Optimization and analysis of lock gates in the framework of the "Seine-Escaut-Est" waterway upgrading

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SUMMARY

This paper presents a research study performed on lock gates. It concerns the downstream lock gate of the one of the four new locks planned within the framework of the "Seine-Escaut Est (SEE)" project in the Walloon Region of Belgium. At the stage of the basic preliminary design, it was decided to use four identical gates, all suspended and moved transversally to the lock. On this basis, the present work tackles different aspects of the lock gate study. The aim is double: on the one hand, to advance in the study of the four SEE project downstream lock gates, and on the other hand, to focus more particularly on lock gate structural analysis, notably the design, optimization and structural behavior in the case of ship impact.

First, the design and optimization of the gate are performed, using the LBR5 lock gate optimization software and a linear elastic analysis. An optimized solution is obtained considering the best compromise between the cost and weight aspects of the structure. Then, this optimized gate is modeled with the nonlinear finite elements software FINELG. This program is used to conduct non linear numerical analysis of the effect of boat impact on the previously optimized downstream gate. Several analyses are performed, which allow for a discussion on the influence of the stiffener dimensions and the impact zone on the structural behavior of the gate submitted to the impact. Two different behaviors are brought to light, a ductile one and a fragile one. The results of the numerical analysis underline the importance of the development of a global plastic mechanism with the purpose of dissipating a large amount of energy. Finally, an analytical model presented in literature allows for the simplified calculation of the gate theoretical strength in case of ship impact, and the calculated value is compared with the computed results.

KEY WORDS: lock gate, structure optimization, ship impact, crashworthiness analysis

1. INTRODUCTION

1.1 The "Seine-Escaut Est" project

"Seine-Escaut Est (SEE)" is an ambitious project with the purpose to connect the river basin of the Seine to the European waterway network towards Northern Europe and Central and Eastern Europe, to the Black Sea (Fig. 1). This connection affects a zone of first importance for Europe:

this zone represents less than 4% of the surface of Europe-25, but it includes 12.6% of its population and concentrates 17% of its GDP (Fig. 2). Besides, the project connects the large seaports from the Havre, Antwerp and Rotterdam, which concentrate 60% of the maritime flows of Western Europe [Eurostat Base Regio, 2002].

Fig. 1 : Seine-Escaut Est (SEE) connection

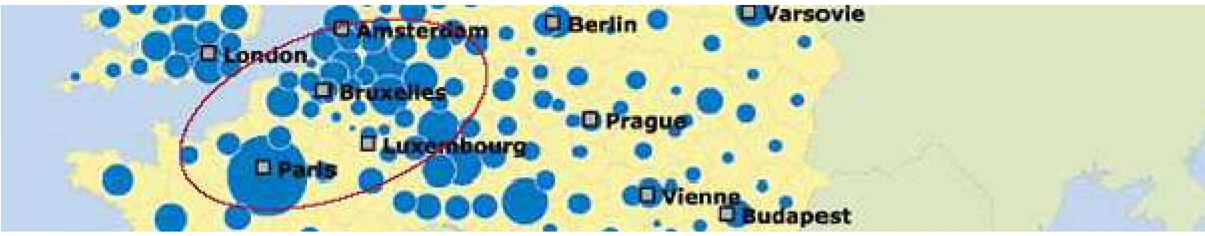

Fig. 2 : GDP by region [Eurostat, 2003]

Within the framework of the SEE project, the Walloon Region plans several works to enlarge some hydraulic structures of its network. The objective of the Walloon Region is to be able to receive the new traffic generated by the "Seine Nord Europe" project, and to keep this way its strategic position within the European waterway network. This paper deals with the construction of four new locks (class V_a in Europe) on the section connecting the Schelde (Escaut) river basin and the Meuse river basin, as part of these developments. The concerned sites are the sites of Obourg, Viesville, Marchienne-au-Pont and Gosselies. The study concerns the downstream gates of these locks, as significantly larger than the upstream gates.

1.2 Characteristics of the gate

At the stage of the basic preliminary design, realized collectively by the University of Liege, the Hydroconsult office and the Service Public of Wallonia (SPW), it was decided to use suspended gates moved transversally to the lock (Fig. 3). It is planned to use four identical downstream gates to take advantage of the standardization (reduced costs of study, facilitated maintenance…). The gates dimensions are summarized in Table 1. The gate width remains to be determined.

Length	Height	Width
13.70 m	13.60 m	1.00 m < l < 1.80 m

Table 1 : Dimensions of the downstream gates

A lock gate is constituted by one or several plate elements called "panels", which provide the function of watertightness, as well as a series of "beam" type linear elements, which support the waterproof panel and provide the gate with strength. Assembling these two types of elements give what is called a stiffened panel. The first objective of the study is to design and to optimize the gate, which implies to determine certain number of parameters:

- The thicknesses of the different panels
- The positions and dimensions of the «beam» elements: stiffeners, frames and girders

In addition, some design choices have to be made, in particular the question of the gate width as well as the possibility of using waterproof compartments (ballast tanks) to lighten the gate. A S235 steel grade was considered (235 MPA as yield stress) leading to an allowable stress of 175 MPa in elastic design.

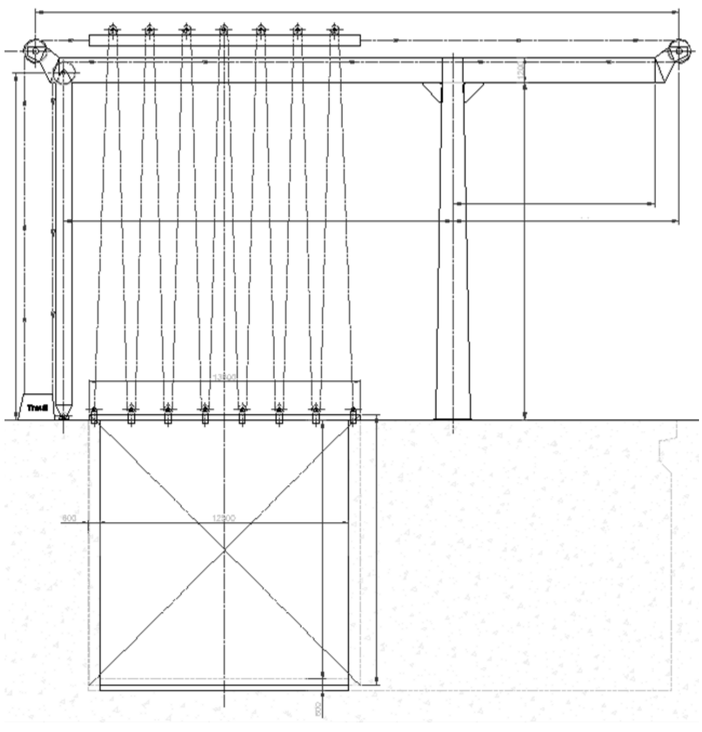

Fig. 3 : Downstream lock gate (elevation sight)

2. DESIGN AND OPTIMIZATION OF THE GATE

2.1 Design and optimization process

Four different models of downstream gates were performed: two models with additional ballast tanks and two different gate widths (Fig. 4a), and two models without ballast tank and also two different gate widths (Fig. 4b). The aim was to optimize each model and then to compare the optimized solutions in order to keep the most interesting design. The optimization of the models is realized by performing a multicriteria optimization; the criteria being the weight and the cost of the gate structure. To make it possible, the Pareto curve is derived for each model using five optimized solutions of the same model but changing the ratio between the criteria. The solutions are optimized according to objectives varying from (100% minimum weight - 0% minimum cost) to (0% minimum weight - 100% minimum cost) with intermediate objectives (Fig. 5). That allows for obtaining the Pareto curve for each gate model. The Pareto curve is the curve giving the zone of the design space (in terms of adimensional cost versus adimensional weight) where the feasible solutions are located (Fig. 6).

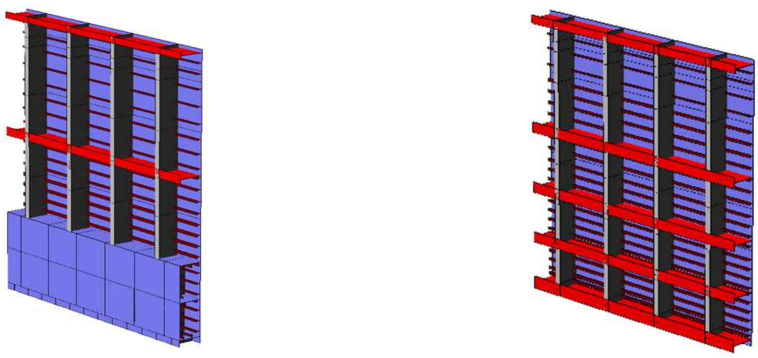

Fig. 4 : a. Gate with lower ballast tanks – b. without lower ballast tank

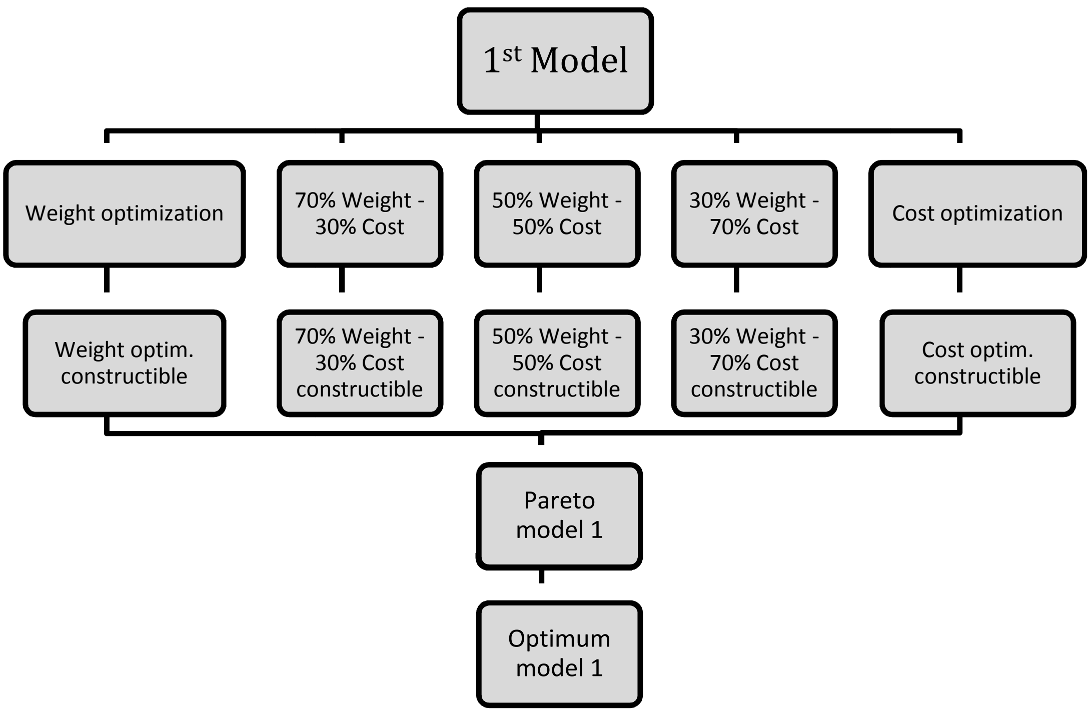

Fig. 5 : Optimization process for one model of lock gate

The considered load case is the exceptional hydrostatic load case for which the downstream section of the canal is empty, so that the maximum hydrostatic pressure is applied on the upstream side. The optimization is based on an elastic structural analysis. The risks of instability (buckling) of the stiffened elements (stiffeners, frames and girders) are taken into account by the definition of adequate slenderness ratio and the assessment of the ultimate capacity of the beam-column components. The risks of plate buckling are considered using the PLTBEN algorithm integrated into the LBR5 software [Hugues, 1983]. The optimization of the downstream gate is realized using the LBR5 software developed by Rigo [Rigo, 2002]. Each model is optimized according to the same process which leads to an optimized feasible solution after several successive stages, as schematized in Fig. 5.

From the gate model Pareto curve, the optimum solution for this model can be determined in agreement with the criteria of selection of the decision-makers. The same process applied to the four models gives four optimum solutions differing by their initial choices of design variables (gate width, ballast tanks). The comparison of these four solutions based on the cost and weight of their structure is a key element to make the best choice of design. Nevertheless, to guarantee the best decision, a more extensive study taking into account all the impacts of the initial design choices, simultaneously on the gate structure and on the other elements of the lock, is required. Such analysis is out of the scope of this study.

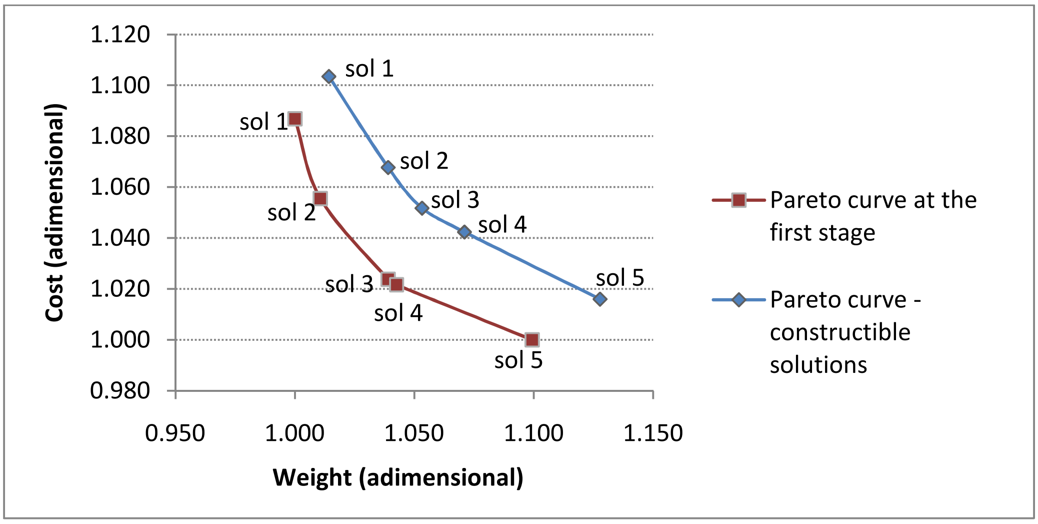

Fig. 6: Pareto model 1 – 1st optimization and optimization of constructible solutions

As shown in Fig. 6, the feasible solutions are slightly more expensive and heavier than the solutions obtained after the first optimization run. Indeed, the first dimensions supplied by the software are rounded off and standardized to obtain at the end a feasible solution, easily constructible.

2.2 Optimum solution

At the end of the design and optimization study, an optimum solution of the downstream gate is obtained. It appears better to select a gate of 1.0 m width (minimal value in order to place a footbridge) without ballast tanks. Indeed the ballast tanks generate a significant additional cost for a rather small benefit (Fig. 7). Indeed, these ballast tanks allow for lightening the structure thanks to the additional buoyancy, what is advantageous for the manoeuvre system (decreases of the effects on fatigue). But this system has still to be capable of supporting the total weight of the structure in case of defect of the tanks or during maintenance of the lock (empty lock). This limits the savings on cables and frames, and makes ballast tanks too expensive with regard to the expected profits.

The optimum solution is presented in Fig. 8. The analysis of this solution was realized under all the hydrostatic load cases susceptible to act in situation of service or exceptional. These analyses were made in both directions of loading (taking into account the two possible orientations of the gate) to consider successively the risks of instability of the plate and the reinforcement elements. The results are summarized in Table 2. The total weight of the gate is 51.4 t and the production cost of the primary structure is estimated to 56,202 €.

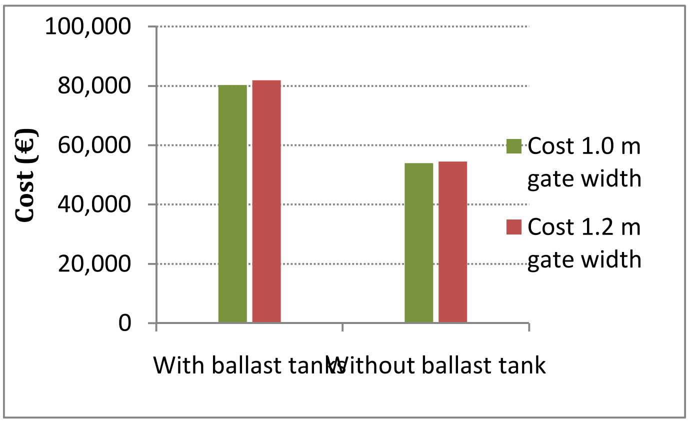

Fig. 7 : Solution costs in relation to their width

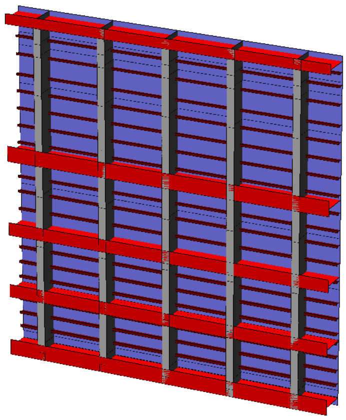

Fig. 8 : Optimum solution

LBR5 analysis	Service case		Water flood		Downstream section empty	
Orientation of the gate	**Direction 1**	**Dir. 2**	**Dir. 1**	**Dir. 2**	**Dir. 1**	**Dir. 2**
σ_{max} girders	151 MPa		168 MPa		159 MPa	
σ_{max} stiffeners	156 MPa		168 MPa		171 MPa	
σ_{max} frames	157 MPa		169 MPa		174 MPa	
Plate (ratio thickness/min thick.)	1.16	1.08	1.11	1.04	1.13	1.06
Deflection	18.3 mm		20.4 mm		19.4 mm	

Table 2 : Analysis results of the optimum solution for different load cases

3. FINITE ELEMENTS MODEL

A finite elements model of the optimum solution was realized using FINELG [de Ville, 1994], a non linear finite elements modeling software. The aim was to realize a non linear numerical analysis of the gate submitted to ship impacts. Taking advantage of the symmetry of the structure, half a gate was modeled using 4,187 shell elements, 850 beam elements and 222 elements for linear constraints (Fig. 9). The mesh size is of 300 mm x 300 mm in the zones of low stress and 150 mm x 150 mm in the zones directly subjected to the ship impact and to higher stresses.

First of all, the model was tested by realizing a linear analysis with the hydrostatic load case studied in the previous stage with the LBR5 software. It allows for the validation of the finite elements model and on the other hand for a discussion about the comparison between the LBR5 software and a finite elements analysis with FINELG. The analysis show a very good concordance of the results given by the two softwares except for the maximum deflections: the maximum deflection given by the finite elements linear analysis is a 20% superior to the value given by the LBR5 analysis. The reason is that LBR5 does not consider the local bending of the plate between two frames and two stiffeners. So, the plate deflections given by LBR5 and FINELG along the girders are equal (no local effect) whereas they can differ by 20% in the middle of an unstiffened plate, the deflections given by FINELG being the biggest. This difference is due to the local plate bending (Fig. 10).

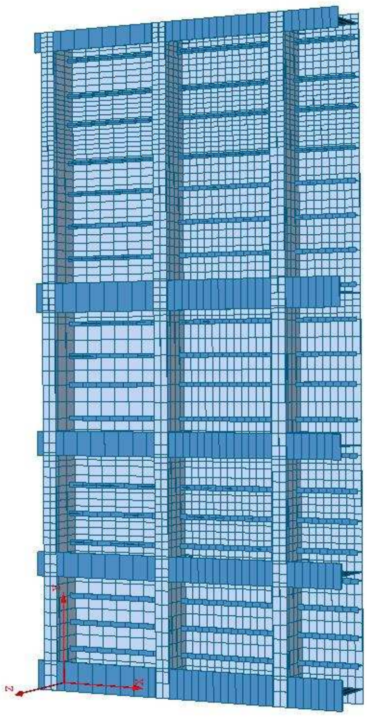

Fig. 9 : FEM of the gate

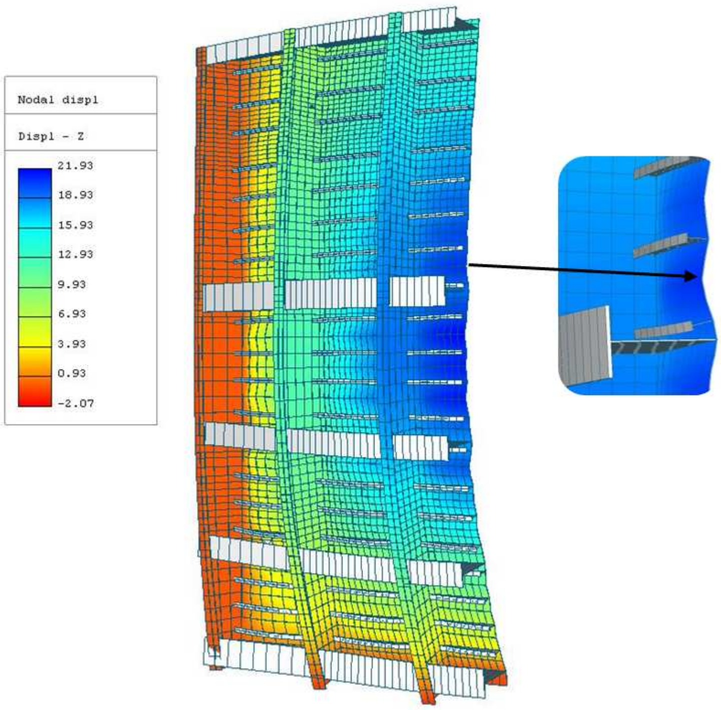

Fig. 10 : Service load deflection and local effect (x80)

4. SHIP IMPACT ANALYSIS

4.1 Assumptions

In this part, the effect of ship impact on the lock gate is studied. The analysis is based on the principle of energy equivalence. An important assumption is that the totality of the energy brought by the ship is dissipated by the gate as strain energy (Eq. 1 and 2). This assumption was validated by several studies [Le Sourne et al., 2002]. Therefore, a quasi-static equivalent load is defined to perform the analysis, so the dynamic effects are not taken into account. According to this approach, it is possible to link the ship initial kinetic energy to a given strain state of the gate (Eq. 2).

$$\begin{cases} W_E = E_{kinetic,initial} \\ E_{dissipated} = F_{impact} \times \delta \end{cases} \tag{1}$$

$$W_E = E_{dissipated} \Rightarrow E_{kinetic,initial} \leftrightarrow \delta \tag{2}$$

The constitutive law of the steel is an elastic – perfectly plastic law. The impact is applied by increasing a uniform force on a perfectly rigid element that represents the ship bow. Three different scenarii of impact are studied to allow for a discussion on the influence of the hydrostatic loads and the impact zone (Fig. 11):

1. The ship impacts the gate in its upper part (at upstream water level: U.W.L.), but the hydrostatic loads are neglected. This first analysis allows for the identification of the only ship impact effect in order to get a better understanding of the phenomenon.
2. The ship impacts the gate in its upper part (at upstream water level) while the hydrostatic service loads are already applied to the gate. This analysis models the case of ship entering in the lock from upstream and hitting the downstream gate.
3. The ship impacts the gate in its lower part (at downstream water level: D.W.L.). There is no hydrostatic load to take into account since the water-levels are identical on both sides of the gate.

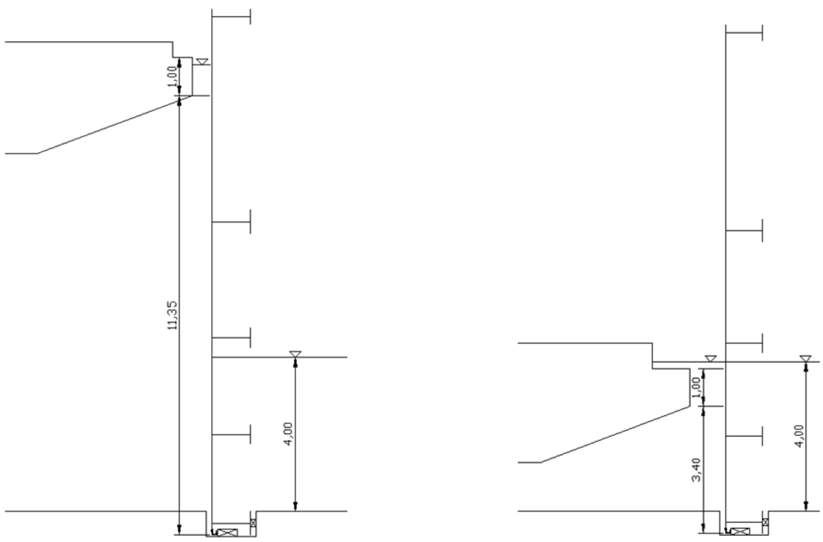

Fig. 11 : Impact at upstream water level and downstream water level

The gate is the structure designed and optimized in the previous part of the study using the LBR5 software, which finite elements model has been previously defined.

4.2 U.W.L. impact with the initially optimized structure

The study concerns the effect of an upstream water level impact on the gate structure elastically designed with LBR5. For this structure, the slenderness ratio of the stiffened panels respects Hugues' criteria for T-elements. Hugues' criteria fit with the Eurocode class 3 [Eurocode 3, 2005]: they guarantee that the section is able to develop its elastic bending moment before collapse through buckling, but not its fully plastic bending moment. These slenderness' are perfectly adapted for structures working in the elastic field but on the other hand they are not enable to take advantage of the plastic field.

The results of the non linear numerical analysis of the impact conducted with FINELG are given in Fig. 12. We give the evolution of the impact force in function of the indentation (in mm). The observed behavior is fragile: the collapse appears suddenly, while the structure stiffness is still considerable. Consequently, the capacity for energy dissipation is weak. The point representing the impact effect of a 2,400 t ship at 0.25 m/s (initial kinetic energy of 75 kJ) is plotted on the curve.

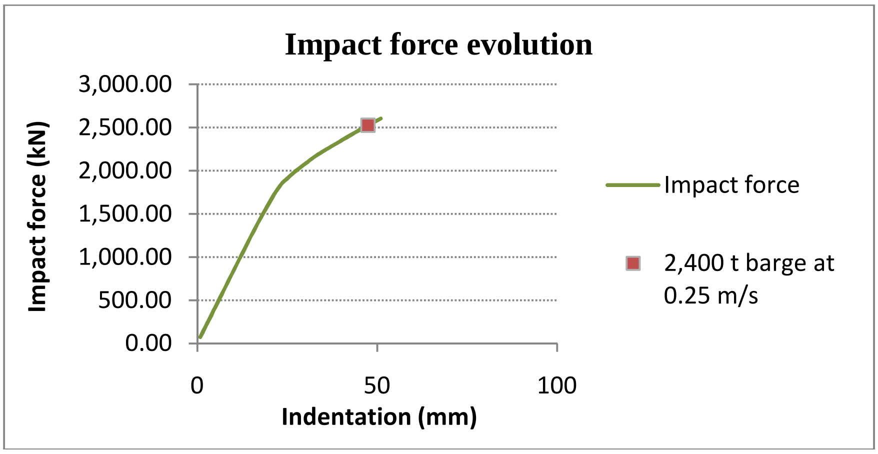

Fig. 12 : Impact force evolution for an upstream water level impact on the initially designed structure

The analysis of the strain level at collapse stage allows for a better understanding of the structure behavior. The buckling of the central frame is clearly visible. This buckling leads to a sudden, fragile collapse. Fig. 13 shows this buckling phenomenon. Therefore, the structure is not able to develop plasticity; its capacity for energy dissipation is indeed extremely weak.

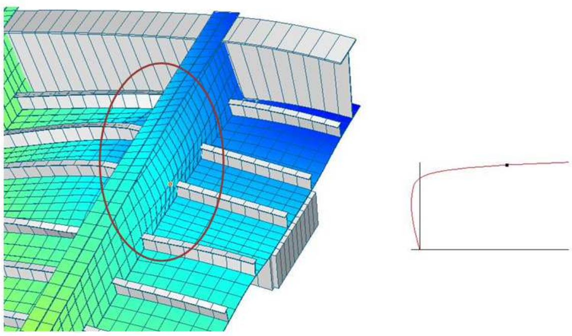

Fig. 13 : Buckling of the central frame and load-displacement curve for one node of this frame

As a result of this first analysis, it was decided to reinforce the structure to provide a better behavior in case of impact. The aim was to avoid that a buckling phenomenon induces prematurely collapse of the gate, preventing the structure from developing yielding behavior. It was so decided to increase the thickness-height ratios of the sections of the primary reinforcement elements (frames and girders) in order to obtain class-1 sections according to the Eurocode classification. As a reminder, a class-1 section is able to develop its fully plastic bending moment and sufficient rotation to allow for the development of a global plastic mechanism in the structure. The next analyses have all been performed with such reinforced structure.

4.3 U.W.L. impact with the reinforced structure

The frames and the girders of the gate have been reinforced to be class-1 elements. Their web thickness was increased from 10 mm to 20 mm and their flange thickness from 17 mm to 25 mm. The total weight of the structure has gone up from 51.4 t to 68.7 t (+34%). The results of the non linear numerical analysis of the impact on this structure are given in Fig. 14, next to the curve of the initially optimized structure. Different impact levels are marked on these curves.

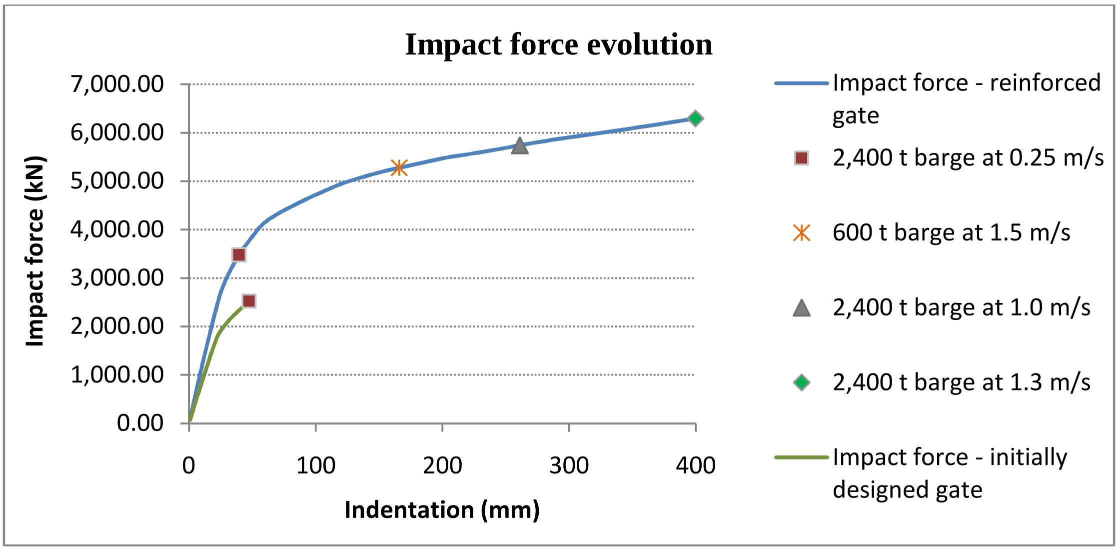

Fig. 14 : Impact force evolution for an upstream water level impact

As shown in Fig. 14, the global behavior of the structure for the same impact scenario is fundamentally different after increasing the gate stiffness. The response of the class-1 higher stiffened structure is ductile; its capacity for energy dissipation is very significant (the kinetic energy of a 2,400 t barge at 1.3 m/s is 2,028 kJ). The choice of such a higher rigid structure gives a much more favorable behavior in case of ship impact.

The analysis of the strain state of the collapsed structure and the plastic hinges highlights the formation of a global plastic failure mechanism (Fig. 15). This plastic failure mechanism strongly contrasts with the fragile failure of the initial structure, in which there is almost no yielding developed before the collapse.

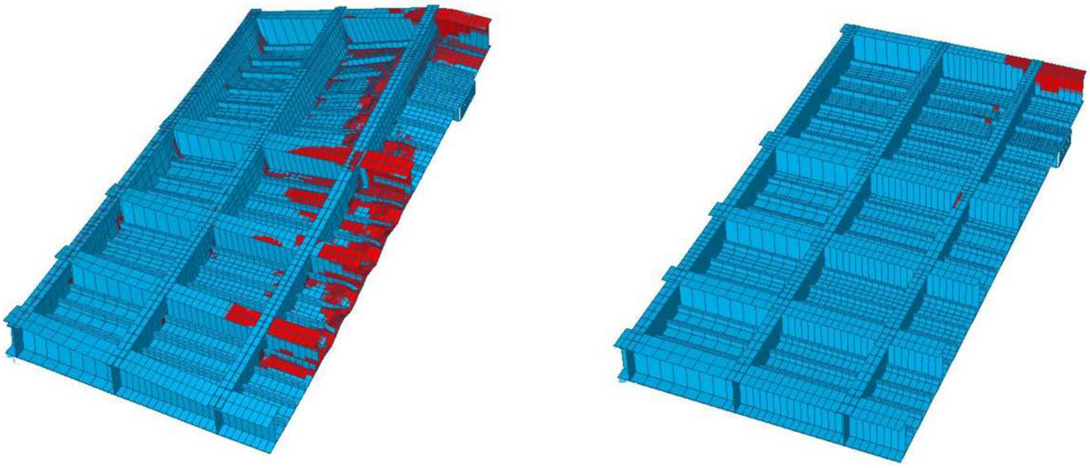

Fig. 15 : Yielding at the collapse stage – reinforced structure (left) and initially designed structure (right)

The analysis of the global plastic failure mechanism shows that the loss of stiffness of the structure, visible in Fig. 14, is due to the successive plastic hinges in the girders. A good ductility of the main girders is thus required to provide the structure with ductility, which is very important to ensure a good capability for impact absorption. Fig. 16 shows the points on the load-displacement curve where successive plastic hinges appear in the girders. These points correspond to the loss of stiffness of the gate. The global plastic failure mechanism developed by the gate includes two plastic hinges lines along the gate height. It is interesting to notice that using this simple failure mechanism, it is possible to calculate analytically, in a simplified way, the gate strength.

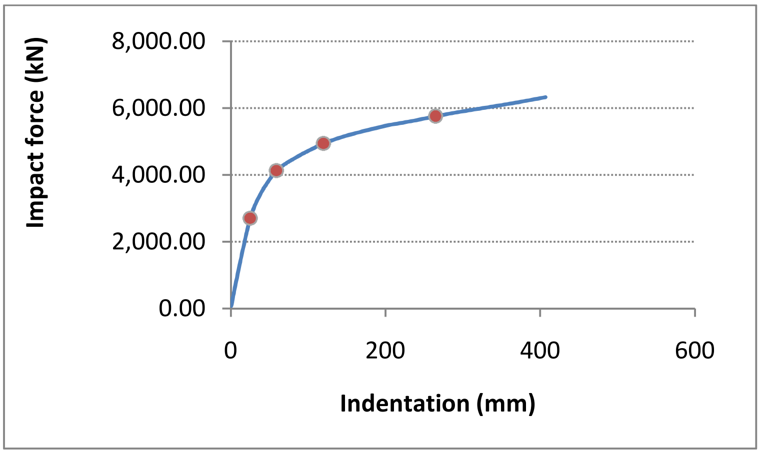

Fig. 16 : Development of the global plastic failure mechanism (after increasing the gate stiffness)

Locally, it is noteworthy that the maximum strain reaches only 6.4% at the failure stage, when the gate is submitted to an impact corresponding to a 2,400 t barge at 1.3 m/s. In the minor collision analysis performed by Mc Dermott [McDermott, 1974], the critical rupture strain for mild steel material in side collision is evaluated from the tensile ductility, so that $\varepsilon_c \approx 10\%$. This means that the gate structure can develop its plastic failure mechanism and absorb a very important amount of energy without apparition of local failure.

4.4 Taking into account the hydrostatic loads

This analysis concerns an upstream water level impact combined with the hydrostatic load applied on the gate. First, the hydrostatic load is applied and then, keeping the water pressure constant, the impact is applied. In Fig. 17, the evolution of the impact force with and without hydrostatic load is presented. Note that the displacement given on the curve with hydrostatic load is the displacement only due to ship impact. It is different from the total displacement, result of the sum of the hydrostatic load and the impact.

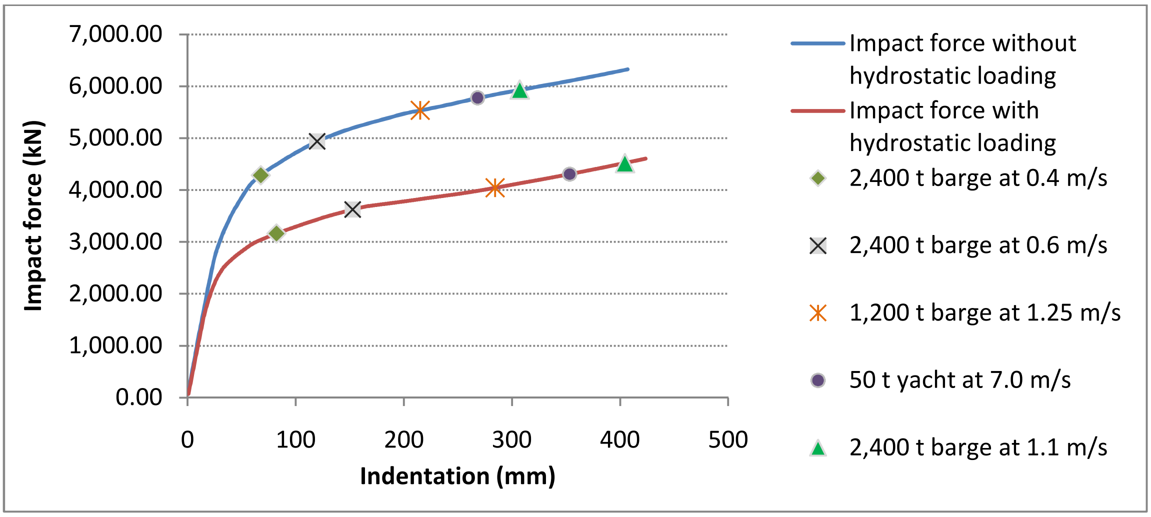

Fig. 17 : Impact force evolution for an U.W.L. impact, with and without hydrostatic load

The global behavior of the gate is identical but the structure is more deformable when it is previously submitted to the hydrostatic load. Indeed, since the water pressure is applied, the gate is already submitted to a stress field. Then when the ship impacts the gate, plasticity appears faster in the gate elements. Consequently, for a same impact load, the indentation is more significant with hydrostatic load. Besides, yielding is increased. On the contrary, the impact force is reduced.

As a conclusion, the global behavior of the gate is unchanged whether the hydrostatic load is applied or not. On the other hand neglecting this load during the impact analysis leads to underestimate the deformation and the yielding of the structure. So, a method which first consider the hydrostatic load, and then adds the deformations due to the only ship impact, underestimates the state of deformation and yielding of the structure. But this approach is safe from the point of view of the impact force. It would be conservative to assess, for example, the maximum reaction susceptible to act on supports.

4.5 Impact on the downstream side of the gate

The next analysis deals with the case of a downstream side impact. In this case, there is no situation where a hydrostatic load could be added to the impact effect and increase this effect. The study of this case allows for the analysis of the impact zone influence. Here, the ship hits the gate in a highly more stiffened zone than in the previous cases. Fig. 18 shows the impact force evolution in the case of a downstream side impact, next to the correspondent curves for an upstream water level impact.

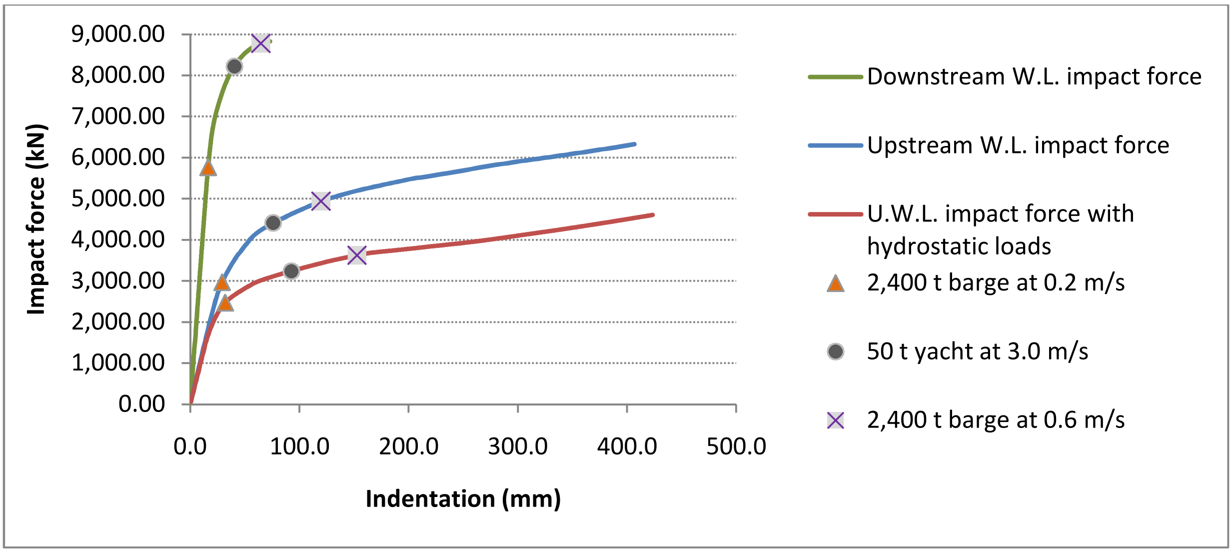

Fig. 18 : Impact force evolution for different impact cases

The gate structure is more fragile for a downstream side impact compared with an upstream side impact. The impact force increases much more quickly and reaches significantly more important values while the indentation remains small. Finally, the collapse arises suddenly for an impact of energy in the order of 450 kJ.

The strain pattern in the gate at the collapse stage shows that there were strain concentrations in the impact zone, mainly in the frame in contact with the barge bow (Fig. 19). This strain peak is due to the small ratio between the transverse and longitudinal stiffness in this zone, which prevented the propagation of yielding and thus the development of a global plastic failure mechanism. It is harmful to the structure ductility and thus to its energy dissipation capacity. Finally, the collapse arises by frame buckling at the level of the plastic hinge, because the rotation of this frame becomes too big. When this collapse occurs, the indentation is still small because the plastic deformations have not propagated much.

Then, the analysis of the downstream side impact shows clearly the importance of a thought on the stiffness ratios in the potential impact zones. For such gate, the transverse elements (frames) stiffness should be higher compared with the longitudinal elements (girders) stiffness, in order to guarantee a good propagation of the plastic deformations and, ideally, the development of a global plastic failure mechanism.

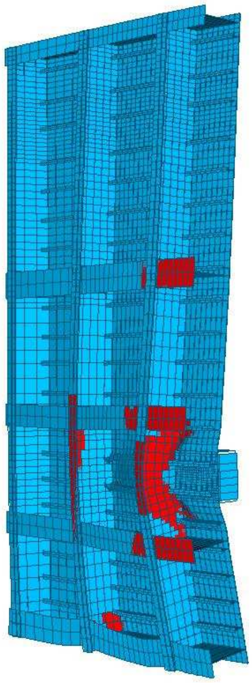

Fig. 19 : Yielding at the collapse stage, downstream side impact

5. ANALYTICAL ASSESSMENT OF THE IMPACT STRENGTH

It is possible to calculate analytically the gate strength in the case of ship impact, considering a collapse mechanism. The considered analysis is based on a method developed by Le Sourne [Le Sourne et al., 2002 & 2003]. The analysis is performed for an upstream side impact, neglecting the hydrostatic load. The considered global plastic failure mechanism is the mechanism highlighted by the numerical analysis presented in the previous sections (Fig. 20). This mechanism shows a global gate bending around the plastic hinge lines determined by the bow shape and by the impact location.

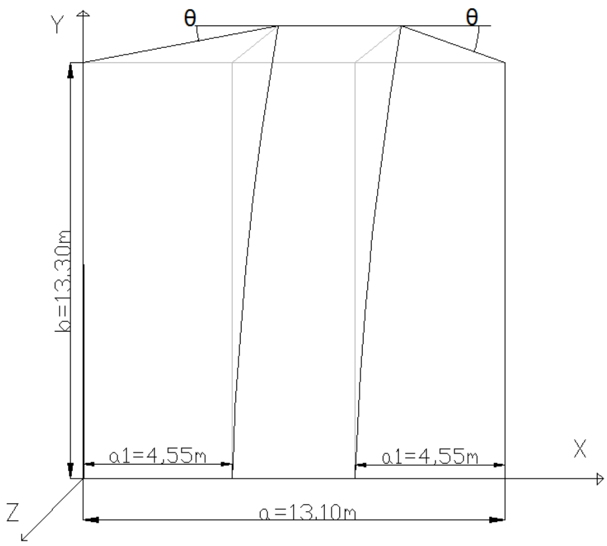

Fig. 20 : Global plastic failure mechanism considered for the analytical analysis

The bending energy rate limited to the two plastic hinge lines can be expressed by Eq. 3.

$$\dot{E}_b = 2\int_0^l \hat{M}_0\,\dot{\theta}\,dl \tag{3}$$

where $\hat{M}_0$ is the fully plastic bending moment per unit length of the plate, θ the local rotation (equal on both side by symmetry) and l the length of the plastic hinges. The local rotation is given by Eq. 4.

$$\theta = \frac{w(a_1, y)}{a_1} \;\rightarrow\; \dot{E}_b = \frac{2}{a_1}\int_0^l \hat{M}_0\,\dot{w}\,dl \tag{4}$$

where w is the transversal displacement and a_1 the distance between the lateral support and the plastic hinge line. In the energy rate formula, two unknowns appear: the fully plastic bending moment per unit length of the plate $\hat{M}_0$, and the displacement field $w(a_1, y)$. Each gate can be considered as the assembly of several elementary horizontal beams. Assuming that all these beams are only submitted to bending around a vertical axe, it is easy to calculate the total plastic bending moment of the gate using Eq. 5:

$$M_p = \sum m_{pi} \tag{5}$$

where the m_{pi} are the plastic bending moments of each elementary beam of the gate in kNm. The m_{pi} are linked to $\hat{M}_0$ by the relationship $\hat{M}_0 = \frac{m_{pi}}{h_{elementary\,beam}}$. The gate is discretized in a set of elementary beams and for each beam the m_{pi} are derived from Eq. 6:

$$m_{pi} = \int_{S_+} \sigma_y \left(z_p - z\right) ds + \int_{S_-} \sigma_y \left(z - z_p\right) ds \tag{6}$$

where S_+ and S_- are the cross sections on either side of the plastic neutral axis $Y'Y$, z is the second coordinate in the beam cross section, z_p is the coordinate of the plastic neutral axis and σ_y is the yield stress. For the considered gate structure, Eq. 5 and 6 give $M_p = 17{,}556.0\ kNm$.

It is then necessary to choose a displacement field $w(a_1, y)$. Though, since the gate structure is not uniform along the plastic hinges, the displacement field should be considered as non linear. The displacement field at the collapse stage has to be estimated and multiplied by the fully plastic bending moment $\hat{M}_0$ according to Eq. 4. Yet, the numerical non linear analysis shows that the displacement field along the plastic hinges tends to get close to a linear field when displacements become important. Fig. 21 shows the displacement field obtained by a linear finite elements analysis of the impact, and the displacement field obtained at the collapse stage with the non linear analysis of the same impact. As we can see, the non-linearity effect is to move the displacement field closer to a linear field. This can be explained by the fact that yielding leads to stiffness variations. While the impact increases, yielding reaches the totality of the plastic hinges, so that the gate structure stiffness becomes theoretically null. The global plastic failure mechanism has developed. At this stage, the gate gets deformed according to a linear field, since there remains no stiffness in the plastic hinges. So, assuming a linear displacement field is valid.

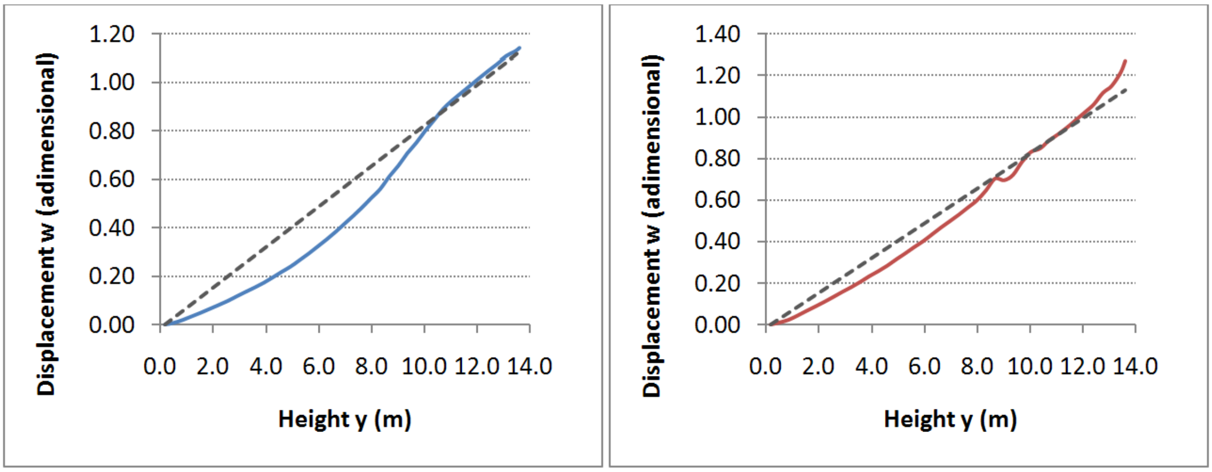

Fig. 21 : Calculated displacement fields compared with a linear displacement field: on the left, linear analysis – on the right, non linear analysis at the collapse stage

The linear displacement field is given by Eq. 7.

$$\theta = \frac{w(a_1, y)}{a_1} = \frac{\delta}{a_1} \frac{y}{h_{impact}} \tag{7}$$

where $\delta = w\left(a_1, h_{impact}\right)$ is the deflection at the impact point. The length of the plastic hinge l is related to the height of the gate b by Eq. 8:

$$l(y) = \left[y^2 + w(a_1, y)^2\right]^{1/2} \approx y\left(1 + \frac{\delta^2}{2\,h_{impact}^2}\right) \rightarrow l = b\left(1 + \frac{\delta^2}{2\,h_{impact}^2}\right) \tag{8}$$

After inserting Eq. 8 into Eq. 4 and neglecting the variation of the fully plastic bending moment per unit length $\hat{M}_0$ along the plastic hinges, the bending energy rate along the two plastic hinge lines is obtained by Eq. 9:

$$\dot{E}_b = 2\frac{\dot{\delta}}{a_1\ h_{impact}}\left(1+\frac{\delta^2}{2\ h_{impact}^2}\right)\int_0^b \hat{M}_0\ y\ dy = \frac{\dot{\delta}\ b^2}{a_1\ h_{impact}}\left(1+\frac{\delta^2}{2\ h_{impact}^2}\right)\hat{M}_0 \tag{9}$$

Finally, the gate strength is given by Eq. 10:

$$P_{gb} = \frac{\dot{E}_b}{\dot{\delta}} = \frac{b^2}{a_1\ h_{impact}}\left(1+\frac{\delta^2}{2\ h_{impact}^2}\right)\hat{M}_0 = \frac{M_p}{a_1}\frac{b}{h_{impact}} \tag{10}$$

where $M_p = l\ \hat{M}_0$ is the fully plastic bending moment of the gate. For the considered gate structure submitted to a symmetrical upstream water level impact, the gate strength calculated by Eq. 10 is $P_{gb} = 4,443.0\ kN$.

Fig. 22 shows the force-indentation curve obtained by the non linear FE analysis for the first impact scenario, next to the value of the strength obtained by the analytical assessment, assuming a linear displacement field. The strength obtained by the numerical analysis noticeably overtake (by +42%) the theoretical strength calculated analytically. Besides, the numerical curve has not reached its yield limit corresponding to the fully plastic failure mechanism, as supposed in the analytical analysis. The numerical result may indicate that the structure has found a new mode of strength overtaking the assumed plastic failure mechanism, for example a membrane mode. On the other hand, a margin of error has to be admitted due to the simplifying assumptions and to the model limitations.

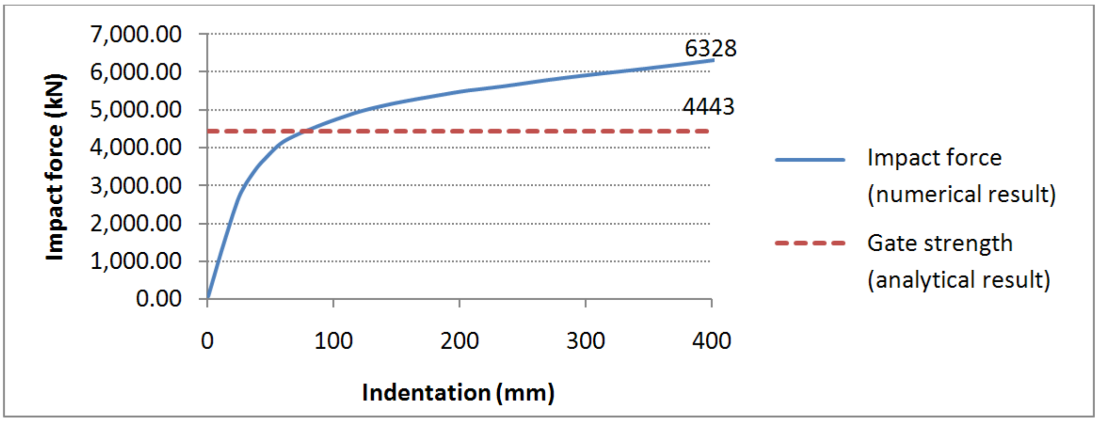

Fig. 22 : Impact force evolution: comparison of the numerical and analytical results in the case of an upstream side impact, neglecting the hydrostatic load

6. RESULTS OF THE IMPACT ANALYSIS

The results of the gate strength in case of ship impact are presented here. Fig. 23 gives the relationship between the impact speed and the indentation for a 2,400 t barge, function of the impact zone. Table 3 gives the effect of a determined impact for the different studied cases.

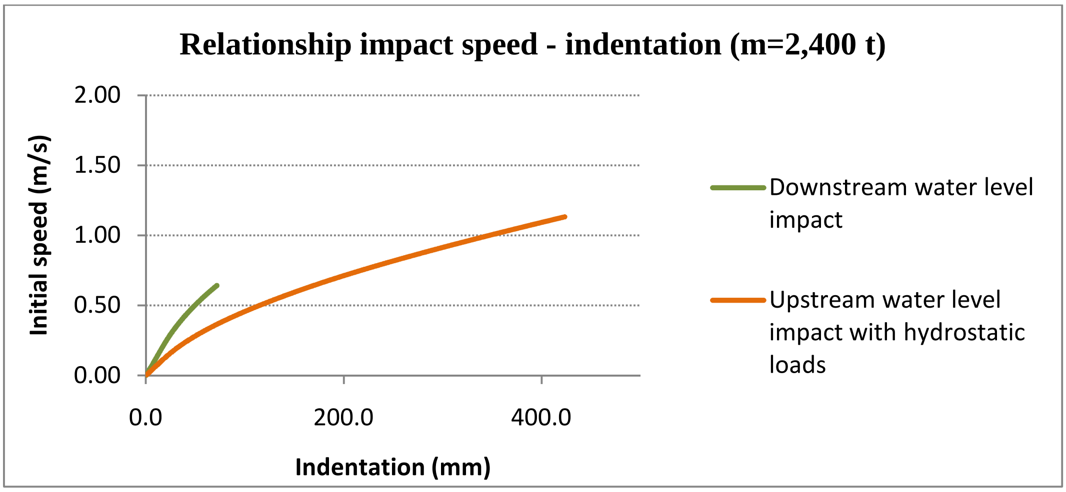

Fig. 23 : Relationship between the impact speed and the indentation for a 2,400 t barge, in the cases of downstream and upstream side impacts

Impact of a 1,200 t barge at 0.8 m/s (384 kJ)	U.W.L. without hydrostatic loads	U.W.L. with hydrostatic loads	D.W.L.
Impact force	4,845 kN	3,550 kN	8,706 kN
Indentation (only due to the impact)	111 mm	139 mm	59 mm
Number of plastic hinges in frames and girders	2 girders	3 girders	1 frame

Table 3 : Synthesis of the effects of a 1,200 t barge impact at 0.8 m/s, function of the impact zone and the hydrostatic loads

The main point learned from this gate impact analysis is the interest of developing a global plastic failure mechanism to provide the structure with a good carrying capacity for energy dissipation. It has been established that a condition to allow this behavior is to use class-1 cross sections for the main reinforcement elements (frames and girders). In addition, it is necessary to guarantee adequate stiffness ratios in the impact zone: the structure response being fundamentally different depending on whether the impact happened at downstream or upstream side. For an upstream side impact, the behavior is ductile, which allows to absorb significant impacts. For a downstream side impact, the behavior is fragile, which dramatically reduces the impact strength.

7. CONCLUSION

The first objective of this study was to provide a design recommendation for the downstream gates of the four new locks projected by the Walloon Region of Belgium (SPW) within the framework of the “Seine-Escaut Est” project. A design and optimization process has been built based on the elaboration of a representative number of solutions, using linear elastic analysis. The optimum solution has been identified using Pareto curves, in order to compare various solutions on the basis of their production cost and weight. Then, the optimum solution has been analyzed in a more complete way and in particular the effect of a ship impact on this gate has been investigated using nonlinear finite elements analysis.

The gate impact analysis has brought the designer to increase the dimensions (cross sections) of the frames and the girders of the optimum solution to obtain class-1 cross sections (Eurocode classification). Consequently, these reinforced elements do not correspond any more to the optimum solution according to an elastic design under hydrostatic loading. The additional cost and weight due to these modifications are respectively of +34% and +14%. Such gate impact study is required to determine in which extent the gate must be reinforced to sustain ship impact. Then, the additional cost of the reinforced solution should be compared with the elastic optimum solution coupled with protective device against ship impact, as “protection beams” for example.

For design purpose the main recommendation is to implement in the optimization software a new constraint that consists in using only class-1 cross sections for the frames and the girders. Including this constraint from the design and optimization stage would permit to obtain optimized solutions considering impact strength. The additional cost necessary to provide impact strength would be reduced if the constraint is integrated from the beginning. In addition, other parameters have to be taken into account because of their influence on the gate behavior in the case of ship impact. In particular, the impact zone stiffness can modify the structure behavior, because of its influence on the yielding propagation. It would be interesting to realize in the future a research focusing on ship impact on lock gate, in order to enhance the understanding of the influence of the various parameters. The aim would be to determine the constraints to fulfill, from the design and optimization stage, in order to provide the gate structure with ductile behavior, guaranteeing it good impact strength. Finally, the optimum solution would present a ductile behavior in case of impact while limiting the additional cost.

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