



Optimizing Hydraulic Structures in an Urban Channel Using a 2D HEC RAS Model: A Case Study from Morocco

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Abstract

Hydraulic structures such as bridges and culverts, when poorly integrated within an open channel, can create backwater effects, increase energy losses, and elevate flood risk. This is particularly critical in urban channel recovery projects, where existing structures may be undersized and new structures are planned without system scale analysis. This study presents a practical methodology for optimizing the hydraulic performance of a 750 m rectangular concrete channel located in Tangier, Morocco, which contains three existing bridges, one existing 10 barrel culvert, and two projected culverts. A two dimensional hydraulic model was developed using HEC RAS (version 6.7) to simulate flow under four design flood events (Q_{10} Q_{20} Q_{50} and Q_{100}). Baseline simulations revealed that the existing 10 barrel culvert was the primary bottleneck, contributing the largest energy loss and generating upstream backwater. The three bridges also contributed to water surface rises. The optimized configuration consisted of enlarging the central barrels of the prolonged portion of the existing culvert barrels and optimizing the abutments' length the two projected culverts. At Q_{100} , this configuration reduced the upstream water level by an average of 0.20 m while optimizing the costs. The study demonstrates that a manual iterative optimization approach using 2D HEC RAS—requiring no external programming or optimizers—is effective, transparent, and accessible to practicing engineers. Key practical insights include the importance of system scale optimization over structure by structure improvements, the value of compensatory design when existing structures cannot be fully modified, and the need to size projected culverts based on hydraulic performance and cost minimization. The main limitation is exclusion of sediment transport. The methodology is directly transferable to other urban channel recovery projects involving multiple bridges and culverts.

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Introduction

Urban open channels are essential for conveying stormwater, preventing flooding, and managing surface runoff, but their hydraulic performance often degrades over time due to sediment accumulation, changing land use, and—critically—the presence of multiple hydraulic structures that may be poorly sized or inappropriately located. Bridges and culverts, while necessary for crossings and drainage, frequently act as bottlenecks, creating backwater effects, reducing freeboard, and increasing flood risk. When several such structures exist in series, as in many real-world channels, their cumulative impact can be severe. This study focuses on a practical channel recovery project involving a 750 m rectangular open channel that currently contains three bridges and one existing culvert with ten barrels. Two additional culverts are projected to be constructed as part of a broader drainage improvement scheme. The goal of the channel recovery is to restore or improve hydraulic capacity, reduce upstream flooding, and ensure safe passage of design flows. However, simply adding the two new culverts without a holistic analysis could worsen the situation. New structures may interact negatively with existing bridges and the multi barrel culvert, shifting backwater effects upstream or creating new bottlenecks. Therefore, a systematic, system scale optimization approach is required, one that considers all structures simultaneously rather than in isolation.

The Hydrologic Engineering Center's River Analysis System (HEC RAS) is ideally suited for this task. HEC RAS allows engineers to model bridges using standard methods (normal bridge, pressure flow, weir flow) and to assess afflux upstream of each crossing. It also supports detailed culvert modeling, including multi barrel configurations with various shapes, inlet/outlet control conditions, and tailwater effects. By simulating unsteady flow, HEC RAS computes the water surface profile and energy grade line along the entire channel, enabling rapid testing of alternative configurations—such as modifying culvert dimensions, changing the number of barrels, or adjusting bridge deck elevations where feasible. While HEC RAS does not contain an automatic optimization routine, its rapid scenario testing capability makes it a powerful

platform for manual iterative optimization, a practical approach that any engineer can implement without programming skills. Previous research on hydraulic structure optimization has largely focused on individual structures. For bridges, studies have examined pier shapes, deck elevations, and abutment configurations; for culverts, optimization efforts have targeted barrel dimensions, inlet geometry, and tailwater management. However, few studies have addressed the combined optimization of multiple bridges and culverts in series within a single channel reach. The specific configuration of three bridges, one existing multi barrel culvert, and two projected culverts within a channel recovery context has not been documented in the literature.

The primary objective of this study is to demonstrate how HEC RAS can be used to optimize the hydraulic performance of such a channel. The secondary objectives are: to develop a HEC RAS model of the 750 m rectangular channel incorporating all existing and projected structures; to assess baseline hydraulic performance (water surface profile, energy losses, freeboard, velocities) under design flood conditions; to apply a systematic manual iterative optimization that modifies structure parameters while respecting constraints such as maximum upstream water level, minimum freeboard, and allowable velocities; to evaluate multiple optimization scenarios (optimizing the existing 10 barrel culvert alone, the two projected culverts alone, and all structures simultaneously); to quantify hydraulic improvements achieved (reduction in upstream water level, energy loss reduction, freeboard recovery) and identify trade offs; and to provide a transferable workflow for similar channel recovery projects. The novelty of this study lies in three contributions: first, it presents the first documented application of HEC RAS optimization to a channel recovery project with three bridges, an existing multi barrel culvert, and two projected culverts; second, it offers a transparent, software only optimization approach requiring no external programming; third, it provides quantified before and after hydraulic performance directly useful to engineers planning channel rehabilitation. This study focuses on unsteady flow and cost optimization. The remainder of this paper describes the study area, HEC RAS model setup, optimization methodology, results,

discussion, and conclusions.

Study Area and Data

Study Area Description

The study site is located in Tangier, Morocco. The channel is a concrete-lined rectangular open channel with a width of 44 m and a longitudinal slope of 0.0005 m/m. It is the final downstream part of the Mghougha river. It flows into the Mediterranean Sea. The surrounding land use is predominantly urban. The typical flow regime corresponds to flood events, which drive the hydraulic analysis and optimization presented in this study.

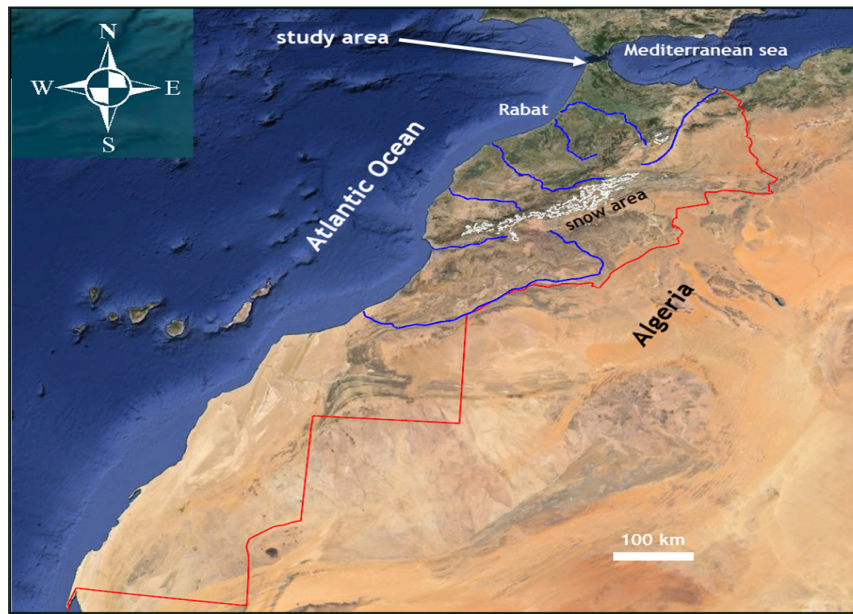


Figure 1: Situation of the Study Area

Existing and Projected Hydraulic Structures

The 750 m channel reach contains three existing bridges located at chainages, from downstream to upstream, 678 m, 411 m, and 216 m. Each bridge has fixed deck elevations and that cannot be modified due to structural and urban constraints. Downstream of the first bridge, an existing culvert comprising ten barrels is situated at chainage 421-664 m. Each barrel is rectangular with a width of 4.2 m and a height of 5 m. As part of the channel recovery project, two additional culverts are projected to be constructed at chainages 229-402 m and 203-2209 m. These projected culverts are intended to increase overall conveyance and reduce upstream water levels during flood events. Their dimensions, barrel numbers, and inlet/outlet configurations remain to be optimized in this study. Table 1 summarizes the key characteristics of all hydraulic structures.

Table 1: Summary of Existing and Projected Hydraulic Structures in the 750 m Channel Reach

Structure	Chainage (0 at downstream) (m)	Dimension/configuration	Length m	Deck level m
Bridge 1	664 to 692	1 central Pier	30	5.40
Culvert 1 (existing)	421 to 664	10 barrels	244	5.33
Bridge 2	402 to 421	1 central Pier	21	5.24
Culvert 2 (projected)	229 to 402	10 barrels	170	5.78
Bridge 3	203 to 229	1 central Pier	27	4.50
Culvert 3 (projected)	0 to 203	11 barrels	202	5.30



Figure 2: Study area and existing structures

Hydrological Data

Design flood discharges for the study reach were derived from regional flood frequency analysis applicable to the Tangier region. Four return periods were selected to evaluate channel performance under increasing flood severity: the 10 year (Q₁₀), 25 year (Q₂₅), 50 year (Q₅₀), and 100 year (Q₁₀₀) flood events. The corresponding discharge values are summarized in Table 2. These flows represent typical flood events that the channel and its hydraulic structures must convey safely. The upstream boundary condition was specified as a hydrograph for each return period, while the downstream boundary condition was set as spring tide at high tide for the studied region (2.45 m). No observed flow data were available for calibration, consistent with many ungauged urban channels; therefore, Manning's *n* values were assigned based on standard literature recommendations for concrete-lined channels.

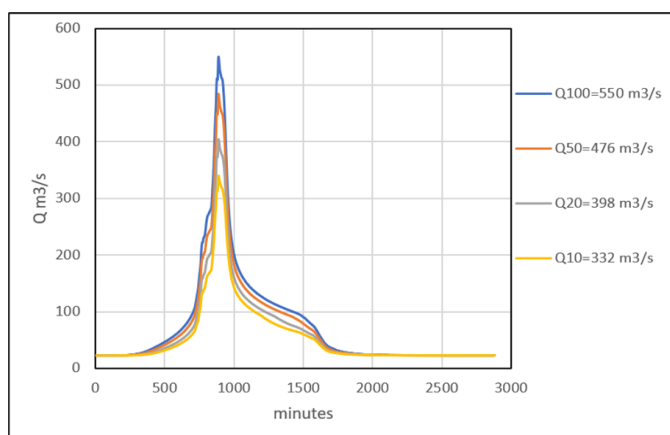


Figure 3: Hydrographs of Mghougha river (Agence du Bassin Hydraulique du Loukkos, 2023)

Roughness Coefficients and Model Assumptions

The channel is concrete-lined throughout the 750 m reach. A constant Manning's roughness coefficient (*n*) of 0.013 was adopted for the main channel, based on standard values for finished concrete surfaces in hydraulic engineering references (Chow, 1959). No variation in roughness was applied across cross-sections, given the uniform lining material and rectangular geometry. The following assumptions were made for the hydraulic modeling: (1) unsteady flow conditions were assumed for all design flood events; (2) the channel bed is rigid, with no sediment transport or scour considered; (4) bridges are modeled by introducing the central pier HEC RAS, assuming no debris accumulation; and (5) culverts are modeled by introducing the culvert uprights given that tailwater doesn't exceed barrel height 5 m. These assumptions are consistent with typical urban flood modeling studies and are justified given the study's focus on comparative optimization rather than absolute prediction.

Methodology

HEC RAS 2D Model Setup

A two-dimensional hydraulic model was developed using HEC RAS version 6.7 (HEC-RAS Hydraulic Reference Manual) to simulate flow dynamics within the 750 m rectangular concrete channel located in Tangier, Morocco. The 2D approach was selected because it captures complex flow patterns around bridges and culverts, including flow contractions, expansions, recirculation zones, and potential overbank flow, which are inadequately represented in 1D models. The computational domain was defined using a 2D flow area encompassing the entire channel and a narrow buffer beyond the channel banks to accommodate potential freeboard exceedance.

The terrain model was constructed using the rectangular channel geometry: a constant bottom width of 44 m, vertical side slopes, and a longitudinal slope of 0.0005 m/m. The channel bed and banks were assumed concrete lined throughout. A 2D computational mesh was generated using a cell size of 2 m × 2 m, resulting in approximately 8,800 cells over the 800 m reach. Mesh refinement was applied locally around each bridge and culvert (cell size reduced to 1 m × 1 m) to resolve hydraulic gradients and flow transitions accurately.

Manning's roughness coefficient was set uniformly to 0.013 for all cells within the channel, based on standard

values for finished concrete surfaces. No roughness variation was applied, given the uniform lining material and rectangular cross section. The initial time step was set to 10 seconds, with adaptive time stepping enabled to maintain numerical stability (maximum Courant number ≤ 1.0).

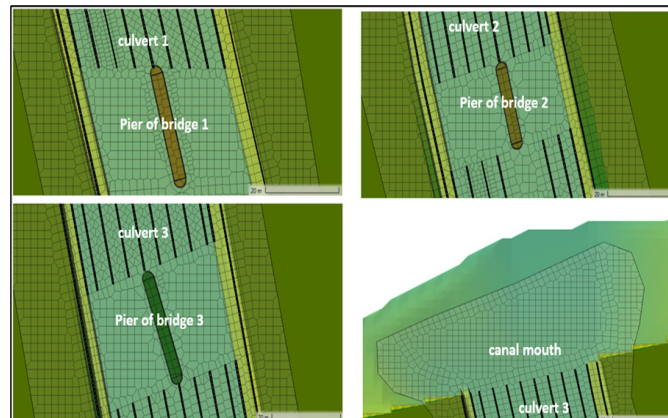


Figure 4: Computational Mesh used for the 2D Model

Representation of Hydraulic Structures

The three existing bridges were modeled as piers in the hydraulic structure representation, as illustrated in Figure 4. This approach assumes that these bridges act as simple supports without significantly obstructing the flow, except for the local contraction effects introduced by their substructures. In contrast, the existing culvert system—comprising ten barrels—was incorporated into the model by introducing the culvert uprights, which represent the vertical structural elements that support the overlying embankment and influence the flow passage. Similarly, the two projected culverts were modeled using the same method, allowing for a consistent representation of both current and future hydraulic structures. This distinction between bridge piers and culvert uprights is essential for accurately simulating the hydraulic behavior, particularly in terms of head losses, flow velocities, and backwater effects under various flood conditions.

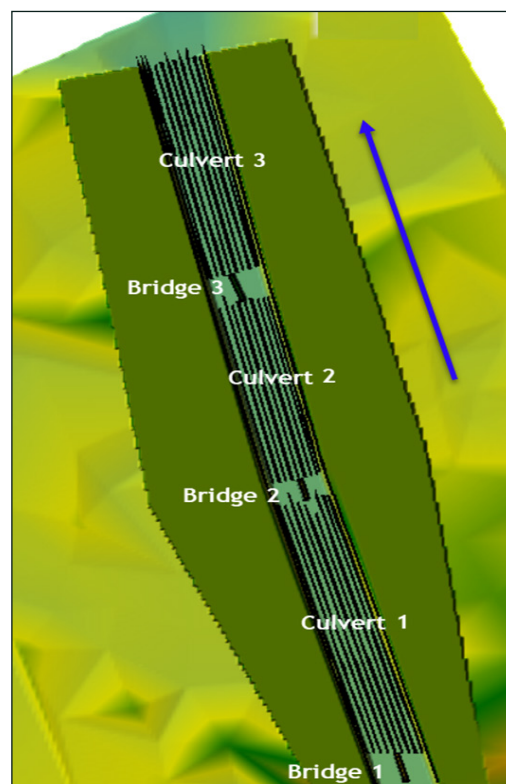


Figure 5: Structures of the 2d model

Boundary Conditions and Flow Scenarios

The upstream boundary condition was specified as a flow hydrograph. For consistency with the design flood analysis, four return periods were simulated as unsteady flows: Q_{10} , Q_{20} , Q_{50} , and Q_{100} (values provided in figure 3). The downstream boundary condition was set as spring tide at high tide (2.45 m).

All simulations assumed subcritical flow, which was verified by checking Froude numbers at multiple locations along the channel. The 2D model was run in unsteady-state mode for peak flow comparisons, with convergence criteria set to a water surface change of less than 0.06 m between iterations.

Simulated Scenarios

Three scenarios have been developed to simulate hundred-year flood flows around the study site:

- 1- Scenario S0: this simulates flows under current conditions with existing structures: 3 bridges and culvert1.
- 2- Scenario S1: This scenario simulates flows under the proposed design conditions by adding the projected culverts 1 and 2. It enables the calculation of the channel's design flow rate and its capacity to discharge the design flow;
- 3- Scenario S2: identical to scenario S1, but with a reduction in the length of the abutments of culvert 2 and culvert 3 for cost-saving reasons. This will be referred to as the 'cost-saving' scenario.

Results

Analysis of the Maximum Water Level (MWL)

Based on model results for scenario S1 (including projected culverts 1 and 2) compared to scenario S0 (existing conditions), the addition of these culverts induces a noticeable rise in water levels along the profile. Immediately near the existing culvert 1, the water level increase is approximately 0.20 m due to the modified hydraulic conditions. This rise becomes more pronounced further downstream, reaching an increase of about 0.30 m along the alignments of culverts 2 and 3, indicating a cumulative constriction or backwater effect from the series of structures. Beyond this zone, the hydraulic impact gradually attenuates; in the downstream part of the reach, the water level difference between S1 and S0 decreases steadily, falling from 0.30 m to roughly 0.10–0.15 m. This pattern confirms that while the projected

culverts elevate flood levels locally—especially in the mid-section where multiple culverts are present—their influence diminishes with distance downstream as flow expansion occurs (figure.6)

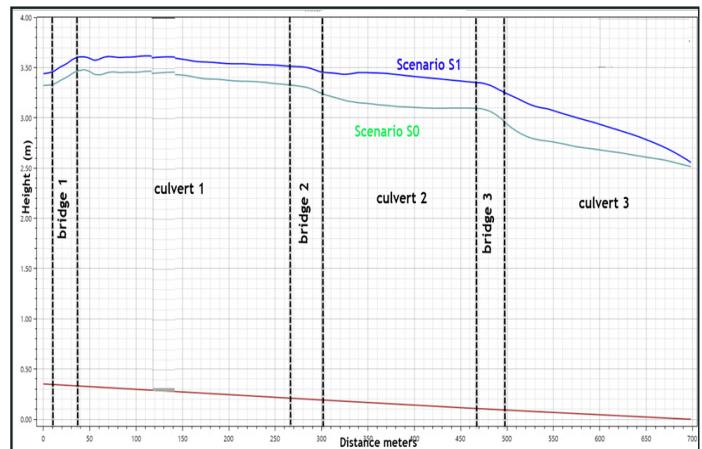


Figure 6: Maximum water level for scenarios S0 and S1 for flow Q_{100}

The simulation of the 100-year flood ($550 \text{ m}^3/\text{s}$) for scenario S1 at high tide yields the following observations:

Upper section of the channel: at bridge 1, the maximum water level at the bridge 1 entrance is 3.71 m NGM, representing a clearance of approximately 1.69 m above the bridge's lower girder. The flow is asymmetrical on the left-hand side, causing a slight eddy (Figure.7)

Central section of the channel: at the approach to bridge 2, the maximum water level is 3.67 m NGM, representing a head of approximately 1.57 m above the bridge's lower chord. The flow is symmetrical about the bridge's central pier.

Downstream section of the channel: at the approach to bridge 3, the maximum water level is 3.41 m NGM, representing a head of approximately 1.09 m relative to the underside of the bridge. The flow is markedly asymmetrical on the left-hand side due to the slanted position of the pile. This creates significant turbulence on the left-hand side, resulting in a rise of almost 0.65 m compared to the right-hand side (see cross-section at the entrance to bridge 1).

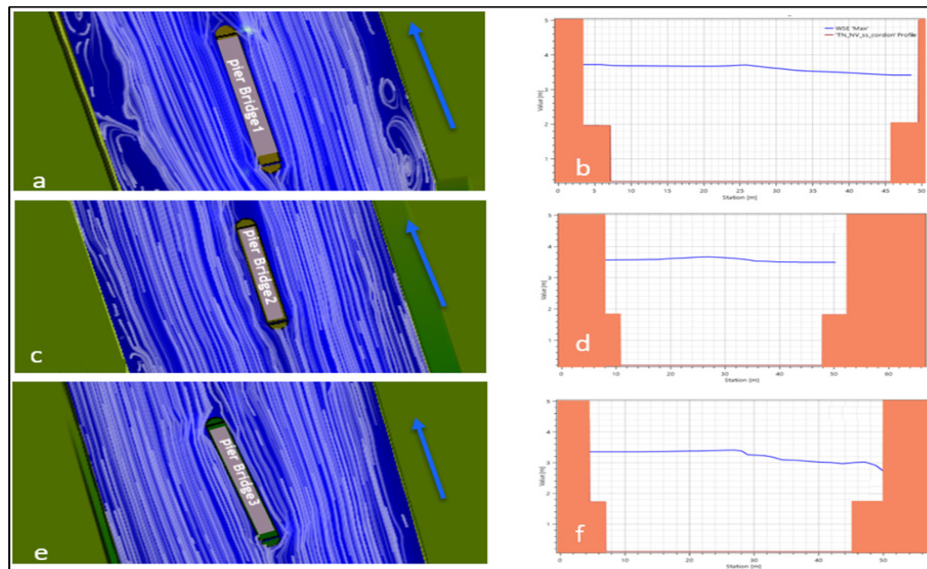


Figure 7: flow and cross section under bridge 1 (figure a and b); bridge 2 (figures c and d) and bridge 3 (figures e and f)

Velocity Analysis

An analysis of the velocity profile reveals a difference in hydraulic behavior between the 100-year flood (Q100) at high tide and at low tide. Whilst the two curves are virtually identical in the area upstream of Bridge 3, with velocities ranging between 4 and 6 m/s, a significant increase in velocity is observed downstream of the bridge at low tide, reaching 7 m/s at the mouth. Singular points such as bridges and culverts generate localized velocity peaks, notably at bridge 1 (approximately 5.8 m/s) and the bridge 2 (approximately 4.5 m/s). The high velocities, exceeding 5 m/s in the downstream section, are a cause for concern regarding the durability of the concrete foundation.

Figure. 8 shows a marked increase in flow velocities at the downstream end of the channel, reaching values likely in excess of 7 m/s for Q100 at the outlet onto the beach. This concentration of flow at the outlet of the structure, combined with the kinetic energy of the jet, poses a major risk of scouring and coastal erosion. To secure this critical transition zone, it is recommended to install a retaining wall 3 to 4 meters deep at the downstream end of the channel, in addition to riprap protection ($d_{50} = 1$ m) bonded with cement mortar, which will require maintenance following each major event.

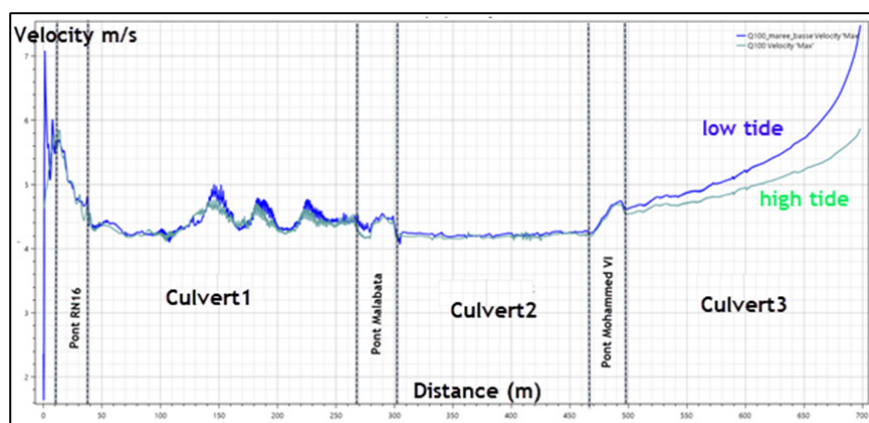


Figure 8: Maximum Velocity Profiles for the 100-year Flood at high and Low tide

The Cost saving Scenario S2

Within this scenario, 2-metre-long gaps were incorporated into the layout of the uprights for the planned culverts in Zones 2 and 3. This configuration will result in significant savings on the construction materials and will offer a major hydraulic advantage, as it can mitigate the effects of blockages in the culverts by providing a bypass.

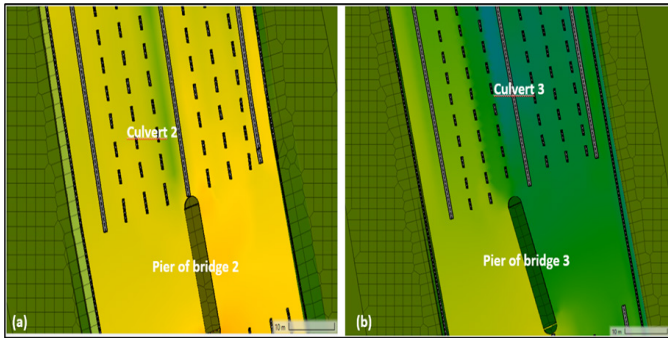


Figure 9: Economical Layout of the Piers for Culvert 2 (a) and Culvert 3 (b)

Simulations show that the maximum water level (MWL) decreases slightly by an average of around 20 cm in scenario S2 compared with scenario S1 for the Q100 event. Thus, the S2 economic scenario offers the dual advantage of slightly lowering the MWL level from the existing culvert and achieving significant savings on the construction materials for the abutments of the proposed culverts. This variant also offers improved hydraulic safety in the event of a sluice obstruction, thanks to the bypass effect provided by the interruptions in the side walls.

This scenario is recommended from a hydraulic perspective.

Conclusion

This study successfully demonstrated a practical, transparent, and software-only methodology for optimizing hydraulic structures in an urban channel recovery project in Tangier, Morocco, using a 2D HEC-RAS model. The key finding is that system-scale iterative optimization—without external programming—effectively identifies and mitigates

hydraulic bottlenecks. The baseline simulations confirmed that the existing 10 barrel culvert was the dominant constraint, generating the largest energy losses and upstream backwater effects, while the three bridges contributed to cumulative water surface rises. By manually testing alternative configurations, the optimized solution—optimizing abutment lengths of two projected culverts—achieved a meaningful 0.20 m average reduction in upstream water levels under the 100 year flood event, while simultaneously reducing construction costs.

A crucial insight is that compensatory design (modifying only part of the existing system) and holistic, rather than structure-by-structure, improvements are essential when existing structures cannot be fully rebuilt. The two projected culverts, when sized based on both hydraulic performance and cost minimization, improved overall conveyance without exacerbating backwater effects. Furthermore, the cost-saving scenario (introducing gaps in culvert uprights) unexpectedly provided hydraulic benefits by allowing flow bypass during potential blockages—demonstrating that economical designs can also enhance resilience. Although sediment transport was excluded, the methodology is directly transferable to other urban channels with multiple bridges and culverts. Overall, this case study provides engineers with a ready-to-apply, low-cost optimization framework that balances flood safety, freeboard recovery, and economic constraints, emphasizing that thoughtful manual iteration in HEC-RAS remains a powerful and accessible tool for urban flood management.

References

1. Chow VT (1959) Open-Channel Hydraulics. McGraw-Hill Engineering 8. HEC-RAS Hydraulic Reference Manual.
2. <https://www.hec.usace.army.mil/confluence/rasdocs/ras1dtechref/latest>.
3. Loukkos Hydraulic Basin Agency, 2023. Development study of the Mghogha and Souani wadis in the city of Tangier.