### SMART RIVERS 2019

#### TOPIC B

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Ref. author:  
Bertrand de Bruyn – Cerema Eau, mer et fleuves  
134, rue de Beauvais, 60280 Margny-lès-Compiègne, France  
bertrand.de-bruyn@cerema.fr  

Co-authors:  
François Ropert – Cerema Eau, mer et fleuves  
134, rue de Beauvais, 60280 Margny-lès-Compiègne, France  
francois.ropert@cerema.fr  

Batoul Fevre – SNCF Réseau - Direction Générale Industrielle et Ingénierie  
Dept. des ouvrages d’art – 5, rue Jean-Philippe Rameau, 93212 La Plaine St Denis  
batoul.fevre@reseau.sncf.fr  

Delphine Lenglet – SNCF Réseau – Direction Modernisation et développement  
Dept. des projets et appuis aux projets, Agence MOEG EOLE (Prolongement RER E vers l’Ouest)  
62 rue de la Chaussée d’Antin, Paris 9ème  
delphine.lenglet@reseau.sncf.fr  

Keywords:  
Inland navigation, Seine River, Construction site protection  

Title:  
Optimization of the protection of a shipyard in the river:  
nautical aspects of the doubling of the SNCF bridge named “Pont des Anglais”  

Full Paper:  
The lengthening rail link from Paris-Hausmann to Mantes la Jolie to the West (RER E) requires the addition  
of a third railway line, one kilometer long. This new rail track will be added near, along the bridge named  
“Pont des Anglais” at the city of Bezons. This bridge crosses the Seine. At this location, the river Seine  
flows in two branches: the “Riviére Neuve” and the “Bras de Marly”. The existing bridge consists of two  
parts, each crossing one of the branches. The navigation channel in the “Riviére Neuve” is equipped with a  
lock (185m x 18m x 5m), in the city of Chatou, which allows the navigation of boats of European  
classification size Vb. The navigation channel “bras de Marly” has locks at Bougival, whose navigable  
depth is lower (3.20m) than at Chatou.  
The current bridge has three piers in each branch of river Seine (six piers in all). The piers and fundation  
footing of the bridge reduce the wetted cross section at the bridge which increases the speed of the water at  
the bridge and the turbulence (cross-flows) downstream of the bridge. Furthermore, the fundation footing of  
the bridge has influenced the downstream bathymetry, which has generated a pit and shoals over time.  
Therefore, the return currents are amplified.  
The orientation of the piers of the bridge are not in the main navigation axis. The piers clear two navigation  
channels in each branch of the river, 25.00m large maximum, one for the navigation downstream, the other  
for the navigation upstream.
It makes the navigation of barge convoy (particularly convoys of 180m long / 4000t of load), difficult around and at the current bridge during floods.

The new rail track is planned on a bridge juxtaposed upstream of the current bridge. The chosen solution is to add the piers of the new bridge in the continuity and upstream of existing piers. The new piers extend about ten meters upstream of existing piers. These new piers are all identical in shape and smaller in width than existing one.

Preliminary studies and discussions with inland barge pilots have shown that under the bridge, navigation with large convoys was possible with the new piers and that the navigation could be facilitated by the improvement of current conditions downstream of the bridge.

However, the construction of piers in the Seine river requires to built a cofferdam around future piers. Cofferdams are wider than existing piers. They are therefore partly located in the navigation channel, reducing it accordingly.

The navigation conditions during the construction phases were characterized by survey of the inland barge pilots. This approach led to specify the worst navigation conditions (flow of 500m³/s). On this basis, current velocities were calculated using Telemac 2D software. The trajectory of the boats was deduced from the currents and the geometry of the site thanks to the NAVMER software and with the help of the inland barge pilots.

This study shows that the construction phases could aggravate the navigation conditions.

Furthermore, staff may be present to work at the bottom of the cofferdam. The difficulties of navigation and the vulnerability of the cofferdams leads to protect them.

The protection devices of the construction site must be optimized. On one hand, a robust protection protects the site but makes navigation more difficult (collision risk with the cofferdams increase) and on the other hand, low protection limits the risk of collision but in case of collision, the damage is significant.

The underlying assumptions to dimension the protections are poorly defined. Indeed, the cofferdams are located nearby and even in the navigation channel. Therefore a collision on the cofferdam by a convoy may occur. No appropriate approach has been found in the literature to assess this risk of collision.

Trajectory studies, conducted on simulator, under unfavorable conditions by confirmed inland barge pilots show that a shock on the protections occurs almost at each test with a large convoy (180 m long / 4000 t of transport). This result calls for caution.

The shocks identified during the simulations are all lateral and occur with a low angle of incidence. However, the configuration of the site does not allow to avoid a frontal impact of a convoy with one of the cofferdams of the piers. The protection of construction cofferdams was therefore planned according to two main components: a side protection of the cofferdam and a device that decrease the risk of frontal impact of a convoy with a cofferdam.
Lateral protection consists of steel piles aligned along the cofferdam, between channel and cofferdam. It protects the building area and guides convoys. It was calculated based on the dimension rules related to the berthing of ships. The energy hypotheses (speed, angle of incidence) have been deduced from:
- geometrical considerations of the site: the values of the angle of incidence are deduced by the width between the passes, the size of the convoy and curvature of the navigation channel;
- trajectory studies: these confirm the angle of incidence of lateral shocks on the site and provide the speed during the impact.

The design characteristics of the protections (particularly the tripod) encountered difficulties. There are, in fact, few technical recommendations for the design of such construction area protection. There are, for instance, no standardized assumptions to take into account in performing the calculation.

However, the mechanisms involved in this case, however, are similar to those related to berthing. There are methods for calculating structures for docking (notably: ROSA 2000 [ROSA 2000], or PIANC recommendations [PIANC]). Speeds and berthing angles are considered low for the development of these recommendations. The speed and the angle of incidence of the boat are major factors in the design of the structures.

These conditions (speed and low angles) are generally not representative of the navigation conditions near the Bezons bridge. Indeed, the boats must have a sufficient speed to navigate under the bridge in order to maintain maneuvering capacities.

For the definition of speed, the standard “Eurocode 1 : The standard NF EN 1991-1-7 of February 2007 does not apply for structures intended to receive shocks of boats (quay wall and Duke of Alva) in normal operation. However, the assumptions made for the shock are similar to those potentially known for tripods. This standard recommends using a speed of 3 m / s plus the speed of the water (about 4 m / s for downstream) or more than 14 km / h.

Furthermore, the maximum speed authorized by the Police reglementation is 20 km / h ie 5.5 m / s and at reduced speed 12 km / h maximum, ie 3.3 m / s.
The estimated speed, that measured near the tripods during simulations is between 8 and 10 km / h, or between 2.7 and 2.2 m / s in 80% of cases. Given the narrowness of the pass pilots must maintain a minimum speed (8 km / h) to cross the bridge and maintain its course.

The speed adopted: 1.67 m / s is lower than the speeds usually performed near the bridge of Bezons.

For the angle of incidence the lateral and frontal protections must be distinguished.
For the lateral protections a first approach led to estimate the angles of incidence lower than 30 °. This important value leads to difficulties to dimension the lateral protection structures. A geometrical study of the site and trajectography simulation allowed to bring this value back to 6 °.
However, for frontal tripods it was not possible to exclude a frontal impact (angle of incidence of 90 °).

The speed (1.67 m / s) and the angle of incidence (6 °) used for the convoys is considered low by the pilots but already very important to dimension the structure. Moreover for the protection in front of the cofferdam nothing could exclude a frontal shock (angle of incidence: 90°). In this case it was not possible to dimension a structure that adequately protects the cofferdam.
Finally, it has not been possible either to design structures for the speeds practiced in the sector of the bridge of Bezons nor to exclude the possibility of a frontal collision with the tripod of protection. It was therefore advisable to check from which type of convoy (weight / speed) the protections were sufficient and that for heavier convoys specific measures are envisaged.

The proposed device to minimize the risk of frontal impacts consists of:
- limiting the navigation convoys: deviated traffic from the “Riviére Neuve” to the “Bras de Marly” is advised if possible (barge with small navigable draught);
- signaling and monitoring the building area: an adequate inland vessel traffic signage was set up; a lookout was proposed during the vulnerable phases of the construction to identify large barges.
- protecting the construction area: an upstream steel pile should help the convoy to change a dangerous trajectory; a tripod located upstream of the cofferdam allows to absorb part of the energy of the convoy on an inappropriate trajectory; when the river flow is strong during the construction phase, a pusher craft helps large barges in convoy to cross the area.

Figure 2: Frontal tripod

The nautical aspects of the doubling of the SNCF bridge named ‘’Pont des Anglais’’ open some perspectives of methodological studies: particularly the characterization of the risk of leaving a navigation channel with the means of simulation and the current statistical knowledge and the characterization of
hypotheses of shock of a convoy (velocities, incidences in particular) on an obstacle in or near the navigation channel.

References


Les barrages gonflables de navigation en France
A ce jour 32 barrages gonflables de navigation sont en service en France : 6 barrages équipent l’Aisne, 24 sont situés sur la Meuse, 2 sont sur la Saône. D’autres sont en projet sur l’Yonne. Avec plus de 60 ans d’expérience dans les infrastructures hydrauliques d’envergure, BRL Ingénierie est un acteur de référence du domaine de la navigation tant en France qu’à l’Export et intervient de la conception à la mise en service de 31 de ces barrages gonflables français. Les deux types de barrages gonflables installés sont schématisés ici :

Pour cette technologie, nouvelle en navigation, la principale question qui se pose est celle de la capacité à jaugez les débits, à réguler un bief, et avec quelle précision comparativement à des vannes classiques.

Position de la bouchure et calcul du débit
A la différence des vannes traditionnelles ou clapets, la structure gonflable est à la fois bouchure et actionneur. Cela implique que la position de la bouchure ne peut pas être déterminée par une règle géométrique simple, en mesurant la course d’un vérin ou d’un treuil. Ceci n’a pas d’incidence particulière sur la régulation, sauf lorsqu’il est besoin de déterminer le débit transant par l’ouvrage. L’ouverture d’un barrage gonflable à volets métalliques peut être déterminée à l’aide d’un inclinomètre fixé sur le clapet. Le coefficient de débitance d’un...
seuil mince permet de calculer le débit passant sur l’ouvrage\(^1\). Pour une membrane seule, les outils de modélisation aux éléments finis 2D permettent de déterminer sa géométrie sous différentes conditions de charge :

\[
H_m = f(H_{am}, H_{av}, H_s, P_i)
\]

La position de la membrane peut être recherchée par la relation : \(H_m = f(H_{am}, H_{av}, H_s, P_i)\) avec \(H_{am}\) : charge d’eau amont ; \(H_{av}\) : charge d’eau aval ; \(H_s\) : hauteur lame de surverse, \(P_i\) : pression intérieure. La littérature sur le sujet reste peu fournie, et au stade des études, les données sont souvent insuffisantes. Lorsqu’on ajoute à cela que le coefficient de débitance d’une bouchure gonflable n’est pas linéaire\(^2\), le calcul de débit transitant par l’ouvrage peut être entaché d’incertitudes pour certaines plages de fonctionnement. C’est pourquoi, pour une bouchure gonflable à l’eau, des solutions pour déterminer le débit transitant sur une bouchure ont été développées in-situ.

La mise en service récente de barrages gonflables de navigation en France permet d’observer les conditions de charge réelles des bouchures, et ainsi d’affiner l’évaluation des débits.

**Performances de régulation : l’exemple du barrage de Saint-Joseph sur la Meuse**

La performance de la régulation peut être liée au temps de manœuvre de ces bouchures. Celui-ci est jugé rapide pour des clapets ou vannes traditionnelles. C’est également le cas des bouchures gonflées à l’air et la précision de régulation pose peu de question pour un barrage gonflable équipé de volets métalliques comme celui d’Auxonne. Le remplissage à l’eau génère des temps de manœuvre plus lents qui peuvent poser question.

Le cas le plus emblématique pour juger de la performance de la régulation est celui d’un barrage associé à une centrale hydroélectrique. Un arrêt intempestif des turbines génère immédiatement un surdébit qui doit franchir le barrage : prenons le cas du barrage de Saint Joseph, en service sur la Meuse.

L’ancien barrage de Saint Joseph a été remplacé en 2017 par un barrage gonflable à l’eau (BGE) automatique associé à une centrale hydroélectrique. Le nouveau site a ainsi les caractéristiques suivantes :

- Barrage équipé de 3 bouchures gonflables à l’eau, de largeur unitaire de 35 m et de hauteur nominale 2,86 m ;
- passe à poissons type double fente, centrale hydroélectrique équipée de 2 turbines VLH ; 500 kW unitaire avec un débit turbiné maximal de 55 m\(^3\)/s, écluse au gabarit européen classe 1 (Freycinet)

A cet endroit, la Meuse a un module de 131 m\(^3\)/s. Le barrage permet le maintien du plan d’eau à sa cote de régulation nominale jusqu’à des débits de l’ordre de 650 m\(^3\)/s environ. Pour des débits plus forts, le BGE est complètement effacé, le plan d’eau n’est plus régulé.

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\(^1\) J.Aubonnet, 2016, PIANC Project Review WG166 Auxonne Gray

\(^2\) M.Gebhardt/ B.Kemnitz, 2007, Hydraulische Bemessung von Schlauchwehren
Performances cibles
Les objectifs en termes de fonctionnement attendus sont les suivants :
- Régulation du plan d’eau amont à +/- 9 cm (cote max cible / cote min cible));
- Variations de niveau dans l’heure inférieures à 10 cm;
- Priorité à la navigation (mais pas d’interface avec l’écluse);
- Respect du débit réservé (6 m³/s);

Performances réelles : exemple du mois de mars 2019
L’évolution des débits transités et turbinés du mois de mars 2019 est donnée ci-après.

Le mois de mars 2019 est un mois riche en perturbations hydrauliques. La Meuse a en effet vu s’écouler une large gamme de débits, variant des eaux moyennes à des conditions de très fortes eaux.
Les gradients d’augmentation des débits (et de diminution) ont atteints des valeurs de l’ordre de 40 m³/s/h.
En réaction à ces conditions d’eau, les turbines ont transférés des débits relativement inconstants : le BGE de Saint Joseph a donc été largement sollicité afin de réguler la ligne d’eau.

Le graphe suivant montre quant à lui l’évolution du niveau d’eau amont, régulé en grande partie par le barrage gonflable, et dans une moindre mesure par les turbines de la centrale hydroélectrique :

On observe ainsi, malgré les conditions hydrauliques très variables, que le niveau amont est resté très proche de la valeur de consigne, et globalement dans une plage de +/- 2 cm. Un comportement constant de la ligne d’eau quels que soient les débits présents dans la rivière est par ailleurs observé.

**Performances réelles : essais de découplage des turbines**

Avant la mise en exploitation du barrage, des essais ont été réalisés sur le site de Saint Joseph afin de vérifier que même pour des perturbations hydrauliques les plus critiques, le barrage gonflable à l’eau était en mesure de respecter les performances ciblées dans son cahier des charges. Dans des conditions de faibles eaux dans la rivière, environ 60 m$^3$/s, la centrale hydroélectrique a été mise en fonctionnement à quasi-pleine ouverture (51 m$^3$/s). Le niveau d’eau amont a été stabilisé à sa cote de régulation : le barrage s’est ainsi progressivement fermé, pour ne transférer qu’un débit 3 m$^3$/s (le débit résiduel étant transféré par la passe à poisson).

Un découplage brutal de la centrale hydroélectrique a alors été déclenché : en moins de 2 minutes le débit turbiné est devenu nul. L’automatisme du barrage gonflable, par l’intermédiaire d’instructions spécifiques et automatiques de compensation, a modifié la position des bouchures. L’intégralité des débits de la Meuse a ainsi été évacuée par le barrage. Le graphe ci-après montre la coupure brutale du débit des turbines.

Le graphe suivant montre quant à lui l’évolution du niveau amont.
Entre le découplage des turbines à l’instant t et t+6min, le niveau amont s’est élevé très rapidement. L’automatisme du barrage a abaissé les bouchures afin d’endiguer la hausse du plan d’eau. À t+6 min, le niveau amont a atteint son maximum, à +8 cm au-dessus de la valeur de consigne. Entre t+6 min et t+30 min, le niveau amont est abaissé progressivement afin d’éviter tout risque de sur-compensation ou de phénomène oscillatoire. À t+30 min, le niveau amont a été ramené à 3 cm au-dessus de la cote de régulation.

Conclusions
Les expériences et observations menées depuis maintenant près de 2 ans sur le site de Saint Joseph tendent à montrer :
- Que le barrage gonflable à l’eau constitue une vraie solution technique pour les sites de navigation, pouvant répondre à un cahier des charges exigeant notamment en termes de capacité de régulation.
- Que dans les débits courants, le barrage gonflable de Saint Joseph, accompagné ou pas d’une centrale hydroélectrique, permet une régulation fine de la ligne d’eau. Selon une moyenne horaire, les enregistrements du mois de mars 2019 montrent une plage de fonctionnement autour de la cote de retenue normale de +/- 2 cm.
- Dans des phénomènes hydrauliques spécifiques, notamment liés à un découplage brutal de la centrale hydroélectrique, l’utilisation d’instructions d’automatisme particulières permet de limiter les élévations du plan d’eau à moins de 10 cm.

Au global, par l’emploi de solutions techniques adaptées, tant sur les organes constitutifs du barrage (bouchure, tuyaux, organes de régulation, etc) que sur les automatismes, le site de Saint Joseph prouve qu’un barrage gonflable à l’eau peut assurer des performances de régulation similaires à celles de barrages traditionnels, par exemple de type clapet métallique.
Conception standardisée sur les itinéraires Aisne et Meuse

Introduction :

En 2004, Voies Navigables de France (VNF) a engagé un programme de reconstruction de barrages mobiles répartis sur l’ensemble de son réseau afin de remplacer d’anciens dispositifs manuels de type Poirée ou Hausses qui présentent des conditions de travail pénibles et à risque.

En 2011, VNF lance un seul et unique projet concernant 31 barrages sur l’Aisne et la Meuse, pour répondre à des enjeux de massification, standardisation et optimisation de la gestion des deux bassins versants sous la forme d’un Partenariat Public Privé (PPP).

Figure 1: Implantation des barrages objet du contrat, à remplacer et à exploiter (source BAMEO)
En 2013, un contrat de partenariat public-privé est signé avec BAMEO pour remplacer, sur les itinéraires Aisne et Meuse, 29 barrages avant 2020 et exploiter un parc de 31 barrages pendant 30 ans. La réalisation du projet est confiée à COREBAM qui regroupe plusieurs entreprises de VINCI Construction. Un groupement conception est constitué, regroupant plusieurs entités : BR Lingénierie mandataire, Hydrostadium, ISM Ingénierie et Ateliers 2/3/4/.

Ce projet permet en un seul acte la mise en œuvre d’un linéaire de 2.5 km de bouchures gonflables, ce qui constitue un record mondial. Il permet également la création de centrales hydroélectriques au fil de l’eau sur trois des barrages. BR Lingénierie, acteur de référence dans le domaine de la navigation tant en France qu’à l’Export depuis plus de 60 ans, assure la conception de ces barrages. Cette conception est réalisée dans un contexte contractuel exigeant avec un objectif fort d’une standardisation d’une chaîne de barrages comme cela n’a jamais pu être effectuée auparavant.

1) Le barrage gonflable : une technologie modulable adaptée à la standardisation

Les bouchures gonflables se composent d’une membrane ancrée, dans notre cas, au moyen de deux rails au droit d’un radier en béton permettant de loger les conduites de remplissage et de vidange (Figure 2).

La conception même des membranes, constituées de couches d’élastomère et de couches de textiles (polyester ou polyamide) qui garantissent la résistance à la traction, permet de moduler les largeurs et hauteurs de bouchure au moyen d’un certain nombre de lés soudés entre eux de manière longitudinale à l’écoulement.

Pour la reconstruction des 29 barrages, les largeurs des 75 passes à créer ont été catégorisées dès l’origine du projet en 5 largeurs type variant de 16,80 m à 34,80 m. Douze classes de hauteur, variant de 1,75 à 2,86 m, ont été déterminées et rationalisées pour conduire à la production de seulement 3 types de membrane (Figure 3).

Dès les études, cette standardisation des largeurs et hauteurs de passe a permis de standardiser les rails d’ancrage selon 4 classes en fonction des hauteurs de bouchure et pour des éléments fixes de 1,2m de long, ainsi que les radiers et piles selon les 12 classes de hauteurs.

1 filiale de VINCI Concessions SIEMA et Meridiam
2) Une conception de 29 barrages ou 29 conceptions de barrage ?

La standardisation des barrages présente un intérêt majeur du projet et doit être pris en considération dès les premières ébauches de conception. Au-delà du gain de temps, elle permet de mener des réflexions à grandes échelles pour affiner ensuite la conception site par site.

Les études hydrauliques peuvent être menées sur tout le cours d’eau puis être affinées sur un site en particulier selon les problématiques. Au stade « faisabilité », de telles études permettent de pré-dimensionner les ouvrages, et de vérifier/valider les grandes orientations stratégiques quant au phasage du chantier, ce qui permet de mieux appréhender les coûts de l’opération et ainsi de minimiser les risques. Les études hydrauliques s’appuient alors sur une modélisation filaire. Puis, il faut s’assurer que le projet dans son ensemble ne conduira pas à une augmentation du risque inondation sur le territoire, aussi bien une fois l’aménagement achevé que durant la durée des travaux. Il est alors nécessaire de mettre en œuvre une modélisation plus complexe (Figure 4) qui intègre le lit mineur et le lit majeur des cours d’eau étudiés, ainsi que des modèles locaux fins bidimensionnels qui permettent de déterminer avec précision les pertes de charge engendrées par les nouveaux ouvrages en état aménagé, mais aussi en phases travaux.

Les études géotechniques peuvent se recouper d’un site à l’autre et permettent d’optimiser les campagnes et les interprétations. Le type de fondations est déterminé par groupe de barrages selon la nature du substratum (rocheux ou marneux) et sa profondeur. Ainsi, les parafouilles sont tantôt constituées (Figure 5) de rideaux de palplanches suffisamment ancrés dans les alluvions pour limiter le risque d’érosion interne ou battus au refus dans le substratum, tantôt constitués de béches en béton dans la continuité du radier.
La standardisation concerne également la conception des locaux techniques, seules émergences architecturales des ouvrages. Rapidement, l’architecte du projet, Ateliers 2/3/4/, propose des parois avec des panneaux de béton matricé préfabriqués (Figure 6), capables de se placer dans un positionnement inversé pour limiter le nombre d’éléments coffrants. Ce choix a engendré une taille standard des locaux techniques pour une grande partie des barrages qu’ils soient équipés de 2 ou 3 bouchures.

Enfin les réflexions sur les passes à poissons sont mutualisées sur l’ensemble des barrages par Hydrostadium, que ce soit sur la typologie de la passe à poissons, la taille des bassins, la position de la passe à poissons par rapport aux barrages, les interfaces par rapport aux barrages, les équipements, ...

3 ) Des gains significatifs en travaux

La standardisation prend ensuite tout son sens pour les entités de VINCI Construction pendant les travaux. Tout d’abord d’un point de vue planning puisque les 29 barrages séparés en 4 groupes ont été réalisés entre avril 2015 pour se terminer en avril 2019. Chaque groupe comportant 7 à 9 barrages était phasé sur 2 années hydrauliquement favorables (avril à novembre). Chaque année, un groupe démarrerait et le précédent se finissait, avec la pose 8 membranes la première année, puis 27, 23 et 17 jusqu’en 2019.
La proximité géographique des sites (quelques kilomètres séparent chaque site) permet de mutualiser les moyens qu’ils soient humains (retour d’expérience pour les méthodes d’exécution, ateliers de travaux pouvant intervertir d’un barrage à l’autre selon les avancements, gestion des aléas mieux appréhendé) dans un objectif de gain de temps ou matériels (optimisations d’aménées et replis des engins de chantiers, réutilisation des outils coffrants, commande de matériels,...).

Figure 7 : Photo d’un des 12 outils coffrants pour les piles des barrages—BRLingénierie

4 ) Une exploitation grandement simplifiée

L’exploitation courante, assurée par la société SeMAO, est facilitée pour le personnel par les similitudes entre les barrages que ce soit au niveau de l’IHM, des procédures de maintenance, du matériel, ..., le tout avec des moyens de télégestion optimisant encore les opérations.

Les opérations de batardage sont souvent complexes et couteuses pour les barrages mobiles en rivière : les moyens nécessaires sont lourds et la mutalisation des batardeaux sur plusieurs ouvrages réalisés lors d’opérations séparées est souvent illusoire. Ici, le batardeau a été conçu dès le début des études pour être utilisé sur plusieurs barrages, avec une modularité qui répond aux spécificités de chaque site.

Le principe de batardage retenu consiste en la mise en place d’un jeu d’éléments porteurs constitués de poteaux en HEB galvanisés ancrés dans les réservations du radier (Figure 8). Ces éléments verticaux sont disposés avec un espacement identique sur tous les barrages. Ils sont complétés in situ par des éléments de remplissage par plaques en aluminium identiques à tous les barrages, empilées et glissés dans les rainures des HEB. En élévation, le principe de batardage amont est représenté par la vue ci-dessous:

Figure 8 : Schéma de principe du batardage de maintenance – VINCI Construction

Avec ce principe inspiré des barrages à aiguilles préexistant, seuls les éléments d’abouts au droit des piles varient selon la classe de longueur et hauteur de la bouchure.

Le batardeau de maintenance n’est qu’un exemple de tout le bénéfice que peut apporter la standardisation. La mise en service des derniers barrages en début d’année 2019 et les 25 ans d’exploitation restants à Semao permettront de mettre ce concept à l’épreuve. Concept qui a pu trouver avec ce projet de reconstruction des barrages de l’Aisne et de la Meuse une occasion unique en France de pouvoir s’appliquer sur un itinéraire complet de barrages.
Title:
Hydraulic loads on bottom constructions behind a storm surge barrier or weir: development of a rapid assessment method

Introduction

Sea level rise will lead to higher maximum water levels to be considered when assessing the structural safety of storm surge barriers or other sea defences. As the sea level rises, so will the water level in the lower sections of rivers, which implies that also many smaller constructions along rivers, such as locks and sluices, will face higher extreme water levels. One of the hydraulic loads to be considered is the load on the bottom construction caused by the overflow and overtopping over the structure. Failure of the bottom construction could lead to a scour hole posing a threat to the structural integrity of the structure. When considering a large number of structures in various scenarios of sea level rise, a need arises for a method for rapid assessment of the hydraulic load on the bottom construction in relation to the strength of that construction.

The hydrodynamic situation of the flow in the lee of an overflowing storm surge barrier is identical to that of an overflowing weir, where, when designed properly, a hydraulic jump occurs directly downstream of the weir. In this location a stilling basin is present with a very strong bottom construction in order to handle the loads as caused by the hydraulic jump. Bottom constructions behind a storm surge barrier, lock or sluice, may not be designed to handle these loads, especially when these would increase due to sea level rise.

The method under development aims to use existing formulae describing hydraulic jumps to estimate bottom velocities that can be linked to formulae describing the strength of a bottom construction.

Approach

The hydrodynamic situation above the bottom construction behind a hydraulic structure can be characterized by the ratio between the required (minimum) water depth for a hydraulic jump, $d_r$, and the available water depth, $d_a$. Three situations can be discerned (see Figure 1), which apply for both overflow and underflow:

- when the available water depth is much larger than required for a hydraulic jump, $d_r/d_a < 1$, a submerged or drowned hydraulic jump occurs,
- when the available water depth is equal to the depth required for a hydraulic jump, $d_r/d_a = 1$, a hydraulic jump will occur close to the construction,
- when the available water depth is less than required for a hydraulic jump, $d_r/d_a > 1$, the hydraulic jump will occur at some distance from the construction, with supercritical flow occurring between the construction and the hydraulic jump, where the flow loses energy until the conditions are met for the hydraulic jump to occur.
The essence of the approach described in this paper is that, as will be explained, it is possible to calculate (an estimate for) the ratio \( \frac{d_r}{d_a} \) from known variables:

- the water level (energy height) upstream of the construction,
- the height of the construction,
- (an estimate of) the discharge coefficient of the construction,
- the water level downstream of the construction,
- the bottom level downstream of the construction.

The next step in the approach is the use of this parameter, \( \frac{d_r}{d_a} \), as an indicator of the degree of flow concentration along the bottom, which can be expressed as a ratio between the velocity at the bottom, \( v_b \), and the depth averaged velocity downstream, \( v_{da} \). Note that \( v_{da} \) can also be calculated from the known variables. This ratio is defined as \( c_b = \frac{v_b}{v_{da}} \).

Thus, the aim of the research is to find the relation between \( c_b \) and \( \frac{d_r}{d_a} \). This relation will make it possible to determine an (estimate for) the bottom velocity from known variables: calculate \( \frac{d_r}{d_a} \), determine \( c_b \) and then find the bottom velocity as \( v_b = c_b \cdot v_{da} \).

As Figure 1 indicates, both a flow over the crest of a construction (weir or storm surge barrier) and a flow under a gate can lead to similar situations of (submerged) hydraulic jumps with or without supercritical flow. This makes the approach equally applicable to the assessment of discharge sluices or lifting gate type weirs.

**The hydraulic jump relations**

For the situation where \( \frac{d_r}{d_a} = 1 \), the hydraulic jump can be described by formulae as presented in various handbooks, such as by Ven Te Chow (1959). For the present purposes, use is made of the presentation developed by Kolkman (1989), which aims at finding the required depth of a stilling basin, \( d_r \). In his presentation, Kolkman uses the critical depth, \( d_c \), to make all other length parameters dimensionless. Using the parameters as defined in Figure 2, these formulae read as follows.

The discharge, \( Q \), over a crest or under a gate can be calculated (or estimated) using Eq. (1) using a known (or estimated) discharge coefficient \( C_D \). Based on this discharge the critical depth, \( d_c \), can be calculated using Eq. (2). This critical depth is used to normalize the head loss in the hydraulic jump, Eq. (3): \( \Delta H = (H_0 - d_c) = (H_{in} - d_r) \) with \( H_{in} \) the energy height of the flow going into the hydraulic jump and \( d_r \) the water depth downstream, both relative to the bottom under the hydraulic jump.

![Figure 1: different hydraulic situations behind a structure, characterized by the ratio between the required water depth, \( d_r \), and the available water depth \( d_a \), both relative to the water level downstream. Note that in all three cases as drawn here \( d_r \) is the same as the drop in water level over the construction and the discharge are the same. Only \( d_a \) is varied by changing the bottom level.](image)

![Figure 2: definition of parameters](image)
jump. In doing this we introduce the approximation
that \( H_{in} = H_0 \), which implies that the flow would
not lose energy along its path from upstream of the
crest towards the hydraulic jump.

Both terms of the normalised head loss can also be
computed from the Froude-number of the flow
going into the hydraulic jump, defined by Eq. (4).
In this way, Eq. (3) transforms into Eq. (5), and
from this equation, the Froude number can be
solved by iteration. From this Froude number we
can calculate \( d_r \) using Eq. (6).

It is important to note that in this approach, \( d_r \)
can be calculated from known variables only.

The computational method

When considering a situation where \( d_r/d_a < 1 \), a
drowned hydraulic jump, the same formulae can
still be applied to calculate \( d_r \). The head loss, \( \Delta H \),
follows from the boundary conditions at both sides
of the structure, \( H_0 \) and \( h_2 \), which will be available
relative to a local reference level, see Figure 3. For
the calculation of \( d_r \) we use a different reference
level (a level which is a distance \( d_r \) below the water
level downstream), but regardless of this difference
in reference level, we can use the same formulae
and calculate \( d_r \). With \( d_{in} \), we find \( d_r/d_{in} \), the parameter we were looking for.

Having found \( d_r/d_{in} \), we now look at the bottom
velocities. From the Froude number \( F_{in} \) and other
known variables, we can calculate the velocity in
the jet going into the hydraulic jump \( v_{in} \) using Eq.
(7). Contrary to our initial approximation, some
energy will be lost while the jet travels from the
crest towards the bottom, and thus, also for \( d_r/d_a = 1 \), the velocity at the bottom will be lower than
\( v_{in} \) \( (v_b < v_{in}) \) In this way \( v_{in} \) provides an upper
limit for \( c_b \), as can be calculated using Eq. (8),
again from known variables only.

In conclusion, for a given situation of overflow over a construction, we can calculate \( d_r/d_a \) and an upper
limit for \( c_b \). For actual values of \( c_b \), as input to the bottom velocities to be expected, we now need
measurements of bottom velocities.

Exploratory measurements for overflowing weir

In a flume at Deltares exploratory measurements have been performed with aerated overflow over a simple
rectangular weir. These tests had a range of purposes, one of which was the exploration of various flow
measuring techniques including Particle Image Velocimetry (PIV).

In these measurements a 2-D PIV technique was applied. As expected, these measurements were only
successful in flow conditions with a limited discharge as air entrainment, increasing with the discharge,
compromised the quality of raw PIV images. Fluorescent seeding particles and a lens filter were used so that
bubbles were masked and only the particles were recorded.
The results from the PIV-measurements, see for instance Figure 5, consisted of time-averaged velocity maps capturing the jet and the region where the jet reaches the bottom of the flume.

The results confirmed what was clear from visual observations, Figure 4, being that even at very low discharges, the jet of overflowing water reaches the bottom of the flume.

As Figures 4 and 5 show, the bottom velocities, \( v_b \), are much larger than the depth averaged velocity, \( V_{da} \), although not as large as \( v_{in} \) in Eq. (7). This is due to the transfer of energy to the surrounding fluid via eddies at both sides of the jet.

The results of these exploratory tests, presented in the framework of the formulae as described in this paper, are shown in Figure 6. The graph presents the value of \( c_b \) against \( d_r/d_a \); three points derived from the (exploratory) PIV-measurements and the general trend that is expected. Also shown is the upper limit of \( c_b \) being \( v_{in}/V_{da} \).

This graph presents the essential relationship in the rapid assessment method, to be further established for all values of \( d_r/d_a \).

Already, the results indicate that, for instance when considering overflow over the gate of a navigation lock, the velocities over the bottom construction can be an order of magnitude larger that the depth average velocity, which confirms that this depth averaged velocity is not a suitable parameter to estimate the hydraulic load on the bottom construction.

Further work

In continuation of this research, further measurements will be done, applying different measuring techniques, to cover the complete range of \( d_r/d_a \). Furthermore, different test series will be done in order to establish whether the same curve applies to different combinations of parameters leading to the same value of \( d_r/d_a \). Also, a similar curve for flow under a gate will be established. Finally, for application of the method, also information on the decay of the jet further away from the construction will be gathered.

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Réf. auteur:
X.Cherradi – ARTELIA Eau et Environnement
6 rue de Lorraine 38130 Echirolles, France
xavier.cherradi@arteliagroup.com

Co-auteurs:
G.Top – ARTELIA Eau et Environnement
6 rue de Lorraine 38130 Echirolles, France
gtp@arteliagroup.com

C.Candelon – ARTELIA Eau et Environnement
6 rue de Lorraine 38130 Echirolles, France
christine.candelon@arteliagroup.com

C.Guilbaud – Voies Navigales de France
175 rue Ludovic Boutleux 62408 Béthune, France
clothilde.guilbaud@vnf.fr

V.Morice – Voies Navigales de France
175 rue Ludovic Boutleux 62408 Béthune, France
vincent.morice@vnf.fr

Mots clés:
diagnostic simplifié, aide à la stratégie d’investissement, approche fonctionnelle

Title:
Inventaire et qualification du patrimoine infrastructures
de Voies Navigables de France
Voies Navigables de France (VNF) a décidé de parfaire la connaissance interne de son patrimoine infrastructures national afin d’optimiser sa politique de maintenance et d’investissement. VNF a mandaté pour cela Artelia Eau & Environnement pour l’assister dans l’inventaire physique et la qualification fonctionnelle de ses quelques 6000 ouvrages et 6700 km de voies d’eau, en évaluant, au-delà des problématiques structurelles, les capacités de chaque ouvrage à assurer les fonctions auxquelles il est dédié. Cette étape est appelée la qualification de l’état fonctionnel.

L’outil existant utilisé, la Base de Données Ouvrages (BDO) interne à VNF, a pour objectif fondamental de capitaliser les données du patrimoine infrastructures issues de divers territoires dans un outil centralisé, commun et partagé de tous au sein de VNF.

Cet outil informatique d’aide à la décision permet de préserver, fiabiliser et valoriser le parc d’infrastructures dans son ensemble, afin d’assurer un niveau homogène de service et de sécurité des personnes sur un itinéraire de navigation. À travers l’état fonctionnel des ouvrages, il contribue à l’amélioration du réseau et à la programmation des investissements.

La BDO est enrichie grâce à la réalisation de visites d’ouvrages en fonctionnement, généralement en eau. Ce sont des visites réalisées sur une courte durée, de l’ordre d’une à deux journées y compris le temps de saisie dans l’outil, qui ont pour but de relever sur le terrain la décomposition d’un ouvrage ainsi que les désordres observés et leur degré d’importance. Les visites reposent essentiellement sur des constats visuels. Elles sont mises en œuvre par des personnes préalablement formées à la méthode, disposant d’une certaine connaissance des ouvrages hydrauliques et notamment des ouvrages spécifiques à la navigation. La méthodologie retenue est rapide et simple puisqu’elle est prévue pour être appliquée par une seule personne, qui a fortiori n’est pas spécialisée dans chacun des domaines techniques concernés par un ouvrage de navigation (génie civil, géotechnique, hydraulique, mécanique, électricité et asservissement, ...). Les personnes en charge des visites ont été formées pour les aspects théoriques de la méthodologie (1j) et pour la mise en pratique (2jrs) afin d’harmoniser l’application des visites à l’échelle du territoire.

Bien qu’elle recoupe d’autres types d’inspection, la méthodologie de visite n’est ni un diagnostic, ni une évaluation exhaustive des travaux à réaliser, ni une GMAO (Gestion de la Maintenance Assistée par Ordinateur). Elle ne se substitue pas aux contrôles et suivis imposés par la réglementation ou par des impératifs de sécurité.

La mise au point de la méthodologie de visite, définie spécifiquement pour les ouvrages ponctuels et linéaires de VNF, a été élaborée conjointement par VNF et ARTELIA, et a permis l’élaboration de guides et de fiches types de visites. Cette méthodologie a fait l’objet de tests in situ avant déploiement sur le territoire couvert par VNF. Inscrite dans un processus d’amélioration continue, la méthode est régulièrement enrichie et améliorée à partir des retours du terrain.

Elle consiste à décomposer chaque ouvrage en parties d’ouvrages, elles-mêmes décomposées en équipements fonctionnels (cf. fig.1 et fig.2). Un équipement fonctionnel est un organe ou élément d’un ouvrage assurant une ou plusieurs fonctions précises.

La liste des équipements pour chaque ouvrage a été établie dans l’objectif d’être exhaustive et avec un niveau de détail suffisant pour assurer la complétude de la visite.

Dans le cas d’une écluse, on retrouve des équipements concernant aussi bien le mode d’exploitation et de commande, le type d’énergie utilisée sur l’ouvrage que des équipements spécifiques à chaque type de porte (vantail/porte, articulation/roulement et guidage, superstructure, ...).

Le principe de la structure de BDO est décrit ci-dessous et développé sur une écluse à portes busquée.
La première étape, lorsque l’ouvrage n’a jamais fait l’objet de visite, consiste à effectuer un inventaire des équipements existants sur l’ouvrage. Pour les ouvrages ponctuels (barrages, écluses,…), il s’agit d’identifier le type de porte, le type d’organe de manœuvre, l’existence de batardeau, … Pour les ouvrages linéaires (diguès, berges,…), il s’agit d’inventorier les défenses de berge, les talus en remblai, … en y renseignant la dimension spatiale (longueur et Points Kilométriques) de l’équipement.

Une fois l’inventaire réalisé (ou lors de la mise à jour d’une visite), le relevé et la notation des désordres s’effectuent au niveau des équipements fonctionnels dans le but d’identifier une incapacité partielle ou totale de l’équipement à remplir son rôle. Cette notion est particulièrement importante, les ouvrages étant parfois dans un état structurel dégradé sans pour autant remettre en cause le fonctionnement à court ou moyen terme. Le relevé des désordres tient compte de l’ampleur pour les ouvrages ponctuels (nombre de bollards, ratio de surface de bajoyer ou de linéaire d’étanchéité, …) et de la longueur pour les ouvrages linéaires (proportion de palplanches).

La méthodologie prévoit la notation type des désordres pour 3 catégories principales : la notation des désordres concernant les structures fixes (génie civil, charpente, … - durée de vie de l’ordre de 50 à 100 ans), les structures mobiles (articulation, organes de manœuvre, étanchéité, … – durée de vie de l’ordre de 5 à 30 ans), les désordres autres (pannes, défauts de conception, absence et manques, …). Les éléments dont la durée de vie est plus faible sont généralement traités dans le cadre de l’entretien/maintenance courant des ouvrages. Ils font l’objet d’une procédure de Gestion Maintenance à part (ex : changement d’huile dans les groupe hydraulique, changement de joints, …).

Quatre niveaux de notation d’un désordre ont été établis (cf. fig.3) : 1 pour un désordre faible, jusqu’à 4 pour un désordre élevé (ruine).
Le principe est présenté ci-après pour les structures mobiles.

Les équipements sont associés selon leur rôle à une ou plusieurs des quatre fonctions retenues pour la qualification de l’état des ouvrages de VNF :
Fonction Maintien du plan d’eau,
Fonction Navigation,
Fonction Sécurité des personnes,
Fonction Pérennité.

<table>
<thead>
<tr>
<th>Niveau de détérioration</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
</table>
| Faible                  |   |   |   | Faible
| Mineur                  |   |   |   | Mineur
| Majeur                  | N’occasionnant pas de gêne dans l’accomplissement de son rôle |   |   | Majeur
| Électe                  | Occasionnant ou susceptible d’occasionner une gêne dans l’accomplissement de son rôle (pour la navigation ou pour le maintien du plan d’eau ou pour l’environnement, ou pour la sécurité) |   |   | Électe

| Alteration superficielle liée à l’usure (stallissement, chocs, ...) | Alteration importante du matériau | Dégradation de la structure, des éléments de fixation, d’ancrage, ... (résistance, déformation, ...) | dégradant son rôle | Capacité de résistance ou de fonctionnement atteinte ou proche de sa limite empêchant de réaliser partiellement ou totalement son rôle |
|présentant un risque pour la pérennité de la structure | |  |  |  |

Fig. 3 – Définition des niveaux de détérioration appliquée aux structures mobiles

Les données sont renseignées dans l’outil BDO qui détermine l’état d’un ouvrage au moyen d’un Indicateur d’Etat Fonctionnel (IEF – cf. fig.4) ainsi que le degré d’accomplissement de chaque fonction.

<table>
<thead>
<tr>
<th>Classe I d’état fonctionnel</th>
<th>Classe II d’état fonctionnel</th>
<th>Classe III d’état fonctionnel</th>
<th>Classe IV d’état fonctionnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_{EF} &lt; 1$</td>
<td>$1_{EF} &lt; 2$</td>
<td>$2 &lt; I_{EF} &lt; 3$</td>
<td>$I_{EF} &gt; 3$</td>
</tr>
<tr>
<td>Ouvrage en bon état fonctionnement</td>
<td>Ouvrage en état de fonctionnement acceptable</td>
<td>Ouvrage en état fonctionnel moyennement dégradé</td>
<td>Ouvrage en état fonctionnel fortement dégradé</td>
</tr>
<tr>
<td>= pas ou peu de désordre</td>
<td>= désordres de faible importance</td>
<td>= nombreux désordres, en majorité de faible importance</td>
<td>= désordres importants par leur quantité et leur dégradation</td>
</tr>
</tbody>
</table>

Fig. 4 – Définition générale des niveaux de détérioration


L’exploitation de l’outil se fait ensuite à l’aide de requêtes SQL et de manière combinée avec les trafics fluviaux, les enjeux de sécurité, de gestion hydraulique, de conformité environnementale, et constitue une aide à la décision stratégique des investissements de VNF.

La méthode mise en œuvre n’est pas adaptée à l’évaluation fine de l’état d’un ouvrage en particulier mais permet l’évaluation de l’état fonctionnel d’un nombre important d’ouvrages (itinéraire, canal, direction territoriale, territoire français). Une évolution de la méthode consisterait à mettre à jour les matrices sur la base d’analyses statistiques des résultats obtenus à grande échelle (itération du modèle). Cette mise à jour permettrait de réaliser une évaluation “relative” des ouvrages (comparaison entre eux) affinant ainsi la stratégie d’investissement de VNF.
Abstract:
Unauthorized intervention is understood as any action carried out by third parties with or without intention to cause damage but that, at the end, reduce the service life or take the infrastructure elements and facilities to an “out of service status” or a worsened or not desired condition.
Experiences have been collected in several variety of fields of action related to Inland Waterways management and maintenance. This Unauthorized Intervention actions have a strong effect in security and safety, as they may affect the infrastructure itself or the electronic devices deployed for River Information Services targeted to safe navigation conditions. Unauthorized Intervention actions are quite common in South American countries, and may also be prevalent in some other developing countries, at least.
In this paper some experiences will be exposed, as well as the solutions that have been carried out, mostly with positive effects and with operative consequences in some cases.

1- First Aids to Navigation Designs. Vandalism. Maintenance costs increase
a) Buoys and Beacons are deployed along the 1654 km of concessioned navigable inland waterway in Argentina. In fact, 960 buoys and 165 beacons make the navigation safe 24 hours a day, 365 days a year. Maintenance is a must. But as buoys and beacons are built with high quality materials and the electric equipment has high standards of quality, maintenance is not (or should not be) a problem. However, vandalism is a big problem that affects the structure itself and electric equipment as well. It has been a game of wits played between the Inland Waterway administrator and the spoilers. Some answers were easily found. Some others not.
Damages to infrastructure itself were the stealing of the buoys towers. The complete tower was stolen, with all the equipment installed. A buoy without its tower turns out to be danger to navigation, as it becomes a floating structure without lights or any kind of signal.

An effective solution was the design of "security devices" that are placed in the fixing points of the buoy.
Security devices for tower fixing

These devices turn out to be effective against vandalism. The point is that each tower assembling and disassembling process takes a 20% more time than a traditional maintenance process.

b) Some years ago lighting systems were deployed in a basis of solar panels, cables, batteries and the lantern itself. Solar panels and batteries were the target of robbers. First step was resigning the autonomous quality of a battery-solar panel design and create a “no-solar panel” tower, with batteries inside a high reinforced steel box.

No solar panel tower- High reinforced battery box

This has been an effective solution but, again, maintenance costs were significantly increased. Aids to Navigation devices subject to vandalism are those located far from big cities, were the Administrator operational base is placed. Maintenance costs due to navigation distances (up to 250 Km) were highly increased. Batteries had to be replaced every three or four months, as they lost their autonomous condition. Furthermore, towers were designed in a way that the battery replacement had to be made in a special way: loading the whole tower with the crane and placing the battery in a safe and protected structure.

A second step in the tower design - always looking for the lost autonomous condition - was the addition of an antivandalism structure.
This structure has been used for about 10 years with positive effects. A steel reinforced box, with a protected solar panel and battery by a special security glass, was designed to guarantee an autonomous system. Despite the security glass, some of them were completely destroyed and batteries and electronic devices were stolen.

2- The self-contained lanterns. New challenges. A social point of view
Technology evolved and the previous battery-solar panel-lantern design turned to be the self-contained lantern. The small 17Ah 12V capacity batteries that these self-contained lanterns have, are not useful for private uses (such as domestic power supply). This is a positive aspect. Batteries of these lanterns were stolen at the beginning, but less vandalism cases were registered afterwards. The first reason is the one mentioned before: batteries are not so useful for domestic purposes. The second reason is the special designed security device developed for these lanterns.

![Security device for self-contained lanterns](image1)

The question that arose was: what was the aim of stealing these small and low capacity lanterns? Batteries are small. Surprisingly, one of the objectives of stealing these lanterns was to get the leds the lanterns contained to deploy a self-design signalling system for the fishing lines. The solution was quite easy: free leds were supplied to fishermen in order to avoid them stealing the lanterns.

3- The wave gauge. A case of systematic unauthorized intervention.
Hidrovia S.A. Concessioner of Rio de la Plata & Parana rivers Waterway has deployed a wave rider in the outer section of Rio de la Plata river since 1996. The position is quite strategic as it is located at the last section of Rio de la Plata river, the outer point of the Inland Waterway. This wave gauge is used for statistical studies and for River Information Systems. In the last years, because of some unknown reasons, it has been removed from this position and located, for instance, near the Uruguayan coast. Last time it was removed, an Automatic Identification System (AIS) data analysis was carried out in order to know the reasons for this lost. This study showed that a fishing vessels tandem hooked the wave gauge and took it out from the position. This was the starting point to carry out an effective solution.

![Two fisherboats "fishing" the wave gauge](image2)
When the wave gauge was relocated, an additional buoy was placed 150 m far from it. The buoy has an AIS transponder equipment installed. It broadcasts an AIS signal with its own position, the wave gauge position itself and a field of 4 other virtual signals around the wave gauge, creating an AIS Fence. From now on, the wave rider is plotted on the AIS vessels screens.
Moreover, frequent meetings organized by the Concession Company and the Uruguayan stakeholders were held in order to inform and remind the importance of the wave gauge and the importance of protecting it and keeping in its right position.
According to the results, it seems that this has been an effective solution.

4- Birds. Another case of unauthorized intervention?
Can we call birds action an unauthorized intervention on Inland Waterway infrastructure? Birds do not ask for authorization... But we can call them “unexpected” interventions in any case.
And one of these unexpected interventions is the action of birds’ depositions on Inland Waterways facilities. In the upper section of the Parana River called Parana Medio, birds’ depositions cause a great problem in buoys. They make them change completely their colour, turning them into white.
In IALA recommendations, colours of signals are one of most important characteristics of Aids to Navigation identification during daytime. The lack of colour affects identification of signals.

[Image: Antigraffiti paint effect on buoys (before – after washing)]

Depositions are not easily removed and they remain for a long time and tend to degrade the painting coats. Even more, extreme sun action make the problem even worse. Solutions were targeted towards two goals: preventing birds approaching buoys and, while accepting them approaching to buoys, focus on solving the depositions effects. The first goal could be solved by electronic or natural scarecrows, but again, they will be subject of vandalism (even more, batteries and solar panels will be needed for electronic ones).

The second one was effectively solved by applying anti-graffiti paint on the buoys surface. It has been a quite affordable solution and easy to be deployed.
Antigraffiti paint makes deposition easier to be washed away, and washing times have been reduced significantly.

Some final notes
Unauthorized intervention was a problem in the past, is a current problem and for sure it will be in the future. Some of them are solved through technical procedures, some others are solved by social ones. Experiences show that results do not come only from technical analysis and security policies, but also from social interchange involving stakeholders in everyday Inland Waterway issues.
Thinking beyond, short time alleviative actions will show the path to long term solutions. It is relevant to show and discuss experiences and solutions carried out in different countries and regions. For sure, some Inland Waterways administrators should face Unauthorized Intervention actions that may have been faced in some other countries.
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Ref. author:  
Chris Willcox – Tetra Tech  
400 112th Ave NE, Suite 300, Bellevue, Washington, USA  
chris.willcox@tetratech.com

Co-authors:  
Brad Houzenga – USACE, Rock Island District  
25549 182nd Street, Pleasant Valley, Iowa, USA  
brad.e.houzenga@usace.mil.gov

Keywords:  
Tainter Gate, Rehabilitation, Design, BIM

Title:  
Mississippi Lock and Dam 21:  
Rehabilitation vs. Replacement for Submergible Tainter Gates

Abstract:  
As the locks and dams on the Upper Mississippi River approach and begin to exceed 85 years of use, the Rock Island District of the US Army Corps of Engineers faces major decisions on how to manage their aging steel spillway gates. Lock and Dam 21 near Quincy, Illinois has 10 submergible spillway Tainter gates that are reaching the end of their useful life. In 2017, the Rock Island District undertook the rehabilitation of one of the gates, replacing many of the deteriorated components and recoating the entire structure. When the final cost of that rehabilitation approached their estimated cost for the fabrication of a new gate, the District decided to let a contract for the design of replacement gates. Tetra Tech, working as a subconsultant to Anderson Engineering of Minnesota, received the contract and produced fabrication drawings and specifications for new gates. This paper describes the rehabilitation effort, the design of the replacement gates, and some of the factors that led the District to decide on new gates.

The rehabilitation work was performed in-place by installing bulkheads upstream of the gate and working off of floating plant underneath and downstream of the dam. In order to provide temporary support, the gate was lowered onto dunnage stacked on a barge underneath the gate. New parts were most often pre-fabricated based on a combination of field measurements and original construction drawings and then painted individually before being installed. Delays were frequently experienced due to high river flows preventing safe access to the dam. Furthermore, pre-fabricated parts often did not fit as expected causing field modifications that led to a general decrease in productivity. While the rehabilitation of three gates was planned, demobilization from the site occurred once the first gate was complete.

The design of the replacement gates had two primary constraints: the gates could weigh no more than the existing gates and, in order to maintain the same flow rates when submerged, they had to be the same shape. In addition, the new gates had to comply with current design codes, meaning that the design had to include loads such as trunnion friction that were not included in the original design, and they had to meet modern fatigue design criteria. The design scope of work also called for a hydrodynamic analysis to investigate the potential for vibration of the gates. In order to meet all the constraints, the new gates were designed as all-welded stressed-skin structures. The efficiency of this type of structure allowed the skin plates to be thickened relative to the original gates to improve their resiliency. The fabrication drawings were developed from a BIM model of the gate and included a nearly complete set of piece drawings.

In the end the decision to proceed with replacement gates came down largely to the fact that while rehabilitated gates would be cheaper, large parts of the gates would still be over 80 years old and not as robust as new gates.
INTRODUCTION

Lock and Dam 21 on the Upper Mississippi River was built in the mid-1930’s and is now over 80 years old. Due to its age, the Rock Island District of the US Army Corps of Engineers is considering what to do with the spillway Tainter gates. The spillway contains ten 64-foot wide by 22-foot high submergible Tainter gates that are the subject of this paper, along with three 100-foot wide roller gates. The gates can submerge up to 8-feet below the normal pool elevation or open upward to pass flows underneath.

The two options on the table were rehabilitation or replacement. Rehabilitation of the existing gates would replace damaged and worn parts but would leave the gate structure essentially it was. Replacement would allow for the new gates to meet modern design criteria, but at a higher cost.

REHABILITATION

In the Fall of 2016, Rock Island District personnel from its Operations Division, Maintenance Section mobilized to Lock and Dam 21. Materials were acquired for the rehabilitation of up to three Tainter gates, to include structural repair, blasting, and painting. Work was accomplished via water-based fleet including an 85-ton crawler crane working from a spud barge, several support barges, two tow boats and two articulating boom lifts for personnel access.

![Floating plant for rehabilitation work](image)

Each lock and dam on the Mississippi has a set of bulkheads that allow for flow to be diverted around a single spillway gate bay. Due to the relatively low head differential at the site, adequate depth exists such that fleet is able to set up immediately downstream of the bulkheads and under the spillway gate. A gate can then be lowered onto dunnage set up on the barge. This allows for safe access to the downstream side of the gate via the articulating boom lifts. There is not enough space on the barge between the gate and bulkheads to allow for easy access to the upstream side of a gate. Personnel and materials are shuttled back and forth via the support barges and tow boats.
Two approaches were identified to repair spillway gates. The first, used exclusively during past repairs, was to remove gate components and then fabricate replacement components using the existing component as a template. This approach is extremely reliable, unless the existing component is severely damaged, but is not economical as fabrication must take place in less than ideal conditions with restricted resources. The second approach was to fabricate replacement components in a shop prior to mobilization utilizing existing spillway gate drawings. These components can be installed immediately upon removing the respective damaged components. This approach is much more economical but often is not as reliable as dimensions cannot be easily field verified ahead of time. This was especially true at this site due to the age of the existing gates (circa 1930’s) as well as the district not possessing a complete set of shop drawings.

Due to the extreme and often unreliable conditions of working on a spillway gate in-place, it was decided to utilize the latter approach to the greatest extent possible. Unfortunately, due to inconsistencies in the contract drawings and the lack of shop drawings, many of the components fabricated ahead of time did not fit up in an acceptable manner and had to be re-fabricated or field modified. This caused significant delays in accomplishing the repairs to the first Tainter gate.

Another challenge to in-place spillway gate repairs are the ever-changing and unpredictable river conditions. Lock and Dam 21 is considered “run of the river”. Thus, as the flow on the river increases, spillway gates are raised to allow more water to pass. When a certain flood stage is reached, the spillway gates are raised completely out of the water. Once this occurs the fleet conducting repairs must be moved out of the dam as it cannot be safely moored. At that point repairs cannot be made until the water level decreases. From November 2016 to October 2017, the time period in which repairs were made to the first Tainter gate, this condition occurred over 143 days resulting in 5 different series of mobilizing and demobilizing the repair fleet.

The cumulative result of the ineffective repair strategy and unusual river conditions was a project that took much longer than anticipated and at a much higher cost. As a result of this and the fact that the funding to accomplish the work was going to expire, senior leaders made the decision to redirect the funding to a replacement strategy rather than repair.

**REPLACEMENT**

After the issues experienced during the gate rehabilitation project, the District took another approach, letting a contract to Anderson Engineering of Minnesota, and their subconsultant Tetra Tech, to design replacement gates and produce a set of fabrication drawings and specifications.

The contract stipulated two primary constraints: in order to utilize the existing hoist machinery, the new gates could weigh no more than the existing ones; and, in order to maintain the same flow rates when submerged, they had to be the same shape. In addition, the design was required to comply with the current requirements for design of hydraulic steel structures. These included loads not included in or increased from the original design, such as trunnion friction and ice impact, and consideration of vibration and fatigue. With
respect to vibration, the contract also required computational fluid dynamic (CFD) modeling of the gate, which was carried out by Alden Labs.

Because the new gates were required to be the same weight as the existing gates while resisting higher loads, a welded, stressed-skin configuration was chosen. The stressed-skin structural system makes more efficient use of the steel in the gate with the skin plate doing double duty as the barrier and the girder flanges. Effectively, the stressed-skin gate body acts as a tubular beam spanning between the end frames, whereas in the existing gates, girders with separate flanges span the width of the gate with no contribution from the skin plates. The new gate contains three horizontal webs and four internal vertical diaphragms, which help to maintain the shape of the gate body, redistribute loads and provide resiliency.

The most significant weight saving came from the reduction from four girders in the existing gates to three horizontal webs in the new ones. In addition, the stressed skin construction eliminated the need for flanges separate from the skin plate, and the welded construction allowed for the elimination of connecting angles. These weight savings made up for an increase in the weight of the arms, which was required due to the increased ice impact load and, more significantly, the inclusion of trunnion friction in the design. The weight savings in the interior of the body also allowed an increase in the thickness of the skin plates, increasing the resiliency of the gates and providing a corrosion allowance on the exterior of the gate body.

Another requirement of the contract was for the drawings to be developed from a BIM model of the gate, and this provision figured in the analysis of the gate as well. After initial hand calculations were used to determine the gate structural configuration and preliminary sizes, Autodesk Inventor was used to build a three-dimensional model of the gate that became the basis of the BIM model. The FEM analysis of the gate was carried out using Nastran In-Cad, a plug-in for Inventor that uses the Nastran solver with front- and back-ends integrated with Inventor. The Nastran results were checked both with hand calculations and with simplified models run in the program SAP 2000. One unexpected result of the FEM modeling was the very small amount of racking in the gate when it was supported by the hoist chains on one end only. The vertical deflection of ¼-inch was about one-third of what had been predicted by hand calculations that did not consider the torsional rigidity of the gate body.

![Diagram of replacement gate with downstream skin plate removed](image)

*Figure 3 - Isometric of replacement gate with downstream skin plate removed*

In order to produce the BIM model and the drawings, the Inventor model was imported into Microstation and the required data—part numbers, materials, specifications, etc.—were added to each of the parts of the model. In addition to Plans, Elevations and Sections of the gate, detail drawings were created for each
individual part of the gate, following a procedure the Rock Island District had used previously for a non-submergible Tainter gate.

CONCLUSION

In the end, the District elected to proceed with the replacement option for three main reasons: it limited the amount of in-water work, reducing the risk of delays; the off-site fabrication of the gates combined with the shorter in-water work means the upgrade can be completed in a shorter time frame, assuming funds are available; and, probably most significantly, the new gates are expected to have a longer effective service life than the rehabilitated gates, partially offsetting the higher cost of the replacement gates.

At this time, the design of the replacement gates has been completed and it is expected that a construction contract for the first set of gates will be put out for bid in 2020, pending appropriation of funds.
Life Cycle Investment Strategies and Operational Risk Exposure for Critical Assets on the McClellan Kerr Arkansas River Navigation System

Short Paper:
This presentation will focus on the development and implementation of the life cycle investment strategies using operational risk exposure for the fifteen locks and dams projects on the McClellan Kerr Arkansas River Navigation System (MKARNS). This paper is a continuation of the US Army Corps of Engineers efforts under their Asset Management Program to develop life cycle strategies as presented by Ellsworth and Patev (Ref 1. 2015) and Patev (Ref 2. 2015) at the Smart Rivers Conference 2015 in Buenos Aires, Argentina.

The lock and dam structures on the MKARNS are over 60 years old and seeing significant aging and deterioration of the 14,000 plus critical subcomponents that are part of their asset management inventory. This prototype project performed for the Little Rock and Tulsa Districts, US Army Corps of Engineers uses an Excel-based operational risk exposure tool to calculate the probabilities of failure (Pf), risk (hazard and current), mean-time-to-failure (MTTF) and remaining service life (RSL) which will guide and focus the life-cycle investment decisions of the MKARNS systems over the next 50 years.
The prototype life-cycle analysis allows the examination of the MKARNS system using both the condition of each component using their operational condition assessments as shown in Figure 2 and the risks of the critical components as they reach their tolerable MTTF and RSL limits where investments are needed to avoid any forced shutdowns and delays to navigation traffic. This is shown in Figure 3 with the investment zone highlighted in red to show which critical components enter this box over the investment period (i.e., 50 years). Figure 3 also shows the GIS layout of the MKARNS system with a pie chart showing the risk for the dam (yellow) and lock (green) that can be varied over the investment interval (this is shown at Year 15).

This paper presents the life cycle concepts will show the methodologies that are used in the ORE tool to calculate the important decision variables and the development of a user interface that focuses the decision-makers to visualize both the investment timing and level of funding required over time for the MKARNS system using a risk-informed decision process. The full paper will highlight key points using examples of critical components and how the life cycle strategies can be developed for these systems at navigation projects.

The ORE tool and interface show how the investments change by looking at how subcomponent to component to system levels are analyzed to determine when it is best to replace or repair and at what level. This roll-up is shown in Figure 4 from components to subsystems levels. In addition, the tool also will account for benefit-cost ratio (BCR) at various levels to inform the decision maker when the optimized time would be to make the
investment. This is shown in Figure 5 below as the difference (red lines) of the rehabilitation investment shows the benefits and risk reduction over time for a sample component.

Overall, the implementation of the life cycle process for MKARNS will help make risk informed decision for both maintenance practice as well as repair and replacement of critical components and systems. This approach will align well with USACE Asset Management practice in the future that will be applied over the entire portfolio of USACE inland marine transportation system (IMTS). The IMTS portfolio look will greatly assist the USACE in determining where their investments should be made and if current major rehabilitation policy should be changed to look at investments across a system instead of at the project level. As part of the presentation, a demonstration of the ORE tool will also be made to show how the interface greatly assists the decision makers in their ability to visualize trends and opportunities and assemble and optimize their best investment decision to budgetary constraints.

References:
Risk-based inspection and maintenance planning of miter gates

Abstract:

Risk-Based Inspection (RBI) has been utilized in the offshore industry, but it is rarely applied for inland navigation lock gates. This research presents a framework where Dynamic Bayesian network (DBN) is used for risk-based inspection planning of a miter gate considering inspection data. The study incorporates two sets of heuristic decision rules: inspections performed at regular time intervals and inspections performed when a certain annual probability failure threshold is reached.

1. Introduction

Miters gates are widely used in navigation locks due to its economical and aesthetic advantages. Due to variations of water levels during operation, miter gates are subjected to cyclic loading. These loads are primarily generated by differential water heads on both sides of a lock gate. Since water heads are not always the same for each lockage due to seasonal flows of the river different obtained stress ranges can be represented by an equivalent stress range [1]. The nature of fatigue damage is complicated because fatigue cracks can develop from unexpected initial defects coming from the manufacturing process. Inspection and maintenance are required to assure an adequate strength and serviceability of the structures. Inspection and repair of developed cracks are costly because the gate needs to be put out of service. Therefore, it is essential to perform a maintenance optimization. The proposed risk-based approach for inspection and maintenance planning is based on the Bayesian pre-posterior decision analysis [2]. RBI has been applied successfully for offshore wind turbines as described in [3]. In this study, a fracture mechanic model is used to predict fatigue crack growth of a miter gate welded joint based on an equivalent stress range. A DBN model is then employed to update the probability of failure whenever new information becomes available from inspections. In order to minimize the total expected
lifetime costs, various inspection and repair strategies are considered to be able to choose the one yielding lowest costs.

2. Fatigue crack growth model in miter gate

To model the fatigue crack growth, a fracture mechanics model is used. The most widely used model is the Paris law [4]. Assuming that the geometry function (Y) is constant, crack size at time t can be calculated as shown in Eq (1).

\[ a = \dot{a} t \]  

(1)

where \( a_0 \) (\( t = 1 \)) is the initial crack size, \( C \) and \( m \) are material parameters. An equivalent nominal stress range \( \Delta \sigma \) was used for fatigue analysis, \( n \) is the number of cycles per year, \( B_s \) is the load uncertainty and \( Y = 1.12 \) is the geometry parameter.

The failure event is defined by the limit state function:

\[ g = a_c - a_t \]  

(2)

where \( a_c \) represents the critical crack depth. The failure occurs if \( g \leq 0 \) and it is safe if \( g > 0 \). The input variables are summarized in the Table 1.

3. Dynamic Bayesian Network (DBN)

A DBN is a special class of Bayesian network, that represents the temporal evolution of variables over time. DBN was developed in the early 1990s by extending static belief-network models to more general dynamic forecasting models [5]. A DBN framework for stochastic modeling of deterioration process and updating the failure probability is proposed in [6]. Here, in order to use a DBN for the fatigue model, the time is discretized in intervals of 1 year. The variable \( q = \left( 1 - \frac{m}{2} \right) C B_s^m \Delta \sigma^m Y^n n^{m/2} \) (see Eq. 1) is defined in order to reduce the dimension of the joint distribution and consequently computational time. The crack depth at the end of each year can be expressed recursively as a function of the crack depth in the previous year as shown in Eq (3).

\[ a_{t+1} = a_t \]  

(3)

In this study, the failure probability of the time-variant model parameters is updated with the observations. The computational performance is determined by the number of states employed for representing the variables, in particular \( a_0 \) and \( q \). So, the discretization of these variables must be performed. The number of intervals is regulated to achieve the optimal balance between accuracy and computational speed. Therefore, the variables \( a_0 \) and \( q \) are discretized in 90 and 60 states, respectively. The discretization scheme for the random variables is shown in the Table 2.

The DBN representation including inspection results \( I_1 \), is shown in Fig 1. In case an inspection is performed, the probability of detection (POD) describes the probability of detecting the crack is given by Eq (4),

\[
\text{Table 1. Variables of the crack growth model}
\begin{array}{|c|c|c|c|}
\hline
\text{Variable} & \text{Distribution} & \text{Mean} & \text{Std/Cov} \\
\hline
a_0 \text{ [mm]} & \text{Exponential} & 0.16 & \\
\hline
a_c \text{ [mm]} & \text{Deterministic} & 25 & \\
\hline
\Delta \sigma \text{ [Mpa]} & \text{Deterministic} & 57 & \\
\text{Ln}(C) & \text{Normal} & -26.80 & \text{Std}=0.29 \\
\text{Bs} & \text{Lognormal} & 1 & \text{Cov}=0.25 \\
\text{n} & \text{Deterministic} & 7048 & \\
\hline
\end{array}
\]

\[
\text{Table 2. Discretization Scheme}
\begin{array}{|c|c|}
\hline
\text{Variable} & \text{Final interval boundaries} \\
\hline
a_0 \text{ [mm]} & [-\infty, \exp(ln(1e-5):(ln(25)-ln(1e-5)))/(90-2):ln(25)), +\infty] \\
\hline
q & [-10, -\exp(0:ln(1e-3)/(60-2):ln(1e-3)), 0] \\
\hline
\end{array}
\]

![Diagram of DBN](image)
\[ \Pr \{ \text{D} \vee a \} = \text{POD}(a) = \frac{\alpha a^d}{(1 + \alpha a^d)} \quad (4) \]

where D is the event of detection, a (mm) is the detectable crack and \( \alpha, \gamma \) are regression parameters. By instantiating the inspection variables I, in the DBN with the observed events at the times of inspection, the failure probability is updated considering the inspection outcomes based on the two heuristic decision rules.

4. Risk based decision analysis

The aim of RBI is to find a balance between the benefit from inspection and repair schedule versus failure cost. The obtained failure probabilities using DBN are combined with the specific cost model. The assumed failure cost \( C_f \) is \( 10^6 \) (money units) and the total time period is 100 years. The different costs are provided in the Table 3.

<table>
<thead>
<tr>
<th>Inspection cost, ( C_{\text{imp}} )</th>
<th>0.002 ( C_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Repair cost, ( C_r )</td>
<td>0.04 ( C_f )</td>
</tr>
<tr>
<td>Failure cost, ( C_f )</td>
<td>( 10^6 )</td>
</tr>
<tr>
<td>Discounting rate, ( \alpha_r )</td>
<td>0.03</td>
</tr>
</tbody>
</table>

Table 3. Cost characteristics

The total expected cost during the lifetime \( E_{\text{tot}} \) may be written as Eq (5).

\[ E_{\text{tot}} = \sum_{i=1}^{N} \frac{P_f(t_i)C_f}{(1+\alpha_r)^i} + \sum_{i=1}^{N} \left( 1 - P_{fc}(t_{\text{imp}}) \right) \frac{C_{\text{imp}} + P_r(t_{\text{imp}})C_r}{(1+\alpha_r)^{i_{\text{imp}}}} \quad (5) \]

where \( P_f \) denotes the annual failure probability in year \( t \) and \( P_r \) is the probability that a repair is performed in the year \( t \) after an inspection has been done in the same year. \( P_{fc} \) is the cumulative failure probability at time \( t \).

Fig 2. Comparison between schedules (a) interval inspection and (b) failure probability threshold

The result in Fig 2 shows that the optimum inspection interval is every 11 years and expected costs \( E_{\text{tot}} = 1.0742 \times 10^4 \) while the optimal annual failure probability threshold is \( P_f = 3 \times 10^{-4} \) and expected costs \( E_{\text{tot}} = 0.96095 \times 10^4 \). The failure probability threshold method was found to give smaller expected costs than periodic inspections.
5. Conclusion
In this paper, a framework where DBN is used for risk-based inspection planning of a miter gate welded joint considering inspection data. Further work can be used to consider the multiple structural components in a gate.

6. Acknowledgements
The authors acknowledge the financial support provided by the Wallonie-Bruxelles International (WBI), Belgium for this research.

7. References
Title:
Mapping the risk of neglecting maintenance on waterways infrastructures

Abstract
We develop a map showing the risk of disregarding maintenance on the German network of waterways infrastructures. The assessment of the risk is carried out applying risk matrices, which are developed considering historical accidents, structural characteristics of the objects, and a classification of the consequences of failures. The visualization of risk improves the definition of failure scenarios and the assessment of the magnitude of the consequences by considering other GIS-based information. The final scope is develop a risk-based approach to the prioritization of maintenance actions.

Introduction
Recent events such as the collapse of the Morandi bridge in Italy and the Brumadinho dam in Brasil have drawn the attention on the aging of infrastructure systems and the risk of disregarding maintenance actions. Although several maintenance management systems for bridges, roads, dams and waterways have already been developed, they often struggle to tackle current challenges, such as to allocate the limited maintenance resources to those repairing actions at which the highest risk is associated, also maintaining the infrastructure system available. For this reason, risk-based maintenance strategies are required.

Scope of the research
We want to develop a risk-based approach to the maintenance of the German network of waterways infrastructure, which allows a prioritization of maintenance actions already classified as urgent. Within this approach, we want to develop a map, which show the risk of neglecting maintenance; the map allows visualizing the connections between risk and GIS-based data, making possible to sharpen the risk assessment by considering failure scenarios in details.

State of the art
In the last decades, the unavoidable aging of civil infrastructures and transportation network fostered the development of IT-based management system for their maintenance. Information about damages is collected, which allows to rate each damage according to simple scoring systems, so that the urgency of the maintenance can be easily communicated. The advantage of such MMS is its feasibility; however, they fail to meet current challenges, and especially to properly allocate the limited resources to the increasing number of damages which could have great, negative consequences in the near future, or in other words which have the highest risk.
For this reason, advanced techniques for the prioritization of maintenance interventions are required. One way to do that is to consider the risk of neglecting maintenance actions, or in other words applying a Risk-based maintenance (Arunraj & Maiti 2006). Before entering into the merits of these methods, it is important to remind a few facts about risk. First of all, the notion of risk involves three important elements: the definition of a failure scenario, the probability that the scenario will happen, and the magnitude of the consequences (Kaplan and Garrick 1981). Consequences might belong to different categories, which are characterized by different unit measures: the damage is thus a vector quantity parameter, rather than a single scalar. When risk is used in order to prioritize some type of actions, it is important that its multidimensional nature could be condensed into a suitable number or a category, which allows a univocal orderings of the failure scenarios. A risk measure can be created, which aggregates several factors through a weighted combination, and express risk in a numerical way. In some cases, for the purpose of enhance risk perception and understanding, it can be useful to convert the risk measure into a risk index, which is usually a number having a certain unit measure or a categorical scale identified by letters, words, colors.

Risk is usually evaluated through risk matrices, in which two criteria are considered: the imminence and the consequences of failure. Each criterion is usually rated on a scale from 0 to 5 and a categorical risk index is obtained by associating categorical values to the multiplication of each rate. Although risk matrices have some limits, and their construction should follow some rules which assure their consistency (Cox 2008), they are wide-spread tools in risk analysis, since they allow a straightforward and easy evaluation.

Another subject which has recently attracted attention is risk visualization. Generally speaking risk visualization enhances risk understanding: in effect many consequences, such as basin flooding or transportation disruption, can be well identified through GIS systems. Visualizing risks might reveal connections among different types of risks, or among risks and the environment: this step can play an importance role in densely populated and urbanized area, where also different infrastructure systems interact, and any failure or malfunctioning could have cascade effects. Risk visualization also provides further insights in risk management, such as suggesting group maintenance strategies or particular surveillance measures. Recently several studies have focused the attention on developing risk maps: Tolone (2008) suggests an interactive visualization for critical infrastructure systems; Repetto et al. (2017) propose a web-based GIS platform for the safe management and risk assessment of complex structural and infrastructural systems exposed to wind; Di Salvo et al. (2018) introduce a GIS-based procedure for preliminary mapping of pluvial flood risk at metropolitan scale.

However, no papers have been found in literature that propose to develop maps for the visualization of the risk of neglecting maintenance.

Our research proposal
We propose to develop a map for visualizing the a-priori risk of neglecting maintenance on the infrastructures of the West-German canals. Recalling that consequences of infrastructures failures and malfunctioning depend, on one side, on characteristics of the object which is affected by the damage, and on the other side, on characteristics of the environment where the failure takes place, we suggest in this paper to determine the magnitude of the consequences dependent only on the characteristics of the object, and to visualize the resultant risk on the map. By considering other GIS-based information, it will be possible to identify where a detailed analysis of the environment is required. The risk is thus a-priori, because it is determined only considering the magnitude of the consequences which depend on the objects typology and structural characteristics. Furthermore, the risk is a-priori because failure scenarios are not determined in details; consequences are rather classified according to their typology (social, environmental, economic), based on the function of the object under examination. The determination of the a-posteriori risk, which is also based on detailed failure scenarios and circumstantial condition, is the main scope of the development of such risk maps.

However the map has also other aims: since expert opinions and engineering judgments about the likelihood of failures and magnitude of the consequences are available, a risk index will be developed, visualized on the map and compared with risk indices generated by our analysis. In this way, divergent risk indices will foster the collection of new information, which will serve to explain the divergence or to improve the risk index, especially where the risk could be significantly high.

The development of such risk map implies the calibration of suitable risk indices. The intention is here to resort to risk matrices, which lead to categorical risk indices. This choice is motivated by the fact that part of the expert opinion and judgments is already in the form of risk indices obtained through risk matrices. However, a particular attention will be devoted to the development of consistent risk matrices, and when
sufficient information about historical case studies is available, the risk matrix is directly learnt from data; if the available information will be insufficient to derive statistics, deduction will be anyhow made, which will improve the development of the risk matrices.

Another important prerequisite to the development of the risk matrices is the classification of the objects according to their structural characteristics. Data about structural characteristics could be both quantitative (i.e. geometry) and qualitative (i.e. material). Here, a cluster algorithm is applied, which is able to analyze both quantitative and qualitative data at the same time, and leads to the identification of the main structural categories. Each category collects objects having similar structural characteristics, and thus similar consequences or similar probability of failures. Before applying the cluster algorithm, data about objects structural characteristics, which are already collected in a database, are cleaned and made suitable for the analysis.

The results of the risk analysis are shown on the map together with other GIS-based data, such as fleet and freight trajectories, land usage, infrastructures systems, intermodal transportation terminals; suitable symbols will be also created, which facilitate the visualization of the relevant information. Conclusion will be finally drawn about the objects whose failure scenarios deserve more attention.

Case Study

The attention is focus on the West-German network of canals, which comprises several types of infrastructures such as locks, weirs, culverts, conduits, flood gates, bridges, canal bridges, dams. Since the construction of the network of canals dates back to the 1930s, many infrastructures show nowadays damages and other signs of aging. Furthermore the area, which is characterized by a high population and industrial density, plays a special role within the waterways system: waterways are there an irreplaceable transportation mean and they provide water to population and industrial activities; at the same time, any failure or malfunctioning of the canals infrastructures could lead to flooding or interruptions of the supply chain, which in turn might have very high social, economic and environmental consequences. A risk-based approach to the prioritization of maintenance actions is there particularly required.

A risk map and a personalized approach to risk definition are developed for each infrastructures typology. This is motivated by the fact that the available information for risk definition and the GIS relevant data are heterogeneous. The information will be extracted from several databases, which contain data about infrastructures structural characteristics and damages, fleet structures and trajectories, delivered goods, transportation interruptions and accidents.

Conclusion

This paper suggests an approach to the prioritization of maintenance intervention based on the risk of neglecting repairing actions. The development of a risk map is proposed in order to enhance risk understanding, and especially to sharpen the definition of failure scenarios and the assessment of failure consequences. Attention is especially focused on the network of West-German canals, which is characterized by several infrastructures typologies, and high density of industries, population and other transportation systems. The results will serve to allocate the limited maintenance resources in a more effective way.

Reference

Title: Maintenance Management Without Using Probabilistic Methods?

Abstract:

1. Introduction
One of the tasks in the management of infrastructure systems is decisions on the allocation of resources. There are countless methods to help systematize the decision-making process and prioritize alternatives. They show decision possibilities and the consequences of decisions. If the resources are limited, only a part of the projects can be realized. Criteria help to evaluate these projects. The criteria usually are object-related (e.g. condition evaluation) or take into account aspects of other systems (e.g. economic significance). The procurement of information often turns out to be difficult or impossible and is time-consuming. The implementation of a decision support system for the planning of maintenance in the portfolio is often an iterative process that lasts for years and in which the information increases continuously.

This paper shows the development of a structure for the provision of infrastructure that is also applicable for different types of infrastructure. The core of the method is (i) the analysis of the function of the structures in the infrastructure system, (ii) the analysis of the failure consequences in relation to the functions and (iii) the derivation of indicators as well as (iv) the evaluation with spatial information systems and official statistics and (v) to communicate risks in a simple way. The presented method is applied to 243 weir systems in Germany.

2. The problem-solving approach
The idea for the approach in this paper can be found in the historical development of transport infrastructures. Human settlements were mainly created alongside rivers and lakes because of the water and food supply and the good soil quality. Even today, waterways are important not only for passenger and freight traffic, but also for many other systems. Due to the variety of tasks and functions of waterway structures, incidents can have multiple effects on different areas:

- Water supply and disposal for industry, trade and agriculture
- Flood management
- Energy production
- Tourism and leisure activities
- Drinking water production
- Groundwater management
- Habitat for animals and plants

Especially the investigation of these areas offers a lot of information about the interactions with other systems. The fact is that the environment of the waterways is related to them. If, for example, many settlement areas are located alongside a river, it can be assumed that many people will spend their leisure time there and in industrial areas the water will be used for cooling. The evaluation of e.g. the spatial data does not provide "perfect" data, but it allows comparisons between different waterway structures, although a comparison has been considered impossible so far.
The following describes how the consequences are systematized and which indicators are selected for the respective area. The investigation framework formed by this structure allows a comparison of the objects regarding consequences after a building failure (collapse) or functional failure.

3. Selection of Proxy Indicators

The following Table 1 shows a selection of possible consequences after a function failure or collapse for each of the four object types. The basis for this list are various systems in which traffic infrastructures and the assumed triggering events "collapse of the structure" or "functional failure" are integrated.

The evaluation results in the following effects for all four object groups:

- Traffic impairments
- Impairments in (drinking) water supply and water disposal
- Flooding of adjacent areas with different types of use
- Endangerment of persons
- Influence on natural habitats
- Extraordinary costs for the infrastructure manager

Depending on the triggering event, the respective consequences for society, economy, environment and infrastructure manager may vary in intensity. Due to a lack of information on realistic occurrences, the following assumes the maximum possible consequences in each area. It is a challenge to identify indicators based on few available data, which nevertheless provide initial conclusions about possible cause-effect relationships. The following Table 1 assigns the consequences to a consequence class and names the selected indicator.

### Table 1: Consequences and indicators

<table>
<thead>
<tr>
<th>Consequences class</th>
<th>Consequences</th>
<th>Indicator</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Economical</strong></td>
<td>Impairment of freight transport</td>
<td>Costs due to traffic disruption</td>
</tr>
<tr>
<td>(Economy, Ec)</td>
<td>Impairment of water supply and sanitation</td>
<td>Size of adjacent areas with industrial or commercial use</td>
</tr>
<tr>
<td></td>
<td>Flooding of adjacent industrial and commercial areas</td>
<td>Hydraulic analyses and size of flooded areas with industrial or commercial use</td>
</tr>
<tr>
<td><strong>Social</strong></td>
<td>Endangerment of persons on adjacent areas</td>
<td>Size of adjacent settlement areas, population density and number</td>
</tr>
<tr>
<td>(Society, S)</td>
<td>Impairment of drinking water supply</td>
<td>Size of adjacent settlement areas, population density and number</td>
</tr>
<tr>
<td></td>
<td>Flooding of adjacent settlement areas</td>
<td>Hydraulic analyses and size of flooded settlement areas</td>
</tr>
<tr>
<td><strong>Ecological</strong></td>
<td>Impairment of natural habitats</td>
<td>Size of adjacent natural areas</td>
</tr>
<tr>
<td>(Environment, En)</td>
<td>Flooding of natural habitats</td>
<td>Hydraulic analyses and size of flooded natural areas</td>
</tr>
<tr>
<td><strong>Structural</strong></td>
<td>Extraordinary costs</td>
<td>Reconstruction costs for the object</td>
</tr>
<tr>
<td>(Infrastructure Manager, I)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The table shows that the following investigations must be made in order to be able to determine values for the proxy indicators:

- Statistics on freight transport and related cost analyses
- Land cover analyses with spatial data
- Hydraulic analyses
- Reconstruction costs

The analysis of consequences is an essential component of risk analysis. The potential of damage is determined as the totality of the values and their spatial distribution in the investigated area.

4. Case study for weir systems

The analyses described above were carried out for 243 weir systems. From the results of the spatial analyses it can be determined which types of land use are available. The population density can be used to determine how many people live in the area under investigation. The link between freight traffic statistics and demurrage fees makes it possible to differentiate the economic importance of waterway sections. The hydraulic analyses illustrate the danger posed by waves to people on the shore and/or ships in the water.

The following figures (see Figure 1) show the compilation of the results for two weir systems on the river Saar in a cobweb chart. The respective values of a weir system result in individual blue plains. Each weir system thus has an object-specific profile. The charts are color-coded so that the user can orientate by himself. The cobweb charts are scaled to the interval [0;1], i.e. the respective maximum value of a key figure of all weir systems corresponds to the value 1.0.

![Figure 1: Different results for two of the weirs](image)

For each weir, it is possible to quickly see where possible consequences are focused. An area center in the upper right area means high values for the number of affected persons and the size of the affected settlement areas. A center of gravity in the upper left area means high reconstruction costs. In this case, there are no costs due to traffic disruption because no freight transport is included in the statistics due to very low volumes. In addition, the share of industrial land in relation to the other 243 weirs is also very low.

The described procedure shows a simple method to record additional parameters of weirs and other projects with relatively little effort. With the additional indicators, it is possible to prioritize structures that have the same condition. The advantage is that the interactions with other systems are now also integrated, even if a detailed evaluation is not yet possible. In addition, different building types can be compared to each other.

5. Summary of the results and outlook on further steps

Due to a lack of resources, the maintenance of waterway infrastructures has been severely neglected in recent years. A maintenance management system is to be developed out of this necessity. It started with regular inspections and damage assessments. As a result, it has been shown that the majority of the waterway infrastructures are still in a sufficient or in an unsatisfactory condition. Due to the high number, a method for prioritizing projects has been developed, after the application of this method many projects are still highly prioritized.

The method developed in this paper offers a solution to address this problem. The idea is to make use of the knowledge about interdependencies with other systems. Due to a lack of data, spatial data analyses were used to investigate the
regional-geographical circumstances. In addition to freight transport statistics and population statistics, initial insights are gained into the historically evolved interactions between the waterway infrastructure and its environment. The data could be collected with little effort. With the visualization in cobweb diagrams, decision makers can gain an initial overview without having to study all the data in detail.

The described procedure is the starting point for further analyses. The results can already be used for weirs for a multi-criteria decision analysis. It is now possible to prioritize the infrastructures in terms of their interactions with their surroundings and their importance for freight transport as well as their condition.

The long-term goal is the development of a decision support system. This system should be designed in this way that the decision makers have the possibility to analyze with the variation in their priority setting how certain decisions affect the entire situation. A matching MCDA method must be identified.

References


Title:
Robustness of Navigation Infrastructure in Context of Risk-Based Maintenance

Motivation:
There are approx. 5,000 infrastructure constructions on the 750 km maritime and 6,600 km inland waterways of the Federal Waterways and Shipping Administration of Germany (WSV). Of these, about 50% were built before 1960. Currently, the state of the construction infrastructure, which is partially critical, requires extensive measures - from repair to new construction. The necessary realizations are not all simultaneously possible. Therefore, an objectified prioritization of individual structures and network sections is required. In this context, the exclusively condition-based consideration of structural damage seems not to be decisive alone.

In the case of risk-based maintenance management, the structure assessment is carried out from two sides (Fig. 1): on the one hand, the probability of failure, determined by the condition of the structure and its robustness, and on the other hand, the consequences of the failure. On the construction side, an advanced damage process does not necessarily lead to failure if the structure reacts robustly to the special damage—e.g. with a redundant load transfer. Thus, the combination of detected damage and general constructional robustness (Fig. 2) results in a qualitative condition assessment, which allows an extended statement on the probability of failure in the form of a damage index.

![Fig. 1. Risk-based maintenance management](image1)

![Fig. 2. Damage index as a combination of robustness and damage classification](image2)

Definition and assessment of robustness:
For waterway structures, robustness is described by the difference in structural performance between an undamaged and a damaged system [1]. The structural robustness is not limited to a preferred fail announcement, but also requires a relation of the extent of the damage or failure to its causes. A total of \( j = 7 \) relevant robustness criteria were defined for the application case waterway structures - in particular locks - which can be divided into the fields of construction (C), utilization (U) and global stability (G) (Table 1). Each robustness criterion is given a robustness grade \( R_j \) and a weighting \( W_j \) in the overall context, from which the overall robustness value \( R \) can be determined:

\[
R = \frac{\sum R_j \cdot W_j}{\sum W_j}.
\]

The robustness values are normalized from 0,0 (no robustness) to 1,0 (highest robustness).

**Tab. 1. Robustness criteria and weighting**

<table>
<thead>
<tr>
<th>robustness criterion</th>
<th>Construction (C)</th>
<th>Utilization (U)</th>
<th>Global stability (G)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>redundancy</td>
<td>progressive collapse</td>
<td>ductility</td>
</tr>
<tr>
<td>weighting ( W_i )</td>
<td>3</td>
<td>3</td>
<td>2</td>
</tr>
</tbody>
</table>

The evaluation of the criteria is carried out analogously to damage classification [2] by means of grades between 1 and 4, which depend on calculation criteria according to structural re-calculation [3] or assessments (Figure 3). Here, a distinction is made between individual evaluation areas and sections of the waterway structure (Figure 4).

**Fig. 3. Determination of robustness grades, example criterion: redundancy**

**Fig. 4. Evaluation sections at a lock cross-section (lock Erlangen, Main-Danube-Canal)**

**Link of robustness with damage classification:**

Periodical structural inspections of the WSV according to [2] lead to damage classes DC at different structural elements (e.g. reinforced concrete, steel elements, seals, joints, etc.). For the assignment of the damage classes DC to the robustness criteria \( R \) a simple 0/1 combination is carried out. In this case, a damage process, if decisive for the respective robustness criterion, has the same effect on all detection formats. The damage index \( DI \) of the investigated area i is then calculated from damage class DC, weighting \( W \) and robustness grade \( R \) per criterion \( j \)

\[
DI_i = \frac{\sum R_j \cdot W_j \cdot DC_{ij}}{\sum W_j}.
\]
Evaluation of application at ship locks:
In the course of an application test on reinforced concrete locks [1] as well as locks with heavyweight chamber walls [4], the efficiency of the system has been tested. It was shown that the extended state analysis with regard to the probability of failure can put into perspective poor structural conditions and thus influences the prioritization of measures.

Seven ship locks on the Main-Danube Canal, the Erlangen, Eibach, Bachhausen, Leerstetten, Kehlheim, Nuremberg and Strullendorf locks and other different reinforced structures were evaluated in order to test system performance. The chamber cross-sections differ importantly in terms of cross-sectional design and utilization. This is reflected in the robustness rating. For example, while the squat locks Kehlheim and Bachhausen have consistently high robustness values between 0,7 and 0,9, there are partial weaknesses in the slender buildings Erlangen and Nuremberg. A higher damage classification does not necessarily mean a higher damage index. Thus, structures with a higher degree of robustness may compensate certain damages (Table 2).

Tab. 2. Comparison of damage index and damage classification of different ship locks

<table>
<thead>
<tr>
<th>Lock</th>
<th>Damage classification</th>
<th>Robustness</th>
<th>Damage index</th>
<th>Main damage process</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erlangen</td>
<td>3,9</td>
<td>0,19</td>
<td>1,00</td>
<td>Joints and seals</td>
</tr>
<tr>
<td>Eibach</td>
<td>3,9</td>
<td>0,23</td>
<td>1,00</td>
<td>Joints and seals</td>
</tr>
<tr>
<td>Leerstetten</td>
<td>4,0</td>
<td>0,66</td>
<td>1,00</td>
<td>Joints and seals</td>
</tr>
<tr>
<td>Strullendorf</td>
<td>4,0</td>
<td>0,29</td>
<td>0,88</td>
<td>Cracks in concrete</td>
</tr>
<tr>
<td>Nuremberg</td>
<td>3,9</td>
<td>0,19</td>
<td>0,61</td>
<td>Concrete surface damage</td>
</tr>
<tr>
<td>Oberhausen</td>
<td>3,2</td>
<td>0,59</td>
<td>0,50</td>
<td>Concrete surface damage</td>
</tr>
<tr>
<td>Heime-Ost</td>
<td>4,0</td>
<td>0,58</td>
<td>0,34</td>
<td>Concrete surface damage</td>
</tr>
<tr>
<td>Kelheim</td>
<td>4,0</td>
<td>0,69</td>
<td>0,32</td>
<td>Joints and seals</td>
</tr>
<tr>
<td>Bachhausen</td>
<td>4,0</td>
<td>0,84</td>
<td>0,30</td>
<td>Concrete surface damage</td>
</tr>
<tr>
<td>Zeitling</td>
<td>2,1</td>
<td>0,62</td>
<td>0,25</td>
<td>Risse im Stahlbeton</td>
</tr>
<tr>
<td>Henschenburg</td>
<td>3,2</td>
<td>0,67</td>
<td>0,18</td>
<td>Concrete surface damage</td>
</tr>
</tbody>
</table>

Locks with heavyweight walls mostly achieve high robustness values. Hence, despite of damage classifications DC > 3,0 low damage indices DI < 0,3 were evaluated.

Summary:
The extended evaluation system with a damage index related to the robustness relativizes the damage classifications determined in the course of the structural inspections and thus allows a more objective prioritization. Robustness characteristics can be classified both within a group of the same structure type and for different typologies. This may better predict and sort out the perspective susceptibility to damage.

Literature:


Title: Innovations in Underwater Imaging of Waterway Infrastructure

Abstract:

Technology related to the survey and inspection of underwater infrastructure has evolved significantly in the past decade allowing tasks that were previously too dangerous, inaccurate, or expensive. PIANC Working Group 182 was created with international representation by a joint effort of several commissions to provide an overview of these technologies and provide useful information about using these innovative techniques in civil engineering tasks in various areas like environmental permitting/archeological studies; underwater construction monitoring; maritime security threat assessments; documented visual representation; diver safety and efficient; rapid condition assessment; erosion/scour detection, and evaluation of submerged infrastructure. This paper and presentation will highlight key aspects from the WG182 report scheduled for release in the near future.

Introduction:

By utilizing these innovative capabilities, facility owners and agencies can accomplish many inspection tasks in new ways that can provide more detail, allow better decision-making, improve safety, and even reduce operating costs. Furthermore, these technologies improve situational communication efficiency and effectiveness since “a picture is worth a thousand words”. Imaging is the quickest and most intuitive way of comprehending information by individuals. Images provide a means of helping an individual visualize what they can’t actually see with their own eyes from the underwater perspective.

Information is provided on the applications, theory, state of the art, deployment methods, georeferencing, and data management concerning underwater imagery and related technologies. Real case examples of beneficial acoustic images for underwater situations include:
- Design Stage Data Gathering (e.g., new marine construction, rehabilitation of port/harbour structures, extension of breakwaters, etc.)
- Environmental permitting and archeological studies (e.g., ship wrecks, historical infrastructure, etc.)
- Construction Monitoring Underwater (e.g., quality control, progress payments, pre-/post site conditions, etc.)
- Maritime Security Threat Assessments (e.g., detection of submerged explosives, intruder detection, etc.)
- Documented Visual Representation (e.g., as-built plans, large scale defects on a structure, or submerged object / obstruction documentation)
- Diver Safety and Efficiency Enhancement (e.g., challenging dive sites such as fast current, heavy debris, extreme depth, polluted water, and dangerous marine life)
- Rapid Condition Assessment (e.g., verification of marine facility structure after an incident such as seismic event, vessel impact, extreme weather, etc.)
- Erosion/Scour Detection and Documentation (e.g., seabed/channel bottom movement monitoring and foundation exposure information)
- Evaluation of Infrastructure (e.g., maintenance decision-making data, asset management input, etc.)

The report briefly explains the principal theory behind acoustic surveys. Acoustic theory is well known and there is more detailed information referenced in the report. It should be noted that the terms acoustic and sonar are often used interchangeably in the industry. It should also be noted that this report uses the common term sonar, but this vernacular was originally derived from the acronym SONAR (SOund Navigation And Ranging). The focus of this report is in everyday usage of various underwater acoustic methods, as well as the benefits and capability that technology on implemented projects worldwide. Nonetheless, other competing technologies are mentioned in the report to better indicate the advantages and limitations of the underwater acoustic methods. Using a laser pulse to produce an image is an active optical imaging technique that can be utilized above and below the water. Laser scanning, often referred to as LiDAR (Light Detection And Ranging), can produce extremely accurate underwater images, but possess limited range due to light transmission factors related to water clarity and other limitations make it more widely used for nearshore coastal surveys or offshore ocean structures than inland waterway infrastructure. Non-optical technologies that have demonstrated success in providing underwater images include sonar and radar. Radar technologies, such as ground penetrating radar (GPR), can produce underwater images with electromagnetic waves primarily to find internal concrete defects or subsurface channel-bottom geotechnical strata layers, while synthetic aperture radar (SAR) has used electromagnetic waves to obtain a large-area perspective image of bathymetry.

The most common technologies and their characteristics are introduced. A discussion on the capability of the these methods to gather necessary information in different situations. The report reviews different possibilities and applications to use methods in fieldwork. This is important when planning best opportunity to gather suitable information for specific management and civil engineering purposes. It is sometimes valuable to have information from different methods that can supplement each other to get even better result for further planning, repair, and maintenance needs.

Deployment methods are also presented to highlight this important part in the usage of acoustic methods to gather the best possible information for different tasks. The suitability of the most common inspection method is discussed considering different construction materials and different type of waterfront structures and parts of the structures (like bridges, channel/canal structures, quays, other waterfront structures and aids to navigation) based on practical knowledge from different cases. In this case, a comparison has been made between multibeam, scanning sonar and diving inspection (as reference method and most used traditional inspection method for underwater structures).

Data management is extremely important part of the process. Typically data must be verified and post processed to eliminate possible disturbances in survey. It is important to understand that acoustic survey is based on sound velocity and reflection of voice impulse from different materials and structures. There is several reasons that may cause problems to gather perfect data. The report will explain briefly most typical phenomenon and gathered experiences in different situations. For data visualization there is wide range of
software. Some of these specially designed to handle and study large amounts of data as it is often case in acoustic data files, especially when we are discussing about point clouds. The capability to show details varies and it is important to understand how well the survey data is expressed in final images.

Obtaining underwater imagery is usually just part of a larger task such as evaluating the waterfront infrastructure for a particular reason, so the related above water data acquisition methods that are often used at the same time will be briefly discussed as well to show how these components can fit together. The challenge with traditionally used underwater photography (either obtained by a diver, ROV, or other means) is that the water clarity, limited available light, limited range, and limited topside location perspective only provides so much value for the evaluating professional in documenting existing conditions and the owner in promoting any recommended follow-up work activity for funding. Commonly used traditional hydrographic survey data provides channel bottom elevations without much further detail. Underwater acoustic technology provides more useful 2-D and 3-D images in dealing with waterway infrastructure.

Waterway infrastructure will greatly be affected by climate change, and underwater imaging can be used by facility owners to deal with higher and lower water levels, as well as other effects of climate change such as scour due to extreme weather events. Rising sea levels will result in flooding and disruption in marine infrastructure. Assessment of the effects of sea level rise requires good understanding of the construction of marine infrastructure, construction, orifices, inlets, and outlets, as well as the condition of such structures.

Advances in underwater imaging and geomatics are occurring around the world. As facility owners, agencies, consultants, and contractors consider their options for underwater observations—in particular, the use of underwater imaging—there needs to be careful consideration for assuring qualified personnel, proper equipment and proper deployment techniques, and a basic understanding of the technical limits of each type of imaging technology. Because of this need, case studies with breath-taking imaging will be presented.

**Conclusion:**

The use of underwater acoustic inspection technology in the planning, design, maintenance, demolition, retrofitting and rehabilitation of existing and new facilities is growing among the facility owners, operators, consultants and contractors. Examples of the facilities and structures for which underwater data collection can be employed on any type of marine situation including: commercial wharves, piles, bulkheads, quay-walls, seawalls, wing-walls, bridge piers, channel and river beds, dams, intake and outlet structures, jetties, groins, gabions, dykes, lined and non-lined water conveyance channels, levees, ripraps and revetments, lighthouse and aid-to-navigation elements, locks, sluicegates, flood-defense systems, offshore windfarms, artificial islands/reefs, and recreational boat launch ramps/docks.

Underwater imaging technologies can be used to help facility owners detect underwater obstructions and build more economical and durable structures, as well as better monitor and manage existing assets. It is anticipated that water transport infrastructure will greatly be affected by future deterioration and climate change. Underwater imaging can be used by facility owners to make better decisions and deal with higher/lower water levels, as well as other effects of climate change such as scour due to extreme weather events.
Allongement de l'écluse de Rochetaillée-sur-Saône – Présentation du contexte de l'opération et description d'une méthode de construction par caisson flottant

LE CONTEXTE ET LES PRINCIPALES CONTRAINTES

Située à 15 km au Nord de Lyon, l'écluse de Rochetaillée –Couzon est localisée au PK17 de la Saône, coté rive gauche. En rive droite se situe un barrage à clapets composé de quatre passes, auquel est accolée une micro-centrale hydroélectrique.


La longueur utile de son sas (184,50 m) restait un point pénalisant de l'itinéraire Rhône/Saône sur lequel circulent des convois poussés d’une longueur totale de 190,00 m (2 barges + pousseur). Cette situation obligeait les transporteurs à adapter leurs convois ou à désaccoupler les barges lors du franchissement ; ce qui s’est révélé très pénalisante pour les transporteurs de la voie fluviale dans un contexte économique très tendu malgré la croissance du trafic commercial vers le nord du bassin : développement des ports de Pagny, de Chalon sur Saône et de Macon.

La Direction territoriale Rhône Saône de Voies navigables de France basée à Lyon a donc lancé une opération pour harmoniser la longueur utile de cette écluse avec celle des autres écluses de l’axe Rhône-Saône, mais aussi sécuriser l’approche des ouvrages par la construction d’un dispositif de guidage coté aval qui faisait cruellement défaut.

L’opération a été financée dans le cadre d’un contrat de projet interrégional avec cofinancement VNF 71 %, REGION RHÔNE-ALPES 22 %, EUROPE 7 %.
LA CONCEPTION

CNR Ingénierie a obtenu le mandat de maîtrise d’œuvre complète pour mener à bien cette opération. Les études d’avant-projet et de projet envisageaient :

- **Allongement du sas** :
  
  Parmi toutes les solutions d’allongement, celle qui fut retenue est un allongement par l’aval à partir de la construction d’une nouvelle tête aval qui serait accolée à la tête existante de l’unique écluse en service. Cette solution tenait compte du bon raccordement des trajectoires de navigation et du maintien de la fonction barrage des ouvrages. Encore fallait-il imaginer comment réaliser cette nouvelle structure dans l’emprise du chenal, sachant qu’il fallait maintenir le passage de la navigation pendant toute l’année, excepté pendant 10 jours au cours desquels un arrêt de la navigation serait accordé (chômage annuel), mais en mars, période où il n’est pas rare de voir la Saône en crue. Plusieurs méthodes de construction ont été étudiées au stade du projet :
  
  - allongement par construction en place à l’aide d’enceintes en palplanches,
  - allongement par immersion d’une structure béton amenée par flottaison,
  - allongement par immersion d’une structure métallique amenée par flottaison.

  C’est finalement la solution avec caissons métalliques flottants qui a fait l’objet d’un appel public à concurrence pour les travaux de construction.

- **Guidage des bateaux à l’aval de l’écluse** :

  Les études de trajectographie menées par CNR Ingénierie ont montré que les accès à l’écluse post-allongement resteraient délicats en raison de la proximité de l’écluse à la berge et des courants de recirculation dus aux écoulements du barrage. Un guidage aval s’est donc avéré nécessaire avec un double objectif : protéger la nouvelle tête vis à vis des chocs de bateaux et aider les navigants dans leur manœuvre d’approche. Il comprend les ouvrages suivants :
  
  - un mur guide coté berge présentant un guidage rectiligne d’environ 90 m de longueur aligné avec le bajoyer rive gauche de l’écluse ;
  - une estacade coté rivière de 18 m de longueur inclinée formant ainsi une sorte d’entonnement avec le mur guide.
Les structures sont de type "poutre sur pieux" avec un plan de glissement des navires plein constitué par des plaques en béton armé préfabriqué.

Grace à la souplesse relative de la fondation, l’énergie de choc des bateaux induite au moment du guidage est absorbée par la déformation élastique des pieux, ce qui réduit les risques d’avarie sur la coque des bateaux et les déformations irréversibles sur les structures.

**Figure 1 : vue aérienne du projet**

**LES METHODES D'EXECUTION MISES EN OEUVRE**

Une des grandes particularités de l’ouvrage réside dans sa méthodologie de construction rendue difficile par la multitude des phases transitoires.

Préfabriquée en atelier, la structure métallique du radier est assemblée à terre et mise à l’eau au port Edouard Herriot au sud de Lyon grâce à 2 grues de 500 Tonnes (Photo 2 : mise à l’eau du radier), puis transféré par voie fluviale jusqu’à l’ancienne écluse de Rochetaillée (Photo 3).

**Photo 2 : mise à l’eau du radier**

Chaque bajoyer métallique (Photo 4 et Figure 2) est constitué de quatre niveaux (N1 à N4) de 2.50 m de hauteur qui se décomposent :
- de tôles latérales de 10 mm d’épaisseur,
- d’une tôle amont de 20 mm d’épaisseur,
- d’une tôle aval de 10 mm d’épaisseur,
- et de raidisseurs multiples, constitués de plaques d’épaisseur 10 mm évidées formant des diaphragmes transversaux.

**Photo 4 : structure métallique des bajoyers**

Les niveaux N2, N3 et N4 sont ensuite levés et assemblés sur le radier en flotaison amarré dans le sas de l’ancienne écluse (Photo 5) au moyen d’une soudure périphérique étanche au niveau des tôles enveloppes.

**Photo 5 : caisson assemblé dans l’ancienne écluse**

A cet instant de l’avancement des travaux de l’allongement, le caisson métallique est achevé et peut être équipé de la porte d’écluse et des batardeaux provisoires insérés dans les rainures de part et d’autre du sas afin de préparer l’échouage.

L’opération suivante consiste, pendant la période de chômage de 10 jours, à déplacer le caisson face à l’écluse, à l’immerger dans sa position finale (Figure 3) puis à bétonner l’interface entre le substratum préalablement purgé des alluvions et fermé par des gabions et la sous-face du radier (figure 12).
La période de chômage étant fixée au préalable et immuable, la méthode d’immersion doit permettre une pose du caisson quel que soit le niveau d’eau aval de l’écluse car, à cette période l’année, le risque de fluctuation est grand. La méthode d’échouage proposée par la Direction Technique de Vinci Construction Maritime et Fluvial n’est donc pas basée sur un simple ballastage mais sur un principe de guidage par transfert progressif des charges sur quatre pieux supports insérés au préalable dans les bajoyers.

Ce transfert se fait par l’intermédiaire de suspentes et de vérins annulaires disposés dans la charpente. Le ballastage à l’eau permet d’équilibrer immédiatement les sous-pressions lors de la descente en eau et de limiter ainsi les efforts sur les tôles.

Cette méthode présente l’avantage d’être réversible jusqu’au bétonnage du radier. En cas de problème, il est possible de renflouer la structure en venant pomper dans les bajoyers et en remontant les suspentes (Figure 4).

Une fois la structure à la bonne profondeur et dans le bon alignement, une première phase de bétonnage vient combler l’espace entre le substratum rocheux et la sous face du radier ; puis l’eau est remplacée par le béton dans le radier d’abord puis dans les bajoyers (Figure 5).

En effet, une autre particularité du projet est de mettre en œuvre un béton structurale auto-plaçant immergé ; la structure de la nouvelle tête d’écluse étant dimensionnée comme une structure mixte acier/béton au sens de l’Eurocode 4.
CONCLUSION

L’allongement de l’écluse de Rochetaillée a constitué un défi majeur pour tous les acteurs du projet, à savoir le maître d’ouvrage VNF, le maître d’œuvre CNR et l’entreprise Vinci Construction Maritime et Fluviale.

En effet, bien que plusieurs fois proposée dans des marchés, la construction d’une tête d’écluse amenée par flottaison et échouée pendant une période de chômage constitue une « première » en France.

Ce défi comporte plusieurs étapes comme définir la liste exhaustive des sollicitations des charges sur la structure métallique durant l’ensemble des phases du chantier, de la mise en flottaison au port Edouard Herriot à l’échouage, en passant par les phases de poussage, de ballastage par de l’eau puis par du béton auto-plaçant, de transfert de charges sur les pieux et cela avec un caractère de réversibilité quel que soit le niveau d’eau.

Pendant la période de chômage de 10 jours de mars 2016, le caisson, construit et en flottaison dans l’ancienne écluse, a été positionné avec succès pour offrir enfin une nouvelle aire commerciale au trafic fluvial sur la Saône.

**=*=*
Title:

Extension of the Quesnoy-sur-Deûle lock—Presentation of the operation context and description of a construction method using shifting

Introduction

The Quesnoy-sur-Deûle canal lock is located in France at 15km from Lille (Diagram 1). Its extension project fits within the framework of the Seine Nord Europe canal reconfiguration mission and within the scope of the modernization of the Seine Escaut connection. The goal is to allow accessibility to VA+ class units which correspond to the 135-metre Rhine vessels and to guarantee continuous navigation to the wide gauge network. These works will make it possible to address anticipated changes in Seine Nord Europe Canal traffic, both internally within the region and externally to Paris and neighbouring countries.
Context

The sector’s geological map is presented in Diagram 2. The base of the works foundation is located in Ypresian formations which are quite malleable. The Deûle’s average flow at Quesnoy-sur-Deûle is 10.3 m$^3$/s with a low flow at 1.72 m$^3$/s. The lock drop is 3.47m.

At present, 13,000 ships, 9000 of them loaded, pass through the lock. Transported tonnage is at 5 million tons. Saturation of the lock is expected on the horizon of 2025. The current lock dates back to 1980. It is in reinforced concrete. It is equipped with 2 mitre gates which limit a 110m x 12m chamber and a 4.30m moorage. On the left bank, a weir carries the Deûle flow up to 125 m$^3$/s.

The New Lock

The overall project (Diagram 3) includes lengthening the lock, fish passes, a pumping station, two lock basins (up- and downstream), the canal recalibration for a 3.5m moorage over 1.1km, the replacement of guide rails, and the refurbishing of three buildings.

The extension will be made on the downstream end of the existing lock which makes it possible to keep the drop wall. The goal is to attain a useful chamber length of 144.6m. The overall extension length is 39.55m (Diagram 4). As with the existing lock, it will be in reinforced concrete, with a U cross-section, and it will be superficially founded on Flanders clay.
The new downstream head will have a mitre gate equipped with 6 paddles activated by electromechanical actuators. It will be protected with a double bumper. Downstream of the head, the new guide rails will be made of DN1000 metallic stakes and 3 levels of sliding rails. The lock will handle 41 cycles every 24 hours. The cost of the works approaches 30 M€, all taxes included.

Related Works

The lock development work required incorporation regulatory obligations and taking into account environmental stakes and hydraulic constraints linked to the site.

Environmental measures
Further to the undertaking of a Fauna-Flora-Habitat diagnosis over a complete year, it was shown that the site presents important ecological stakes. The zone impacted by the works is in fact home to protected species and sensitive habitats which are totally or in part impacted by the works. Application of the “Avoid, Reduce, Compensate” principle (Diagram 5) necessary for obtaining the work permits drove:
i) choices for installation of certain works or work zones (in particular, avoiding a wooded area initially intended for storage on the right bank and the decision to position the fish pass upstream),
ii) planning work periods and defining conservation measures (reducing impacts linked to deforestation and work in the river as well as different avoidance measures such as seed collection, conservation fish catches, and signage for protected species),
iii) but also proposing compensatory measures:

- destruction of 3900m² of copses and riparian forests is compensated by 4500m² of reforestation of a field located beyond the works zone.
- destruction of 925m² of reed bed is compensated by transplanting the reef bed over 1500m² in the immediate vicinity of the destroyed zone.

Monitoring during the work and operations phases (over 5 to 30 years depending on the measures) is planned.

**Fish Traffic**
The building of a weir on the left bank in 2009 required the creation of a lock fish pass which was incorporated into the extension operation. The presence of a becque, a small water stream, in the immediate vicinity of the lock informed the choice of a continuous fish pass up-and-downstream on the right bank for 2 target species (pike and eel): an upstream fish pass with rough bottom from the Dewasier becque for the intermediate portion, and a pool-type pass downstream (Diagram 6). The construction’s attractiveness was called into question when the file was reviewed so an eel ramp was designed on the left bank near the weir.

**Measures to Ensure Navigational Flow**
The pumping station must be sized to cover compensation needs during low flow periods (3 months/year on average) until 2050, by taking into account the future construction of a second lock (Chart 1). Further to a hydraulic study, the following pumping needs were identified:

![Diagram 6 - Fish Works](image)

<table>
<thead>
<tr>
<th>Horizon</th>
<th>Situation</th>
<th>Navigational Flow</th>
<th>Low water flow</th>
<th>Fish pass flow</th>
<th>Pumping flow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>m³/s</td>
<td>m³/s</td>
<td>m³/s</td>
<td>m³/s</td>
</tr>
<tr>
<td>2015</td>
<td>Current lock</td>
<td>2,31</td>
<td>1,72</td>
<td>0,50</td>
<td>1,09</td>
</tr>
<tr>
<td>2030</td>
<td>Extended lock</td>
<td>3,06</td>
<td>1,72</td>
<td>0,50</td>
<td>1,84</td>
</tr>
<tr>
<td>2050</td>
<td>Doubled lock</td>
<td>2,84</td>
<td>1,72</td>
<td>0,50</td>
<td>1,62</td>
</tr>
</tbody>
</table>

The size situation is that of an extended lock at saturation in 2030, and not that of the second lock in 2050, as the latter will have the benefit of water-savings devices. A pumping capacity of 1.84 m³/s in 24 hours is necessary. The equipment will be 2 groups of 920 l/s, with a second as back-up.
Construction Details

Changeable Cofferdams

Undertaking the lengthening poses several problems. As the decision to build a traditional work in reinforced concrete was taken, ordinary realisation time - with interrupted navigation - can be estimated at 4 months. However, for the operator, it is out of the question to stop navigation for more than several weeks. A construction method that maintained ship traffic had to be proposed. To be certain, strictly speaking the lock sides walls do not affect the navigation gauge. However, their construction requires a cofferdam and protecting the workers inside from ship impacts. These constraints make it necessary to build side walls with a minimum 0.70m recess with regards to the existing lock’s bare side walls. The designer suggested two solutions: to leave the side walls in place or to align the side walls with the existing ones. The 1st solution required designing a door and wider bumper, with the opening moving from 12 to 13.4m. Further, it created sharp angles that are dangerous for ships. That’s why the 2nd solution was chosen. It was then decided to:

- first build the extension side walls with a recess of approximately 3.5m with respect to the existing side wall layout, away from two small cofferdams,
- and, then to shift each side wall - by sliding on metallic sections - until their definitive position away from a large cofferdam unifying the two previous smaller ones.

These construction principles led to a 3-year work sequence (Chart 2) with 3 idle time periods.

Chart 2 - Work phases

- Step 1 - Starting point
- Step 2 - Idle time 1 – Small cofferdams and stakes
- Step 3 - Period between idle times 1 and 2 – Shifting path
- Step 4 - Period between idle times 1 and 2 - Side walls
- Step 5 - Idle time 2 – Uniting the small cofferdams
- Step 6 - Idle time 2 – Building the large cofferdam
**Shifting Tracks**

The shifting beams rest on two rows of stakes (Diagram 7). The side wall rests on the tracks by way of engraved PTFE-coated switch plates. Actuators push on each track.

*Diagram 7 - Shifting device*

**Measures for Respecting Short Idle Times**

Intense river traffic requires that idle times be drastically reduced. They are limited to 3 periods, at approximately 1-year intervals, of respectively 15, 30, and 20 days. This requires working in 6 day/week shifts, with strict control of the planning by pre-defined markers. A margin of 2 to 3 days for the idle times was nonetheless planned.

**Issue of Flanders Clays**

The geotechnical works which make up the cofferdams and their foundations are sunk into Flanders clay (Ypresian grounds) which is a material that is very malleable, very firm, and subject to swelling. The works size entails the necessity of using heavy sections. But our experience on ramming sheet piles or stakes in this geotechnical context shows a high risk of rejection. As a result, ramming tests took place at end 2018 in order to confirm feasibility as well as the conditions for implementation of the intended works. These tests also allowed us to measure the stakes load capacity in light of a possible optimisation. At the completion of the tests, feasibility was confirmed, further, with possibilities for optimisation in the execution phase.

**Conclusion**

The start of on-site work is planned for 2023. Their smooth running, in compliance with environmental and river requirements, will be the guarantee of a successful operation. The construction method by shifting thus validated will be extended to other sites. The issue of lengthening locks is in fact common in the Hauts-de-France wide gauge network.
Réf. auteur:
Philippe SCHALKWIJK – BRL Ingénierie
1105 Avenue Pierre Mendès-France – BP 94001 – 30001 Nîmes, France
philippe.schalkwijk@sfr.fr

Co-auteurs:
Sophie LEGRAND - VNF
Service Maîtrise d'Ouvrage / Cellule Etudes et Grands Travaux 1, Direction Territoriale Nord-Pas-de-Calais, 3 rue Jeanne Maillotte 59000 Lille, France
Sophie.legrand@vnf.fr
Chloé CHENE
1105 Avenue Pierre Mendès-France – BP 94001 – 30001 Nîmes, France
chlo.chene@brl.fr

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Titre:
Allongement de l'écluse de Quesnoy-sur-Deûle – Présentation du contexte de l'opération et description d'une méthode de construction par ripage

Introduction
Le projet de l’allongement de l’écluse de Quesnoy-sur-Deûle, en France, à 15km au nord de Lille (fig.1), s’inscrit dans le cadre de la mission de reconfiguration du canal Seine Nord Europe et fait partie du périmètre de modernisation de la liaison Seine Escaut. L’objectif est de permettre l’accessibilité aux unités Va+ correspondant aux grands-rhénans de 135m et d’assurer la continuité de la navigation au réseau à grand gabarit. Ces travaux permettront de faire face aux évolutions de trafics attendues avec le Canal Seine Nord Europe, tant internes à la région qu’externes vers Paris et les pays voisins.
Le contexte

La carte géologique du secteur est donnée en figure 2. La base des fondations de l’ouvrage est donc situées dans les formations Yprésiennes qui sont très plastiques.
Le débit moyen de la Deûle à Quesnoy-sur-Deûle est de 10,3 m$^3$/s avec un étiage à 1,72 m$^3$/s. La chute de l’écluse est de 3,47m.
Aujourd’hui, 13000 bateaux, dont 9000 chargés, franchissent l’écluse. Le tonnage transporté est de 5 millions de tonnes. Une saturation de l’écluse est attendue à l’horizon 2025. L’écluse actuelle date de 1980. Elle est en béton armé. Elle est équipée de 2 portes busquées délimitant un sas de 110m x 12m avec mouillage de 4,30m. En rive gauche un barrage évacue les débit de la Deûle jusqu’à 125 m$^3$/s.

La nouvelle écluse

Le projet global (fig.3) comprend l’allongement de l’écluse, des passes à poissons, une station de pompage, deux garages d’écluse (amont et aval), le recalibrage du canal pour un mouillage de 3,5m sur 1,1km, le remplacement des rails de guidage et le rafraîchissement de trois bâtiments.

L’allongement est réalisé en extrémité aval de l’écluse existante, ce qui permet de conserver le mur de chute. Il vise à obtenir une longueur utile de sas de 144,6m. La longueur totale de l’extension est de 39,55m (fig.4). Comme pour l’écluse actuelle, elle est en béton armé, de section transversale en U, et est fondée superficiellement sur les argiles des Flandres.
La nouvelle tête aval accueille une porte busquée équipée de 6 vantelles de vidange et actionnée par des vérins électromécaniques. Elle est protégée par un parechoc à deux battants. A l’aval de la tête les nouveaux rails guide sont composés de pieux métalliques DN1000 et de 3 niveaux de glissières. L’écluse peut réaliser 41 cycles par 24h. Le montant des travaux approche les 30 M€TTC.

**Les ouvrages connexes**

La réalisation de l’aménagement de l’écluse nécessite l’intégration d’obligations réglementaires, la prise en compte des enjeux environnementaux et des contraintes hydrauliques liés au site.

*Mesures environnementales*

Suite à la réalisation d’un diagnostic Faune Flore Habitat sur une année complète, il s’avère que le site présente des enjeux écologiques importants. En effet la zone impactée par les travaux héberge des espèces protégées et des habitats sensibles, qui sont en tout ou partie impactés par les travaux. L’application du principe “Eviter, Réduire, Compenser” (fig.5) nécessaire à l’obtention des autorisations de travaux a piloté :

i) le choix de l’implantation de certains ouvrages ou zones de travaux (notamment l’évitement d’une zone boisée initialement prévue pour le stockage en rive droite et le choix de la position de la passe à poissons amont),

ii) la planification des périodes de travaux et la définition de mesures de préservation (réduction des impacts en lien avec le déboisement et les travaux en rivière ainsi que diverses mesures d’évitement comme la récolte de graines, les pêches de sauvegarde et le balisage des espèces protégées),
iii) mais aussi la proposition de mesures compensatoires :

- la destruction de bosquets et ripisylves sur 3900 m² est compensée par 4500 m² de reboisement sur un terrain situé en dehors de la zone de travaux
- la destruction de 925 m² de roselière est compensée par la transplantation de la roselière sur 1500 m² à proximité immédiate de la zone de destruction.

Un suivi en phases travaux et d’exploitation (sur 5 à 30 ans selon les mesures) est prévu.

**Circulations piscicoles**

La construction du barrage en rive gauche en 2009 nécessitait la création d’un ouvrage de franchissement piscicole de l’écluse qui a donc été intégré à l’opération d’allongement. La présence d’une becque à proximité immédiate de l’écluse a orienté le choix d’une continuité piscicole pour les 2 espèces cibles (brochet et anguilles) entre l’amont et l’aval, en rive droite par le biais d’un ouvrage amont de type passe à macrorugosité, de la becque Dewasier pour la partie intermédiaire, et d’un ouvrage aval de type passe à bassin (fig.6). L’attrait de cet ouvrage ayant été remis en cause lors de l’instruction du dossier, une rampe à anguilles a été conçue en rive gauche près du barrage.

**Mesures pour garantir le débit de navigation**

La station de pompage doit être dimensionnée pour couvrir les besoins de compensation en période d’été (3 mois/an en moyenne) jusqu’à l’horizon 2050, en intégrant la réalisation d’un doublement ultérieur de l’écluse (tabl.1). Suite à l’étude hydraulique, les besoins de pompage suivants ont été identifiés :

<p>| Tableau 1 - Calcul des débits de pompage nécessaires aux différents horizons |
|-----------------------------|------------------|------------------|------------------|------------------|</p>
<table>
<thead>
<tr>
<th>Horizon</th>
<th>Situation</th>
<th>Débit de navigation</th>
<th>Débit d’été</th>
<th>Débit passe à poissons</th>
<th>Débit pompage sur 24h</th>
</tr>
</thead>
<tbody>
<tr>
<td>2015</td>
<td>Écluse actuelle saturée</td>
<td>2,31 m³/s</td>
<td>1,72 m³/s</td>
<td>0,50 m³/s</td>
<td>1,09 m³/s</td>
</tr>
<tr>
<td>2030</td>
<td>Écluse allongée saturée</td>
<td>3,06 m³/s</td>
<td>1,72 m³/s</td>
<td>0,50 m³/s</td>
<td>1,84 m³/s</td>
</tr>
<tr>
<td>2050</td>
<td>Écluse jumelée (doublement)</td>
<td>2,84 m³/s</td>
<td>1,72 m³/s</td>
<td>0,50 m³/s</td>
<td>1,62 m³/s</td>
</tr>
</tbody>
</table>

La situation dimensionnante est celle de l’écluse allongée saturée en 2030 et non celle de l’écluse doublée en 2050, celle-ci bénéficiant de dispositifs d’épargne. Une capacité de pompage de 1,84 m³/s sur 24h est nécessaire. L’équipement sera donc de 2 groupes de 920 l/s, plus un en secours.
Les détails de la construction

Batardeaux évolutifs
La réalisation de l’extension pose plusieurs difficultés. Dès lors que le choix de construire un ouvrage traditionnel en béton armé a été fait, le temps de réalisation ordinaire – sous interruption de navigation - peut être estimé à 4 mois. Or pour l’exploitant il n’est pas question de couper la navigation plus que quelques semaines. Il faut proposer une méthode de construction préservant la circulation des bateaux. Certes, les bajoyers de l’écluse n’engagent pas strictement le gabarit de navigation. Cependant leur construction nécessite la réalisation d’un batardeau et la protection des ouvriers à l’intérieur contre les chocs de bateaux. Ces contraintes imposent de construire les bajoyers avec un retrait de 0,70m minimum par rapport aux nus des bajoyers de l’écluse existante. Deux solutions s’offrent au concepteur : soit laisser ces bajoyers en place, soit ramener les bajoyers dans l’alignement des existants. La 1ère solution nécessite de concevoir une porte et un parechoc plus larges, l’ouverture passant de 12 à 13,4m. Elle crée de plus des angles saillants dangereux pour les bateaux. C’est pourquoi la 2ème solution a été retenue. Il a alors été décidé :

- dans un 1er temps, de construire les bajoyers de l’extension en retrait d’environ 3,50 m par rapport aux plans des bajoyers existants, à l’abri de deux petits batardeaux ;
- et, dans un 2ème temps, de riper chaque bajoyer – par glissement sur des profilés métalliques – jusqu’à son emplacement définitif à l’abri d’un grand batardeau unifiant les deux petits précédents.

Ces principes constructifs ont conduit à un phasage de travaux (tabl.2) sur 3 années avec 3 chômages.

Tableau 2 - Phasage des travaux

<table>
<thead>
<tr>
<th>Étape 1 - État 0</th>
<th>Étape 2 - Chômage 1 – Petits batardeaux et pieux</th>
<th>Étape 3 - Période entre chômages 1 et 2 – Voie de ripage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Etape 4 - Période entre chômages 1 et 2 - Bajoyers</td>
<td>Etape 5 - Chômage 2 – Unification des petits batardeaux</td>
<td>Etape 6 - Chômage 2 – Réalisation du grand batardeau</td>
</tr>
</tbody>
</table>

- 5 -
Rails de ripage
Les poutres de ripages sont appuyées sur deux files de pieux (fig.7). Le bajoyer repose sur les rails par l’intermédiaire de platines engravées revêtues de PTFE. Le poussage s’effectue sur chaque rail par vérin.

Figure 7 - Dispositif de ripage

Dispositions pour respecter les courtes périodes de chômage
Le trafic fluvial intense impose de réduire drastiquement les périodes de chômages. Elles ont été limitées à 3 périodes, espacées de 1 an environ, de respectivement, 15 ,30 et 20 jours. Cela nécessite de travailler en postes 6j/semaine, avec un contrôle strict du planning par jalons pré-identifiés. Des marges de 2 à 3 jours sur les durées de chômage ont néanmoins été prévues.

Problématique des argiles des Flandres
Les ouvrages géotechniques constituant les batardeaux et leurs fondations sont fichés dans l’Argile des Flandres (terrains Yprésiens) qui est un matériau très plastique, très raide et sujet au gonflement. Leur dimensionnement induit la nécessité d’utiliser des profilés lourds. Mais le retour d’expérience sur le fonçage de palplanches ou de pieux dans ce contexte géotechnique indique un risque de refus important. Ainsi, des essais de fonçage préalables ont eu lieu fin 2018 afin de confirmer la faisabilité ainsi que les conditions de mise en œuvre des ouvrages prévus. Ces essais ont aussi permis de mesurer la capacité portante des pieux en vue d’une éventuelle optimisation. A l’issue de ces essais, la faisabilité est confirmée avec, de plus, des possibilités d’optimisation en phase d’exécution.

Conclusion
Le démarrage des travaux sur site est envisagé en 2023. Leur bon déroulement, dans le respect des impératifs environnementaux et fluviaux, sera le gage d’une opération réussie. La méthode de construction par ripage ainsi validée pourra alors être étendue à d’autres sites. La problématique des allongements d’écluses est en effet courante sur le réseau à grand gabarit des Hauts-de-France.
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Ref. author:  
Claus Kunz – Bundesanstalt fuer Wasserbau  
Kussmaulstrasse 17, D-78187 Karlsruhe, Germany  
claus.kunz@baw.de  

Concept for standardised restoration and extension of ship locks

1 Introduction

The river Neckar was expanded as a waterway from 1921 to 1968 in three construction phases. Twenty-seven barrages, aged now between 60 and 100 years, overcome altogether a height difference of about 160 m over a distance of 203 km. The locks are mainly double locks, each 110 m long and 12 m wide. On the Neckar currently cargo vessels with a maximum length of 105 m and a width limited to 11.45 m width operate. The fairway with a minimum width of 36 m has been cleared since the year 2000 to 2.80 m depth. Traffic on the Neckar in 2016 was around 5.6 million tonnes. Expansion and maintenance of a waterway are the responsibility of the German Federal Waterway and Shipping Administration (WSV), represented by waterways and shipping offices and waterway construction offices, here the ANH.

The need for systematic basic repairs of the existing locks and the extension of at least one lock per barrage led to a concept for a standardised repair and extension concept [1]. Responsible for this work was Bundesanstalt fuer Wasserbau (BAW), Karlsruhe, as the central consultant of the WSV in waterway engineering issues. Experts of the WSV and ANH had been involved.

The characteristics of the project had been:  
• view of the entire lock chain on the river Neckar  
• development of system solutions (standards)  
• optimization or minimization of future maintenance  
• faster action in the event of replacement or replacement in the event of failure or in case of an accident (operational and economic benefit).

Figure 1 a/b: Simulation of the lock extension – usable length - towards downstream (right lock chamber Neckargemünd; a: present b: future (Source: ANH, [2])
2 Structure of the project

Standardisation efforts in existing structures, such as on the Neckar in the double locks and while maintaining the lock operation in the respective neighboring lock, represent a special degree of difficulty. This is due to the different construction times of the individual locks, in the different types of hydraulic steel structures as well as in different equipments. In addition, there are different locations of the locks to be extended, sometimes on the water side, sometimes on the land side, and there are different neighboring structures such as weirs or power plants or fish passages or traffic areas.

The work was based on analyses of the existing structures, materials, components, systems and boundary conditions and included experiences from BAW’s expert work as well as WSV’s experience. The aim was to find best practice solutions. Group formations were made for fall height-dependent constructions, as well as for the different lock positions on the water side and land side, as for the directions of extension after upstream or downstream direction, as well as for the different neighboring structures. As a result, standardisation options with different depths were developed within modules, which are further developed according to the planning and execution process [1].

The general procedure can be described by acquisition of the actual condition, determination of the repair requirement, determination of the extension requirement, development of a repair concept, development of extensions and has been performed and documented in modules.

3 Standardisations

First, general planning principles and special design principles laid down for the Neckar were compiled in the sense of a "specification". Loads have been specially identified and defined for some actions not included in the relevant codes. These stipulations served as a guideline for the other modules and are intended to provide the project-specific planning for unified design principles that are indispensable for execution [1].

The design ship was defined with $L = 135 \text{ m}$, $B = 11.45 \text{ m}$ and $T_{\text{max}} = 2.8 \text{ m}$. The chamber width of currently 12 m should be maintained at the extension. The effective length should be at least $L_N = 140 \text{ m}$, the usable width $B_N = 12 \text{ m}$, the water depth at least $T_N = 3.2 \text{ m}$. The freeboard should be 1.0 m. Locks with a fall of more than 5 m will be equipped on one side with 3 floating bollards. The calculated number of lockings per lock chamber is 15 locks/day x 365 days/year x 100 years $\approx$ 550,000 load cycles. Neckar-specific impacts were determined for ship impact (also for rope impact protection) and for ice loads.

Since the planned repairs also require a static assessment of the existing structural substance, the development of a verification methodology for the stability and load-bearing capacity of existing structural parts as a Neckar specific solution was unavoidable. Furthermore, old building materials (plain concrete, reinforcing steel BSt I or III) and old construction methods (unreinforced constructions), which are no longer covered by the current codes, had to be taken into account. DIN 19702, [3], allows the inclusion of knowledge during the operating phase of a hydraulic structure in the respective expenditure, which can be used to deviations from the new construction standard. For this reason, standardisation required a modification of the current codes and verification formats in which the special conditions at the Neckar locks were sufficiently taken into account.

Further on it was examined which basic repair and reinforcement requirements exist and to what extent the necessary static verifications and repair measures can be standardised, [1], [2]. Essential parts of the lock chambers on the Neckar that are to be extended should remain in place and be used
on a long-term basis. The basis for an analysis had been the material characteristics and structural conditions. With regard to durability requirements, types and groups of damage were formed and assessed for durability risks. On this basis, possible repair solutions for the water-side, vertical wall areas and horizontal platforms of locks were developed. These include local concrete repairs and crack injections concepts as well as concrete removal and reproduction of anchored and reinforced concrete shells. In most cases it was found that, if the old substance was more damaged, the cast-in-situ facing shell with approximately 40 cm wall thickness was the optimal solution. Similar standardised restoration had been worked out for the remaining lock heads.

Standardisations for the new heads as part of the extension had been worked out by the development of unified energy dissipation systems and shape definitions of the solid construction in the longitudinal direction and for cross sections.

Concerning hydraulic steel structures the development of modular mitre gate constructions had been adapted to the different falls in river Neckar. For a range of up to 10 m fall, the same system components for the lower and upper gate parts are all the same and a variable middle gate part and were recommended. The lower gate part contains 4 radial gate fillers. In case of required flood protection, radial gates with a filling shell (flood discharge through the lock) had been designed for the head gate.

The equipment in the lock (bollards, ladders, ship impact protection, etc.) had been standardised.

4 Conclusion and outlook

The standardisation of restoration and extension of locks in river Neckar as part of a special concept was already a structured help for the engineering offices entrusted with the individual projects. The concept enables standardised methods and structural elements. In the meantime, there has also been an incorporation benefit for new employees of the ANH.

Over the past few years, experience has been gained with the standardisation of projects carried out in the Neckar based on the described concept [1], and preferred solutions with possible standardisation alternatives have been emerged. In addition, codes have changed in the past time. Therefore, an update of the technical concept is currently in progress. For example, the load assumptions are adjusted to new codes. Verifications will be based on the meanwhile edited BAW guideline "Tragfähigkeitsbewertung für bestehende massive Wasserbauwerke (TbW)", [4]. For the repair of the chamber walls, the 40 cm thick facing shell will be exposed as preferential. There are also further developments in the static reinforcement by means of tension anchorages in the chamber walls.

References
Title:
Performance of a lock with vertical lift gates after renovation; case study including measurements and optimisation of levelling system

Abstract:
Lock Delden, an 80-year-old lock in the Twentekanaal (a channel in the eastern part of The Netherlands) was renovated in 2018. The renovation of the lock included additional safety measures to support a bigger class of vessel ("Class Va"), see also Veldman (2018). The levelling system with slowly opening lift gates remained unchanged. Despite the original gate lifting schedule was re-implemented, skippers noticed an increase in lock-levelling time, and thus in lock-cycle time after the renovation.

This paper describes the analyses of the levelling system after the renovation and the proposed improvements. The reengineering of the gate-lifting schedule of the renovated lock looked promising. The computed levelling of lock Delden is even faster than before the renovation. The optimised gate-lifting schedule has been implementation on 12 September 2019. Results of validation measurement are satisfactory.

Introduction to lock Delden
The lock chamber is 140 m long (between the gates), 12 m wide, and has a lift height of 6 m. Levelling is provided by controlled lifting of the lift gate in the lock head. The water flows through the horizontal slit between the lock gate and the upper sill through the stilling basin into the lock (see figure with upper lock head and detail).

The slit below the upper gate has a special shape that controls the discharge. At the bottom of the upper gate a 1 m high flow guiding sheet is fitted, that guides the flow down to the stilling basin. The present guiding sheet (see red line) origins from 1994, when the gate was replaced. However, the original drawings from the lock (1933) indicate a different S-shaped guiding sheet (see overlay with purple line).
The original S-shaped flow guiding sheet has been designed to slowly increase the slit and the discharge in the first minutes of the gate lifting. In an introduction to the Twenthe-kanaal, Wentholt (1932) explains that the original system was developed and tested on a 1:30 scale at the Waterloopkundig Laboratorium in Delft (now Deltares). The advantage at that time was, that it allows a low (0.005 m/s) but constant gate-lifting speed while at the same time the tranquillity in the lock chamber remains acceptable. Disadvantage is that the sheet at the bottom of the gate develops a huge upward hydraulic pressure on the gate during the lifting. This upward force must be accounted in the counterbalance of the gate lifting system.

The gate in the lower lock head is lifted from a sill in the horizontal floor of the lock head. This results in a linear increase of the slit with the height the gate is lifted. The vertical lifting of the gates follows the gate lifting schedules.

Increase of levelling time after renovation
In spring 2017, about a year before the renovation of the lock, prototype measurements were carried out in lock Delden to investigate the effect of larger vessels on the locking process (Veldman, 2018). The measurement included the position of the vessel and the water level in the canal and in the lock. The analyses of the measurements indicated that the filling of the lock took about 13.5 min and the emptying 9.5 min till the gate was completely lifted and the vessel could leave the lock (Aktis, 2017).

The gut feelings of the skippers on an increase of the levelling time was confirmed from a video registration in July 2018, few weeks after the completion of the renovation. The video indicated that the filling took 14.5 min (=1 min extra) and the emptying 12.5 min (=3 min extra) till the gate was completely lifted and the vessel could leave the lock (Aktis, 2019a). According to the conditions for the renovation, an increase of levelling time was not allowed. Bousmar (2018) showed that for some locks, the reengineering of the valve opening law (or gate lifting schedule) can be applied successfully to optimise the levelling. For lock Delden, Rijkswaterstaat decided to reengineer the gate-lifting schedule to reduce the levelling time to the original duration or less.

Analyses of levelling process
The renovation of the lock included also the mechanical parts for the lifting of the lock gates. The requirement was to maintain the levelling time. It was decided to reproduce the original gate lifting schedule. In addition is was decided to start the final lifting of the gate already when the remaining head difference over the gate is less than 0.1 m. For this purpose, the levels of the gates and the water levels in the lock chamber and the outer harbours are permanently measured and stored by the lock operation system.

Registration of the gate level indicated, that, despite the exact reproduction of the gate lifting schedule, the time required for locking increased. Detailed analyses of 5 lock cycles indicated that the lifting schedules are indeed reproduced accurately. Nevertheless, the time from the start of levelling up to the gates being completely lifted varied from 14 to 19 min for the filling of the lock and from 13 to 17 min for the emptying of the lock. Both levelling times are significantly longer than before the renovation.

Analysis of the water-level registration from the lock-operation system questioned the reliability of the registration. Measurements of the water level in the lock and the outer harbours confirmed several imperfections in the water level registration of the lock-operation system. Moreover, the system produces only a moving average over 100 s, with a time-lag of 50 s.

These findings indicate that the water level registration by the lock operation system is not a reliable data source for controlling the opening of the lock gate. The inaccurate water level registration is most probably one of the reasons for the increase of the lock-levelling time, especially, since the final opening of the gate is related to the (measured) water head over the lock gate being less than 0.10 m.

Options to reduce the lock levelling time
The options to reduce the time for the levelling of the lock till the gate is fully lifted are:
1. Improve the accuracy of the water level registration by the lock operation system;
2. Reengineering of the gate-lifting schedules.
The accuracy of the water level registration is included in the lock renovation programme. The present work concentrates on the reengineering of the gate lifting schedules for filling and emptying.

Important aspects are:
- The levelling-discharge gradient to reduce wave amplitude in the lock chamber (PIANC, 2015);
- The upward hydraulic pressure on the flow guiding sheet under the upper gate (<50 kN);

The gate lifting scheme has been optimised in several steps by try and error. For each try the force on the vessel in the lock chamber was checked with LockFill Version 5.1 (Deltares, 2016). Finally, the following measures have been implemented in the optimised gate lifting schedule that improve the levelling:
- Variation of the gate lifting speed;
- Reduce the number of stops during the gate lifting to one (prior to final opening); and
- Final opening of the lock gate on fixed time from start of levelling (not on measured water levels).

The original and the optimised gate-lifting schedules are presented in the figures below. The dashed lines indicate the gate lifting schedule, the solid lines the water level in the lock chamber. The purple and red lines present measurements (21 and 22 February 2019) with the original gate-lifting schedules. The green lines represent the optimised gate-lift schedules implemented on 12 September 2019.

The upper-gate was lifted in three steps with three stops in between: lifting up to 0.2 m, a 30 s stop, lifting up to 0.5 m, a 270 s stop, lifting up to 1.2 m and a stop till the remaining head is less than 0.1 m before the gate was lifted at higher speed for the final opening. The solid lines indicate that the filling of the lock was completed after about 12 min. However, for yet unknown reasons, the total time for the levelling and opening of the gate varied from 13.9 to 17.7 min.

The green lines show that filling has been accelerated. The total time for filling and opening of the gate has been reduced to 11.9 min.

The lower gate was lifted in one step and one stop: lifting to 0.5 m and a stop till the remaining head was less than 0.1 m before the gate was lifted at higher speed for the final opening. The solid lines indicate that the emptying of the lock was completed after about 9 min. The total time for the levelling and opening of the gate varied from 10.4 to 13.3 min.

The green lines show that emptying has been accelerated. The total time for emptying and opening of the gate has been reduced to 8.5 min.

LockFill simulations for the optimised gate-lifting schedules indicate that the maximum longitudinal forces exerted on the vessel is reduced to 0.9% of the displacement (from 1.3%).

**Conclusion:**
After the renovation of the mechanical and electronic part for the lifting of the lock gates the original gate lifting schedule was reproduced, but the levelling time was increased. The reengineering of the lock levelling system and the implementation of an adapted lifting schedule accelerated the lock levelling and improved the tranquillity in the lock. The main adaptations in the gate lifting schedule are:
- Remove the stops (the initial lifting schedule comprised various stops);
- Vary the gate lifting velocity (the initial lifting schedule was based on one fixed lifting velocity);
- Accelerate the procedure for the determination of the remaining head over the gate for the final opening of the gate (the moving average period was reduced and the head at final opening increased).

The reengineering of the gate-lifting schedules reduces both the filling and the emptying with at about 2 min or even more, while the longitudinal forces exerted on the vessel have been reduced to near the criterion for a Class Va lock. The lock-cycle time has been reduced with at least 10%.
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Challenges in the design and construction of the New Lock in Terneuzen; A special Dutch - Flemish collaboration

Abstract:

Introduction
The existing lock complex at Terneuzen connects the Ghent-Terneuzen Canal to the Western Scheldt Estuary and is a bottleneck for both inland vessels and sea going vessels.

In the current situation the lock complex consists of three locks:
- the Eastern Lock, built in 1963, 280m long and 24m wide, for inland navigation only;
- the Middle Lock, built in 1910, 140m long and 18m wide, for inland navigation and sea going vessels with a draught up to 7.2m;
- the Western Lock, built in 1968, 290m long and 40m wide, for inland navigation and sea going vessels with a draught up to 12.5m;

At present the locks have reached their maximum capacity. This causes increasing waiting times. For this reason and because the locks are getting old, the current situation is no longer sufficient for the future.

New Lock
To ensure a safe and reliable passage in the future at this important link in the Rotterdam-Paris inland waterway route and to provide access for larger vessels to the port of Ghent, a new large lock is being built amidst the existing locks.

The new lock is sized for “New Panamax” size vessels, and will have a chamber size of 427m long by 55m wide and a depth of approximately 16 m.

In addition the project also includes more works like improvement of flood defense, approach channels and buildings. This requires extensive earthwork, demolition of the existing Middle lock and relocation of existing facilities.
**Project**
Client is the Flemish-Dutch Commission for the river Scheldt, named VNSC. This is a Flemish and Dutch governmental co-operation on projects with common interest in the Scheldt estuary along the Dutch-Belgium border. Both the Dutch Rijkswaterstaat and the Flemish Department of Mobility and Public Works take part in this cooperation to realise the New Lock in Terneuzen.

**Contract**
The Design and Build contract for the project has been awarded in 2017 to the Dutch/Flemish Joint Venture Sassevaart which consists of the following companies:
- DIMCO B.V. (DEME Infra Marine Contractor)
- Dredging International N.V.
- BAM Infra B.V.
- BAM Contractors N.V.
- Algemene Aannemingen Van Laere N.V.

Contract price is 626M€ (excl. VAT).

**Major challenges**
This project comprises some unique elements and challenges which have to be dealt with.

The new lock is situated in the Netherlands, while the project is largely financed by the Flemish Government. A combined Flemish/Dutch project team has been formed. This offers both a political and cultural challenge as well as an opportunity to use “best of both worlds”. Between the Flemish and Dutch culture, the approach varies towards design and construction techniques, project organisation, contractual terms and contract management philosophy.

**Design and lay-out**
Situating a new lock of this size in between the existing locks, is complicated. There needs to be enough space for the new lock itself, but also for the lock approaches, in such way that ships and road traffic both can pass on a reliable, safe and smooth way during both construction time and its operational lifetime.

Within the framework of keeping costs for both construction and maintenance at an acceptable level, meeting the RAMS requirements (Reliability, Availability, Maintainability and Safety) is a challenge.

**Levelling of the lock**
The required optimisation of the levelling system to provide for a smooth and safe lockage is difficult, given lock size, locking conditions and the density difference between fresh and salt water.

As the target water level of the canal is close to high tide, the prevailing condition at Terneuzen is the filling of the salty lock with fresh (brackish) water from the canal. The maximum head during levelling, with the sea at the lowest astronomical tide, is 5.07 m, and the target levelling time is 20 min. Because there is a density difference between the approach harbours, during levelling the
hydrodynamic forces on the moored vessel in the lock chamber do not only result from translatory waves but also from density currents.

Based on elaborate scale model studies, it has been decided to build a levelling system with a longitudinal culvert. The lock will be filled and emptied through two bottom grids, located at about one quarter and three quarters of the chamber length. By distributing the discharge over these two grids, the resulting translatory waves and density currents in the longitudinal direction are significantly reduced, and the corresponding longitudinal forces as well. Then, the determining forces are the transversal forces caused by the density currents. Too high transversal forces, only occurring with the largest vessels at the deepest drafts, can be avoided by mooring the vessel not too close to the gate. For non-standard conditions additional operational measures will be defined to ensure a safe levelling process.

**Construction**

The new lock has to be built in the limited space of the existing lock complex, while throughout the project duration the primary flood defence, the locking process, water discharge and road traffic have to be maintained on a reliable and safe level.

Damage to the existing locks has to be avoided at all times. This requires specific building techniques.

The geotechnical and geohydrological conditions at the site combined with the lock dimensions require specific construction methods. Lowering of the ground water level is not allowed to prevent soil deformations and damage to surrounding structures. Diaphragm walls are designed for parts of the lock chamber wall and the building pit. The building method to create diaphragm walls is very effective, but requires specific attention to ensure durable quality.

**Conclusion**

All together it is clear that it takes quiet some effort to realize this new large lock within this border crossing project in this specific area, but when the new lock is finished and ready for use, a safe and reliable passage will be available for the coming decades.

Currently (2019) the construction of the works is in full progress. The new lock should be operational by the beginning of 2023.
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Réf. auteur :
Gwenaël CHEVALLET – BRLi
1105 avenue Pierre Mendès France, 30001 Nîmes Cedex 5, FRANCE
Gwenael.Chevallet@brl.fr

Co-auteurs :
Chloé CHENE, Antoine HALBARDIER, Franck RANGOGNIO
Chloe.Chene@brl.fr ; Antoine.Halbardier@brl.fr ; Franck.Rangognio@brl.fr

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Modélisation 3D-CFD, infrastructures de navigation, hydraulique, écluse, sassée

Titre : Reconstruction et allongement des écluses de Méricourt : étude du fonctionnement hydraulique de l’ouvrage

Avec plus de 60 ans d’expérience dans les infrastructures hydrauliques d’envergure, BRL Ingénierie est un acteur de référence du domaine de la navigation tant en France qu’à l’Export.

Introduction
Le dimensionnement d’une écluse et la définition des consignes de gestion des sassées associées sont des problèmes complexes abordés jusqu’à aujourd’hui à l’aide :
- Soit de modèles physiques réduits qui peuvent être lourds à mettre en œuvre,
- Soit de méthodes empiriques souvent couplées avec des approches calculatoires spécifiques à chacune des problématiques soulevées :
  - Études hydrauliques 1D en régime transitoire permettant de vérifier le respect de critères de vitesses moyennes et de pente de ligne d’eau dans le sas ainsi que les temps de sassée associés,
  - Modélisations hydrauliques 3D en régime permanent centrées sur les organes vannés de remplissage ou de vidange,
  - Application d’abaques ou mise en œuvre d’approches calculatoires simplifiées pour la problématique liée aux amarrages,
  - Retour d’expérience des exploitants.

Les équipes de BRLingénierie ont mis en œuvre une méthodologie permettant de répondre à toutes ces interrogations de manière calculatoire et combinée à l’aide d’une modélisation hydraulique transitoire 3D-CFD (menée avec le logiciel Flow-3D© développé par Flow Science).

Méthode
Le projet de rénovation et d’allongement des écluses de Méricourt sur la Seine vise à reconstruire les écluses existantes car ces dernières présentent des désordres structurels visibles notamment à travers la déformée des bajoyers. Actuellement, le site est équipé de deux écluses fonctionnelles en parallèle, l’une disposant d’un sas de 160 m utile et l’autre d’un sas de 185 m utile. Le maître d’ouvrage profite de ces travaux pour entre autre :
- allonger l’écluse de 160 m de manière à uniformiser les longueurs utiles des sas, et ainsi sécuriser l’axe de navigation ; cet allongement induit de fait une augmentation des volumes de sassée,
- mettre en place des bollards flottants en lieu et place des bollards fixes existants,
- remplacer les organes vannés aval (2 aqueducs remplacés par 18 vантelles).

Ces changements s’accompagnent d’une exigence forte du maître d’ouvrage : le maintien a minima des temps de sassée actuels de l’ordre de 15 min en respectant principalement les efforts maximaux sur les bollards (250 à 300 kN par bollard [25 à 30 tonnes]).
La modélisation présentée ici concerne l’état projeté de l’écluse n°1 (L=185 m, l=17 m). Ce modèle intègre :
- une géométrie 3D des écluses réalisée à l’aide d’outils dédiés (Autodesk Revit™ et Allplan™),
- une modélisation hydraulique transitoire 3D-CFD pouvant simuler tous les écoulements complexes [écoulements stationnaires, remous, entraînement d’air, cavitation, coups de bélier…],
- des objets mobiles :
  o couplés au fluide :
    ▪ un bateau de type Grand Rhénan (classe CEMT Va, L=110 m, l=11,4 m, tonnage 1500 à 3000 tonnes),
    ▪ des bollards flottants,
  o respectant des consignes de gestion : vannes des aqueducs amont ou vantelles aval.
- un module d’amarrage liant le bateau aux bollards,
- un module de collision entre le bateau et les bajoyers de l’écluse.
Le modèle mis en œuvre comprend environ 500 000 mailles. Les temps de calculs associés pour simuler un remplissage ou une vidange du sas (temps réel de l’ordre de 10 à 15 min) sont compris entre 6 h et 12 h environ sur un ordinateur de calcul dédié. La définition des bonnes pratiques de maillage pour concilier les temps de calculs et la précision/fiabilité des résultats attendus a demandé de nombreux tests itératifs.

Par ailleurs, compte tenu des importants temps de calculs sur Flow-3D©, des estimations préliminaires des lois d’ouverture des organes de vidange/remplissage ont été effectuées à l’aide de simples tableurs ou de modélisations 1D à partir de critères hydrauliques issus de la littérature, de données du programme du maître d’ouvrage et éventuellement du retour d’expérience de l’exploitant.

Enfin, le code Flow-3D© est partiellement ouvert ce qui a permis à BRLi de développer des routines de calculs répondant spécifiquement aux problèmes posés.

Durant les quelques mois qui ont été nécessaires à BRLi pour mettre en œuvre cette méthodologie, de nombreux phases de test et de validation ont eu lieu :
- prise en compte des bollards flottants (calcul itératif d’un compromis concernant la densité des bollards pour éviter de trop grandes oscillations ou une inertie trop forte),
- nombre, caractéristiques et positions des amarres,
- niveau de remplissage des bateaux/barges, type de convois,
- consignes de gestion de remplissage ou de vidange.

Figure 3 : Comparaison des lois de vidange/remplissage analytique et issue de Flow-3D©

Figure 4 : Validation du comportement des bollards flottants
Résultats

Une fois les conditions limites imposées (niveaux d’eau des biefs amont et aval), les caractéristiques du bateau et du plan d’amarrage choisies, le modèle mis en œuvre a permis de répondre aux points suivants :
- Durée d’une sasée (c’est-à-dire un cycle de vidange ou de remplissage de l’écluse) pour des consignes de gestion données,
- Conditions hydrauliques 3D des écoulements dans le sas (distribution des vitesses principalement),
- Efforts transmis dans les bollards lors d’une sasée.

*Figure 5: Simulation de remplissage de l’écluse n°1 en état projeté (2 amarres) - Grand Rhénan*

Il a ainsi été possible d’optimiser les consignes de gestion de remplissage ou de vidange, à savoir les lois d’ouverture des vannes, de manière à :
- garantir le respect des efforts cibles dans les bollards (250 à 300 kN par bollard [25 à 30 tonnes]),
- minimiser la durée des sasées (de l’ordre de 10 à 11 minutes) en respectant les contraintes matérielles des organes vannés (plage des vitesses de manœuvre de la centrale à huile notamment).
Discussion

La modélisation 3D-CFD qui a révolutionné l’hydraulique des ouvrages (notamment le dimensionnement des barrages) était jusqu’à présent limitée pour sa mise en œuvre dans les ouvrages de navigation par l’absence d’objets mobiles couplés ou non aux mouvements du fluide et l’absence de modules spécifiques d’amarres.

Le modèle 3D mis en œuvre a permis de répondre grâce à un seul et même outil à toutes les problématiques liées à une sassée (durée de vidange/remplissage, sollicitations hydrauliques, efforts sur le bateau et efforts sur les bollards flottants…). Il constitue de fait une réelle avancée pour l’état de l’art. En effet, cette méthodologie est applicable à tout type d’écluse et tout type de bateau.

Les résultats des modélisations effectuées jusqu’à aujourd’hui sont particulièrement satisfaisants. En effet, ils respectent tous les ordres de grandeur calculés à l’aide d’abaques, de méthodes simplifiées ou issus du retour d’expérience de l’exploitant (lois de vidange/remplissage, coefficients de débits des organes vannés, efforts maximaux sur les bollards…).

Concernant les efforts sur les bollards (paramètres essentiels de dimensionnement), les résultats sont évidemment sensibles aux lois de remplissage mais également à la longueur libre des amarres et à leur rigidité ainsi qu’au plan général d’amarrage (nombre et position des amarres).

Les axes d’amélioration qui ont été identifiés pourraient être les suivants :
- Possibilité d’asservir les consignes de manœuvre des organes vannés à des niveaux d’eau, à des vitesses… : en effet, Flow-3D© autorise d’asservir les mouvements d’objets mobiles à différents paramètres, ce qui constitue un atout majeur par rapport à un modèle réduit par exemple,
- Prise en compte d’une géométrie plus détaillée des bateaux et des convois : les embarcations ont été modélisées de manière simplifiée en l’absence d’objets 3D existants ; il est envisageable de mener ce même type de modélisation avec des objets 3D décrivant les bateaux ou les barges de manière très fine,
- Calage du modèle sur des données observées in situ : même si les résultats de la modélisation respectent tous les ordres de grandeur issus de la littérature ou du retour d’expérience de l’exploitant, il serait intéressant de pouvoir les confronter à des données de terrain (données réelles de débits, de courantométrie, d’efforts sur les amarres…).

Références
Title: Reconstruction and extension of the Méricourt locks: study of the hydraulic operation of the structure

With more than 60 years of experience in large scale hydraulic infrastructures, BRL Ingénierie is a leading company in the navigation sector both in France and abroad.

Introduction

The dimensioning of a lock and the definition of the associated lock management instructions are complex problems that have been addressed to this day using:
- Either scaled physical models that can be heavy to implement,
- Or empirical methods often coupled with calculation approaches that are specific to each of the issues raised:
  - 1D transient hydraulic studies to verify compliance with the criteria of average speed and water line slope in the lock as well as the associated locking times,
  - 3D steady-state hydraulic models centered on the filling and emptying valve elements,
  - Application of charts or simplified calculation approaches for mooring problems,
  - Feedback from operators.

The BRLingénierie teams have implemented a methodology to answer all these questions in a computational and combined way through transient 3D-CFD hydraulic modelling (using Flow-3D© software developed by Flow Science).

Methodology

The renovation and extension project of the Méricourt locks on the Seine aims to rebuild the existing locks as they present visible structural disorders, particularly through the deformation of the lock walls. The site is currently equipped with two parallel functional locks, one with a 160m capacity lock chamber and the other with a 185m capacity lock chamber. The project owner takes advantage of this project to, among other things:
- Extend the 160m lock in order to standardize the capacity of the locks, thus securing the navigation axis. This extension induces an increase in the filling and emptying volumes.
- Install floating bollards instead of the existing fixed bollards,
- Replace the downstream valve parts (2 aqueducts replaced by 18 valves).

These changes come with a strong requirement from the owner to maintain locking times close to the current 15min locking time, and at the same time respect the maximum forces on the bollards (250 to 300kN per bollard [25 to 30 tons]).
The model presented here concerns the project situation of lock n°1 (L=185m, w=17m). This model includes:
- A 3D lock geometry designed with dedicated tools (Autodesk Revit™ and Allplan™),
- 3D-CFD transient hydraulic modelling capable of simulating all the complexities of the flow [stationary flows, eddies, air entrainment, cavitation, water hammer…],
- Moving objects:
  - Coupled to the fluid:
    - A Grand Rhénan type boat (ECMT class Va, L=110 m, w =11,4 m, capacity 1500 to 3000 tons),
    - Floating bollards,
  - Respecting management instructions: upstream aqueduct gates or downstream valves.
- A mooring module linking the boat to the bollards,
- A collision module between the boat and the lock walls.
The model implemented includes about 500,000 mesh elements. The associated calculation times to simulate a filling or emptying of the lock (real time of 10 to 15 minutes) are between approximately 6h and 12h on a dedicated calculation computer. The definition of good meshing practices to reconcile computation times and the precision/reliability of expected results required numerous iterative tests.

In addition, given the long calculation times on Flow-3D©, preliminary estimates of the opening laws of the filling/emptying devices were made using simple spreadsheets or 1D models based on hydraulic criteria from the literature, data from the contracting authority’s program and feedback from the operator.

Finally, the Flow-3D© code is partially open, which has allowed BRLi to develop computation routines that specifically address the problems encountered.

During the few months it took BRLi to implement this methodology, many tests and validation phases took place:
- Consideration of floating bollards (iterative process to find a compromise concerning the density of the bollards to avoid oscillations or too large inertia),
- Number, characteristics and positions of the mooring lines,
- Level of filling of boats/barges, type of convoys,
- Instructions for filling or emptying management.

Figure 3: Comparison of analytical emptying/filling laws and Flow-3D© output

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- Consideration of floating bollards (iterative process to find a compromise concerning the density of the bollards to avoid oscillations or too large inertia),
- Number, characteristics and positions of the mooring lines,
- Level of filling of boats/barges, type of convoys,
- Instructions for filling or emptying management.

Figure 4: Validation of floating bollards behaviour
Results

Once the boundary conditions implemented (forebay and tailbay water levels) and the characteristics of the vessel and the mooring plan are chosen, the model implemented made it possible to meet the following points:

- Duration of a filling or emptying cycle for given management instructions,
- 3D hydraulic conditions of the flows in the airlock (mainly velocity distribution),
- Efforts transmitted in the bollards during a filling or emptying cycle.

*Figure 5: Simulation of filling of lock n°1 – project situation (2 mooring lines) - Grand Rhénan*

It was then possible to optimize the filling or emptying management instructions - i.e. the laws governing the opening of the valves – in order to:

- Ensure compliance with the maximum forces in the bollards (250 to 300 kN per bollard [25 to 30 tons]),
- Minimize the duration of the locking times (about 10 to 11 minutes) while respecting the material constraints of the valve components (range of operating speeds of the oil circuit pump in particular).

*Figure 6: Simulation de emptying of lock n°1 – project situation (2 mooring lines) - Grand Rhénan*
Discussion

3D-CFD modelling, which has revolutionized the hydraulics of structures (in particular the dimensioning of dams), has so far been limited for its implementation in navigation structures by the absence of moving objects coupled or not to the movements of the fluid and the absence of specific mooring modules.

The 3D model used made it possible to answer all the problems related to locking with a single tool (emptying/filling time, hydraulic loads, forces on the boat and forces on floating bollards, etc.). It is in fact a real step forward for the state of the art. Indeed, this methodology is applicable to all types of locks and all types of vessels.

The results of the modelling carried out so far are particularly satisfactory. Indeed, they comply with all orders of magnitude calculated using charts, simplified methods or based on the operator’s feedback (emptying/filling laws, flow coefficients of the valves, maximum forces on the bollards, etc.)

Concerning the forces on the bollards (essential dimensioning parameters), the results are obviously sensitive to the filling laws but also to the free length of the mooring lines and their rigidity as well as to the general mooring plan (number and position of mooring lines).

The areas for improvement that have been identified could be as follows:
- Possibility to control the operating instructions of the valves using water levels, speeds…: indeed, Flow-3D© allows the control of moving objects movement based on various parameters, which is a major advantage compared to a reduced model for example,
- Taking into account a more detailed geometry of boats and convoys: boats have been modelled in a simplified way in the absence of existing 3D objects; it is possible to carry out this same type of modelling with a 3D objects describing boats and barges in a very detailed way,
- Calibration of the model on observed in situ data: even if the modelling results respect all orders of magnitude from the literature or the operator’s feedback, it would be interesting to be able to compare them with field data (actual data on flow rates, current measurement, mooring forces, etc.).

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Ref. author:
Tom Stephens – TenCate Geosynthetics
Pendergrass, Georgia, USA
t.stephens@tencategeo.com

Co-author:
Nicolas Ruiz – TenCate Geosynthetics
Bogota, Colombia
n.ruiz@tencategeo.com

Keywords:
Geobag, Geogrid, Canal, Coffer Dam

Title:
Innovative Technology Saves 100-Year-Old Panama Canal Locks

Paper:

Abstract
The Panama Canal opened for business 105 years ago and was hailed as the eighth wonder of the world and an engineering marvel of the time. Since 1914, the originally three sets of dual parallel lane locks are still in operation in their original chamber dimensions of 33m wide x 300m long x 12m deep. However, ships have become larger and the tugs to maneuver them in and out of the locks have become more powerful. The latest version of Panama Canal tug is a 25m long 4,400 HP tug of azimuthal propulsion. With this increase in tug power prop wash combined with the continual flow of water out of the canal has created erosion along the dividing wall foundation and 2.0m thick approach slab. This erosion reached the extent that all three sets of locks were in danger of a catastrophic structural collapse. Seour erosion was detected using sonar digital imaging in areas to - 20m of depth. This paper will detail how the Panama Canal Authority (ACP) Engineering and Maintenance Division’s used an innovative technology and marine construction techniques to keep the canal operating 24 hours a day without any unscheduled interruptions to tonnage of operations. The paper will detail the science of testing of this innovative technology and the resulting design that
enabled this process to successfully solve the erosion problem using more than 300 sand-filled
gotextile bag units that weighed up to 90 metric tons (99 Imperial tons) each.

Keywords: Panama, Canal, Locks, Innovative, Technology, Scour, Erosion, Coffler Dam, Geotextile Bags

HISTORY OF PANAMA CANAL

The Panama Canal became the most consequential global waterway when the passenger ship SS Ancon entered the
canal thru the Gatun Locks from the Atlantic Ocean on the morning of August 15, 1914, and it has continued to be
so today. Traffic thru the canal has risen year over year thru the original locks from 807 transits the first year of
operation up to 13,114 transits in 2016 when the new set of locks opened. On September 4, 2010 the Chinese
freighter Fortune Plumb became the 1,000,000th ship to pass thru the canal. Total tonnage of transit has increased to
more than 330 million metric tons (363 tons) in 2016 with revenue exceeding $2.4 billion/USD per year. All of this
has been accomplished thru the three sets of 104-year-old locks.

![Image of SS Ancon](image)

**Figure 1 SS Ancon first ship to transit the Panama Canal Aug. 14, 1914**

The Locks

The original French design of the Panama Canal by Ferdinand de Lesseps was a sea level design without locks.
Construction started on January 1, 1881 and continued until 1987 when it was realized that the sea level design was
not practical, and an elevated canal and lock design was adapted. However, by this time tropical disease and
construction difficulties had bankrupt the French effort. In 1903 a revolution occurred in Panama creating the
Republic of Panama followed by the Hay – Bunau- Varilla Treaty. These events allowed the Unites States to take
over the French property where the canal was being constructed. On May 4, 1904 and a new canal design was
approved and construction restarted.

The new US design required three sets of locks. The Gatun Locks with three sets of parallel chambers would be
constructed on the Atlantic side. The Pedro Miguel Locks with one step of parallel chambers and the Miraflores
Locks two sets of parallel chambers would be built on the Pacific side. Each chamber would be 33.5 meters (110 ft.)
wide by 305 meters (1,000 ft.) long. The Gatun Locks would raise or lower ships 26 meters (85 ft.) to and from the
Atlantic Ocean allowing ships to transverse over the continental divide. The Pedro Miguel and the Miraflores Locks
would raise or lower ships 26 meters (85 ft.) to and from the Pacific Ocean. The first concrete was poured at the
Gatun Locks in August 1909. The final set of locks were completed four years later in May 1913. At the time or
their construction, their overall mass, dimensions and innovative design surpassed any similar existing structures,
and they are still considered to be an engineering wonder of the world. However, time, the environment, and
shipping technology have taken a toll on these magnificently designed yet simplistic operating structures.
The Problem

After construction, the Panama Canal was administered by the United States until 1979 when the Canal Zone was transferred to Panama and it has since been administered and operated by the Autoridad del Canal de Panama (ACP). The ACP Engineering and Maintenance Division has insured that the canal has been continually operated under safe conditions with modern upgrades and expansions. In December 2013 the ACP Engineering and Maintenance team conducted the annual comprehensive sonar investigation of the entrance and exit of each chamber of all three sets of locks. It was discovered that there were numerous areas of erosion scour that were occurring at the leading edge and under the approach slabs, along the dividing wall foundations, and around the base of several wing walls. This erosion was significantly more advanced and extensive than had been observed in previous years as detailed in Figures 4. The worst areas of erosion scour which were occurring at the Atlantic entrance to the Gatun Locks approach slab and dividing wall. These areas were monitored every month during 2014. In early 2015 ACP determined that the scour areas were continuing to progress, and two areas were becoming critical. Figure 5 details the extent of the erosion under the Gatun Locks Atlantic dividing wall in July 2015.
The Challenge

The ACP engineers determined in March 2015 that immediate action should be taken and that the challenge was three-fold; 1) the repair of the damage must not inhibit the flow of traffic thru the canal, 2) that the repairs must be done during routine maintenance schedule windows when one side of the locks was temporarily shut down for a 12 hour maintenance operation, and 3) the repairs must be able to be performed by the ACP construction and maintenance division due to the emergency nature of the repair.

Given these challenges, it was obvious that the repairs would be performed in water depths of up to -20m (66 ft.) with limited visibility and there was zero margin for error. Also, all the preparations for the repairs would be required to be performed at an offsite location and be able to mobilize and then demobilize in short order to stay within each 12-hour maintenance window.
The Solution

At the time the erosion scour was defined as critical, the ACP Construction and Maintenance division was working with the TentCate Geosynthetics group on a separate land-based slope erosion project. A joint meeting was scheduled between the two to brainstorm innovative solutions that may be incorporated to overcome the project challenges and to make the required repairs that would solve the erosion problems. This team quickly arrived at a potential solution that would meet all the challenges and could be rapidly executed.

The solution called for an underwater coffer dam to be installed adjacent to the base of the concrete structures under which erosion scour was occurring. The coffer dam would be formed using large flexible geosynthetic Geobag® containers that would be filled above water and lifted and placed underwater with a crane. Once in place, concrete would be pumped behind the coffer dam filling the void caused by the erosion scour. This could be a two-step process; Step 1) construction of the underwater coffer dam, and Step 2) filling the void with concrete. Each step would be performed separately during successive scheduled 12-hour maintenance windows.

TenCate had previously developed the system of filling, lifting, and placing 50m³ (65.4 yd³) sand filled Geobag® containers weighing 90 metric tons (99 Imperial tons). See Figure 6. The bags were fabricated from a 200 kN/m (1,142 lb./in.) woven polypropylene geotextile in a 5.0m wide x 5.0m long x 2.0m high (16.4 ft. long x 16.4 ft. wide x 6.56 ft. high) configuration. The bag had a top lid that was open for filling with sand and could be closed and mechanically sealed to permanently contain the sand. A unique 400 kN/m (2,284 lb./in.) friction harness was designed to provide a flexible system of lifting and placing. See Figure 7 for the calculations of the Friction Lifting Harness. The keys were: 1.) to have a barge mounted crane that could lift and place these mega bags as required within the Panama Canal Locks, 2.) that these Geobag® containers were massive enough that the propwash of 4,400 HP tugs could not move the 90 metric ton (99 tons) Geobag® containers, and 3.) yet to be flexible enough to contour to the eroded bottom of the canal to protect against future erosion scour.

ACP decided that the first repair operation would be the Atlantic side approach slab of the Gatun Locks. The design was finalized and the Geobag® and Lifting Harness materials were ordered and delivered to the site in August 2015.

Figure 6 First Test Lifting 50 m³ Sand Filled Geobag Container weighing 90 Metric Tons
Figure 7 Friction Harness Calculation

The Operation

The ACP determined that the Geobag® filling operation would be conducted at an ACP maintenance dock facility adjacent to the canal near the Gaillard Cut. The ACP 300 metric ton (330 Imperial ton) Titan barge mounted crane lifted the sand filled 90 metric ton (99 Imperial ton) Geobag® containers on to a transfer barge that was moved to the Gatun Locks to be ready to start as soon as the 12-hour maintenance window opened. See Figure 8. The first repair was to the scour erosion underneath the approach slab to the left chamber of the Atlantic approach to the Gatun Locks. The scour was the entire 33.5m width (110 ft.) of the slab down to a depth of 4.0m (13.1 ft.) below the slab and 8.0m (26.2 ft.) underneath the slab. This repair would require constructing the coffer dam using 10 or 12 of the 90 metric ton (99 Imperial ton) sand filled 50m3 (65.4 yd3) Geobag® containers. See Figure 9 for the first Geobag® placement using the 300 metric ton (330 Imperial ton) Titan barge crane. Figure 11 is the engineers sketch of the first 8 Geobag® positions as they were placed and the position of the Geobag® #9 ad #10 that are to be placed. It only took 10 bags to form the coffer dam and all were placed within the first scheduled 12-hour maintenance window that was a nighttime operation. During the next month’s 12-hour nighttime maintenance window more 300 m3 (393 yd3) of concrete was pumped behind the Geobag coffer dam and under the approach slab to fill the scour void under the slab.

This same procedure was followed to make the scour erosion repair to both sides of the dividing wall to the approach to the Gatun Locks as detailed in Figure 12 and similar repairs to the Miraflores and Pedro Miguel Locks.

Figure 8 The 300 Ton Crane Placing the 90 Ton Geobag® Containers on Transfer Barge
Figure 9 Placing the first 50m3 Geobag® in the Bottom of the Gatun Locks

Figure 10 Engineers Drawing of Geobag® Placement
Figure 11 Cross Section of Gatun Locks Dividing Wall Before Repair

Figure 12 Phase 1 Repair to the Gatun Locks Dividing Wall
Figure 13 Installing Geobag® Units Along Gatun Locks Dividing Wall

Figure 14 Method of Repair to Scour Erosion Under Slabs and Dividing Walls
The Results

There has been on scour erosion detected under the two Gatun Locks structures since the repairs using the Geobag® coffer dam technology in September 2015. In addition, the ACP has installed more than 300 Geobag® units to construct underwater coffer dams within the Panama Canal to allow the filling of under structure voids with concrete at other detected scour erosion at the Gatun Locks and the other two sets of Panama Canal locks.

CONCLUSIONS

This innovative technology and installation methodology are credited in allowing the ACP’s Construction and Maintenance division to be able to respond rapidly to a serious threat to the +100-year-old Panama Canal Locks that if not addressed in a timely fashion could have caused serious consequences or catastrophic damage resulting in the loss of millions of dollars of revenue and cost millions in construction cost to the Panama Canal. In addition, the innovative solution allowed for all the required repairs to take place during scheduled maintenance windows without any loss of traffic volume or revenue to the ACP.
Introduction
The existing lock complex at Terneuzen connects the Ghent-Terneuzen Canal to the Western Scheldt Estuary and is a bottleneck for both inland vessels and sea going vessels.

In the current situation the lock complex consists of three locks:
- the Eastern Lock, built in 1963, 280m long and 24m wide, for inland navigation only;
- the Middle Lock, built in 1910, 140m long and 18m wide, for inland navigation and sea going vessels with a draught up to 7.2m;
- the Western Lock, built in 1968, 290m long and 40m wide, for inland navigation and sea going vessels with a draught up to 12.5m;

At present the locks have reached their maximum capacity. This causes increasing waiting times. For this reason and because the locks are getting old, the current situation is no longer sufficient for the future.

New Lock
To ensure a safe and reliable passage in the future at this important link in the Rotterdam-Paris inland waterway route and to provide access for larger vessels to the port of Ghent, a new large lock is being built amidst the existing locks.

The new lock is sized for “New Panamax” size vessels, and will have a chamber size of 427m long by 55m wide and a depth of approximately 16 m.

In addition the project also includes more works like improvement of flood defense, approach channels and buildings. This requires extensive earthwork, demolition of the existing Middle lock and relocation of existing facilities.
**Project**
Client is the Flemish-Dutch Commission for the river Scheldt, named VNSC. This is a Flemish and Dutch governmental co-operation on projects with common interest in the Scheldt estuary along the Dutch-Belgium border. Both the Dutch Rijkswaterstaat and the Flemish Department of Mobility and Public Works take part in this cooperation to realise the New Lock in Terneuzen.

**Contract**
The Design and Build contract for the project has been awarded in 2017 to the Dutch/Flemish Joint Venture Sassevaart which consists of the following companies:
- DIMCO B.V. (DEME Infra Marine Contractor)
- Dredging International N.V.
- BAM Infra B.V.
- BAM Contractors N.V.
- Algemene Aannemingen Van Laere N.V.

Contract price is 626M€ (excl. VAT).

**Major challenges**
This project comprises some unique elements and challenges which have to be dealt with.

The new lock is situated in the Netherlands, while the project is largely financed by the Flemish Government. A combined Flemish/Dutch project team has been formed. This offers both a political and cultural challenge as well as an opportunity to use “best of both worlds”. Between the Flemish and Dutch culture, the approach varies towards design and construction techniques, project organisation, contractual terms and contract management philosophy.

**Design and lay-out**
Situating a new lock of this size in between the existing locks, is complicated. There needs to be enough space for the new lock itself, but also for the lock approaches, in such way that ships and road traffic both can pass on a reliable, safe and smooth way during both construction time and its operational lifetime.

Within the framework of keeping costs for both construction and maintenance at an acceptable level, meeting the RAMS requirements (Reliability, Availability, Maintainability and Safety) is a challenge.

**Levelling of the lock**
The required optimisation of the levelling system to provide for a smooth and safe lockage is difficult, given lock size, locking conditions and the density difference between fresh and salt water.

As the target water level of the canal is close to high tide, the prevailing condition at Terneuzen is the filling of the salty lock with fresh (brackish) water from the canal. The maximum head during levelling, with the sea at the lowest astronomical tide, is 5.07 m, and the target levelling time is 20 min. Because there is a density difference between the approach harbours, during levelling the
hydrodynamic forces on the moored vessel in the lock chamber do not only result from translatory waves but also from density currents.

Based on elaborate scale model studies, it has been decided to build a levelling system with a longitudinal culvert. The lock will be filled and emptied through two bottom grids, located at about one quarter and three quarters of the chamber length. By distributing the discharge over these two grids, the resulting translatory waves and density currents in the longitudinal direction are significantly reduced, and the corresponding longitudinal forces as well. Then, the determining forces are the transversal forces caused by the density currents. Too high transversal forces, only occurring with the largest vessels at the deepest drafts, can be avoided by mooring the vessel not too close to the gate. For non-standard conditions additional operational measures will be defined to ensure a safe levelling process.

**Construction**
The new lock has to be built in the limited space of the existing lock complex, while throughout the project duration the primary flood defence, the locking process, water discharge and road traffic have to be maintained on a reliable and safe level.

Damage to the existing locks has to be avoided at all times. This requires specific building techniques.

The geotechnical and geohydrological conditions at the site combined with the lock dimensions require specific construction methods. Lowering of the ground water level is not allowed to prevent soil deformations and damage to surrounding structures. Diaphragm walls are designed for parts of the lock chamber wall and the building pit. The building method to create diaphragm walls is very effective, but requires specific attention to ensure durable quality.

**Conclusion**
All together it is clear that it takes quiet some effort to realize this new large lock within this border crossing project in this specific area, but when the new lock is finished and ready for use, a safe and reliable passage will be available for the coming decades.

Currently (2019) the construction of the works is in full progress. The new lock should be operational by the beginning of 2023.
Travaux de réhabilitation d'écluses et maintien de la navigation: impacts sur la stabilité des ouvrages

Introduction
Voies Navigables de France (VNF) s’est engagé depuis plusieurs années dans une politique de restauration visant à fiabiliser le réseau pour garantir les niveaux de service et envisager la modernisation complète de certains de ses ouvrages. La réhabilitation des écluses s’inscrit dans ce contexte de mise aux normes ainsi que dans la perspective d’une standardisation globale de ce type d’infrastructure. En parallèle, du fait du nombre croissant d’utilisateurs du réseau fluvial sur le territoire ainsi que des impacts sociaux-économiques importants pouvant résulter d’une fermeture de ce dernier, les périodes de chomages sont réglementées et leurs durées parfois insuffisantes pour réaliser l’ensemble des travaux de réhabilitation lors de la mise à sec de l’écluse.

1) Problématiques principales
BRL Ingénierie est régulièrement confronté à ces problématiques qui sont désormais inhérentes aux projets de rénovation d’ouvrages fluviaux. Les travaux de réhabilitation (reprise de l’étanchéité des joints interplots, du réseau de drainage, traitement de la pollution des terre-pleins,...) conduisent souvent à décaisser tout ou partie des remblais situés à l’arrière des bajoyers d’écluse. Il en découle des situations de projets et des conditions spécifiques d’exploitation qui n’avaient pas été imaginées et prises en compte lors des dimensionnements historiques de ces ouvrages : ouvrage mis à nu sans terrain en place à l’arrière, avec un sas plein ou vide.

Dans ces configurations, les bajoyers travaillent en sens inverse de celui pour lequel ils étaient initialement prévus et leur stabilité d’ensemble peut être engagée. Cet article vise à apporter un éclairage à l’appui de notre expérience sur le contexte normatif à utiliser et l’impact du dimensionnement sur les méthodes de travaux.

2) Retour d’expérience sur les projets de Gambsheim et Denain
Deux projets traités récemment par BRL Ingénierie sont emblématiques :

- Remise en état des écluses de Gambsheim situées sur le Rhin: le projet prévoit le changement du réseau de drainage en arrière des bajoyers de rives via la réalisation d’une paroi blindée nécessitant un terrassement partiel du remblai ; cet exemple permet de comparer deux approches normatives différentes ;

2.1) Cas de la stabilité externe : écluses de Gambsheim
Les deux écluses de Gambsheim présentent une longueur totale de 310.08m. Les sas offrent une largeur de 24m et les bajoyers ont une hauteur de 21.05m. Ces derniers sont composés d’un empilement de blocs béton.
L’un des objectifs des travaux est de remplacer les drains actuellement bouchés par de nouveaux drains posés à l’air libre dans une tranchée réalisée sous blindage. En phase conception, il a été envisagé une technique de terrassement et remplacement en lieu et place.

En phase des travaux, le bajoyer de rive sera “allégé” du fait du terrassement partiel à l’arrière de l’ouvrage combiné à une alternance de sas plein et de sas vide compte tenu du maintien de la navigation durant cette opération. L’étude a consisté à vérifier la stabilité de cet ouvrage dans ces conditions très spécifiques. Le dimensionnement en phase projet a été mené suivant les Eurocodes puis contre-vérifié avec les recommandations du Comité Français des Barrages et Réservoirs (CFBR).

**Vérifications Eurocodes**
En première approche, la stabilité externe est étudiée selon l’Eurocode 7 et plus précisément la norme NF P 94-281. La stabilité au soulèvement (renversement) des redans est vérifiée par la justification de l’excentrement qui revient à vérifier que:

\[ 1 - \frac{2 \times e}{B} \geq \frac{1}{15} \text{ à l’ELU} \]
\[ 1 - \frac{2 \times e}{B} \geq \frac{1}{2} \text{ à l’ELS} \]

Avec:
- \( e \): excentrement de la composante de charge verticale par rapport au milieu du redan étudié;
- \( B \): la largeur du redan.

**Vérifications CFBR**
Le CFBR recommande de mener les vérifications suivantes:
- Vérification de l’état limite d’extension des fissures:
La condition de non fissuration s’écrit :

\[ \sigma'_N(x) = \frac{f_{tk}}{\gamma_{mft}} \]

Où :

- \( f_{tk} \) est la valeur caractéristique de la résistance à la traction du matériau examiné (en pratique, la valeur dimensionnante est celle au droit des joints de reprises) ;
- \( \gamma_{mft} \) est le coefficient partiel venant affecter la valeur caractéristique de la résistance à la traction du matériau et dépendant de la combinaison d’action examinée.

Les conditions d’état-limite à examiner pour l’état-limite d’extension des fissures s’expriment à partir de la longueur d’ouverture de la fissure, obtenue par un calcul itératif dans lequel on considère que la pleine sous-pression s’introduit dans la partie fissurée de la section.

Pour le cas des redans, on retient qu’en combinaison rare, il est admis une ouverture maximum de 25% de la largeur totale du redan.

- **Vérification de l’état limite de résistance à l’effort tranchant:**

La condition d’état-limite de résistance à l’effort tranchant s’exprime de la façon suivante :

\[ \left[ \frac{C_k}{\gamma_{mc} \times L'} + \frac{N' \times (\tan \varphi)_k}{\gamma_{mtan\varphi}} \right] > \gamma_{d1} \times T \]

Où :

- \( C_k \) et \( (\tan \phi)_k \) sont les valeurs caractéristiques de la cohésion et de la tangente de l’angle de frottement interne du matériau (en pratique, la valeur dimensionnante est celle au droit des joints de reprises) ;
- \( L' \) est la longueur de la section non fissurée étudiée, telle que calculée sous la combinaison d’actions considérée ;
- \( N \) et \( T \) sont les composantes normale et tangentielle des actions agissant sur la section étudiée, issue de la combinaison d’actions considérée ;
- \( \gamma_{mc} \) et \( \gamma_{mtan} \) sont les coefficients partiels venant affecter les valeurs caractéristiques des résistances au cisaillement du matériau et dépendant de la combinaison d’actions considérée ;
- \( \gamma_{d1} \) est le coefficient de modèle de l’état-limite de résistance à l’effort tranchant, dépendant de la combinaison d’actions considérée.

- **Vérification de l’état limite de résistance à la compression:**

Il s’agit de vérifier la non-plastification du matériau en partie aval du redan. La condition d’état-limite de résistance à la compression s’écrit :

\[ \gamma_{d2} \times \sigma'_C < \frac{f_{ck}}{\gamma_{mfc}} \]

Où :

- \( f_{ck} \) est la valeur caractéristique de la résistance à la compression du matériau examiné ;
- \( \gamma_{mfc} \) est le coefficient partiel venant affecter la valeur caractéristique de la résistance à la compression du matériau et dépendant de la combinaison d’action examinée ;
- \( \gamma_{d2} \) est le coefficient de modèle de l’état-limite de résistance à la compression, dépendant de la combinaison d’actions considérée.

Il est à noter que les vérifications de stabilité externe suivant l’EC7 ont été menées manuellement en réalisant pour chaque redan un bilan des forces qui s’y appliquent. Pour les vérifications suivant le CFBR, une modélisation aux éléments finis sur le logiciel Autodesk Robot Structural Analysis© a permis de tirer les contraintes normales et tangentielles nécessaires aux calculs.
Résultats

Les résultats obtenus pour ces différentes vérifications sont présentés dans les tableaux ci-dessous : 

### Vérification excentrement ELU EUROCODES

<table>
<thead>
<tr>
<th>Élément</th>
<th>Valeur admissible</th>
<th>CAS 3 : Nappe à +127 NN + SAS VIDE</th>
<th>CAS 4 : Nappe à +127 NN + SAS PLEIN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/15</td>
<td>1-2e/B</td>
<td>γ</td>
<td>1-2e/B</td>
</tr>
<tr>
<td>1/15</td>
<td>1-2e/B</td>
<td>γ</td>
<td>1-2e/B</td>
</tr>
</tbody>
</table>

\[ \gamma = \frac{1 - \frac{2e}{B}}{15} \]

### Vérification excentrement ELS EUROCODES

<table>
<thead>
<tr>
<th>Élément</th>
<th>Valeur admissible</th>
<th>CAS 1 : Nappe à +126,15 NN + SAS VIDE</th>
<th>CAS 2 : Nappe à +126,15 NN + SAS PLEIN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>1-2e/B</td>
<td>γ</td>
<td>1-2e/B</td>
</tr>
<tr>
<td>1/2</td>
<td>1-2e/B</td>
<td>γ</td>
<td>1-2e/B</td>
</tr>
</tbody>
</table>

\[ \gamma = \frac{1 - \frac{2e}{B}}{2} \]

### Etat limite d'extension des fissures CFBR

<table>
<thead>
<tr>
<th>Élément</th>
<th>Valeur admissible</th>
<th>CAS 3 : Nappe à +127 NN + SAS VIDE</th>
<th>CAS 4 : Nappe à +127 NN + SAS PLEIN</th>
</tr>
</thead>
<tbody>
<tr>
<td>25%</td>
<td>% calculé</td>
<td>γ</td>
<td>% calculé</td>
</tr>
<tr>
<td>25%</td>
<td>% calculé</td>
<td>γ</td>
<td>% calculé</td>
</tr>
</tbody>
</table>

\[ \gamma = \frac{25\%}{\text{calculé}} \]

### Etat limite de résistance à l'effort tranchant CFBR

<table>
<thead>
<tr>
<th>Élément</th>
<th>CAS 3 : Nappe à +127 NN + SAS VIDE</th>
<th>CAS 4 : Nappe à +127 NN + SAS PLEIN</th>
</tr>
</thead>
<tbody>
<tr>
<td>4,83</td>
<td>Effort résistant γ</td>
<td>26,33</td>
</tr>
<tr>
<td>35,94</td>
<td>Effort résistant γ</td>
<td>60,85</td>
</tr>
<tr>
<td>170,35</td>
<td>Effort résistant γ</td>
<td>128,07</td>
</tr>
<tr>
<td>292,61</td>
<td>Effort résistant γ</td>
<td>146,97</td>
</tr>
<tr>
<td>466,11</td>
<td>Effort résistant γ</td>
<td>149,88</td>
</tr>
</tbody>
</table>

\[ \gamma = \frac{\text{Effort résistant}}{Y_{d1} \times T} \]

Figures 3: Modélisation aux éléments finis 3D du bajoyer de l’écluse de Gamshein (à gauche) et cartographie de moment de flexion du redan n°3 (en haut, en kN.m/m)
Dans le cadre des écluses de Gambsheim, et quel que soit le contexte normatif employé, il ressort qu’en présence de nappe fluviale, le cas {Terrassement partiel + sas vide} est toujours plus instable que le cas {Terrassement partiel + sas plein}. L’analyse croisée de ces différentes vérifications met également en évidence que la vérification d’excentrement à l’ELS de l’EC7 est la condition limite la plus rapidement atteinte.

2.2) Cas de la stabilité interne : écluse de Denain
Le site de Denain comprend deux écluses. Une première et ancienne écluse à petit gabarit située en rive gauche qui est désormais désaffectée et une deuxième écluse à grand gabarit en rive droite, plus récente. Les deux ouvrages sont séparés par un terre-plein central.

Le terre-plein central est actuellement constitué de scories gonflantes qui génèrent d’importants dégâts sur les ouvrages à proximité. L’un des objectifs des futurs travaux est de stopper ce phénomène de gonflement en substituant l’intégralité des matériaux du terre-plein central par un matériau inerte convenablement compacté.

Lors de la phase d’excavation des scories du terre-plein central, le bajoyer rive gauche de l’écluse à grand gabarit sera découvert dans sa grande majorité alors que l’exploitation du sas sera maintenue afin de permettre la poursuite de la navigation.

Il existe une phase durant laquelle les efforts appliqués coté sas dépassent ceux déployés côté terre-plein.

Lors des études, l’objectif premier de la vérification de stabilité interne est de s’assurer que les armatures existantes dans l’ouvrage sont suffisantes pour reprendre les efforts engendrés dans les différents scénarii étudiés.
- **Cas 1:** Terre-plein décaissé sans nappe (pompage) + sas plein
- **Cas 2:** Terre-plein décaissé avec nappe (défaut pompage) + sas vide

Pour l’écluse de Denain, une modélisation aux éléments finis suivant un profil 2D a permis de déterminer les contraintes de compression/traction qui transitent dans les différentes parties de l’ouvrage.

Dans la configuration {terre-plein décaissé sans nappe (pompage) + sas plein}, le pied intérieur du bajoyer et la partie supérieure du radier sont logiquement tendus.

![Figures 6: Modélisation aux éléments finis 2D des bajoyers de l’écluse de Denain - contraintes dans le béton](image)

L’analyse croisée des plans de ferraillage et des résultats de la modélisation montrent une insuffisance d’armatures en pied de bajoyer. Elles sont insuffisantes pour reprendre à elles seules les efforts de traction qui s’y développent. Toutefois, les efforts de traction résiduels sont compatibles avec les contraintes de traction admissibles dans le béton.

3) **Discussion et perspectives**

Il est essentiel en phase conception d’étudier tous les scénarios possibles et de s’assurer de la faisabilité technique du projet sous ses contraintes d’exploitation particulières. A la lumière des éléments présentés dans cet article, une comparaison directe entre la méthode Eurocodes et CFBR n’est pas évidente du fait que:

- Les deux méthodes nécessitent des combinaisons d’états limites différentes (bien que toutefois très similaires);
- Les résultats CFBR dépendent du choix des valeurs à retenir pour les coefficients partiels et des caractéristiques intrinsèques des interfaces entre blocs;
- Les conditions limites recherchées sont différentes et plus ou moins drastiques suivant la méthode retenue;
- La méthode CFBR, bien que très précise (calcul itératif, profil de sous pression plus réaliste,...), est très orientée ouvrage de type barrage et l’application aux écluses n’est pas clairement spécifiée.

Quoiqu’il en soit, les philosophies générales de calculs restent proches et ces deux méthodologies peuvent être croisées afin de s’approcher au plus juste des phénomènes physiques réels qui s’observent sur ces ouvrages anciens à réhabiliter.
Introduction

Depuis plus de 60 ans, le groupe BRL assure conjointement la maîtrise d’ouvrage, la maîtrise d’œuvre et l’exploitation de nombreux ouvrages hydrauliques dans le Languedoc-Roussillon. Ces ouvrages comprennent notamment des canaux, des adducteurs, des stations de pompage, des grands barrages de stockage, des ponts-canaux et des réservoirs d’eau au sol ou sur tour.

Les premiers besoins en BIM sont apparus chez le maître d’ouvrage pour développer une base de données patrimoniale.

Les filiales d’exploitation et d’ingénierie de BRL ont trouvé un intérêt majeur dans cette démarche : l’optimisation ; d’une part de la phase de conception des ouvrages et d’autre part de la phase d’exploitation et de maintenance, en couplant les maquettes numériques avec leur logiciels de GMAO (Gestion de Maintenance Assistée par Ordinateur).

Une démarche BIM a ainsi été mise en place au sein des équipes d’ingénierie dans le cadre de maîtrises d’œuvre réalisées pour le compte du groupe, notamment sur les projets de construction ou de modification de stations de pompage liées au développement du réseau régional hydraulique AquaDomitia.

Fort de ces expériences réussies, le développement du BIM a été poursuivi pour le compte d’autres Maîtres d’Ouvrages, sur des infrastructures hydrauliques de plus en plus diversifiées, pour aboutir à la réalisation de maquettes numériques sur la majeure partie des ouvrages de navigation fluviale étudiés : barrages, écluses, quais, ponts-canaux, etc...

Mots clés :
Infrastructure de navigation, BIM

Le BIM pour les infrastructures hydrauliques : Analyse d’une démarche regroupant Maîtrise d’ouvrage, exploitation et ingénierie
**Les apports du BIM sur les ouvrages de navigation**

Avec la réalisation depuis plusieurs années de maquettes numériques de différents types d’ouvrages sur le réseau à grand gabarit français, le BIM a ouvert successivement pour les concepteurs, maîtres d’ouvrage et exploitants, des perspectives très intéressantes.

**De nouvelles méthodes de travail pour le maître d’œuvre**

Les maquettes numériques ont un coût, mais aussi un intérêt indéniable pour le Maître d’œuvre dans l’optimisation de ses méthodes de travail :

- **Gestion des interfaces** entre bureaux d’études spécialisés dans différentes parties d’ouvrages et/ou architectes. Les maquettes numériques permettent aux différents partenaires de travailler sur un support unique permettant la mise en place d’une démarche qualité commune, malgré des méthodologies de travail souvent différentes.

- **Interopérabilité entre différents outils** utilisés en phase études : les calculs de dimensionnements structurels, mais également hydrauliques, des ouvrages font de plus en plus appel à des modélisations 3D, souvent consommant des temps. Ces phases de modélisations peuvent être optimisées avec une maquette numérique comportant des ouvrages paramétriques (ouvrages dont l’ensemble des cotes s’ajustent automatiquement en fonction de dimensions paramétrées sous forme de champs à renseigner). Ceci permet un gain de temps significatif lors d’itérations entre les phases de calculs de dimensionnements structurels ou hydrauliques et les phases de correction des modèles géométriques alimentant les logiciels de calcul.

Une maquette numérique paramétrique a notamment été utilisée pour le dimensionnement hydraulique 3D réalisé dans le cadre des études d’avant-projet de l’opération de reconstruction des écluses de Méricourt sur la Seine. Cette démarche a permis de tester de nombreuses configurations géométriques des organes d’alimentation/vidange des sas et de gestion, afin d’aboutir à un ouvrage totalement optimisé à la fois pour les efforts sur les bollards flottants mais aussi des temps de sasées.

**Figure 3 : Extrait de la maquette numérique BRLi des écluses de Méricourt – Maquette construite sous Nemetschek Allplan™ – Modélisations hydrauliques 3D réalisées sous Flow-3D©**

- **Optimisation des temps de production** : les maquettes numériques montrent un réel avantage dans la standardisation d’ouvrages répétitifs. Ainsi, elles permettent une « paramétrisation » des équipements, de différentes parties de génie civil qui, si elle est bien pensée, permet un gain de temps important lors de l’élaboration des livrables.

Une démarche BIM a été mise en place dans le cadre du Partenariat Public Privé de reconstruction de 29 barrages sur l’Aisne et la Meuse. La maquette numérique a été utilisée pour centraliser les productions des intervenants en charge de la conception d’ouvrages imbriqués (barrage, passe à poissons, microcentrale hydroélectrique, local de commande et équipements). Elle a permis l’optimisation des systèmes de bouchures, et des équipements associés, pour les décliner de manière standardisée sur l’ensemble des barrages du projet.
Une fiabilité améliorée des livrables pour les maîtres d’ouvrage

Au-delà de leur intérêt pour les maîtres d’œuvre, lors des études de conception, les maquettes numériques offrent également aux Maîtres d’Ouvrage un outil de visualisation, permettant une meilleure compréhension des problématiques rencontrées et des conceptions imaginées par les ingénieurs pour y répondre. C’est également une garantie de fiabilité des différents livrables produits par ces derniers. Cela est permis par la fiabilisation des coûts estimatifs, via la réduction des approximations et risques d’erreurs en lien avec la modélisation 3D de l’ensemble des ouvrages, mais également par l’élaboration détaillée des phasages (et donc des plannings) de réalisation.

Une maquette numérique a ainsi été utilisée dans le cadre des études du projet d’allongement de l’écluse de Quesnoy-sur-Deûle dans le Nord-Pas-de-Calais. Ce projet vise à allonger l’écluse existante tout en limitant au maximum la durée des périodes de chômage de l’ouvrage. Cela implique un phasage des travaux très précis et très complexe nécessitant un raisonnement en trois dimensions dans l’espace mais également dans le temps.

Figure 4 : Extrait de la maquette numérique BRLi du site de Saint Joseph - PPP de reconstruction de 29 barrages de l’Aisne et de la Meuse – Maquette construite sous Autodesk Revit™

Figure 5 : Extrait de la maquette numérique BRLi sous Nemetschek AllplanTM des travaux d’allongement de l’écluse de Quesnoy-sur-Deûle (Voies Navigables de France)

Outre le fait que la maquette numérique a permis de générer des métrés fiabilisés des ouvrages provisoires et définitifs à réaliser, elle a surtout permis d’identifier les contraintes de réalisation de chacune des phases du chantier ; le planning de réalisation a été généré de manière sécurisée, en s’articulant autour de trois périodes de chômage.
Un outil de gestion des ouvrages pour les exploitants

L’intérêt principal des maquettes numériques est surtout de permettre aux exploitants de disposer d’un outil de gestion de leurs ouvrages facile d’accès et intuitif. Cela vaut pour la consultation des plans d’archives ou pour la gestion et la maintenance des équipements à travers leur totale standardisation. Il peut s’agir d’une simple défense d’accostage à des équipements plus complexes comme des bouchures de barrage ou des portes d’écluses.

Une démarche BIM a été, par exemple, mise en application sur le réseau à grand gabarit du Nord-Pas-de-Calais pour l’étude de 55 zones de stationnement et la création d’un cahier de standardisation des équipements de ces ouvrages. Une maquette numérique a été créée afin de mettre à disposition du Maître d’Ouvrage un outil interactif intégrant l’ensemble des plans et notices technique d’une zone de stationnement standardisée accessibles par un simple clic sur les équipements en question.

Figure 6 : Extrait maquette numérique interactive des zones de stationnement du réseau à grand gabarit du Nord-Pas-de-Calais (Voies Navigables de France) – Maquette BRLi construite sous Autodesk Revit™

Des apports qui demandent un investissement de la part de tous les acteurs

La mise en place d’une démarche BIM dans le domaine des ouvrages de navigation demande un investissement important de la part de l’ensemble des acteurs du projet.

Ainsi, au-delà de l’investissement financier lié à l’acquisition des outils eux-mêmes (logiciels de modélisation, outils de visualisation et de gestion, développements spécifiques aux spécificités des ouvrages de navigation) les différents acteurs du projet doivent intégrer des formations des intervenants pour la mise en place du processus :

- Pour la maîtrise d’œuvre, en plus de la formation technique du personnel sur les nouveaux outils et méthodes de travail, les besoins d’interopérabilité nécessitent également la formation ou le recrutement de personnel dédié (coordonnateur ou Manager BIM) ou la mobilisation de cabinets d’expertise BIM,
- Concernant la maîtrise d’ouvrage, la démarche BIM nécessite une formation des prescripteurs ou la mobilisation de cabinets d’expertise BIM afin de permettre l’élaboration de cahier des charges précis définissant clairement les usages attendus des maquettes impactant directement les niveaux de détails de ces dernières et leur coût,
- Enfin, côté exploitation, la mise à disposition d’une maquette numérique nécessite non seulement la mise en place de formations des agents pour sa utilisation courante mais également pour sa mise à jour durant toute la vie de l’ouvrage. Au-delà de cela, un couplage des maquettes numériques avec les logiciels de GMAO (Gestion de Maintenance Assistée par Ordinateur) impose au préalable aux
exploitants de réaliser un travail indispensable de définition de la « granulométrie » de l’information qu’ils souhaitent gérer.

Conclusion

Si le BIM a trouvé sa place depuis de nombreuses années dans le bâtiment, puis plus récemment pour des ouvrages d’adduction et de traitement de l’eau (stations de traitement, station de pompage, etc.), il reste encore assez peu utilisé en France dans le domaine des infrastructures de navigation. Cela peut notamment s’expliquer par l’existence d’outils de modélisation historiquement attendus et développés pour la réalisation d’ouvrages du bâtiment (géométrie assez simple mais nombreux corps de métiers, d’équipements et de réseaux). Le développement d’outils appropriés aux ouvrages d’infrastructures, aux géométries complexes, sans interfaces entre corps de métiers, et avec peu d’équipements et peu de réseaux, a dû attendre.

Il faut aussi nuancer la conception et l’exploitation, l’usage du BIM pour cette dernière étant encore limité, y compris dans le bâtiment, malgré un intérêt croissant de la part des différents acteurs.

Les retours d’expérience des différentes maquettes réalisées par BRL ingénierie sur les infrastructures de navigation montrent que les outils de modélisation ont bénéficié ces dernières années d’efforts de la part des fournisseurs de logiciels. Ils offrent des perspectives grandissantes et des fonctionnalités de plus en plus adaptées aux ouvrages de navigation : dans les années à venir, les modes de conception et d’exploitation des projets devraient muter et se généraliser pour tous les acteurs du domaine.
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Ref. author:  
Cyril Guidoux – geophyConsult  
159 Allobroges, 73000 Chambéry, France  
cyril.guidoux@geophyconsult.com

Co-authors:  
Fanny Dubié – EDF CIH  
Savoie Technolac 73373 Le Bourget du Lac cedex, France  
fanny.dubie@geophyconsult.com

Jean-Robert Courivaud – EDF CIH  
Savoie Technolac 73373 Le Bourget du Lac cedex, France  
jean-robert.courivaud@geophyconsult.com

Vincent Speisser – Voies Navigables de France  
4, quai de Paris, 67000 Strasbourg, France  
vincen.t.speisser@vnf.fr

Keywords:  
Safety, monitoring, waterways dyke, embankment, internal erosion, leakage, flood, real time

Title:  
OPERATION OF A MONITORING SYSTEM INCLUDING FIBER OPTICS ON THE RHINE LEVEES BETWEEN STRASBOURG AND IFFEZHEIM

Context:  
In 2010, Voies Navigables de France (VNF) chose to deploy on the dikes on the left bank of the Rhine, north of Strasbourg, an innovative surveillance system using optical fiber. Project management for this project was entrusted in 2010 to the SAFEGE / EDF / geophyConsult consortium.

The objectives of the monitoring system of the dikes Gambsheim (13 km) and Iffezeheim (25 km) are to monitor their hydraulic and mechanical behavior in the long-term, but also during hazards like flood or seism. In order to achieve these objectives, the monitoring system includes piezometric and Rhine level measurements as well as temperature measurements distributed by optical fibers. The purpose of distributed temperature measurements is to detect the occurrence of significant leaks in the embankment and thus to send early warnings the the owner. For several decades indeed, temperature measurement has proven to be one of the most relevant measures to detect the presence of leaks in earthworks. The advent of optical fiber technology, which allows for distributed temperature measurements, complements the monitoring of the hydraulic behavior ensured by conventional instrumentation ([1], [3], [4] and [8]).

Surveillance system:  
The location of the optical fiber within the structure has been determined by applying the EDF-geophyConsult methodology for the design of such systems, which consists of simulating the temperature measurements that optical fiber would generate at location where it is desired to install it, taking into account a typical anomaly within the structure, and then checking with the raw fiber temperature data processing models that have been developed by EDF, if the leak is well detected by at least one of the models. The application of this methodology has led to the conclusion that an optical fiber installed in a shallow trench (1 m to 1.5 m deep) dug along the downstream dike toe would be able to detect leaks into the core or at the embankment-foundation interface for a flow rate ≥ 1 l × min⁻¹ × m⁻¹ (Figure 1).
Data are acquired at a frequency of one measurement per hour. All measurements are uploaded daily by radio (fixed station) or GSM (mobile stations) to the control room at Gamsbeim. A backup of this data is performed in real time on a sftp server feeding the geophyConsult database, on which the processing methods are then applied. This monitoring system was implemented in 2013 and 2014 [5], then it was reinforced (at the end of 2016 and then in 2017), with the equipment of 16 existing piezometers and the construction of 18 new piezometers.

The rest of this article focuses on the optical fiber surveillance system.

**Data analysis:**

The main analysis method is based on the singular value decomposition of the matrix of the temperature data recorded by optical fiber, corrected for each measurement day ([7], [8]). From this decomposition, a signal subspace (consisting of the first singular value, containing most of the measurement information) and a residual subspace are defined, and the detection parameter is defined as the Euclidean norm of each vector of the subspace residual to the right of each measuring point. This parameter is therefore calculated for each measurement day and every meter along the optical fiber on the homogeneous measurement sections (Figure 2).

The detection criterion applied to detection parameter consists is a fixed value, measured during in situ tests of artificial leaks by injection of water with 1 liter / minute / m flow rate, performed along a similar dyke equipped with an optical fiber surveillance system similar to that of the dikes of Gamsbeim and Ifezheim. If this value is exceeded by the detection parameter, this may mean that a leak of at least 1 liter / minute / m has occurred at the measurement site, provided that the optical fiber measurement at this location has not been disturbed by the presence of a crossing structure (pipeline, etc.), by a variation of its laying method (directional drilling, depth variation, etc.), or by a significant change in the fiber environment (distance to dike / drainage channel, etc.). The method applies to short data sets (1 day minimum), provided that there are 12 measurements / day minimum, as with longer time series (weeks, months, years).

**Real-time monitoring:**

The trigger point for real-time monitoring was initially set by VNF at a flow rate in the Rhine above 2 100 m3 / s at the Kehl-Kronenhof gauging station [3]. This flow was durably exceeded:

- from 02/02/2016 to 21:00 to 02/02/2016 at 05:00. During this period, the dykes were monitored in real time, despite the low duration of the episode and the low reported flow rates, to test the human organization associated with real-time monitoring;
- from 13/05/2016 at 14:15 to 17/05/2016 at 16:00;
- from 04/06/2016 at 17:00 to 01/07/2016 at 17:00.

During these periods, the owner provided the on-call team at least once a day with the flow records at Gamsbeim and the levels of the Rhine at the ends of the two controlled reaches. A restitution of the data processing to the owner was carried out every eight to twelve hours approximately, during all the duration of the real time monitoring. The holding of this organization, since August 2015, represented 104 hours of work in working days and daytime hours, and 55 hours of night and weekend work, for a linear of dikes surveyed 38 km.

**Gamsbeim reach:** The detection parameter is lower than the detection criterion over the entire length of the linear, including the portions that have undergone a significant increase in hydraulic head during floods, with the exception of the portions of the dykes comprising crossing structures near which passes the optical fiber. The presence of these structures induces temperature variations unrelated to the hydraulic loading of the dike.

**Ifezheim reach:** as in Gamsbeim, the detection parameter is lower than the detection criterion over the entire linear, including the portions that have undergone a significant increase in hydraulic head, with the exception of the portions of dykes comprising crossing structures. During the May and June 2016 floods, a number of peaks appeared along the Mole and risberme on the left bank of the River Ill, as well as on the Offendorf harbor bypass dike (Figure 3).
A visual inspection was carried out on June 21st during the flood, and these zones were identified by the VNF agents as follows:

• mole of Gambbsheim and risberme on the left bank of the Ill: l'Ill being in flood, the risberme where the optical fiber is installed was submerged of 20 to 50 cm of water. The peaks of the detection parameter thus correspond to the infiltrations of the water of the Ill in the risberme following its submersion;

• Offendorf harbor bypass dike: the Rhine level was high enough to flood the usually dry upstream dike toe by 20 to 40 cm of water. The downstream toe, inspected 72 hours after the flood peak, remained dry. The peaks of the detection parameter thus correspond to local infiltration of the Rhine water into the structure and / or to the rise of the local aquifer, without any consequence for the structure.

**Conclusion:**

The application of distributed fiber optic measurement technology for the detection of thermal or mechanical anomalies, associated with internal erosion or instability type disturbances within the structures, constitutes a complementary tool for monitoring and maintenance of dikes. This tool, successfully used in the Gambbsheim and Iffezheim reaches since 2015, allowed to monitor in real time the state of the dikes during the 2016 floods. The data delivered by the system (fiber optics and telemetered piezometers) allowed to draw up a very satisfactory inventory of the behavior of the structures during exceptional hydraulic loadings. All events detected by the monitoring system were confirmed by field elements. An almost real-time flood monitoring organization has been implemented and proved relevant to the objectives set by the owner. The analysis of the temperature measurements for the localization of the leaks has been implemented both in situation of long term periodic monitoring and in flood monitoring situation in near real time. This method proved to be well suited to these different surveillance situations. The assessment of its qualification for flood monitoring needs to be consolidated by means of greater experience feedback.

**Figure 1: Overview of the reaches (left) and optical fibre installed at the downstream dike toe (right)**
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**Ref. author:**  
Jean-François Seignol – IFSTTAR  
Boulevard Isaac Newton, Cité Descartes, 77447 Marne-la-Vallée Cedex, France  
jean-francois.seignol@ifsttar.fr

**Co-authors:**  
François Sataro – Cerema  
25, avenue François Mitterrand, 69500 Bron, France  
francois.sataro@cerema.fr

Didier Germain – Cerema  
25, avenue François Mitterrand, 69500 Bron, France  
didier.germain@cerema.fr

Laurent Labourie – Cerema  
44ter, rue Jean Bart, 59000 Lille, France  
laurent.labourie@cerema.fr

Pierre-Yves Scordia – VNF  
37, rue du Plat, 59034 Lille Cedex, France  
pierre-yves.scordia@vfn.fr

**Keywords:**  
Lock, prestressed concrete, delayed ettringite formation, durability, finite-element method

**Title:**  
**DEF-pathology in concrete supporting a lock-gate: use of numerical modelling to re-assess structural behaviour**

**Abstract:**  
Delayed ettringite formation (DEF) is a concrete pathology which may occur in structural parts submitted to high temperature during their casting, such as some massive elements cast in-situ. DEF consequences can lead to structural disorders (expansions, cracking, deterioration of mechanical properties), which modify and jeopardize structures serviceability and sometimes, safety.  
This communication deals with a concrete lock for which a large prestressed beam supporting the lower gate is affected by DEF for several years.  
First, the structure and its pathology are described. The global strategy of the study, mixing together documents analysis, laboratory tests on concrete samples and use of finite-element models is then presented. These numerical models have multiple purposes: assessing the early-age thermal history of the beam in order to confirm the origin of DEF; assessing the structural behaviour and its modification due to DEF, based on chemo-mechanical coupling models; mechanical interaction between the beam and the other parts of the lock. This modelling is performed to assess structural state at present time, as well as to predict its evolution in the future.  
Finally, conclusions drawn from the study are used to help structure owner in their planning of his maintenance policy.

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1 - **The structure and its pathology**  
Ettringite is one of the products formed during cement hydratation. In sound concrete, ettringite crystallizes at early-age, while concrete is still plastic, without any consequences. In some cases, if temperature raises during cement hydration, ettringite cannot be formed immediately. Hence, its formation occurs later, in
hardened concrete and crystallization pressure induces material damage and global swelling. This pathology is then called Delayed Ettringite Formation (DEF) (Divet, 2001). One of the common causes responsible for excessive temperature at early-age is the heat produced by cement hydratation, especially in massive structural elements.

The Fontinette lock (built in 1965, this lock with a 144.60 m x 12 m chamber is on the Neufossé channel, part of the Dunkerque-Escault-Lille link) is equipped with two miter gates (upper and middle) and a lower guillotine gate (see Ill. 2). Due to the exceptional 13 m rise, the lower gate is topped by a “mask” girder. The mask girder consists in two parts (see Ill. 1) between which the gate slides: the upstream part protect the gate from boat shocks and the downstream one support water pressure when the chamber is full and bears a technical gallery. For resisting these high loads, the downstream girder is prestressed in two directions (22 cables “STUP” consisting of 12 8-mm tendons in the vertical direction, 122 cables of the same type in the transversal one).

![Illustration 1: Cross-section of the mask-girders and the lower gate (opened), downstream on the left.](image1)

Illustration 2: Downstream part of the lock with the lower gate and its mask-girders.

In 1997, evidences of internal swelling reactions have been discovered in the downstream girder. Alkali-silica reaction (ASR) has been observed on samples, and DEF is also suspected (investigations still on-going). Possible long-term consequences of these pathologies could be a decreased of concrete resistance due to internal cracking, compression stresses in the girder due to prescribed swelling and excessive traction in the tendons. This is why a numerical model has been developed to: first, confirm the possibility of DEF in conjunction with the truthful ASR; second, to estimate the mechanical consequences of the pathology onto the lock structural safety.

2 - Global strategy of the study

The numerical modeling of DEF-effect in the downstream girder is a three-step strategy, connected to the global management of the lock which includes documentation research, monitoring, sampling of concrete and laboratory analysis. We decided to focus on the sole DEF because there exists no numerical model taking into account simultaneous occurrence of both ASR and DEF and because DEF is well-known to be more dangerous (quicker and with larger swelling) than ASR.

The first step consists in re-assessing the early-age thermal history occurring during the casting of the girder, based on documentation from the enterprise in charge of construction and from meteorological statistics. This thermal history is then transformed into a swelling potential, which requires fitting some unknown parameters relative to the development of DEF in this particular concrete. Since, at this step, these parameters are unknown, typical values from the literatures are used. Once the model for concrete swelling is done, it is coupled in a mechanical model (forming a chemo-mechanical model) and allows us to assess the mechanical consequences of DEF in the girder. By comparing these consequences to values from the monitoring and to experiment results obtained on concrete samples from the structures, the chemo-
mechanical, the chemo-mechanical model is then fitted. This last step being still on-going, we will only describe the two first steps in the following sections.

3 - Numerical model for DEF-affected concrete
The first step is to re-assess the early-age thermal history. This step is performed by solving the heat transfer equation in each lift of the girder (6 lifts have been successively cast) and introducing cement hydration heat as a source (Tailhan et al. 2010). At each point of the girder, the thermal history is transformed into swelling potential using model developed by (Baghdadi et al. 2008).

The swelling potential is then converted into an incremental prescribed strain representing swelling of the concrete and its effects are computed taking into account boundary conditions, loads, as well as the presence of reinforcements and prestressing cables. The influence of stress-state is also considered, both as inducing anisotropy in the swelling and as a classical creep mechanism. Evolution of DEF is also represented by a degradation of mechanical properties through a damage equation. This chemo-mechanical procedure is described in (Seignon et al. 2012).

4 - Application to the beam supporting the lower gate
For numerical modeling, the downstream girder of the lower gate is represented by about 97,000 finite elements and divided into six parts, for each successive lift. The early-age thermal history is then computed and, as shown on Ill. 3 for the 3rd lift, we confirm that temperature exceeded 65 °C in the central part of the structure (this value is usually considered as the threshold temperature above which DEF can occur).

In the second step, we introduce classical values for the swelling parameters, and the structural effects of DEF are observed on the same mesh (into which bar elements have been introduced to reproduce groups of prestressing cables). The evolution of the girder versus time can be observed, for instance 50 years after construction, as seen on Illustration 4: the central and upper part of the girder is subjected to high displacements (about 10 mm) whereas the bottom and side of the element are blocked by the surrounding parts of the lock, leading to internal compressive stresses. This model will soon be fitted to be more consistent with observed measurements as well with recent observations on concrete samples.

5 - Conclusion
In this (on-going) work, we try to put the stress on two important points concerning the management of waterway infrastructures affected by delayed-etrtringite formation:
- the complex and serious consequences of DEF onto mechanical behaviour of concrete elements, hence onto structural safety of critical infrastructures;
- the possibility offered by numerical modelling to test assumptions and to estimate consequences (now and in the future) of DEF, and its conjunction with informations obtained from documentation research, in-situ monitoring and laboratory analysis.

6 - References
Illustration 3: Zone where temperature at early-age reaches 65°C in lift #3 (numerical simulation).

Illustration 4: Deformation of the downstream girder after 50 years of DEF, and map of global displacement (numerical simulation).
A French experience of Structural Health Monitoring of scour affecting river infrastructures

Because of their ability to connect territories and populations, any bridge failure can have a significant financial, human and social impact. As an example, the weakening of the Mayou Bridge following a flood in 2009 permanently disrupted traffic in the centre of Bayonne even though an inspection carried out two years earlier had not identified any defects/problems. In fact, flow/structure interaction can lead to scouring process of river beds or banks that represents a significant contributing factor in the deterioration of structures and which can lead to their collapse during major floods. Moreover, in a rail or road transportation infrastructure system, bridge maintenance represents a high operating cost. For example, the French rail network has approximately 8,000 river crossings, of which 1,700 are potentially exposed to scouring processes. The stakes are even higher in the case of the national road network. Cofiroute, manager of approximately 10% of the French motorway network has, out of the 1,200 structures it manages, 20 viaducts under a specific monitoring programme to identify scouring problems. Across the US, 60% of reported bridge failures are due to scour of which about 20,000 bridges are identified as “scour critical” and the same number as “scour susceptible”. These examples clearly show the importance and fragility of transport structures in the face of the environmental risks of floods and the importance of having observation and
warning systems in place to identify disorders and their consequences in order to move towards an integrated forecasting system and integrated risk management procedures.

Financed by the French National Research Agency (ANR), the SSHEAR project, "Soils, Structures and Hydraulics: Expertise and Applied Research", comprises six partners, whose complementarity offers a major asset to the project. Specialists in soilsand in fluid mechanics; geotechnical and hydraulic engineers together with sedimentologists, physicists, infrastructure management companies and a technological research institute contribute to the project (Chevalier et al, 2018).

The project proposes an intensive research effort on scour erosion based on a multi-scale and multi-scientific approach using:

- physical processes of flow and erosion in the vicinity of structures (e.g. bridges, dams, embankments, quay walls),
- laboratory experiments featuring multi-scale observation,
- an innovative numerical approach built around a two-phase model,
- observations and field recordings of actual structures subjected to hydro-sedimentary forcing imposed due to environmental or anthropogenic actions.

The first step has been to identify pilot sites representative of scour vulnerability. This selection has been undertaken using the databases of SSHEAR project Partners (SNCF, Cofiroute), and validated by some field campaigns. Several criteria were defined to select the sites that would be monitored:

- bridge vulnerability to scour,
- bridge geometry (type of bridge and its representativeness, type of piers, ...),
- river conditions (flow conditions, hydro-morphology, nature of sediment...),
- geology and geotechnical parameters (foundations type, bedrock or reinforcement presence...),
- site accessibility,
- staff and materials safety...

The selection has then been finalized and restricted to 3 sites:

- Site 1 corresponds to the scouring of a backfill in a meander. Such situations are common in valley bottom structures, while all infrastructures are concentrated near the river, as is the case in the mountainous regions of France. This is associated with a semi mountainous climate. It has been chosen to make precise hydraulic and bathymetric surveys once a year.
- Site 2 (A71 road bridge over la Loire at Orléans) is representative of large structures crossing a river in France in an oceanic climate. This structure is founded on 6 piers, 4 of which are located in the main channel. It has been chosen to implement continuous monitoring devices.
- Site 3 (Aurence railway crossing near Limoges) is a characteristic arch bridge built on the banks of watercourses. It is also in the oceanic zone. It is representative of the structures of secondary road and rail networks. It has been chosen to implement continuous monitoring devices.

According to Breusers et al. (1977), scour can be described by equation (1)

\[ dse = f(\rho, \mu, U, Y, g, d, Uc, D) \]  

where \( g \) is the acceleration of gravity; \( Y \) is the depth of water; \( U \) is the average upstream velocity; \( D \) is the diameter of the pile, \( \rho \) is the density; \( \mu \) is the dynamic viscosity of the fluid, \( Uc \) is the critical value of the velocity associated with the movement of the bed particles, \( d \) is the diameter of the particles. For each measure the following items have been taken into account:

- the min-max measuring range,
- the desired characteristics (uncertainties, resolution,...)
- intrusiveness
- innovative techniques / more robust techniques
- homogeneity of a fleet of equipment (on 1 site if redundancy or between sites)
- equipment acquisition costs
- operating and maintenance costs
- data recovery costs
- on-site deployment constraints
- installation constraints
- power supply (mains, battery, if batteries what autonomy…)
- post processing (validation criteria, operation,...)

A bibliographic review of experimental technics have been undertaken, a table of specifications has then been sent to potential suppliers. For the two sites selected for continuous monitoring, the proposed set-ups have been developed and implemented as part of the project. On the A71 site, radar level sensor have been fixed on the bridge to measure the water level relative to a fixed point. In addition a raft made of two boards (figure 1a) is used to support a sonar instrument, used to scan the bathymetry, and a velocity profiler used to measure the velocity field. The sensors are connected to a data logger that is also sending the data to a supervisor. On the Aurence site, we have implemented an aerial water level gauge, an ultrasonic velocity profiler that is mounted on a floating device in order to follow the free surface variations and a fixed drone network camera that allows us to watch the sensors periodically in order to identify any emergency situation. Due to the rural location of the site, both energy and data transmission are made by GSM (Figure 1b).

On the Aurence site, the instrumentation has been working for 5 months and we already have feedback information. The first on is the usefulness of the camera. It has allowed the team to visualize a tree that had damaged the raft so we have been able to act quickly and repair the damaged apparatus. The second is the reactivity to the flow rate to rain events, that is also corroborating the data acquired by the French data base “Banque Hydro” (http://www.hydro.cafrance.fr/) about 1.5 km upstream. The third reason is the importance of measuring the velocity profiles. Concerning this aspect, the first results show a hysteresis effect in relation to flowrate variations. More time is needed to analyse the hydraulic behavior in order to automatize the data processing for management purposes.

On the A71 site, the datalogger has been damaged and it has not yet been possible to recover data. However, those two sites give a useful feedback for continuous monitoring of scour affected structures over rivers.

References
Title: 
Solution innovante pour la surveillance de porte d'écluses

La CNR, exploitant et mainteneur de 18 aménagements sur le Rhône, s’intéresse de très près aux organes de vantellerie qui équipent ses principaux ouvrages : usines hydroélectriques, barrage mobiles et écluses de navigation. Le parc exploité comporte plus de 300 structures de vantellerie de grandes dimensions : 10 m<largeur<45m ; 6m< Hauteur< 21m ; 50t< poids < 200 t. Ce sont les vannes mobiles de barrages (segment ou wagon), les vannes de sécurité aval des groupes hydroélectriques, les vannes déchargeurs et évacuateurs de corps flottants, les portes d’écluses (busquées, latérales coulissantes ou baissantes) et les batardeaux (barrages, usines et sas d’écluses). Le nombre élevé de ces équipements ainsi que leur importance stratégique en matière de sécurité et sûreté des installations, en font des actifs prioritaires parmi les quelques 5 000 actifs électromécaniques gérés. Parmi ces organes de vantellerie, les portes d’écluses sont, avec les vannes de sécurité aval des groupes de production, les équipements à la fois les plus sollicités et ceux ayant subi le plus grand nombre de cycles de fonctionnement. Le parc CNR comporte 52 portes d’écluses, dont la répartition des cycles de fonctionnement ( 1cycle = fermeture porte, remplissage sas, vidange sas, ouverture porte) est la suivante :

Figure 1 : Nombre de cycle de fonctionnement pour chaque porte d’écluse

La moitié du parc des portes d’écluses est âgé de plus de 50 ans, sachant que la limite de calcul donnée par les normes de référence est basée sur une durée d’utilisation maximale de 70 ans (cf DIN 19704-1 2014-11 § 7-6 « Fatigue »). La CNR est donc confrontée aujourd’hui à la problématique d’atteinte de « vie mature » d’un nombre significatif de portes d’écluses. De plus, environ la moitié des écluses, ne sont pas munies de portes
redondées : en cas d’avarie majeure sur l’une d’entre elles, une interruption longue de la navigation fluviale serait la première conséquence immédiate, ce qui n’est pas acceptable.

La politique actuelle de maintenance et de surveillance de ces organes est liée aux contraintes de la voie navigable : intervalle élevé entre 2 arrêts pour maintenance (une année) et durée réduite de ces arrêts (8 jours). La maintenance est adaptée à ces contraintes du cahier des charges imposé par le Concédant. Elle est essentiellement basée sur la surveillance régulière lors des rondes, ainsi que sur l’analyse des résultats des expertises, porte à sec, effectuées pendant les arrêts de navigation : contrôles non destructifs de la structure, contrôle de l’état de la protection anticorrosion, contrôle d’usure des mécanismes et dispositifs d’étanchéités pour l’essentiel. Cette approche est basée sur de la maintenance corrective, et donc non prévisionnelle. Cette maintenance corrective allant jusqu’à de la rénovation « lourde » s’avère parfois inadaptée à l’état réel des structures : le scénario du remplacement, a déjà été retenu pour plusieurs portes et va s’accélérer dans les prochaines années.

Afin d’avoir une vision plus objective sur l’état de santé de ces organes de vantellerie, la CNR a souhaité développer un système de maintenance prédictive destiné à l’aider à la priorisation et programmation du remplacement, dans les toutes proches années, des portes les plus vulnérables. Cette approche prédictive de la maintenance, par la mesure de la flèche du bordé de la porte « on line », est l’objectif du projet de recherche et développement présenté ici.

Le premier démonstrateur de ce projet est la porte d’écluse aval de l’écluse de Grand gabarit d’Avignon. La porte a été instrumentée en 2018 avec un système NEURON de la société MORPHOSENSE composé de 17 capteurs (accéléromètres 3-axes de pointe) synchronisés pour le monitoring de la déformation de la structure. Le système NEURON permet la détermination des déformations statiques 1D et 2D de la porte à partir de la mesure d’inclinaisons. Il permet également le suivi du composante dynamique tel que les fréquences propres de l’ouvrage. Dans le cadre de cette etude, la deformation statique constitue le critère principal d’intérêt. Le maillage réalisé par l’instrumentation permet de reconstruire, via un algorithme développé par le CEA-Leti, une surface de déformation autour du maximum de déformation. La Figure 2 montre une représentation 2D de la déformation de la porte où l’on peut correctement identifier le maximum de déformation de la porte.

![Image de l'écluse d'Avignon](image1.png)  
**Figure 2.** (Gauche : de haut en bas) Usine d’Avignon, Porte d’écluse Aval, Chariot mobile de la porte où est localisée l’amoire d’acquisition du système NEURON, Capteur fixé à la porte via des aimants puissants. (Droite) Déformée surfacique reconstruite lorsque la porte est en charge. En haut, l’évolution du maximum de déformation et de la hauteur de chute pendant tout le cycle de charge/décharge. À droite, la hauteur de chute et le profil de déformée sur la ligne centrale pour l’instant choisi.

Ces mesures font l’objet de corrélation avec des modèles de simulation FEM en cours de réalisation par le LaMCoS (INSA Lyon).

Pour donner quelques détails de l’algorithme, la première étape consiste à déterminer les inclinaisons à partir de chaque accéléromètre puis, par une méthode d’approximation, à monter à la surface de déformation (voir
[1] pour plus de détails) et notamment à la déflexion de la ligne centrale de la porte. De cette déflexion sont extraits le maximum de déflexion et sa localisation. Le maximum de déflexion se calcule à partir de la corde reliant les 2 extrémités de la courbe (cf. Figure 2). Le calcul de la déflexion à partir de la corde permet de s’affranchir de l’inclinaison globale de la porte et décrit alors réellement la déformation subie. La Figure 3 présente l’évolution de la déformation maximum de la porte au cours d’une journée de monitoring. Ce tableau de bord permet le suivi en temps réel de la structure où l’on observe bien les cycles de charge et de décharge de la porte.

La bibliographie sur le suivi des portes d’écluses révèle que le maximum de déflexion seul n’est pas un indicateur suffisant pour traduire l’état de santé de la porte puisque ce dernier dépend des conditions environnementales : température, niveau d’eau amont et aval. Dans l’article [2], il est explicité que la corrélation entre le maximum de déflexion et la hauteur de chute pendant une phase de chargement (ou de déchargement) est un indicateur plus robuste. Cet indicateur doit rester stable dans le temps, une éventuelle dérive peut constituer un signe de fatigue ou de dommage de la porte. La Figure 4 montre un exemple d’évolution du coefficient de corrélation sur 2 jours de monitoring.

Cette étude, initiée par CNR en collaboration avec MORPHOSENSE et le CEA-Leti, permet une compréhension du comportement statique des portes d’écluse à travers un monitoring dédié. En plus d’un suivi des portes, un recalage avec des modélisations numériques en cours d’étude permettra d’anticiper les opérations de maintenance futures ou les eventuels changements de porte.

Références

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Ref. author:  
Céline Savary – Service public de Wallonie – Hydraulic Research Laboratory  
Rue de l’Abattoir 164, 6200 Châtelet, Belgium  
celine.savary@spw.wallonie.be

Co-authors:  
Didier Bousmar – Service public de Wallonie – Hydraulic Research Laboratory  
Rue de l’Abattoir 164, 6200 Châtelet, Belgium  
didier.bousmar@spw.wallonie.be

Catherine Swartenbroekx – Service public de Wallonie – Hydraulic Research Laboratory  
Rue de l’Abattoir 164, 6200 Châtelet, Belgium  
catherine.swartenbroekx@spw.wallonie.be

Gil Zorzaz – Service public de Wallonie – Hydraulic Research Laboratory  
Rue de l’Abattoir 164, 6200 Châtelet, Belgium  
gil.zorzaz@spw.wallonie.be

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measurement, navigation lock, lock leveling, mooring forces

Title:  
Lanaye Lock  
Perturbation in the lock chamber induced by asymmetrical filling

Introduction  
In the frame of the 18th priority project of the Trans-European Transport Network (TEN-T) concerning the corridor Rhine-Main-Danube, a new navigation lock was inaugurated in 2015 in Lanaye at the borderline between Belgium and The Netherlands. The new lock is 225 m long and 25 m wide and allows vessels up to 9000 tons (ECMT Class VIb) to cross a difference in height of 13.7 m between the Albert canal and the river Meuse.  
The lock is leveled through two longitudinal culverts with side ports, the filling and emptying discharge is regulated with butterfly valves of 3.5 m diameter. It was designed to address concurrent objectives: (1) minimizing the transit duration, to increase the lock efficiency; (2) reduce the perturbation in the lock chamber, to limit the mooring forces and the risk of incident during leveling; and (3) limit the leveling waves amplitude in the downstream reach. One drawback of the chosen system is the asymmetrical flow in the lock chamber during the leveling when one of the culverts (valve) is out of order.

Commissioning tests – measurement of the free-surface slope  
Before opening the lock of Lanaye to navigation, different measurements were realized on site to check the conformity of the lock with the hydraulic design criteria. To estimate the perturbation in the lock chamber during the leveling, the water surface slope was calculated on the basis of synchronized pressure measurements at different locations of lock chamber. A perfect synchronization in time is needed to deduce the free-surface slope from the individual water level measurements. A lack of synchronization of 1s could lead to an error of 0.1% on the slope which is quite important regarding the small value expected (<0.5%).  
During the design, it was considered that the longitudinal slope of the free-surface in the lock chamber during the lockage has to remain below 0.5% for all kind of vessels. This criterion is conservative compared to
admissible forces as listed by PIANC (2015): longer is the ship, smaller is the admissible force, the smallest admissible force is 0.85%o for a Va CEMT vessel. This conservative criterion accounts for considering the water slope instead of the actual force. However, in situ measurements on other locks revealed larger water-surface slopes, which do not imply problem for vessels during the leveling. No value is proposed regarding the transversal slope.

The site measurements showed that during asymmetrical leveling, the mean slope of the water surface in the lock chamber always remains below the conservative value of 0.5 %o (Figure 1), but that the transversal slope for an asymmetrical filling reaches 6 %o (Figure 2). The measured transversal slope during asymmetrical emptying remains below the conservative value of 0.5 %o which reveals that it is not an issue.

In order to avoid safety problems, operational guidelines were proposed in case of asymmetrical filling: only one boat is allowed on the width of the lock chamber and it has to be moored on the side of the operational culvert. Then, due to transversal currents, the vessel is held against the wall. After few months of navigation, during the maintenance of one of the upstream valves (one culvert was out of order), these recommendations were not followed. An incident occurred due to the asymmetrical filling of the lock and to the important perturbation in the lock chamber.

**Definition of alternative opening schedules on basis of a simple numerical approach**

The opening schedule of the valves, recommended from the design, was a linear opening of the valve with a constant rate of 10°/min (law A in Figure 3). Regarding the slope measurements in asymmetrical filling, the valve had to open slower. Nevertheless, the impact on the duration of the leveling due to the modification of the valve schedule had to remain moderate, the filling of the lock could not last more than 35 minutes in asymmetric mode.

It was not possible to test a large amount of valve opening schedules on site. A simplified numerical model ALFREDO (Christiaens et al. 2014) was used, as a first approach, to define three opening schedules that could be interesting to test on site. This numerical model consists of two interacting parts, one calculates the transient flows in the culverts (1D), and the other calculates the free surface flow (1D) in the lock chamber. The head loss coefficients were calibrated on basis of the measurements realized during the commissioning of the lock (comparison between measured and modeled hydrograms).

Different valve opening schedules were tested with the numerical model. After analysis of the numerical results, only three opening schedules were selected for site measurements (Figure 3). These were chosen on the basis of the video record of the incident and the previous measurements realized on site. (1) The measurements revealed that the transversal slope increases sharply when the opening angle of the valve exceeds 50° (after 5 minutes) (Figure 2), which corresponds approximately to a discharge of 40 m³/s and a water level of 6.5 m from the bottom. It was deduced that it is not necessary to change the opening rate before reaching 40°-50°. (2) On the video, the transversal flow seemed reduced 12 min after the opening of the valve, which, regarding the hydrogram, corresponds to a discharge of 80 m³/s and a water level of 12 m from the bottom. These two points corresponding to convenient conditions are connected by a red line in Figure 4 which represents the evolution of the discharge regarding the water level in the lock chamber during an asymmetrical filling for the selected valve opening schedules.
An alternative solution consisting in a bi-linear opening of the valve had to be aborted: to be efficient, the opening rate for angle higher than 50° should drop to 5°/min. This would require substantial modifications on the hydraulic control circuits of the jacks which activate the valve on site. Therefore, to decrease the discharge, despite the fact that it is usually not recommended, the opening schedules with one or two steps were proposed.

**Site measurements and final optimization**

For the proposed opening schedules, the transversal water surface slope was measured during asymmetrical filling. Moreover, to have a better idea of the transversal current, the vertical velocity profile was measured during the leveling with an ADCP at the middle of the lock chamber (in front of a port and between two ports). In the best case, the transversal slope only decreases from 6 to 4 % (Figure 5).

To further reduce this extreme value, the opening schedule of the valve was optimized a second time using the simplified numerical model. The resulting transversal slope should be a little smaller (not measured on site), but it was not possible to find an opening schedule which allows decreasing it drastically with an acceptable duration for the leveling. Even if the safety conditions are improved, operational recommendations have always to be applied.

**References**


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Ref. author:
Wu Peng - CCCC Water Transportation Consultants Co. Ltd
No.28 Guozijian Street, Dongcheng District, Beijing, China
email: wupeng@pdiwt.com.cn

Co-authors:
Cao Fengshuai – CCCC Water Transportation Consultants Co. Ltd
No.28 Guozijian Street, Dongcheng District, Beijing, China
email: caofengshuai@pdiwt.com.cn
Xuan Guoxiang – Nanjing Hydraulic Research Institute
No.223 Guangzhou Road, Nanjing City, Jiangsu Province, China
email: xuan@nhri.cn

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Study on the Filling and Emptying System of Wanan Second Lock

Wanan Hydropower Station is located in the middle reaches of Ganjiang River, which is a tributary of the Yangtze River. Its main function is to generate electricity, and it has comprehensive utilization benefits such as flood control, shipping, irrigation and aquaculture. The total storage capacity of the reservoirs is 2.2 billion m³. It was completed and put into operation in January 1996.

The existing lock of Wanan Hydropower Station is arranged on the right bank with a single-step arrangement. The effective size of the lock chamber is 175m×14m×2.5m (length×width×water depth on sill), and it can pass a fleet of two 500-ton barges. The filling and emptying system is the two sections of long culvert under the lock bottom with top branch hole covered by plate for energy dissipation. The partial side water intake mode is adopted for the inlet of filling and emptying system. The left inlet draw water from outside approach channel and the right one from inside approach channel. The partial side water discharge mode is also adopted for outlet of filling and emptying system.

Considering the development of vessels and the improvement of navigation conditions of the waterway in Ganjiang River, it is proposed to build the Second Lane of Wanan Lock, and it will pass the 1000t class vessels. The effective size of the new lock chamber is 180m×23m×4.5m, and the design lift is 32.5m.

The lift height of Wanan Second Lock is at the forefront of the world in single-step locks, and the volume of water conveyance is 164.45 thousand m³ for one lockage process. How to ensure the safety and efficiency of filling and emptying is the key technical problem to be solved in the construction of Wanan Second Lock.

1. Selection and layout of filling and emptying system

The following formula is always used for the preliminary type selection of filling and emptying system, which is taken from Chinese "Design Code for Filling and Emptying System of Locks (JTJ306-2001)"

\[ m = \frac{T}{\sqrt{H}} \]

In the formula: \( T \) is the required time of filling the chamber (min), \( H \) is the design lift (m). This is an empirical formula. Normally when \( m \) is bigger a simple system could be used and when it is smaller a
complicated one should be used. The following factors shall be taken into consideration for the selection of a filling and emptying system: lift height, time for filling or emptying the lock, lock chamber sizes, investments and permissible maximal forces on the vessel. In the above formula only two factors are included. So the other factors should be considered in practice.

According to the design requirements of Wanan Lock, it can be obtained that:

\[
m = \frac{10-12}{\sqrt{32.5}} = 1.75-2.10
\]

The lift of Wanan second lock is 32.5m, and the gravity lock wall could be adopted for the structure of lock chamber according to geological conditions. Therefore, a very complicated type of distributed filling and emptying system can be chosen. It is composed of the main culvert in the lock wall, the flow division in the middle of lock chamber, four longitudinal branch culverts in two sections at the bottom of lock, side ports and open ditch for energy dissipation (Fig.1).

![Fig.1 Layout of filling and emptying system of Wanan Second Lock](image)

Then the sectional area of culvert in the valve section could be calculated according to the following formula which is also taken from the Chinese code.

\[
\omega = \frac{2C(\sqrt{H+d} - \sqrt{d})}{\mu T \sqrt{2g \left[ 1-(1-\alpha)k_v \right]}}
\]

In the above formula, \( \omega \) is the Sectional area of culvert in the valve section (m\(^2\)); \( C \) is the calculation water area of lock chamber (m\(^2\)). \( H \) is the design lift (m); \( d \) is the inertia head when the valve totally opened; \( \mu \) is the discharge coefficient of filling and emptying system when the valve entirely opened; \( T \) is the required time for filling the chamber (s); \( \alpha \) is the coefficient to be selected according to the table in the code and \( k_v \) is a ratio of valve open time divided by filling time; \( g \) is the acceleration of gravity (m/s\(^2\)).

For the Wanan Second Lock: \( C=220 \times 23=5060 \text{m}^2 \), \( H=32.5 \text{m} \), \( T=10-12 \text{min}=600-720 \text{s} \), \( d=0.5 \text{m} \), \( \mu=0.8 \), \( \alpha=0.43 \), \( k_v=0.6 \). It can be obtained that:

\[
\omega = \frac{2 \times 5060 \times (\sqrt{32.5+0.5} - \sqrt{0.5})}{0.8 \times (600 \times 720) \times \sqrt{2 \times 9.81 \left[ 1-(1-0.43) \times 0.5 \right]}} \approx 36.4 \quad 30.3 \text{m}^2
\]

So the value of the sectional area of culvert in the valve section is decided to be 36.0 m\(^2\).

When the sectional area of culvert in the valve section is determined, it is very important to determine the elevation and the shape of the culvert section with valve for the high lift lock. The lift of Wanan Second Lock is very high and the downstream water level varies nearly 10m, therefore, the shallow burial of the valve and natural ventilation at the top of culvert could not be used to solve the cavitation problem. It can only be considered that the valve should be arranged at the lower elevation, relying on the greater pressure behind the valve to reduce the occurrence of cavitation behind the valve. Through preliminary analysis, the burial depth of culvert of the valve section would be 13m, and the elevation of the culvert top should be 54.50 m.

The filling valve section is connected with the upstream inlet through a vertical turning, while the culvert behind the filling valve is directly connected with the main culvert in the lock wall. The emptying valve section is also connected to the main culvert in the lock wall through a vertical turning, and the culvert behind emptying valve is directly connected to the side emptying culvert of the lower lock head (Fig.2). The
height of the culvert is adjusted in the vertical turning and sudden expanding culvert section.

![Fig.2 Layout of water filling and emptying culvert](image)

The intake manifold is arranged on the front wall of the upper lock head. The total area of the intake manifold is determined with the criteria that the flow velocity should be less than 2.5 m/s. So total area is 2-6×4.0m×6.0m(width×height)=288 m². The submerged depth of the inlet should be more than 0.4 times the design lift according to the Chinese code.

The wall culvert is connected with filling and emptying valve sections and lock chamber diversion ports respectively. The flow coefficient and cross-section velocity of the filling and emptying system should be taken into account in the determination of cross-section area. The cross-section velocity is generally controlled within 10~12 m/s. According to the estimated maximum flow of water filling, the culvert section needs to be more than 48.7~40.6 m². So the lock wall culvert section should be 2-4.0 m×6.0 m=48.0 m², the area ratio of culvert section to the water filling valve is 1.33, and correspondingly the velocity is about 10 m/s in culvert.

The water depth of the downstream approach channel is only 4.5 m at the lowest navigation water level, so the downstream lock head emptying system adopts side discharge arrangement. The discharge culverts on both sides behind the valve culvert are connected with the side culvert through horizontal turning, and the elevation is adjusted. After turning, the section of the discharge culvert is enlarged to make it consistent with the section of the side discharge culvert, and the side culvert passes through the bottom of the first lock approach channel then reach the main flow of the river.

The diversion structure in the middle of the lock chamber refers to the layout of the vertical diversion structure of the Three Gorges Lock. The horizontal diaphragm of the vertical diversion is arranged in the lock chamber. In order to further control the cross-section velocity, the cross-section of the diversion is appropriately enlarged, and 4-6.0×2.5m (width×height) = 60 m² is adopted.

The section of the culvert in lock chamber is 4-4.0×5.0 (width×height) = 80.0 m². The ports on each culvert along the flow direction are divided into three groups with the sizes (width×height) of 0.48 m×1.0 m (6 holes), 0.44 m×1.0 m (6 holes) and 0.40 m×1.0 m (6 holes), respectively. The total area is 63.36 m². The ports are arranged with equal distance, the center distance of adjacent ports is 4.0m, and the total length of the outlet section is 2×18×4.0m=144.0m, accounting for 80% of the effective length of the lock chamber.

The open ditch for energy dissipation is set outside the port to adjust and dissipate the flow into the chamber. The width of the open ditch is 2.5 m and the depth is 4.0 m.

2. Hydraulic characteristics of the filling and emptying system

Because of the height lift the cavitation of valve culvert section is one of the key technical problems to be solved in the design of filling and emptying system. Through the test of physical model of filling and emptying system with scale of 1:30 and physical model for filling valve with scale of 1:20, two kinds of layout of filling and emptying system are analyzed and compared, namely, flat bottom culvert (Fig.3) and sudden expansion culvert (Fig.4) at valve section.

For the flat bottom culvert, the flow coefficients of filling and emptying are 0.838 and 0.734 respectively. To meet the requirement of safe mooring condition stipulated in the Chinese code, the valve opening time is decided to be 8 min when filling the lock chamber, and the maximum flow rate is 457 m³/s, and the filling time is 11.44 min. The valve opening time is 7 min when emptying the chamber, and the maximum flow rate is 433 m³/s, and the emptying time is 11.45 min.
Fig. 3 Diagrammatic sketch of the valve section culvert and flow pattern for the flat bottom culvert

For the sudden expansion culvert, the flow coefficients of filling and emptying water are 0.780 and 0.692 respectively. With the same safe mooring condition, the valve opening time is 8 min when filling the lock chamber, and the maximum flow rate is 403 m³/s, and the filling time is 11.12 min. The valve opening time is 7 min when emptying the lock chamber, and the maximum flow rate is 402 m³/s, and the emptying time is 11.23 min.

Fig. 4 Diagrammatic sketch of the valve section culvert and flow pattern for the sudden expansion culvert

3. Hydraulic model test of valve and selection of filling and emptying system

The flow pattern behind the culvert valve in the flat bottom section can be divided into three areas, a main flow area, a main rotational flow area formed by the shrinkage and expansion of the flow at the bottom edge behind the valve and a secondary rotational flow area at the upper boundary of the main flow. The area of the culvert behind the valve does not change in the horizontal direction, so the water flow is not separated in the horizontal direction.

The flow pattern behind the culvert valve in the sudden expansion section can be divided into 4 areas, a main flow area, a main rotation area formed by the shrinkage and expansion of the flow at the bottom edge after the valve, a secondary rotation area at the upper boundary of the main flow and a secondary rotation area caused by the falling jet. Because the area of the corridor behind the valve does not change in the plane, the flow is not separated in the plane.

The flat bottom culvert section is relatively simple and easy to construct. The sudden expansion culvert has shorter filling time and a lower maximum flow velocity. The flow pattern at the upstream inlet and approach channel also improved as the flow velocity in the valve section and main culvert decreased. In the sudden expansion culvert, the vertical space behind the valve is increased, and the main rotation area behind the valve is fully whirlied. Compared with flat bottom culvert, although the top pressure of culvert decreases when the valve opening $n=0.1-0.3$, the natural ventilation of lintel is smoother, and the bottom cavitation can be sufficiently inhibited by the natural ventilation of lintel.

When the valve opening is equal or greater than to 0.4, the main flow can diffuse rapidly along the way,
which reduces the main flow speed and increases the pressure obviously.

4. Conclusion

Both two layout schemes of filling and emptying system (flat bottom type and sudden expansion type in culvert behind the valve) can meet the requirements of design and the code. The sudden expansion type has shorter water filling time and lower maximum flow rate.

In the sudden expansion culvert, the vertical space behind the valve is increased in the top and at the bottom of culvert, and the main rotation area behind the valve is fully whirled. Combining with natural ventilation measures of lintel, the problem of valve cavitation of Wanan second lock can be solved.

The distribution of ports with different areas and open ditches outside the ports has achieved the anticipated results in improving the mooring conditions of vessels in the lock chamber. The flow in the chamber is relatively stable under different operation conditions, and the vessel has no drift during the rising process, and the flow at diversion structure is balanced.

Reference


Retour d’expériences sur le fonctionnement des écluses avec blocs de remplissage dissociés et porte intermédiaire en période d’étiage

1 - Contexte
La Compagnie Nationale du Rhône (CNR) a reçu la concession du fleuve Rhône par l’Etat pour son aménagement et sa valorisation selon trois missions solidaires : production d’électricité, navigation et irrigation des terres agricoles. En aval de Lyon, CNR a aménagé 330 km de voie navigable à grand gabarit (195 m de long et 12 m de large), depuis le confluent du Rhône et de la Saône jusqu’aux écluses de Barcarin et de Port-Saint-Louis ouvrant sur la Méditerranée. Cette voie comporte 13 biefs d’une longueur moyenne de 25 km et 14 écluses à grand gabarit pour franchir les centrales hydroélectriques. Le dénivelé entre Lyon et la mer est de 162 m et la chute maximale des écluses varie entre 6,70 m et 23 m.
Pour les deux écluses de plus fortes chutes - 17 m pour l’écluse de Châteauneuf-du-Rhône et 23 m pour l’écluse de Bollène (les autres écluses disposant de chutes inférieures à 12.5 m) - le sas a été conçu avec une porte intermédiaire pour permettre un fonctionnement en demi-sas et une économie d’eau de 50% lors du passage de bateaux de faibles dimensions. Le système hydraulique de remplissage et de vidange de ces écluses est donc constitué de deux demi-blocs indépendants.

Système d’alimentation de l’écluse de Châteauneuf – Plan des circuits

Lorsque la porte intermédiaire n’est pas utilisée, il est important que les ouvertures des vannes de remplissage des deux demi-blocs soient synchrones afin d’obtenir une répartition longitudinale équilibrée de l’alimentation du sas. En évitant la création de pentes de lignes d’eau trop importantes dans le sas, la rupture éventuelle d’amarres est ainsi prévenue. Une attention particulière doit être également portée lorsqu’une vanne est en maintenance car dans ce cas, seul un demi-sas est alimenté directement, augmentant ainsi les efforts sur les amarres des bateaux. Ces phénomènes sont augmentés en période d’étiage du Rhône pour au

2 - Valeurs maximales admissibles des contraintes longitudinales
Les valeurs des contraintes admissibles en France sont celles établies par la Direction des Ports Maritimes et des Voies Navigables qui propose pour la résultante longitudinale des efforts hydrodynamiques les valeurs maximales suivantes (notice SCT n°76-1 de mai 1976) :
- 1/450 du poids d’eau déplacé par un automoteur (jusqu’à 400 tonnes) ;
- 1/600 du poids d’eau déplacé pour les petits convois (jusqu’à 5 800 tonnes) ;
- 1/1000 du poids d’eau déplacé pour les grands convois (poids d’eau déplacé supérieur à 5 800 tonnes).

Pour un convoi poussé composé d’un pousseur et de deux barges, cette classification présente l’inconvénient d’exiger pour un enfoncement de 3 m des efforts inférieurs à ceux admis pour 2,50 m. En effet, un convoi d’enfoncement de 2,50 m correspond à un déplacement total de 4 950 T et un convoi d’enfoncement 3 m à 5 850 T. Les efforts maximaux admis seraient donc respectivement pour 2.50 m et 3 m de 8.25 T (Tonne Force) et 5.85 TF. Afin de supprimer cette discontinuité, la valeur maximale de 1/600 du poids d’eau déplacé est pris en compte jusqu’à 6000 T.

Ainsi, la valeur maximale de la résultante longitudinale des efforts hydrodynamiques pour un convoi constitué d’un pousseur et de 2 barges est de 9.75 TF (5 850/600).

3 - Pente longitudinale de la ligne d’eau maximale admissible
Les pentes de lignes d’eau génèrent des efforts sur les amarres des bateaux. Les différentes études réalisées sur modèles physiques montrent que l’effort longitudinal exercé sur le bateau est expliqué à 90% par la pente longitudinale de la ligne d’eau dans le sas :

\[ F = \frac{1}{90\%} P \times i = 1.1 \times P \times i, \] où F est l’effort, P le poids du bateau et i la pente dans le sas de l’écluse.

Les 10% restants sont dus aux autres composantes des efforts exercés sur le bateau, notamment les efforts d’inertie (ou de masse ajoutée), les efforts de trainée, les efforts de frottements (ou de viscosité) ainsi que des efforts liés aux fluctuations turbulentes dans le sas.

En considérant l’effort maximal admissible F=P/600, on obtient la valeur maximale de la pente :

\[ i_{\text{max}} = 90\%. \frac{F}{P} = 90\%. \frac{1}{600} = 1.5\% \] (1.5 pour mille)

Cette pente est la pente maximale avec un bateau dans le sas.

Pour des raisons de sécurité, les essais site sont réalisés sans bateau dans le sas. Ainsi il est nécessaire de déterminer la valeur limite des pentes de la ligne d’eau dans cette configuration.

Lorsqu’il n’y a pas de bateau dans le sas, les pentes de la ligne d’eau sont plus faibles. Cette influence a été clairement identifiée sur le modèle physique des écluses de Panama réalisé par CNR où on a relevé une augmentation de l’ordre de 40% des pentes lorsque le bateau est présent dans le sas. Cette augmentation dépend de nombreux paramètres tels que :
- Forme du bateau (coefficient de blocage du bateau) ;
- Caractéristiques géométriques du sas ;
- Débits injectés dans le sas.
Pour déterminer les efforts sur les amarres du bateau, il serait nécessaire de réaliser un modèle physique couplé avec un modèle mathématique de représentation des amarres comme cela a été réalisé pour les écluses de Panama.
Par défaut il a été pris en compte un coefficient d’augmentation de 50% pour les essais aux écluses de Châteauneuf et Bollène.

**Ainsi la valeur de la pente de la ligne d’eau maximale admissible sans bateau dans le sas est estimée à 1 %**.

4 - Campagnes de mesures sur site
Afin de déterminer les pentes de lignes d’eau dans les sas, ceux-ci ont été équipés de sondes piézométriques (doublées) en amont et en aval. Les deux vannes de remplissage et les deux vannes de vidange ont été équipées de capteurs de positions pour permettre l’analyse des vitesses de manœuvre et les écarts éventuels de positionnements entre les vannes (désynchronisation). Une station d’acquisition a permis d’enregistrer l’ensemble des mesures. Pour ne pas être géné par le passage des bateaux (les essais doivent être faits sans bateau dans le sas), les essais ont été réalisés de nuit. Afin de tester la sensibilité des résultats de mesures en fonction des différents paramètres (gradient d’ouverture, niveau initial dans le sas, chute à l’écluse), il a été nécessaire de réaliser 3 campagnes de mesures pour chaque écluse.

5 - Résultats des mesures réalisées à l’écluse de Châteauneuf
Les essais ont été réalisés en phase de remplissage du sas avec les deux vannes de remplissage mais également avec une seule vanne de remplissage (pour simuler les cas de maintenance d’une des deux vannes). Quelques essais ont été réalisés en phase de vidange pour valider le fait que pour ce type d’opération les pentes de lignes d’eau dans le sas sont très faibles (la dissipation de l’énergie ne se faisant plus dans le sas, mais à l’aval de l’écluse).

Utilisation de 2 vannes de remplissage :
Des tests préalables ont montré qu’il existe une désynchronisation variant de quelques secondes jusqu’à une trentaine de secondes de l’ouverture des vannes. Avec un décalage de 30 secondes, la valeur maximale de la ligne d’eau est de l’ordre de 1,80 %, donc environ 2 fois supérieure à la valeur admissible. Cet effet lié à la désynchronisation des vannes reste important même si on adopte une ouverture de vannes très faible en début de phase de remplissage (50 cm durant 3 minutes – mode dit « stabilisé » - voir graphe ci-dessous). En conclusion, il est essentiel de supprimer tout décalage dans l’ouverture des vannes de remplissage. Lorsque les ouvertures des deux vannes sont synchrones (gradient d’ouverture constant de 1m/mm), les pentes de la ligne d’eau longitudinale dans le sas sont alors largement admissibles (inférieures à 20% de la valeur maximale).
Ecluse de Châteauneuf – Essai avec désynchronisation en début d’ouverture des vannes de remplissage et mode «stabilisé»

Utilisation d’une vanne de remplissage :
Les essais ont montré la nécessité de limiter le gradient d’ouverture de la vanne à 0.5m/mn. La pente obtenue est de l’ordre de 75% de la valeur maximale admissible. A noter que le mode « stabilisé » est à proscrire car la pente de la ligne d’eau est alors supérieure à la pente admissible et que la durée de remplissage est plus importante (+3.5 mn).

6 - Résultats des mesures réalisées à l’écluse de Bollène
Avec l’utilisation des deux vannes de remplissage, le gradient adopté est de 0.6m/mn et permet de limiter les pentes à 50% de la valeur maximale admissible.
Avec une seule vanne, le gradient adopté est de 0.3m/mn, et permet de limiter les pentes de la ligne d’eau à 70 % de la valeur maximale admissible. Quelques tests ont été réalisés en augmentant progressivement la vitesse d’ouverture de la vanne, ce qui permet de diminuer la durée de remplissage du sas tout en conservant des pentes de lignes d’eau admissibles.

7 - Actions mises en œuvre et retour d’expérience
Pour l’écluse de Châteauneuf-du-Rhone, la commande hydraulique des servomoteurs actionnant les vannes de remplissage a été modernisée afin de permettre une sélection et une maîtrise de leur gradient d’ouverture ainsi que leur parfaite synchronisation. Pour l’écluse de Bollène, la modernisation du système est à l’étude.
Ce retour d’expérience montre que les écluses avec porte intermédiaire présentent un avantage évident en termes d’économies d’eau lorsque les sasements en demi-sas peuvent être anticipés au regard du trafic, mais que le choix d’une telle conception doit être mis en regard des enjeux liés aux coûts de construction et à la sûreté d’exploitation.

8 - Bibliographie
R Merlin - Notice SCT n° 76-1 Mai 1976 – Ecluses standards (185 m x 12 m) de chute moyenne / Etude de l’alimentation par contournement des têtes – Service Technique Central des ports maritimes et voies navigables.
Roumie Pierre – Compagnie Nationale du Rhône
Hydraulic Structure Behavioural Analysis Centre (CACHO)
4 rue de Chalon sur Saône 69007 Lyon - France
p.roumieuf@cnr.tm.fr
Devillers Samuel - Compagnie Nationale du Rhône
Operational Coordination and Safety Department (CES)
2 rue André Bonin 69004 Lyon - France
s.devillers@cnr.tm.fr

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Lock, stress on moorings, intermediate gate, on-site tests

Feedback on the operation of locks with separate filling blocks and an intermediate gate in low-flow periods

1 - Context
The Compagnie Nationale du Rhône (CNR) was awarded the concession for the River Rhône by the State to develop and upgrade it in line with three interdependent missions: electricity production, navigation and irrigation of agricultural land. Downstream of Lyon, CNR has developed 330 km of wide-gauge navigable channel (195 m long and 12 m wide), from the confluence of the Rhône and the Saône down to the Barcarin and Port-Saint-Louis locks opening into the Mediterranean. This waterway contains 13 reaches with an average length of 25 km and 14 wide-gauge locks to pass the hydroelectricity power stations. The level difference between Lyon and the sea is 162 m and the maximum fall of the locks ranges from 6.70 m to 23 m.

For the two locks with the largest falls – 17 m for the Châteauneuf-du-Rhone lock and 23 m for the Bollène lock (the other locks having falls of less than 12.5 m) – the lock chamber was designed with an intermediate gate to allow for semi-lock chamber operation and a 50% water saving when small boats pass through. The hydraulic filling and emptying system for these locks thus comprises two independent semi-blocks.

![Lock filling and emptying system](image)

Where the intermediate gate is not used, it is important for the openings of the two half-block filling valves to be synchronous to obtain a balanced longitudinal distribution of the lock chamber feed. By avoiding the creation of excessively large water surface gradients in the lock chamber, the possible rupture of moorings is thus prevented. Particular attention must also be paid where a valve is undergoing maintenance as in this case, only a half-lock chamber is filled directly, thereby increasing the stresses on boat moorings. These phenomena are increased in low-flow periods on the Rhône for at least two reasons. On the one hand, the
heads, and thus the lock filling flowrates, are larger than in normal conditions. And on the other, the initial levels in the lock chambers are low which greatly reduces the dissipation of energy from the “mattress” of water at the start of the filling phase.

Incidents associated with mooring ruptures occurred at the Châteauneuf-du-Rhône lock in 2009, and the Bolléne lock in 2018. They initially prompted preventive measures with regard to operating conditions: reduction in filling valve opening gradients, indication to crew of the particular operating configuration, observation of a minimum safety distance between boats in the lock chamber etc. Then site tests were undertaken to determine the precise causes of these malfunctions.

2 – Maximum admissible values of longitudinal forces
The forces admissible in France are those set by the Maritime Port and Navigable Waterways Department which proposes for the longitudinal resultant of hydrodynamic stresses, the following maximum values (SCT notice no. 76-1 of May 1976):

- 1/450 of weight of water displaced by a powered craft (up to 400 tonnes);
- 1/600 of weight of water displaced for small convoys (up to 5,800 tonnes);
- 1/1000 of weight of water displaced for large convoys (weight of water displaced above 5,800 tonnes).

For a pusher convoy comprising a pushing tug and two barges, this classification presents the disadvantage of requiring lower stresses for a 3 m draught than those permitted for 2.50 m. This is because a convoy with a 2.50 m draught corresponds to a total displacement of 4,950 T and a convoy with a 3 m draught to 5,850 T. The maximum permitted stresses would thus be for 2.50 m and 3 m, 8.25 TF (Tonne Force) and 5.85 TF respectively. To eliminate this discontinuity, the maximum value of 1/600 of weight of water displaced is used up to 6000 T.

Thus, the maximum value of the longitudinal resultant of the hydrodynamic stresses for a convoy comprising a pushing tug and 2 barges is 9.75 TF (5,850/600).

3 – Longitudinal gradient of the maximum admissible water surface
The water surface gradients generate stresses on boat moorings. The various studies carried out on physical models show that the longitudinal stress exerted on the boat is 90% explained by the longitudinal gradient of the water surface in the lock chamber:

\[ F = \frac{1}{90\%} P \times i = 1.1 \times P \times i, \]

where F is the stress, P the weight of the boat and i the gradient in the lock chamber.

The remaining 10% is due to other components of the stresses exerted on the boat, in particular inertia (or added mass) stresses, drag stresses, friction (or viscosity) stresses and stresses associated with turbulence fluctuations in the lock chamber.

Where the maximum admissible stress F= P/600, the following maximum gradient value is obtained:

\[ i_{\text{max}} = 90\%, \quad \frac{F}{P} = 90\%, \quad \frac{1}{600} = 1.5\% \text{ (1.5 per thousand)} \]

This gradient is the maximum gradient with a boat in the lock chamber.

For safety reasons, on-site tests are carried out without a boat in the lock chamber. Thus, it is necessary to determine the limit value of the water surface gradients in this configuration.

Where there is no boat in the lock chamber, the water surface gradients are smaller. This influence was clearly identified on the physical model of the Panama locks made by CNR where an increase was observed in the order of 40% for gradients when the boat was present in the lock chamber. This increase depends on many parameters such as:

- The shape of the boat (blocking coefficient of the boat);
- Geometric characteristics of the lock chamber;
- Flows injected in the lock chamber.
To determine the stresses on boat moorings, it would be necessary to make a physical model together with a mathematical model representing moorings as was done for the Panama locks. By default, a 50% coefficient of increase was taken into account for the tests on the Châteauneuf and Bollène locks.

**Thus, the value of the maximum water surface gradient without a boat in the lock chamber is estimated at 1 %.**

4 – **On-site measurement campaigns**

In order to establish the water surface gradients in the lock chambers, they were fitted with (double) piezometric sensors upstream and downstream. The two filling valves and the two emptying valves were fitted with position detectors to allow the analysis of manoeuvring speeds and any positioning differences between the valves (desynchronisation). An acquisition station allowed all the measurements to be recorded. So as not to be hampered by boats passing (tests must be conducted without any boat in the lock chamber), the tests were carried out at night. To test the sensitivity of the measurement results based on different parameters (opening gradient, initial level in the lock chamber, fall at the lock), 3 measurement campaigns had to be carried out for each lock.

5 – **Results of measurements taken at the Châteauneuf lock**

The tests were carried out in the lock chamber filling phase with two filling valves and also with just one filling valve (to simulate cases where one of the valves is undergoing maintenance). Some tests were carried out in the emptying phase to validate the fact that for this type of operation, the water surface gradients in the lock chamber are very low (as the energy is no longer dissipated into the lock chamber, but downstream of the lock).

**Use of 2 filling valves:**

Preliminary tests showed that there is a desynchronisation ranging from a few seconds to about thirty seconds from the opening of the valves. With a 30-second time lag, the maximum water surface value is in the order of 1.80 %, thus around twice the admissible value. This effect associated with the desynchronisation of the valves remains considerable even if a very small valve opening is adopted at the start of the filling phase (50 cm for 3 minutes – a so-called “stabilised” mode – see graph below). In conclusion, it is essential to eliminate any discrepancy in the opening of the filling valves. Where the opening of the two valves is synchronous (constant opening gradient of 1 m/min.), the longitudinal water surface gradients in the lock chamber are then generally admissible (less than 20% of the maximum value).
Châteauneuf lock – Test with desynchronisation at the start of the opening of the filling valves in “stabilised” mode

Use of one filling valve:
Tests have shown the need to limit the valve opening gradient to 0.5 m / min. The gradient obtained is in the order of 75% of the maximum admissible value. It should be noted that “stabilised” mode is not to be permitted as the water surface gradient is then greater than the admissible gradient and the filling time is longer (+ 3.5 min.).

6 – Results of measurements taken at the Bollène lock
Where two filling valves are used, the gradient adopted is 0.6 m / min. and allows gradients to be restricted to 50% of the maximum admissible value.
Where just one valve is used, the gradient adopted is 0.3 m / min. and allows the water surface gradients to be restricted to 70% of the maximum admissible value. Some tests were carried out by gradually increasing the valve opening speed, which allows the lock chamber filling time to be reduced while maintaining admissible water surface gradients.

7 – Action taken and feedback
For the Châteauneuf-du-Rhone lock, the hydraulic control of the servo-motors actuating the filling valves has been modernised in order to allow the selection and control of their opening gradient and their perfect synchronisation. For the Bollène lock, modernisation of the system is currently being studied. This feedback of experience shows that locks with an intermediate gate have a clear advantage in terms of water savings where lockages with semi-lock chambers can be anticipated in line with traffic, but that the choice of this design must be made with consideration given to the issues of building cost and operating safety.

8 - Bibliography
R Merlin - Notice SCT no 76-1 Mai 1976 – Ecluses standards (185 m x 12 m) de chute moyenne / Etude de l’alimentation par contournement des têtes – Service Technique Central des ports maritimes et voies navigables.
Feasibility study of Itaipu Binacional Locks

Introduction
The Itaipu dam and hydropower plant were built between 1975 and 1983 and are operated by Itaipu Binacional. It provides around 15% of the energy consumed in Brazil and 90% of the energy consumed in Paraguay (2018). During the normal operational conditions of the Itaipu dam (up to a discharge of 15,178 m³/s), the water of the Itaipu reservoir is discharged through the turbines only. When upstream discharges exceed the capacity of the 20 turbines, additional water is discharged through a spillway allowing to control the water level of the Itaipu reservoir. This section of the Paraná River was never used as a waterway, due to the existing natural barriers. The projected bypass for the Itaipu dam will offer the possibility for navigation between the lower and upper reaches of the Paraná River. This bypass would allow to transit the 120 m water level difference between the upstream and downstream side of the dam.

In 2017, Witteveen+Bos and CNR were requested by Itaipu Binacional to prepare an up-to-date feasibility study for the construction of a bypass along the Itaipu dam. An important first step of the design of the shipping bypass, which will consist of a navigation channel and four locks, was to determine if the hydrodynamic conditions in the curve of the Paraná River south of the bypass allow for safe navigation of the design convoy into and out of the downstream lock. Indeed, current velocities in this section of the Paraná River can reach up to 3-4 m/s. Therefore, downtime of the bypass system due to too high velocities in the river south of the bypass was considered as a serious risk at the start of the feasibility study. To enter the bypass system, ships need to navigate at a relative close distance (~1km) of the Itaipu turbines and the spillway (Figure 1). Fast-time nautical manoeuvring simulations were performed to determine the navigability of the design vessel (convoy with pusher) over 4 km downstream to the future by-pass entrance. This required detailed information on the current field downstream of the Itaipu dam for various discharges. Obtaining reliable current fields in this part of the Paraná River was challenging because of the turbulent flow, complex geometry and large variations in water level and discharge. This paper describes how the required current fields were obtained and validated.

Methodology
Representative current fields were derived using the numerical model Telemac-3D together with a physical scale model (scale: 1:100) of the Paraná River and Itaipu Dam. The output of the numerical model are spatial
current fields that are used in the manoeuvring studies. The physical scale model and on-site measurements were used together for the calibration of the numerical model.

Hydrological, topographic and bathymetric data, among others, were made available by Itaipu through the CIH, an applied hydroinformatics research center of Itaipu. Daily recordings of water levels at various locations and of discharge through the Itaipu dam (subdivided over the turbines and the spillway, for the period 1983 - 2015), were used to determine:

- The variation and the probability of exceedance of water levels in the downstream Paraná River
- The variation and the probability of exceedance of discharges in the downstream Paraná River
- The relationship between water level and discharge of the downstream Paraná River

ADCP measurements of the current velocity in the downstream Paraná River were obtained between 2002 and 2018 and used to validate the numerical model output. The spatial variety of the currents was measured during a detailed ADCP measurement campaign executed on 15/02/2018. A total of 18 ADCP transects across the river were measured during this campaign (Figure 2). Thereby providing information on the spatial current field at many locations. The discharge at the time of the measurements varied between 12,500 and 14,000 m³/s.

Telemac-3D uses a finite element numerical method to solve the Saint-Venant equations that describe the depth-averaged flow of the river. The effect of helicoidal flow in river bends is taken into account. Multiple schemes are available for computing the advection of model variables. The default is the 'method of characteristics'. This method treats advection and diffusion as mutually independent and assumes that the value of the advected variable is equal to the value of the same variable in the previous instant traced back on the path travelled during the time step. In this study, next to the default scheme, an explicit scheme is used. This scheme requires smaller time steps (Courant number < 1), but is known to perform better in terms of mass conservation. The bathymetry in the numerical model had to be compiled from three data sources due to specific situation with varying waterlevels and complex geometry from:

- Data from the multi-beam measurement carried out at the beginning of 2018,
- Data of the complementary multi-beam measurement carried out in May 2018,
- Topography and bathymetry of the river banks before the construction of the Itaipu dam.

Next to the ADCP measurements, a physical scale model was used for the calibration of the numerical model results. This 1/100 scale model was built in 1978 to support the design of the Itaipu dam and covers the downstream river until approximately 25 km downstream from the dam. This allowed to capture possible backwater effect from the Yguazu River. It was revitalised and instrumented for this project specifically, after being used as a tourist attraction in the previous years. The bathymetry in the scale model is based upon measurements conducted prior to the construction of the dam. For comparison with the numerical model, ADCP measurements of the current were conducted at different transects.

Results

The water level varied over more than 35 meter in the period for which data was available. Water level variations are strongly correlated to the discharge through the Itaipu dam and remain between 98.6 and 111.4 m + MCT at a station just upstream from the bypass system for 90% of the time. The total daily average discharge through the spillway and turbines varies from approximately 6,000 to 25,000 m³/s, with some extreme events of up to 40,000 m³/s. In 2001, changes were made to the operational use of the Itaipu dam.

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1 Telemac Modelling system - TELEMAC-3D Software - Operating Manual. EDF R&D July 2016
downtime is considered acceptable, current fields for the nautical manoeuvring simulations were produced for discharges of up to 20,000 m³/s.

The first results of the Telemac-3D simulations showed significant discrepancies with the ADCP measurements of current fields that were obtained in the project area. Specifically:
1. Recirculation in front of the entrance to the bypass was not well captured by the numerical model.
2. Systematic underestimation of the (average + surface) velocity in the river.
A detailed sensitivity study revealed that the numerical model results were steered by parts of the bathymetry in the numerical model, based upon the physical model (thus obtained prior to the construction of the dam). An improved bathymetry was constructed implementing the recent multibeam echosounder measurements (see methods) and strongly improved the modeled velocity magnitudes in the river. Moreover, it turned out to be crucial to use the explicit advection scheme in the numerical model to realistically model the velocity field in the river bend where the bypass will connect to the downstream Paraná River. Both the ADCP measurements and the physical scale model revealed that a clockwise rotating eddy forms in the river bend at this location. Figure 4 shows a comparison between the velocity magnitude as obtained from the physical scale model and the numerical model at transect P3 (see Figure 2). The explicit scheme is clearly needed to model the (return) flow that is present in the outer bend of the river.

The Telemac-3D model was used to provide velocity fields that correspond to discharges of 8,000 m³/s, 14,000 m³/s, 18,000 m³/s, and 20,000 m³/s for the nautical manoeuvring simulations. Additional velocity fields were derived to assess the sensitivity of the velocity fields on the water level. Fast-time nautical manoeuvring simulations of the design vessel were performed by MARIN. Based on the results of the fast-time analysis it is concluded that a design vessel existing of 2x2 barges pushed convoy of 160 m length and 16 m width can safely enter the designed bypass system for discharges up to 20,000 m³/s. This showed that the expected downtime due to conditions that limit the navigability is limited to 2%.

**Conclusions**

The Paraná River downstream of the Itaipu dam is highly variable and dynamic. Typical water level fluctuations range over 12 m and typical discharges cause flow velocities over 2 m/s. The bathymetry of the river changed significantly since the installation of the Itaipu dam. It was essential to include the recent bathymetric surveys in the hydrodynamic model to obtain realistic velocity fields that could be used for the manoeuvring simulations. Moreover, an explicit advection scheme was needed to be used to correctly model the return flow that occurs in the outer river bend at the location where the bypass connects to the downstream Paraná River. Fast-time nautical manoeuvring simulations showed that a design vessel existing of 2x2 barges pushed convoy of 160 m length and 16 m width can safely enter the designed bypass system for discharges up to 20,000 m³/s.