InCom Working Group 26

Design of Movable Weirs and Storm Surge Barriers

Final Report Version 6.2
29 March 2005

SUMMARY

The PIANC InCom-WG26 (Working Group) performed a comprehensive review (state-of-the-art) of the modern technologies, design tools, and recent researches used to design and build structures controlling water level and flow in rivers, waterways, and ports (for navigation and flood protection).

The WG considered regulatory structures of river control weirs and storm surge barriers, focussing on the gate design. This includes:

- Gates controlling water level and flow in rivers (even those not navigable) and waterways (lifting gate, tilting gate, radial gate, sector, etc.; designed in one piece or with an upper flap). These are MOVABLE WEIRS.
- Gates controlling water level and flow in estuaries with regard to high tides and storms (lifting gate, articulated, tilting, rolling, floating, sliding, etc.). These are flood BARRIERS.

The WG Report focuses on the following aspects:

- List of the recent movable weir and barrier projects (see Project Reviews), presentation of their concepts and innovations, and the driving forces considered for selecting these particular designs (Section 2.1).
- A terminology review of the technical terms and names used to define weirs and barriers (Section 2.2)
- Design Procedure for the design of weirs and barriers (Section 3).
- A review of the various multi-criteria assessment approaches that can be used to select the most relevant designs (Section 4). List of criteria for weirs and barriers, are proposed.
- Technical considerations including environmental, economic and safety aspects, for design, construction, maintenance and operation (Section 5).
- Structural considerations on various gate-types with an advantage-disadvantage comparison (Section 5.1).
- Technical background required to perform hydraulic and flow analysis of various gate-types (Section 5.2)
- Interaction between foundation and weir-barrier structure (Section 5.3).
- Control procedures of the gate operations and their maintenance (Section 5.4)
- Survey of the temporary closure systems (e.g. bulkheads) used for inspection and maintenance (Section 5.5).
- State-of-the-art of the risk-based design methods. With applications to navigation weirs and flood barriers (Section 5.6)
- Interactions between the technical aspects of a weir/barrier design with environmental and aesthetic considerations (Section 5.7)
- Procedure to assess the global construction cost of a weir at the design stage (Section 5.8)
- Design assessment tools for preliminary and detailed design stages (Section 6 and Annex A)
- Prefabrication techniques (Section 7)
- Codes, rules and standards: at national and international level; including the use of the semi-probabilistic Eurocode format (Section 8)
- An extensive list of relevant technical books, web sites, and guidelines (Section 10).

The present hardcopy WG-26 report is a reduced version of the full report, which is available on the companion CD-ROM, attached to this PIANC hardcopy report (Directory /A2- REPORT WG-26 (Extended Version)/.

The CD includes

- About 50 Project Reviews of movable weirs and storm surge barriers with various flat, radial, lifting, sector, and inflatable gates (Directory A1 on CD)
- A PDF Copy of this Report (Directory A2 on CD)
- Sponsor Company References (Directory A3 on CD)
- Various additional information about Sections 3; 4; 5; 6; 7 and 8 of this report (Directory Annex Section # on the CD)
- Various technical guidelines (Directories B on CD) such as
    - B1: PIANC's "Illustrated Technical Dictionary" (Locks, Gates, Dewatering services and Protection from Ship Impact).
    - B2: "Design of Mobile and Marine Metallic Structures using the Limit States and Partial Safety Factor Concepts" (France) & "ROSA 2000: Guidelines for the limit state design of harbour and waterways structures"
    - B3: Movable Weirs (Guide du chef de projet)
    - B4: Inflatable Weirs (Germany)
    - B5: Maintenance bulkhead types and Temporary and Demountable Flood Protection. Some technical reports are also given.
    - B6: Examples of rehabilitation Weirs
    - B7: Flood Protection in UK,
    - B8: Environmentally Considerate Lubricants
- WG26's Meeting Pictures, Directory C on the CD

DESIGN OF MOVABLE WEIRS AND STORM SURGE BARRIERS

PIANC WORKING GROUP 26 InCom

CONTENT

WG-26's CD-Rom

- About 50 Project Reviews of movable weirs and storm surge barriers (Directory A1)
- WG-26 Report (FDF Full version), (Directory A2)
- Sponsor Company References (Directory A3)
- Various additional information about Sections 3; 4; 5; 6; 7 and 8 of this report (See Directory Annex Section #)
- Various technical guidelines (Directories B):
  - B1: PIANC's "Illustrated Technical Dictionary"
  - B2: Guidelines: "Design of Mobile and Marine Metallic Structures" & "ROSA 2000:
  - B3: Movable Weirs (Guide du chef de projet - France)
  - B4: Inflatable Weirs (Germany)
  - B5: Maintenance bulkhead types and Temporary and Demountable Flood Protection.
  - B6: Examples of rehabilitation Weirs
  - B7: Flood Protection in UK
  - B8: Environmentally Considerate Lubricants
- WG-26 meeting Pictures, (Directory C)

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The Environment Agency (UK), Voies Navigables de France (VNF), BAW (Germany) and Balkema Publ. for their Copyright Agreements.

Meetings of the Working Group

Working Group (WG-26) had 6 meetings at Brussels (February 2003), London (June 2003), Pittsburgh (November 2003), Rotterdam (March 2004) and Edinburgh (October 2004).

Thanks to the organisations that provided rooms and funded our meetings: TECHNUM and ANAST-University of Liege (BE), British Waterways and the Environment Agency (UK), the U.S. Army Corps of Engineering, INCA (US) and the Rijckwaterstaat (NL).

CD-Sponsors.

The WG acknowledges the following organisations and companies for their sponsorships to support the CD fees: BRIGESTONE (Japan - UK), BRL (France), BUYCK (Belgium), CNR (France), COYNE et BELLIER (France), DREDGING- TV SVKS (Belgium), ISM (France), OBERMEYER-DYRHOFF (USA), RUTTEN (Belgium), SCALDIS SALVAGE (Belgium)

Extensive technical references of these companies are available on the Directory A3 on the CD.

1. INTRODUCTION

INCOM (PIANC's Inland Navigation Commission) launched, in the last 30 years, working groups (WG) on various subjects such as 'Inland waterway vessels', 'Standardization of ships and inland waterways for river/sea navigation', 'Locks', 'Shiplifts', 'Automatic management of canalized waterways and its hydraulic problems', etc.

For one reason or another, movable weirs, and particularly the design of their movable parts (the gates), have not been addressed by a PIANC WG. While locks, ship lifts, bridges, waterways dimensions, bank protection, contaminated dredge material etc. have been studied, key structures that provide waterway navigability, such as movable weirs, have not.

There are several reasons for this, some of which include the following:

- On rivers, movable weirs are often overlooked. Such is the case of the oldest types (needles, wicket gate, *hausse Aubert, ... in France*, and stoplogs). In a similar way bear-trap, radial gates (most of the time) and flap gates are not visible. Only lifting gates are visible throughout the year. Therefore, such "*invisible and quiet structures*" do not seem very important (even if they are usually critical for the surrounding people).
- River weirs are not spectacular. Ships interact with locks, ship lifts, etc. but seldom sail through weirs (unless when is it dismounted or the gate is hidden). River weirs definitely do not attract attention.
- In Europe, most of the rivers are equipped with movable weirs (when required to allow navigation throughout the year). So, most of the projects concern rehabilitation or replacement (as in France) on small rivers having only local traffic and pleasure navigation. This is, of course, less attractive than new outstanding structures. Since about 1970, with infrastructure funds lacking, the emphasis on weirs is no longer a priority (contrary to new canals, locks etc.).
- Movable weirs are massive structures whereas movable parts (needles, stoplogs) are relatively simple and thus do not receive high attention from the head offices.

Field engineers involved in river engineering and particularly those designing river weirs, usually agree that in recent memory, the design of movable river weirs has not progressed as other engineering works have.

- A new weir is usually built like the previous one.
- There is not enough room for innovation, as weir owners (usually public administration) do not want to face any "problems". The risk of using a new concept is usually assessed as being too high as compared to the advantages. This is evidence of how important these gates really are. For standardization reasons (at the operational level), changes are also often avoided.
- Gate type (or weir type) is usually decided based on the experience of the head officer(s) (even if some general assessment is provided). Selection procedure is often more a justification procedure than a thorough investigation for a best solution. Often, various gates types are discarded as not relevant. Then, for the 5 or 6 remaining types, a solution is selected using a series of good and obvious reasons (too expensive, not adapted to sediment transport, movable parts in water must be avoided, too complex, difficult to regulate, aesthetic or integration is doubtful, not reliable, require extensive validation, etc.).

Fortunately, since about 1970, the need to protect estuaries and ports against high tides and storm surges has induced the construction of a new type of movable weirs called barriers. These barriers do not control daily flows for irrigation, navigation or industrial purposes but are designed to prevent a major disaster in case of exceptional high rise of sea/river water level (tide, storm surge, typhoon etc.). Due to the enormous size of these barriers, the traditional conservative designs were avoided and public officers had to challenge designers to develop new and innovative concepts. Outstanding examples are the Thames Barrier, the Nieuwe Waterweg Barrier in Rotterdam and in the near future the Venice Barriers. Such designs required multi-disciplinary teams, thorough economic and technical assessment, multicriteria and risk assessments.

Knowing this situation, this WG report provides some relevant contributions to improve the design (and the gate selection) of movable weirs and storm surge barriers. These contributions are:

- general design methodology
- reviews of the various types of weirs and a listing of new innovative concepts (floating structures, prefabricated elements, inflatable weirs, …)
- an up-to-date review of design tools
- a multicriteria assessment guideline
- a survey of the technical, economical and environmental aspects of movable weirs
- integration of traditional weir design procedures with risk assessment, maintenance and control, codes and standards (Eurocodes), and design concept (limit states and partial safety factors)

It is hoped that, with this information, those responsible for these matters will look at the options in a new light.

1.1 AIMS OF THE WG-26

Based on the WG26's terms of reference the aim of the WG (Working Group) was to conduct a comprehensive review (state-of-the-art) of the modern technologies, design tools and recent research used to design and build structures controlling water level and flow in rivers, waterways and ports (for navigation & flood protection).

The WG considered regulatory structures such as:

- Gates controlling water level and flow in rivers (even non navigable) and waterways (lifting gate, tilting gate, radial gate, sector, etc.; designed in one piece or with an upper flap). These are referred to as WEIRS. This does not include spillway gates of fixed dams. For this specific aspect see ICOLD (www.icold-cigb.org). Irrigation weirs are also not considered in this report. Old weir types such as needle weirs, weir-boards, etc. are not reviewed even though many of these weirs are still used and their improvement investigated.

- Gates controlling water level and flow in estuaries with regards to high tides and storms (lifting gate, articulated, tilting, rolling, floating, sliding, etc.). These structures are referred to as BARRIERS.

The civil engineering aspects related to strength, stability, etc. of the fixed elements (pier, abutments, floor, ..) of moveable structures were in principle not considered unless there is a direct relation between the design of the movable structures and the fixed parts. This is for instance the case of the foundations, as there pattern and strength have a direct effect on the selection of the relevant weir-types and therefore, on the gate-types.

The WG Report focuses on the following aspects:

- List of the recent movable weir and barrier projects (see Project Reviews), presentation of their concepts and innovations, and the driving forces considered for selecting these particular designs (Section 2.1).
- A terminology review of the technical terms and names used to define weirs and barriers (Section 2.2)
- Design Procedure for the design of weirs and barriers (Section 3).
- A review of the various multi-criteria assessment approaches that can be used to select the most relevant designs (Section 4). List of criteria for weirs and barriers, are proposed.
- Technical considerations including environmental, economic and safety aspects, for design, construction, maintenance and operation (Section 5).
- Structural considerations on various gate-types with an advantage-disadvantage comparison (Section 5.1).
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- Procedure to assess the global construction cost of a weir at the design stage (Section 5.8).
- Design assessment tools for preliminary and detailed design stages (Section 6 and Annex A of the report).
- Prefabrication techniques (Section 7),
- Codes, rules and standards: at national and international level; including the use of the semi-probabilistic Eurocode format (Section 8)
- An extensive list of relevant technical books, web sites, and guidelines (Section 10)

1.2 WG26's CD-ROM

Due to editing constraints the number of pages of WG26's hardcopy report was limited. Therefore all the following information have been saved on a companion CD-ROM (attached to this PIANC hardcopy report). This CD includes:

- About 50 Project Reviews of movable weirs and storm surge barriers with various flat, radial, lifting, sector, inflatable... gates (Directory A1)
- Copy of this Report (Full version) in PDF. (Directory A2)
- Sponsor Company's References (Directory A3)
- Various additional information about Sections 3; 4; 5; 6; 7 and 8 of this report (Directory Annex Section #) including a survey of maintenance bulkhead types.
- Various technical guidelines (Directories B) such as:
  - B1: PIANC's "Illustrated Technical Dictionary" (Locks, Gates, Dewatering services and Protection from Ship Impact).
  - B2: "Design of Mobile and Marine Metallic Structures using the Limit States and Partial Safety Factor Concepts" (France) & "ROSA 2000: Guidelines for the limit state design of harbour and waterways structures"
  - B3: Movable Weirs (Guide du chef de projet)
  - B4: Inflatable Weirs (BAW, Germany)
  - B5: Maintenance bulkhead types (survey) and some technical reports are also given. Temporary and Demountable Flood Protection, DEFRA, (www.environment-agency.gov.uk/floodresearch)
  - B6: Examples of rehabilitation Weirs (Belgium, Germany)
  - B7: Flood Protection in UK (Environment Agency)
  - B8: Environmentally Considerate Lubricants (UK)
- WG-26 meeting pictures, (Directory C)

Other relevant documents used by the WG are:
- Manual for River Work in Japan, Japan (In English)
- Technical Standards and Commentaries for Port and Harbour Facilities in Japan (in English).

Unfortunately we were not allowed to paste copies of these 2 documents on the WG26's CD.

1.3 LIST OF PROJECT REVIEWS

The WG completed about 50 project reviews of movable weirs and storm surge barriers. The list is presented in Table 1.1.

The project reviews (full version) are available on the Directory A1 on the CD. Here after is presented (Section 2.1) a brief description of each.

In addition, a descriptive summary of the different weir and barrier types is also available on the Directory A1 on CD.

Code	Gate Type	Project Title	Country	Author	Closure	Purpose
A1	Arch/Visor	Rhine Visor Weirs	NL	Daniel	Frequent	Flow
A2	Arch/Visor	Osaka Arch Gate	Japan	Nagao	2-3 / Year	Flood
B1	Flap Gate	Lagan Weir(Storm surge barrier)	UK	Dixon	Frequent	Flow
B2	Flap Gate	Tees Barrage (Tidal weir)	UK	Dixon	Frequent	Flow
B3	Flap Gate	Libcice-Donaly (river navigation weir)	Czech Rep	Kupsky	Frequent	Flow
B4	Flap Gate	Veseli (24m long)	Czech Rep	Kupsky	Frequent	Flow
B5	Flap Gate	Bremen Weser Weir (navigation weir)	Germany	Meinhold	Frequent	Flow
B6	Flap Gate	Torque-tube at Montgomery Dam	USA	Stockstill	Annual	Flow
B7	Flap Gate	Sauer Closure Gate - Short Review	France	Daly	Frequent	Flood
B8	Flap - Wicket	Denouval	France	Daly	Frequent	Flow
B9	Flap - Wicket	Olmsted, Wicket Gates	USA	Stockstill	Annual	Flow
B10	Flap - Inflatable	Sinnissippi Weir (Obermeyer)	USA	Lagache	Frequent	Flow
B11	Flap - Bouyant	Venice storm surge barrier	Italy	Perillo	Annual	Flood
C1	Inflatable Weirs	Inflatable Weir	Canada	Abdelnour	Frequent	Flow
C2	Inflatable Weirs	Ramspol Barrier	NL	Daniel	Annual	Flood
C3	Inflatable Weirs	Pocaply (river weir)	Czech Rep	Kupsky	Frequent	Flow
C4	Inflatable Weirs	Inflatable Weirs Presentation	Germany	Meinhold	Frequent	Flow
C5	Inflatable Weirs	Rubber Dam at the river Lech	Germany	Meinhold	Frequent	Flow
D1	Miter Gates	Goole Caisson	UK	Dixon		Emergency
E1	Radial - Single	Upper Meuse	Belgium	Hiver	Frequent	Flow
E2	Radial - Single	Steti (river navigation weir)	Czech Rep	Kupsky	Frequent	Flow
E3	Radial - Single	Stör Storm Surge Barrier	Germany	Meinhold	Frequent	Flood
E4	Radial - Single	Braddock Dam	USA	Miller	Frequent	Flow
E5	Radial - Single	Iron Gates (Nagivation river weir)	Romania	Sarghiuta	Frequent	Flow
E6	Radial - Single	Olt River Lower Course	Romania	Sarghiuta	Annual	Flow
E7	Radial - Double	Eider Barrage (storm surge barrier)	Germany	Meinhold	Frequent	Flood
E8	Radial - Double	Haringvliet Storm Surge Barrier	NL	Daniel	Annual	Both
E9	Radial - Innovative	Radial Gate w/ Under/Overflow (Concept)	Belgium	Rigo	Frequent	Flow
E10	Radial - Innovative	Prefab Floating Weirs: Alu + Fibres Conc	Belgium	Rigo	Frequent	Flow
F1	Rolling & Trolley	Selby Lock Rolling Gate	UK	Dixon	3 per year	Flood
F2	Rolling & Trolley	Berendrecht Flood Control Rolling Gate	Belgium	Bulckaen	Annual	Flow
G1	Roof or Bear Trap	Tee Gate	UK	Dixon	Frequent	Flow
H1	Sector - Horiz.	Roudnice (river weir)	Czech Rep	Kupsky	Frequent	Flow
H2	Sector - Horiz.	Mosel River Weir Lehmen(Nav. Weir)	Germany	Meinhold	Frequent	Flow
H3	Sector - Rising	Thames River Barrier	UK	Wilkes	5 - 30/year	Flood
H4	Sector - Rising	EMS (storm surge/nav. Channel gate)	Germany	Meinhold	Frequent	Both
I1	Sector - Vertical	Maeslant Storm Surge Barrier	NL	Dan.& Bulk.	Annual	Flood
I2	Sector - Vertical	Storm Surge Barrier: Alternative Concepts	NL	Rigo	Frequent	Flood
I3	Sector - Vertical	Amagasaki ock gate	Japan	Nagao	2-3 / Year	Flood
J1	Stoplogs & B/H	Kentucky Lock Floating Caisson	USA	Miller	Annual	Flood
J2	Stoplogs & B/H	Olmsted Maintenance Bulkheads	USA	Miller	Annual	Flood
J3	Stoplogs & B/H	Tees Stoplog	UK	Dixon	Annual	Maintenance
J4	Stoplogs & B/H	Murray River Stop Logs	Australia	Rigo	Frequent	Flow
K1	Swing	Bayou DuLarge : 17m Barge Gate	USA	Miller	Annual	Flood
K2	Swing	Bayou Lafourche Barge Gate	USA	Miller	Annual	Flood
K3	Swing Floating	Storm Surge Barrier: Alternative Concept	BE, NL	Rigo	Frequent	Flood
L1	Vertical Lift	Beernem Weir	Belgium	Bulckaen	Frequent	Flood
L2	Vertical Lift	Hartel Canal Barrier	NL	Daniel	Annual	Flood
L3	Vertical Lift	Ivoz-Ramet (Renovation weir + B/H)	Belgium	Dermience	Frequent	Flow
L4	Vertical Lift	Kamihirai Gate	Japan	Nagao	2-3 / Year	Flood
L5	Vertical Lift	Shinanogawa River Gate	Japan	Nagao	2-3 / Year	Flood
L6	Vertical Lift	Blanc Pain (Emergency gate)	Belgium	Rigo	Frequent	Emergency
L7	Vertical Lift	Hull Barrier	UK	Wilkes	10-30/year	Flood
L8	Vertical Lift	Cardiff Bay	UK	Wilkes	Frequent	Tide
M1	Floating boom	Ice Boom - Lac St. Pierre	Canada	Abdelnour	Annual	Flood
M2	Unclassified	Curtain Barriers – Temporary	Canada	Abdelnour	Annual	Flow
	Maintenance Bulkheads and Cofferdams- *See CD Annex Section 5.5*			Rigo	Annual	Maintenance

Table 1.1 : List of Project Reviews

2. GATES OF MOVABLE WEIRS AND BARRIERS

2.1 PROJECT REVIEWS

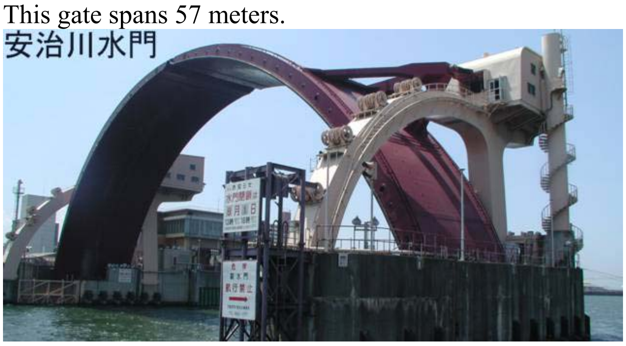
Representative samples of each gate type included in this document are summarized in this chapter. Case studies of each of these gates are included on the WG25-CD /Directory A1/. The case studies include a more complete description of the gate, foundations, abutments, operating characteristics and, where available, cost. Photographs and select engineering drawings are also presented for many of the gates.

A. ARCH or VISOR GATES
An arch gate is a three-hinged arch that spans from abutment to abutment across the waterway. It is hinged at the abutments and rotates upward for storage and downward to close the channel.

A.1 Rhine Visor Weirs
These double visor gates each span 54 meters and are used to control flow for power generation and navigation. This is one of 3 weirs of similar construction on the Rhine River.

Hagestein, The Netherlands (~1960)

A.2 Aji River Barrier
This is one of 3 lock gates constructed as flood protection measures from storm surges for the city of Osaka, Japan. This gate spans 57 meters.



Osaka, Japan, 1970

B FLAP GATES
Flap gates are hinged along the upstream edge of the gate and attached to a sill foundation. They are stored submerged and flat to the bottom. To close the flow, the downstream edge is rotated upward.

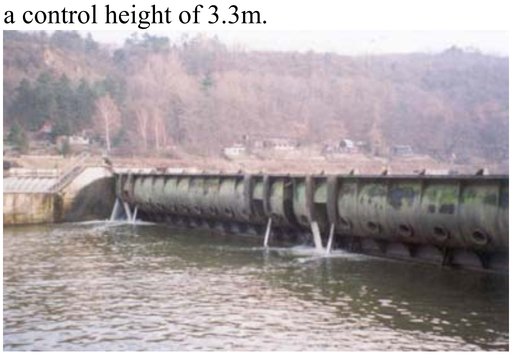
B.1 Lagan Weir (Storm surge barrier)
The barrier is composed of 5 Fish Belly, bottom hinged, flap gates. Each gate is 20m wide by 4.5m tall. These gates are used for flood control and to improve water quality.

Belfast, Northern Ireland, 1994

B.2 Tees Barrage (Tidal weir)
This barrage was established to improve water quality and to provide flood protection. The barrage has 4 bottom hinged fish-belly flap gates. Each gate is 13.5m wide by 8m high.

Stockton on Tees/Teesside, UK, 1995

B.3 Libcice-Dolany (river navigation weir)
The three sluiceway openings serve navigation and hydropower interests on the Vltava River. The right sluiceway is 19.85 m wide and the others are 43.0 m, with a control height of 3.3m.



Libcice, Vltava River, Czech Republic, 1989

B.4 Veseli (24m long)
The weir Veselí consists of two 24.4 m wide hollow flap gates with a 1.4 m control head. The dam provides support for navigation and hydropower. A fish ladder is also provided.

Veseli, Morava River, Czech Republic, 2002

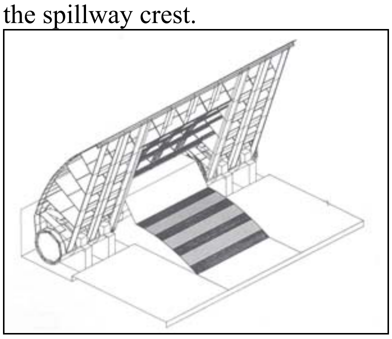
B.5 Bremen Weser Weir (navigation weir)

The five fish belly flap gates span 31 m and provide a control height of 3.8m. The weir provides for flood protection and maintains draft for navigation.

Bremen, Germany, 1993

B.6 Torque-tube at Montgomery Dam

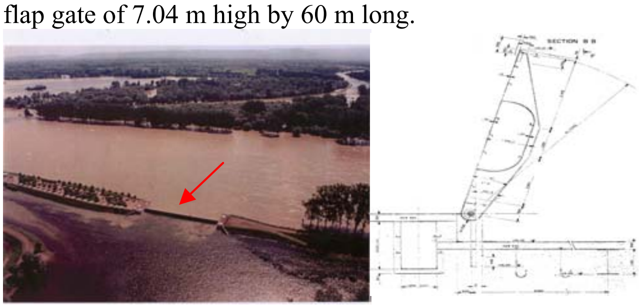
The project consists of a navigation lock, a 91.4-m-wide controlled navigation pass spillway with 10 torque-tube gates, and a 61.0-m-wide fixed uncontrolled overflow spillway. Each gate is 9.1 m wide and rises 3.96 m above the spillway crest.



Desha County, Arkansas, USA, about 2004

B.7 Sauer Closure Gate

The goal of this project is the protection of cities and lands against flood created by the river Rhine. There is a single flap gate of 7.04 m high by 60 m long.



Sauer Flood Barrier – Munchhausen, France, 1993

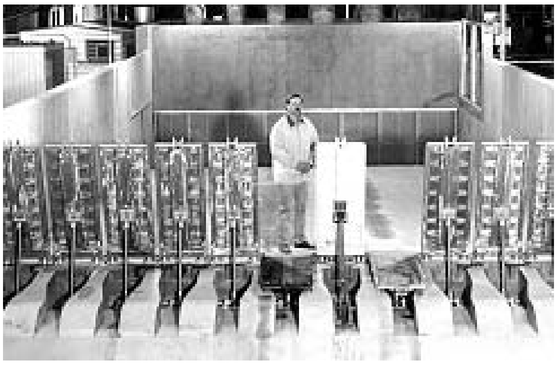
B.8 Denouval Wicket Gates

These 30 wicket gates dam a river width of 70 m. Each wicket has a height of 3.3 m and a width of 2.5 m. The gates are hydraulically operated and can be placed in one of four possible positions. The gates facilitate navigation on the Seine.

Andresy, France, 1980

B.9 Olmsted Wicket Gates

The navigable pass section of the dam will be 420-m long with 140 x 2.95-m wide, boat-operated steel wicket gates. The project provides navigation and flood control.



Olmsted, Illinois, USA, Estimated 2009.

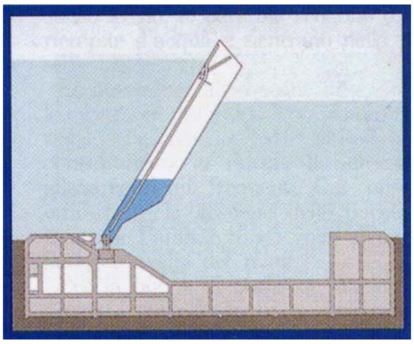
B.10 Sinnissippi Dam

The dam has three 16m (48-foot) long and four 32m (96-foot) long pneumatically operated hinged-leaf gates and a 168m (504-foot) long conventional concrete ogee spillway and provides for flood protection, hydropower and navigation (Obermeyer system).

Sterling – Rock Falls, Illinois, 2002

B.11 Mose Buoyant Flap Gate

These oscillating buoyant retractable floodgates will provide flood protection to Venice. Seventy-eight flood gates will be provided at 4 locations. They will vary in width from 3.6m to 5m and the length will vary from 18 to 28m.



Venice, Italy (planned project)

C INFLATABLE WEIRS

These are operable weirs that are composed of long bladders, secured to a bottom foundation. The weir is raised by inflating the bladders with air or water.

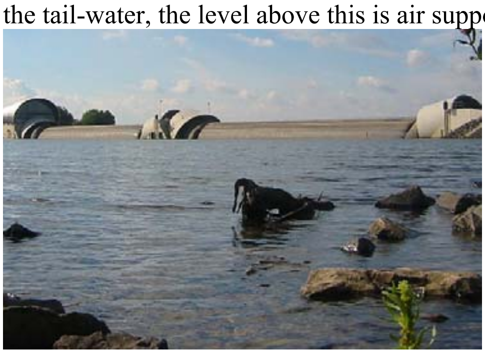
C.1 Canadian Inflatable Weir

An inflatable weir was built upstream of a fall, downstream from a power plant intake structure, to control and optimize the water level while maintaining a minimum flow over the weir at all times.

Chute Bell, Rivière Rouge, Québec, Canada, 1994

C.2 Ramspol Barrier

These 3 inflatable fabric bellows barriers with a width of 60m, provide 2.7m of flood protection from inland river flood waters. The water level inside the barrier matches the tail-water, the level above this is air supported.



Kampen, the Netherlands, 2002

C.3 Pocaply Inflatable Weir

This rubber dam is 21m wide with a design height of 1.6m. It is water filled and provides a pool for hydropower generation.

Pocaply, Loucna River, Czech Republic, 1998

C.4 German Inflatable Weir Reference Document

This pdf document shows a presentation on the operation and design of inflatable weirs (BAW, Germany).

C.5 Rubber Dam at the river Lech

This dam provides a pool for hydropower. Four sections are used, one with a width of 26.65m and a height of 3.35m. The other three are 46.67m wide by 1.25m high.

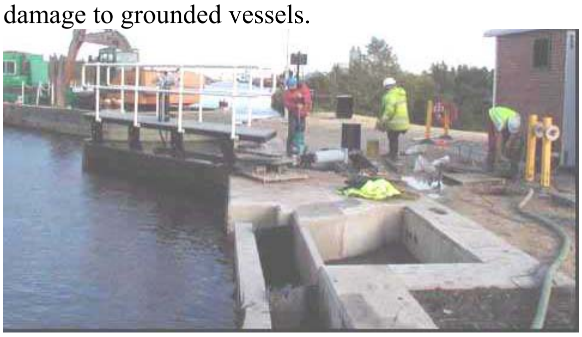
Füssen, Germany, 2001

D MITER GATES

Miter gates are typically used for navigation locks rather than flood control. However, they are used at Goole to prevent the harbour draining if the canal wall collapses. Miter gates are only operated when the water level is equal on both sides of the gate. A miter gate has two leaves that are hinged like doors on either side of the channel. They meet at an angle of about 30 degrees and rely on the mitering action to span the opening. This carries significant thrust to the abutments.

D.1 Goole Caisson

These gates are closed if a breach in the canal wall occurs. This prevents the harbour from draining with subsequent damage to grounded vessels.



Goole, Great Britain, 2002

E RADIAL GATES

A Radial or Tainter gate has a skin plate mounted on an open structural steel frame supported by strut arms at each side of the gate. The strut arms extend to trunnion bearings mounted on abutment walls on either side of the gate opening. Radial gates may have the trunnion bearing either upstream or downstream and the gates may be stored submerged and raised to close flow or stored overhead and lowered to close flow.

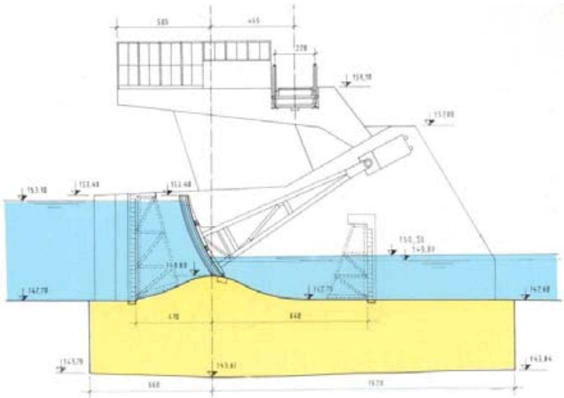
E.1 Upper Meuse

This project will rebuild a number of locks and dams on the upper Meuse River to improve navigation and power generation. These radial gates have an upper flap that allows more economical and precise flow control.

Upper Meuse Basin, Belgium, 1985-95

E.2 Steti Radial Gates

The weir is provided with seven sluiceway openings, two are fixed, two are locked by a steel radial gate, and three openings are locked by a steel radial gate with a control flap. 4.4m of control height is provided.



Steti, Labe River, Czech Republic, 1972

E.3 Stör Storm Surge Barriers

Double Tainter gates are provided on each side of two lock chambers to provide redundant flood protection in support of navigation. The tainter gates span 43 m and are 13 m high.

Federal State Schleswig-Holstein, Germany, 1974

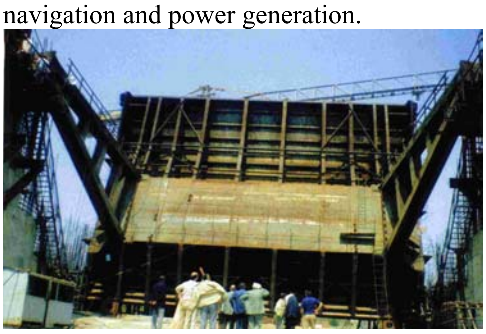
E.4 Braddock Dam

The 4 radial gates are 33.53m long with a total damming height of 6.4m. The gates are used for flood protection and navigation and are hydraulically operated.

Braddock, PA, USA,2003

E.5 Iron Gates

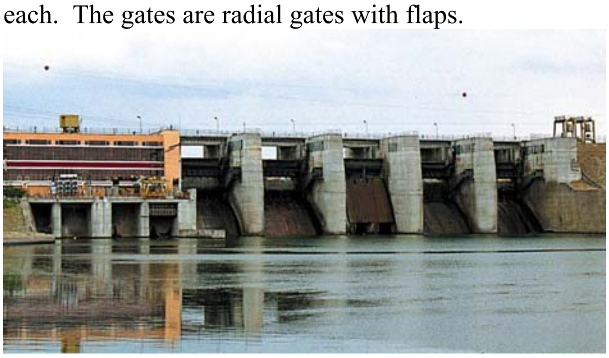
The two spillway dams on each river branch with seven 21m wide gates, three of which are equipped with overflow flaps of 2.50 m height. The dams are used for navigation and power generation.



Danube, Romania and Yugoslavia, 2000

E.6 Olt River Lower Course

Five dams were constructed in 13.5m steps along the Olt River to provide for hydroelectric power generation. Each of them consist of a gated dam with 5 openings of 15 m each. The gates are radial gates with flaps.



Olt River – lower course, Romania, 1990

E.7 Eider Barrage (storm surge barrier)

The floodgate section consists of five 40m wide spillways. Each opening has two radial floodgates for double protection. Seaside: High: 10.1m Riverside: High: 11.10 m

Schleswig-Holstein/Nordfriesland, Germany, 1973

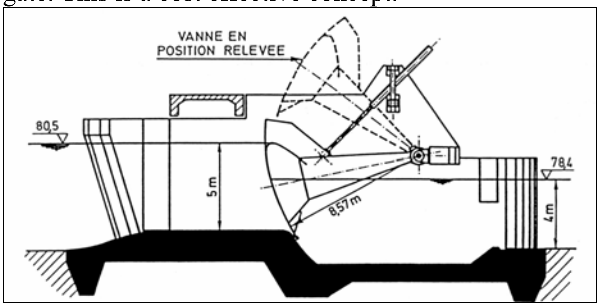
E.8 Haringvliet Storm Surge Barrier

This flood control structure provides two rows of 17 seaside and 17 riverside radial gates. The barrier is 1048.5m wide and the gates span 62m.

Hellevoetsluis, The Netherlands, 1970

E.9 Radial Gate with Under and Overflow

This gate concept has not yet been implemented, but would allow fine control of flow by lowering the gate and allowing surface flow over the top or would provide for high discharges and passage of sediment by raising the gate. This is a cost effective concept.



Upper-Meuse, Belgium (not built)

E.10 Prefabricated Floating Weirs - Innovative Concept

A series of 9 prefabricated navigation control weir sections are constructed in 4 floating sections that are transported afloat to the site and placed on a prepared foundation. Elements are made of aluminium to float in shallow water (60cm) steel can also be used. The structure (30m long, 29.5m wide and 7.6m high) includes 2 radial gates of 12m. The infill concrete is reinforced with steel fibbers rather than traditional rebar. This facilitates underwater placement.

The concept was developed for the Sambre river, Belgium, (not yet built).

F ROLLING or TROLLEY GATES

Rolling and Trolley gates are closure panels stored adjacent to the waterway. They are rolled into position in anticipation of a flood event. Rolling gates are bottom supported and trolley gates are top supported.

F.1 Selby Lock Rolling Gate

This flood control gate is stored in a slot at the side of the waterway and is winched across the canal. The gate is 6.4m wide, 3.85m high and 0.35m deep. It is partially buoyant and seals to a timber sill.

F.2 Berendrecht Flood Control Rolling Gate

These rolling lock gates are used to provide navigation access through a flood control barrier. The gates are buoyant and supported by a submerged trolley on the leading edge and an above water trolley on the aft end. The gates are 69.69 m long and have a height of circa 22.60 m. The average width is 9 m.

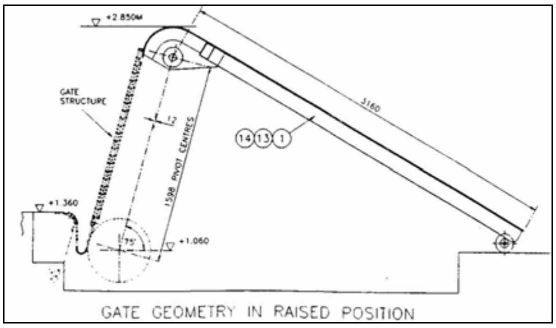
Antwerp, Belgium, 1989

G ROOF or BEAR TRAP GATES

Bear trap gates are not as common today as in years past. A bear trap gate is constructed of two leaves that slide over one another and seal together. They are stored on the bottom of the waterway. Typically water is allowed to enter the space beneath the gate and the upstream water pressurizes the space beneath the leaves and the gate leaves rise to block the flow. Resurgence has been found in two projects in England. They are used in recreational water parks to provide a “whitewater” rafting and canoeing experience. The course is configurable by adjusting the bear trap gates to adjust the flow characteristics. One example is provided at Tees Barrage in England.

G.1 Tees Barrage Bear Trap Gate

This bear trap gate is 5.950 m wide. The upstream leaf is 1.598 m centre to centre and the downstream leaf is 3.160m. The gate is used to control flows for white water canoe and kayak recreation.



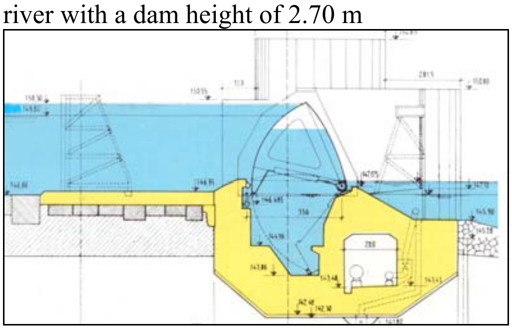
Tees, United Kingdom,1984

H SECTOR GATES - HORIZONTAL AXIS

Horizontal axis sector gates are circular sections hinged on the downstream side with a skin plate on the upper 2 sides. A horizontal axis sector gate rotates in a vertical plane about a horizontal axis. When lowered the upper skin plate of the gate coincides with the overflow section of the sill. Rotating or Rising sector gates are included here also. These gates provide skin plates on a segment of a circular arc and are supported at the sides of the spillway.

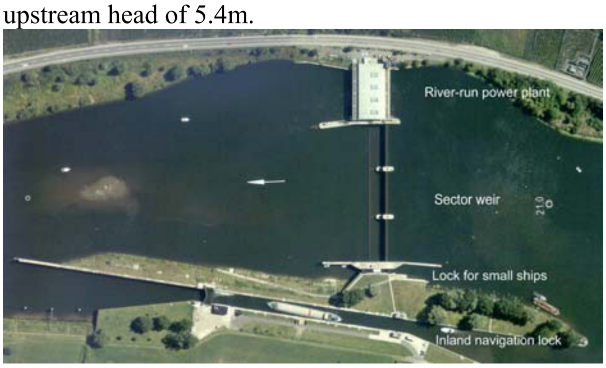
H.1 Roudnice

These gates are used for navigation and irrigation. Three sluiceways of the same clear width of 54.05m span the river with a dam height of 2.70 m


Roudnice, Labe River, Czech Republic, 1972

H.2 Mosel River Weir Lehmen (Navigation Weir)

11 of the 14 weirs built on this section of the Mosel use sector gates to control flows for navigation and hydropower generation. Three 40m spans dam an upstream head of 5.4m.



Mosel river, Germany,1963

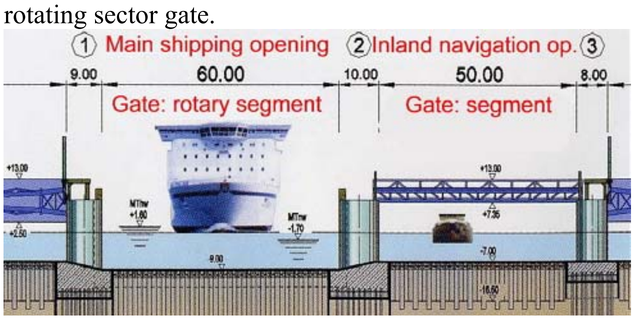
H.3 Thames River Barrier

This massive flood protection barrier protects London from flooding on the river Thames. The barrier extends 520m across the river and uses four 20 m high rising sector gates that span 61m.


London, United Kingdom, (1982)

H.4 Ems Barrier

The Ems barrier provides flood protection and supports navigation, it has a length of 476m between bank lines with 7 openings. The main shipping opening uses a rotating sector gate.



Ems river, Germany, 2002

I SECTOR GATES - VERTICAL AXIS

Vertical Axis Sector Gates are circular sections supported on a vertical hinge at the center of a circular arc. The skin plate is only on the face of the circular arc. Because the hydraulic thrust is directed radially inward toward the vertical axis there is very little unbalanced load and they can be opened and closed with differential head across the gate.

I.1 Maeslant Storm Surge Barrier

This flood protection barrier spans 360m. The gate is made buoyant when it is moved by locomotive engines on each shore. The gates pivot on specially fabricated spherical bearings.


Hoek van Holland, Netherlands, 1997

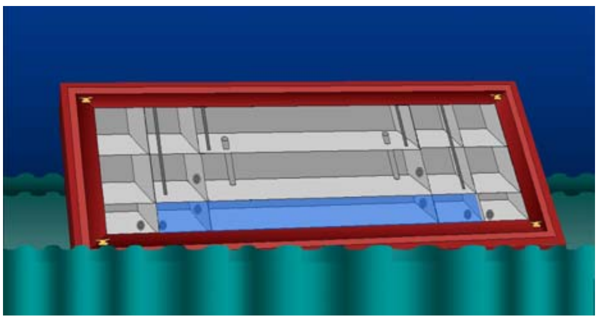
I.2 Maeslant Alternative Barriers
This paper discusses the alternatives to the sector gates finally selected for the Maeslant barrier. A pneumatic tumble gate, a segment gate, hydraulic tumble gate, sliding gate, boat gate and floating sector gates are discussed.

I.3 Amagasaki lock gate
These Vertical axis sector gates provide 17m wide lock access for navigation while providing flood protection to the lowland city from offshore storms and surges.

Amagasaki City, Japan, 2003

J STOPLOGS and BULKHEADS
Stop Logs and Maintenance bulkheads are typically constructed with a pair of horizontal trusses supporting a vertical skin plate on one face. They are stored separately from the gate opening and lifted into place by an overhead or mobile crane. They are designed to span across the opening or between intermediate posts that can be installed at intervals across the opening. They may extend vertically from the sill to the top in one piece or smaller units may be stacked and seal against one another to close the opening.

J.1 Kentucky Lock Floating Caisson
This floating gate is used to dewater lock chambers for maintenance. The bulkhead is towed from one site to another as a barge. It is then filled with water in a sequence to rotate it vertically, move it into position, and lower it into final position. The gate is 34.3m wide and 9m high with a depth of 3.2m.

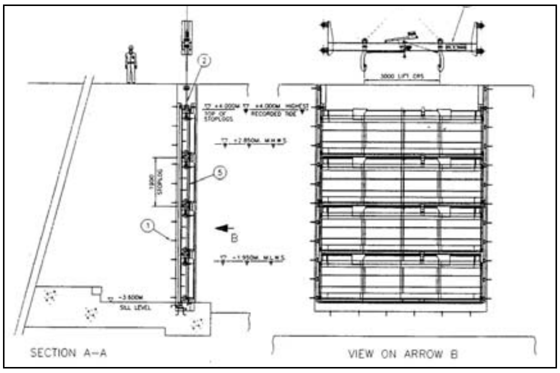


Locks on Tennessee & Kentucky Rivers, USA, 1969

J.2 Olmsted Maintenance Bulkheads
Four bulkhead sections were built to allow maintenance dewatering of the locks and radial gates. The bulkheads are stacked to meeting varying site conditions. Two lower sections 3.4m and 5.5m high are designed to support one of 2 upper sections 11.6m high. The bulkheads span 34.1m.

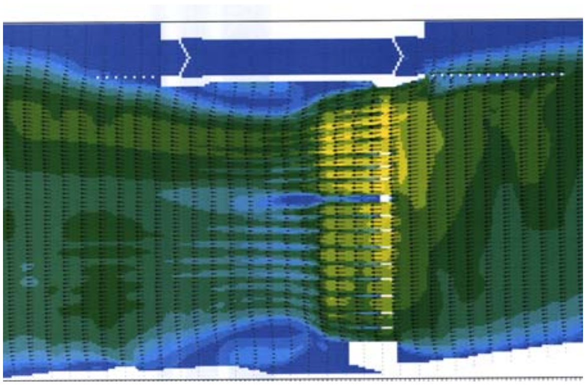
Olmsted, Illinois, USA, 2004

J.3 Tees Stoplog
Thirteen stoplogs, 1.9 m high, close an opening 6m wide. Eight are used on the downstream side of a gate bay and 5 are used upstream. They are placed with a crane and a lifting beam that will automatically engage or disengage from the stoplog.



Stockton on Tees/Teeside, UK,1995

J.4 Murray River Stop Logs
These stop logs are used in support of navigation and flood control. They resist heads varying from 4.5 to 6m



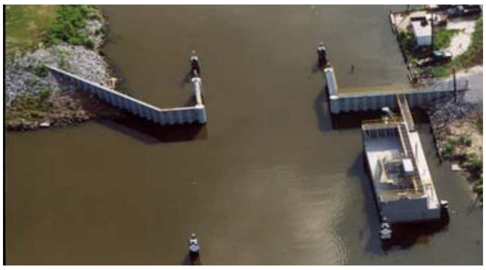
Between Adelaide and Mildura, Australia, around 2000

K SWING GATES

A swing gate is stored on one side of a waterway and pivots about a vertical axis to close against abutments on either side of the waterway. A Swing Gate may be buoyant to reduce hinge and operating forces.

K.1 Bayou DuLarge Barge Gate

This flood control barrier is made buoyant and floated into position by winches in advance of a flood. It spans 18.3m. When in position, it is ballasted onto the sill and has a height of 6.25m.



Bayou DuLarge, Louisiana, USA, 1996

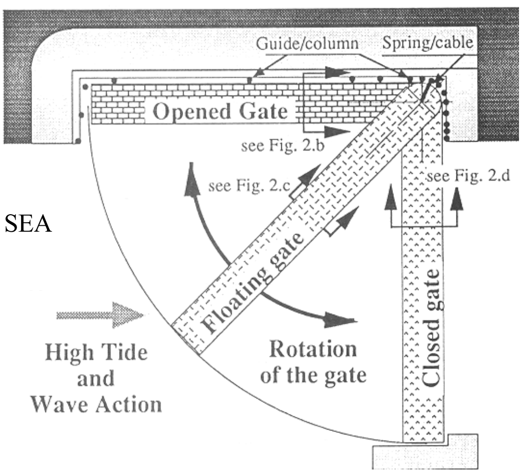
K.2 Bayou Lafourche Barge Gate

This flood control barrier is similar to Bayou DuLarge. It spans 22.9m and has a depth of 3m with a water-tight parapet extending up an additional 1.5m.

Bayou Lafourche, Louisiana, USA

K.3 Antwerp and Rotterdam Swing barriers

This innovative concept of floating rotating barrier was developed for closure of large spans (up to 400m) without any limitation on draft or air clearance, during construction or operation (Rigo et al. 1996).



Project in Belgium and The Netherlands (not built)

L VERTICAL LIFT GATES

Vertical lift gates are raised and lowered vertically. They may be stored underwater and raised to close flow, or stored above a channel on towers and lowered to close flow.

L.1 Beernem Weir

This vertical lift gate provides flood protection and is 8.05m high and 17.9m wide.

Beernem, Flanders, Belgium, 1998

L.2 Hartel Canal Barrier

This large storm surge barrier consists of two lens-shaped vertical lift gates with spans of 98m and 49.3 m with a height of 9.3m. To facilitate water storage the gate never fully closes and at high flood stages the gates are overtopped.

Spijkenisse, Netherlands, 1996

L.3 Ivoz-Ramet

This is a nice example of a rehabilitated weir.

Liege, Belgium, 2000-2001

L.4 Kamihirai Gate

These 4 gates are closed in advance of a flood event. Each gate is 30m wide, 2 gates are 9.2m high and the other 2 are 9.5m

Tokyo, Japan, 1990

L.5 Shinanogawa River Gate

This flood protection structure has 3 spans each 30m wide with a height of 24.5m.

Niigata prefecture, Japan, 1974

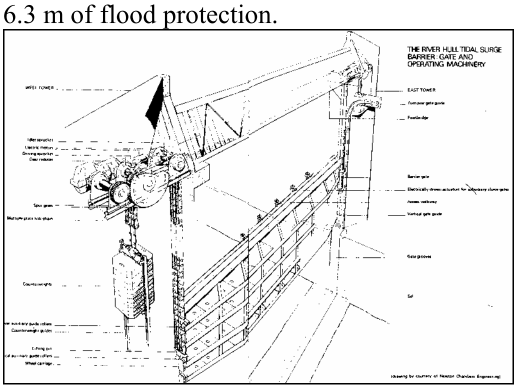
L.6 Blanc Pain

This emergency lift gate protects the 73m high shiplift at Strépy and the surrounding countryside from a flood event in the event of riverbank or structural collapse. The gate closes a channel width of 32.4m and has an air clearance of 7m when raised.

La Louvière, Canal du Centre, Belgium 2003

L.7 Hull

The flood protection barrier is a vertical lift gate which provides a 30 meter wide navigation opening and provides 6.3 m of flood protection.



Hull, UK, 1979

The gate is designed to be aesthetically pleasing and the gate rotates 90 degrees when raised to maximize navigation clearance and minimize visual impact.

L.8 Cardiff Bay Barrier

Cardiff Bay Barrage is a tidal exclusion barrier designed for flood control with 5 sluices (9m wide x 7.5 m high) with double-leaf vertical lifting gates (Faganello E., 2004).

Cardiff Bay, UK, 1998-99.

M UNCLASSIFIED GATES

M.1 Ice Boom - Lac St. Pierre

This floating structure protects a major shipping channel from closure by ice. The floating boom segments are restrained by steel cables to anchors on the lake bottom.

Trois Rivières, Québec, Canada, 1994

M.2 Curtain Barriers – Temporary

This curtain barrier was designed to create a headloss and temporarily force the diversion of the flow away from a tributary. The barrier consists of a long steel pipes with a curtain attached to the bottom. The curtain can be a rubber liner or a plastic pipe(s).

Laboratory test and the field deployment of a curtain, 2004.

2.2 TERMINOLOGY REVIEW

2.2.1 TECHNICAL TERMS IN DIFFERENT LANGUAGES

2.2.1.1 PIANC Dictionaries

To promote the use of homogeneous technical terms in different languages the PIANC's *Illustrated Technical Dictionaries* (written in the six languages: French, German, English, Spanish, Italian, and Dutch) may be very helpful.

Since 1930 different PIANC dictionaries have been published. Unfortunately, some have not yet been published or are no longer available. These dictionaries (published or not) are:

- Chapter I: The Sea (*)
- Chapter II: Rivers, Streams, Canals (*)
- Chapter IV: Boats and Ships, Propulsion (1967)
- Chapter V: Materials (1951)
- Chapter VI: Construction Plant and Methods (1959)
- Chapter VII: Ports (1938)
- Chapter VIII: Locks and Dry Docks, (1936)
- Chapter IX: Maritime Signals (1963)
- Chapter X: River Weirs (Fixed weirs & Movable weirs), (1935, *)
- Illustrated Technical Dictionary (PIANC, 1985, Draft)

(*) Not (or no longer) included in the actual PIANC-Catalogue

The *Dictionnaire Technique Illustré* (PIANC, 1985) is currently unpublished. It's content concerns elements of locks, power stations, weirs, dewatering systems, impact-protection systems and different equipment-parts belonging to them. The draft includes terms concerning water and hydraulic engineering and terms for special hydraulic steel structures (different lock and weir gates).

Nevertheless a draft is available in four languages (German, English, French, and Dutch) but it is not fully complete. The dictionary, converted in PDF-files, can be found on CD, Directory /B1- DICTIONARY (PIANC 1986)/.

The Table of Contents (pdf-files) includes:

- Page 02-19: Locks (Types, Elements, Cross-Sections)
- Page 20-41: Gates (Including Equipments)
- Page 42-53: Dewatering Devices
- Page 54-57: Protection from Ship Impact
- Page 58-65: Water Levels / Navigation Conditions

2.2.1.2 ELSEVIER'S Dictionary

The Dictionary "*Water and Hydraulic Engineering*" (Elsevier 1987) is also recommended. This dictionary contains translations in English, French, Spanish, Dutch and German.

2.2.1.3 ICOLD's terminology

ICOLD (International Commission on Large Dams) has also edited a valuable terminology guideline, which mainly relates to gates of spillways rather than river navigation weirs.

The ICOLD website contains the reference to their technical dictionary,
http://www.icold-cigb.org/anpubli.html
and there is also an online dictionary at:
http://www.icold-cigb.org/services.htm

2.2.2 Standard technical terms of gates of movable weirs and barriers

Before starting with the technical aspects of weir design, it is necessary to introduce the following information:

- The types of barriers and weirs and main characteristic dimensions.
- The name (terminology) of the constitutive elements of barriers and weirs.

Here after technical terms are explained using pictures and sketches (self explanatory pictures).

Figure 2.1 shows a generic view of the main elements of a movable weir structure and its movable parts (hydraulic steel constructions) and the meaning of the terms (associated with numbers of this figure) are given below.

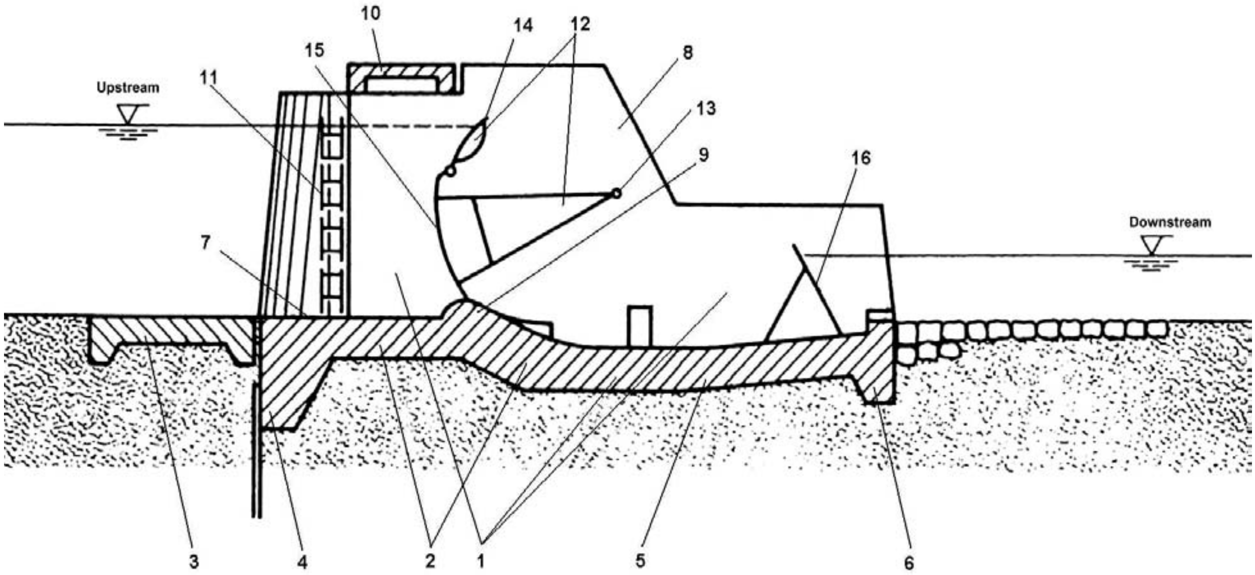


Figure 2.1 : Generic view of the main elements of a movable weir structure

Number	**Meaning**
1	Weir structure
2	Weir sill (or slab)
3	Upstream floor
4	Upstream diaphragm wall (or apron) with cutoffs (here sheetpiles)
5	Stilling basin
6	Downstream diaphragm wall (or apron)
7	Intake floor
8	Weir pier
9	Sill
10	Service bridge
11	Upstream dewatering structure or Bulkheads (here: stop logs)
12	Gate (here: radial gate with fishbelly flap)
13	Bearings
14	Breaker (for flow aeration)
15	Upstream face (water retaining front of the gate)
16	Downstream dewatering structure or Bulkheads

Some generic types of gates of movable weirs are presented below on Table 2.1 to Table 2.3.

Table 2.1: Generic types of gates of movable weirs (Part I)

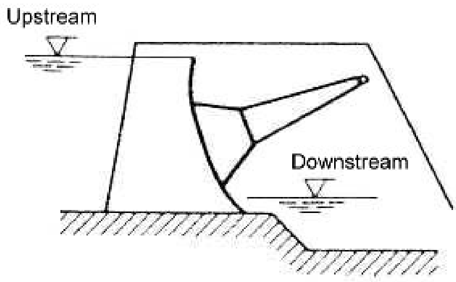
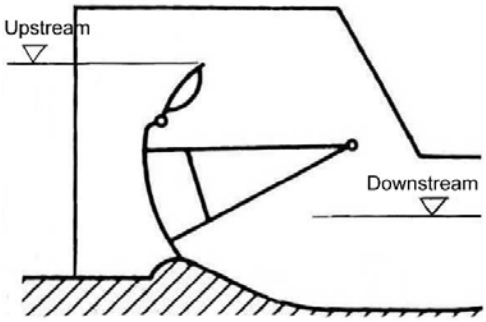
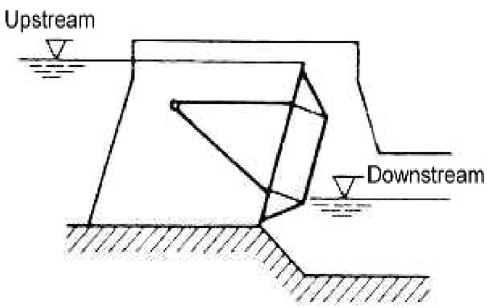
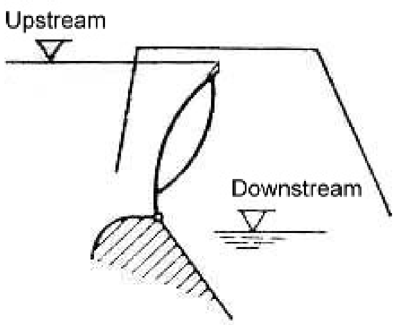
Code	**Gate type** in English German (D), French (F) and Dutch (NL)	**Sketch of gate-type**
1	Radial or taintor gate with compression gate arms D: Drucksegment F: Vanne segment avec bras en compression NL: Segmentschuif	
2	Radial gate (or Taintor Gate) with compression gate arms and upper flap gate D: Drucksegment mit Aufsatzklappe F: Vanne segment avec un clapet supérieur NL: Segmentschuif met klep	
3	Radial gate (or Taintor Gate) with tension gate arms D: Zugsegment F: Vanne segment à bras tendu NL: Segmentschuif met trekarmen	
4	Flap gate (Fishbelly-type) D: Stauklappe, Fischbauchklappe F: Vanne Clapet NL: Bodemklep	

Table 2.2 : Generic types of gates of movable weirs (Part II)

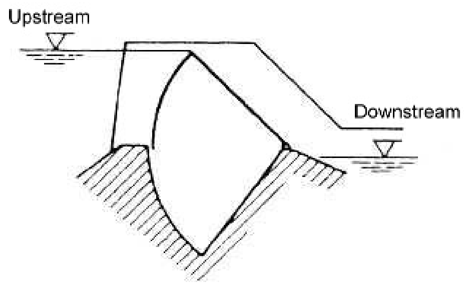
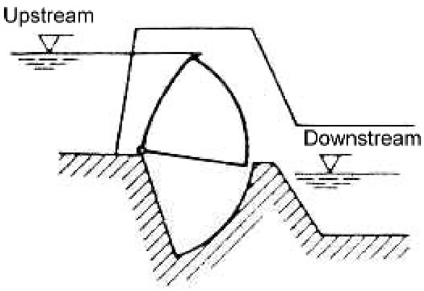
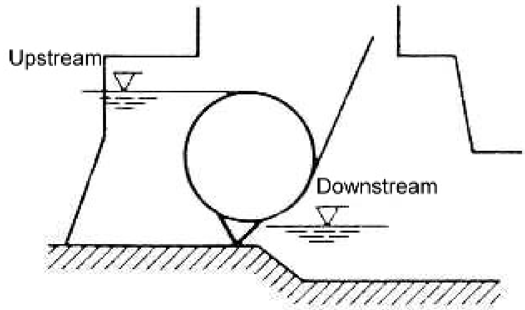
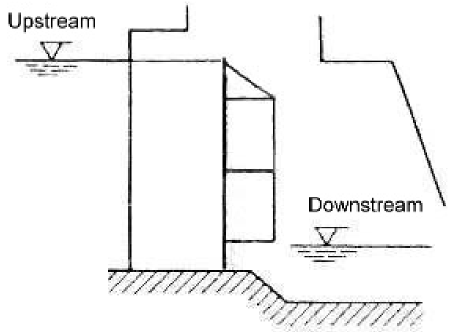
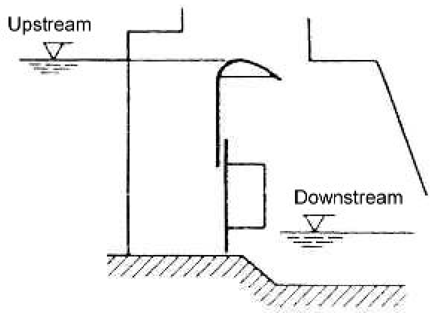
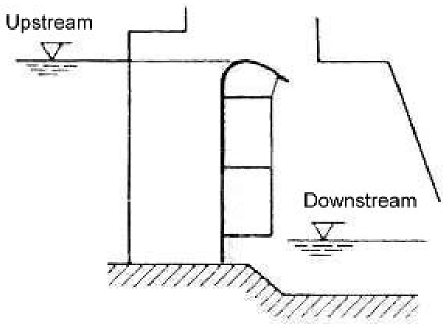
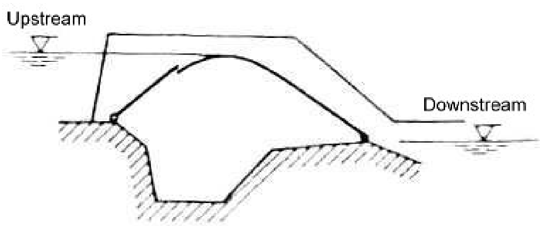
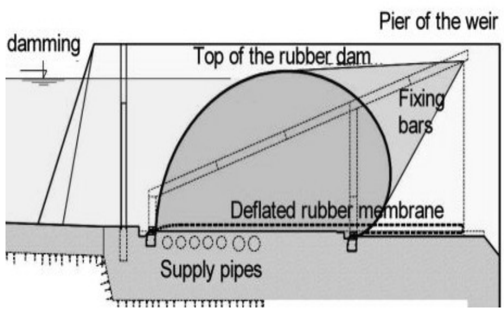
Code	**Gate type** in English German (D), French (F) and Dutch (NL)	**Sketch of gate-type**
5	Sector gate D: Sektor F: Vanne secteur NL: Verticale sectordeur	
6	Drum gate D: Trommel F: Vanne Tambour NL: Luchtkistdeur (trommeldeur)	
7	Roller drum gate D: Walze F: Vanne Cylindrique NL: Cilinderdeur	
8	Vertical lift gate (one-piece gate) D: Einteiliges Hubschütz F: Vanne levante (en une pièce) NL: Hefschuif	

Table 2.3 : Generic types of gates of movable weirs (Part III)

Code	**Gate type** in English German (D), French (F) and Dutch (NL)	**Sketch of gate-type**
9	Double leaf gate (Upper gate: Lifting hook type) D: Hakendoppelschütz F: Vanne levante avec hausse supérieure NL: Dubbele hefschuif met overlaat	
10	Vertical lift gate (Lifting hook type) D: Hakenschütz F: Vanne levante avec lame déversante NL: Hefschuif met overlaat	
11	Beartrap gate, roof weir D: Doppelklappe, Dachwehr F: Vanne toit NL: Dubbelklep, dakstuw	
12	Inflatable weir / Rubber dam D: Schlauchwehr F: Vanne gonflable NL: Balgstuw	

3. DESIGN PROCEDURE

This section provides a summary of the design procedures of the controllable weirs and gate structures essential for safe operation under environmental or other loading conditions expected during its operational life.

As an introduction (WG26-CD Directory /B3…/), the reader should note that the Voies Navigables de France (VNF) published a comprehensive guide "*Les Barrages Mobiles de Navigation*", for use by the project manager to design movable navigation weirs (VNF 1998).

Other publications worth mentioning are the U.S. Army Corps of Engineers "*Engineering and Design, River Hydraulics*" (1993) that discusses the design criteria for hydraulic structures including locks, dams, gates and spillways. "*Hydraulic Design of Navigation Dams*" (1987), "*Vertical Lift Gates*" (1997) and "*Design of Spillway Tainter Gates*" (2000) are other pertinent sources published by the U.S. Army Corps of Engineers.

Many of the existing documents on movable structures are very elaborate and detailed, and should be used as references. For examples: Bouvard (1991), Burt (1996) and Mockett et al. (2003).

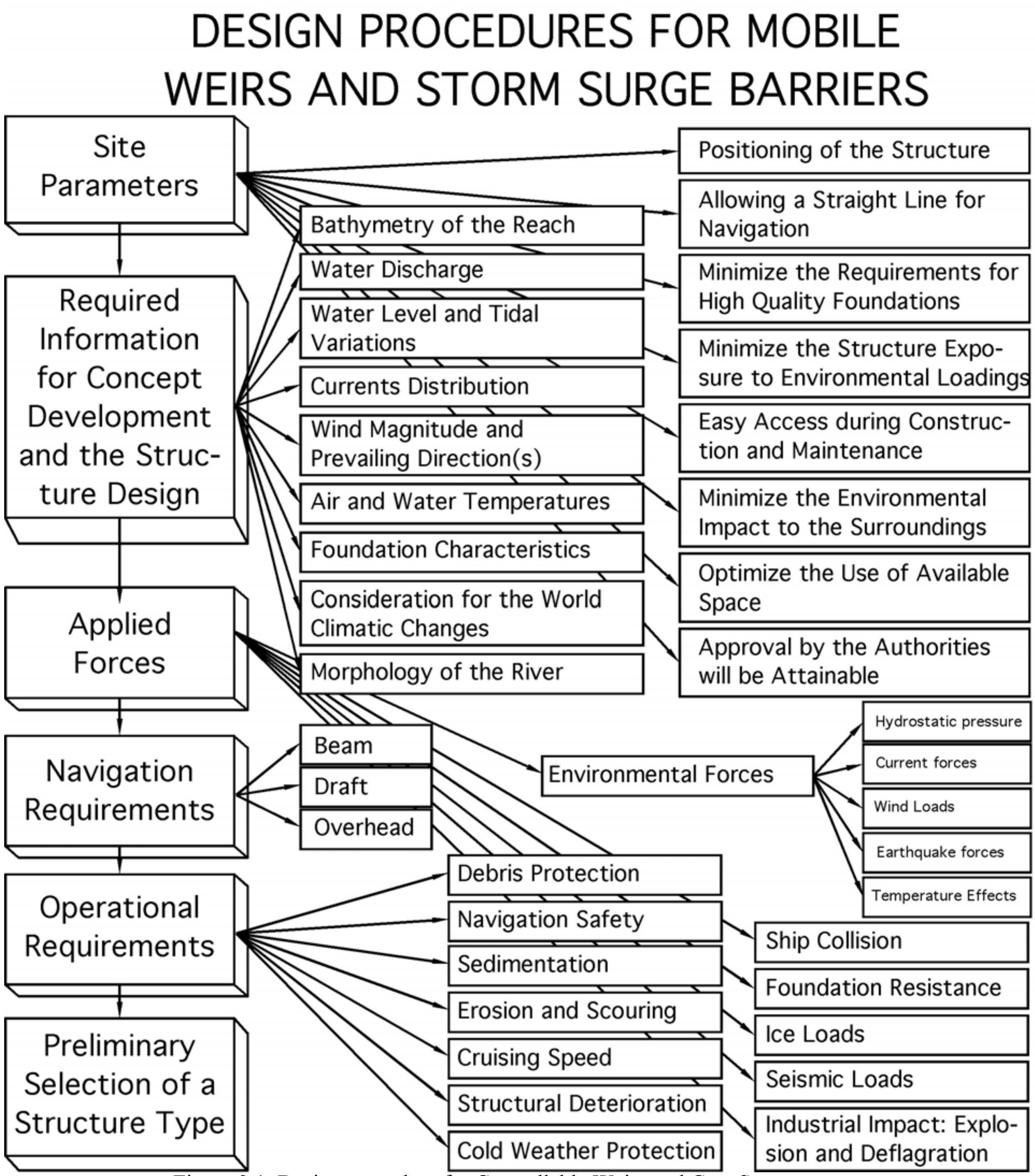


Figure 3.1: Design procedure for Controllable Weirs and Gate Structures

The design procedures of movable gates and barrier structures include a number of steps and associated parameters, which are (Figure 3.1):

- Site Parameters, as the selection of the site, depends on several factors (called here parameters).
- Required Information such as bathymetry, water discharge, wind magnitude, … and Loads that are necessary for technical analysis at concept development and later for the weir structure design.
- Navigation and Operational Requirements such as debris flow protection, navigation safety, sedimentation … that correspond to the user requirements to have save, efficient and reliable operations of the weir.
- Design Criteria that help the development of a preliminary analysis by assessing the degree of applicability of each type of structure to the proposed project site.

3.1 SITE PARAMETERS

The selection of the site depends on several factors that are interrelated and are weighted by technical, environmental, economical and political issues. Some of these factors are listed and discussed below.

3.1.1 POSITIONING OF THE STRUCTURE

The structure should be situated in a position that will minimize cross-current in the areas where ships navigate. The magnitude of the crosscurrents expected during the year should be considered in view of the ships expected to navigate this gate and their capabilities to manoeuvre and cross the gate. This is most important when the flow in the river is at its maximum or minimum discharges.

3.1.2 ALLOWING A STRAIGHT LINE FOR NAVIGATION

The structure should be situated in order to provide a straight line of sight with the structure navigational openings to facilitate the incoming ship to enter and exit the gate without the need to make sharp turns. The magnitude of the wind and its effect on the ship navigation capability expected during the year should be considered.

3.1.3 REQUIREMENTS FOR EXTENSIVE FOUNDATION WORKS

The softer the foundation, the more massive the structure is likely to be built. The site selection should include an evaluation of the existing geological conditions in order to minimize the foundation works. The increased size of the structure can influence other factors including cost and environmental impact.

3.1.4 MINIMIZE THE STRUCTURE EXPOSURE TO ENVIRONMENTAL LOADINGS

The applied winds, currents and associated hydrodynamic forces including hydrostatic pressure should be optimized. Consideration should be given so the layout of the structure minimizes its exposure to environmental loading. For example, the structure should be placed so it will not face the prevailing wind direction in order to avoid the associated waves. This is particularly important when the structure is built at the entrance of a long lake where the wind fetch is important. A long fetch will result in strong waves and large slamming loads on the structure.

For installations in a cold environment, the position of the structure should minimize ice loads, while ensuring all other factors are optimized. In addition, the structure layout should also minimise the ice fetch, which increases the loads applied due to the thermal expansion of the ice sheet.

For installation in warm climates, the position of the structure should be optimized with respect to the factors like: rainy season or monsoon flows, periodic droughts and floods, human or wildlife migrations etc.

3.1.5 EASY ACCESS DURING CONSTRUCTION AND MAINTENANCE

Easy access to the site during the construction period and later during its operation and maintenance is an important consideration.
In some cases, the structure will also incorporate a bridge, which adds a second dimension to the design that may not be considered otherwise.

3.1.6 MINIMIZE THE ENVIRONMENTAL IMPACT TO THE SURROUNDINGS

Modifications to the existing river channel should be selected in a way to minimize the environmental impact of the structure and its size.

Therefore, after all the other factors have been considered, the gate should be built at a site where there is a natural restriction of the river.

It is important to assess the long term and short-term effects of the construction of the structure on the fish habitat in the area, especially in critical areas.

Most projects will require an Environmental Impact Statement prior to planning permission being granted.

3.1.7 OPTIMIZE THE USE OF AVAILABLE SPACE

The type of the structure should take into consideration space that is available in the area. This is very important especially in densely populated areas where minimal disturbance to the residents and businesses already located on the river/channel is important to diminish any potential resistance to the project construction.

3.1.8 APPROVAL BY THE AUTHORITIES WILL BE ATTAINABLE

The design should be compliant with the rules and regulations of all concerned authorities (Coast Guard, Department of Environment, Department of Transport, etc.).

During the progress of the work, a preliminary design should be completed and submitted to the appropriate authorities for approval. This will bring forward any missed guidelines, regulations or rules that may not have been considered in the design.

3.2 REQUIRED INFORMATION FOR CONCEPT DEVELOPMENT AND THE STRUCTURE DESIGN

3.2.1 BATHYMETRY OF THE REACH

Technological advances in the area of Geographic Information Systems (GIS) and river mapping and currents measurements made the collection of accurate data affordable.

3.2.2 WATER DISCHARGE

The structure should be designed to take into consideration the design discharges, including maximum, minimum, and the average expected during the year and during each season. The flow restriction across the structure and its impact on flood sensitive areas upstream and downstream should also be considered.

Changes to the historical discharge of the river that may affect future predictions based on statistical analysis, expected during the life of the structure, should also be considered. For example, water diversion, water loss due to evaporation, and seepage due to construction of dams upstream that may affect the flow, should be considered, including global warming. The design discharges should consider the watershed, the future development of the area, which affects the discharge due to runoff from rain and snowmelt.

The historical discharge of the navigation channel should be analysed and the design discharge should be defined based on a desired recurrence interval (100-, 1000- or 10,000-year storm).

3.2.3 WATER LEVEL AND TIDAL VARIATIONS

The water levels are affected by the variations in the river discharge and the water control structures upstream and downstream of the structure. The design water levels should be obtained using the historical water level data. The design minimum, maximum, average and water level should be calculated.

The maximum water level difference between upstream and downstream for all possible scenarios including during an open and closed gate (design water level difference) should be defined. This is important for the development of the design including, the hydrostatic pressure on the structure, the excavation required to allow the design vessel to pass through, the resulting currents, and the associated erosion of the river bottom.

3.2.4 DISTRIBUTION OF CURRENTS

The structure should be positioned to ensure the current circulation surrounding the structure, upstream and downstream, is acceptable for navigation. The current magnitude and direction should also be used to calculate potential drift velocity of a vessel to ensure that no damage would occur to the structure should the vessel strike any of the gate structures.

Numerical and/or physical models should be used to define the current distribution (see a list of these numerical models and the tools used to obtain currents and bathymetry data in the Appendix A). The design current distribution depends on the water discharge. The design discharges and the water levels, 100- to 10000-year maximum of minimum recurrence intervals, should be obtained and used as input to a numerical model for the prediction of the current distributions.

Obtaining the current distribution is particularly important in tidal estuaries where the currents change significantly over the period of a tidal cycle.

3.2.5 WIND MAGNITUDE AND PREVAILING DIRECTION(S)

The wind speed and its magnitude should be obtained from the historical database. The wind speed and its direction should be analysed and the design wind rosette should be obtained for the desired recurrence intervals. The design maximum wind for each of the main prevailing directions should be obtained.

The maximum wind speed and its prevailing direction are used as input for many purposes including:

- Ship manoeuvring and effect of loss of control on the drift of the vessel
- Wind effect on the water levels (seiche effect) and waves generated by wind, especially for large wind fetches
- In cold region environments where ice covers the reservoir upstream of the structure, wind force due to wind drag on the ice surface generates an ice force on the structure

In the case of two large lakes connected with a channel with relatively short distance, long lasting, unidirectional winds can drag the water masses towards one lake away from the other leaving a significant water level change at the gates. This phenomenon appears next to the so-called seiche effect. It can significantly contribute to the gate hydraulic loads

3.2.6 AIR AND WATER TEMPERATURES

The structure should be designed to resist thermal expansion and the associated internal stresses from the daily and seasonal temperature fluctuations expected during the life of the structure.

The historical temperature records in the region should be

analysed and the design temperatures including the maximum, minimum, freezing degree-days, and heating degree-days should be defined based on a desired recurrence interval (100- or 1000-year event). For the structures above water, the design temperatures should include the warmth radiation effect, which produces higher temperature differences than the ambient temperatures.
The extreme values of the air and water should be used independently. In some cases these values can be correlated to select the peaks for each season.

3.2.7 FOUNDATION CHARACTERISTICS

The type of bottom materials should be obtained from past projects carried out along the stretch of the channel. If data is not available, drilling to obtain cores should be performed to identify the depth of the soil layer over the rock and its sediment distribution. A sediment distribution map should be drawn to clearly illustrate the dominance of each type of material, sand, silt, and clay. The type and various rock layers should also be defined up to the required depth defined by the geotechnical engineers.

In some (usually highly urban) areas, the river foundation at site location might significantly be affected by human activities in the past. This can include objects of historical value, old foundations, vessel wrecks, munitions, other warfare, industrial contaminations, etc. Proper investigations and a reliable excavation procedure should be carried on in such cases.

This topic is more developed in the structure foundation section (Section 5.3).

3.2.8 CONSIDERATION FOR THE WORLD CLIMATIC CHANGES

Sea level rise is one of the main issues in the design of barriers (storm surge barriers). Therefore, the design should be flexible to take into consideration the world climatic changes. This can include changes in the water discharge, water level, air temperature, wind magnitude, etc.
IPPC(2001) published a report “Greenhouse Effect and Sea Level Rise: The Cost of Holding Back the Sea”, which provided the prediction presented in Figure 3.2.

Based on this figure, during the next 100 years, the sea level may rise between 10 cm and 100 cm (depending of the future earth warming trend). To simplify the design criteria and reduce any potential risk of structural damage or flooding, a value should be selected. It is recommended, as a worldwide average design figure, that the value of 40 cm be selected for the sea level rise in the next 100 years. In the Netherlands, the sea level is considered to rise 25 cm in 50 years and 50 cm in 100 years. Note that the Dutch consider 65 cm for the next 100 years as local land sinking is included (Mockett and Simm, 2003).

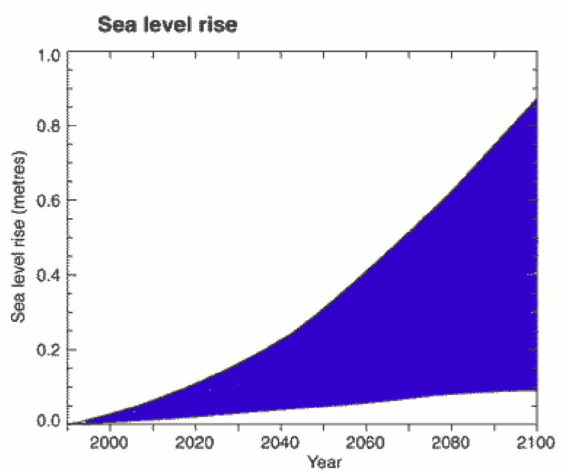


Figure 3.2: IPCC Third Assessment Report (2001)

3.2.9 MORPHOLOGY OF THE RIVERS

Morphology of the river is an important factor to consider when assessing the response of the river to the structure. The presence of a structure in a river or an estuary will affect the hydraulic regime of the channel by modifying the flow and affecting the sediment transport regime.

The water levels and the flow will be affected by the operation of the structure gates during the opening and closing. The ship navigation will add to this change of regime by introducing other physical phenomenon including the propeller wash and squad effect (PIANC’s InCom-WG27 reviews this topic).

The project should include procedures to deal with the regime change and ensure it would not result in significant problems. For example, estimates of the sediment deposits along the channel should be made and solutions developed and implemented.

3.3 APPLIED FORCES

The applied forces on the structure are examined in this section. These forces are generated and their magnitudes are directly related to the environmental parameters defined in Section 10.2 as follows:

3.3.1 ENVIRONMENTAL FORCES

- **Hydrostatic pressure**: Define the hydrostatic pressure and the distribution of the pressure on the structure and the gates. The hydrostatic pressure should be calculated for the maximum differential water level between upstream and downstream of the structure. In tidal estuaries, the hydrostatic pressure is to be calculated for both upstream and downstream flows. The water level difference should be obtained from the historical records and considering the expected life of the structure.

- **Current forces**: Current forces are a combination of the current distribution in the upstream end of the structure and the local current generated when the gates are opened. The forces should also include the turbulence generated and the associated vibratory motions during the opening and closing of the gates.

- **Wind Loads**: The wind effect on the loads on the structures and the associated safety and operation problems should be calculated. The loads include:
  - Wind Loads on the structure due to the drag forces
  - The wind generated waves (which depend on the fetch) and the associated waves slamming forces
  - The additional hydrostatic pressure induced by the water level rise (seiche)
  - The wind forces on the transiting ships to evaluate the risk of accidents and therefore shut down the system for winds that exceeds set limits

- **Operating forces**: Friction forces need to be considered where applicable as neglecting or underestimation friction forces may lead to gate failure.

- **Earthquake forces (seismic)**: The seismic loads should be calculated for the structures based on existing guidelines and seismic maps (Section 3.3.5).

- **Temperature Effects**: Thermal stresses and strains due to change of the temperature of the structure mass should be considered.

- **Icing Loads**: Icing of the structure can add a considerable additional deadweight load on the structure. Estimates for the accumulated ice should be calculated and considered in the design (Section 3.3.4).

3.3.2 SHIP COLLISION

Surge barriers constructed on navigation routes have to be designed to take into account the impact forces resulting from ship collision against a fixed structure.

Ship collision is a loading case of low probability but potentially with high consequences. It is practically not feasible to let the structure withstand all theoretically possible ship collisions. In such cases, the designer should try to impose the least harmful (for both the structure and the surroundings) damage mechanism on the structure, rather than to avoid any damage.

Typically, analyses are made for a number of ship interaction scenarios and for various ship characteristics such as type of transiting ships, the barrier position, the ship's probable speed (vs. allowable speed), the angles of impact against the gate piers. The use of ship simulation models help assessing the difficulties expected during transit. Physical modelling is also another option and it could be more cost effective for some projects.

Once the ship and environmental parameters are defined, impact forces are determined using data from a look-up table published in the literature or using computer simulation using finite element models. Collision forces are developed during plastic deformation of the ship's structure against a relatively rigid concrete or steel structure (Le Sourne, 2003). These forces can be used for dimensioning of the pile foundation and the pile structure (PIANC-WG 19, 2001).

On the other hand, this process can also give rise to the development of shipping regulations in the vicinity of the barrier thus limiting the probability and magnitude of the ship collision impact. This regulation can limit navigation speed and imposes limits for ships above a certain tonnage.

3.3.3 FOUNDATION RESISTANCE

The foundations should be capable of resisting the forces applied by the environment plus the structural dead weight. The amount of the structure settlement should not exceed the allowable settlement. For example, the type of bottom sediments and the presence of rocks can affect the type and the layout of the structure selected. The cost of the structure can increase significantly if the bottom sediments are weak in comparison to a rock foundation.

Piles or post tensioning dam anchor requirements should be considered to reinforce soft soils.

3.3.4 ICE LOADS

The structure should be capable to resist ice forces on the structure for the determined life duration of the structure. These forces are expected if ice forms on the water. The loads include thermal pressure and dynamic loading resulting from impact of ice during the break-up. The type of ice, its thickness, its properties, particularly strength, should be defined during the development of the concept design stage of the project.

Various processes generate ice loads. The most important ones are:
- Static pressure due to thermal expansion
- Static pressure due to water level fluctuations
- Static loads due the driving forces applied by the drag forces from currents and winds on the ice upstream of the structure.
- Dynamic loads due to the impact of ice floes with the structure and its components.

State-of-the-art design procedures for static loads are: Comfort et al. (2003 and 2004); "Ice Engineering and Design", U.S. Army Corp of Engineers (2002), and "Technical and Economic Problems of Channel Icing" (PIANC WG23, 2004).

On the CD-Directory /Annex Section 3/ a short description of the load calculations for a structure and the gates is provided.

3.3.5 SEISMIC LOADS

Scientists and engineers around the world have studied the effect of seismic motions and their effect on civil engineering structures (Hadjian 2004, Dowrick 2003, and

Naeim et al. 1999). Recently, numerous studies have been carried out on seismic response of dams (http://www.struc.polymtl.ca/dams/chapter15.htm). These studies provide guidelines and standards to take into consideration during the design of hydroelectric dams.

3.3.6 INDUSTRIAL IMPACT AND TERRORISM: EXPLOSION, DEFLAGRATION

Surge barriers constructed in an industrial surrounding (e.g. a harbour), have to be dimensioned to withstand explosion forces and planned activities.

Existing industrial activities in the vicinity of the (future) barrier have to be analysed, as well as regulations regarding permitted activities now and during project life of the barrier. Depending on the type of existing and/or future activities in the surroundings, explosion forces may occur. Typically a distinction is made between detonation and deflagration pressure waves, and their probability of occurrence.

Furthermore, probable propagation directions of the shock waves have to be determined. On the leeside of the barrier (pile or gate), forces are different than on the windward side where reflection of the pressure wave exists. The exposed pile or gate is then calculated for the resulting pressure taking into account time shift of the wave pressure between windside and leeside, together with its reflection on the windside.

If those forces cannot be resisted, and no other alternative designs are possible to develop, a physical protection system should be considered. Several designs exist on the market with good potential to reduce the risk of these explosions (See “Seacor” and “Whisper Wave” in the web sites in Section 10.1).

The protection against terrorism is an important consideration, especially for those structures with grave consequences. The structure should be designed to minimise the risk from terrorism related damage.

3.4 NAVIGATION REQUIREMENTS

For Inland navigation, ship sizes are defined through rules. In Europe each river corresponds to a class (i.e., Class Vb is the larger class), which refers to the maximum ship sizes. So, navigation requirements are totally defined by specifying the river class. Similar regulation exists in USA (CEMT, US Class). Therefore, the structure should not, in any way, compromise the existing or potential navigation capabilities of the waterway. For example, The Tees Barrage (see Project Review on the CD Directory A1) was not going to have a lock due to cost constraints, but under public pressure from boat clubs and others petitioned against the Act of Parliament to have a lock included.

The structure should allow the passage of the largest vessel class that is expected to transit the navigation channel. In an estuary, this will be limited to the width and draft of the vessel to transit the narrowest and shallowest point along the vessel route, upstream and downstream of the structure. Change in sedimentation and erosion that may affect the depth of water, restricting navigation should also be considered. This will be discussed in more details in Sections 3.5.3 and 3.5.4.

The dimensions of a typical vessel should not exceed the allowable clearances selected to ensure the safe operation of the structure. The length of the vessel is limited by the infrastructure of the navigation channel. Therefore, the following vessel characteristics should be considered in the design procedure:

- **Beam**: In addition to the beam of the vessel, the structure should allow sufficient clearance for minimising the damage to either the vessel or to the structure.
- **Draft**: The structure should allow the ship to cross the structure as safely and as quickly as possible.
- **Overhead**: The total height of the vessel (loaded) should be below the height of the structure for the highest water level.

The following operating requirements should take into consideration:

- The maximum and minimum water discharge range and maximum water level changes expected when the ship is allowed to cross the structure.
- The currents in the navigation channel should be below the critical current speed that would create a safety risk to the ship while manoeuvring in the navigation channel.
- The vessel cruising speed should be regulated if the water depth is below the critical depth that would cause a vessel to squat, which may cause damage to the vessel and the structure and present a safety risk to the navigation system.

3.5 OPERATIONAL REQUIREMENTS

3.5.1 DEBRIS PROTECTION

The presence of debris and its accumulation along the face of the structure can represent another hazard to the structure. This can be manifested by improper operation of the structure gates and perhaps allowing higher than expected loads on the structural components.

This problem is particularly important in rivers where debris is a common occurrence. Debris can prevent the safe operation of the structure’s gates and associated equipments and can result in significant maintenance cost.

The structure should be positioned to minimize the effect of debris, algae, grass, etc. and to facilitate their collection and disposal.

Floating booms specially designed for the diversion and collection of debris are available on the market. There are various types and selection will depend on the type of debris expected. Wooden logs or steel beams have been used successfully to divert the debris toward a specially designed collection area, where easy access by on land equipment can remove the debris. State-of-the-art boom design procedures are provided in Abdelnour et al. (2003).

River bank management such as pruning trees and collecting debris on land can be less costly than collecting the debris from water.

3.5.2 NAVIGATION SAFETY AND SIGN

Should any area be a danger zone, warning signs should be displayed so it can be seen from a far away distance.

Depending on the structure size and the hydraulic conditions of the channel, warning signs may not be sufficient and a boom structure is more likely to be a more effective deterrent to boaters. Specially designed booms have been used successfully at hydroelectric dams, on the non-navigable part of the structure. Various types and shapes are available on the market. Note that design criteria for booms should be developed for each specific site. Considerations should be made so the booms can keep small boats and people from danger. The boom should be designed for the highest discharge conditions, to ensure failure will not occur when the danger is greatest. So the design safety margin must be large.

The boom should also be designed so that under no circumstances a failed boom may become a projectile that may impact and harm the gate. The damage mechanism of the boom must properly be controlled.

3.5.3 SEDIMENTATION

Sedimentation occurs due to sediment erosion upstream, which are put into suspension then transported along the channel downstream. These sediments get deposited when the current velocity becomes less than the critical current velocity that is required to maintain the particles in suspension.

Several factors cause soil erosion. Sediment is eroded due mainly to water currents, particularly during the runoff during the rainy season. The sediment usually comes from a distance upstream and is very different from one location and another, and can vary significantly from one day to the next and from one season to the next.

In a navigable channel, erosion can also occur due to the propeller wash, where the currents generated are very significant, especially along the shallow stretch of the channel. The erosion can also be significant if the ship mass (or draft and beam) is relatively large in relation to the channel. The density of the shipping activity, the typical ship power and speed in the navigation channel are also very important factors.

Therefore potential sedimentation along the navigation channel, and close to the gates should be evaluated and minimized. The main area of potential problem is the water depth along the reach leading to the structure, where upstream and downstream of the structure may be filled with sediment and rest below the minimum water depth (design water depth). The problem can be very severe if the sediment content is high and if the current velocity distribution close to the structure is less (or if the current fluctuates during the opening and closing of the gates).

Another important problem is the disturbance of the polluted bottom materials from old industrial areas due to discarded toxic chemicals. This is a serious problem in most countries including the United States, Germany, Netherlands, Canada, Britain and Belgium. Therefore, neutralising or cleaning these sites would be necessary to eliminate the potential problem.

Based on the type of soil, the soil particle distribution and the current distribution, physical model and/or numerical models can be used to predict susceptible areas where sedimentation can occur.

3.5.4 EROSION AND SCOURING

Erosion at various locations around the structure and along the navigation channel can occur due to the obstruction to the natural flow of the river caused by the structure.

The erosion is due to local currents generated by waves or from ship movements close to the structure. Opening the structure gates also generates large local currents that should be considered during the design of the bottom scour protection of the upstream and downstream ends of the structure.

Hydraulic models can be used here as well to define susceptible areas where erosion can occur.

3.5.5 CRUISING SPEED

As discussed in above sub-sections, the vessel speed is the cause of many problems. The cruising speed should be regulated (reduced) in some reaches where the currents are relatively high and/or unpredictable.

Cruising speed of transiting vessels affects the safe operation of the gate structure. In most navigation channels, there are regulations to limit the vessel speed (PIANC-WG 41, 2003).

3.5.6 STRUCTURAL DETERIORATION

Minimize wear of moving parts and corrosion of all steel structures and steel used in concrete. The structure should be designed to resist wear and corrosion and ensure that its structural integrity will not be compromised during the lifetime of the structure.

The actions expected on the structure from waves and currents should be evaluated. The parts of the structure exposed to various levels of wear should be designed to minimize the wear of the components used.

Prevention of corrosion should also be considered, especially when the structure is exposed to seawater. Cathodic protection should be considered on all the structural components exposed to seawater (PIANC-WG 17, 2004).

The applied protection systems against corrosion should be selected taking into account the environmental issues. These systems must periodically be renewed. If the blast-cleaned old coating is not properly collected, this operation may harm the environment.

3.5.7 COLD AND WARM WEATHER PROTECTION

3.5.7.1 Cold Weather Protection

This issue should be considered when the construction would be in cold regions. The coldness of a region is expressed by an index related to the mean daily temperature. The freezing degree-days (^{o}C-days) index is the accumulated degree-day for the entire winter season (Assel, 2000). A freezing degree-days above 50 ^{o}C-days, will result in the start of the formation of ice on the water. Therefore, the analysis of past temperature records should be carried out to decide if the design accurately considers the cold temperature.

When a decision is made to design for cold weather, the following should be considered:

- Calculate the average and maximum natural ice thickness growth expected in the navigation channel.
- Estimate the ice loads expected on the structure from published information in the literature (Comfort et al. 2003, 2004, and CD's Directory /Annex Section 3/).
- Design the structure to minimise the effect of ice and icing build up on the structure walls and gates. Ice, if not considered, may prevent the opening of the gates when required. Solutions to gate icing problems consist of using air bubbler systems, water circulating pumps, plates heating systems of the gate guiding system, the use of steam to deice the gates. This approach is manual and is justified if icing is considered a rare event at this particular location.
- Ice in the navigation channel may also require removal to allow navigation to proceed. Icebreakers are used to manage the ice and ensure the ice cover remain stable throughout the winter season so flooding does not occur. Solutions to this problem are discussed in PIANC-WG 23 (2004).

3.5.7.2 Warm Weather Protection

The warm weather and direct sun exposure should also be considered when the construction would be in relatively warm environment. The warm temperature will affect many construction materials. It accelerates for example the corrosion of steel and the deterioration of concrete in salt environment.

3.6 PRELIMINARY DESIGN OF A STRUCTURE TYPE

The review of the design criteria in this section helps the development of a preliminary analysis to assess the degree of applicability of each type of structure to the proposed project site. These alternatives are then rated (See Section 4: Multi-criteria Assessment) and the best two to three alternatives are selected for further examination and analysis. The analysis to be considered are those relevant to the structure design, construction and operation.

A non-exhaustive list of criteria is introduced here that can be used for the preliminary design selection of the most appropriate structure type(s) for the specific site and its environment.

For a given project, the selected criteria should be assessed with significant input from the communities living in the region where the proposed project is to be implemented and its surroundings. In the following Section, the multi-criteria assessment methodology is extensively discussed. In this section, only an introduction is discussed:

- Reliability and Operational performance: Provide a structure that is efficient, i.e., has high river flow control accuracy, high closure reliability.
- Safety: Ensure the top level of safety for the operation staff and the users.
- Environmental Impact: Minimize the impact of the structure to the environment, locally and globally. This helps reduce resistance that may arise during the public hearing for the project approval by the authorities.
  - Sedimentation and erosion of the river bottom
  - Affecting the fish habitat
  - Environmental oil spill
  - Shore erosion from ship generated waves
  - Level of noise generation from traffic and from the operation of heavy machineries
  - Architecture of the structure so it sits in harmony with the environment
- Cost: The cost, capital, operation and maintenance, are essential factor that influence the selection of the proposed structure. Cost benefit studies are usually complex to execute since many of the benefits are not tangible enough to be included. Considerations for the benefits analysis of the project include:
  - Construction cost
  - Cost associated with the fabrication of the structure components
  - Definition of the total maximum allowable time and cost implications of completion deadline
  - Cost allocation for interruption to navigation
  - Number of direct jobs during execution and during operation of the structure
  - Post-construction repair maintenance and management cost

3.7 RELIABILITY AND SERVICE LIFE

The reliability and service life are two important parameters for the design of movable structures. Reliability is expressed in terms of probability of failure and service life in a minimum number of operation years under the (globally) agreed maintenance approach. This topic is discussed in details in Section 5.6.

4. MULTI-CRITERIA ASSESSMENT

4.1 NECESSITY OF A MULTI-CRITERIA ASSESSMENT

Both river movable weirs and costal barriers are structures that have great economical, environmental, and other impacts to large areas. The weir and the barrier projects usually affect many people in many different ways, varying from the safety of their homes to the nature of their means of income. The processes, which generate these effects, are often complex, and can be short-term (e.g. immediate solution to flood problems) as well as long-term (e.g. agricultural, ecological, or even climatic changes).

A gate type selection is a significant part of these processes. There are far-reaching consequences of choosing one gate type above another. Though gate type selections usually take place when the global project requirements are known, they can still affect such principal issues as:

- Weir/gate location – as not all gate types are suitable for all locations;
- Waterway navigability – as the gate type selected may promote or halt navigation.;
- Flooding risk – as not all gate types are equally stable, watertight etc.;
- Water flows, bottom and shore erosion – as different gates give different flow patterns;
- Water ecosystem – as not all gate types allow, for example, for a fish passage;
- Local economy – as gates can provide one kind of work and/or destroy another;
- Local energy balance – as gates can be suitable for energy generation or not.

It should, therefore, be clear that the gate type selection is a matter of engineering, economy, politics, or any other privileged discipline, and its people. It is, in fact, a matter of the entire communities living or having other interests in the areas in question. These communities and areas can be very large. In extreme cases, different interests in this matter result in international disagreements. For practical reasons, the gate type selection is usually made by the engineers. They should, however, be aware of all different interests involved; and seek a balance between those interests. The gate type selection can be assisted using multi-criteria assessment methods.

4.2 SOME HISTORICAL BACKGROUND

Historically – as far as known – engineers always analyzed the feasibility of various gate types and submitted their recommendations to the community or political leaders to make the final decisions[1]. What differed from the current practice was mainly a smaller depth of the analysis and a potentially less balanced (often arbitrary) selection. This did not prevent many remarkable feats of gate engineering from happening. For example, the world's first miter gates were - in all likelihood - constructed in the 15th century in a link of the Italian Navigilio Grande Canal to Milan. However, in terms of selection criteria, there was just one, which mattered in that project: The gate had to facilitate boat passage for the transport of marble to build the Milan cathedral (Erbisti 2004), (Canal Monuments Web site, 2004).

The medieval revival of hydraulic engineering taking place in Europe, gradually introduced additional criteria. At least three of them (navigation – not only to build cathedrals, water supply and land safety) became a common practice. The relation between them and the gate type selection was, however, still a case of lead engineer's and his principal's personal view. In some countries (Netherlands, Flanders, and later, France) relatively more people were involved in decision-making than in other countries (United Kingdom, Austria, and Germany), but it did not necessarily mean that the selections were better balanced. In the Netherlands, for example, the deciding parties were large but their members represented oligarchs (merchants, bankers etc.) who would not mind to set farmlands under water in order to win a local war. They would also do a lot to obstruct the navigation heading for the concurrent Belgian harbor of Antwerp.

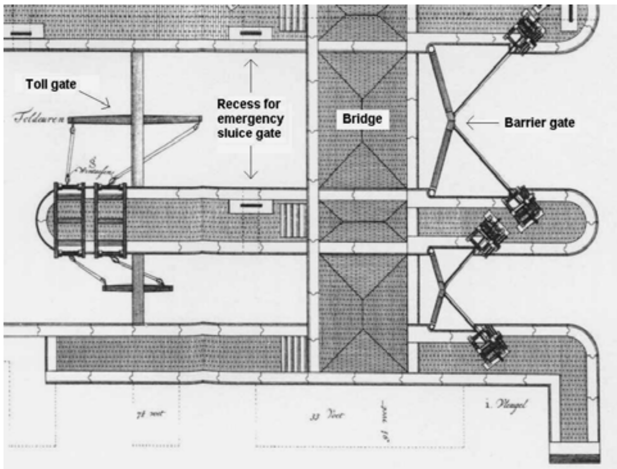


Fig. 4.1: Part of a sea lock in Muiden, Holland, 17th century, (Van den Horst, 1981)

In the 17th and 18th century, there were already good drawings showing gate layouts and details. A typical barrier controlled the access to city moats; and consisted of outer gates and tollgates. Toll charging was a cherished income source for many small states of Germany and the Dutch provinces. Fig. 4.1 shows that different gate systems were

[1] The exceptions – like the czar Peter the Great who studied maritime engineering in the Netherlands and had personal contributions to some projects in St. Petersburg; or George Washington, a surveyor and an engineer himself, who initiated a number of canal projects in the USA – serve only to prove this rule.

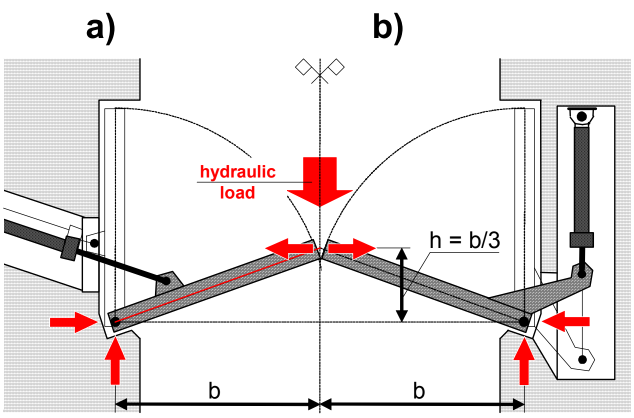
used for different purposes. This proves that a kind of gate type assessment has been performed.

The Industrial Revolution brought further changes. The French Revolution and the emergence of the United States gave voice to new social groups, while the industrialization lead by Britain and Germany resulted in large hydraulic projects. Many new gate systems were developed which gave more significance to the issue of gate selection. Although nobody spoke about the selection criteria yet, gate types were considered to have different advantages and disadvantages in various fields. Apparently, there was little consensus about it between the engineers. Everybody had their own preferences. These were the times when, e.g., Isambard Kingdom Brunel built the 42.7 m wide mitre gates in the Portbury Locks in Bristol, still the largest of this kind in the world (Pugsley, 1974). We would probably select another gate type for that project today.

Such a consensus begins to appear at the end of 19th and the beginning of 20th century. Advancing technical education and the emergence of technical literature played a major role in it. Engineering is not an elitist activity with the air of secrecy any more. The lead in this process belongs to two famous French schools, *Ecole des Ponts et Chausses* and *Ecole Polytechnic*, the British *Mechanics' Institutes* and the German *Technische Hochschulen* - not to minimize the role of educational institutions in other countries. A typical example is the discussion by H. Kulka (1928) on advantages and disadvantages of the known weir gate systems. The author concludes each gate type presentation by a section "Kritik der …wehre" (Weir evaluation), where quite a wide range of criteria is considered. Several of these judgments may well be applied today still.

The multi-criteria analyses, as performed today, were largely introduced after the World War II. The main reasons for their growth were (and still are) as follows:

Fig. 4.2 : Gate types considered as flood barrier on the Meuse-Waal Canal in Heumen



- growing urbanization and complexity of large infrastructural projects;
- emancipation of different social groups, interests, ideas etc.;
- call for more transparency and balance in all projects concerning the environment;
- demand for cost optimization and good cost control;
- the logics of computerization, desire for programmable selection procedures.

4.3 METHOD OF QUALITATIVE ASSESSMENT

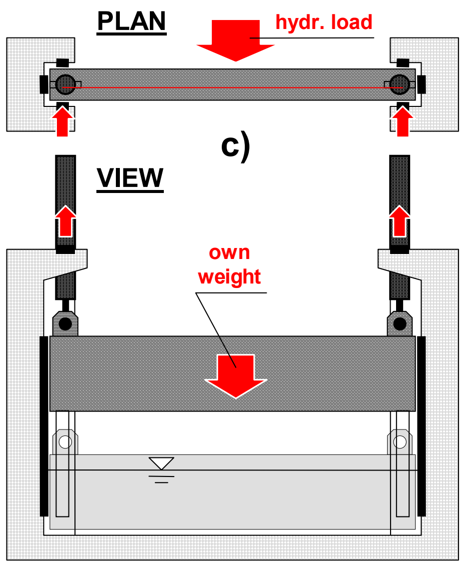
In general, a multi-criteria analysis is a procedure which should result in a matrix in which different options are evaluated with respect to different criteria, see Table 4.1. Completing such an analysis means - simply speaking - giving values to the matrix elements.

Option Criterion	Gate type 1	Gate type 2	Gate type n
Criterion 1			
Criterion 2	SCORE RATING		
Criterion m			
Total			

Table 4.1 : Basic matrix of a multi-criteria analysis

The two main questions of a multi-criteria gate type assessment are:

1. How and in which units to measure the scores of gate types in each criterion?
2. How to convert these scores to the same units in order to make a total assessment?



Option Criterion	Mitre gate (a)	Mitre gate (b)	Vert. lift gate (c)
Total costs	-	--	+
Operation	+/-	+/-	++
Navigation	+/-	+	-
Maintenance	+/-	+	+
Environment	+	+	++
Aesthetics	+	+	-
Total	**+/-**	**+/-**	**+**

Herein:
++ very good;
+ good;
+/- fair;
- poor;
-- bad

Table 4.2.: Simple (only qualitative) analysis for the flood barrier in Heumen

The simplest solution is to ignore these questions by using qualitative descriptions with no quantitative values. Several gate type selections have been performed in this way. Figure 4.2 shows an example of the gate type assessment for a flood barrier on one of the Dutch navigable waterways, the Meuse-Waal Canal, in Heumen. In a rough pre-selection only three gate types had been considered suitable for this project: two mitre gate options and a vertical lift gate. The final matrix of a multi-criteria analysis was filled as shown in Table 4.2.

The vertical lift gate appeared to be the preferable solution in this particular example. Both types of miter gates were more expensive and were also less reliable when an emergency closure is required. The unlimited overhead space for navigation and the aesthetics were not considered substantial enough to compensate these two disadvantages.

Such an analysis is entirely based on subjective judgments of a person or a team. As the matrix contains no numerical values, there is practically no way to verify the performance assessments of the gate types considered. Nevertheless, this simple method can be considered sufficient in a number of situations when, e.g.:

- There is no time or money to perform a better, quantitative analysis. The latter (money) is actually a poor reason to avoid performing a good analysis – wrong gate selection always cost more – but the principal may have a different view in this matter. This happens e.g. when the money does not all come from the same pocket.
- The analyzed case is rather simple – like in the discussed example. It may be efficient then to make a simple, qualitative assessment; and decide later whether more effort should be expended on gate selection. This in fact was the approach taken in the quoted case.
- The customer has already made a choice and he does not want any discussion about it. Yet, he appreciates some kind of "educated justification" in case he is asked to give an account of it. If this does not conflict with the engineer's ideas, he may do it.

The last situation shows that the method of qualitative assessment is manipulative. In general, one is advised to lay it in the hands of more specialists, if possible from different organizations, profiles, etc. However, this method can delay progress. A correct, quick assessment is often preferable to long discussions, which can result in a general impotence to get anything done.

4.4 METHODS OF QUANTITATIVE ASSESSMENT

4.4.1 ASSESSMENT IN TERMS OF COSTS

In order to provide a better, traceable gate type assessment, answers must be found to the two main questions from Section 4.3, i.e.:

1. How and in which units to measure the scores of gate types in each criterion?
2. How to convert these scores to the same units in order to make a total assessment?

These answers are not easy, as the gate performances can clearly be quantified in some criteria (e.g. costs in euros or dollars), less clearly in other ones (e.g. navigation in ship passages) or not at all in still other ones (e.g. aesthetics or environment in Section 5.7). Not to mention one uniform unit for all the criteria.

As far as is known, there are two strategies to deal with this problem (excluding the one which ignores it, see Section 4.3):

- Expressing everything in terms of costs (in currency units);
- Performance rating and the use of weighting factors.

An argument for the first strategy is that project costs are always one of the most important selection criteria – and this criterion is certainly the best quantifiable. As this criterion often dominates the analysis, the idea is to give values in currency units to gate performances in all other criteria as well. Such an approach answers both questions from the beginning of this section. In support of this strategy, some other criteria – like maintenance or operation – can indeed be measured in currency units to some extent. The maintenance and operation costs over the entire service life must first be capitalized, then added to construction costs.

An example of this approach is a recent study on modernization options for the existing weir on the Meuse in Sambeek, the Netherlands (Pouw et al. 2000). Four options were considered suitable for this project (Fig. 4.3):

a) Vertical lift gate with a flap section;
b) Sector gate of Thames Barrier type;
c) Top-hinged (suspended) flap gate;
d) Bottom-hinged flap gate.

The analysis in terms of costs resulted in the matrix as shown in Table 4.3.

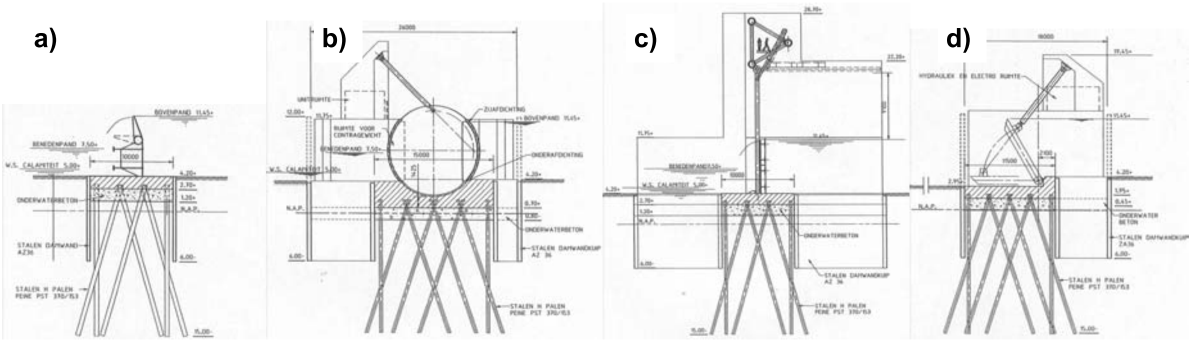


Fig. 4.3 Considered gate types for the Meuse weir in Sambeek

Option Costs (€)	Vertical lift gate (a)	Sector gate (b)	Suspended flap gate (c)	Bottom flap gate (d)
Construction	36,000,000	37,000,000	34,000,000	32,000,000
Maintenance: - per year - capitalized	 340,000 7,596,000	 447,000 9,987,000	 365,000 8,155,000	 421,000 9,406,000
Operation: - per year - capitalized	 246,000 5,496,000	 246,000 5,496,000	 246,000 5,496,000	 246,000 5,496,000
Totally (€)	**49,092,000**	**52,483,000**	**47,651,000**	**46,902,000**

Table 4.3. Analysis in terms of costs for the Meuse weir in Sambeek

In this case, a period of n = 50 years was assumed for capitalizing maintenance and operation costs. The difference between the rate of interest and the inflation rate was assumed uniform in this period i = 0.04 (4%).

The capitalized maintenance and operation costs C_c were calculated from the estimated yearly costs C_y, using the following formula:

$$C_c = C_y \cdot \frac{1-(1+i)^{-n}}{1-(1+i)^{-1}} \quad (4.1)$$

E.g., for the vertical lift gate with a flap section, option (a), the total costs were as follows:

$$C_c = 36{,}000{,}000 + [340{,}000 + 246{,}000] \cdot \frac{1-(1+0.04)^{-50}}{1-(1+0.04)^{-1}}$$
$$= 49{,}092{,}000 \text{ euros} \quad (4.2)$$

Despite the clearly defined, recognizable measure unit (money), this approach has a number of disadvantages. The most important ones are:

- Not all criteria can be quantified in currency units. The performances in such criteria as navigation, environment, aesthetics, local constrains (traffic, consumed land, local economy, social effects etc.) can hardly or not at all be measured in this way.
- Strict financial assessment in maintenance and operation says little about e.g. inspection conditions, (→affecting the sufficiency and reliability of the inspection), risks and obstructions due to maintenance, ease of operation, safety for operation personnel, etc.
- The owner always wants his costs accurately counted. The costs of other parties (e.g. obstacles for navigation, impact on infrastructure) are often underestimated.
- A strongly cost-oriented analysis creates a desire to give everything a price in currency units. This can be considered morally or otherwise controversial[2], e.g. with respect to human life, irreversible damage to the environment, etc.

[2] *This report issues no moral judgments. Giving no price to human life might also be considered controversial.*

Yet, this method can successfully be used in several cases, e.g.:

- A so-called “quick-scan assessment” of different gate types, which does not necessarily (anyway not immediately) lead to a project. The discussed example was in fact such a case.
- In gate type assessments, where other than financial criteria are not very relevant, e.g. weir or barrier in little inhabited area of small ecological significance.
- In project assessments, where the performances of gate types considered do not differ much from each other in other than financial criteria. One should, however, be careful in classifying a project as such.
- In countries, regions, times, situations, etc. where engineers are forced (often with good reasons) to consider costs as the main or the only selection criterion.

The presented example is rather simple. For example, no effort was made to estimate the differences in operation costs for the gate types under consideration. This certainly should be done in more sophisticated assessments. In such assessments, costs are often estimated as stochastical values (means with standard deviations) instead of deterministically. Diverse financial and other risks can also be taken into account. A discussion on these method extensions goes beyond the scope of this section.

4.4.2 PERFORMANCE RATING WITH WEIGHTING FACTORS - GENERAL

As mentioned in sub-section 4.4.1, another assessment strategy is to use performance ratings with weighting factors. Such a strategy does not make use of measure units from any single criterion, but it introduces its own measuring system which is applicable to all the criteria. Usually, a rating scale from 0 to 10 points is assumed to quantify gate performances in each single criterion, though other scales (e.g. from 0 to 5) are known as well. Higher marks usually represent better scores, although reverse systems (the higher, the worse) are also possible. In this report, a decimal scale with a progressive performance rating will be discussed.

In general, the rating of gate performance takes place in one of the two following ways:

- For quantifiable criteria: Measure the gate performances in quantity units of a criterion (e.g. in money for the costs criterion); choose a rating range covering the performance range; and convert the measured values to the rating system.
- For criteria that are not-quantifiable: Allow a representative group of specialists rate the gate performances subjectively; ask them to come up with a consensus or mean scores.

Having done the rating, one can not simply add the rates in different criteria to each other, because the importance of these are not the same. In order to produce the total scores, the relative importance of each of the criteria must be assessed. This is done using weighting factors. A weighting factor represents the importance of a particular criterion in the analysis, in relation to the total of all the criteria. The most convenient way is to assign the weighting factors of the range 0.00 to 1.00 for all criteria, in such a way that the sum of these factors equals 1.00. The total scores will then emerge in the same rating scale as the scores for each criterion, which gives the method more recognition and helps to avoid confusion. Nevertheless, other scales of weighting factors (e.g. in percents) can also be used.

It is advised to let the weighting factors be chosen by a team representing the project initiator (local authorities and other parties involved), acting independently from the team of professionals, which actually rates the gate performances. This decreases the risk that the team members will “drag” the analysis towards their favorite gate types. A good approach is to ask a multi-disciplinary team (deciders and, if possible, other involved parties) to set up the criteria and their weighting factors; and a team of specialists to generate solution ideas and to do the rating. The communication between both teams is a sensitive mater. On the one side it must produce well-understood and workable criteria; and on the other it should not be used for lobbying or other manipulating of each other’s domains. Such an approach is especially advised for large water management projects.

4.4.3 PERFORMANCE RATING - EXAMPLES

Below are two cases of performance marking assessments. The first case represents gate type selection for a double barrier lock between two large lakes, the IJsselmeer and the Markermeer, (Fig. 4.4) which emerged after closing of the Dutch internal sea, the Zuiderzee. The barrier lock was constructed in 2003 as a so-called “naviduct”, a lock on an aqueduct (Daniel et al., 2003). Combining the barrier and the lock functions in one project is a very common practice in the Netherlands and Belgium. The idea is that the gates operate as lock gates under normal conditions. When a storm surge approaches, the navigation stops, and the closed gates operate as a barrier protecting land and inland waterways against flooding.

In a primary selection, four gate types were considered suitable for this project:

1. Mitre gates;
2. Single leaf gates;
3. Rolling (or slide) gates;
4. Sector gates.

The considered criteria and their weighting factors are shown in a selection matrix in Table 4.4. “Total costs” represent here the construction and the (capitalized) maintenance cost. “Operation” has been assessed separately; considering more than financial aspects. “Local constraints” cover the aspects like aesthetics, required space, walkway possibility, disturbance to radar communication, etc. Criterion “Navigation” requires no comments. “Environment” is focused on environmental impact in a global sense (e.g. energy consumption, diverse emissions due to material winning, manufacturing, etc.) as well as in local sense (e.g. pollutions due to lubrication, painting, impact on wild life, etc.).

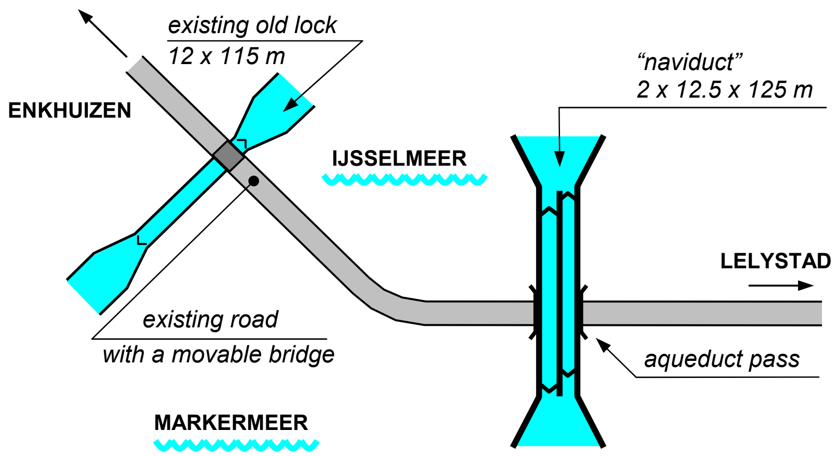


Fig. 4.4 : “Naviduct” in Enkhuizen, location map

Criterion / Option	Weighting factor	Gate type: Mitre gates	Single leaf gates	Rolling Gates	Sector gates
1. Total costs	0.40	8	9	6	6
2. Operation	0.35	9	8	8	7
3. Local constrains	0.10	8	7	8	7
4. Navigation	0.10	8	7	8	6
5. Environment	0.05	7	7	6	7
Total score	1.00	**8.30**	**8.15**	**7.10**	**6.50**

Table 4.4. Gate type assessment by performance rating for the Naviduct Enkhuizen (*)

(*) This example is presented in an MS Excel file, see on the CD’s Directory /Annex Section 4/GateSelectExample3.xls/

The second example represents the gate type selection for a storm surge barrier in the Hartel Canal, one of the two waterways to the harbor of Rotterdam. This project was completed in 1996 and the gate type selection took place 4 years earlier. For details of this see the “project review” included on the PIANC-WG26’s CD (Directory A1).

In the first (initial) phase, about 40 diverse gate types were proposed in a brainstorming session. Then, six of them were considered suitable for this project in a pre-selection, based on some feasibility studies. These gate types were (Fig. 4.5) (Daniel, 1996):

1. Vertical lift gate;
2. Visor (arch) gate;
3. Single rolling gate;
4. Double rolling gate;
5. Suspended flap gate;
6. Turn-over gate.

The criteria to be considered were in the first instance, also collected in a brainstorming session. Then they were clustered in the six following groups:

1. <u>Reliability:</u> Probability of closing and opening failure, stability under hydraulic loads from both directions, sensitivity to obstacles, sedimentation, operation comfort, etc.;
2. <u>Project control:</u> Risks in design process and project execution, time and money impact;

3. Navigation: Navigable width, depth, overhead space, hindrance during construction and due to maintenance, disturbances to radar communication, etc.;
4. Local constraints: Aesthetics, required space, impact to traffic, construction nuisance;
5. Total costs: Design and construction costs, capitalized operation and maintenance;
6. Realization time: Time required for research, design and construction.

The team of specialists involved in the selection process was much larger than in the case presented earlier (Naviduct). Some 20 people of different disciplines took part in it, while in the Naviduct project this number was 4 to 5. This is traceable in the final selection matrix, as presented in Table 4.5.

Observe that the weighting factors are not only quite different - they are also not round figures any more. The reason is that while these factors were agreed by consensus in the Naviduct project, the Hartel team could not reach a consensus. Therefore mean values had to be taken. Another difference is a much narrower range of the weighting factors and – in particular – a lower ranking of the costs criterion. Also this can be explained by different sizes and profiles of assessing teams. In multidisciplinary teams, the opinions tend to be more divergent when the teams are large. Fortunately, the selection matrix produced one clear winner: a vertical lift gate.

The considered criteria, their weighting factors, and the ranking of the final six gate types are shown in a selection matrix in Table 4.5.

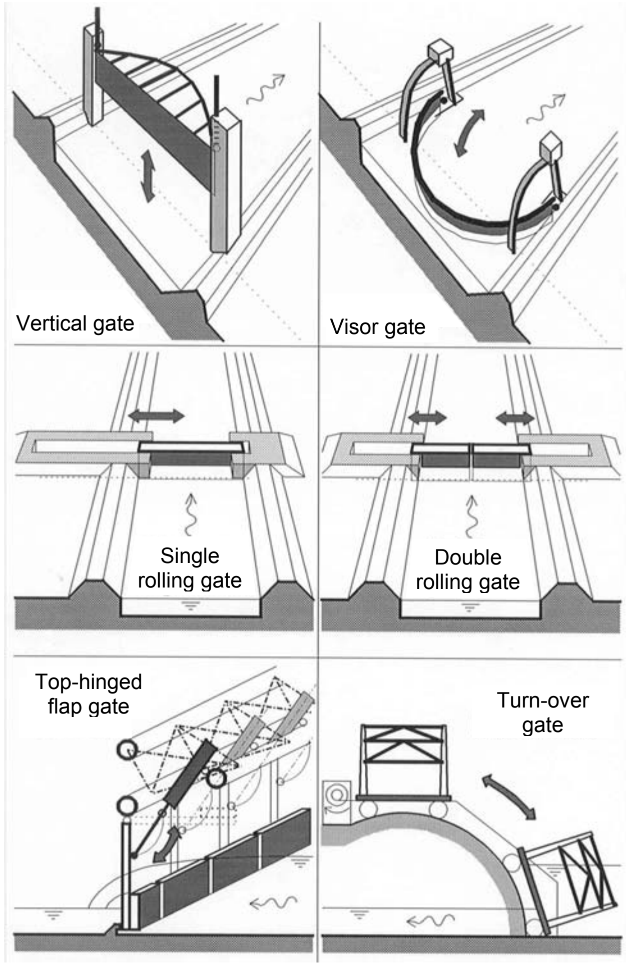


Fig. 4.5 : Gate options for the Hartel Canal Barrier

Option / Criterion	Weighting factor	Gate type: Vertical lift gate	Visor (arch) gate	Single rolling gate	Double rolling gate	Top-hinged flap	Turn-over gate
1. Reliability	0.27	9.0	8.5	8.0	7.0	6.0	8.0
2. Project control & operation	0.20	8.5	6.0	7.0	6.0	6.0	6.0
3. Navigation	0.19	8.0	7.0	8.0	8.0	8.0	7.0
4. Local constrains	0.12	7.0	8.0	7.0	7.0	6.5	7.0
5. Costs (total)	0.11	9.0	8.0	6.0	6.0	7.5	5.0
6. Realization time	0.11	8.0	6.0	6.0	6.0	6.0	6.0
Total score	1.00	**8.36**	**7.33**	**7.24**	**6.77**	**6.61**	**6.74**

Table 4.5 : Gate type assessment by performance rating for the Hartel Canal Barrier (*)

(*) This example is presented in an MS Excel file, see CD's Directory /Annex Section 4/GateSelectExample4.xls/

Presented gate type selections were accepted in both cases. The mitre gates of the Naviduct are in operation since 2003 and the vertical lift gates of the Hartel Barrier have operated since 1997.

4.4.4 PERFORMANCE RATING - SENSITIVITY ANALYSIS

As shown above, the performance rating method is rather vulnerable to arbitrary opinions. Except for the costs related criteria, it remains quite difficult to set up an objective, traceable marking system. The choice of weighting factors remains arbitrary as well. It is practically impossible to eliminate the arbitrariness, but it is possible to estimate its influence on the final results. A way to do it is the so-called "sensitivity analysis". We shall focus on the sensitivity to different assumptions of the criteria weighting factors, which is a crucial, final numeric decision to be made. However, this does not cover the whole subject. One can also analyze the sensitivity to the assessment approach as such, i.e. to the way in which proper groups of interest are involved in the decision making process. Interesting examples in this field are the Belgian and French experiences within the so-called "concertation" approach – a multi-criteria analysis for multi-actor decision making.

Focusing on weighting factors, below is an example of a sensitivity analysis using the data from the gate type assessment for Naviduct Enkhuizen, as presented in the previous section (Table 4.4).

Let us investigate the assessment sensitivity to the costs criterion. In other words: Let's say that we are not certain about the weighting factor of 0.40 for that criterion; and we want to see how much it maters when it assumes other values. In order to do that, we now take another value – say 0.10 – for this weighting factor, and divide the difference proportionally between the remaining criteria. The resulting weighting factors are then:

1. Costs (total): 0.100
2. Operation: 0.525
3. Local constraints: 0.150
4. Navigation: 0.150
5. Environment: 0.075

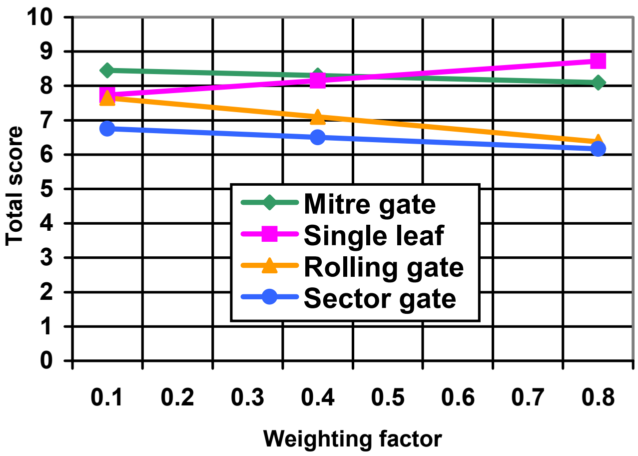


Fig. 4.6 : Naviduct Enkhuizen, sensitivity analysis for the costs criterion

Using these weighting factors in the Excel file *GateSelectExample3.xls* will give other total scores than those shown in Table 4.4. For each gate type, we now have two points with coordinates (f_w, s), where f_w is the weighting factor for the costs criterion, and s is the total gate score. These points define the linear functions of the total scores with respect to the costs criterion, which can be pictured in a diagram as shown in Fig. 4.6.

It can be observed that a mitre gate holds the highest score up to the weighting factor of about 0.46 for the costs criterion. If the weight of this criterion is still higher, a single leaf gate begins to score better. The other gates are not competitive in this analysis.

In a similar way, sensitivity analyses regarding all other criteria can be performed. Fig. 4.7 presents the sensitivity diagram with respect to the navigation criterion. The mitre gate now scores highest in the whole range of the weighting factors. Note that the rolling gate takes the second position when the navigation criterion weights heavier than about $f_w = 0.56$.

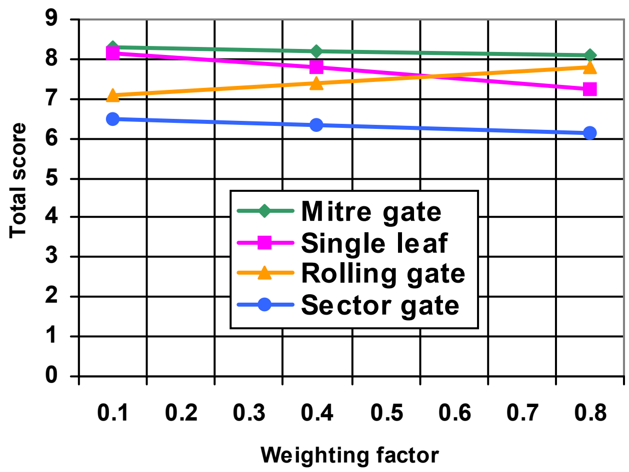


Fig. 4.7 : Naviduct Enkhuizen, sensitivity analysis for the navigation criterion

Such diagrams can be used to investigate the assessment sensitivity with respect to just one "uncertain" criterion at a time. It is, however, also possible to do it for two such criteria. The diagrams are then three-dimensional; and the lines representing gate types become planes. Such presentations are quite spectacular, but their essential value becomes questionable due to uncertain bases. They turn the attention away from the merits to the form – an approach called by the Dutch "selling baked air". It is, therefore, advisable not to focus on sensitivity analyses but to use them as the last tool in gate type assessments. Sensitivity analyses also help to determine the importance of improving the accuracy of a score or weight. Improving accuracy costs time and money. Its value should, therefore, always be weighted against these factors.

4.4.5 PERFORMANCE RATING – CRITERIA CLUSTERS

The examples provided in Section 4.4.3 indicate that different projects require different systems of criteria and their weighting factors. Therefore, it is not the intention of this report to establish a uniform system, for all weir and barrier projects, apart from locations, local conditions, preferences, etc. Nevertheless, it can be helpful to have an example of such a system when approaching the question of gate assessment. In this sense, as an example – not as advice, two systems of hypothetical gate criteria will be given, one for a weir and one for a barrier project (see Table 4.6).

In both cases, the criteria are clustered in a relatively small number of main criteria, which, in turn, cover a number of sub-criteria. The sub-criteria have been selected taking the following principal guidelines into account:

- There is no doubling of issues between the criteria. Every relevant issue is represented in only one (sub-) criterion.
- Each sub-criterion is more or less independent. There is no or little correlation between the criteria. In case some correlation cannot be avoided (e.g. service life and maintenance), a clear division between the domains of the sub-criteria can be drawn.
- The proposed criteria and weighting factors reflect the average views in the so-called "industrially developed" countries.

Table 4.6. Indication of gate assessment criteria for weir and barrier projects

Criteria	Weir projects		Barrier projects	
	W.f.	**Sub-criteria**	**W.f.**	**Sub-criteria**
Generalized costs	0.30	Initial costs (engineering, land purchase, construction etc.);	0.15	Initial costs (engineering, land purchase, construction etc.);
		Periodic costs (inspections and maintenance);		Periodic costs (inspections, testing and maintenance);
		Operation costs (personnel, energy, facilities, etc.);		Operation costs (personnel, energy, facilities, etc.);
		Costs of dismantling / modernization after service life;		Costs of dismantling / modernization after service life;
Reliability	0.15	Sensitivity to malfunctions, human errors, ship collisions;	0.25	Failure chance to close, when closed and loaded, to open;
		Vulnerability to foundation distortions, vibrations, bottom erosion, earthquake, etc.;		Vulnerability to foundation distortions, bottom erosion, earthquake, etc.;
		Vulnerability to sediments, ice, debris, algae etc.;		Sensitivity to malfunctions, human errors, ship collisions;
Operation	0.15	Capacity and accuracy of river control in all seasons, operation vulnerability to calamities;	0.15	Convenience and clarity of procedures, especially under extreme conditions;
		Convenience of operation, procedure clarity;		Unavailability for operation due to maintenance;
		Unavailability for operation due to maintenance;		Construction time, especially in reconstruction projects;
		Construction time, especially in reconstruction projects;		Sensitivity to technological aging, patented technology etc.
Navigation	0.10	Construction impact on navigation conditions;	0.15	Free navigation width, overhead space and depth;
		Maintenance impact on navigation conditions		Clarity of navigation regulations during closing and opening;
		Navigation safety and convenience (distances, currents etc.)		Construction impact on navigation conditions;
		Disturbances to maneuvering, radar signals etc.;		Maintenance impact on navigation conditions;
Maintenance	0.05	Maintainability (not in terms of costs!) of all areas and details	0.05	Compliance with ban on maintenance in stormy seasons;

		Access to maintenance sensible components		Maintainability (not in terms of costs!) of all areas and details
		Maintainability under operation conditions		Access to maintenance sensible components
		Health and safety of maintenance crews		health and safety of maintenance crews
Environment	0.15	Operation impact on eco-system (vegetation, wide life etc.);	0.10	Required area, construction impact on eco-systems;
		Environmental "footprint" of materials (pollutions, energy consumption);		Environmental "footprint" of materials (pollutions, energy consumption);
		Environmental impact of gate construction and maintenance (e.g. painting, lubrication);		Residual environmental impact of storm surge passage;
		Possibility of winning "clean" (water) energy;		Environmental impact of gate maintenance (e.g. painting, lubrication);
Social impacts	0.10	Aesthetics, harmony with landscape, local culture etc.;	0.15	Aesthetics, harmony with landscape, local culture etc.;
		Daily impact on local community (jobs, economy, transport, agriculture, social contacts);		Daily impact on local community (economy, transport, agriculture, social contacts);
		Noise (water flow, machineries, maintenance vessels, etc.)		General image, feeling of safety for the local community;
		Tourism, sport and recreation benefit, science and technology popularization effect;		Tourism, sport and recreation benefit, science and technology popularization effect;

4.5 OTHER GATE ASSESSMENT METHODS

Other approaches to the problem of gate type selection are also possible, but they do not have such universal character as the methods discussed earlier. In these approaches, the starting point is usually one specific criterion – other than cost, which is assumed to play a decisive role. Such a criterion then becomes deliberately privileged.

This criterion can be local constrains. Local constrains may dominate the assessment in all situations where a barrier or weir is to be constructed e.g. in a highly urban surrounding, with complex aesthetic, traffic or other requirements. Traces of such an assessment can probably be found in the selection of sector gates for the Thames Barrier in London, or a turnover vertical lift gate in the Hull Barrier. Outside urban locations, other local constrains may play a major role. These can be, e.g.: exceptional landscape, military reasons, prestige aspects, intensive radar communication, risk of water pollution, etc. In such cases, assessment methods will partly or entirely be determined by the dominating local constraint criteria.

Another dominating criterion can be global environment. While such situations are rather exceptional nowadays, they will probably be more frequent in the future. An important aspect of ecological analysis is the selection of construction materials. There are examples of successful, well-quantified ecological analyses in this field for other structures than barriers and weirs. One of them is the construction of a new faculty building on the Carnegie-Mellon University campus in Pittsburgh, USA (Mahadvi, 1998). Another is the construction of a pedestrian bridge in the Noordland inner harbor in the Netherlands (Daniel, 2003). The scope of this report does not allow for a detailed discussion on the assessment methods used in these projects. Those methods can, however, be applied to barrier and weir gates as well.

The environment can also be a dominating criterion when considered in local rather than a global sense. In this matter, a tool suitable for providing solutions to a complex scenario of feasibility indicators is the Environmental Impact Assessment (E.I.A.). A global methodology outline of E.I.A. has been presented in (Perillo, 1997), along with a number of usable formulas for assessment of different gate alternatives.

The assessment methods of this report can, in principle, be applied to new construction and to renovation projects alike. However, the latter category usually comprises projects of more specific boundary conditions and requirements than the first one. Therefore, the assessments are also deeper rooted in local conditions. An interesting approach in this field is the so-called "unity value analysis", which is similar to performance rating with weighting factors (see section 4.4.2) – but especially focused on renovation projects. The method and its application to a rehabilitation strategy selection for an old weir in Germany is presented by Jansen et al (1996). In this case an old system of two fixed-wheel main gates and two smaller fine control gates were replaced by three flap gates of equal spans.

4.6 CONCLUDING REMARKS

Gate type selection is an important stage in a barrier or weir project. The operational, financial, and other consequences of this selection are often more important than the detailed engineering. It is, therefore, advisable to give thorough consideration to the gate type selection. This report gives some background information and a review of existing assessment methods in this field. The general conclusions of this section are as follows:

- There are always a number of criteria to be considered in gate type selection. These criteria are, however, different for every individual project. Therefore, it is not advisable to standardize them, nor to establish strict procedures to be followed in this matter. Nevertheless, every effort should be made to get a clear, well-balanced inventory of all criteria significant to a particular project.
- Analogously, there are always a number of gate types suitable for a project. These types are also different for every project; and should therefore not be part of any standard selection procedure. A list of gate types to be considered can, e.g., be obtained in a brainstorming session, possibly helped by check lists from different publications.
- A multi-criteria gate type selection can be performed in one or more phases. The first takes place when there is a general understanding about significant criteria and suitable gate types – and when the numbers of both are not large. This happens often in small projects. In large projects, the chance of it is usually small. A better strategy then is to make a selection in two or more phases, focusing still deeper on the crucial criteria.
- The assessment methods for gate type selection can be qualitative or quantitative. The choice of a method depends on the size and complexity of the project, required transparency, etc. – but also on individual skills and preferences of the selecting team.
- Qualitative assessments are procedurally simple and fast – but, on the other hand, quite arbitrary and not very transparent. Such assessments are discussed in section 4.3. They should, in general, be used in projects of small complexity, or as the first phase (pre-selection) in large, complex projects.
- Quantitative assessments require more effort and time. In these methods, gate performances are measured in the units of the dominating criterion (e.g. in money for the costs criterion) or in another numerical score system. Such assessment methods are less arbitrary and more transparent than the quantitative assessments.
- The assessments based on costs analyses are probably the best quantifiable. These assessments are discussed in Section 4.4.1 of this report. An important disadvantage of such assessments is, however, that not all selection criteria can be quantified in money. The criteria which cannot, are e.g.: hindrance for navigation, environment, aesthetics, diverse local constraints, etc.
- A more universal assessment method is the performance rating with weighting factors for different criteria. This method is discussed in Sections 4.4.2 through 4.4.4. The quantifying possibilities of other methods, and the non-quantifiable subjective assessments, are in this method converted into performance rates. The assessment criteria are given weighting factors expressing their significance in the analysis.
- The performance rating method is not free of arbitrariness, but it is more transparent than the qualitative methods; and better balanced than the methods based on costs analyses. As the rating and the choice of weighting factors are compliant to individual opinions, it is crucial to give these tasks to a representative team of specialists. When selecting such a team, one should keep in mind the remarks in Section 4.4.3.
- It is advised to let the criteria and their weighting factors be determined by a team representing the project initiator (local authorities and other parties involved) – and the actual rating by a multidisciplinary team of professionals. Both teams should act independently, but they must preserve some communication in order to have the same image of the selected criteria, practical meaning of determining weighting factors, etc.
- It is advised to keep the number of gate types under investigation small, e.g. not larger than 4 to 6. If more types are submitted, it is advisable to make a pre-selection. It is also advisable to keep the number of assessment criteria not larger than 6 to 8. If more criteria are submitted, a clustering should be considered. It is difficult to produce a well-balanced assessment when the number of gates and/or criteria is higher.
- As the choice of weighting factors is arbitrary, it is often disputable. A tool to help solve such discussions is a sensitivity analysis. This analysis is discussed in section 4.4.4. It is important to keep in mind that the diagrams of sensitivity only help viewing the results, they do not introduce any new information. They should never replace the essential arguments.
- Although no uniform system of criteria and their weighting factors can be given, it is possible – if desired – to set up such a system using existing examples in this field. In Section 4.4.5, some general principles for this task have been given followed by an indicative criteria system for a hypothetical weir and barrier project.

5. DESIGN CONSIDERATIONS (Parameters and Criteria)

5.1 STRUCTURAL CONSIDERATIONS

The aim of this section is to give an overview about the gate structural aspects and to survey the advantages and disadvantages of the structural aspects of the various gate-types for their intended purposes. These advantages-disadvantages will vary according to how closely the gate type matches its expected uses.

The assessment of the advantages and disadvantages of various gate-types can only be considered and performed for a given context and situation. It is necessary to provide a complete investigation of the local site characteristics, the user requirements and the design objectives (weir functions) before an effective assessment of gate types can be made.

If the situation-context changes, then the advantages and disadvantages of a given gate also change. Therefore, the limits of use and the optimum ranges of application of a gate-type can vary with the operational requirements (barrier or river flow control weir, rural or industrial area, etc.).

This chapter will first present the Main Steps of a Structural Design (Section 5.1.1), and then present three additional areas of consideration for gate selection:

- Structural Characteristics of various gate-types (Section 5.1.2).
- Analysis of specific constraints and functions (Section 5.1.3).
- The Typical Structural concerns (problems, malfunctions) that may occur in movable weirs (Section 5.1.4),

The last section (5.1.5) of this chapter compares the advantages and disadvantages of the design, construction, maintenance and operational characteristics for each of 5 major gate types. The typical range of operation and use are provided to assist the designer in selection of the most appropriate gate type for a specific application.

5.1.1 MAIN STEPS OF A GATE STRUCTURAL DESIGN

Steel structures (gates) of a weir have to be designed more carefully than fixed ground-based structures for several reasons:

- They are movable.
- Loads are difficult to calculate (particularly hydrodynamic effects, varying loads, fluid structure interactions).
- Shapes can be complex (3D stiffened shells) which make stresses difficult to calculate.
- These structures are mainly under-water and often difficult to inspect and to maintain.
- They are subject to deterioration from various causes: vibration, corrosion, wear, flow, …
- Structures are typically kept in use significantly longer than their design life.

This requires robust solutions and high safety factors.

In the past, relatively complex structures were often fabricated or simplified for analysis as frame structures modeled with beam-column elements. Improved analysis tools now allow more optimized fabrication and accurate analysis of the welded, stiffened plates and shapes used to construct the complex 3D structures to be modeled using 3D shell and brick finite elements.

Hereafter the main steps of design are described, using a semi-probabilistic verification method:

→ *Global and geometric design*

The geometric characteristics of the gate have to be optimized using hydraulic and structural considerations in order to:

- Transmit the loads to the civil work,
- Improve hydraulic efficiency,
- Avoid vibrations,
- Control deflections,
- Resist torsion and bending forces,
- Minimize weight (for movable gates),
- Simplify fabrication,
- Provide corrosion protection,
- Simplify maintenance (access to different parts),
- Guarantee long service life.

For example, the radius of a skin plate is a geometric parameter to optimise for hydraulic flow and load transmission.

→ *Determination of characteristic actions (Design loads)*

- Hydraulic (static and dynamic),
- Operating (reaction to the hydraulic loads),
- Accidental (induced for instance by hoisting devices that are not synchronized),
- Deadweight,
- Friction,
- Ice and debris,
- Other actions: earthquakes, waves, wind, blast, etc.

These actions are calculated by numerical and physical models, but can also be estimated by simple first principle assessment methods.

For each one, the designer has to determine the calculation values according to the various situations and combinations.
The choice of values has to take into account the difficulty to assess them because of mobility and hydrodynamic effects.

→ *Structural analysis*

In order to calculate the strengths in the structure, it is necessary to analyze:

- Stresses in the fixed and operable structural elements of the gate,
- Forces transmitted to the foundation or supporting structures,

- Reaction forces on hinges, trunnions, rails,
- Deformations, etc.

In addition the designer has to take into account:

- Immersion of some components (ropes, chains, articulated parts, actuators, etc.),
- The wear of guiding devices leading to additional loads or changes in distribution,
- Corrosion in all the service life.

→ *Load cases*

Different load cases have to be determined for:

- Permanent situations (typical case: normal water levels),
- Transient situations (typical cases: maintenance…),
- Accidental situations (typical case: floating debris chock, malfunction of hoisting device…).

They have to be realistic according to hydraulic and operating conditions or probability of occurrence.

→ *Verifications*

The designer has to form combinations (with partial factors applied to the actions) in order to make the verifications for all the load cases and for various limit states (serviceability, ultimate limit…).
Then, in each case, he will use the appropriate factors of safety to be applied to each element of the structure.

These verifications apply to:

- Global torsion and bending moment that induce shear stress, longitudinal and transverse stresses in the stiffened plates,
- Strength of local components as plates, beams and stiffeners (loads transmitted by the skin plate),
- Local deflection of skin plate and members,
- Fatigue and vibration (if relevant).

→ *Design of operating equipment*

Attention must be paid to design seals and hoisting devices. A high safety factor has to be applied, at least 5 or 6.

→ *Catastrophic events*

For catastrophic events, failure mechanisms should be designed to provide an orderly reduction of forces and to minimize the costs of repair.

5.1.2 STRUCTURAL CHARACTERISTICS OF VARIOUS GATE-TYPES

Tables 5.1 and 5.2 compare five commonly used gate-types (flap gate, radial gate, vertical lift gate, sector gate and inflatable gate) with regard to their structural and mechanical proprieties. Other types can also be considered; Drum Gates, Visor Gate, Stop Logs…

From a structural point of view the various types of gates mainly differ by the means to transmit the load to the civil work and the means to move (translation, rotation...)
.

Table 5.1 : Structural and mechanical characteristics of gate-types (Part 1)

	Description (See Tables 5.1-5.3)	**Other types or variants to considerer**	**Foundation & Operator Supports** (transmission of loads)
Flap Gate	Skin plate generally curved, stiffened, and hinged on the floor.	- Torque tube - Wicket gate - Obermeyer gate (see inflatable weir Table 5.2)	Hinged on the floor by several points on the lower and downstream side of the gate.
Radial Gate	Skin plate (usually curved) - linked to 2 arms, - hinged on the piers.	- Reverse radial gate with upstream arms and trunnions. - Flap gat on the top of radial gate	Hinged (trunnions) in 2 points at the ends of the arms on the piers.
Vertical Lift Gate	Leaf with a rectangular skin plate, stiffened with stiff vertical and horizontal members. Often with a wheel carriage on each side but sliding gates are also considered (i.e. for emergency closure).	- Double leaf gate, superposed. - With a flap at the upper part.	On the lateral end-sides of the gate (in slots) with cables or hydraulic cylinders
Sector Gate	Plate formed of an upstream circular curved plate and a downstream flat plate, articulated at the bottom of the upstream side of the gate. See	Drum Gate is not very different but with an axis on the upstream side	Hinged at the bottom of the downstream plate
Inflatabl e	Sealed tube made of flexible material (usually reinforced membrane). Inflatable by air or water.	Inflatable air-bags that support metallic flaps (Obermeyer system). This alternative may also be considered as a flap gate.	Anchored to the sill by stiff bolts (1 or 2 lines), These require careful design and maintenance to insure reliability in an inflatable weir!

Table 5.2 : Structural and mechanical characteristics of gate-types (Part 2)

	Movement and Handling	Flow type	Applied Loads: - Hydraulic load - Weight and friction - Handling load	Internal forces
Flap Gate	Rotation around a bottom hinge, Handled by one or two hoisting devices	Overflow (typical)	Hydraulic load - To the floor (2/3) and to piers (1/3). Weight and friction - Small forces (to the floor and to the piers) Handling load → opposite to the hydraulic load - To the pier, - Handling loads are large	- Longitudinal bending moment combined with torsion. - Large torsion if driven from 1 side.
Radial Gate	Rotation around the trunnion axis. Hoisting systems applied on the arms on one or two sides.	Underflow (typically). Or both over and under flow (if with an upper flap)	Hydraulic load 50% on each pier Weight and friction To the piers Handling load → opposite to the weight - To the pier, - Small handling load,	- Bending moments in vertical and longitudinal directions - Limited torsion unless operated from one side (not both).
Vertical Lift Gate	Vertical up/down translation to open/close the gate. Hoisting system on both side of the gate.	Underflow (typically). Or both over and under flow (if designed in 2 elements).	Hydraulic load - To the pier (slot) on the both side of the gate. Application points change with the vertical gate position. Weight and friction - To the piers Handling load→ opposite to the weight - To the pier above the gate,	- Longitudinal bending moment. - Deadweight is significant.
Sector Gate	Rotation around the hinge. The internal water pressure acts as driving force.	Overflow (only)	Hydraulic load - To the floor Weight and friction - To the floor and to piers Handling load - No real handling load (pressure from weir water head is used)	Lateral pressure and Transverse bending moment on upstream plate.
Inflatabl e	Inflated (air or water) Pressure from the compressed air or water head	Overflow (only)	Hydraulic load - To the floor through the anchorage bolts Weight and friction - Very small Handling load - No real handling load (internal pressure in the membrane/bag)	High tensile stress in the flexible membrane.

5.1.3 GLOBAL ANALYSIS OF CONSTRAINTS AND FUNCTIONS

This analysis includes, but is not limited to the site location, maximum flood discharge, foundation type, and navigational requirements for which the gate is designed and will be used.

Before determining if a gate-type is appropriate for a specific location and operational requirements, it is necessary to have precisely determined or quantified these requirements and constraints. The requirements can be defined using a few key parameters or characteristics that relate to geometric data, operations and functions, and environmental considerations.

These parameters are:

- Geometric: width, water height (upstream, downstream, flood), crest height, bed position, air clearance,
- Operation: functions and performances such as accuracy of control, operating range, frequency of handling, frequency of maintenance (with an objective and a tolerance), dewatering requirements, …
- Environmental conditions: debris flow, sediment transport, geotechnical conditions for foundation support and seepage control during construction and operations.

→ **Geometric parameters** (Ract et al., 1970) :

- Global dimensions (height, width) of the weir, which have an influence on global hydraulic loads
- Relative elevation of the weir sill compared to the downstream riverbed to determine the available depth for the downstream floor without additional excavation.
  Hence, for flap and sector gates, a minimum depth is necessary to totally recess the gate under the crest level (when the gate is down).
  If the available depth is too small, it can lead to an expensive solution. But if the crest is high above the riverbed, it may be easier to design a flap gate or a sector gate.
- Height and width of the spans versus required torsional and bending gate rigidities.
  The structure must respect deformation limits, and the capacity of the different gate-types to resist the applied torsion and bending moments.
- Water level elevations: upstream head, tail water (downstream water level) and flood height.
  These values determine the highest position of the gate and the trunnions (for a radial gate) the trunnions should not be immersed or at least not frequently (for instance once a year).

→ **Operational parameters**

- The operational requirements, or the purpose, of the weir (navigation, flood control...); for example:
  - When navigation through the weir is possible, a higher air clearance is necessary
  - Capacity of the gate to accept an inversion of hydraulic loads.
- Expected performance: accuracy of control (tolerance on upstream pool height), manoeuvring speed, hydraulic head, discharge rate and volume,... these depend on the weir requirements. For instance, navigation needs a good accuracy on the guaranteed depth and on the controlled flow speed.
- Range of use relates to opening possibilities. Depending on the expected weir functions, there are various situations, for example:
  - The gate is used for all opening possibilities (0 to 100%)
  - The gate is either closed or opened (0 or 100%)
  - The gate is used for small discharge regulation and then totally opened (for instance, opened from 0 to 30% and then 100%). In this case a double leaf gate may be more suitable.
- Maintenance: The availability of staff and resources for effective and economic operations has to be compared to the durability of materials. If maintenance resources are limited, the operational reliability can be compromised with less durable materials. Operational constraints also include the capacity to stop the operation anytime, hoisting capacity for dewatering system, etc.
- Reliability: expected level of reliability
  Capacity to guaranty a safe opening in case of flood.
  Capacity to guaranty a safe closure if it's more important the safe opening
  Probability of failures accepted
- Other conditions: environment, aesthetic, …

→ **Environnemental conditions**

- Debris flow that needs to be evacuated: type (trees…) and quantity,
- Sediment transport (gravel and sand) that creates abrasion on structures: type and quantity,
- Climate: icing,
- Geotechnical conditions for foundation support and seepage control during construction and operations: structural capacity to accept high loads in the piers or the floor.

5.1.4 TYPICAL STRUCTURAL CONCERNS

In this section various structural concerns (problems, advantages or disadvantages…) that may be encountered from gate design to its operation are discussed. For each concern, some general statements are given.

Structural concerns may be classified in three groups:

- Design and construction concerns.
- Operation and maintenance concerns.
- Deterioration of the structure during its life.

During the design stage is the best time to look for economic and reliable solutions or to solve technical problems. Latter, it will be more difficult, sometimes impossible, but always more expensive.

5.1.4.1 Design and construction concerns

The main structural concerns related to design are structure complexity, weight and impact on civil work:

- Complexity of structure:
  A more complex structure is difficult to design and later to build. A complex structure has inherently higher risks of errors and weaknesses that may in turn have a negative impact on maintenance and durability. This can also lead to a higher cost.
- Weight, thickness and superstructures:
  The problems occur when theses parameters are out of proportion to their operational requirements or handling capabilities. They have a negative impact on the cost and handling loads. The consequence is that some concepts (gate-type) are not feasible and/or too expensive for large dimensions. For example, vertical lift gates can (in some cases) become prohibitively expensive when they are very large.
- Impact of gate design on civil works:
  Often the technical problems do not concern the steel structure itself (gate) but the civil engineering works (concrete and foundations), which may become more important, complex and expensive.
  The larger impact comes from the load transfer (i.e. gate to piers) and the shape of the contact lines or surfaces where seals are often located.

If the loads transmitted to piers are too concentrated, heavy and expensive concrete reinforcement is required and design limits may be reached (i.e. load on the trunnions of radial gates)
If the loads are mobile (for example a lift-carriage on a rail) special devices and reinforcement have to be considered, specifically related to wear and blockage.

5.1.4.2 Operations and maintenance concerns

The choice of the structure has a specific impact on operations and maintenance. That's why some specific concerns have to be considered in structural design.

The operation concerns are the ability to control water height, and to handle debris and sediments.

Some of the key points for maintenance of a movable gate are to define, inspect and maintain the parts that are always (or often) under water, and to assess the consequences of submergence on design and maintenance. The following must be considered:

- Robustness and durability of all pieces, probability of mechanical failure,
- Methods to maintain and replace all the mechanical pieces, both those submerged and those above water,
- Redundancy to maintain acceptable operability in the event that key elements, that are difficult to replace, fail (actuators, hinges..)
- Probability of accident (chocks…) and capacity of the structure to accept major accidents without massive destruction or collapse (for examples: a flap gate with several hinges, guaranties a redundancy).

5.1.4.3 Degradations of the structure during its life

Physical phenomena causing degradations

Usually, structural degradation does not depend specifically on the gate type. Structural degradation has to be considered as normal, but the extent of the degradation has to be limited to the expected life cycle of the structure.

Typical sources of degradation are:

Wear:

Wear due to friction is important in the articulated parts of the gate, such as hinges and wheels. For each piece, the wear tolerance must be known. Excessive wear can create deformation, vibrations and lead to a different load distribution that may induce failure. For instance trunnion friction moment omitted in the original design of the Folsom tainter gates induced a major gate failure (Todd, 1999)

Fatigue

Fatigue is generally not a major problem for weir gates because there are generally few operating cycles throughout the life of the gates relative to fatigue capacity of the gate elements. Nevertheless, some flow configurations (at gate edges) may induce local and global vibrations (See Section 5.2). As fatigue is usually not a main concern, designers consider yielding, buckling and excessive deflections as design limit states.

Abrasion

This kind of degradation is the result of contact with water current, mainly in the presence of sediment transport. This is more important for gates that have underwater hinges (as flap gates).

Corrosion

It can develop for all steel structures near water.

- Problem of accessibility (for inspection and repair) is an important issue.
- Use of stainless steel and aluminium or synthetic materials could be a solution against corrosion but contact between dissimilar metals should be taken with care or corrosion is greatly accelerated.
- Abrasion can lead to corrosion

An efficient protection can be obtained by various techniques: cathodic protection, sacrificial anode.
An efficient corrosion prevention policy requires a "planned maintenance" rather than a "repair on failure" approach.

Vibrations

Vibrations can be the result of either mechanical or fluid-flow causes. The lack of aeration in gate overflow is one of the major causes of vibration.
They can cause higher stresses, large alternate deflection, noise and wear, but fatigue cracks are usually not induced.

If amplitudes are small and happen during limited times (for special flow configurations), the vibration may only be a nuisance and will not cause structural failure or operational problems. Vibrations such as these must be avoided by appropriate gate management. For instance avoiding critical underwater opening conditions (small bottom opening has to be avoided).

If the amplitude of vibration is large enough to produce significant levels of stress, and the vibration persists for a long enough time, serious damage or complete failure may occur through fatigue of structural components.

Anyway, such vibrations must be avoided by appropriate gate management, for instance avoiding critical underwater opening (small bottom opening must be avoided), combined with an appropriate design of the gate edges.

Indeed, the way in which the gate is operated may have an effect on the occurrence and the nature of vibrations, i.e. the most serious vibrations of gates often occurs for underflow when gates are operated at small openings. In this case, eddies may generate alternative pulse loads on the gate that induce vibration.

The seals themselves are a major cause of vibrations if not designed and installed properly.

Design guidelines must be considered to avoid (or reduce) vibration problems (see Section 5.2).

Cavitation

It can produce damage to concrete structures due to:

- Inappropriate design of gate slots,
- High velocity jet flow caused by serious water leakage,
- Inappropriate design of gate's bottom edge.

Consequences of degradations

These physical phenomena (vibration, wear, …) lead to several problems:

- Reduction of plate thickness,
- Change in load distributions,
- Excessive deflections,
- Geometric distortions,
- Leakages,
- Higher stresses in the gate structure,
- Cracks.

These degradations can lead to a structural failure or weir malfunctions.

5.1.5 ADVANTAGES AND LIMITATIONS OF VARIOUS GATE TYPES

Table 5.3 reviews the advantages and disadvantages of various gate-types, discusses the characteristics explained above, and the various "problems". This table presents the attributes of these gate-types with regard to, its design, degradations, maintenance, and operation.

Attention must be paid to the fact that advantages and disadvantages strongly depend on local conditions and specific requirements. Therefore, the following table cannot be considered totally objective.

Table 5.3: Advantages and disadvantages of various gate types (Part 1)

	Flap Gate	**Radial Gate**	**Lifting Gate**	**Sector Gate**	**Inflatable Dam**
Design	Advantages - Economic - Simplicity of civil work and hoist equipment - Hydraulic load on floor → (better for stability and allows narrow piers) - Not visible	Advantages - Robustness and high stiffness (less rigid for low height and if there is an upper flap gate - Concentrated loads (radial shape) - Low hoisting capacity - No slot	Advantages - Simple shape - Ease of fabrication - Large span is possible - Possibility of double leaf gate - Avoid long piers - Erection time is short	Advantages - No length limit - No torsion and no longitudinal bending: simple structural concept - Not visible	Advantages - No length limit - Erection time is short - Not visible - Simple foundation
	Disadvantages - Low height only - Lack of torsion rigidity, especially if there is one side handling - Difficulty of alignment, particularly with two operators.	Disadvantages - Difficult to design if the downstream pool or the flood height is too high (needs high posi-tion of the trunnion) - Long/large downstream piers with heavy steel bar reinforcement - Hydraulic load on downstream pier (bad for stability) - Visible (gate, piers, jacks, …)	Disadvantages - Heavy and complex mechanical system (if wheel carriage is used) - High friction forces - Needs a tower for hoisting - Large slot in the piers for rails - Moving loads in the slot pads - Under water mecha-nisms (wheel, rails) - High superstructures and complex mechanic equipment for hoisting system	Disadvantages - Needs a high civil work under the crest - Usually expensive, - Heavy steel structure - Friction generated by compacted sediments in the underwater slots	Disadvantages - Low height only (limited head) –Head depends of the material strength - Lack of experiences for navigation in some countries. - Service life shorter for the rubber
Degra-dations	- Sensitive to vibrations - Hinges partially or always under water: corrosion risks - Sensitive to abrasion when there is a strong sediment transport	- Sensitive to vibrations in case of sealing defects	- Sensitive to vibrations for small bottom opening or sealing defects -Sensitive to sediments	- Hinges partially or always under water: corrosion risks - Sensitive to abrasion	- No "metallic" degradations - Specific risks for vandalism and debris abrasion (punctures or guns)

Table 5.3: Advantages and disadvantages of various gate types (Part 2)

	Flap Gate	Radial Gate	Lifting Gate	Sector Gate	Inflatable Dam
Mainte-nance	Advantages	Advantages - Easy to inspect and to maintain in upper position (nothing under water)	Advantages - Easy to inspection and operate.	Advantages *See Project Review : Sector Weir Lehmen*	Advantages Low cost *See Project Review : Inflatable Weir Lechbruck*
	Disadvantages - Requires bulkheads (cofferdams) for inspection and maintenance	Disadvantages	Disadvantages - Complexity of systems → higher maintenance	Disadvantages - Movable parts under water - Maintenance is difficult under water axes	Disadvantages ---
Opera-tion	Advantages - Easy removal of debris - Adapted for navigation - Reliable for opening in case of operation failure - Smaller risk of massive destruction in case of accident (ship impact) - Reliable operation even if an actuator fails (for a gate with 2 actuators)	Advantages - Possibility to add an upper flap gate - Adapted to a unsymmetrical operation (from one side only)	Advantages - Possibility to add an upper flap gate - Simple and reliable (most reliable closure system for emergencies)	Advantages - Low cost	Advantages o No risk of blockage o Very reliable for safe opening
	Disadvantages - Sedimentation in the gate recess - Danger of damage if not enough clearance for shipping	Disadvantages - Free air clearance limited (danger of damage) - Energy dissipation for underflow (dissipation basin often required) - Arms and trunnion encroach on water pass: stop debris and not always protected against impact with debris flow	Disadvantages - Energy dissipation for underflow (dissipation basin often required) - Need large air clearance for shipping through the weir	Disadvantages - Danger of damage if not enough clearance for shipping -	Disadvantages - Less accurate for medium discharges - Most of the mechanical parts are under water - For inspecting fabric the gate has to be isolated - Danger of damage if not enough clearance for shipping

5.1.6 DESIGN RANGE OF THE DIFFERENT GATES

Table 5.4 indicates the optimal ranges of various gate-types associated with their limitations and operation requirements.

There is no argument to definitively discard a gate-type as they all encounter advantages and shortcomings and none of them provide only advantages. Selection can only be achieved for a given context.

However, the optimal sizes differ from gate type to gate type because of the differences of physical capacities and weight that may lead to a significant difference in cost.

In Table 5.4, optimal ranges are indicated for the dimensions (span and water height): it means that in this range, common use of the considered gate has already been made and can be recommended, but it does not mean that it is technically impossible to go further or that it has not been made before in an exceptional example. However, a maximum value is sometimes indicated.

In fact, the considerations about the limiting dimensions are more complex than presented in the next table as for each potential gate width there is a maximum water height, and such limits can only be represented by graphs.

Generally, the maximum water height decreases when the span increases (so that the global hydraulic load do not increase too much). However, it may be different for some gate types:

- Maximum water height decreases with the span:
  → Flap, radial, lift gates
- Maximum water height is not dependent on the span:
  → Inflatable, wicket, sector gates
- Maximum water height slightly increases with the span: → Bear trap, drum gates

The limiting parameters or situations indicate the type of gate that is not recommended (but not always impossible to use) and that specific solutions have to be found (probably with a higher cost).

Table 5.4: Optimal ranges of each gate type, with their limiting factors.

	Flap Gate	Radial Gate	Lifting Gate	Sector Gate	Inflatable Dam
Optimal dimensions: Water height and Span	H = 2 to 5 m Span = 4-15 m (1 side supported), = 15–35m (2 sides supported)	H = 4 to10 m Span = 15-30m	H = 2 to15 m Span = 2- 30 m	H = 4 to 8m Span = 15-40m	H = 1 to 4 m Span = 20-40 m
Limiting technical parameters	- Torsional rigidity water height (7 m) - No length limit for wicket gate	Global load on trunnions (40 MN)	- Loads on the bearings or wheels. - Extreme deformations	Water height	Water height (7m)
Limiting situations (configuration)	- Sediment transport, - When it's difficult to realize a thick weir floor under the crest - When there is no possibility of dewatering during the gate life	- A high tail-water would prevent keeping the trunnion above the water level - High air clearance needed - High debris transport (or with an upper flap)	High air clearance needed High debris transport (or with 2 leaves)	-Sediment transport. -When it's difficult to realize a thick weir floor under the crest	- Specific danger of punctures. - If long life is needed for the structure

Additional information is available on the WG26-CD in the Directory /Annex Section 5.1 – Structure/ taken from Erbisti 2004 and Sehgal 2000 (ICOLD'2000 in Beijing).

5.2 HYDRAULIC AND FLOW

This section evaluates various gate configurations from a hydraulic perspective. The discharge characteristics are quantified in terms of discharge coefficients (where available), that is, the head/discharge relation. Vibration tendencies that may be associated with the gate geometrical configuration or seal locations are identified. Gate performance in regards to their ability to control flow/pool by throttling flow is compared. Some gate types lend themselves to simply a fully open or fully closed operation. Another issue that can be important is the speed of gate operation. What type of gates can be opened or closed rapidly relative to other choices. Venting of the lower nappe of the jet is required for certain types of gates to avoid harmful vibrations. A gate's efficiency at passing floating material such as ice and debris can be an important project consideration. Wider gates are more efficient at passing floating material and are better at avoiding jams of floating material between piers. Effects of high tailwater, potential for unusual hydrodynamic loads, and potential for problems associated with sediment accumulation are also addressed.

A list of hydraulic performance evaluation metrics is provided. Each of the gate types is described in terms of these metrics (where metrics have been identified in the literature). Any appurtenances that should be avoided (e.g. a seal location) or included (e.g. air vent for nappe aeration) are also mentioned.

5.2.1 DISCHARGE EFFICIENCY

Discharge at navigation projects is controlled using gates and other devices. A generalization of the head-discharge relation is through the use of the discharge equation, which can be expressed in various ways, but is given here as:

$$Q = C_D G_o L_E \sqrt{2gH} \qquad (5.1)$$

where Q is the volumetric flow rate (discharge), in L^3/T; C_D is a dimensionless discharge coefficient; G_o is the gate opening, in L; L_E is the effective weir length (see next section for description of effective length), in L; g is the acceleration due to gravity, in L/T^2; and H is the head on the centre of the gate, in L. Care should be exercised in using C_D from the literature because it often includes $g^{1/2}$ and therefore, has units. In this case, the system of units of the discharge coefficient must be considered.

This section is by no means an exhaustive treatise on quantifying discharge over/under all gate types and geometries, but rather, it is meant to serve as an indication of relative efficiency for various flow control mechanisms. The more efficient a structure is at passing flow for a given head (i.e. pool conditions) is reflected in the discharge coefficient.

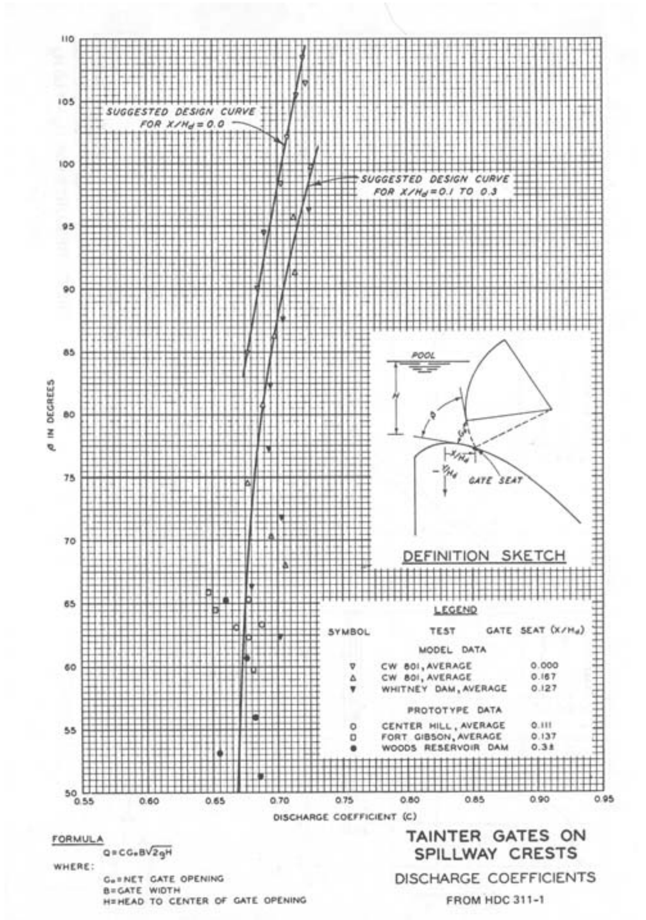


Fig. 5.1: Discharge coefficients for radial spillway gates (from USA Corps of Engineers)

Discharge coefficients associated with spillway flow control using radial gates are provided on Fig. 5.1 (US Corps of Engineers, 1990). The efficiency is affected by the gate seat location relative to the spillway crest. Radial gate discharge coefficients generally range from 0.67 to 0.73 depending on the gate geometry and seat location.

Discharge coefficients for vertical lift gates are shown on Fig. 5.2 (US Corps of Engineers, 1952). The boundary effects are small, but the coefficient does increase as the relative gate opening goes toward 100 percent. The coefficients vary from 0.73 for small gate openings to 0.8 at gate openings of 80 percent. Head loss at gate full-open position is attributed to gate recesses and other boundary discontinuities.

Flow Under Gates: Discharge efficiency is affected by the gate seat location relative to the spillway crest. Radial gate discharge coefficients associated with spillway flow control, generally range from 0.51 to 0.76 depending on the gate geometry and seat location. These values are for free discharge conditions in which the flow rate is not affected by the tailwater. Submerged flow is the case where tailwater influences the discharge under the gate. The case of submerged flow reduces the discharge coefficient and the discharge is not only dependent of the gate geometry, but also the upstream to downstream flow depth ratio.

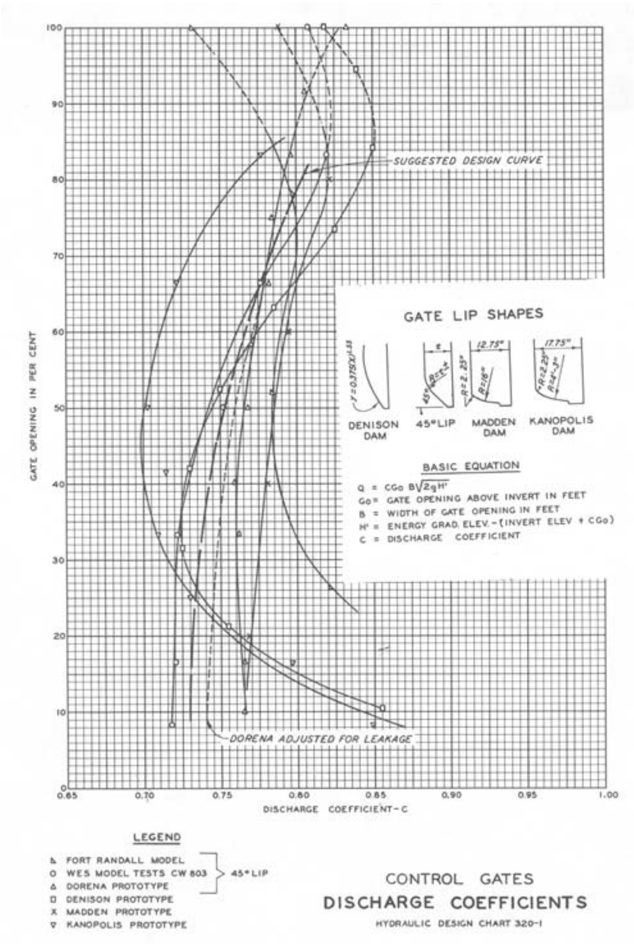


Fig. 5.2 : Discharge coefficients for vertical lift gates (from USA Corps of Engineers)

Discharge coefficients for vertical lift gates are also affected by whether the flow is submerged or freely discharging. The boundary effects are small, but the coefficient does increase as the relative gate opening goes toward 100 percent. The coefficient for free discharge varies from 0.68 for small gate openings to 0.80 for gate openings approaching the full-open position. Head loss at full-open gate position is attributed to gate recesses and other boundary discontinuities.

The different types of gate lips produce significant differences in the head to discharge relationships. Therefore, it is difficult to quantify specific discharge coefficient for a vertical lift gate because it is strongly dependent on the lip configuration.

The shape and elasticity of components such as structural members and seals can affect vibration tendencies, hydraulic loads, and discharge efficiency.

The head/discharge characteristics for many gate shapes differ as gates are moved in raising direction as compared to that associated with gate lowering. This phenomenon is commonly known as the hysteresis effect of gate discharge during raising and lowering.

Flow Over Gates: Flow conditions over gates (typically for flap gates), range in performance from that associated with a sharp-crested weir when the gates are in the fully raised position, to that associated with a broad-crested weir when the gates are in the lowered position.

5.2.2 EFFECTS OF PIERS AND ABUTMENTS

Special consideration must be given to the design of crest piers and abutments. The drawdown of the water surface as the flow accelerates around these features reduces the effective width of the gated opening, L_E. The contractions reduce the discharge at the gate. The contraction caused by abutments and piers are quantified using contraction coefficients that empirically provide the effective width of flow at the gate opening. Values of contraction coefficients depend on the pier shape. Contractions can be as large as 10 percent of the span width. Contraction coefficients are presented in manuals such as EM 1110-2-1603 (1990). http://www.usace.army.mil/publications/eng-manuals/

Surges can be produced by eddies shedding from adjacent piers supporting one radial gate. These shedding eddies produce cross-flow. The intersection of cross flow is inherently unstable. This condition induces self-excited surges that oscillate between the upstream pool and the gate's surface. The result is a low-frequency loading on the gate and discharge variations due to wave oscillations.

5.2.3 VIBRATION TENDENCIES

Flow-induced vibrations of gates in a free-surface environment can be a forced motion resulting from shedding vortices from gate lips. Researchers have focused on vertical lift gates suspended via wire rope (e.g. Bhargava and Narasimhan 1988, Neilson and Pickett 1979, and Campbell 1961). Findings of these efforts have shown that flat-bottomed gates are to be avoided. The inclination of the gate lip should be 45 degrees and that the skin plate should be on the upstream side of the gate.

The major cause of radial gate vibration is the gate lip and bottom seal designs. Fluttering of the rubber seal has generated vibrations of gates. These flow-induced vibrations can be caused either by the shifting of the flow control point between the skin plate lip and other gate bottom members or by the flexibility of the rubber seals which causes them to flutter (Pickering 1971). Schmidgall (1972) found that vibration tendencies could be reduced if a rubber seal is not used. This of course leads to leakage when the gates are closed and so leakage must be tolerated at the project for this strategy to be useful.

5.2.4 GATE AERATION

Many hydraulic structures used to manage rivers and canals, are characterized by the presence of a waterfall. The sections just downstream from weirs are prime areas for major oxygen transfers that can improve the water quality considerably, especially when the waterfall is located in a stretch with high oxygen deficit.

Re-aeration at hydraulic structures is usually characterized by the "reoxygenation rate," which is defined classically as the ratio between upstream and downstream oxygen deficits. The reoxygenation rate depends of the hydraulic structure, the type of flow (underflow, overflow, or mixed flow), the waterfall, the discharge, and the structure of the jet. Experimental results have illustrated the important influence of spoilers or breakers incorporated on the tops of gates.

Bottom-hinged flap gates are subjected to rapid load variations as they are deployed into the raised position. The maximum loads occur, as the nappe formed under the raising gate is broken. The low pressures under the raising gate are quickly relieved to pool pressures as the nappe is ventilated (see Fig. 5.3). This shock on the gate can act as a vibration forcing on the mechanism. Documentation in the literature primarily pertains to gate vibration problems for flow over flap gates held in fixed position. Theoretical discussions of the modes of oscillation of bottom-hinged flap gates are given by Naudascher (1991), Homma and Ogihara (1976), Partenscky and Swain (1971), and Schwartz (1964). These discussions do not mention methods of gate nappe aeration.

Ogihara and Ueda (1980), Pulpitel (1980), Kolkman (1980), and Gill and May (1989) have each reported the results of experiments conducted to alleviate flap gate oscillations. In each of these studies, the oscillations were attributed to an unaerated nappe. There are basically two methods used to aerate a gate nappe. One method involves introducing air to the underside of the gate via an air supply conduit. This method is successful when adequate time is allowed for the air to be drawn in. Air delivery is used successfully when the gate is maintained in a fixed position. Aeration conduits have been designed for projects having structural piers adjacent to each flap gate (Zipparro and Hasen 1993). These piers provide a means of day lighting the air conduit using only a short conduit length.

The second method uses nappe spoilers (or breakers). These are appurtenances placed at the top end on the upstream face of the gate to change the nappe's upper and/or lower profiles. In each of the investigations mentioned above, the spoiler designs were optimized using field tests or hydraulic physical models.

Various spoiler designs have been tested. Only Ogihara and Ueda (1980) provide full documentation of the spoiler details in relation to the flap gate. The shapes tested by Ogihara and Ueda are provided in Fig. 5.4, where the spoiler shape is shown using dimensionless lengths as functions of the gate length (L), the head on the gate (H), and the spoiler spacing (S). Fig. 5.6 shows various spoilers tested by other investigators. Schwartz (1964) suggests that details of spoiler designs are situation dependent.

The results of model tests carried out in 2001 by the Federal Waterways Engineering and Research Institute, Karlsruhe, Germany, have shown, that under prevailing conditions breakers (spoilers) can be built considerably less than proposed by Ogihara and Ueada (see Fig. 5.5)

Fig. 5.3 : Flow conditions during raising of a bottom-hinged flap gate just before the gate breaks the nappe

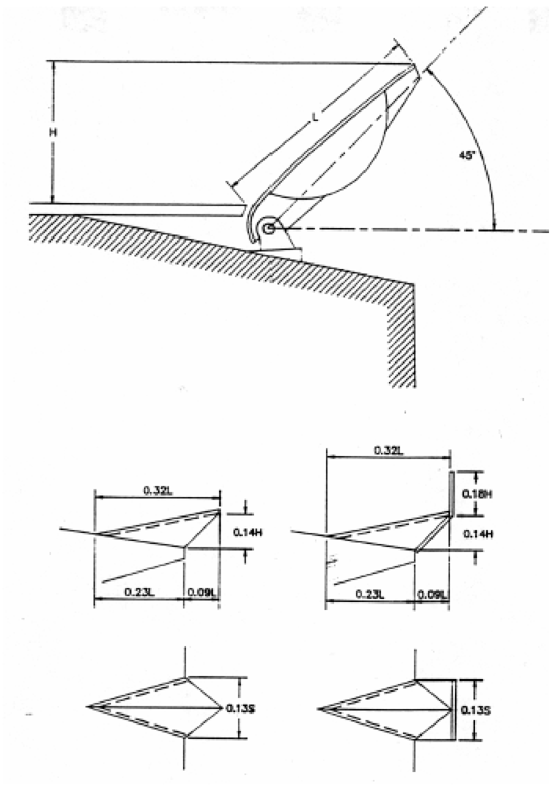


Fig. 5.4 : Spoilers Tested by Ogihara and Ueada (1980)

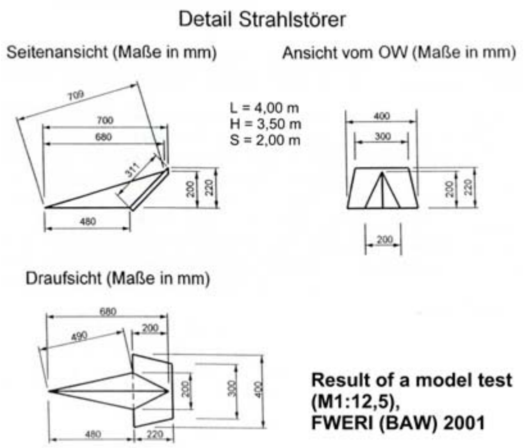


Fig. 5.5 : Test result breaker, FWERI (2001)

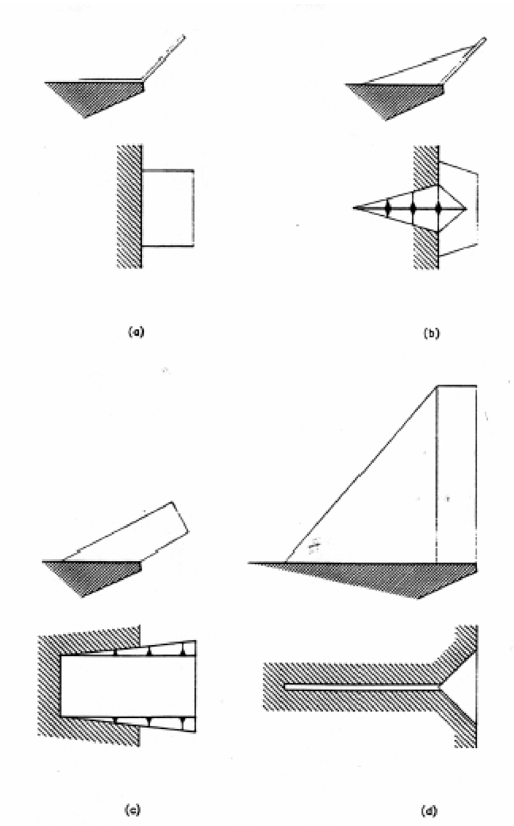


Fig. 5.6 : Various Types of Spoilers

Concerning the effect of air supply conduits it has been proved to be advantageous to place them in different levels (see Fig. 5.7).

Fig. 5.7 : Arrangement of air supply conduits in different levels

An innovative design was employed for the vertical lift gate at the Hartel Barrier (see Project Review on the CD-Directory A1). Air ducts were incorporated into the structural members on the downstream side of the gate. As flow passes over the vertical gate, air is drawn through these ducts thereby aerating the nappe. Fig. 5.8 illustrates the Hartel design concept of venting the nappe for flow over a vertical lift gate.

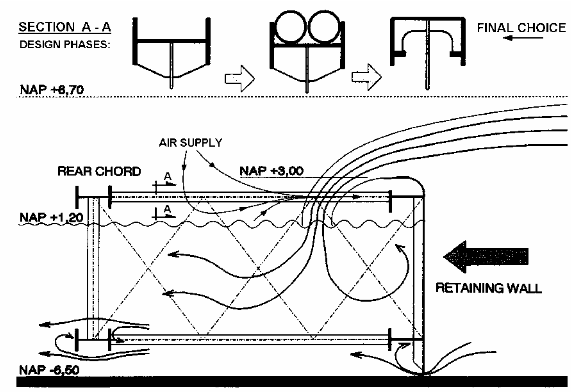


Fig. 5.8 : Nappe ventilation design, vertical lift gate at the Hartel Barrier.

5.2.5 SCOUR TENDENCIES AND SEDIMENTATION

Operation of a single gate can produce an eddy downstream that may pull sediment or riprap into the stilling basin. This material will then tend to erode the stilling basin concrete especially baffle blocks and end sills (Hite 1993).

5.2.6 SUMMARY OF HYDRAULIC METRICS

Table 5.5 is a matrix of hydraulic parameters for various gate types. This is a simplification for comparison of hydraulic conditions associated with various geometric configurations of gates.

Parameters	Gate Types								
	Radial Gates (without upper flap)			Vertical Lift	Flap & Wicket	Bear Trap	Inflatable	Vertical Axis Sector	Horizontal Axis Sector
	Underflow	Overflow	Mixed						
Flow Control	- Efficient, see Figure on C_D - Discharge coefficient varies with gate opening			- See figure on C_D - Discharge coefficient is function of gate opening	Efficient, similar to sharp-crested weir when raised and broad-crested weir when lowered	Raised or lowered; can pass flow over when raised or partially raised	Similar to broad-crested weir, difficult to regulate flow in partially open/closed position	Discharge coefficient varies with gate opening	Discharge coefficient varies with gate opening
Speed of Operation	Variable	Variable	Variable	Variable	Flap: Rapid Wicket: Slow, dangerous and resources consuming	Slow, controlled by opening sluice valves to raise buoyant gate	Site specific, Basically slow	Variable	Variable
Nappe Ventilation	Not needed	Usually air supplied via venting	May require venting	Not needed	Requires flow breakers and/or air supplied via venting.	Not needed	Not needed but fins are used	Not needed	Not needed
Vibration Tendency	- Lip configuration - Critical for small openings	Large potential due to vortex formation downstream	Can experience interaction of over and under flows	- Lip configuration is crucial - Critical for small openings	Vibration reduced with flow breakers and/or aeration	Hinge seal can produce vibration that has led to fatigue	Fins are added to reduce vibrations. Air supplied via venting does not work as large span is used.	Problem only with small gate openings	None
Floating Material Passage	Underflow gates tend to trap floating material	Efficient at passing floating material	Efficient at passing floating material	Underflow gates tend to trap floating material	Efficient at passing floating material, unless it gets caught on flow breakers	Easily passes material, often used in log-sluicing	Subject to damage from floating material	Effective at passing material	Underflow gates tend to trap material
Effects of High Tailwater	Trunnion must be higher than tailwater	Trunnion must be higher than tailwater	Trunnion must be higher than tailwater	C_D is reduced	Can produce tension in the support beams, ➔ risks of unexpected closure.	Has little effect	None	None	Trunnion must be designed for reverse head
Potential for Unusual Hydrodynamic Loads	None	Loads may be caused by flow striking gate arms	Interaction of flow passing over and under	Flow under gate can reduce pressure causing downward forces	Transient loads during gate raising as the nappe is broken	None	None	None	None
Potential for Problems due to Sediment Accumulation	None	Sediment can accumulate during long periods of submergence		None	Sediment tends to accumulate in the bottom recess (depends on the shape of the bottom)	Traps sediments which are difficult to remove	Sediment tends to accumulate in the bottom recess	Potential to silt up	

Table 5.5: Hydraulic Parameters and Performance Metrics of Common Gates

5.3 FOUNDATION AND CIVIL ENGINEERING

This subtask intends to emphasize the main aspects of foundations and civil works related to movable weirs and storm surge barriers. The foundation of movable weirs and storm surge barriers shall be designed to be safe against loads transmitted from the weirs and barriers body, to possess the required water tightness against seepage flow.

The regional and site geologic setting are critical in evaluating the adequacy of a proposed weir or barrier and a given situation (e.g. site location). The foundation conditions available may have a significant effect on the site arrangement, on the design of the structure and on the sequence of construction.

The selection of the most appropriate foundation type is largely based on the site geology, the available geologic and geotechnical information, as well as the performance requirements of the foundation. The type of structure should also be considered. The final decision on the foundation type will affect the total project cost. Foundation investigations and field data are required to assess whether or not a safe and economical structure can be built at a selected site (Fig. 5.9). Especially, in a seismic environment and in locations where differential settling is expected will affect the foundation design. Therefore, foundation investigation is one of the most important issues at the design stage.

Investigations to collect such information are conducted in the field and in the laboratory. Analyses and reference work are performed in the office.

Additionally, the seismic environment of the site will affect the design of the foundation. At the feasibility stages the designers should undertake an appropriate seismic risk assessment and must be aware that some sites are not suitable for barriers, barrages, or dams.

Fig. 5.9(a): Collapse of the Shih-Kang weir (Taiwan)

Fig. 5.9(b): Collapse of a weir due to foundation failure.

For instance, the Shih-Kang weir (Taiwan) was designed with 2 sluiceways and 18 spillway gates. On September 1999, the concrete weir was severely damaged during an earthquake of magnitude 7.3, and the reservoir was released through two destroyed spillway gates (Fig. 5.9). The most spectacular damage occurred near the right abutment and was due to fault movements (reverse faulting) of several metres mainly in the vertical direction. During the excavation of the dam foundations, no fault trace was detected or reported. From this case, it can be concluded that dams cannot be designed economically to resist fault movements of such magnitude (Wieland, 2003).

5.3.1 FOUNDATIONS REQUIREMENTS

The foundation requirements are largely based on the objectives of structure and must be adapted to the site conditions. Foundation must satisfy, in general, two essential requirements:

- Provide a stable support for the entire structure;
- Provide resistance to under-seepage, preventing excessive water losses and degradation of soil components, and prevent the sand from washing away, out from under the barrier piers.

Foundation of the movable weirs and barriers can be achieved on sites with large variety of geological and geotechnical characteristics.

The capacity of a foundation to support the loads imposed by the various structures is primarily dependent on the:

- Water tightness and its associated uplift control,
- Deformability of the foundation (deflection and differential settlements must be within acceptable limits for the serviceability of the gates and other operating equipments),
- Foundation stability.

The complexity of these problems varies significantly and depends on the soil type, stratification, permeability, homogeneity, and other properties of the foundation materials, as well as the size and physical requirements of the structure itself.

Rock foundations have a very large load bearing capacity, resist erosion, and reduce permeability. Regarding weirs founded on rock, the rules applied are classical ones used for concrete dams, i.e. rock consolidation and binding injections and drainage galleries.

Many movable weirs and storm surge barriers are founded however, on alluvial soils or sands. A soil with a high permeability allows water to flow underneath the weir.

The actual foundation design cannot suppress the seepage completely; it can only reduce it by specific methods.
Consequently depending on the upper structure design and site geology, the underground contour of a movable weir/barrier may have the following components: cutoff walls; horizontal aprons, filters and drains, grouting of the rock foundation, grouting of alluvial deposits, and slurry trench cutoff (Fig. 5.10), (Razvan, 1998).

This requires:
- Upstream concrete apron, usually in conjunction with cutoffs, sheetpile walls, cement-bound piles or grouting (Fig. 5.10).
- Downstream concrete apron, with scour cutoffs at the downstream end, and with or without filters, and drains under the apron (Fig. 5.11).
- Cutoffs at the upstream and/or downstream ends under the apron wall and up to the impervious soil (if any) (Fig. 5.11).

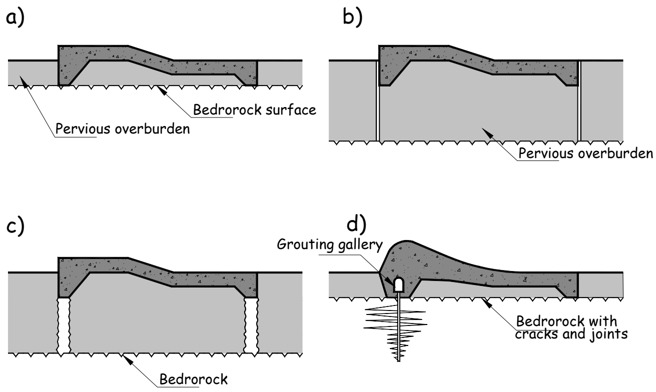


Fig. 5.10: Watertight barriers in dam foundation

a) concrete cutoff wall; b) sheetpile wall; c) cement-bound piles; d) grouting.

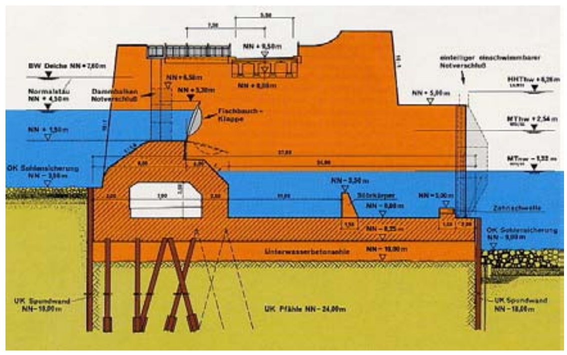


Fig. 5.11: Cross section of the Bremen Weser Weir (Germany)

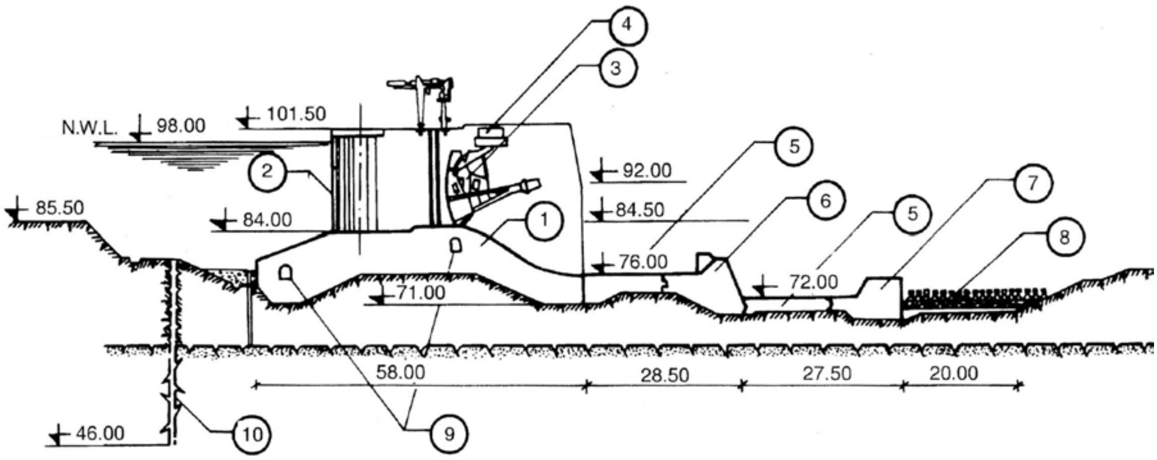


Fig. 5.12: Cross section of Ipotesti Dam (Romania)

1 Spillway; 2 Pier; 3 Radial gate with flap; 4 Machinery chamber; 5 Stilling basin; 6 Baffle; 7 Chute block; 8 Riprap; 9 Drainage gallery; 10 Cut off wall

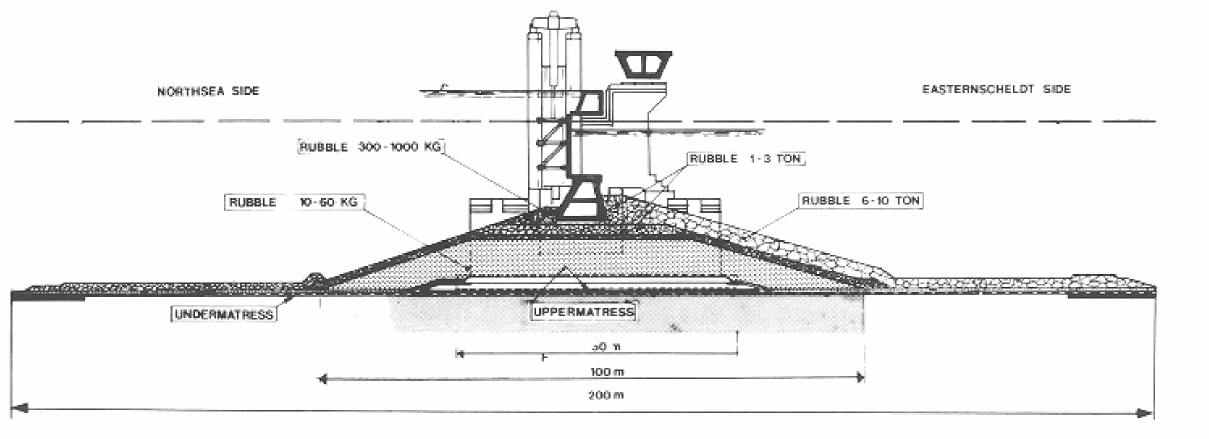


Fig. 5.13: Cross section of the Easter Scheldt Storm Surge Barrier (The Netherlands)

An innovative foundation design example, developed in the case of storm surge barrier, is Easter Scheldt barrier (Fig. 5.13). Since the floor of the Eastern Scheldt is in a constant state of flux, in order to maintain the stability of the structure, a foundation and block mats were constructed. Dredging up sediment and replacing it with sand improved the seabed (Rijckwaterstaat, 1982).

One of the most important functions of the foundation mattress is keeping in place the bottom sand under the influence of the static and dynamic hydraulic forces, caused by difference in water level on either side of the barrier. The mattresses were made of synthetic material and filled with sand and gravel.

Lastly, there is a sill of rocky material, which serves to protect the foundation mattress and so, indirectly, the bottom soil by preventing erosion, caused by currents and waves. The sill is built up from layers of rocky material of varying weight; basalt blocks weighing between 6 to 10 t form the top layer.

Such great weight was necessary, as the sills must resist the strong currents that may arise in the unlikely event one of the gates cannot be closed.

5.3.2 COMPONENT STRUCTURES

A movable weir and barrier is composed of one or more types of structures that operate together to dam a pool of water. The choice of the components and their sizes are dictated by site flow conditions, geotechnical considerations, operational and maintenance requirements, type of gates, construction considerations, and requirements of the user.

The main component structures that compose the fixed part of a gated weir or a barrier are (see Figure 2.1, Section 2.2):

- Piers and abutments,
- Weir sill,
- Stilling basin.

The shapes of these structural components depend of their functions, but also on their serviceability.

Piers and abutments are, respectively, used to divide the river dam discharge field and to provide connection with other structures, which close the retention front or the banks.

The main functions of the piers are to support the gates, bulkheads, gate operating machinery, the operation deck, the service bridge or any road bridge, and to transfer the water pressure to the foundation.

Their thickness depends upon structural requirements and generally has a direct link with the type of gate, as well as with the span width and the pattern of the expansion joints.

The pier cross-section contour may include slots for the stoplogs and vertical lift gate, or steel guidance plates for radial gates, embedded in the final concrete. The slots represent a disturbing factor for the flow that can affect the closure operation and the hydromechanics equipment. Therefore, dimensions of slots should be minimal. In the case of radial gates, the absence of the slots can be considered an advantage.

The pier length depends on the selected type of gates: for vertical-lift gates, this length is minimal; for radial gates, the pier length is determined by the position of the trunnion bearing.

In the case of vertical-lift gates the piers need to be wide enough and long enough to have sufficient space for bearing surfaces for the gates, gate operating machinery, gantry cranes and slots.

Pier shapes and configurations affect the hydraulic performance and discharge capacity of movable weirs. The cross-section should be selected between a hydrodynamic profile, offering the minimal resistance to the flow, and a rectangular shape, the simplest one. In practice, the most common and usually most satisfactory design is a semicircular pier nose shape. The downstream end of the pier can be flat.

Weir sill and ***upstream and downstream aprons*** accomplish the sealing of the weir at the bottom face. The main functions of a sill weir are: to be a foundation structure, support the main gates, enable energy dissipation, and prevent seepage. The sill is a massive structure with an elevation close to the elevation of streambed.

The lower the head on the crest of weir sill is, the lower the unit discharge. This results in a longer crest but less requirements for the stilling basin and downstream channel protection. Conversely, the higher the head on the crest is, the higher the unit discharge. This results in a shorter crest length but greater requirements for the stilling basin and downstream channel protection.

In order to provide sufficient space for operating hydro-mechanical equipment, a broad-crested weir is often indicated and structural requirements usually dictate the width of the crest to be approximately the same as the damming height of the gates (U.S. Army Corps of Engineers, 1987 and 1995).

For structures that do not operate under submerged flow conditions, an ogee crest is often used to improve the efficiency of the spillway.

On the sill crest at the gate and stoplogs seats, steel parts are embedded in the final concrete. Their purpose is to ensure a tight contact with the seals of the gates.

***Stilling basins*:** After the sill, a horizontal concrete apron is designed on a lower elevation in order to increase the length of the path of percolation, to reduce uplift downstream of the dam and to provide a basin where the energy of the overflowing water can be safely dissipated. Energy dissipation on the concrete apron helps to prevent dangerous erosion at the toe of the weir.

In case of pervious (soil-like) foundations, an end sill should be provided with a concrete cut-off, riprap deposit or gabion mattress or a combination of the two. This serves as a safety measure against piping and local scour effects.

The dimension of the stilling basin and other possible energy-dissipation structures are determined by the hydraulic design (Novak et al. 1997; U.S. Army Corps of Engineers, 1987 and 1995).

Additional component structures in the case of movable weirs are *draining walls*. They are designed to control flows upstream and downstream of the dam where variations in the project features may cause unwanted hydraulic effects.

5.3.3 STRUCTURAL CONFIGURATION OF FIXED CONCRETE PART

Weir and barrier concrete structures can be designed as a monolithic structure. However, none can be constructed as a single monolith, but rather as a set of monoliths linked together in some way. These links are realized by joints, which allow the set of monoliths to behave as a single structure.

The sources of discontinuities in a large concrete structure are the construction conditions, and the required flexibility of the structure. The cast-in-place versus prefabrication is described in Section 7 “Prefabrication”.

The possible structural configurations depending on foundation conditions are: (Fig. 5.14):

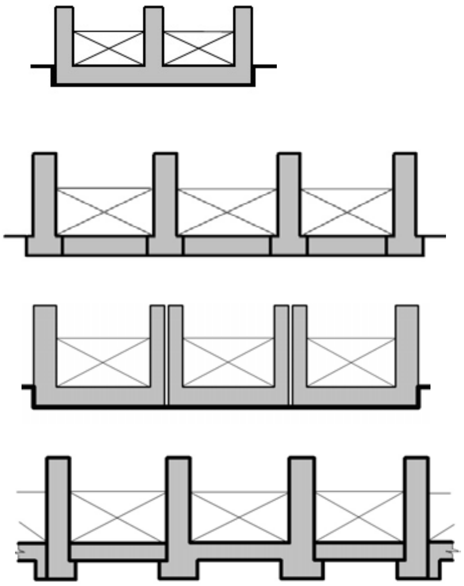
a) General foundation without expansion joints,
b) Independent piers, with joints between piers and slabs,
c) Independent bay structures,
d) Mixed configuration.

a) The main advantages are: the construction of the structure in one stage; the concrete works are achieved in one step; homogeneous foundation conditions are required.

b) Independent piers are separated from the weir slab by expansion joints. The main advantages are: it allows execution of the works in several stages; it allows flexibility in the schedule of excavation and concrete works; it requires minimal reinforced steel, as the bending moments are minimal. The main disadvantage is the sensitivity of the gates’ operation to differential settling of the piers and the slab.

c) An expansion joint divides each pier along its middle vertical section. The system is suitable for compressible soils foundation, with different characteristics along the dam axis. The main advantages are: safe operation of the gates, because there is no risk of differential settlement between two half piers supporting a gate; it allows a flexible works-execution schedule. The main disadvantages are: the quantity of reinforced steel must be increased due to the important bending moments; reduction of the bay width because of the increase of the total pier thickness.

a)



b)

c)

d)

Fig. 5.14: Typical types of foundation

The expansion joints patterns must be compatible with the operation of the gates and must prevent blocking due to differential settling.

d) The mixed configuration combines the advantages of the systems *b)* and *c)*. Movable weir and storm surge barrier construction normally requires a dry construction site. As these structures are sometimes located across or near streams, cofferdams are required for site dewatering and a reasonable degree of flood protection. The cofferdam used for the construction may reduce the river cross section. Usually several alternate diversion schemes are investigated to find the best feasible and economical solution. Alternative construction methods, based on prefabrication, can be applied when dewatering is not possible (see Section 7 and Project Review E10 on the WG26-CD).

5.3.4 EARTHQUAKE AND TSUNAMI

The effects of earthquake on the structure are due to seismic motion, fault displacement, liquefaction of the ground and water level and current (caused by tsunami).

The seismic motions are appropriately determined in each limited states. The seismic performance of the structure is usually described by a performance matrix. The weir and barrier are required maintaining serviceability during and after an earthquake (seismic motion, fault displacement, ground failure, settling). At the design stage of a long and large structure, the spatial variation of seismic effect should be considered. Saturated loose sandy subsoil tends to liquefy during the earthquake, causing damage to structures. When designing in a seismic region, effect of liquefaction should be considered (see PIANC Guideline of MARCOM-WG34, 2000).

5.4 CONTROL, OPERATION AND MAINTENANCE

5.4.1 INTRODUCTION

This section investigates the control systems used on the Movable Weirs and Barriers reviewed by the WG. The investigation should enable an informed decision on the advantages and disadvantages of the various systems in use and assist in the selection of a control system for a new construction.

As well as the control functions of the mechanical, electrical and computer systems the investigation shall include the controls imposed on the operation by statutory bodies such as the Environment Agency in England and VNF in France (see their web sites in Section 10.1).

The investigation will also consider operational aspects including the manning implications of the systems adopted and the method to isolate the gate for maintenance.

5.4.2 METHOD

A detailed questionnaire was sent to each reporting member, along with guidelines to assist in its completion. Both the guidelines and the questionnaire are available on the CD-Directory /Annex Section 5.4 /. This was followed by further questions depending on the issues raised within the initial response, either specific to a structure or to satisfy a global issue. The results from the questionnaires and the author's own experience was used to complete the task.

5.4.3 CONCERNED STRUCTURES

5.4.3.1 Control Weirs

These structures are designed to operate over a specific range of movements controlling the level in an impounded body of water. Most of the Control Weirs in this report include some flood defence function and are capable of rising further to prevent or reduce flooding from high tides.

Control weirs are prone to generate a reflected wave when brought into use but with good control developed by physical modelling this effect can be minimised.

There are complete reviews of these structures in the CD version of this report (CD's Directory A1).

5.4.3.2 Storm Surge or Flood Defence Gates

These structures are dedicated to flood defence rather than level control. This means they are either fully open or fully closed, unlike the control weirs that can assume any intermediate position within the range of open to closed.

For flood defence, swinging gates have been designed to seal the river against tidal intrusions. These types of gates can only be moved in low flows or with minimal differential head (See Project Reviews: Bayou Gates, Louisiana). These last two examples are in direct contrast to some of the flood defence structures in the Netherlands (Delta Plan) and Belgium (Blanc Pain) that operate entirely automatically and are merely observed from a remote location.

Again, there are reviews of all these structures on the CD.

5.4.4 MAINTENANCE AND RELIABILITY

A) Although maintenance is covered in more detail elsewhere in this report, there are some aspects that can be mentioned here. When designing a structure it is important to recognise that reliability and ease of maintenance cost money but may also save money, long-term. A contractor may seek to win a job by offering a low capital cost but leave the owner with higher operating costs. It is important in the design stage to recognise these factors and budget accordingly. See also "Maintenance and Renovation of Navigation Infrastructures" (PIANC-WG25, 2005).

For operational purposes, it is easier if items requiring maintenance can be accessed easily and without extensive dewatering procedures. Two examples, the Thames Barrier protecting London and the Maeslant Barrier defending Rotterdam, came about as a result of the devastating 1953 floods. Although full reviews are available on the WG's CD, it is worth noting that both these structures are capable of being maintained in the dry, without the use of stoplogs or dewatering. For the Thames Barrier the drive machinery is installed in the pier voids and the gates can be rotated out of the water for maintenance. At Maeslant the gates are normally held in "dry docks" cut into the riverbank and these are flooded to take some of the weight off the bearers or when slewing the gates into the river. The drive machines sit above the gates and move them in or out by a gear train.

The procedure for gate maintenance at the Tees is described more fully in Section 5.5 "Temporary Closure Devices", but to access the sluices or the gates and gate seals requires heavy lifting gear to be hired in to manoeuvre the stoplogs in or out.

At the Lagan Weir the gates are lifted out vertically rather than floated in, as they are wider than the openings, which use the pier noses to protect the gate edges and hydraulic cylinders. The gates are lifted in and out with a large floating crane and because of this difficulty are designed for a longer service interval than usual, ten years as opposed to annual. This is partly achieved by selecting a heavier duty seal for the gate/pier interface and because total saline exclusion is not required it is set up with less 'pre-compression' leading to reduce wear. Pre-compression refers to how tight the rubber seal is between the gate and the pier. There is a seal to stop water from leaking through the gap. It is fastened to the gate and its position can be adjusted for wear. The tighter the seal pushes against the pier the better the seal, but also the greater the friction and so the seal will wear out quicker. The gate may have some tolerance so that it can move on its hinges or expand with temperature changes. Pre-compression is adjusting the seal so that it is under compression with the gate in its neutral position. Then if the gate moves either way one seal will compress more and the other will relax but because it has some compression in it already, it will still seal the gap. At Lagan the sealing is not critical and access is difficult for maintenance so the seals are set up with less compression to reduce wear. At the Tees Barrage the upstream is used for recreation and the downstream is quite polluted so it was considered important to have a very good seal arrangement. The seals keep out the polluted water when the high tide exceeds the impounded level, about eight times a month.

B) The legislation governing the structure may influence its operation. If a certain reliability is quoted, for example only to fail once in every 1000 years, then the equipment and the operation will reflect this with multiple redundancy and top quality components whereas a lower value might be achieved with a simpler, cheaper solution.

An example of reliability versus consequences of failure concerns vertical lift gates and whether it is "better" to latch the gates in the raised position or support them by the pressurised rams. When the gates are needed will the latches release? Will the rams pressurise to take the weight off the latches? Where rams support the gate, it can be dropped by gravity using a hand operated dump valve if the electrically operated valve fails. But what is the risk – and consequence – of the gate dropping inadvertently on a passing vessel? Or dropping and impeding the fluvial flow with resultant increase in upstream level? And do you still need a latch to support the gate when a cylinder is removed for maintenance?

Issues to do with Safety, Reliability and Risks are dealt with more fully in Section 5.6.

5.4.5 MANNING ISSUES

A) Running a structure on a 24-hour basis, 7 days a week, has serious cost implications. Shift patterns vary but the usual patterns are 3-shift: 6am to 2pm, 2pm to 10pm and 10pm to 6am or 2-shifts: 6 (or 7) am to 6 (or 7) pm and then another equal 12 hours night shift. This shift pattern is efficient when staffs are likely to be working on various tasks away from the base, as it avoids the disruption of a changeover in the middle of the day. Since the introduction of the Working Hours Directive it is even more important to consider the social aspects of continuous manning. Countries outside the European Union will have their own employment legislation but the principal of socially acceptable terms and conditions should still apply. It may be reasonable to have people working alone but a full risk assessment must be undertaken and complied with at all times.

B) Contingency measures for absence must be in place. There may be day workers normally on maintenance duties that can cover the duties of the shift operators for sickness or holiday periods. If safe operation can be achieved on a purely automatic basis and the risks of vandalism are small then the investment in equipment - capital cost and maintenance – can be offset against the savings in payroll costs. A decision needs to be made on the skill level of the staff employed, do they carry out the maintenance or is it contracted out? Again, better-qualified staff will cost more but may still be economical.

C) Overheads in administration, holiday pay, sick pay and any pension will be significant, although there may be some economies of scale within a larger organisation. Some welfare facilities would be required even at an unmanned site but for a manned site these could be quite extensive. Toilets, kitchen, rest rooms, office space and car parking as well as the extra heating and lighting will all add to the costs, both capital and revenue.

D) Closed Circuit Television (CCTV) may be required for both operational and security purposes and the images can be transmitted to another location if the site is to be unmanned.

E) Investment in automation may be economical in reducing manning levels, although the maintenance of a more sophisticated system may have further cost implications.

5.4.6 RESULTS OF THE QUESTIONNAIRE

The completed questionnaires are available on the CD version of the report (Directory /Annex Section 5.4/)

5.4.7 ANALYSIS

It is more accurate to refer to PLCs (Programmable Logic Controllers) for controlling the system rather than PC's (Personal Computers) that we are more familiar with in an office environment but 'computer' will be used throughout.

A) Depending on the criticality of a structure, the computer systems can be duplicated. They all have some level of battery back up and emergency generators or alternative power source for gate movements. Compliance with national and international standards, such as the British Standard BS.61508 "Functional Safety of Electrical, Electronic and Programmable Electronic Safety-Related Systems" is vital. Different countries will have their own standards and codes of practice, (see Section 9). The expected life of most computer and electrical equipment is 10 to 15 years and the structure itself may have a design life of 75-120 years, so a true cost analysis will need to include this equipment being replaced several times over.

B) Where a choice of 'Manual' or 'Automatic' is available it is still common for the manual actions to be in the form of inputs to a computer and the outputs regulated according to all the safety considerations programmed into the system. In automatic, the desired setting is entered into the computer via some MMI (Man Machine Interface) or - more simply - a desk and keyboard! See also SCADA (Supervisory, Control and Data Acquisition) below. The computer recognises the action required and carries out the command, according to the programme loaded within it.

C) 'Hard wired', where the input switch or push button is connected by electrical cables going directly to the device being operated, is usually only found on the front of a motor starter, electrical panel or a local control station. On short cable runs like these the volt drop may allow cable sizes of 0.5 or 0.75 mm^2 to be used but for mechanical strength a limit of 1.0 or 1.5 mm^2 is common. Stranded conductors are preferable to solid as they have more flexibility making them less prone to failure due to vibration.

D) Control actions are generally classified into four types, Step, Proportional, Integral and Derivative.

- ➔ Step Control is the simplest, often as basic as on/off such as a thermostat or open/closed for a valve but also applies where the corrective action, such as gate movement, is the same fixed amount each time.
- ➔ Proportional Control is where the movement is not fixed but varies in proportion to the error, so the greater the difference between the measurement, say water level, and the desired value or set-point, the greater the corrective action.
- ➔ Proportional can be used in conjunction with Integral where the longer the error exists, the greater the correction applied.
- ➔ Derivative can also be added where the rate of change of the error affects the corrective action. This can be useful to pre-empt a flood event on a river. The rate of change can also be calculated in the computer for any measurement and used to warn operators of the event, leaving them to decide on the appropriate action.

E) The more sophisticated systems are quite common in chemical and process industries but, in general, step control is more than adequate for the type of structure we are considering.
By the nature of the application, the gate movements are quite coarse, often measured in tens of centimetres rather than fractions of a millimetre. Similarly, the time constants are large due to the sluggish nature of the parameter being controlled, usually the level of a large mass of water.

F) A gate or weir can generate a reflected wave when moved, but with good control developed by physical modelling this effect can be minimised. The step size, its frequency, and the actual speed at which the gate moves, are all factors to take into consideration. Where a gate is used for flood defence and is either open or closed and the "step" is full stroke. Introducing stop/run timers or variable speed to slow the movement as the stroke increases can smooth the action. Again, modelling should be able to identify these situations and provide solutions.

G) Nowadays the gate operation is generally by hydraulic cylinders although there are some examples of lifting gates that are electrically driven. These include for instance the Hull Tidal Barrier in England and the Rotterdam Flood Defence Gates in the Netherlands. Other examples of mitre or sector gates not reported on but with electric drives are Goole (See Project Review), England, used for protection against loss of water should the canal bank fail and the lock gates at Hull Marina, again in England. They use gearboxes or winches to move the gates and both are owned by British Waterways. Hydraulics is reliable and simple to control and can easily generate the large forces needed to move the gates. To produce the smooth stroke with electric drives requires sophisticated 'Soft Start/Soft Stop' speed control with associated problems of high starting currents and harmonics in the power supply. There have also been reported cases of bearings in electric motors failing due to circulating currents induced by variable speed drives (see www.abb.com/motors&drives, Bearing Currents and Electrical Discharge Machining - EDM).

H) It is also evident that there is no means of directly measuring the height of the gate lip. It is always derived by calculation from some other measurement, usually the extension of the ram. This is easily built into the system at the manufacturing stage, where the ram is 'corrugated' along its length, with a ceramic coating providing a smooth finish. A counter reads the number of high points passing it as the ram moves and these 'pulses' are translated into a distance by the computer. The computer then uses a 'look-up' table to convert the distance i.e. stroke of the ram, into a value for gate lip height. For reliability two or three counters can be installed and the readings compared, generating an alarm if a discrepancy occurs. Cumulative errors can occur over time if the pulses are miscounted for any reason or from drift in the instruments. The error can be removed by periodically fully retracting the ram to a limit switch that resets the counter to zero.

I) Where alternative drives are installed then other types of position sensors such as potentiometers, LVDT (Linear Variable Displacement Transformers) and shaft encoders can be used to produce a reading of gate height.

J) Apart from using a limit switch at zero stroke to reset some position measuring devices, a similar arrangement can be used to stop the drive unit at the end of the stroke, either fully raised or fully lowered. Alternatives are pressure switches that trip the system when the ram is fully extended and the pressure rises. However, they may give false readings due to pressure rise from frictional forces or physical obstructions to gate movement. Limit switches or relays often have duplicate contacts with one set used for the actual control and the other for inputs to the SCADA.

K) An example (Tee Barrage) is presented (Fig. 5.15), where analogue signals from position or level sensors being repeated to a SCADA system should be via Isolating Amplifiers so that any fault on the field wiring does not affect the computer circuits, and vice versa. See other examples on CD's Directory /Annex Section 5.3/.

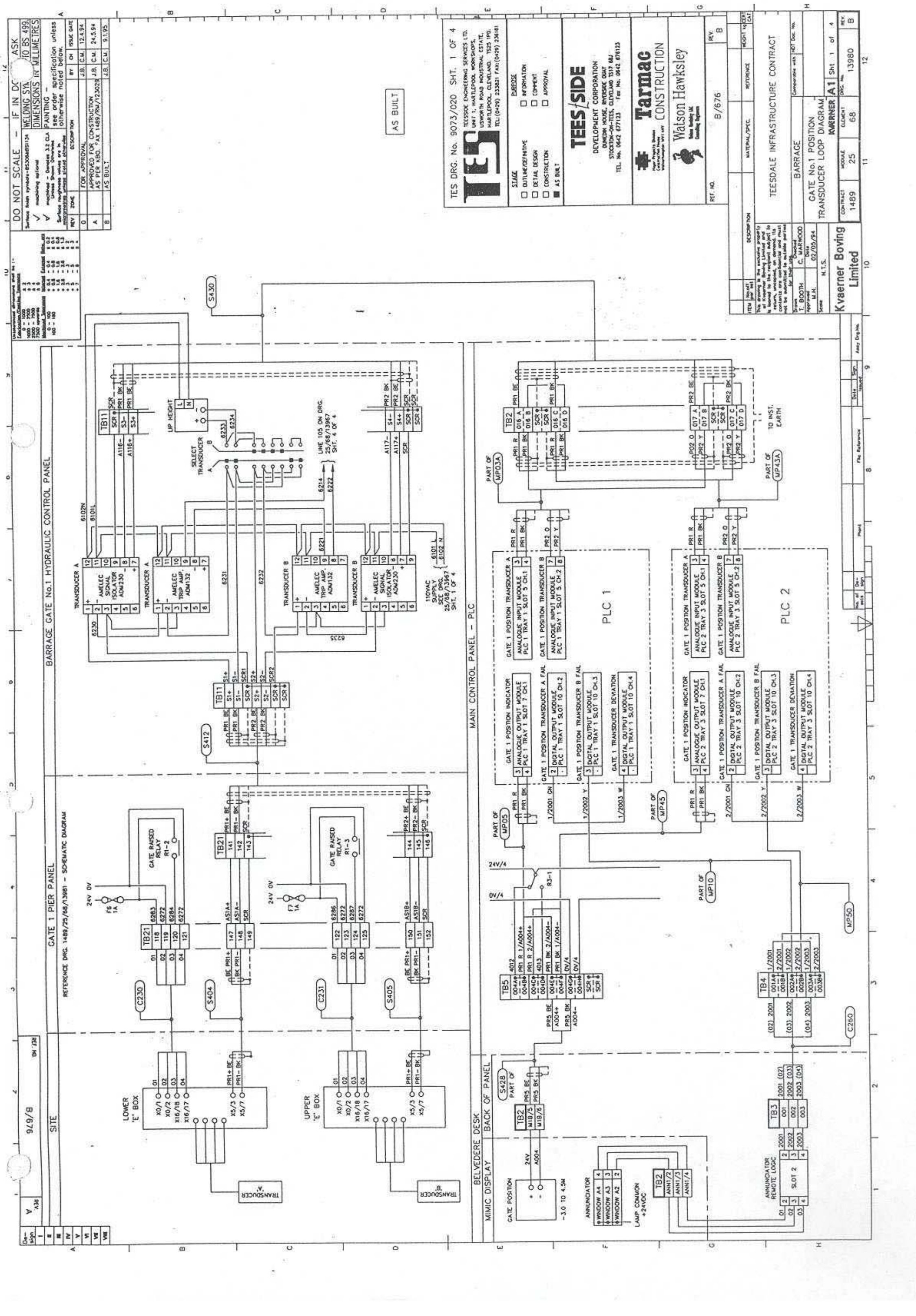


Fig. 5.15: SCADA system

L) Water Level Measurement
Whatever the function of the structure, it is nearly always necessary to know the water level on either side of the gate(s). There are a number of ways of doing this and all have their own strengths and weaknesses.

➔ Devices in contact with the water:
- Pressure sensor
- Float and shaft encoder
- Capacitance sensor

➔ Devices not in contact with the water:
- Ultrasonic
- Radar
- Bubble pipes

➔The basic difference between contact or non-contact devices is that the sensor may either be immersed in the water or separate from it. Non-contact may be easier to access but more complicated needing skilled maintenance and are also more vulnerable to damage or vandalism. Devices in contact with the water such as pressure or capacitance sensors can be better hidden but also be as complicated as the non contact type so still require as much maintenance. Floats, with shaft encoders and chart recorders are very simple to operate and maintain, but need stilling wells. These can be remote from the water providing security for the equipment and for a lock, weir or dam the extra structural works is probably insignificant.

➔ Pressure sensors can have a piezo crystal in the sensing head that generates a voltage proportional to the pressure – head of water – acting on the device. This voltage is detected by the transmitter remote from the sensor and can be processed for data logging on site or transmission by telemetry.

➔ Other types of pressure sensors use compressors or gas cartridges and work on the “bubble” principle. This is where air/gas is bubbled through a tube laid to the riverbed and the pressure measured is at equilibrium with the static head. These can suffer from silt or turbulence.

➔ Ultra-sonic devices transmit a beam onto the surface of the water and detect the reflected beam on its return. By measuring the time taken and knowing the speed of the beam it can calculate the distance and therefore the water level. A tube or stilling pipe usually restrains the beam, also serving as a mounting platform for the instrument. Fitting a blanking plate with a small (say 80mm diameter) hole in it at the bottom of the tube will damp out level changes from external wave action. This improves the accuracy and also helps to keep out debris.

➔ Radar devices work on the same principle but at different frequencies. They are more reliable than the Ultra-sonics and are becoming increasingly popular.

➔ It is common to have multiple sensors allowing the computer to establish a reading based on the average of 2 or best 2 out of 3. This choice depends on how critical the reading is to the operation. Having the same system for both sides of the gate, with multiple sensors, can cause a problem. If, for some reason, the sensors develop a common fault then all readings will be lost. Such a scenario is more often found in extremely sensitive applications such as nuclear power plants or chemical works. To improve reliability it is possible to mix sensors from different suppliers or use a combination of devices, for example ultra-sonic and pressure.

➔ It is recommended that an ultimate fail proof device always be included. At the Tees Barrage, in the North of England, this was a simple gauge board, visible from the control room, allowing operators to maintain some control even if all the instrumentation failed!

M) Flow, over a gate, is a function of the head and the discharge coefficient. The latter is difficult to ascertain and approximations are often adequate as in most cases it is repeatability rather than absolute accuracy that are important. There are not many instances of using flow as the control measurement, level is much more appropriate and an accurate figure more easily obtained.

N) Flow in rivers is determined by a ‘Stage-Discharge-Calibration’ where a cross section is scanned by a velocity meter and an approximate formula derived for flow at different water levels. It can be useful for anticipating flood events allowing pre-emptive adjustments to the system. The Lagan[2] in particular uses flow measurements on tributaries for this reason whilst the Tees Barrage[2] has sensors at the estuary and at two points on the upstream river and tributary for the same purpose. At the Tees, it was found that the tide level at the estuary was only a few minutes ahead of the level immediately downstream of the gates. Such information is not always constantly available in real time, often being obtained by telemetry although when needed, the communications channel can be left open. This is uneconomic under normal circumstances where the outstation is usually polled (interrogated by the master station) on a routine basis and accumulated readings downloaded to the operating system.

O) SCADA (Supervisory, Control and Data Acquisition)

➔ It is beneficial to store operational measurements for legal and managerial purposes. Storage devices such as tape and optical discs are being replaced by hard disc drives where larger quantities of information can be stored and quickly accessed when needed. As part of the management information system it is common to find a SCADA system alongside the industrial computers used to operate the structure. The "Supervisory" part of SCADA allows operators to supervise or watch over installation, usually by "mimics" or graphical representations of the processes on a VDU (Visual Display Unit). The level of detail can be as much or as little as the customer wants, but it must be decided during the configuration stage for economic reasons. The "Control" was previously referred to as the

Man Machine Interface. The "Data Acquisition" is the measuring, display and recording of such values as may be of interest to the operators or regulatory bodies. Such a system can plot graphs of instantaneous flows and levels etc. or record integrated values over periods of time varying from hourly to annual. It can also be configured to generate scheduled reports of any logged value or event.

➔ The SCADA screen and keyboard are also the easy way to issue commands to the operating computer and to view plant status on mimics. Touch screens and drop down menus are becoming more common now making the operators' task both easier and less liable to error. It is possible to programme the mimics to guide the operator through his task, actually highlighting the preferred responses. The Falkirk Wheel in Scotland, is a good example of this where not only is the expected action highlighted but confirmation of the entry is also required before the operator is allowed to proceed to the next step. A message pops up asking "Are you sure you want to do…" with "yes" or "no" tick boxes for the answer. The SCADA can be remote from the site allowing operators to manage multiple locations simultaneously. As highlighted in the questionnaire, personnel costs can be quite significant and the economies of remote operation must be considered.

➔ As expected, few of the structures are operated without the extensive use of computerised controls. The floating gates[2] from Tennessee and Louisiana in the United States of America, are purely manual operation. In some instances, usually flood defence structures, elements of the decision-making are left to human operators but to achieve the required safety levels computers provide the necessary protection. Blanc Pain[2], in Belgium, is one example of an unmanned, totally automatic flood defence operation, although it is monitored from a nearby site.

➔ In almost every case the primary value being monitored is the level of the water above the control structure. It is only in tidal defence cases such as the Thames Barrier[2] that the downstream level is of primary interest. Blanc Pain, the flood defence on the "Canal du Centre", Belgium, monitors level and flow and seismic disturbance to determine when it should be closed.

➔ When a structure has a dual role in navigation and tidal exclusion, then both upstream and downstream are closely monitored. A rising downstream level will instigate the raising of the gates to prevent flooding of the impounded waterway. Measuring the downstream level has a secondary function in flow calculations once the gate lip drops to the point that the formula changes from 'broad crested' to 'drowned' weir. Knowing the level either side of a lifting gate is essential if the gate can only be moved under near equilibrium or zero differential head conditions. Similar constraints are used in mechanised locks where the levels can be measured upstream, downstream, and in the lock chamber to prevent the gates from being moved prematurely. Where no such interlocking between levels and gate movement is employed, then the drive systems must have some protection against overloading. Fuses, circuit breakers and pressure relief valves to cater for the normal operation and fault protection of the system can also deal with these extra loads.

➔ Typical is the control philosophy employed at the Tees Barrage, where the intention is to maintain the upstream river level within a fixed band. The Environment Agency set this band but the operators have the discretion to select a desired level within that range. Using the storage capacity of the impounded river it may be necessary to set the level high or low within the band depending on the state of the tides and the flow in the river. Hydraulics Research at HR Wallingford was employed to analyse all the permutations of the control philosophy. The upper and lower limits of the desired level could be set individually or by applying a 'deadband' either side of the desired value or set-point. The corrective movement of the gate could be fixed or variable amounts depending on the difference between the actual level of the river and the desired value. The interval between sampling the river level could be set short or long, seconds to minutes.

➔ The main findings of the research were:
- Narrow deadband with large gate movements and short sampling interval meant unstable control with a large number of corrections in a short period of time.
- Wide deadband with small gate movements and long sampling interval meant poor control with large errors persisting, even allowing the level to vary outside the EA (Environment Agency) fixed band.
- Frequent corrections mean that electric motors for the hydraulic system must be higher rated to withstand the stresses of so many starts in a short period.
- Ideally the corrections should be subtle enough not to be noticed!
- Modelling means that the installed system will require very little tuning
- Modelling means that the operation and effects of any adjustments can be explained to the staff during commissioning.

P) Each structure will be unique in terms of the relationship between the size of the gates, their discharge capability compared to the volume of the impounded river and statistical flows so it is recommended that modelling be carried out in every case. Mathematical modelling is more appropriate than physical modelling for control but physical modelling can be used to advantage for structural considerations. It can also assist in determining maximum discharges and effects on sedimentation. At the Tees Barrage (UK), physical modelling led to the design changing from five gates to four with resultant savings in the civil works and hydraulic systems.

Q) Information about of friendly environment lubricants are available at *CD-Directory/B8- Environmentally Lubricants (BW,UK)/.*

5.4.8 SUMMARY

The philosophy "Keep it Simple" is always good but not always realisable! There are examples here of very simple flood defence structures that work well but need a lot of manual input, both in initial decision making and then in actual operation. There are also some very sophisticated structures that operate entirely by automation. The real question lies in the reliability of the system and the consequences of failure. It is recommended that all critical elements of the control system be duplicated and that the power supply and drives be backed up to some extent.

As long as the proper maintenance is carried out, a properly designed system will operate reliably over its predicted design life. The problem is that the more sophisticated systems require more maintenance by a skilled workforce. Any shortcoming in the application will be reflected in the performance of the structure, perhaps not noticed until it is too late.

Comprehensive test routines to validate the performance of the entire system should be in place and used regularly. Times and dates of these tests should be recorded along with any observations on system performance.

5.5 TEMPORARY CLOSURE ARRANGEMENTS

5.5.1 DEFINITION OF 'TEMPORARY'

It is important to separate "maintenance closure" from "emergency closure" and "site construction closure" systems. This report mainly deals with maintenance closure.

Typically, emergency closure systems are vertical-lift gates that remain suspended. They are expensive systems.

Few emergency systems were considered in the WG's Project Reviews (Blanc-Pain Gate in Belgium and the Hartel Canal in the Netherlands).

Site construction closure systems can be quite similar to maintenance closure. The "Pallet Barrier" is probably the best example.

For our purposes, a temporary closure is defined as either:

- a closure required to make the structure available for maintenance or repair.
- a closure required to ameliorate the effects of a flood event or breach where no fixed device is available.

Examples for the first situation would be stoplogs used to seal off a structure so it can be dewatered and accessed for maintenance.
For the second it could be the use of a floating cofferdam brought to the site of a breach in the canal bank or a damaged gate and deployed to control the leak until a permanent repair can be carried out.

5.5.2 CLOSURE DEVICES OR BULKHEADS

A *bulkhead* is a vertical partition used to seal off one space from another, capable of withstanding the differential head without significant deformation or leakage. Bulkheads are a variation on Stoplogs and are generally one piece construction rather than sectional or modular.

There are several devices capable of being deployed to be a temporary closure. A few common examples are:

- Stoplogs (Fig. 5.16, Fig. 5.17)
- Needles (Fig. 5.18)
- Cofferdams
- Caissons
- Air or water bags
- Palets, etc.

Stoplogs are planks or beams that fit into slots in the sidewalls of an opening. The size depends on the span and the lifting facilities. In some cases intermediate vertical supports are employed to reduce the width of the individual stoplogs, making them easier to handle, or to increase their versatility.

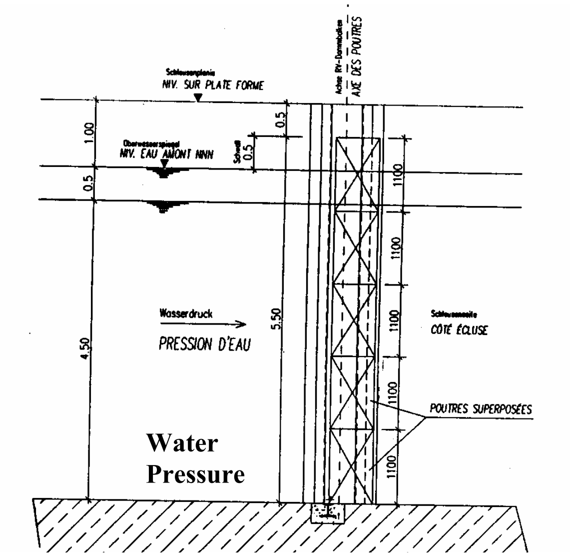


Fig. 5.16 : Beam-Stoplogs (cross section)
Size: 1.10m x 0.7m by 12.5 m; Germany

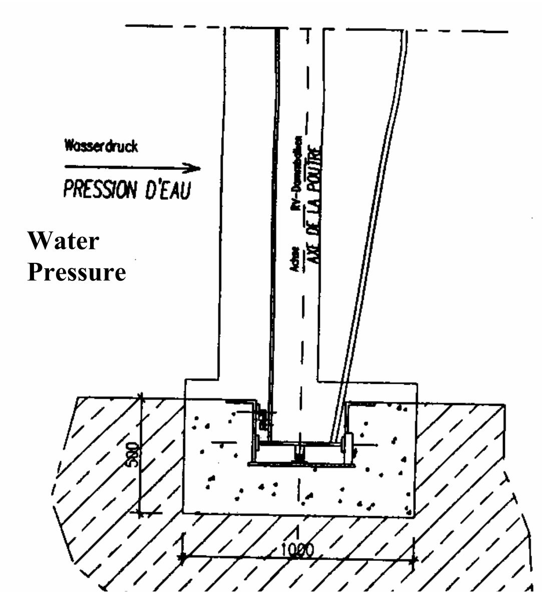


Fig. 5.17: Beam-Stoplogs (Bird View)

Needles are beams of 2 to 4 m long and typical section of 7x7 cm to 10 to 20 cm. They can be in wood, aluminium or steel. They are located vertically, supported at their upper part on a beam-girder or a part of an upper bridge (Fig. 5.18). A slot in the floor is used for lower support. Needle weirs are not 100% watertight. To reach a good water-tightness a plastic sheet can be placed before dewatering. Such systems usually require divers.

Fig. 5.18: Bulkhead needles supported by an upper beam

Cofferdams are box like structures that can be floated into position and ballasted in such a way as to seal the opening. (See: Kentucky Floating Caisson Project Review).

Caissons are similar to cofferdams but may also be part of a fixed structure to facilitate their use

Other Bulkheads Systems:

German bulkheads (canal):
Prefabricated elements of about 2 m wide (Fig. 5.19):
- H shape vertical beams in slot,
- Steel Panel (1x 2 m),
- Inclined beam support (need slots in floor),
- Require divers and floating pontoon.

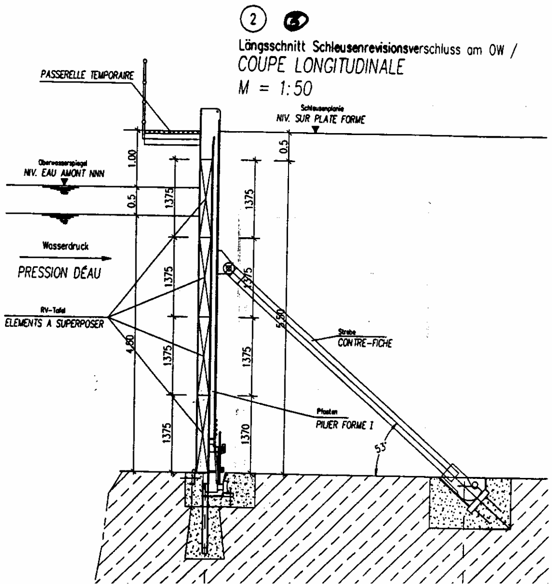


Fig. 5.19: Elementary Bulkhead Element (Germany)

Pallet barrier:
Self-supported bulkhead with watertight membrane
(See: Annex Section 5.5 on CD)

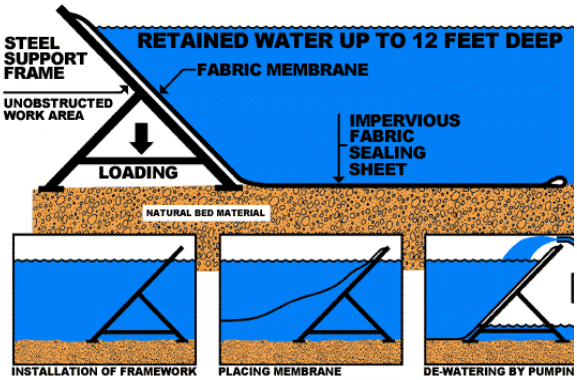


Fig. 5.20: Pallet Barrier (cross section)

French Bulkhead System:
(for small water head (< 3m))
- Vertical H shape column,
- Wood stoplogs (2.5 m span)

Fig. 5.21: Beam Bulkhead System

Inflatable Bulkhead (Ivoz-Ramet):
See additional data in Project Review (L3) on the CD (Directory A1).

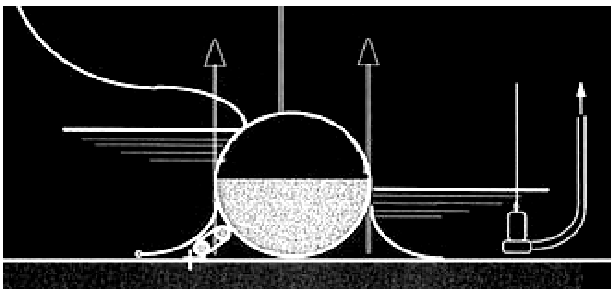


Fig. 5.22: Inflatable Bulkhead
(with the Courtesy of Mr. Dermiance , MET, Belgium)

Rotating walking-bridge used as bulkhead (France):

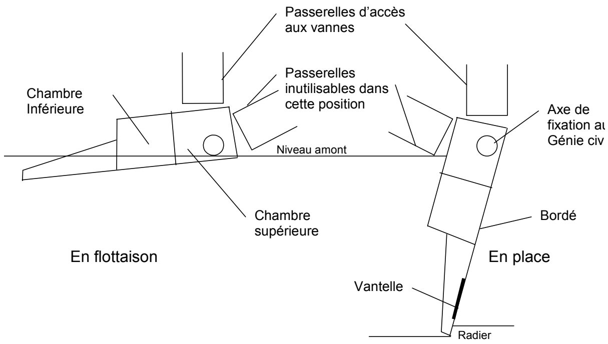


Fig. 5.23: Bulkhead articulated on a walking bridge
(with the Courtesy of VNF and ISM, France)

Documents that constitute an extensive survey of the different types of maintenance bulkheads are available on the CD's Directory /Annex Section 5.5/. This Annex includes a comparison analysis (*pro and con*).

5.5.3 STANDARDISATION

For several reasons (economical, manufacturing, maintenance, etc.) it is very important to standardise such temporary closure structures. Here after are some examples of standardisation.

5.5.3.1 In England, British Waterways is trying to standardise the size of stop logs for their locks to achieve some economies. This is being achieved with locks that generally use wooden or steel planks stacked sequentially across the canal and supported in vertical slots in the canal wall. One set of planks can be stored locally to be used at any of a group of locks having the same dimensions, rather than having numerous sets of planks with slightly different measurements.

At Lagan, Northern Ireland, the stoplogs are standardised from other sites and the gate openings were made to fit these existing stoplogs, rather than the other way round.

An example of a flood defence gate from the United States of America is interesting: a large floating caisson has been designed to provide a standardised sealing arrangement for a number of locks on the Tennessee and Cumberland Rivers around Nashville, Tennessee. Although expensive in its own right, the versatility it provides and the removal of expensive craneage, makes it cost effective.

For flood defence at Bayou Dularge and Bayou Lafourche around Lockport, Lousiana, swinging gates have been designed to seal the river against tidal intrusions (See Project Reviews). These gates are normally held back against the river/canal bank and winched out in advance of a flood event. Once in place they are ballasted to sit onto the sill. The tension of the winch ropes and the hydrostatic head complete the seal. These operations are entirely manual using electric pumps and hydraulic winches controlled by the attendant crew called out in response to a flood warning. These types of gates can only be moved in low flows or with minimal differential head.

5.5.3.2 An interesting conception is used in the Tees Barrage project (UK) that is now owned by British Waterways although it was originally built and operated by the Teesside Development Corporation. It has four flap gates controlling the level in the river, a navigation lock on the south side and canoe slalom on the North. To allow dewatering for maintenance of the gates or the lock there are a variety of stop logs supplied under the contract. They are unique within British Waterways and deserve special mention.

There are four gates regulating flow down the river. Each gate is housed between piers with grooves to accept the stoplogs. The grooves have stainless seal faces built into the concrete to improve the effectiveness of the sealing arrangement. There are 13 stoplogs (8 logs needed for the downstream and 5 for the upstream), each 13.888 m wide, 1.250 m deep and weighing 7,018 kg. A special frame is used for handling the stoplogs. It weighs 3,124 kg itself and has a safe working load of 12,500 kg. This frame is left on the crane during the lifting sequence and is 'latched' onto the stop log for the lift. It then automatically disconnects itself when its landing sensor makes contact with the ground or the previously lowered stop log. This is to ensure that the lifting slings do not become detached if the stop log should jam before reaching the fully lowered position. For the operation of the stoplogs and the frames it is required to use a 120T mobile crane situated on the road bridge. The stop logs are stored across the piers, in three groups of three on the downstream side and four singles on the upstream. For more details see Tees Barrier Project review on the CD.

5.5.4 RESPONSE TIMES

A further consideration is the speed of response if a catastrophic failure occurs and a breach or gateway needs to be sealed off. Off site storage of closure devices will increase the time it takes to transport the equipment to the site in an emergency.

Schemes of Operation by regulatory bodies may impose a response time that determines where the devices are kept in order to ensure compliance.
At the Tees Barrage the storage location is not ideal from an aesthetic point of view but tolerated for reasons of availability. Off site storage would also be an additional cost over the lifetime of the Barrage.

5.5.5 MAINTENANCE

Maintenance of stoplogs consists of touching up the landing area where the seals rub the paint away and repairing any damaged seal sections as and when necessary. This can be scheduled to precede annual gate maintenance.

Caissons and cofferdams require internal inspection of ballast tanks as well as functional checks on valves and pumping arrangements.

Air Bags will require pressure testing on a regular basis.

5.5.6 SUMMARY

Although maintenance is covered elsewhere in this report, it must be considered as part of the overall design package. A structure will only meet its design life if properly maintained, so provision must be made right from the start. The ability to access the moving parts in a safe manner depends on the design of the temporary closure devices.

Risk is defined as a function of probability and consequences of a failure. If the consequences of a failure can be reduced by the availability of temporary closure devices then the risk is equally reduced.

For both these reasons temporary closure devices must be considered as vital in their own right.

5.6 SAFETY, RELIABILITY AND RISKS

5.6.1 USE OF RELIABILITY AND RISK

5.6.1.1 Definitions

Reliability analysis (REA) means the investigation towards the probability that a structure or part of a structure (existing or to be designed) will not fulfil its task.

Increasing complexity of structures and their equipment (machinery, electrical/electronical systems and so on) have increased dangers to society and the environment and have increased the importance of reliability as a quality characteristic.

The determination of the probability of function loss, or probability of failure, is important, because the probability of failure has to remain between economical and legal restraints.

Very generally, reliability is defined as the probability that an item will perform a required function:

- Under specified conditions,
- For a specified period of time.

Reliability, as the characteristic of a structure or a structural element, is expressed as a probability, which includes three independent concepts:

- Time,
- Spatial factors (such as operating, maintenance, and environmental conditions),
- Rules for determining whether or not the structure or part of a structure performs as specified (definition of failure).

Reliability of a structure (or also of a product) can be defined as a function of time, because time is the only factor that changes for every device.

Risk

A lot of discussion is still going on in the scientific world upon the use and definition of risk. An overview can be found in Vlek (1996). Informal definitions of risk, such as "a set of possible negative consequences" or "lack of perceived controllability" which all are an expression of uncertainty, also exist but will not be used in this text.

The following definition of risk is frequently used in the engineering community because of its ability to quantify the risk:

> *"Risk is the measure of the probability and severity of an adverse effect to life, health, property, or the environment (an adverse impact). The scale or significance of risk is described by a combination of probability of failure (reliability) and consequences of a particular outcome or set of outcomes. Probability and consequences can be multiplied together to assess the size of a risk. "*

An example shows the shortcomings of this definition: a 0.5 probability (or 50% chance) of incurring a loss of 1000 EUR may be considered similar, in risk terms, to a 0.01 probability (or 1% chance) of a loss of 50000 EUR. Both have mean or mathematical expectation values of 500 EUR within the time period. Despite their similar risk values, attitudes to and management of these risks may differ because of their very different scales of loss, should they arise. Therefore, for the complete assessment of risk, it may be necessary to take into account the component probabilities and consequences.

A **Risk-Analysis** (RIA) links the different possibilities of failures and the current probability of their appearing with the consequences belonging to them.

A reliability analysis is a part of a risk analysis. In a risk analysis, not only the probability of failure is determined, but also the material and immaterial consequences. Since damage is also related to local circumstances (is there industry and/or housing downstream the failing gates, or agricultural land, or natural areas…?), it is difficult to generalize the results of a risk analysis.

5.6.1.2 Why use risk based methods

Risk based methods are used more and more frequently employed for the design of flood protection schemes.

Some reasons for it are (ICOLD Question 76, 2000):

- The fact that risk analysis allows evaluating margins of safety more realistically than traditional (deterministic) safety criteria,
- The possibility to achieve economic benefits from risk based assessments,
- To provide a common approach for comparing a wide variety of options and enable the risks due to flood defence to be compared with the risks due to other natural and man-induced hazards,
- Judgment of evolution of safety considering the changing climatic conditions,
- Public desire to quantify the risk of catastrophic events with and without protection measures. For example, risk-based approaches consider not just the likelihood of high water levels against a defence (barriers, dikes, etc.) but also the likelihood of defence failure and the degree of harm resulting to people/property, etc. behind the defences.

Whenever uncertainty is a governing factor in the conceptualization of safety and for the evaluation of cost decisions, risk analysis is a reasonable approach.

5.6.1.3 Fields of application

A) Structural design of a weir or storm barrier

The general aim of designing a structure, against all limit states that have to be considered, is to ensure (with an acceptable level of probability) that its performance is satisfactory during its erection and entire design working life, such that the structure is unlikely to fail by any real loading events or require repair of damages caused by such events.

A reliability analysis (REA) can be done for constructive failure or excessive deformation, but also related to failure of processes, management and maintenance systems, and of quality assurance.

The reasons to perform a reliability assessment of a structure can be various:

- Identifying the weak parts in a construction or process, in order to improve the design;
- Tailoring the solution, when standard engineering proves to be too expensive. Generally the safety, reliability and risk assessments should be considered on the understanding that design, calculation, manufacturing, operating and maintenance (servicing/up-keep - inspection - repair/reconditioning) of the various structures must be done according to national or international technical rules, standards, guide lines ... and in conformity with them. However, applying these rules on complex structures may lead to high costs. It is also possible that design rules are not adapted enough to the specific construction, this makes it difficult to judge if the so designed structure would satisfy the imposed reliability criteria;
- If tailoring has been done because standard solutions do not satisfy local (environmental) interests;
- In order to define priorities for allocation of people or means. By knowing the probability of failure, objective decision making is possible;
- To establish the relationship between "design safety", "working life", and "maintenance". Using a REA, how much a probability of failure is influenced by a chosen maintenance strategy can be determined (see Fig. 5.24).

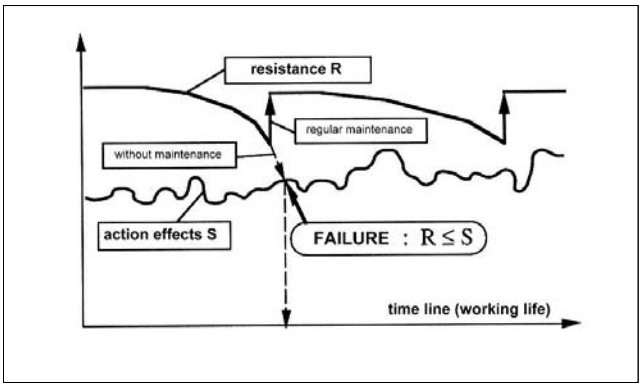


Fig. 5.24: Relationship between "safety", "working life" and "maintenance".

In certain situations it is less evident to perform a reliability analysis:

- With standard constructions or parts of construction, it is often more attractive to follow the already established design rules. Because of repetition, design efforts will be minimal but also the risk of problems during maintenance will be minimal. In these cases it is often not worthwhile to consider each construction or construction element as unique and perform a special analysis and design for it.
- When the techniques and specialist knowledge necessary for a detailed REA are not available, it is better to rely upon traditional design procedures.

The **International Standard ISO/FDIS 2394** "*General principles on reliability of structures*" constitutes a common basis for defining design rules relevant to the construction and use of a wide majority of buildings and civil engineering works, whatever the nature or combination of the materials used. It specifies "general principles" for the verification of the reliability of structures subjected to known or foreseeable actions. Reliability is considered in relation to the performance of the structure throughout its design working life.

Nevertheless, it is also important to consider that rules and codes cannot be a dogma. For instance, the actual "*state of art*" must always be considered and it is also required to evaluate the experiences (and knowledge) that were gained and collected during all phases of design, manufacturing and operation. In addition, it is necessary to take into account new results found in the field of research and development. In this context, the corresponding proceedings may be handled very differently in different countries.

B) Decision making: design and comparison of alternative flood protection schemes

Different types of risk can be considered in hydraulic design of storm surge barriers. These can be categorized as follows (Mockett et al. 2002 and DEFRA 2000): engineering, financial, economic, insurance, construction, operation, environment and heritage, health and safety, political, and societal risks.

Part of these risks cover the project specific risks (financing of the structure, risks during construction, risks to be covered by insurance premiums...), whereas other risks are more related to the choice of the flood protection scheme: how will risk caused by flooding diminish (and is probability of failure of the structure low enough to guarantee this reduced flooding risk?). Is there any impact on environment or on human activity? What is risk reduction by monitoring and pre-warning, etc.?

In this concept, the investments (construction cost and maintenance costs, summarized as "the average predicted costs") are compared to the reduction of the presumed risks in the protection area of the flood defense structure (summarized as the "average predicted benefits" for society). These studies also include the societal risk (however giving a monetary value to human life) and the effects of warning systems.

Alternative design options for flood defense projects can thus be compared on an economical basis. The decision maker should normally choose the option, which maximizes expected net benefit (or which has the shortest pay-back period). In designing flood defense schemes, the concept of "equal risk" for all areas along a river can be used. This results in a lower standard of protection for areas with a low monetary value for flood damage (which finally results in different design return periods for flood protection of cities, agricultural lands, ... as it is already the case in Great Britain, see MAFF guidance quoted in Environment Agency (2000).

C) Guidelines and legislation: definition of flood protection standards.

Some countries are using the concept of risk to redefine their policy towards safety and flood protection.

Flood protection standards have mostly been defined in terms of probability of flooding. In the Netherlands for example, a standard probability of flooding of once in ten thousand years is maintained for the central part of Holland, and a return period of once in four thousand years for the Western Scheldt area.

This legal restraint implicitly takes into account the consequences of failure. When high damage is expected, a low probability of overtopping of dike crests has legally been imposed. However, this restraint does not take into account that the resulting risk of flooding is obtained by the sum of the individual risks of each part of a long defense. When "uncommon" or high-probability primary failures exist on one of these composing elements, it will result in a unexpectedly high risk for the area to be protected (ICOLD 2000).

For this reason, investigations are going on in various countries, trying to redefine the flood protection standards from a concept of probability of overtopping into a concept of risk, where all the elements in the flood defense scheme are submitted to a REA or RIA.

5.6.2 RELIABILITY/RISK ASSESSMENT TOOLS AND TECHNIQUES

5.6.2.1 Risk assessment techniques

All previously mentioned risks can be assessed using various techniques. An overview of existing techniques is given in Mockett et al. (2002) and DEFRA (2000), where more details can be found.

They can be summarized as follows:

A) Broad brush qualitative method (establishing risk registers).

These are used to help in hazard identification, to record information about risks, and to document decisions taken. The structure of a risk register reflects the overall risk assessment and management process. Establishing a risk register implies following steps:

- Risk identification (screening),
- Probability assessment, e.g. by a panel of experts, or by expert elicitation,
- Evaluation of consequences, using monetary values where available,
- Mitigation and evaluating residual risks.

When risks of different nature have to be combined to facilitate a decision, use can be made of a multi-criteria analysis (see specific chapter in this report).

B) Quantitative and qualitative risk assessment:

Various methods are documented in the literature to perform a risk assessment (DEFRA 2000, Mockett 2002). Some important methods are:

- Event trees and fault trees,
- Analytical methods,
- Monte Carlo modelling.

Event trees and fault trees are a primary tool for understanding the components of a problem and combining probabilities in a logical manner. Event trees show the range of likely consequences (i.e. flooding/no flooding) that may arise from a given initiating event (storm surge). Fault trees work backwards from the consequence (flooding) to determine a range of possible initiating events (failure of a mechanism, storm surge, etc.).

Fault tree analysis is particularly useful for analysis of mechanical and electrical systems such as floodgates. An example is given further on in this text. It enables critical elements in the system to be identified and if necessary reinforced or duplicated in order to improve the reliability of the whole system.

Quantitative modelling of risks is also performed by modelling the system. In a deterministic model a unique set of input parameters is used. In a probabilistic approach, a full range of input values is tested in the system model, each one weighted by the probability of encountering it. Therefore, this method starts with obtaining input probability distributions for the boundary conditions of the model (for example: the distributions of water levels and accompanying storm wind velocity).

The Monte Carlo method is based upon a random sampling in the input distribution, followed by a calculation of the system response. By carrying out a large number of simulations of the system response based on this random sampling, an output distribution and statistical insight of reliability of the structure is obtained, by counting the number of failures in the long series of simulations.

In the analytical methods, the failure rate is not obtained by performing a large number of simulations (as Monte Carlo) but by integrating a pre-established analytical function (representing the response of the system upon the imposed boundary conditions) in a narrow domain containing the failure situations.

The quantitative risk assessment is often followed by a sensitivity testing. This method is used to identify by how much key variables can change before a different preferred option is identified. Sensitivity testing usually involves varying each parameter in turn with other parameters at their "best estimate" value. Because of the uncertainties, sensitivity tests will usually give a rather uncertain indication of the robustness of a preferred option, often resulting in further probabilistic analysis.

5.6.2.2 Flood impact assessment

To make the step from reliability analysis to risk analysis, an important effort has to be done to evaluate the consequences of failure i.e. the damage and loss of life.

In a first approach, rough risk contour maps can be constructed, using digital terrain models (DTM) and GIS soil use maps and assuming horizontal inundation levels. These maps are mostly used to define areas to be protected (high potential damage areas) or inversely to define the necessary reliability level of the storm surge barrier or weir.

In quantitative risk assessment, used for scheme optimisation and selection, it is important to use more detailed damage assessment techniques.

For barriers, failure generally means “gate not closed”, which then leads to high water levels and eventually to dike overtopping and dike overflow combined with dike failure (resulting in breaches in the dike).

Volumes of water in the inundated areas are generally estimated using 1D hydrodynamic models connected to DTM models and GIS maps of the flood prone areas.

Often the considered failure system of dikes along rivers, is limited to overflow combined with dike breaches. The breach mechanisms are sometimes simulated by time series, but more often by erosion formulas taking into account some physical factors governing the breach development. At the time being, breach mechanisms are mostly taken into account using erosion formulas, but determining the start of breaches and the erosion parameters remains subjective (lack of geotechnical information, lack of observations to allow model calibration, etc.). Often pragmatic choices are made, and afterwards a sensitivity analysis on the results is performed.

Research on this theme is still going on (see for example www.delftcluster.nl and www.floodrisknet.org.uk).

From the previous mentioned hydraulic calculations, flood inundation maps presenting water depths, water rise velocity and horizontal velocity are produced.

From this information, combined with landcover maps (such as Corinne satellite information), flood damage is calculated using damage functions, which are chosen as a function of soil use in the inundated area. A literature study on available methods can be found on the Delftcluster website (previously mentioned). Research is on going to refine damage calculations for buildings, for pollution, for economical (indirect) losses, to evaluate loss of human life (taking into account warning systems and evacuation procedures), etc.

5.6.3 APPLICATION OF RISK ASSESSMENT: EXAMPLE OF DECISION MAKING

5.6.3.1 A flood protection scheme along the Scheldt in Flanders

In 1978, a flood protection scheme was conceived in Flanders (Belgium), departing from the premise that a general protection against flooding of 1/10000 year had to be guaranteed.

This solution implied the construction of a storm surge barrier on the river Scheldt. However, this barrier was never constructed, because a benefit-cost analysis (performed in 1982) demonstrated that the project was not economically viable.

Aware of the dangers of a possible sea level rise, the Flemish government launched a new design study in 2001 for a flood protection scheme, imposing the risk approach for determining the necessary degree of protection against flooding (AWZ 2004).

In the design study, all possible methods of protection against flooding were considered: storm surge barrier, dike heightening, flood storage areas, and combinations thereof. In a first approach, different schemes were designed using different levels of probability of overtopping as a design criterion. For storm surge barrier solutions, design for 1/10000 events was assumed with probability of failure on closure of 1/100.

Flood maps were calculated for 11 different return periods, considering overtopping of dikes and failure by breach formation in the overtopping dikes and in dikes where freeboard was not respected.

Damage was calculated using these flood maps, and translating flood depths, combined with land use into damage estimations using damage functions (giving for each type of damage, the damage in function of inundation depth). Integrating damage in function of probability of occurrence, results in average annual risk during the project lifetime. Each flood protection scheme results in a reduction of the average annual risk when compared to a do-nothing scenario. This reduction of risk can be considered as a benefit for the project, and is used to perform a benefit/cost analysis and to compare and optimise the various possible flood protection schemes. Therefore, the expected average annual risk reductions (benefits) are discounted to the present, to obtain (combined with the discounted investment and maintenance costs) economic figures such as net present value and a payback time, which can be used to compare alternative projects.

Study results demonstrated:

- That a 1/10000 protection is not economically justified,
- That, in the case of the Belgian part of the river Scheldt, flood protection schemes using dike heightening and strengthening, and flood storage areas, are more economical than building a storm surge barrier.

In a following step, risk analysis was used to define, in each sector of the Scheldt basin, the optimal local flood defense scheme: different combinations of dike heightening and flood storage areas were compared on economical basis, and each time the optimal solution was selected. The final combined total flood defense scheme presented a higher net benefit (this is lower residual risk) and a shorter pay-back period than a scheme established using the conventional uniform "probability of overtopping" design criterion. Resulting probabilities of overtopping ranged between 1/4000 and 1/1000.

5.6.4 APPLICATION OF RISK ASSESSMENT: FAILURE TREE FOR STORM SURGE BARRIER

The below methodology is proposed by TAW (1997) as an approach for risk analysis for storm surge barriers.

5.6.4.1 Definition of failure

In the REA of storm surge barriers, safety against flooding is the central point. Therefore, failure can be defined as "not fulfill anymore the function of retaining the high water levels".

5.6.4.2 Failure mechanisms

The state of failure can be reached in various ways, called "failure mechanisms".

For a surge barrier, main failure mechanisms are:
- Overflow or overtopping by waves
- Loss of stability or loss of strength
- Failure of the closure operation of the gates.

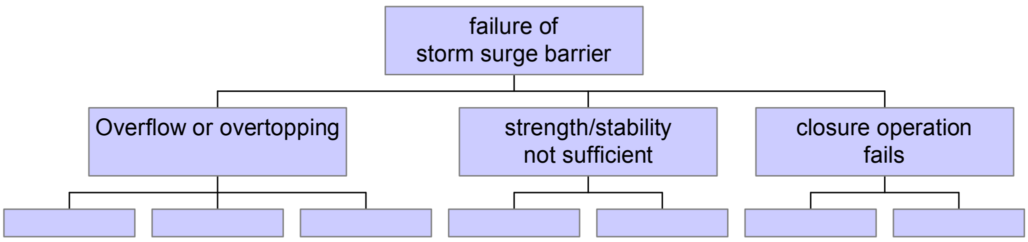


Fig. 5.25: Failure Mechanisms tree of a storm surge barrier

5.6.4.3 Fault and event trees

The ways in which failure can be reached, can be shown systematically in a fault tree. The top event is failure. In the branches of the tree, it is shown which chain of events (from bottom up) can give rise to the top event. In this way, insight is created in sometimes very complex systems.

The relationship between the elements in the fault tree has to be such that they can provoke the "higher" situated event. The top event has to be a clearly defined event and can only be one state of failure. When constructing a fault tree, it is important to consider systematically all parts of the structure, and to take into account effects of order of appearance of the events and effects of time. Therefore, it is advisable to construct first event trees, permitting to analyse chained events.

5.6.4.4 Methods of calculating reliability

A fault tree analysis consists of a qualitative and a quantitative part. The qualitative part analyses how the structure can fail. In the quantitative part, each event is given a probability of occurrence, and the probability of the top event is calculated.

For quantitative analysis, two approaches are possible:
- Bottom-up: the probability of failure of each element is determined, next it is verified if the top event satisfies the imposed reliability criteria,
- Top-down: an allowable failure rate of the top event is fixed. On the basis of maintenance reasons, the allowable failure rate of the components and mechanisms is fixed. Next, the design is made and it is verified if the allowable failure rate of the top event is satisfied. If not, the design is adapted.

The top-down approach is mostly used in hydraulic engineering.

When calculating probabilities of failure, mutual dependency and succession of failure mechanisms is important.

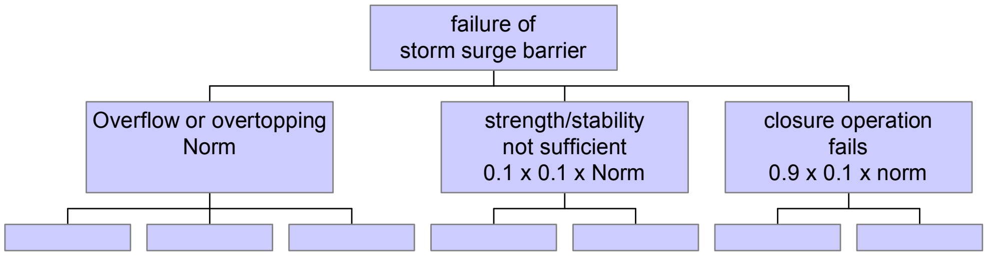


Fig. 5.26: Probability of failure of the Fault and event tree of a storm surge barrier

5.6.4.4 Quantifying the fault tree

In the following example, it is shown how the fault tree analysis is performed in the Netherlands (TAW 1997). The approach is not through risk, but through a reliability approach. The probability of overflow and overtopping is fixed by law:

$$P(q>qt) < \text{normfrequency}$$

with:

P = probability of exceedance,
q = overflow discharge;
qt = allowed overflow discharge (not generating damage);
norm = frequency fixed by law.

The second criterion is that the probability of failure of all other failure mechanisms together has to be very low (translated as 10% of the probability of exceedance allowed by law).

Because of the fact that the closure operation in general is more difficult to satisfy in terms of failure probability, whereas the failure space for loss of stability can easily be restricted, the Dutch attribute a failure space of 1% of normfrequency to the stability criterion (Fig. 5.27).

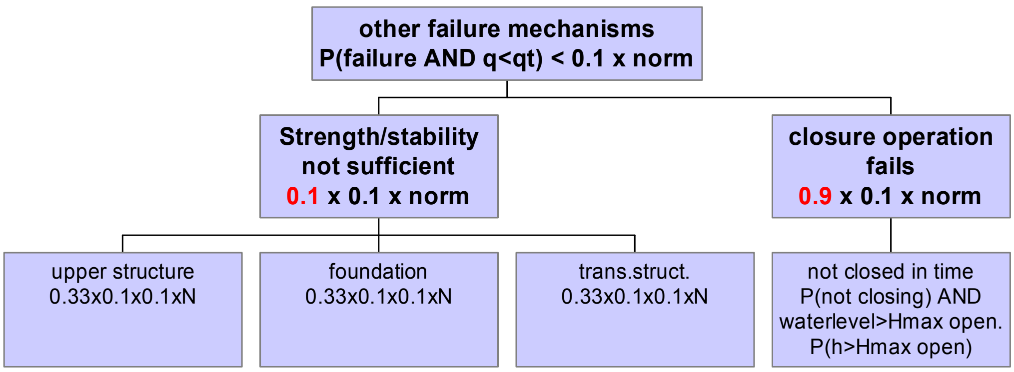


Fig. 5.27: Quantified Fault Tree of a storm surge barrier

note: Hmax open = maximum allowable water level with open barrier which does not cause flooding
N = normfrequency for admitted failure of whole structure

5.6.4.5 Reliability related to stability/Strength

In the picture above, a possible distribution of the 1% failure space over the submechanisms of stability failure of the barrier is shown.
The procedure to verify if this reliability criterion is satisfied is as follows:

- Determine design conditions,
- Determine properties of construction and (part) mechanisms,
- Determine limit states,
- Construct fault trees,
- Determine target values for failures and verify if target values are reached for all limit states using a (semi-) probabilistic method.

5.6.4.6 Reliability related to closure operation

In the above approach a larger failure space is attributed to failure due to impossibility to close (all or part of) the gates. The attributed failure space is about 9% of the normfrequency.

The measures to be taken to satisfy this criterion will be far more dependent on gate type than the stability criterion.
In general, the closure of the barrier gates can fail due to human and to technical errors.

The probability of human errors can be reduced by:

- Making or improving procedures and agreements,
- Improving the man/machine relationship,
- Make the human errors detectable and correctible,
- Training of the human actions to be performed to close the barrier,
- Control the actions through technical systems.

Technical failure of the closure operation can be due to mechanical, electrical, or electronical failures. These systems can fail because:

- They are not operational at the moment of closure: the "average" failure rates of the components depend on the maintenance plan. The unforeseen non-availability is harder to estimate,
- Because they fail at the start of the closure: by means of a system description and a data analysis, it must be determined which elements have a probability of failure at the start-up. The influence upon general failure is detected through a fault tree,
- Or, because they fail during closure: same as above. It also has to be found out if reparation during closure is still possible.

In general, the technical analysis of non-closure can be performed in various ways. Theoretically, a detailed analysis can be made, by means of fault trees, down to each element of the mechanical, electrical of electronic system. Databases containing tables with the failure rate of different components exist in the industry, and permit calculation of the global failure rate.

In practice, general impression on existing barriers is that these failure rates are valid under controlled circumstances, but are not easily applicable in practice for the evaluation of failure rate in less controlled circumstances such as a with a storm surge barrier.

5.6.5 APPLICATION OF RISK ASSESSMENT: SIMPLIFIED RISK DESIGN FOR WEIRS

British Waterways proposes following a simplified approach for categorizing risks of hydraulic structures.

A number from 1 to 5 is used to quantify the potential consequences if the asset were to fail. Table 5.6 gives a general guide as to the Consequence of Failure Category to be applied. In general these definitions are rather simplistic and do not take into account the variations in potential risk associated with different asset types.

Table 5.6 : General Guide of the Consequence of Failure Category

Category	**Personal**	**Neighbours**	**Affected Property Values**
5	Multiple Deaths	Widespread Urban Flooding (>0.5 km^2)	In excess of £5m
4	Multiple Serious Injuries Single Death	Flooding of small community	£2m to £5m
3	Serious Injury (1 to 2 victims)	Disruption of a major transport link. Widespread flooding of agricultural land (>0.5 km^2)	£250k to £2m
2	Minor Injuries	Limited flooding to gardens Limited flooding to agricultural land (<0.5 km^2) Disruption of a minor transport link	£25k to £250k
1	Single Minor Injury	Seepage to gardens/agricultural land No Consequences	£1k to £25k

Next, Tables 5.7 to 5.10 provide additional guidance and propose "*Consequence of Failure Grades*" to each of about 20 asset types (only a few are given hereafter).

Table 5.7: Consequence of Failure Grades for Canal Sluices and Canal Weirs

<table>
<tr><td rowspan="7">Canal Sluices and Canal Weirs</td><td colspan="2">The purpose of these structures is to dispose of storm water to prevent the canal overtopping and breaching. Should a breach occur sluices and canal weirs can be used to mitigate the effect of flooding.
Consider the effect of the collapse of the structure itself and the consequences if it failed to function as designed. The canal sluice collapse is unlikely to lead to high consequences, unlike the canal weirs.</td></tr>
<tr><td>5</td><td>Not Applicable</td></tr>
<tr><td>4</td><td>Flooding of urban areas</td></tr>
<tr><td>3</td><td>Flooding of villages</td></tr>
<tr><td>2</td><td>Flooding of main roads and railways</td></tr>
<tr><td>1</td><td>Rural flooding and flooding of minor roads</td></tr>
<tr><td colspan="2">When allocating grades consider:
- Length of pound – volume of water which could escape,
- Numbers of sluices and weirs in the pound – the relevance of the individual structure,
- Crest lengths,
- Location within the pound compared with inflows, embankments, urban areas, etc.
- Local knowledge and operational experience – e.g: is the sluice regularly used?
- Take a pragmatic view</td></tr>
</table>

Table 5.8: Consequence of Failure Grades for Stop Gates and Safety Gates

Stop Gates and Safety Gates	In the event of breach stop gates and safety gates can be used to reduce loss of water and consequent damage by reducing the length of long pounds. Some canals have gates at each end of an embankment. Mitre gates are often inoperable due to siltation. Consider each case on its merits. A valid approach is to assign a grade one category below that given to the embankment (dikes) that is protected. Some river navigations, eg Calder & Hebble, have flood gates at the upper end of lock cuts. These are similar to flood locks but prevent navigation when the gates are closed. The effect of failure leading to overtopping of the lock cut and consequential flooding should be considered.

Table 5.9: Consequence of Failure Grades for River Weirs

<table>
<tr><td rowspan="5">River Weirs</td><td colspan="2">The aspects to be considered are the proximity of the navigation at the head of the weir, the likelihood of a catastrophic collapse, the effect of rapid draw down on property upstream, the possibility of flooding downstream if the river has a low freeboard and the possibility of scour.</td></tr>
<tr><td>4, 5</td><td>Not applicable</td></tr>
<tr><td>3</td><td>Navigation in close proximity to the head of the weir</td></tr>
<tr><td>2</td><td>Those weirs where rapid draw down flooding and scour in urban areas might occur</td></tr>
<tr><td>1</td><td>Most weirs</td></tr>
</table>

Table 5.10: Consequence of Failure Grades for Locks

Locks (having also a role of water regulation)	*It has been suggested that in a large ship lock should the wall be in such a state that a heavy vessel could cause masonry to fall on a small boat, the consequences of failure are greater than 1. On the majority of canals there are no such cases.* *In the case of river locks, where it is conceivable that a barge could demolish a set of bottom gates, the consequences to third parties would be slight because waiting craft would generally be using lock landings.* Cases where lock failure could lead to other than minimal consequences are: - Flood locks where a lock cut passing through an urban area is maintained below the flood level of a river. - Locks where a collapse could lead to breach of the canal or affect adjacent buildings. These cases should be considered on their merits. Consider also the length of pound above the lock and whether weirs below could cope with the water which might be released	
	5, 4, 3	Flood locks based on individual cases
	2	Cottages affected
	1	Most Locks

5.7 ENVIRONMENTAL IMPACTS AND AESTHETICS

5.7.1 ENVIRONMENTAL IMPACTS

It is recommended that clients, designers and planning authorities are mindful of the "*whole life cycle*" impact of their projects – it would be unfortunate if a chosen design was resource effective at the building stage, but proved resource intensive during operation and posed major wastage and impact at decommissioning.

Similarly, it is important to consider the "*whole environmental footprint*" of the project and not just factors relevant to the site of construction and operation. For example, avoid specifying timber or stone from sources which are not sustainably managed and/or require transport over large distances; instead, use more innovation in the specification and seek out managed, local sources of materials.

As with so many designs and conceptual processes, recognise that achieving high standards of environmental acceptability is an iterative process – allow one good idea to lead into another.

Environmental headings that must be considered include:

- Storage and handling of all materials;
- Construction materials;
- Materials, resources and energy required to operate;
- Impacts, particularly waste streams at times of major overhaul, e.g. removal and surface preparation from old paint, especially over water.

Some of the UK standards such as the Institution of Civil Engineers CEEQUAL Standards, BREEAM standards for buildings and the Environment Agency's own Environmental Audit provide much useful guidance (http://www.ceequal.com/ and http://www.bre.co.uk/).

Inspired environmental design will also consider the impact of the installation in its locality. Factors, which should be considered, include:

- The scale of disruption to natural tidal regimes or fluvial flows. A major impact is likely to be bad, a smaller impact is likely to be the optimum (incursion of less than 15% of the natural cross section is often seen as a reasonable objective). Consider modelling the dynamic effect of the new structure.
- The physical disturbance to humans, migratory fish, birds and other ecosystems consider the effects of noise, light or chemical pollution. The client or planning authority may legitimately choose to seek environmental gain out of the project – new or improved facilities, larger and more diverse areas of habitat.

5.7.2 AESTHETICS

By its very nature, aesthetics is very subjective. Perhaps any system of classification could be under three broad headings:

- Poor or negative impact,
- Average or acceptable,
- Good or with added value.

For any major structure, we would recommend that an artistic impression should be commissioned to create a "vision" of the possible options. These artistic impressions will have many purposes including:

- Evaluation of options and optimising the preferred solution,
- Satisfying the expectations of client, stakeholders or the planning authority,
- Used as a visual and conceptual guide for the design team.

It is often wise to include structural and landscape architects as part of the design team.

Installations in urban sites or sites visited by a large number of people for recreation (sailing, walking, cycling, bird watching, etc.) may warrant closer attention to aesthetics than installations rarely seen by others.

Some examples of installations, which may fall into the good or added value category, are:

A) Thames Barrier, London

Detailed technical information is available on the CD (see Project Review, Directory A1 and Directory /B7-Flood Protection in UK/).

Key points of the integration of the Thames barrier are:

- The absence of overhead machinery, which is beneficial for minimising the restrictions on shipping also means that the structure is low and unobtrusive from distant views,
- The shape of the machinery roofs echoes maritime profiles, e.g. upturned boats and sails, helping the structure blend with its setting,
- The roofs are clad in stainless steel, which acts like a mirror reflecting the colours of the sky and the water. The bulk of this massive structure is minimised as it changes to mimic "the moods of ambient light and the weather".

B) The New Waterway, Rotterdam

Detailed technical information's are available on the CD (, Directory /A1-Project Review …/).

Main integration issues are:

- The absence of overhead machinery to minimise the restrictions on shipping also means that the operating structure can be laid out on a low horizontal axis, which minimises intrusion from distant views,
- The structure and support buildings are sand coloured to blend with the surrounding estuarine environment,
- Close up, the structural form of the barrier is apparent which, for many, creates a "wow and excitement factor" at the scale of the engineering.

5.7.3 REMARKS

Designers are usually reluctant to highlight any individual structures, which fall into the poor or negative impact category, but there are several examples that come to mind. Sites where the bulk of the structure is oppressive, where the colour, texture and profile of the structure is out of place with their surroundings. Frequently such structures are industrial and merely functional in form.

5.8 COST (Construction, Maintenance and Operation)

Global cost for construction of a navigation weir is related to the site's physical constraints (geology, hydraulics, sediments science, aesthetics, etc.) and to the adopted weir type (flap gates, sills, etc.). Fig. 5.28 shows the different steps of a weir project including Conception, Design, Construction, and Operation and Maintenance.

But to obtain a real estimation, the operation and the maintenance cost should also be taken into account, these costs depend on the expected safety level. That is what is called "**global cost".**

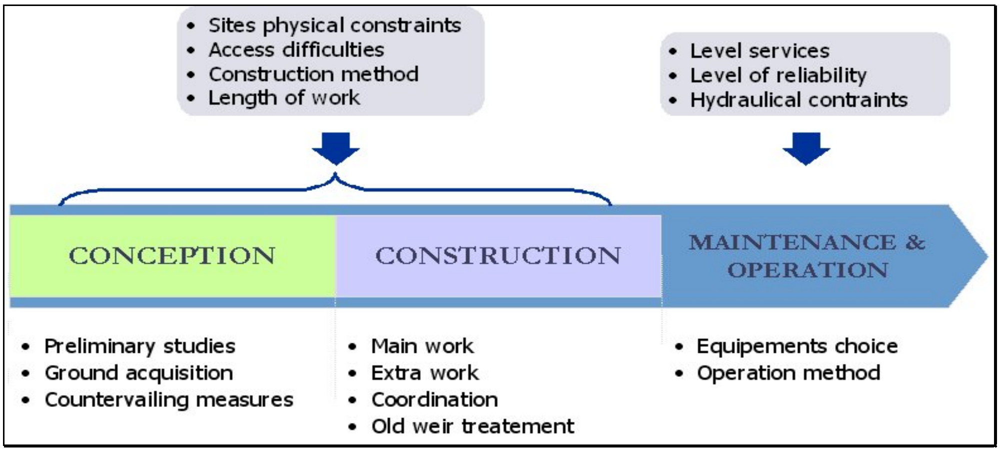


Fig. 5.28: Steps of a weir project (Conception-Design, Construction, and Operation and Maintenance)

5.8.1 GLOBAL COST

According to the hydraulic head, the site characteristics (rural or urban zone) and the weir type, the construction cost of a navigation weir in France is estimated to be between 35 k€ and 200k€ (taxes included) per linear meter. As a reference *(Source: SN Seine District)*, weirs recently built with flap gates of 17 m x 3.50 m and operated by an electric gear motor and chains cost 60,979.61 € per linear meter (1992 value). Fig. 5.29 shows a comparison of relative unit costs for three French small size river navigation weirs: Le Vezoult (on the Seine river): 3 spans with flap gate, new navigation weir in a new site, Villevallier (on the river Yonne): 2 spans with flap gate, weir built upstream of the previous one (used as support) and the Roanne weir, which is detailed on Section 5.8.2 and presented Fig. 5.30.

Fig. 5.29 shows a rather homogeneous distribution between all kinds of works. Therefore, savings can deal with all aspects/topics: construction procedure, optimisation of the civil engineering works, operating devices, downstream protection, gates, …

A study (Daly, 1995) on the weir building costs was based upon 10 French weirs. It gives an empirical formula to define the order of magnitude of the cost. This formula was used in France to estimate the magnitude of cost on a weir reconstruction program (not valid for barrier or other types of structures).

$$\frac{P}{L} = K\left(A.h + B.h^2\right) \qquad (5.2)$$

with :

- P is the weir cost excluding approach works (in Euros, without taxes, 1994 value),
- L is the total length of the weir, between abutments, in meters,
- h is the maximum upstream water depth (above the sill), in meters,
- K is a coefficient ranging from 0.6 to 1.4 that takes into consideration the auxiliary works (compensatory measures, access track, dry connection with river banks, etc.) and the site characteristics (regional differential pricing, etc.). K partially accounts for the life cycle duration and the factor of safety to be used in design. This coefficient includes an uncertainty of 40% on the assessment,
- A and B are empirical constants with A = 6.070€ and B = 1.821€. These parameters cannot be considered as universal values and must be validated by each potential user.

Nature of Expenses (%)	Sites		
	Le Vezoult	**Villevallier**	**Roanne**
A- MAIN WORKS	**76**	**80**	**86**
A1- Civil work	46	52	47
Installation	*4*	*10*	*9*
Earth works	*22*	*20*	*26*
Structure	*20*	*22*	*12*
A2- Equipements	30	28	39
Gates	*18*	*15*	*25*
Mechanism	*12*	*13*	*14*
B- ADDITIONAL STRUCTURES	**24**	**20**	**14**
TOTAL (%)(A+B)	**100**	**100**	**100**

Fig. 5.29: Cost distributions (in %) of three French small size river navigation weirs

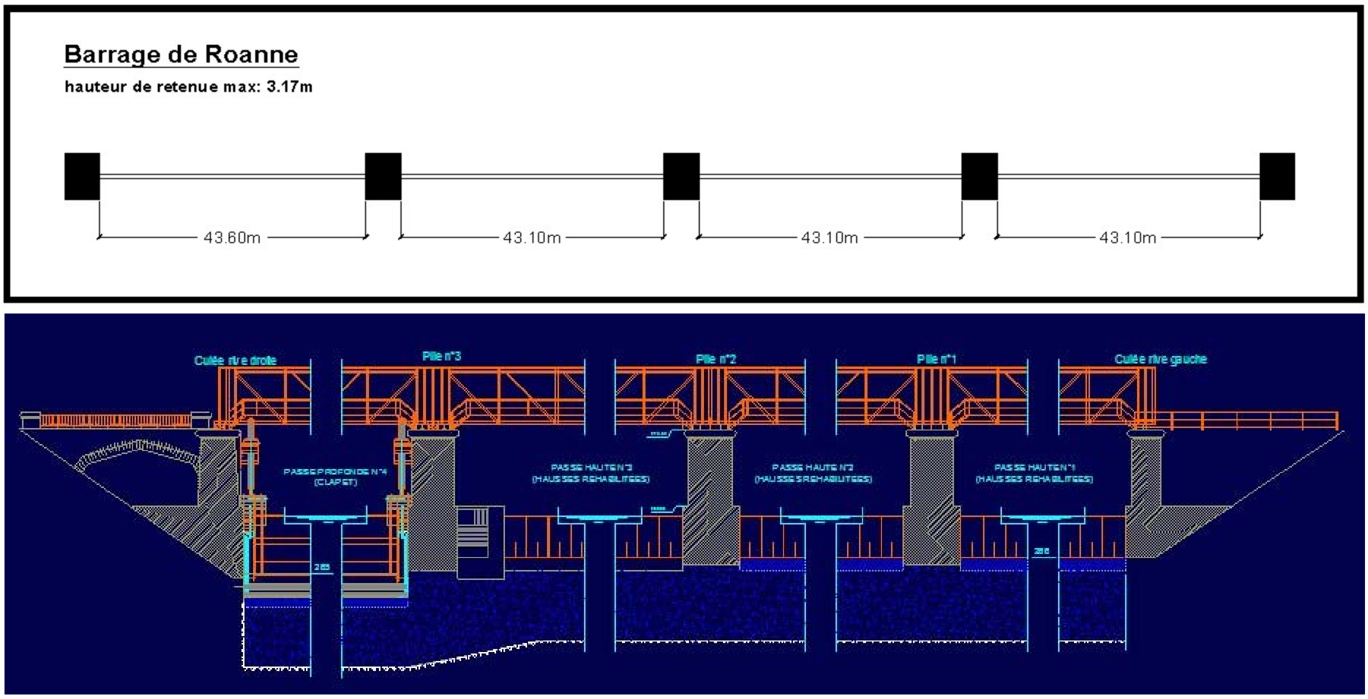


Fig. 5.30 : Weir of Roanne (France)

5.8.2 ASSESSMENT OF THE CONSTRUCTION COST

At the preliminary design stage (or feasibility study), construction cost can be figured with an uncertainty of 15 to 20 %. To get an accurate and reliable cost assessment, all of the cost items must be considered. The following information needs to be specified:

- Organisation(s) to lead the design effort (engineering and design department, architect),
- Location of the weir as labour costs and even raw material (as concrete) vary from region to region,
- Construction methods (cofferdam, phasing, prefabrication, etc.),
- Existing access (roads) to the site location,
- Need of major environmental compensatory and coaching measures (fish pass, planting, etc.).

To assess the different cost units, it is necessary to perform a preliminary estimate of each quantity (volume, surface, length, etc.) and use experience gained from similar works.

5.8.3 EXAMPLE OF THE ROANNE WEIR

The Roanne weir (France) gives an example of the cost distribution (see Fig. 5.31). It is composed of 4 spans equipped with wicket gates, with a maximum water head of 3.17m.

To rebuild this dam, the following procedure was used:

- The civil engineering was re-used and the original design was kept,
- Span 1 was renovated using flap gates,
- Span 2 and Span 3: the former gates were replaced by a system similar to the old one,
- Span 4 was kept unchanged.

Items	Description	Cost (%)		
I100	**Preliminary studies and additional surveys**			(*)
I200	**Land acquisition**			(*)
I300	**Compensatory measures (provisions)**			(*)
I400	**Main structures**			
	Civil engineering			**47.17%**
	Land preparation and energy piping		(*)	
	Site preparation		**9.40%**	
	Mobilisation and installation	9.40%		
	Dredging		**0.52%**	
	Upstream dredging	0.52%		
	Bulkheads, cofferdams and terraforming		**25.66%**	
	Earth deposit and rock protection of the upstream apron/floor	2.67%		
	Downstream slab rock protection	10.71%		
	Upstream work cofferdams	6.21%		
	Downstream work cofferdams	5.95%		
	Geotextile	0.11%		
	Piers and slab		**10.10%**	
	Concrete	6.17%		
	Coffers	1.59%		
	Frame	1.44%		
	Anchoring	0.90%		
	Cylinders piers anchoring			
	Active piers anchoring			
	Piers		(*)	
	Abutments		(*)	
	Slab		(*)	
	Inverts		(*)	
	Downstream and upstream protections		(*)	
	Dikes surface restoration		**1.48%**	
	Grass	0.18%		
	Masonnery	1.30%		
	Equipements			**39.13%**
	Closure devices		**24.95%**	
	Deep gate (nb 4) flap	10.72%		
	Gates 2 & 3 refurbishment	14.23%		
	Operation system		**7.09%**	
	Deep gate (nb 4) cylinder	2.62%		
	Movable carriages for span 2 and span 3	4.47%		
	Operation cofferdam		**5.24%**	
	Floating boom	5.24%		
	Electrical equipement		**1.85%**	
	Electrical equipement	1.85%		
	Automatism		(*)	
I500	**Additional structure**			**6.92%**
	Equipement room		**0.40%**	
	Equipement room security work	0.40%		
	Foot bridge or bridge		**6.29%**	
	Foot bridge rehabilitation	6.29%		
	Fish ladder		**0.23%**	
	Right bank fish ladder refurbishment	0.23%		
	Canoeing and Kayaking pass		(*)	
	Upstream embankment protection		(*)	
	Metal work		**0.37%**	
	Ladders and persons protection	0.19%		
	Discharge pipes diam. 200Mm and valves	0.18%		
I600	**Old weir treatment**			**6.41%**
	Demolition		**2.61%**	
	Concrete and masonnery demolition	2.61%		
	Masonnery work		**3.80%**	
	Repairing & Make stone/brick joins	1.29%		
	Gaps filling	0.12%		
	Cement coating repair	0.11%		
	New concrete nailing on old structure	0.83%		
	Anchorages of the sill in the masonnery	1.46%		
I700	**Master of work and safety and hygien coordination**			(*)
I800	**Contingencies**			(*)
	TOTAL (Taxes Excluded)			**100%**

(*) Items not considered in this example of a small weir reconstruction

Fig. 5.31 : Cost distribution for the Roanne weir (Reconstruction)

5.8.4 STORM SURGE BARRIERS

Total Cost of Nieuwe Waterweg barrier	**Value 2002 (Euro)**	
Overhead costs Staff and general administration, geotechnical and hydraulic investigation, public relations, etc.	30 600 000	7.70%
Civil works	189 800 000	47.74%
Steel constructions Including delivery, assembly and mounting of all steelworks	142 000 000	35.71%
Electromechanical parts Moving parts on each door, energy systems, gate control systems	35 200 000	8.85%
TOTAL EXCL VAT (€)	**397 600 000**	100.00%
VAT	83 496 000	21.00%
TOTAL INCL VAT (€)	**481 096 000**	

Fig. 5.32: Cost distribution of the Nieuwe Waterweg barrier.

Fig. 5.32 shows the cost distribution between various types of works for the Nieuwe Waterweg storm surge barrier (See Project Review). Although weirs and barriers have common elements, they are rather different structures. So data concerning barriers cannot be compared with weir data. Moreover, as each barrier is a unique work, it is not feasible to extrapolate their costs from previous projects.

5.8.5 MAINTENANCE COST

Maintenance costs are closely tied to the operating and mechanical systems selected. An estimate of these costs can be made with the assistance of suppliers and previous maintenance experience at other projects. For various aspects of the project, maintenance costs will be known when the structure is placed in operation. But this does not preclude the necessity of estimating these costs and integrating them into the total project cost at the earliest design stages. This provides a comprehensive view of the total project costs.

Use of monitoring equipments (levelling, verticality, constraints, deformation, wears, temperature, energy consumption, etc.) and the staff's safety during repair and replacement of materials on the system, must be considered at beginning of the project. Moreover, building procedures and the design concepts generally account for major maintenance with the inclusion of special features and devices.

In addition, maintenance often influences the civil engineering design, and has impact on the operation of the facility itself. For instance, a corrosion protection system can be achieved:

- Either with cathodic protections (anode durability is nearing approximately 15 years),
- Or by using a painting system, of which durability ranges from 7 to 25 years and sometimes more (50 years).

Actually, these works or those concerning submerged wearing parts like bronze rings of flap gates bearings, fasteners and watertight systems require setting up rather heavy logistics (draining, security, accessibility and power supply). The preparation cost can be more expensive than the work itself (replacement, repair). To limit maintenance, structures are sometimes oversized (for an example, adding 1-2 mm to the plate thickness). Survey, control and maintenance of protections (fenders) and watertight systems (embankments, downstream floor, piers, etc.) must not be forgotten.

Finally a specific maintenance plan must be established simultaneously with the construction.

An example is provided on the WG's CD-Directory /Annex Section 5.8 …/Maintenance Planning …/. It concerns the maintenance of the Lith lock in The Netherlands. It includes costs used for (or during) planning and scheduling. Unfortunately these assessments only relate to locks and not to weirs and are only theoretical and are not yet checked in practice.

5.8.6 OPERATION COST

Operational Cost is a budgetary item that can be high. In 2004, France still has about 150 manual weirs (especially needle dams, weir-boards) where each operation requires from 2 to 10 people. To evaluate this work, we need to establish the hydraulic operating requirements, the allowable water surface variation (especially for navigation weirs).

According to the complexity of these rules, designers have to consider the possibility of using automation or a remote control system (distance operation). Furthermore, it is important to think about organization of the weir control and of its different elements, and to use specialists for this task.

If the weir structure is to accommodate visitors or tourists, the additional cost in terms of safety measures, personnel, and infrastructure can be significant and should be included.

5.8.7 FINANCIAL PART

5.8.7.1 Financial planning

When many organizations are involved (planning, contract, flood control program, etc.) everything has to be contractually defined to clearly set up the organisation of the project and the financial commitments of each party. The funding for studies, design, and administrative procedures should be separated from construction financing to avoid a delay in the implementation of the project.

In France, weir study costs are usually estimated as a percentage of total cost as follows:

- 3% of the global cost for the preliminary studies concerning the whole river or a section where construction of several weirs is planned. These studies can last 3 years because of the necessary data collection.
- 9% for the project studies including detailed design. This step should last 2 years and includes the tender.
- 9% for the control and the follow-up of the works (1 or 2 years).

The main steps (schedule) of a project are shown in Fig. 5.34

Fig. 5.33 shows an example of a global financial schedule of the project. It gives the estimated cost for each operation and per quarter. Updates of this plan during the studies and the various operations give the project manager a continuously updated financial balance sheet. In this example the time period, refers to a quarter but could be changed according to the total duration of the project.

It is a planning document to assist the project team in obtaining, on time, the relevant authorizations to proceed, and gives the picture of the necessary budget allowance at the beginning of each year and each quarter (or any chosen time period).

5.8.7.2 Financial Schedule

Work financial schedule												
Stage	**Schedule**											**Total**
YEARS	**1**	**1**	**1**	**1**	**2**	**…**	**2**	**3 ….**	**… 3**	**4…**	**..5, ..6**	
Preliminary studies												
Project survey and notice to bidders												
Safety coordination survey												
Administrative procedures												
Dialogues with companies												
Safety coordination												
Works												
Totals per quarter (3 months)	**€**	**€**	**€**	**€**	**€**	**…**	**€**	**€**	**€**	**€**	**€**	
Annual Totals				**€**			**€**		**€**		**€**	**€**

Fig. 5.33 : Example of a Work Financial Schedule (Roanne, France)

Work schedule

Main stage	Schedule																							
Months	3	6	9	12	15	18	21	24	.	.	.	.	.	.	.									
Preliminary studies																								
Topographical survey	1																							
Bathymetrical survey		1																						
Dialogue with waterway user	1																							
Public network holders brainpicking	1																							
Geotechnical survey			1		1																			
Negociated notice to bidders for environmental study				1																				
Environmental study and hydraulic survey					1	1																		
Coordination for safety during works						1																		
Notice to bidders for surveys execution					1																			
Preliminary study						1	1																	
Administrative procedures																								
Dialogues with users						1	1																	
Public survey and application of "Rules on Water"								1	1	1														
Prefectoral clearance for construction of weir (Rules on water)												1	1											
Statement for works on equipment room														1										
Operational survey																								
Project survey						1	1																	
Notice to bidders							1	1	1															
Maintenance specific plan (included in notice to bidders)							1	1																
Operation specific plan (from project studies)						1	1																	
Financial procedure																								
Studies authorization request	1																							
Works authorization request											1													
Notice to bidders																								
Limited tendering (advertisement and bidders declaration)							1																	
Opening of candidacy								1																
Tenderers analysis and selection									1	1														
Dialogues with tenderers											1	1												
Offers opening													1											
Offers analysis														1										
Notice to bidders signature															1									
Execution of works																								
Coordination for safety during works																1								
Tender notification																1								
Routine start order																	1							
Preliminary works																		1						
Civil engineering works																			1	1	1			
Steel construction works																					1	1		
Hydraulics, power lines and automatism																					1	1		
Works receipt																							1	
Weir commissioning																								1

Fig. 5.34: Example of a Work Schedule of a navigation movable weir (Roanne, France).

6. DESIGN AND ASSESSMENT TOOLS

This section presents design and assessment tools currently used in standard practice for the design of movable weirs. Also, new trends in the use of advanced analysis are introduced.

The section is based on a questionnaire sent to about 20 design companies in about 12 countries (see Table 6.1). About half on these organisations (5 publics and 6 privates) replied (Belgium, Czech Rep., Germany, Japan, the Netherlands, UK, USA).

The list of companies/organisations, questionnaire and the answers received from survey participants, are available on the CD's Directory /Annex Section 6/.

The questionnaire focuses on the <u>standard design tools</u> used nowadays by engineers in the current practice of designing movable weirs and barriers. It also surveys <u>the engineer's needs for specific and advanced tools</u> taking into account, the design requirements that become more and more demanding (economic, technical, and environmental aspects).

In the following sub-sections, the design tools are categorized according to the different technical problems that an engineer faces during the design of a movable weir/barrier:

(1) CAD software for project drawing and plans,
(2) EARLY DESIGN tools including optimisation capability,
(3) HYDRAULIC: Flow pattern and discharge assessment,
(4) PHYSICAL MODELS in laboratories,
(5) LOADS assessment including dynamic water pressure, wind, wave, tide, snow, ice, etc.,
(6) Strength assessment of STEEL structures,
(7) Strength assessment of CONCRETE structures,
(8) Strength assessment of FOUNDATIONS,
(9) Static and dynamic FLOATING STABILITY assessment,
(10) FINANCIAL assessment,
(11) Other specific tools and software (RISK assessment, ENVIRONMENTAL assessment, GIS, etc.).

Tools, specificities and user requirements are discussed in relation with the tool purposes. For each technical problem (see points (1) to (11) above), the WG proposes a list of relevant tools with, if possible, recommendations and reference to previous experiences (with links to project reviews). According to the design stage (preliminary design stage, detailed design stage) specific problems with their associated assessment tools are discussed like structure optimization, cost assessment, nonlinear behaviour, large deflection, shock and impact, etc.

Some tool specificities are briefly presented/described with reference to annexes and/or web sites (when available). General-purpose tools like commercial finite element packages are considered as well as specialized tools, which are specific for particular applications/problems.

Table 6.1: List of participants in the survey.

Participant	Company	Location	Surveyed Topics
1	AQUATIS a.s. (JSC)	Brno, Czech Republic	CAD, Hydraulic
2	Bundesanstalt für Wasserbau	BAW, Karlsruhe, Germany	Steel Concrete
3	Port Design Standard Division National Institute for Land and Infrastructure Management Ministry of Land, Infrastructure and Transport (Japan)		CAD Loads Steel Concrete Foundation Reliability
4	Bureau d'Etudes Greisch (GEI) (Mr. De Ville V.)	Parc Scientifique du Sart Tilman, Liege, Belgium	CAD Loads Steel Concrete Foundation Spatial analysis
5	Black & Veatch Consulting Ltd (Mr. J. Waller)	Surrey , United Kingdom	CAD Hydraulic Loads Steel Concrete Foundation
6	Ministry of Transport, Public Works and Water Management of the Netherlands, Civil Engineering Division	Bouwdienst Rijkswaterstaat Zoetermeer, The Netherlands	CAD Hydraulic Loads Steel Concrete Foundation Floating Structures Financial anal. Others

Participant	Company	Address	Surveyed Topics
7	British Waterways, Bridges Dept. who provides support to the rest of British Waterways	National Support Unit British Waterways Leeds, UK	CAD Steel Concrete Foundation Financial anal
8a	International Marine and Dredging Consultants (IMDC nv)	Antwerp, Belgium	CAD Hydraulic Loads GIS & others
8b	Tractebel Engg; Environment & Safety Energy and Industrial Solutions	Brussels, Belgium	Steel Concrete GIS & Risk
8c	Tractebel Development Engineering	Brussels, Belgium	CAD Hydraulic Loads Steel Concrete Foundation
9	INCA Engineers, Inc.	Bellevue (WA), USA	CAD Hydraulic Loads Steel Concrete Foundation Floating Structures Financial analyses
10	Technical University of Brno	Brno, Czech Republic	Hydraulics

Table 6.1: List of participants in the survey.

6.1 TYPES OF TOOLS REQUIRED BY ENGINEERS/COMPANIES/EXPERTS (based on the survey)

In this section, for each technical problem (Sections 6.1.1 to 6.1.11), the WG gives the results of the survey (tools used; user requirements, user needs, …). When relevant, the WG refers to some tools (i.e. most commonly used, innovative tools, …).

In principle, the WG tried to avoid mentioning software names. Nevertheless, commercial names are sometimes mentioned to easier categorize the tools. The mentioned software's are only given as examples (to give an idea) and it does not mean that these tools are better than others; only that they are only more popular. In addition, the lists are not exhaustive; they are only the image of a survey based on 11 participants from 7 countries.

6.1.1 CAD SOFTWARE FOR PROJECT DRAWING AND PLANS

CAD software is used throughout the project period. This includes the first master project plan through the final manufacturing documentation and drawings.

Nowadays the production companies often use advanced parametric and object oriented software programs for the detail design stage (like ProEngineer, CATIA, SolidWorks, Solid Edge, etc.). Then, the models made with these CAD programs can be easily transformed to the detail documentation, manufacturing program (CME) or are used as basic data for the strength assessment programs.

Based on the survey, the main user requirement is to have a CAD (computer aided design) tool with the following features:

- Entities (lines, circles, arcs, etc.),
- 2D, Surfaces,
- 3D Solids.

Operating System (Windows XP, NT, Unix, etc.) is also a major user requirement.

Less demanding CAD features concern:

- Components (bearings, joining pieces, screw…)
- Design mechanical pieces (articulation, shaft,….)
- Design Piping and electro-mechanic systems
- Used to interface with numerical tools (FEM, …)
- Visualization, raster-graphic files (JPG, PDF,…)

The most often cited software are *AUTOCAD (85%)* and *MICROSTATION (50%)* (within a list of more than 15 software packages).

<u>WG comments:</u>
All the participants of the survey said the use CAD tools starting at the early design stage. These CAD tools are used to define the master plan, the main geometry, etc. First, the main dimensions are defined (spans, floor thickness' and

elevations, pier sizes and shapes, etc.), then gates, valves, mechanical and electromechanical parts are selected. The CAD can also be used as a convenient tool to assess the quantities (volume of material, surface wall, length of weld…) and then latter the cost.

6.1.2 EARLY DESIGN TOOLS

The question was if individuals use specific tools at the early design stage (feasibility analysis, preliminary design).

The survey shows that about 50% use specific early design tools for:

- Strength assessment, and particularly the LBR5 innovative tool (least cost optimisation tool for gates of weir and barriers),
- Hydraulic and flow assessment.

But only 10% use specific early design tools for:

- Cost assessment
- Least Cost Optimisation.

WG comments and recommendations

A) Use of direct analysis

Use of advanced design tools at the preliminary design stage is now a standard procedure in bridge design and civil engineering buildings as well as in the ship and offshore industry.

For movable weirs, the survey confirms that in 2004, the current practice at the early design stage is not to perform direct analysis (excepted in few countries like the Netherlands). Often preliminary design (including selection of weir type and concept design) is only based on previous experience of the designers and from previous projects. It is common to select a weir type without an extensive investigation on the "best type". Tradition is the main driving force.

When technical aspects like flow speed, discharge, stress and load have to be assessed, simplified formulas from rule books are often used. Each company and each country have their habits and own "convenient formulations – rules of thumb" (even if quite conservative).

Such usual practice in civil engineering and particularly for standard hydraulic structures like movable weirs does not leave enough room for optimisation and innovation. It is indeed difficult to develop an innovative concept if the design is mainly based on tradition and experience.

In the future, the WG recommends that the weir design procedures continue to integrate previous experiences but also be based on direct analysis. This means that engineers should consider direct analysis tools at the early design stages. This also means that they must be familiar with the specific tools applicable to the early design stage.

B) Early design analysis versus detailed analysis

It is necessary to differentiate the early design stage tools (that are very specific design tools) from the traditional detailed analysis tools (used to validate an existing design - typically the standard commercial Finite Element packages -FEM) and the simplified formulations (which are rule based). On one hand, using a standard FE package at the early design stage does not usually fit with the design time frame and the available budget. On the other hand, using traditional oversimplified formations at the design stage does not allow the designer to optimise and design innovative structures. It is therefore recommended that the designers use specific analysis tools specifically designed for the early design stage (quick, flexible and user friendly).

C) New innovative software

It is unfortunate to hear from experienced engineers, "*We cannot perform such analysis at the early design tools, we have no time, we do not have the data, it is not worthwhile, …*". This was true with traditional heavy and time-consuming numerical models but it is no longer true with new tools that allow optimisation at the early design stage of the weight and the construction cost. This is accomplished using a simplified computational model including fast structural constraints assessment (yielding, buckling, etc.).

For instance, for the design of gate structures, there are now specific software (Rigo, 1999 and 2000) that:

- Gives a quick and fast strength assessment.
- Performs automatic production cost assessment.
- Compares alternative designs based on their strength, their weight and their cost (multi objective optimisation tool).
- Automatically defines the scantling (plate, stiffeners and frames dimensions), which is sized to satisfy structural constraints and to fit with the design objectives (cost, weight, safety, etc.).

D) Cost assessment

Cost is a main concern for all designers but it does not seem that designers have the ability and the opportunity to assess cost at the early design stage. This means that the selection between alternatives is often based on rough assessments or typically using the lead engineer's experience. In the future, we recommend that these procedures be changed using specific tools like cost assessment, risk assessment and the use of multi-criteria analysis (see Section 4).

Optimisation tools are now available but designers are reluctant to use them since most of the contacts are established in Euro/kg or Euro/m^3. Therefore, there are no financial motivations for builders to design lighter (*means cheaper for the state*). To have safer, cheaper (lighter) structures, the ministries and public organizations (that fund weirs and barriers) have to specify in their "work specifications" that the company's design methodology must include design analysis tools capable of reaching a specified objective (i.e. low cost, highest safety index, etc.).

In this aspect, the WG recommends that for large and expensive public works elements like weirs, the following objective function be required: "*For a given set of safety indexes, the design should correspond to the lowest cost for the owner*". This means that there should be no other alternative design that gives the same safety indexes and a lower cost. By safety indexes we means the safety factors corresponding to the different limit states (yielding, serviceability, ultimate....).

6.1.3 HYDRAULIC: FLOW PATTERN AND DISCHARGE ASSESSMENT

Based on the survey, about 80% of designers use numerical models to estimate the flow fields adjoining the gate under study.

Standard analyses are (for 80 to 100% of users):

- One-dimensional steady-flow equations,
- 2-D steady- or unsteady-flow models,
  - Depth-averaged or width-averaged
  - Finite difference, finite volume, or finite element
- 3-D Shallow-water equations.

Note that 3-D Navier-Stokes models were found to be used by 60% of users.

In the design stage, main user demands (70-100% of the users) are:

- Flow patterns in the river,
- Flow around the weir/gate,
- For environmental impact: excessive sedimentation, water table effects, impact upon wetlands, increased turbidity,
- Salt water intrusion (in coastal areas).

Less demanding target concerns:

- Polluted flow, contaminant transport (15%)
  (using conservative or nonconservative model)

Within about 20 cited software packages (see Annex A), standards are HEC-RAS from the US Corps of Engineers and MIKE11, from DHI (Denmark).

6.1.4 PHYSICAL MODELS

Concerning the need of physical modelling (laboratory experiment) about 80% of the users reply YES, but only for complex or large and expensive projects. Main objectives of the physical modelling are flow patterns, load assessment and sedimentation/erosion.

Secondary concerns are vibration, dynamic behaviour, microbiological problems and for environmental issues.

WG Comments:
Physical models in laboratories are usually performed when the early design is finished, but before the detailed design starts. It is therefore an intermediate assessment tool.

Even if numerical models became more and more reliable and commonly used, scale models are still required, especially for hydraulic concerns. It is often used in parallel with numerical codes to validate the codes and sometimes for calibration purpose by '*tuning'* some parameters (for instance for sediment transport codes). After validation (calibration), the numerical code becomes, at low cost compared to experiment, a reliable tool for comparative analysis.

6.1.5 LOADS ASSESSMENT

The question was regarding the relevant and most frequently assessed loads and how they are currently assessed.

The following paragraphs give a comprehensive list of loads that are assessed. The values in brackets show those, which are the most frequently required.

* *Water pressure:*
- Static [100%],
- Dynamic (flow loads) [80%],
- Unsteady flow [60%],
- 2D and 3D model [80%],
- Uplift water pressure (interface soil-Structure) [80%],
- Hydraulic Added Mass [15%].

* *Seismic effect* (inertial forces from the structure mass, hydrodynamic pressures, etc.) [80%]

* *Wind* (Static and dynamic effects) [80%]

* *Wave loads and wave pattern* (dimensions and forces depending on the extend of water surface or fetch, the wind velocity and duration and other factors) [80%]

* *Tide* (induce flow and variation of water level) [50%]

* *Snow and ice loads* [50%]

* *Others*:
- Soil (earth) and sediment pressures: [100%],
- Impact forces (ship, floating body, etc.) [100%],
- Thermal loads (particularly for bodies in both air and water) [100%],
- Mooring loads and towing forces [80%],
- Various pre-stressing loads [20%],
- Obstacles on the way of the gate motion (closure/opening) [20%],
- Vessel propeller loads [20%].

* *Loading combinations* and computational cases (specified or not by rules) [100%

About 25 software packages were mentioned in the survey, but none of these are specifically 'load oriented'. They are 'structure analysis oriented (Finite Element packages)', 'hydraulic-hydrodynamic oriented (CFD)' or 'soil mechanic and hydrogeology oriented'. Therefore, users may find the relevant tools to assess loads in these categories (see Appendix A).

6.1.6 STRENGTH ASSESSMENT OF STEEL STRUCTURES (yielding, buckling, ultimate strength, fatigue, vibration, …);

The question was to define the relevant and most frequent *strength assessment of steel structures* and how they are currently assessed.

The next list gives potential limit-states to assess in the design process. The value in bracket shows those, which are the most frequently performed.

Considered limit-states:

- Yielding assessment (von-Mises stresses) [100%],
- Buckling (plate and beam) [85%],
- Ultimate strength [100%],
- Fatigue [100%],
- Vibration (flow or wind induced) [85%],
- Seismic [75%],
- Fire (incidentally) [50%],
- Shock and collision (ship or floating bodies on gate) [75%].

About 40% of the users have at least one special requirement within the following: brittle fracture, delamination of composite material, service life assessment, geometrical and physical nonlinearity, diverse contact problems, friction, wear and lubrication problems and plastic analysis.

Concerning computational models used for strength assessment of steel structures (of weirs), about 80-100% of the users can perform:

- Beam model (3D frame) [100%],
- Plate/shell model (3D) [100%],
- Shell element + beam element (3D) [85%],
- Linear and static analysis [100%],
- Dynamic analysis [85%].

Less than 60% of users require non linear analysis for the design of weirs and barriers.

About 40% of the users use at least one other computational model within the following categories: incidentally chain, membrane (with plate) element, combinations of beam and chain, soil-structure interaction, implicit and explicit code.

6.1.7 STRENGTH ASSESSMENT OF CONCRETE STRUCTURES (reinforced, pre-stressed, light, high strength …);

The question was to define the relevant and most frequent *strength assessment of concrete structures* and how they are currently assessed.

Potential limit-states are:

- Yielding assessment (Tresca, ..) [100%],
- Pre-stressed & post-tension concrete [100%],
- Cracks [100%],
- Impact (ship) [85%],
- Seismic [85%],
- Heat dissipation [70%],
- Fire (incidentally) [50%],
- Serviceability proofs, Compactability (water tightness) [50%].

About 50% of the users have at least one special requirement within the following: ultimate limit state analysis, young concrete, time effects due to aging-creep-shrinkage, building stages, explosion (terrorist), frost impact, and chemical stability.

Seismic loading is not considered in those countries that are not in a seismic zone. However 100% of the users have the capabilities to consider seismic effect using the “Seismic coefficient method (pseudo-static method)” or the “Dynamic analysis procedure (Response spectrum analysis or Time history analysis)”.

➔ 100% of the users assess the following points:

- Stability of the structure against overturning
- Stability against sliding on the contact surface between the structure and the foundation soil
- Stability of the structure against uplift pressure (the structure must not float)
- Strength - unitary stresses in the body of structure (Tresca, ..)

➔ 100% of the users in current practice use the following computational models:

- Beam model (3D frame)
- Plate/shell model (3D)
- Shell element + beam element (3D)
- Non linear analysis
- Dynamic analysis
- Non linear incremental stress analysis (heat dissipation, …)

➔ 100% of the users are able to study the following concrete structures:

- Gravity structures,
- Thick shell structures,
- Thick reinforced plates.

➔ 80% of the users perform 3D analysis to study piers, sills and monoliths. A few use 2D models, for instance for sills.

For strength assessment (steel and concrete), there are hundreds of FE-analysis commercial packages. About 20 software packages were cited. None of them were mentioned twice. This indicates there is not, one standard for usual finite element analysis (steel and concrete). For linear analysis, there are international codes like ANSYS, NASTRAN, SAP2000, etc. but there are also in-house programs that provide more flexibility. There are also programs that have special capabilities (seismic, ...). For explicit finite element analysis LS-Dyna is a standard (crash, ...). These codes are usually not relevant for foundations and soil mechanics (Sub-section 4.1.8).

6.1.8 STRENGTH ASSESSMENT OF FOUNDATIONS AND GROUND WATER MODELLING

The question was to define the relevant and most frequently used *design criteria for the assessment of foundations* and how they are currently assessed.

➔ 100% of the users consider the following requirements (design criteria for foundation stability under external loads):

- *Sliding – sliding factor*
- *Settlement:*
  - Deformability of foundation soil
  - Limited irreversible deformation
  - Reversible deformation
  - Unlimited irreversible deformation, followed by rupture
- *Internal stability*
- *Uplift and flotation*
- *Strength: pressure on the foundation soil / bearing capacity of the foundation soil*

➔ Only 70 % of the users consider the following requirements concerning "Ground water modelling":

- Seepage and piping in dam foundation (soil hydrology, fractured-rock seepage, etc.)
- Saturated Porous Media, Saturated-Unsaturated Flows
- Contaminant transport

Concerning computational models about 100% of the users are able to perform 2D and 3D models: As foundation stability closely relates to dam configuration, analyses are usually done for the whole system (dam body + rock or soil foundation), using:

- 2D model,
- 2D horizontal flow, 2D vertical flow, 3D flow, steady, unsteady,
- 3D model.

For foundation assessment, there are also many commercial packages available on the market (see Annex A). About 25 software packages were cited in the survey but none of them were mentioned twice. This suggests that there is no standard software for such analysis. Several of these tools are in-house tools. Listed tools are often specific to a particular application/problem. None of them can solve all the foundation problems; a set of different programs is therefore required.

6.1.9 FLOATING STABILITY

It becomes more and more economic and therefore popular to use floating structures (or sub-structures) for the design of movable weirs and barriers. The movable parts can be the gates themselves. (Project Reviews 3b, 9c, 9d and 12d, the huge sector gates in Rotterdam, the Swing gate…). Use of floating elements can also be a relevant technique to transport prefabricated elements (Project Review 12b-Floating Prefabricated Weir in Belgium).

In order to design such floating structures, a series of specific requirements must be satisfied. For the survey, we asked the participants to list their specific design requirements for such structures. Of the 11 replies received, only one user replied to this part. This clearly shows that, on the contrary to steel structures, concrete structures and foundations for which standard design procedures are well established and for which many efficient commercial software packages are available (and well known). This is not the case for the floating structures assessment.

For civil engineering floating structures (not ship structures) the relevant requirements (design criteria, limit states) are the following:

- Static floating stability usually assessed by the metacentric height,
- Ballast procedure (considering reduced stability induced by free water surfaces),
- Dynamic stability (when towed, manoeuvred, under wave, etc.).

which require at least the ability to assess:

- Gravity centre, buoyancy centre and metacentric height assessment,
- Mass distribution (included the added mass).

To perform these floating stability assessments, available computational models are (see Annex A):

- 2D model (for structures like a box girder for which the ratio length/width is larger than 3, having transverse watertight bulkheads and a uniform distributed deadweight),
  or
  3D model (for other configurations),
- Small displacement model (usually enough for civil engineering structures which are usually partly moored or have some displacement restraints (as the Swing gate, Project Review 9d),
- Dynamic model (seakeeping model) with added mass are only relevant for ship structures and structures submitted to severe wave actions (to check the self excitation frequency of the structures - usually roll and heave are considered).

Concerning the strength assessment of such structures, the requirements and models are the same as those mentioned above in Sections 6.1.6 and 6.1.7 (steel and concrete assessment).

6.1.10 FINANCIAL ANALYSIS

Only 20 % of the questionnaires (mainly filled in by civil engineers) contain information about financial and economic tools. This suggests that different teams do financial assessment and engineering and the communication between these two groups is weak.

Financial analysis, economic return, etc. are in principle, performed at the preliminary design stage (economical or feasibility studies). This means that it is conducted at the same stage that the preliminary design is done by engineers. Despite this similarity, financial analyses do not seem to be available to designers and to the technical departments.

On the market, tools are available from very *simple spreadsheets* to *sophisticated tools*, which can be in fact generic accounting software that can also support all the business and financial administration of a company.

Lack of an answer in the questionnaire concerning this aspect shows that the engineers and companies do not put enough effort in assessing the cost at the different steps of their design procedure.

The financial departments assess costs. This is usually based on few parameters (unitary cost parameters: €/m^2, €/kg, span, kg/m, etc.). Afterwards, when the contact is signed, the designer focuses on technical aspects keeping in mind maintaining the company cost at the lowest level, but often he is not able to have a reliable indicator of the current cost. Particularly, he is not able to make a selection between alternatives based on a direct cost assessment.

6.1.11 OTHER SPECIFIC TOOLS AND SOFTWARE

Concerning other specific analyses used for a weir/barrier design about 60% of the users require:

- Reliability assessment,
- Risk assessment,
- Environmental impact assessment,
- Spatial analysis,
- Geographic, GIS (Geographical Information System),
- Time schedule software.

About 40% of the users require software to consider:

- Bathymetry,
- Cut and fill assessment,
- Aluminium structures, composite materials,
- Current and flow measurement (field measurement).

And only 20% of the users require tools for:

- Site selection,
- Ice modelling,
- Shock, impact, and collision.

List of software are given at Annex A.

6.1.12 SELECTION OF RELEVANT TOOLS ACCORDING TO THE DESIGN STAGES AND THE SPECIFIC PROBLEMS

According to the design stage (preliminary design stage, …, detailed design stage) specific problems with their associated with assessment tools are discussed like structure optimisation, cost assessment, nonlinear behaviour, large deflection, shock and impact, risk analysis, environmental impact, etc.

It is sometimes necessary to perform advanced analysis like nonlinear behaviour, large deflection, shock and impact, fire and blast, etc. Many tools are available, but few are user friendly and their use must be limited to experienced specialists.

Standard river weirs usually do not required advanced hydraulic and structural analysis.

Barriers subjected to extreme conditions, require ultimate capacity assessment (extreme load, ultimate strength, crash, etc.). It is recommended that such advanced analysis be performed at the detailed design stage to increase the reliability of the barrier closure.

➔ Detailed design stage

A) Hydraulic: flow pattern and discharge assessment.

Reliable 2D and 3D numerical codes are now available to assess the river flow pattern upstream and downstream of the weirs. They can also be used to assess the flow above and below the weirs. The main difficulty occurs when a hydraulic jump is located just downstream of the gate. In that case, vibration can be induced and the flow cannot be studied independently of the structure. However, only specialists can use hydro-elastic codes.

B) Loads

Load assessment, including dynamic water pressure, wind, wave, tide, snow, ice, etc. are usually assessed through standards. For weirs, it is not the current practice to assess the loads using direct analysis, neither numerical, nor experimental. For large structures (barriers), vibration is often studied experimentally due to the difficulty of modelling the interaction between the structure and the water.

C) Structure: steel and concrete

The current practice is to use finite element commercial packages. For standard linear analysis, there are now reliable, user friendly and rather inexpensive software (1000 to 2000 Euros). Nowadays most of the engineers have the capability to use such software. FE-codes can now be used on a standard desktop computer and provide direct post-analysis processor (location of maximum stress, 2D and 3D deflection pattern, stress 3D pattern, etc.).

Even if it is now internationally recommended to use plate (shell) elements to model steel structure like gates and valves, it seems the common practice is to use beam mesh models. Beam elements were used 20 years ago when shell elements, were not yet reliable and were used to reduce the number of degrees of freedom. Now, this practice is obsolete. Computing time is no more a problem. Thin shell elements (with 4 or 8 nodes) provide a higher accuracy and allow better simulation of the real behaviours of the structure (bending, torsion, buckling, etc.). In 2005, it does not take more time to make a 3D plate model than a beam model. In conclusion: beam-shell elements can/should be used for 3D steel gate structures.

Nowadays codes usually also include linear buckling capabilities and often modal analysis.

For complex problems like:

- crashworthiness analysis between a ship and the weir gate (*Le Sourne et al. 2003*),
- nonlinear analysis (ultimate strength),
- dynamic analysis (vibration induced by flow),

advanced tools are now available, but only specialist can use them.

D) Foundations

There are also a lot of commercial packages that assess foundations, earthquake assessment, ground water modelling and hydrological models, seepage models, transport models. Basic foundation analyses are based on standard FE packages. Still highly experienced specialists are required to get reliable strength assessment of the foundations. Risk of miss-modelling is high, especially in the non isotropic mechanical characteristic that attribute to the different elements (3D element with usually 8-16 nodes). Modelling of cracks in foundations is one of the main difficulties. There is almost no possibility to calibrate the numerical tools (excepted using an expensive and not highly reliable scale model).

E) Floating structures

Only specialist should perform the study of floating structures. Even if there are tools on the market, standard civil engineering companies do not usually have the experience and the capability to perform such analysis. Their use seems simple but a background in naval architecture is highly desirable, not so much to run the model but to understand the physics of the behaviour of floating structures during the ballasting stages. Ballasting is the most sensitive stage due to the loss of stability induced by free-surface moving water. During the ballasting procedure or in case of flooding (accident), transverse and longitudinal bulkheads must be designed to avoid capsizing (loss of floating stability).

In addition to commercial packages, classification societies (Bureau Veritas, Lloyds Register, ABS,) provide reliable tools to assess static and dynamic analysis of floating structures.

F) Other aspects

There are also computer programs used by contactors for the time schedule determination (planning) during the entire (or part of) project procedure.

Other technical concerns that require specific tools are:

- Reliability (safety) and risk assessment,
- Environmental assessment,
- GIS (geographical information system),
- Etc.

7. PREFABRICATION TECHNIQUES

7.1 DESCRIPTION

Flood control projects have traditionally been constructed in cofferdams. This allows traditional construction methods and equipment as well as conventional quality control inspections and measures to be used. The cost of this method is high; it requires the temporary construction of a large cofferdam that serves no final purpose and needs to be removed after construction. There is the risk of overtopping and potential damage to work in progress as well as delays to construction for demobilization, flooding, cleanup and start up efforts.

Prefabrication has long been used on flood control projects for various gate components. Typically the steel gates themselves and their operating components are fabricated offsite and then placed by crane. If the gates are too large to be handled in one piece, they may be brought to the site in sections and assembled in place.

Improvements in technology and engineering knowledge have increased the viability of prefabrication. It is now possible to completely construct hydraulic structures without a cofferdam. The subgrade and foundation can be prepared "in-the-wet" by floating construction equipment that prepares the river bottom and supporting structures from the surface. Templates or guide structures that extend above the water surface can provide great accuracy in placement.

Shells for the substructure and/or superstructure are constructed offsite, transported via a navigable waterway to the site and set in place, see Fig. 7.1. The structure is then filled with concrete to complete the structure and join it to the foundation. If necessary, the gate openings can be closed with bulkheads and dewatered for installation and final adjustment of the gates. It may also be possible to preinstall the gates in the concrete shell prior to their transport and set-down.

A prefabricated gate foundation structure is typically built as a shell structure fabricated of reinforced concrete. Steel or aluminium plate can also serve this purpose but are not as common. The shell is designed to provide a finished surface for the final structure and to provide a "stay-in-place" form for the in-fill concrete that is added at the project site. The shell may also function as its own floating vessel, allowing the shell to be floated and towed to the site. Temporary bulkheads can be installed in openings at the periphery of the shell to allow an otherwise open structure to float.

As an alternative, the prefabricated units may be designed to be lifted into place by large capacity floating cranes. These units can be fabricated adjacent to the final site, launched via a marine railway or skidway and picked up near shore by the crane, carried to their final destination, and lowered into place. If the units are not too large, they can be fabricated further away and transported to the site by barge.

In-the-Wet construction allows rapid completion of construction, minimizes disruption to existing river traffic, and has less environmental impact than conventional techniques.

Flood control structure site locations are typically chosen as a place where hydraulics, topography and geologic requirements can best be met. When selecting a site, the availability of good roads, access to a trained labor force, and availability of materials and equipment are not necessarily part of the selection process, but they are important to the construction of the structure. Prefabrication allows a significant portion of the gate structure to be fabricated in the dry, at a more advantageous site and greatly reduces the area of the site that is involved in construction. This may allow the consideration of a larger number of potential flood control structure sites.

7.2 EVALUATION OF ALTERNATIVES

7.2.1 CONVENTIONAL CONSTRUCTION

Conventional construction for hydraulic flow control structures requires that the site be dewatered prior to construction. This is accomplished one of two ways. Typically a cofferdam is built to block a portion of a river to allow work to proceed in that area in the dry, see Fig. 7.2. The cofferdam is then removed and rebuilt over the next river section and this process is repeated until the structure is completed.

In many cases, a diversion canal or tunnel is built to carry the water around the site and a temporary dam is built upstream and downstream of the construction site. This provides a good work site where the project can be constructed in its entirety and each element of construction is available for visual inspection by all relevant personnel.

7.2.2 IN THE WET CONSTRUCTION

"In-the-Wet" construction is a term used to describe construction that takes place in or on a body of water, typically using floating construction equipment. In-the-wet construction can be divided into two major types – float-in or lift-in construction. Both methods require "in-the-wet" foundation preparation to receive the shell structures. Float-in construction is used for larger sections that can be made watertight and can be moved to the site as a floating vessel, see Fig. 7.3. Float-in structures can also be constructed on a barge and then transported to the site, see Fig. 7.4. When they arrive at the construction site, the barge is flooded and submerged, allowing the shell structure to float off the barge and be towed into place.

Lift-in methods are used for smaller units although catamaran crane barges are available with a lifting capacity up to 8,500 tons for lift beams (Fig. 7.5 to Fig. 7.7) and 20,000 tons for linear jacks. In this case, units are brought to the site on a barge or are launched from shore on a skidway and picked up by a crane and transported to their final location.

There are a wide variety of lifting mechanisms available. These include:

Types	Capacity
Fully Revolving Crane Barges	600 tons
A-frame Shear-Leg Crane Barge	700 tons
Off-Shore Shear-Leg Crane Barge	2400 tons
Jack-up Crane Barge	2520 tons
Catamaran Crane Barge w/ Lift Beams	8500 tons
Catamaran Crane Barge w/ Linear Jacks	20000 tons

7.2.3 SELECTION OF METHOD

A number of criteria should be evaluated in the process of selecting a construction method. Some points to consider are -

- Gate Type and Size - Can it be divided into modules that allow easy transport to the site?
- River Traffic – What limitations and/or closures on traffic could be allowed?
- Site Layout – Is there room and infrastructure to support a large construction project?
- Geology – What foundation types are possible? Is it possible to grade, install and control seepage of the foundation
- Construction Site – Is the site close to necessary materials and a good labor force.
- Hydraulic Conditions
- Transport and available Graving sites

In-the-Wet construction gives the designer greater flexibility in the choices available for both gate types and site location for the structures. The ability to fabricate significant portions of the structure independent of the gate site can improve the quality and reduce the cost of component fabrication. It may also benefit the project to select a combination of the above methods. It may be appropriate to build cofferdams adjacent to each shore and complete shoreside abutments, in the dry, while allowing traffic to proceed during construction. The gate structures can be built offsite and then floated into place when the abutments and foundations are completed, with minimal impact on traffic. The combinations and choices are as varied as the sites and circumstances that face the designer for each project.

OFFSITE FABRICATION ALTERNATIVES

Graving Docks

A graving dock is a basin prepared adjacent to a navigable waterway that can be dewatered so that construction can proceed In-the-Dry. It should have protection from flooding, which may require that a berm be constructed surrounding the basin. For project specific sites this is typically an earthen berm that is removed upon completion of construction. For a reusable site it may be worth the cost to construct a gate with permanent abutments and a sill, see Fig. 7.8 and Fig. 7.9.

In order to minimize dewatering efforts and to protect against flooding, a two-stage graving dock may be constructed. In this case, the graving dock enclosed inside the berm has two levels, a lower level that matches the depth of the adjacent navigation channel, and an upper level above the adjacent water level where the construction will occur, see Fig. 7.10. This requires a substantially higher berm that extends above the level of the casting bed an amount equal to the draft of the precast shells when floating, plus an allowance for clearance under the structure. A clearance of about 0.6 m or more should be provided to allow for transportation and adjustment of heel and trim.

When a precast shell structure is completed, the graving site is “super-flooded” with pumps such that the shell will float in the upper basin. The floating structures are then moved over the lower level and the water level in the graving dock is lowered to match the water level in the adjacent waterway. The berm is then removed and the shell is floated out and begins its journey to the final construction site.

The 2-stage graving site can allow for a larger construction area with less total excavation. During construction, access to and around the shell can require an area 2-3 times the footprint of the shell itself. The excavation from the lower basin can serve to provide the berms for the upper basin. Dewatering is not required, because the lower basin can be allowed to remain flooded to a level below the elevation of the casting slab on the upper basin.

7.2.4 DRY DOCKS AND FLOATING BARGES

A dry dock is essentially a submersible barge with high side walls that remain above water, for control and stability when the barge is submerged. The dry dock is first submerged, then a vessel is floated in place and set on prepared blocks or stands. The water is pumped out of the dry dock and is raised to the point where the vessel and the deck of the dry dock are above the water level and work can begin. These are used extensively in the ship industry for ship repair and maintenance. They tend to be in high demand and their cost for the relatively long duration required for concrete construction can be high.

A less expensive variant is a submersible barge. This is a barge with floodable compartments. A prefabricated structure is built on the deck of this barge, when completed; the barge is then towed to the construction site. Flood and vent tubes are used to control flooding of the barge. This requires a location adjacent to the construction site with the proper depth. It needs to be deep enough to accommodate the total height of the barge plus the draft of the floating structure. But it can not be too deep, because when the submersible barge is completely flooded it can become unstable. One end of the barge is flooded first and set on the bottom, the other end is then slowly flooded to finally release the floating structure. The Ford Island floating bridge for Pearl Harbor was constructed in Seattle on an ocean-going barge and then shipped across the Pacific Ocean to Hawaii where it was unloaded. The unloading took almost a full day. A barge was used to significantly improve the towing speed and eliminate ocean wave loads on the bridge section.

7.2.5 SKIDS AND MARINE WAYS

If the bank conditions on a navigable waterway are appropriate, it is possible to construct the shell structures on shore and then slide them on prepared guides down the bank and into the water.

7.3 SHELL CONSTRUCTION

The shell is a weight controlled structure, whether it is to be lifted or floated into place. The most accurate and precise construction of the project will be completed in the dry; this can then allow greater tolerances for the work that needs to be completed in the wet. If the shell is concrete it can be constructed of precast or cast-in-place concrete. The shell needs to accommodate reasonable construction tolerances for the foundation. For a pile supported foundation a tolerance on pile location of about 0.25m or larger can be accommodated with either an open base on the shell or larger blockouts that are then grouted solid after placement. This allows a reasonable tolerance for pile placement and reduces the costs of the efforts required for template construction and quality control.

7.4 FOUNDATION CONSTRUCTION

In-the-Wet construction requires that the bottom be prepared underwater. While the river bottom does not need to support the deadload of the structures above, it will need to support the dead load of the tremie concrete that is pumped in place to join a pile foundation to the shell and the tremie concrete that is used to close the space between the cutoff walls and the shell. The steps required to prepare the foundation include the following:

- Dredge to remove soft materials
- Grade and place subgrade materials
- Drive Perimeter Sheet Pile Cutoff Walls
- Drive Bearing Piles
- Prepare Landing Pads/Piles

Foundation support can be provided by load bearing soils, rock, H piles, drilled shafts, or pipe piles. Geotechnical studies should be undertaken to investigate the best approach.

Depending on the river conditions, scour and deposition of sediment also need to be controlled during construction. When the shell is set in place, local water velocities may increase significantly and scour the river bottom. Rock, articulating concrete mattresses or other materials can be used to reduce scour.

The construction sequence and schedule should be carefully controlled in areas of high silt deposition to minimize the amount of silt deposited after dredging and prior to placement of the shell.

For rock foundations large hydraulic excavators can excavate bedload materials as well as rip relatively weak bedrock to obtain a satisfactory bearing surface. Grout bags can be used to accommodate significant geometric irregularities in the prepared foundation and, when the concrete sets, provide significant loading carrying capacity, see Fig. 7.11, Fig. 7.12 and Fig. 7.13.

Due to the filling qualities of tremie concrete, significant tolerances in bottom topography can be accommodated in underwater construction. However, accurate placement of the prefabricated shells requires an accurate means of leveling the structure as it is released from the lifting crane or as buoyancy is reduced by ballast.

Specially prepared landing pads can be placed on top of piles or pile clusters to support the shell. Flat jacks are large load capacity, limited movement hydraulic jacks that can provide adjustment up to about 15 mm, see Fig. 7.14.

These jacks can be stacked to provide additional adjustment. If possible the elevation of the landing pads or piles should be verified by optical means carried above the water level. For the Charleroi lower guard wall design, the land pads are supported by 2 m diameter drilled shafts. The steel casing is extended above the water level during construction to allow the bearing pads to be placed, adjusted, and surveyed in the dry. After they are completed, they will be flooded and divers will be used to cut off the casing so that it can be removed by a crane.

7.5 SUPERSTRUCTURE CONSTRUCTION

7.5.1 ALIGNMENT AND ADJACENT STRUCTURES

Optical methods work best for alignment and set down. With floating structures, high degrees of accuracy for alignment can be obtained. The forces required to adjust the location of a floating structure are relatively small. Allowances need to be made for hydraulic changes in flowing conditions as the structure is lowered. Mechanical alignment alternatives include –

- Winch Lines – Fig. 7.15
- Guide Piles and Dolphins
- Spotting Towers – Fig. 7.16
- Horned Guides – Fig. 7.17 and Fig. 7.18.
- Pintles – Fig. 7.19.

Without proper controls the structure may become unstable during setdown. If the structure is going to be completely submerged as it is set down, auxiliary means of control will be required, such as winches to barges or pile-supported structures.

Dolphins or pile-supported guide structures can be constructed in advance of the structure delivery to the site. With the use of shims and accurate pre-measurement of the shell, excellent tolerances can be obtained during set down; accuracy of a few millimetres is achievable.

Mechanical fabrications on pre-placed structures whether alignment dolphins, prepared shoreside abutments, or a previously placed shell can ensure that the shell is accurately placed. A Horned guide is a wide opening that narrows to the final geometry on one structure and mates with a projection on the other.

Pintle guide pins were used with stainless steel shims to align the 122 m long sections of the Olmsted floating guard wall as they were joined to make up the 509 m long upper guard wall. This allowed adjustments of as small as a millimetre longitudinally and transversely.

7.5.2 SETDOWN

Setdown of the shell should not occur until controls are in place to control lateral placement in both directions and the final setdown elevation. This can be done by the use of landing piles or keel blocks for vertical control and guide dolphins, anchors, tag lines, dynamic positioning with thrusters, tug boats or other methods.

7.5.3 BALLASTING

A ballasting plan needs to be prepared that verifies stresses in the structure at each stage as it is lowered. Ballast sequences for filling chambers and venting the air as they are filled, need to be calculated and included as part of the ballasting plan. Chambers may be permanently ballasted with in-fill concrete, structural concrete, gravel, water, or other suitable materials.

If concrete is used as structural in-fill, and not just as ballast, the condition of the interface between the existing structure and the new concrete needs to be properly detailed. Silt and sediment may inhibit a bond between the two materials, the concrete may not be completely consolidated and the compartment may not be completely filled. The engineer needs to balance the benefits of composite concrete with the effort required to ensure the structures work as designed and the quality assurance required to verify that the structure is built as specified.

The last ballast added should be removable – not concrete. If the structure moves laterally as it finally sits down, the benefits of backing up and starting over can be immeasurable. Permanent connections to the foundation can then be made and the temporary, removable ballast replaced with permanent ballast.

7.5.4 FOUNDATION INTEGRATION

Once it is set down on the landing piles or landing pads the structure should be secured before the underbase grouting is placed. This can be accomplished by mechanical connections, additional ballast or a combination of methods.

Once set down the shell is probably not ready to sustain the full design deadload until all of the design piles have been integrated into the structure. Load bearing piles can be integrated into the structure in a number of ways. One is to provide sleeves into the base of the shell structure, when the shell is lowered over the piles, they extend into the shell and the annulus is grouted to bond the two structures together. This can provide both tension and compression capacities. Tension capacity may be required to resist uplift forces if the gate bay is dewatered or to resist uplift pressures due to the underbase grouting. The upper portion of the pile that will be embedded in concrete can have deformations welded around the circumference that improve the shear capacity of the connection.

The annulus between the pile and the shell can be enclosed by use of a wiper seal preinstalled in the base of the shell that closes the gap as the shell is lowered. A grout bag or pneumatic seal can be preinstalled and inflated. Either method would help to confine the grout and make a secure connection.

The space between the bottom of the shell and the top of the subgrade should be between 0.5 and 1.0 meters to allow adequate tolerances for the subgrade preparation. Upon installation of the shell, this space needs to be filled and also join a hydraulic cutoff wall to the shell structure. The perimeter of this space can be confined by grout bags, which are filled after set down.

Grout bags are fabric tubes or bags that are attached to the bottom of the shell structure with grout and vent tubes that lead to the surface. When pumped full of grout they inflate to fill an irregular void, when the grout sets, it forms a permanent seal.

Other techniques include the installation (either before or after setdown) of a sheet pile perimeter wall that extends above the subgrade and vertically overlaps the perimeter of the shell by about a meter. Rock can also be placed around the perimeter of the structure. When the perimeter is secure, the underbase tremie concrete placement can begin.

A pattern of grout and vent tubes should be preinstalled in the shell structure and extend above the water surface. Placement of the tubes depends on the quantity of grout or concrete to be placed, the zones to be used for placement, and any areas under the shell that may trap air or water that will need to be forced out by the grout or concrete. Grout or concrete is pumped into one zone until it can be observed filling adjacent zones, whereby the tremie pipe is moved to the next zone until the entire substructure area is filled. The grout or concrete for this mix is specially designed to be pumpable, self-consolidating and flowable. If the grout or concrete is designed for high compressive strength (35 MPa or higher), the mix sometimes contains an anti-washout admixture to prevent dilution when placed underwater.

7.5.5 SUPERSTRUCTURE

When the pile connections and underbase grout or concrete have achieved their design strength, final in-fill concrete in the upper portions of the shell can be completed. It may be that additional precast sections are designed to be added on top of the base shell to provide towers for gate piers or operating structures. They may be added because of draft or height restriction for transport to and installation precluded their construction at the prefabrication site.

When the piers are completed, the gates can be brought into place and floated in on a barge or lifted in by crane. It is possible for some gate configurations to be provided with dewatering slots in the shell that allow the gate bay to be dewatered, see Fig. 7.20 to Fig. 7.23. It is then possible to make adjustments and final installation of the gates in the dry. If necessary, blockouts can be left in the sill and sidewalls to allow fine adjustment for in the dry placement of the sill and side seals.

7.6 LIST OF FIGURES

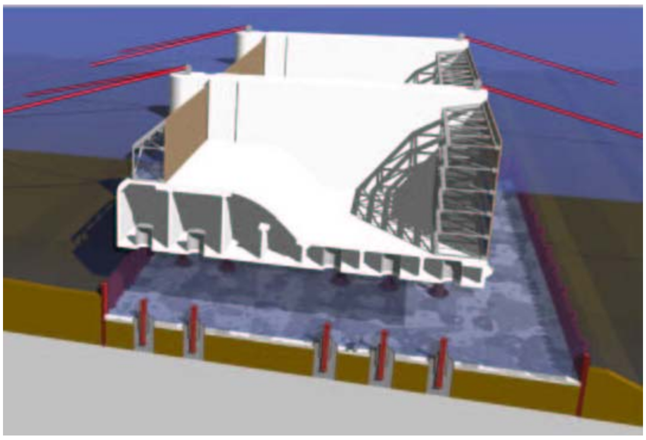


Fig. 7.1: Braddock Lock & Dam Tainter Gate Bay Float in Segment (prefabricated civil works)

Fig. 7.2: Olmsted Locks Cofferdam being removed

Fig. 7.3: Braddock Dam Float-in Tainter Gate Foundation

Fig. 7.4: Montezuma Slough Salinity Barrier Precast Construction on a Barge

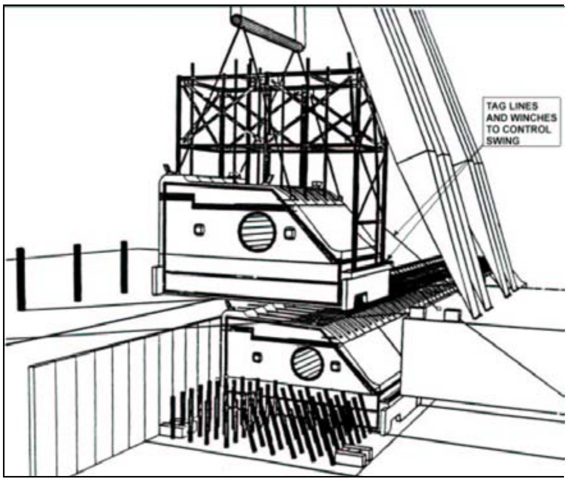


Fig. 7.5: Heavy Lift Dam Segment

Fig. 7.6: Jackup Barge with a Heavy Lift, ERDC TR-02-22

Fig. 7.7: Catamaran Crane Barge of 8500 tons capacity, ERDC/GSL TR-00-2

Fig. 7.8: Concrete Tech Graving Dock w/ Port of Bremerton Floating Dock

Fig. 7.9: Graving Dock Construction of Concrete Bridge Pier Caissons for Oresund Crossing, EM1110-2-2611, C-8

Fig. 7.10: Braddock 2 Level Graving Dock at Leetsdale

Fig. 7.11: Grout Bag Placement Template

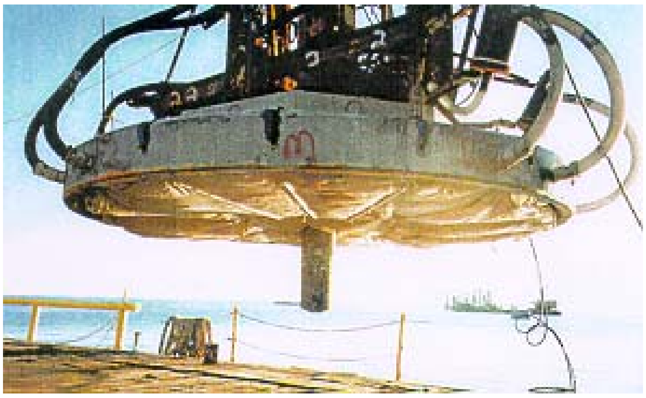


Fig. 7.12: Grout Bag with Grout and Vent Tubes Attached

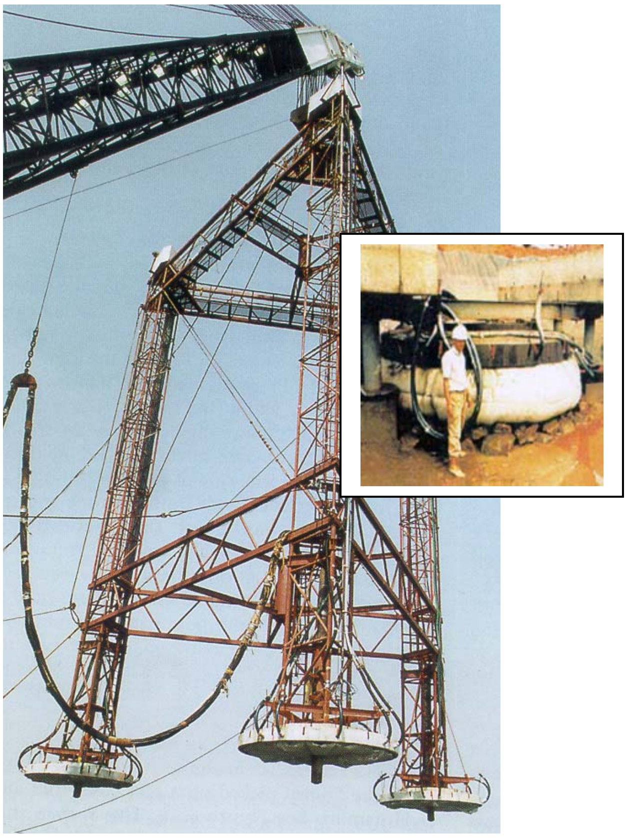


Fig. 7.13: Grout Bag used as a Foundation Leveling Pad

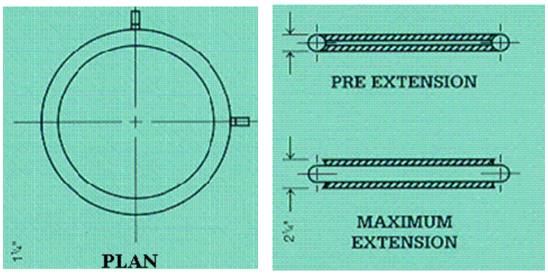


Fig. 7.14: Flat jack Geometry

Fig. 7.15: Alignment with Winch Lines and Tugs

Fig. 7.16: Immersed Tube Bridge Segment w/ Spotting Towers (EM1110-2-2611, page C-9)

Fig. 7.17: Horned Guide

Fig. 7.18: Horned Guide w/ Mated Section

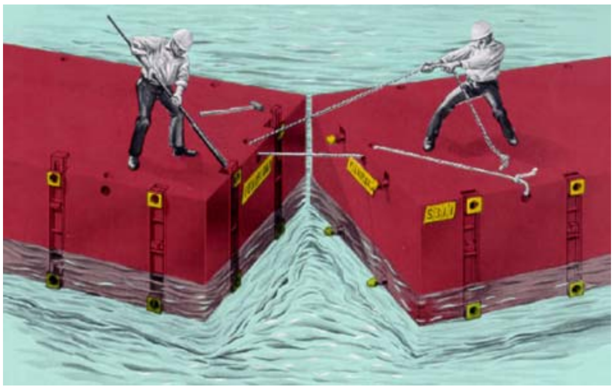


Fig. 7.19: Guide Pintles used for Alignment

Fig. 7.20: Tainter Gate Transported by Barge to Braddock

Fig. 7.21: Tainter Gate Installation by Barge

Fig. 7 22: Tainter Gate Bulkheads in Place to Dewater the Gate Bay

Fig. 7.23: Braddock Tainter Gate – Final Adjustment "In-the-Dry"

8. CODES, RULES and STANDARDS

8.1 APPLICATION OF NEW STANDARDS TO HYDRAULIC STRUCTURES

This section is based on a report kindly provided by Jean-Bernard Kovarik (1998).

8.1.1 THE LIMIT STATES DESIGN AND THE SEMI-PROBABILISTIC FORMAT

The development of new standards (like Eurocodes) based on limit states and partial factors format, has been focusing on the need to express harmonized design standards in practical terms. So far, hydraulic structures have been mainly designed using different rules according to the relevant part of the structure (structural vs foundation design) that leads to tricky situations when different formats are used simultaneously.

On the other hand, several actions [static and dynamic water pressure, waves, currents, … as well as actions due to vessels (berthing, mooring) and to port activities (live loads, cranes, equipments…)] fall out of the scope of existing standards, which are mostly devoted to buildings and bridges (wind, snow, exploitation loads, traffic actions). To overcome this problem, some aspects of the semi probabilistic format were developed, by unifying the «*source factors*» and by diversifying the «*model factors*». The most important issues to be addressed when developing a limit states verification format are then: partial factors, characteristic values for actions with emphasis on water actions, assessment of safety level, and calibration procedures.

In Europe, some aspects of the Eurocodes' format were developed by unifying the «*source factors*» and by diversifying the «*model factors*». The «*source factors*» are related to actions, materials and resistances; they allow only for the intrinsic uncertainty on parameters and their values are mainly derived from existing codes or regulations. The «*model factors*» are introduced in the limit state function at the last stage in the verification process and must be calibrated in order to fit with traditional design rules.

In France, this has led to the publication of «*Guidelines for the limit state design of harbour and waterways structures*» such "ROSA 2000" based on the Eurocodes' format (CD's Directory /B2 ../).

8.1.2 A HARMONISED DESIGN PROCEDURE

In Europe, a major development began at the end of the 1970s, with the progressive substitution of the traditional «allowable stress» methods by semi probabilistic methods in the rules for checking structural safety.

The considered limit states are:

- Ultimate limit states (ULS) which, if exceeded, would result in the destruction of the structure through loss of static equilibrium, mechanical strength, shape stability, etc.; ULS are those phenomena whose occurrences have so dramatic consequences that it is economically consistent to prevent them by severe predetermined safety margins.
- Serviceability limit states (SLS) which, if exceeded, would result in a malfunction that would jeopardise the intended use of the structure; SLS are those phenomena whose occurrences have only limited consequences so that it is economically consistent to assess less severe safety margins.

In Eurocodes, formats are used depending to the limit states and the nature of the basic variables. To do that, partial factors are divided into:

- «*Source factors*», noted γ_f, γ_M and γ_R, which apply to the basic variables, like ground properties, structural loads, material properties, etc.
- A unique γ_d «*model factor*» (for the sake of simplicity). This "*model factor*" is supposed to be located on the left side of the limit state condition, i.e. increasing the action effect.

The general expression of a limit state condition with partial factors, for ultimate limit states, reads:

$$\gamma_d . E(\Sigma\gamma_f . F_k) \leq R[\Sigma(X_k / \gamma_M)] \qquad (8.1)$$

where:

- F_k are loads, R is the design value of the resistance and X is a material parameter (soil, concrete, steel …).
- E is a function of several parameters (geometry, loads....). It symbolizes the model equation (can be a simple analytic model to a complex 3D FE analysis), which for instance, gives the stress at a specified location.

In practice, it means for instance, that the usual Eurocode load factor for permanent actions (1.35) is the product of a source factor (γ_f = 1.20) and a model factor (γ_d = 1.125). The same holds for the 1.50 Eurocode factor used for variable actions which is the product of γ_f = 1.33 and γ_d = 1.125.

8.1.3 A PRACTICAL TOOL FOR ENGINEERS

The above ideas and methods are not specific to hydraulic structures like weirs, but relate to the general trend towards harmonisation of design codes in Europe during the 1990s. A major issue in the development of any new safety format lies in the sustainable combination of the following items:

- Full consistency with the European rules «Basis of design»
- Allowance for a national adaptation of safety levels (see: model factors)
- Mitigation of the possible gap between prior national formats

- Important role to still be played by the practitioner in evaluating the results of the new calculation models, and in determining the relevant characteristic values to be specific for the project.

Sharing a common way for the verification procedure is better for quality than over-refining a partial factor. It is very valuable to get a common wording and a good appreciation of safety conditions. In that sense it is expected that the efforts started for several years in many countries to compare and upgrade maritime codes of practice, will continue and contribute to improve the qualification of engineers and the final safety of the works and particularly movable weirs.

8.2 SELECTED CODES OF PRACTICE, RULES, STANDARDS AND GUIDELINES WITH REFERENCE TO THE WG26'S SUBJECTS

In the different countries of the WG26 members, several codes of practice, rules, standards and guidelines concerning the river weirs and barriers are available and in use. As the number of pages of the report is limited, the general overview concerning this special item is available in the WG-26's CD (Directory /Annex Section 8 – Codes/). Here we only introduce this overview.

In part **"A: Compilation based on information of the WG-members",** for a better understanding, the items on the CD are ranked by: user's country, origin, name, year of edition, and specification/title.

Part **"B: CEN On-line catalogue / ISO On-line catalogue"** deals with basis data, which are available on the CEN On-line catalogue (Metal Structures and Technical Aspects), (http://www.cenorm.be) and the ISO On-line catalogue (Steel structures), (www.iso.ch/iso/en/CatalogueListPage.CatalogueList).

Part **"C: List of British Standards"** includes a list of available British Standards.

9. CONCLUSIONS & RECOMMENDATIONS

The WG has investigated a variety of projects and concludes that much knowledge and information particularly relevant to the design of movable weirs is available, but not being taken advantage of. We hope that this report will enable designers of future projects to take advantage of that knowledge and information, leading to improvements in design and economies in construction.

As it was stated that the '*design of movable river weirs is a conservative world*', the WG recommends:

- About Innovation
  The Public Administrations, who are usually the weir owners and managers, should leave more room for innovation and new concepts.
- About Prefabrication and Standardisation
  Prefabrication usage that closely relates to standardisation should be investigated, as it is a source of savings, fast construction, and friendly environment construction modes.
- About Temporary Closure Devices
  Temporary closure devices and maintenance bulkheads must be considered as a key issue of an efficient design.
- About Design Procedure and Multidisciplinary team
  It is now time to integrate the traditional weir design procedures with risk assessment, maintenance and control, codes and standards (Eurocodes), and design concept (limit states and partial safety factors). Such integration requires a multidisciplinary team composed of engineers, economist, biologist, social analyst, etc.
  Limit state concepts and semi-probabilistic approaches (as included in the EUROCODES) should be commonly used in the future.
- About Computational Tools
  We should promote the development and use of specific computational tools for preliminary design. Advanced analysis can now be performed at the early design stage to show the feasibility of new innovative concepts. Optimisation can also be performed at the early stage, as it can induce large savings. Delaying will reduce the potential benefits.
- About Gate type selection
  Gate selection is an important stage in a barrier or weir project. The operational, financial, and other consequences of this selection are often more severe than are the detailed engineering. It is, therefore, advisable to give thorough consideration to the gate type selection.
- About Multi-criteria Analysis
  Previous experiences of skilled engineers may be used, but cannot replace a brainstorm meeting to get innovative concepts and then a fair multi-criteria assessment.

  Recommendations about multi-criteria assessment are:
  - It is advised to let the criteria and their weighting factors be determined by a team representing the project initiator (local authorities, other parties involved) – and the actual rating by a multidisciplinary team of professionals. <u>Both teams should act independently.</u>
  - Effort should be made to get a clear, well-balanced inventory of all criteria significant for a particular project. Clusters of criteria may be considered. In addition, it should be advised to keep the number of gate types under investigation small, e.g. not larger than 4 to 6.
  - Qualitative assessments are procedurally simple and fast – but, on the other hand, quite arbitrary and not very transparent. Quantitative assessments require more effort and time, but are less arbitrary and more transparent. The assessments based on cost analyses are probably the best quantifiable approaches, but a more universal assessment method is the <u>performance rating with weighting factors</u> for different criteria. The performance rating method is not free of arbitrariness, but it is more transparent than the qualitative methods; and better balanced than the methods based on costs analyses.
- About Maintenance and Standardisation
  Maintenance is one on the major hidden issues of a weir design. Maintenance must be considered at the early design stage in order to reach a high efficiency/cost ratio and a high operational standard. Considering maintenance at the design stage may incur higher investment costs but, for sure, will reduce the 30-50 years life-cycle global operational cost.
- About Floating Structures
  Designing movable a structure as floating structures should be used more as it usually leads to simple, cheaper, and more reliable structures. Floating structures require the use of specialist and specific tools to assess floating stability at any stage. Floatability can also be used as a construction mode (see prefabrication techniques).
- About Control of Operation
  The philosophy "Keep it Simple" is always good, but not always realisable! There are examples of very simple flood defence structures that work well, but need a lot of manual input. There are also some very sophisticated structures that operate entirely by automation. The real question lies in the reliability of the system and the consequences of failure. It is recommended that all critical elements of the control system be duplicated and that the power supply and drives be backed up to some extent.
- About Risk Based Design:
  Risk analysis is now an accessible tool for the design of weirs and barriers. It is particularly useful when failure

may induce important damages to nature, cities, and the human lives.

Benefits of using a risk-based design are:

- Evaluating margins of safety more realistically than traditional (deterministic) safety criteria,
- The possibility to achieve economic benefits,
- Comparing a wide variety of options and enable the risks due to flood defence to be compared with the risks due to other natural and man-induced hazards,
- Consider not just the likelihood of high water levels against a defence (barriers, dikes, etc.), but also the likelihood of defence failure and the degree of harm resulting to people/property, etc. behind the defences.

- About Environmental Impact and Aesthetics

  It is recommended that clients, designers, and planning authorities be mindful of the "*whole life cycle*" impact of their projects.

  Similarly, it is important to consider the "*whole environmental footprint*" of the project and not just factors relevant to the site of construction and operation.

10. REFERENCES

ASSEL, R.A., 2000 : “Maximum Freezing Degree-Days as a Winter Severity Index for the Great Lakes”, 1897-1977, Monthly Weather Review, Volume 106.

AWZ, 2004: “Actualization of the Sigmaplan, Social benefit cost analysis”, Belgian Ministry for waterways, Department Seascheldt.

BHARGAVA, V.P.,NARASIMHAN, S., 1988 : "Pressure Fluctuations on Gates", J. of Hydraulic Res., Vol. 27.

BOUVARD, M., 1991 : “Mobile Barrages and Intakes on Sediment Transporting Rivers”, IAHR Monograph, 320 pages.

BS.61508, Functional Safety of Electrical, Electronic and Programmable Electronic Safety-Related Systems. See also http://www.bsi-global.com/

BURT et al., 1996 : "Barrages: Engineering Design & Environmental Impacts", Edited by. N. Burt & J. Watts, John Wiley & Sons., 504p.

BURT, N., REESE, A., 2001: "Guidelines for the Assessment and Planning of Estuarine Barrages", Thomas Telford. London

CASSIDY J., 2000: “Gated Spillway and other controlled release facilities, and dam safety”, ICOLD Beijing congress Q79, pp 735-778

CAMPBELL, F., 1961: "Vibration Problems in Hydraulic Structures", J. of the Hydraulics Division, Proc. of the Am. Society of Civil Engineers, Vol. 87.

COMFORT, G., GONG, Y., SINGH, S., ABDELNOUR, R. and LIDDIARD, A., 2003 : “Static Ice Loads On Hydro-Electric Structures: Summary Report; Ice Load Design Guide, and; Ice Load Prediction Computer Program”, CEATI Report T002700-0206.

COMFORT, G., LIDDIARD, A., ABDELNOUR, R., 2004: “A Method and Tool for Predicting Static Ice Loads on Dams”, IAHR Conference, St. Petersburg.

DALY F., 1995: "Etude sommaire sur les facteurs de coût des reconstructions de barrages en rivère par des ouvrages à vannes clapets", Rapport SOR 95.02, November, CETMEF-VNF, France,

DANIEL, R.A., 1996: "Hartelkering - Schuiven met Mogelijkheden", Bouwen met Staal, No. 130.

DANIEL, R.A., 2000: "Keersluis Heumen - R. Waterkerende Staalconstructies", 3904-S&SM-D-99.304, Bouwdienst Rijkswaterstaat, Utrecht.

DANIEL, R.A., 2003 : "Construction Material for a Bridge with Regard to the Environment", Bautechnik, Heft 1.

DANIEL, R.A., DONKERS, A.L.J.M., DE RIJK, M.C., 2003 : "Naviduct Krabbersgat in Enkhuizen - Sluisdeuren naar een hoger niveau", Bouwen met Staal, No. 175.

DEFRA, 2002: “Temporary and Demountable Flood Protection”, Environmental Agency, UK. (*Available on the WG26's CD),* http://www.defra.gov.uk/environ/fcd/pubs/pagn/default.htm

DIN EN 1050:1997-01 Safety of machinery - Principles for risk assessment

DOWRICK, D.J., 2003 : “Earthquake Risk Reduction”, John Wiley & Sons.

ELSEVIER, 1987: "Water and Hydraulic Engineering", Dictionary of Elsevier Science Publishers B.V., Amsterdam-London-New York-Tokyo.

ENVIRONMENT AGENCY, 2000: The Humber Estuary, Shoreline management plan, http://www.environment-agency.gov.uk/

ERBISTI, P.C.F., 1981: "Hydraulic Gates: The State of Art." Water Power Dam Construction, April 1981, pp43-48

ERBISTI, P.C.F., 1994: "Developments in Hydraulic Gates in Hydropower and Dams", Hydropower and Dams, January 1994, pp 51-57

ERBISTI, P.C.F., 2004: "Design of Hydraulic Gates", A.A. Balkema Publishers, The Netherlands, www.balkema.nl or www.tandf.co.uk (see WG26-CD in the Directory /Annex Section 5.1 – Structure/)

FAGANELLO E., 2004: “Hydraulic Modelling to Revise the Sluice Gate Control Logic at Cardiff Bay Barrage”, PIANC, Bull. n 117.

GERWICK,B.C., YAO, S.X., BERNER, D.E., BITTNER, R.R., 2000: "Assessment of Heavy-Life Equipment for In-the-Wet Construction of Navigation Structures", ERDC/GSL TR-00-2, U.S. Army Corps of Engineers, Vicksburg, Mississippi.

GERWICK, B.C., YAO, S.X., 2002: "Positioning Systems for Float-In and Lift-In Construction in Inland Waterways", ERDC/GSL TR-02-22, U.S. Army Corps of Engineers, Vicksburg, Mississippi.

GILL, H., MAY, C., 1989: "Crest Gates Design for Upper Mechanicville", Proc. of the Int. Conference on Hydropower, New York, NY, Vol. 3.

HARTUNG, F., 1973: "Gates in Spillways of Large Dams", Int. Commission on Large Dams, Madrid Congress, Q41 R72, pp.1361-1374.

GISZAS, H., 2002: Sturmflutschutz: Herausforderungen und Sicherheitskonzept (Protection against storm floods: Challenges and safety concept) HTG-Kongress 2003, Stuttgart, 17-20.09.2003, Thomas Telford, London. Available at http://www.thomastelford.com/ or http://www.ttbooks.co.uk/

HADJIAN, A., 2004: "Basic Elements of Earthquake Engineering", John Wiley & Sons.

HITE, J.E. 1993: "Scour Protection for Dam No. 2, Arkansas River, Arkansas, Hydraulic Model Investigation", Technical Rep. HL-93-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

HOMA, M., OGIHARA, K., 1976: "Theoretical Analysis of Flap Gate Oscillation", Trans. of the Japan Society of Civil Engineers, Vol. 7.

ICOLD-CIGB : "Technical Dictionary on Dams and Glossary appended", International Commission of Large Dam, www.icold-cigb.org .

ICOLD Question 76, 2000: The use of risk to support dam safety decisions and management, Beijing.

IPPC, 2001: "Greenhouse Effect and Sea Level Rise: The Cost of Holding Back the Sea", Report of the Intergovernmental Panel on Climate Change.

ISO/FDIS 2394: 1998(E), "General principles on reliability for structures", International Standard, Final Draft 1998.

JANSEN, D., ROUVE, G., 1996 : "Safety Evaluation and Utility Value Analysis for Rehabilitation of Old Barrages", Barrages: Engineering, Design and Environmental Impacts, Int. Conference, Cardif, U.K.

KLINGMUELLER, O., BOURGUND, U., 1992: "Sicherheit und Risiko im Konstruktiven Ingenieurbau (Safety and Risk in the Field of Structural Engineering)", Verlag Vieweg & Sohn, Braunschweig/Wiesbaden.

KOLKMAN, P. A., 1980 : "Development of Vibration-Free Gate Design: Learning from Experience and Theory", Practical Experiences with Flow-Induced Vibrations, Int. Association for Hydraulic Res., Karlsruhe, Germany, September 3-6, 1979, Edited by E. Naudascher and D. Rockwell Springer-Verlag, Berlin.

KOVARIK, J.B., 1998: "De l'application des Eurocodes aux ouvrages maritimes et fluviaux", Revue Française de Génie Civil, Vol.2.

KULKA, A., 1928: "Der Eisenwasserbau", Verlag von Wilhelm Ernst & Sohn, Berlin.

LE SOURNE, H., RODET, JC, CLANET, C., 2003: "Crashworthiness Analysis of a Plane Lock Gate Impacted by Different River Ships," PIANC Bull., No. 113.

LEWIN, J., 1995: "Hydraulic Gates and Valves in Free Surface Flow and Submerged Outlets", Thomas Telford Publications, London.

MAHADVI, A., RIES R., 1998: "Towards Computational Eco-Analysis of Building Designs," Computers & Structures, No. 67.

MOCKETT, I.D., SIMM, J.D., 2003: "Risk Levels in Coastal and River Engineering: A Guidance Framework for Design", TAW, Leidraad Kunstwerken, (May 2003)

NAEIM, F., KELLY, J.M., 1999: "Design of Seismic Isolated Structures: From Theory to Practice", John Wiley & Sons.

NAUDASHER, E., 1991: "Hydrodynamic Forces", IAHR publications.

NEILSON, F. M., PICKETT, E.B., 1980 : "Corps of Engineers Experiences with Flow-Induced Vibrations", Practical Experiences with Flow-Induced Vibrations, Int. Association for Hydraulic Res., Karlsruhe, Germany, September 3-6, 1979, edited by E. Naudascher and D. Rockwell, Springer-Verlag, Berlin.

NOVAK, MOFFAT, NALLURI, NARAYANAN, 1997, "Hydraulic structures", E&FN SPON,

OGIHARA, K., UEDA, S., 1980 : "Flap Gate Oscillation, Practical Experiences with Flow-Induced Vibrations", Int. Association for Hydraulic Res., Karlsruhe, Germany, September 3-6, 1979, edited by E. Naudascher and D. Rockwell Springer-Verlag, Berlin.

PARTENSKY, H., SWAIN, A., 1971 : "Theoretical Study on Flap Gate Oscillation", Hydraulic Research and Its Impact on the Environment, Fourteenth Congress of the Int. Association for Hydraulic Res., August 29-September 3, 1971, Vol. 2.

PERILLO, G., 1997: "La valutazione di impatto ambientale degli impianti di depurazione mediante analisi e comparazione delle alternative progettuali", Int Symposium: Ingegneria Sanitaria ed Ambientale (SIDISA), Ravello (SA), Italy. (*In Italian*). See English Draft "E.I.A. as an "Environmental Planning" Tool - Analysis and criteria for comparing alternatives, on the WG26's CD-Directory /Annex Section 5.7 – Environment.

PETITJEAN, 1981: "Evolution Technologique des Barrages Mobiles", La Houille Blanche, No.2-3.

PETRIKAT, K., 1980: "Discussion of 'Flap Gate Oscillation' by Ogihara, K. and Ueda, S.", Practical Experiences with Flow-Induced Vibrations, Int. Association for Hydraulic Res., Karlsruhe, Germany, September 3-6, 1979, edited by E. Naudascher and D. Rockwell, Springer-Verlag, Berlin.

PIANC, 1985: "Illustrated Technical Dictionary", Editor: PIANC, Author: Werner Röhle, Austria, 6th Draft, February 1985 (Reporter as final draft).

PIANC-WG19, 2001: "Ship Collision Due to Presence of Bridges", PIANC, Report of InCom WG 19, 2001.

PIANC-WG23, 2004: "Technical and Economic Problems of Channel Icing", PIANC, InCom Report of WG23.

PIANC-WG25, 2005: "Maintenance and Renovation of Navigation Infrastructures", PIANC, InCom Report of WG25 (to be published).

PIANC-WG17, 2004: "Inspection, Maintenance and Repair of Maritime Structures Exposed to Damage and Material Degradation Caused by Sault Water Environment", MarCom Report of PIANC WG-17 (Revision 2004).

PIANC-WG34, 2000: "Seismic Design Guidelines for Port Structures", MarCom Report of PIANC WG-34; ISBN 90 265 1818 8

PIANC-WG41, 2003: presenting Guidelines for Managing Wake Wash from High Speed Vessels, MarCom Report of PIANC WG-41.

PETITJEAN 1981: «Evolution technologique des barrages mobiles» – La Houille blanche, n°2-3, pp 141-144°

PICKERING, G. A., 1971: "Spillway Gate Vibrations on Arkansas River Dams, Arkansas and Oklahoma; Hydraulic Model Investigation." T.R.-H-71-5, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

POUW, C., NAGTEGAAL, J.B., 2000: "Maaswerken - Report Varianentstudie Stuw Sambeek", OTB-S&K-S&S-D-00.025, Bouwdienst Rijkswaterstaat, Utrecht (In Dutch).

PUGSLEY, Sir A., 1974 : "The works of Isambard Kingdom Brunel", Institution of Civil Engineers, London.

PULPITEL, L., 1980 : "Some Experiences with Curing Flap-Gate Vibration", Practical Experiences with Flow-Induced Vibrations, Int. Association for Hydraulic Res., Karlsruhe, Germany, September 3-6, 1979, edited by E. Naudascher and D. Rockwell, Springer-Verlag, Berlin.

RACT, COTILLON, LEMAY, 1970 : "Les Barrages Mobiles en Rivière", Travaux, n°423, p4 (In French)

RAZEK, A., LIDDIARD, A., COMFORT, G., 2003 : "Development of Boom Design Procedures and Site Selection to Improve the Warning and Efficiency of Hydro Plants", Commission Internationale des grands Barrages, Montreal.

RAZVAN E., 1989, "River Intakes and Diversion Dams", Elsevier,

RIGO, P., MARCHAL, RODRIGUEZ, 1996: "The Use of Floating Gates for Storm Surge Barrier", Barrages: Engineering Design & Environmental Impacts", Edited by N. Burt & J. Watts, John Wiley & Sons.

RIGO, P., 1999: "Least Cost Optimum Design of Stiffened Hydraulic and Floating Structures", P.I.A.N.C. Bulletin, No.101.

RIGO, P., 2000 : "Un Outil d'Optimisation des Structures Orthotropes Intégré à l'Avant-Projet", Revue Construction Métallique (CTICM), No.2-2000, Paris.

RIJCKWATERSTAAT, 1982: "Eastern Scheldt Storm Surge Barrier", Proceedings of the Delta Barrier Symposium Rotterdam, 13-15 October 1982.

ROSA-2000, (2001): «Rosa 2000 : "Recommandations pour le Calcul aux Etats-Limites des Ouvrages en Site Aquatique», (French Ministry of Equipement)", Centre d'Etudes Techniques Maritimes et Fluviales (CETMEF-VNF), Ponts-Formation-Editions, France (in French).

SCHWARTZ, H. I., 1964: "Nappe Oscillation", J. of the Hydraulics Division, Proc. of the Am. Society of Civil Engineers, Vol. 90.

SCHMIDGAL, T., 1972: "Spillway Gate Vibrations on Arkansas River Dams", J. of the Hydraulics Division, Proc. of the Am. Society of Civil Engineers, Vol. 98, No. HY1.

SEHGAL, C., 1996: "Design Guidelines for Spillway Gates", J. of Hydraulic Engineering, Vol. 122, pp155-165.

SEHGAL, C., 2000: "Selection Criteria for Gates and Operating Equipment for Spillway", ICOLD, Beijing Congress, Q79, pp 275-298. (see WG26-CD in the Directory /Annex Section 5.1 – Structure/)

SLATTERY, K. T, RIVEROS, G. A., 2003 : "Design by Analysis of Innovative Navigation Structures", ERDC/ITL T.R.-02-4, U. S. Army Corps of Engineers, Vicksburg, Mississippi.

TAW 1997:, Basisrapport Waterkerende Kunstwerken en Bijzondere Constructies, Delft, June 1997.

TODD R.V., 1999: "Spillway Tainter Gate failure at Folsom Dam, California", WaterPower 1999, www.ascelibrary.org (ASCE 2004)

U.S. Army, 1952: “Hydraulic Design Criteria”, prepared by the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi in series since 1952.

U.S. Army Corps of Engineers, 1987: “Hydraulic Design of Navigation Dams”, Engineering Manual 1110-2-1605, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

U.S. Army Corps of Engineers, 1990: “Hydraulic Design of Spillways”, Engineering Manual 1110-2-1603, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

U.S. Army Corps of Engineers, 1993: “Engineering and Design, River Hydraulics”, Engineering Manual 1110-2-1416, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

U.S. Army Corps of Engineers, 1995: “Planning and Design of Navigation Dams”, EM 1110-2-2607, 31 July 1995

U.S. Army Corps of Engineers, 1997: “Engineering and Design – Vertical Lift Gates”, Engineering Manual 1110-2-2701, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

U.S. Army Corps of Engineers, 2000: “Engineering and Design – Design of Spillway Tainter Gates”, Engineering Manual 1110-2-2702, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

U.S. Army Corps of Engineers, 1995 (July), “Planning and Design of Navigation Dams”-Engineer Manual

U.S. Army Corp of Engineers, 2002 (October), Engineering and Design, Ice Engineering.

U. S. Army Corps of Engineers. (February 11, 2004). EM 1110-2-2611, “Engineering for Prefabricated Construction of Navigation Projects”.

VAN DEN HORST, T., SCHENK, J., 1981 : "Theatrum Machinarum Universale", Petrus Schenk, Amsterdam, 1736, new edition by T. Wever, B.V., Franeker (In Dutch).

VLEK, C.A.J., 1996: A multi-level, multi-stage and multi-attribute perspective on risk assessment decision-making and risk control, Risk decision and Policy, Chapman &Hall

VNF, 1998 : “Les Barrages Mobiles de Navigation (Navigation Movable Weirs), Guide du chef de Projet, Voies Navigables de France (VNF)”, Ed. Le Moniteur, Paris (*See WG25-CD Directory B3 for a PDF version, with the courtesy of VNF and acknowledgment to R. Abdelnour who made the PDF file)*

WIELAND, W., 2003: “Seismic Aspects of Dam”, Proceedings of 21st Congress on Large Dams (ICOLD), June 2003, Montreal Canada, Vol. 3 Question 83, p.1291.

YAO, S.X., BERNER, D.E., GERWICK, B.C., 1999 : "Assessment of Underwater Concrete Technologies for In-the-Wet Construction of Navigation Structures", T.R INP-SL-1, U. S. Army Corps of Engineers, Vicksburg, Mississippi.

YAO, S.X., BERNER, D.E., MILES,W., FEHL, B.D., WALKER, M., WOODSON, S.D., 2003 : "Proposed Design Criteria on Thin-Wall Precast Panels for Hydraulic Concrete Structures", ERDC/GSL TR-03-14, U. S. Army Corps of Engineers, Vicksburg, Mississippi.

ZIPPARRO, V.J., HASEN, H., 1993 : "Davis' Handbook of Applied Hydraulics", 4th ed., McGraw-Hill, Inc., New York, N.Y.

10.1 WEB SITES

British Waterways: http://www.britishwaterways.co.uk/

VNF (Voies Navigables de France) : http://www.vnf.fr/

DEFRA - MAFF, FCDPAG4, Flood and Coastal Defence Project Appraisal Guidance, Approaches to Risk: http://www.defra.gov.uk/environ/fcd/pubs/pagn/default.htm

The International Canal Monuments List : http://www.icomos.org/studies/canals2a.htm

ICOLD-CIGB, Technical Dictionary on Dams and Glossary appended, International Commission of Large Dam: www.icold-cigb.org

U.S Army Corps of Engineers: http://chl.wes.army.mil/library/publications/

Seacor environmental products http://www.fossenv.com/worldwide/products/boom/security/index.html

Whisper Wave http://www.whisprwave.com/port.htm

APPENDIX A: NUMERICAL TOOLS FOR WEIR AND BARRIER DESIGN

Appendix of Section 6 "DESIGN AND ASSESSMENT TOOLS"

The following software list was established based on a survey made by the WG (Section 6). This list is obviously not a comprehensive list. It is more a quantitative list that gives a relevant sample of tools used in 2004 by designers, contractors and civil engineering companies in the field of movable weirs and storm surge barriers.

Note that physical modelling is also another option and it could be more cost effective for some aspects.

CAD	USER COMMENT	REFERENCES
AUTOCAD	General 2D/3D modelling/design	www.autodesk.com
MICROSTATION	General 2D/3D modelling/design and data conversions	www.bentley.com
SOLIDWORKS	Feature design development	www.solidworks.com
CANVAS	2D surfaces	www.deneba.com/default.html
RGS-CAD & CADSRC	Structural 3D-Modeling (Steel and Concrete)	www.rgs-cad.com
WISE IMAGE	Raster editing package for AutoCAD	www.cadsoftware.se/produkter
AUTOPLANT	3D modelling software for piping and plant	www.rebis.com/products/ www.bentley.com
NAVIS PRESENTER	Desktop 3D model viewer	www.spi.de/navisworks/navis.htm#Presenter
CAD CHECKER	Ensures that all CAD data complies with BVCLtd and clients standards	www.excitech.co.uk/
ProENGINEER	Parametric 3D-Modeling/design System	www.ptc.com
CATIA	3D-Modeling/design System	www.3ds.com/products-solutions/brands/CATIA
SOLID EDGE	3D-Modeling/design System	www.solidedge.com

EARLY DESIGN ANALYSES TOOLS	USER COMMENT	REFERENCES
LBR5	Optimisation of steel structures based on Construction Cost	www.anast.ulg.ac.be/main.php?LGID=2&MID=34

HYDRAULIC	USER COMMENT	REFERENCES
MIKE 11	1D hydrodynamic software for river flow, quality and sediment modelling	www.dhigroup.com/DHISoftware.htm
MIKE 21	2D unsteady flow	
MIKEFLOOD	combining 1D and 2D	
MIKE 12	Two layer stratified flow (salt/fresh water), width averaged	
MIKE AD	Salt intrusion, water quality, sediment transport	
MIKE 3	3D simulation	
MIKEBASSIN	Flow regulation studies, reservoir regulation	
DELFT 3D	3D simulation hydrodynamic, sediments, salt, pollution ;	www.wldelft.nl/soft/d3d
FLUENT	General CFD code	www.fluent.com
WOLF	Package of hydrological and hydrodynamic software (1D, 2D)	www.ulg.ac.be/hach/en
HYDROWORKS	1D hydrodynamic flow simulation	www.hydroworks.org
INFOWORKS RS	1D unsteady flow simulations, calculating flood extents	www.wallingfordsoftware.com
HEC-RAS	Steady state back water program for simple or extensive river networks with all types of hydraulic structures	www.hec.usace.army.mil/software/hec-ras/hecras-hecras.html
DIVAST	A depth averaged 2D flow modelling tool for estuaries and coastal waters. Includes sediment transport and water quality	www.bullen.co.uk/hydromod

SSIIM	3D numerical model for simulation of sediment movements in water intakes with multiblock option	www.bygg.ntnu.no/~nilsol/ssiimwin
PCSWMM	storm water management modelling	www.computationalhydraulics.com
SMS	Surface Water Modelling System: 1D, 2D and 3D hydrodynamic.	www.ems-i.com
FLOW 3D	3D complete hydrodynamic simulations of hydraulic structures	www.flow3d.com
RMA2, RMA4	2D hydrodynamic, depth averaged, free surface (FEM)	www.bossintl.com/html/sms_details.html

WAVE MODELS	**USER COMMENT**	**REFERENCES**
DUROSTA	Beach and dune morphology (erosion)	www.netcoast.nl/tools/rikz/durosta.htm
SWAN	Near shore wave modelling	www.porl.nus.edu.sg/wave_modeling.htm http://128.160.23.41/Products/modeling/swan
MIKE21-BW	Diffraction wave model	www.dhigroup.com/DHISoftware.htm

GENERAL FEM CODES for STEEL & CONCRETE STRUCTURES		
	USER COMMENT	**REFERENCES**
NASTRAN	Beam model (2D), 3D Finite element analysis	www.mscsoftware.com/products/products_detail.cfm?PI=7
FINELG	Strength assessment	www.ulg.ac.be/matstruc/Logiciels.html
LUSAS	Finite element analysis	www.lusas.com/products/
TEDDS	Structural calculations	www.cscworld.com/tedds/tedds.html
SUPERSTRESS	Structural analysis, finite element	www.integer-software.co.uk
SYSTUS	Implicit finite element calculation	www.esi-group.com
LS-DYNA	Explicit finite element calculation	www.lstc.com
FEMAP	Finite element modelling	www.femap.com
ESAPRIMA WIN	Structure finite element code	www.scia-online.com/esawin/
ROBOT	FEM (linear, 2D and 3D, static and dynamic, …)	robot-structures.com
EFFEL-ARCHE	FEM (linear, 2D and 3D, static and dynamic, …)	www.graitec.com/en/effel.asp
SAP2000	FEM (linear, 2D and 3D, static and dynamic)	www.csiberkeley.com/
GTSTRUDL	Finite element analysis	www.gtstrudl.com
ANSYS	Finite element analysis of irregular shapes	www.ansys.com
STEEL STRUCTURES OPTIMISATION		
LBR5	Optimisation of steel structures based on Construction Cost	www.anast.ulg.ac.be/files/doc/Publication003.pdf
STRENGTH ASSESSMENT OF CONCRETE STRUCTURES (seismic, etc.)		
SHAKE	Site effects for seismic analysis	http://nisee.berkeley.edu/software/shake91
SASSI	Dynamic soil-structure interaction analysis	www.vecsa.com/Software/Sassi/Sassi.htm

FOUNDATIONS	**USER COMMENT**	**REFERENCES** (http://www.ejge.com/GVL/)
GEOSTAB	Slope stability (2D model)	www.geos.ch/logiciel-geostab.htm
FLOWPATH	Seepage and Ground water modelling	www.waterloohydrogeologic.com/
WALLAP REWARD	Retaining wall design / Steel sheet piling wall design	www.geosolve.co.uk/wallap1.htm www.geocentrix.co.uk/
SEEP/W	(FEM) seepage, groundwater modelling system.	www.geo-slope.com
SIGMA/W	FEM - stress and deformation	
SLOPE/W	FEM- slope stability -	
SEEP 3D	3D simulation of (un)saturated ground flow	
GINT	Interprets site investigation and laboratory test data	www.gintsoftware.com/
CADS-RETAIN	stability of retaining wall	www.cads.co.uk/software/retain/retain.htm
M-SHEET; M-PILE	Design of sheet piling walls	www.scia-online.com/www/Products.nsf/

FLAC 2D and 3D	2D (3D) finite difference program for geotechnical models	www.itascacg.com/flac.html
PLAXIS 2D	2D finite difference program for geotechnical model	www.ramcadds.com/plaxis.htm
AQUA 3D	Pseudo 3D numerical model (FEM) for simulation of groundwater flow and contaminant migration	www.digimindsoft.com/aqua3d.htm
MODFLOW	Pseudo 3D numerical model (finite difference) for simulation of groundwater flow and contaminant migration	www.waterloohydrogeologic.com/
PCSTABL6	Slope stability analysis program	www.ecn.purdue.edu/STABL/
LPILE	Analysis of Piles and Drilled Shafts Under Lateral Loads	www.ensoftinc.com
GROUP	Analysis of Piles in a Group	
FLORIDA PIER	Analysis of a pile group	www.ce.ufl.edu/software/software.htm

FLOATING STRUCTURES	USER COMMENT	REFERENCES
(BHS) BASIC HYDROSTATICS	Hydrostatic properties, heeling and trumming moments, wired heeling, damage and wave stability, intermediate stages of flooding	www.ghsport.com/csi/ www.aerohydro.com/products/marine/hydro.htm
EXCEL SPREADSHEET	Weight distribution, stability, moment and shear diagrams, small displacement	www.microsoft.com
HECSALV v7.0	Salvage Response and Design Software	www.herbert.com www.herbertsoftware.com
ARGOS	Stability (Ship structures), Classification Society	www.bureauveritas.com/pages/ship_builders.html
MAXSURF	CAD software with stability package	www.formsys.com/Maxsurf/MSIndex.html

FINANCIAL ANALYZISE	USER COMMENT	REFERENCES
S.A.P	Financial analysis (general business tool)	www.sap.com
EXCEL	In-house spreadsheets for Financial analysis	www.microsoft.com

OTHERS	USER COMMENT	REFERENCES
COMREL & SYSREL	Reliability assessment	www.strurel.de/Epages/index.html
SAFETI	Quantitative risk assessment	www.dnv.com/software
RISKSPECTRUM	Reliability assessment	www.riskspectrum.com
ARCGIS	GIS	www.esri.com
ARCVIEW	GIS applications: creating and visualizing DEM's, generating flood maps	www.esri.com
IDRISI Kilimanjaro	GIS application: generate damage maps	www.clarklabs.org
MAPINFO	GIS system	www.mapinfo.com
PC REMBRANDT	Ship simulation models help assessing the difficulties expected during transit.	www.bmtseatech.co.uk
CEDEX		www.cedex.es/ingles/home.html
MS Project	Time Scheduler	www.microsoft.com

Trends of Using Numerical Tools for Automatic Field Measurement and Water Modelling

The availability of new technologies presents an opportunity to collect very large amounts of data with relative ease, low cost and very good accuracy. These instruments include the Acoustic Doppler Current Profiler (ADCP), which provides instantaneous current profiles. Combined with the Global Positioning Systems (GPS), the bathymetry and the current profile are collected simultaneously in addition to the position of the ADCP (the boat). The data is transferred to a computer and presented on a Geographic Information System (GIS system). The data is then used as input to a range of numerical models.

The current distribution (2D or 3D) can later be obtained using FEM. The model is used to evaluate the current distribution for a range of discharges and to assess their effects on the structure and its overall efficiency and cost effectiveness. This data, and the associated numerical modelling, helps in optimising the position of the structure.

An example is presented at the Fig. A.1, where the data was collected to a discharge of 700 m^3/sec, then using a numeric water-modelling tool; the current velocity distribution was obtained for the 10-year flow of 2000 m^3/sec.

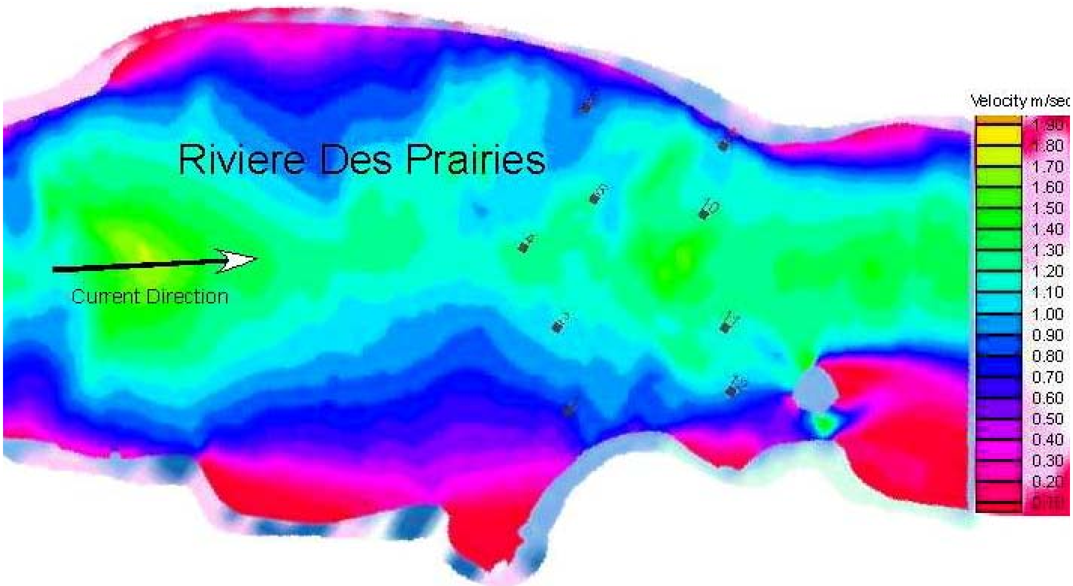


Fig. A.1: Current distribution in "*Rivière des Prairies*" for 1,200 m^3/sec using a numerical model (River width 300 m, Stretch length 800 m)

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