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Simulation and dimensioning of a hydraulic crossing structure on wadi Al Khairat in the Saouaf city-Tunisia

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Abstract: In a global context of climate change, extreme events such as floods have become the preponderant risk threatening living beings, ecosystems and their infrastructures. In this context, the Saouaf, city is among the areas with a particular vulnerability to the flooding risk since it is confronted at the wadi Al Khairat with the problem of the river banks erosion on the one hand, and the crossing structures insufficiency the RL630 road on the other hand, lead to the flooding of the RL6 30 road and the sports complex located nearby. The purpose of this study is to achieve a structure that respects the economic and functional constraints. To proceed we will deal with two main parts, the first consists in the realization of a 1D hydraulic model of Al Khairat River with the HEC RAS software, on which we will install the existing structure to check its condition in the flooding event. The second part will be centered on the determination of the suitable installation to satisfy the conditions imposed using the optimization. The study of the hydraulic modeling as well as the comparative study established throughout this study enabled us to choose the bridge variant with prefabricated beam in reinforced concrete (TIBA) to be the most favorable solution, both in the technical and economical aspect, ensuring the durability of the structure and minimizing the risk of submersion for a hundred-year return period.

1. Introduction

The current situation confronting the climate change is marked by a series of challenges, including recurrent episodes of drought and the increasing of flood risks (Romdhane *et al.,* 2020; El Morabet & Ouadrim (2016)). Floods have devastating consequences, damaging infrastructure, dragging people down and causing human and economic loss (Roche *et al.,* 2012). To face these challenges, it is essential to implement climate change adaptation and mitigation measures, as well as to strengthen disaster prevention and management infrastructures to protect vulnerable communities (Rodier *et al.,* 1981; Ward *et al.,* 2020).

The design of a hydraulic crossing structure most often results from an iterative approach, the objective of which is the optimization with respect to all the natural and functional constraints imposed, while integrating a certain number of architectural and landscape quality requirements (if it is a work of art). (Williams, *et al.* 1984 ; Rangari *et al.,* 2019). Always, flooding is regarded as a major threat that provokes a horrible risk to human survival and development worldwide. In this way, researchers to propose flood simulation models to predict the adverse effects of floods (Shen & Jiang, 2023; Guido *et al.* 2023; Piadeh *et al.* 2022; liu *et al.* 2019; Brouziyne *et al.* 2018).

Whether they are related to maintenance, the environment, installation or the appearance of an extreme flood, there are many reasons for culvert failures. These failures can be:

- Functional related to transit capacity and flow velocity that cause soil erosion around culverts.
- Structural due to the collapse or corrosion of the materials that compose them.

If the failure is sudden and catastrophic, it could result in injury or loss of life. Indeed, an under-dimensioning of the structure could cause the scouring of the surrounding soil over time and even the road traffic cutting and hence the villages isolation. (Williams *et al.,* 1984)

The crossing structure problem was related to solid transport comes down to the fact that builders generally try to compensate for the structure's effects (piles, abutments, etc.), (Zhang *et al.,* 2014) by expanding the bed watercourse to maintain the same passage section during a flood, this result in favoring the sediments transport at the bed bottom. (Wu, 2000; Wu, 2008)

Scour at the pile bottom is one of the main causes of bridge collapse. Indeed, the pile causes a local flow acceleration near its base creating a system of complex three-dimensional vortices which leads to the sediments erosion.

2. Study area presentation

The Saouaf city is located on the left bank of Wadi Al Khairat, one of the most important rivers in the region, bounded to the north by the Zaghouan governorate and to the south by Ennadhour. (Ezzine *et al.,* 2020). Topographically, the city is located 150 meters above sea level and is characterized by hilly to rugged terrain with medium slopes heading towards Wadi El Khairat. (BTE, 2017) On the other hand, a mountain range is located in the North and North-West part of the city and whose maximum heights reach 500m NGT **(Figure 1).**

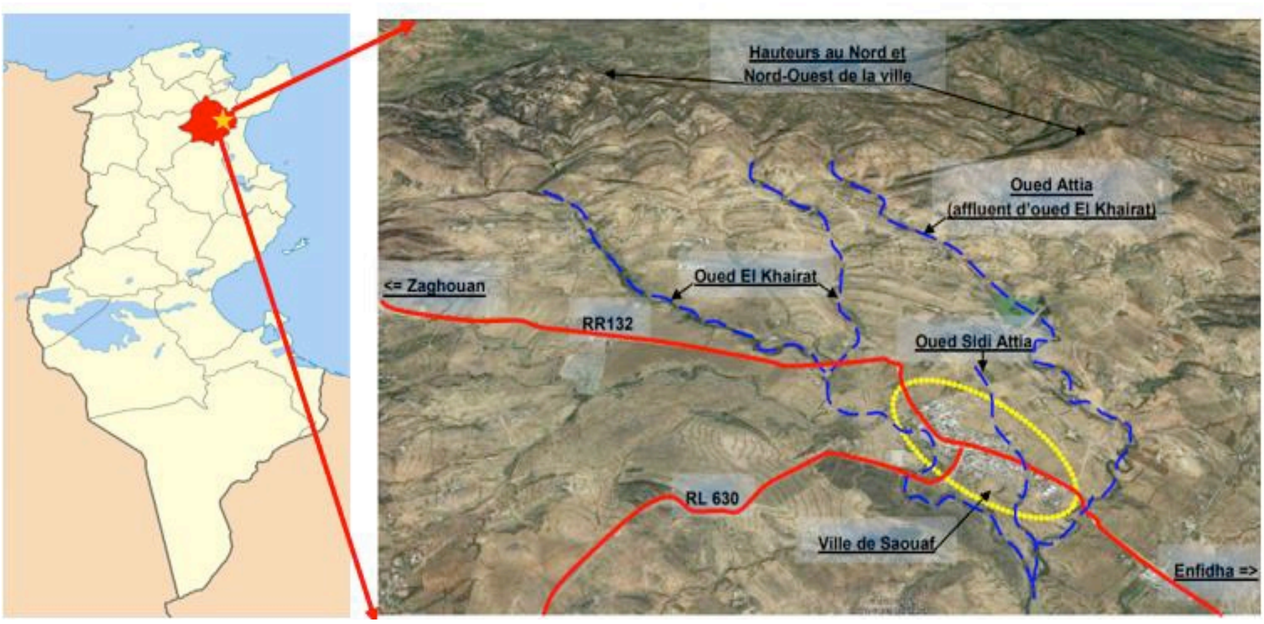

Figure 1. Saouaf city localisation

The rainfall regional distribution shows that the Governorate experiences an annual rainfall between 350 and 550mm. The Saouaf region is crossed by Oued Al Khairat which represents a main river draining a watershed of 16,000ha. It should be noted that there are also several secondary wadis, among others, we cite: Oued el Ksasba, Oued el Ksab, Oued el Ogla, Oued el Melah, Oued Sidi Attia. (Ezzine *et al.,* 2020). In addition, at the level of the Saouaf city, several tributaries originate from the hills located to the north and then cross the town. At the RL630 road, the crossing of the Oued Al

Khairat is ensured by a cassis on which is placed a multiple scupper composed of 5 cells having a dimension of (2×2 m) and a total evacuation capacity equal to 54.31 m^3/ s. we also note that the structure is poorly maintained. We note the accumulation of waste and vegetation at the level of the upstream opening culverts, which leads to the structure clogging. **(Figure 2).** At the cassis, the solid sediments deposit above the structure was noticed, which proves the insufficient transit capacity of the scupper and the overflow of the wadi in the flood events. (Ezzine *et al.,* 2020).

Figure 2. River problems

The watercourse is characterized by a marked bed with banks of significant heights exceeding ten meters in certain sections. The minor bed width varies from 30 to 60m. (BTE, 2017). On the other hand, and taking into account the erosion problem in the project area, it is remarkable the instability and the wadi banks scouring. Also, the watercourse near the Saouaf city is meandering, this causes the wadi overflow and the banks erosion. **(Figure 2).** The low areas and bordering the major bed of Wadi El Khairat have often suffered from flooding problems following the raising of the Wadi El Khairat water line and its overflow. These are the city's sports complex (municipal stadium) and buildings near the Christian cemetery. (BTE, 2017). From the above, it can be seen that the Saouaf city is a favorable place for the flooding and erosion phenomena. Indeed, its location between mountains, the vegetation cover poverty and the average annual precipitation which is between 300 and 400mm, are factors that increase its vulnerability to these problems. (Romdhane *et al.,* 2017)

3. Methodology

3.1 Delimitation of the "Al Khairat" watershed

The watershed delimitation constitutes a preliminary stage for the hydrological contribution's determination, from which one can deduce the surface drained by wadi Al Khairat. This delimitation was carried out using the Global mapper software, through a DTM with a 25 m resolution satellite image and on the basis of a topographic map at 1/25,000. The watershed characteristic thus determined are given in the following table (**Table 1**).

3.2 The rainfall statistical analysis

To study the precipitation, we need a rainfall series that includes the maximum daily precipitation for the longest possible period. (Gary, 2010). In our project, the rainfall series calculation will be based on the data provided by the Hammam Zriba rainfall station. In our study we chose to work with

Gumbel's law, Log Normal law and Gamma law, and to adopt the moments method. The results of these three laws calculated by HYFRAN are illustrated in **Figure 4** linking the maximum daily rainfall as a function of the not exceeding probabilities. It can be seen from these curves that the observations (measured rainfall values) are well within the confidence interval which corresponds to the interval delimited by the two blue curves for the three laws. This leads us to conclude that the three laws Gumbel, Lognrmal and Gamma fit well with our 22-year rainfall series.

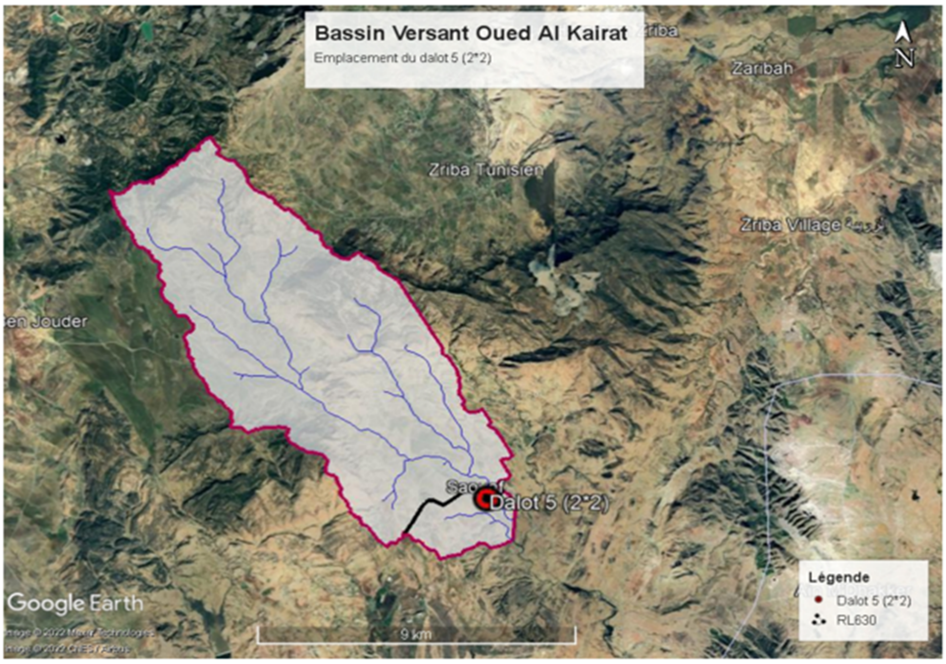

Figure 3. Watershed delimitation

Table 1. Watershed characteristic

$S(Km^2)$	P(Km)	Z_{min} (m)	Z_{max} (m)	L Talwég (Km)	I(%)	I(m/m)	Kg	L rect éq (Km)	l rect éq (Km)
64.54	40.583	131.8	657.3	15.583	3	0.03	1.41	16.25	3.96

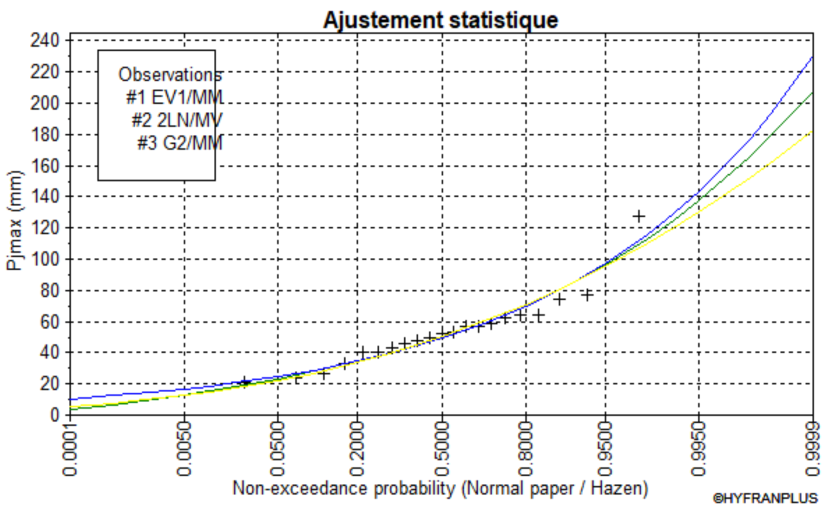

Figure 4. Statistical laws Adjustment (Gumbel, Gamma and Lognormal), Hammam Zriba Station (1999-2020)

On the other hand, we note that the values of the maximum daily rainfall calculated from these three adjustment laws are very close. for this, the maximum daily rainfall values (Pjmax) will be adopted, for the four frequency periods (10, 20, 50, 100 years), is the average calculated by the three statistical adjustment laws. **(Table 2)**

Table 2. The adopted Pluviometry

	Pjmax (mm)			
T (ans)	**Gumbel**	**Lognormal**	**Gamma**	**Adopted**
10	106	107	108	**107**
20	130	132	131	**131**
50	158	160	162	**160**
100	179	185	183	**182**

3.3 The frequency flows determination

The hydrological study is of great importance for the crossing structure realization. It is therefore a question of highlighting the Al Khairat wadi hydrological parameters such as the flood flows as well as the liquid inflows of the watershed in question, from which the crossing structures dimensioning is based **(Table 3)**

Table 3. Flows rate Calculation by different hydrological methods for different return periods

	Q (m^3/s)			
	T=10 ans	**T=20ans**	**T=50ans**	**T=100ans**
Rationnelle	116.04	127.67	134.67	140.27
Speed	165,08	210.62	265.65	307.39
Francou rodier	94,67	155.91	193.07	275.73
Frigui	145,24	210,59	312,26	363,07
Hec-Hms	114,20	156,00	208,50	249,20
SWMM	86.08	113.48	197.71	350.18
Kallel	113.55	150.9	219.71	291.92

The results obtained by the previous calculations allowed us to observe that:

- The rational method underestimates the watershed contributions.
- The Speed and Frigui methods overestimate the flows obtained for the four return periods.
- The discharges estimated by the Francou Rodier, Kallel, HEC HMS and SWMM methods are relatively close. The flow rates to be adopted will therefore be the average of these four methods, which are given in the following table:

For the project flows choice, we finally decided to take the average between the Francou Rodier, Kallel, HEC HMS and SWMM methods. These flows will be of great use to us for the current state diagnosis of the crossing structure considered (**Table 4).**

Table 4. Adopted flow for Al Khairat catchment for different return period

Return Period (ans)	**10**	**20**	**50**	**100**
Flow Rate (m^3/s)	102,12	144.07	204.74	291.76

4. Simulation and results

The under-dimensioning and/or the incorrect orientation of the crossing structures often cause overflows in the floods event (Rangari *et al.,* 2019). The goal is therefore to produce a structure that respects economic and functional constraints. To do this we will deal with two main parts, the first consists in the realization of a 1D hydraulic model of the Al Khairat wadi with the HEC RAS software, on which we will implement the existing structure to check its condition in case of floods. The second part will be centered on the determination of the suitable installation to satisfy the conditions imposed using the optimization. (Gary, 2014)

4.1 1D hydraulic simulation of the current situation

The construction of the 1D model for our case study makes it possible to determine a total of 12 cross sections with a spacing of 50 m, as well as the crossing structure location. The data relating to culvert 5(2*2) (length, slope, upstream and downstream elevations) are deduced from the topographic map of the Saouef city. (Gharbi *et al.,* 2014; Gharbi *et al.,* 2016a; Gharbi *et al.,* 2016b). The simulation results for the different return periods are shown in the below figure (**Figure 5**):

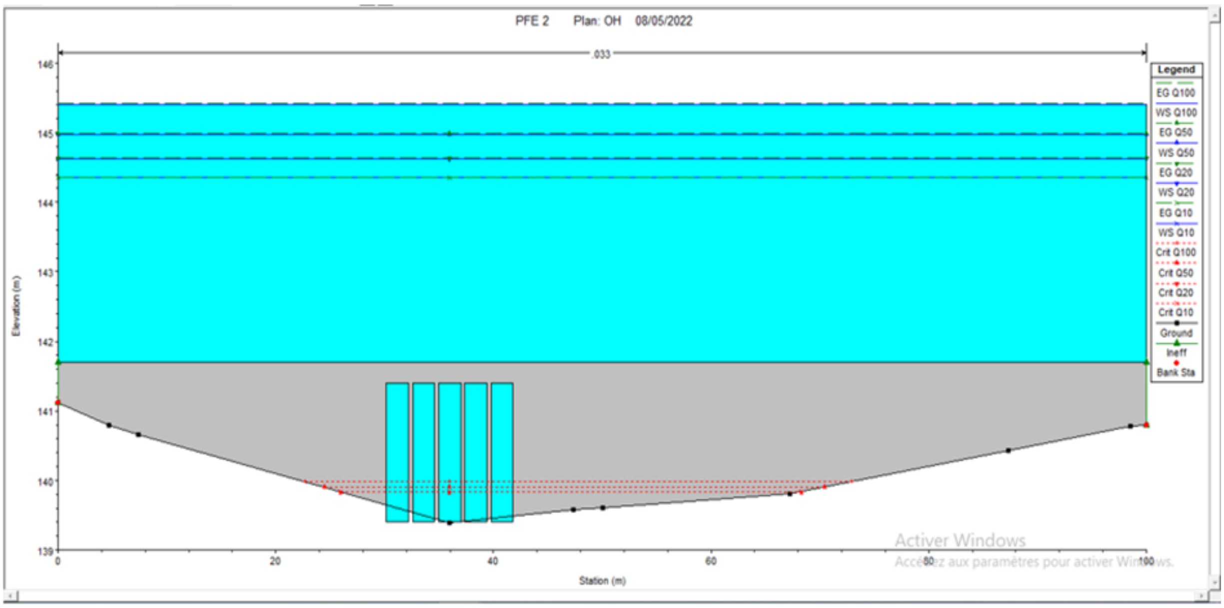

Figure 5. Culvert cross section for the different return periods

The water line simulation for the current situation, revealed that the structure is incapable of transiting the flows transited for the four return periods. The cross-sections analysis shows overflows even in the case of a low ten-year frequency. This highlights the structure submersion during flooding periods and confirms the flooding of the RL630 road.

4.2 Choice and optimization of the projected structure

The optimization of the structure resizing at the RL630 road is a problem with two objectives, conditioned by two constraints***:***

Objective 1: Ensure the passage of the design flow through the structure, minimizing the difference between the transited flow $Q_{transit}$ and the projected flow Q_{100}**.**

Objective 2: Minimize the projected variant cost**.**

Constraint 1: The height corresponding to the transited flow H $_{amont}$ must be less than or equal to the height of the project, which is defined as the difference between the level of the road and the level of the entrance to the structure*.*

Constraint 2: As mentioned in the previous section, the admissible value of the flow velocity in the structure must be between 0.5 and 4 m/s.

Table 5. Choice of the structure to be projected

OH	Q_{100} (m^3/s)	$Q_{transit}$ (m^3/s)	$\lvert Q_{tr} - Q_{100} \rvert$	H_{projet} (m)	H_{amont} (m)	V(m/s)	Prix (Dt)
Dalot 5(1.5*1.5)	291.76	95.44	196.32	5.6	11.5	8.48	2301.75
Dalot 5(2*2)	291.76	160.31	131.63	5.6	10.62	8.01	5276
Dalot 5(3*3)	291.76	288.73	3.03	5.6	7.8	5.47	15112.5
Pont 3 travées portée 10	291.76	553.03	261.29	5.6	3.62	4.82	389057.44
Pont 3 travées portée 15	291.76	829.54	537.8	5.6	2.89	4.28	449507.44
Pont 3 travées portée 20	291.76	1105.96	814.2	5.6	2.43	3.92	509957.44 ✓
Pont 4 travées portée 20	291.76	1474.64	1182.88	5.6	2.01	3.57	629927.44 ✓
Pont 4 travées portée25	291.76	1843.28	1551.52	5.6	1.73	3.33	710527.44 ✓
Pont 5 travées portée 25	291.76	2304.09	2012.33	5.6	1.49	3.09	850647.44 ✓
Pont 5 travées portée 30	291.76	2764.9	2473.14	5.6	1.32	2.91	951397.44 ✓
Pont 5 travées portée 35	291.76	3225.71	2933.95	5.6	1.19	2.13	1052147.4 ✓
Pont 5 travées portée 40	291.76	3681.17	3389.41	5.6	1.04	1.86	1152897.4 ✓

Proceeding by elimination, the results provided by the software enabled us to reject the 3 variants of the culvert as well as the two first variants of the bridge (3-span bridge span 10 and 3-span bridge span 15), since they do not respect the constraints imposed by the optimization problem (high velocity) (**Table 5**). Finally, to choose the appropriate variant, we will proceed to the calculation of the global objective function Fg (**Eqn. 1**) for each variant retained.

$$F_g \ = Min \left[\alpha \left|\frac{Q_{tr}-Q_T}{Q_{tr}}\right|^2 + (1-\alpha)(\frac{Z(x)}{Z_{MAX}})^2\right] \qquad \textbf{Eqn. 1}$$

α : weighting coefficient, between 0 and 1. Z_{MAX} : cost of the most expensive variant $Z(x)$: cost of “x” variant

The variant to be chosen must comply with the following criteria:

- Adequate return period.
- Complies with the constraints illustrated above.
- Optimal cost.
- An appreciable weighting coefficient α.

The results are summarized in Table 6. The results obtained do not justify the choice of the structure type. However, we have found that the "4-span bridge span 20" variant is the most balanced from the point of view of coefficient risk α, speed and cost. Indeed, the design of an additional span risks considerably increasing the cost, moreover the simulation with the HEC RAS software has shown that the width at the mirror corresponding to the section where we have installed the existing structure is worth 100 meters. This means that the structure total width to be projected must be less than this value (the bridge solution with 4 spans of span 25 will in this case be rejected). It is according to this logic that we opt for the variant "bridge 4 span of 20m span" for the rest of our work (**Table 6**).

Table 6. The variant to be considered

Ouvrage	$\lvert(Q_{tr}-Q_{100})/Q_{tr}\rvert^2$	$(Z_{OH}/Z_{max})^2$	V(m/s)	α	Côut éstimé (Dt)	$F_{globale}$
Pont 3 travées portée 20	0.54	0.195	3.92	0.7	509957.44	0.436
Pont 4 travées portée 20	**0.64**	**0.298**	**3.57**	**0.6**	**629927.44**	**0.5**
Pont 4 travées portée25	0.7	0.38	3.33	0.55	710527.44	0.55
Pont 5 travées portée 25	0.76	0.54	3.09	0.45	850647.44	0.64
Pont 5 travées portée 30	0.79	0.68	2.91	0.4	951397.44	0.72
Pont 5 travées portée 35	0.83	0.83	2.13	0.25	1052147.44	0.83
Pont 5 travées portée 40	0.85	1	1.86	0.15	1152897.44	0.9775

4.3 1D hydraulic simulation of development scenarios

It would then be wiser to work with a hundred-year return period, in order to guarantee the structure durability and minimize the submersion risk**.** The water line simulation for this bridge variant clearly shows their effectiveness in greatly reducing overflow for a hundred-year return period (**Figure 6**), and thus ensuring the protection of the city against flooding.

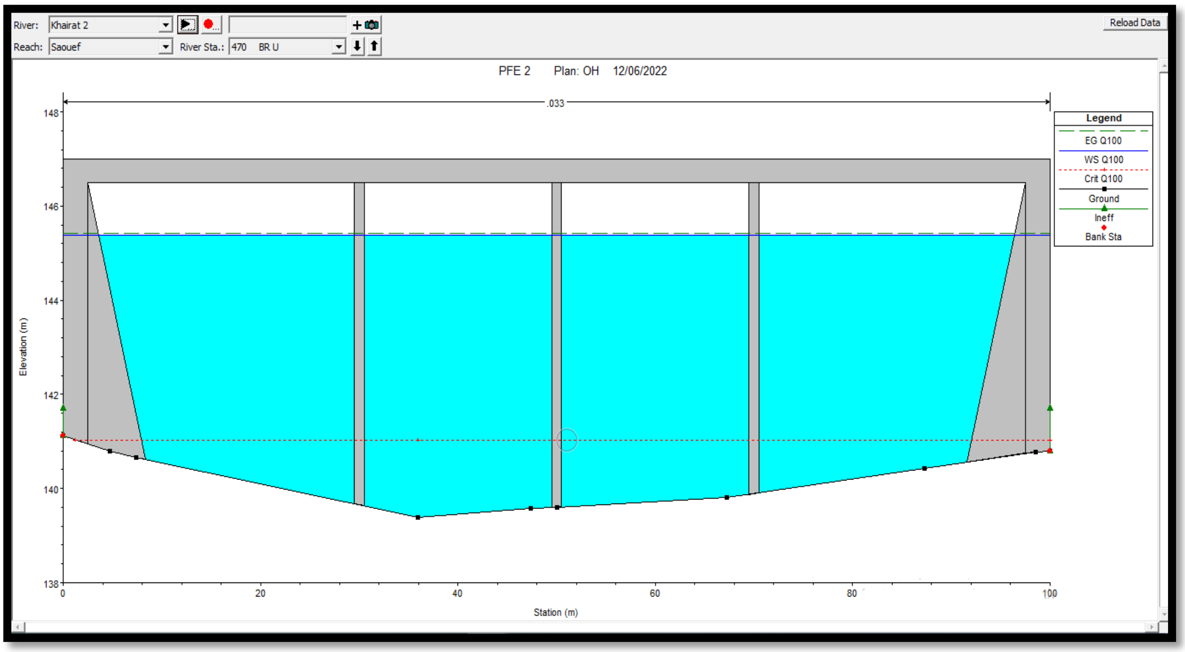

Figure 6. 4-span bridge cross section

The hydraulic study of the bridge chosen led us to design a minimum intrados height of 6 meters and to estimate a scour depth of 4 meters. Such results allowed us to pre-dimension the integrity of the structure constituent elements of our work (**Figure 7**). Our findings are in good agreement with others works aimed to adapt selected formulas for the estimation of the maximum depth of local scour in the area of the bridge pillar model (Kiraga *et al.* 2020; Silvia *et al.* 2023; Abdelhaleem *et al.* 2023; Al-Jubouri *et al.* 2023).

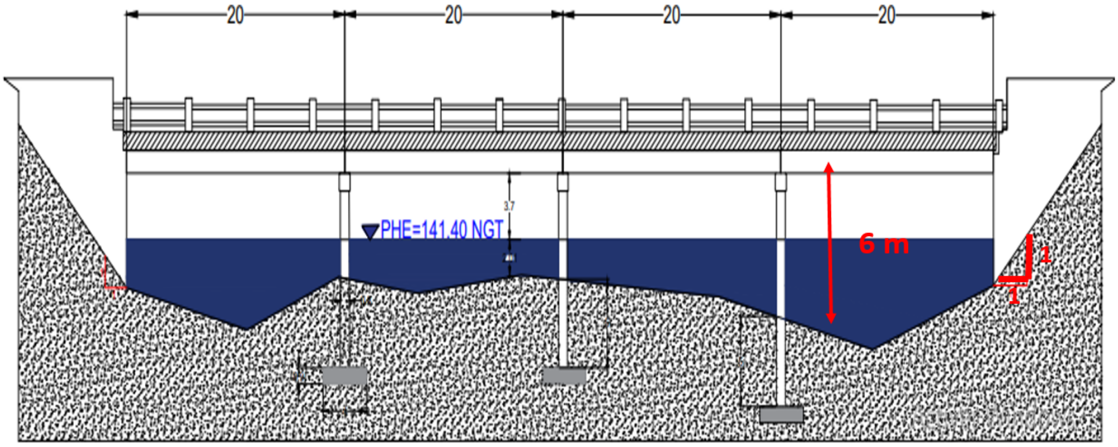

Figure 7. Cross section of the TIBA 4-span bridge to be projected

Conclusion

The study area is facing a problem which occurs mainly at Al Khairat wadi, that is the main heart of Saouaf. The wadi berges erosion on the one hand and the crossing structure insufficiency at the level of the route RL630 on the other hand, lead to submersion of the route RL6 30 and the sportif complexe located nearby.

In order to find a solution to the problem, hydrological, hydraulic studies, are tacked. The use of multicriteria decision-making and methods has enabled us to make a more informed choice that meets the criteria required by the realities of the area on all plans.

According to this study we noticed that we retain the bridge variant with prefabricated beam in reinforced concrete (TIBA) to be the most favorable solution, both in the Technical and economical aspect, ensuring the durability of the structure and minimizing the submersion risk for a hundred-year return period.

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Disclosure statement

Conflict of Interest: The authors declare that there are no conflicts of interest.

Compliance with Ethical Standards: This article does not contain any studies involving human or animal subjects.

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