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Integrated Hydrologic–Hydraulic Framework for Culvert Siting at Road–Waterway Intersections in Ungauged Basins

Nawbahar F. Mustafa
University of Sulaimani
Anwer H. Dwood
Koya University
Luqman S. Othman
University of Halabja
Kawa Zaidan Abdulrahman
University of Sulaimani
Nadhir Al-Ansari
nadhir.alansari@ltu.se
Luleå University of Technology

Article

Keywords: Culvert sitting, Ungauged basins, HEC-RAS 2D, Flood mapping, infrastructure resilience

Posted Date: August 3rd, 2026

DOI: https://doi.org/10.21203/rs.3.rs-10469056/v1

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Additional Declarations: No competing interests reported.

Abstract

Road culvert failures are a major cause of infrastructure damage in developing regions. This study presents an integrated hydrologic–hydraulic framework for culvert siting and sizing at road–waterway intersections, using New Zakho City, Kurdistan Region, Iraq, as a case study. The methodology combines the Watershed Modeling System (WMS) for watershed delineation and rainfall–runoff simulation with two-dimensional HEC-RAS 2D for flood inundation modeling. Design storms were derived from maximum 24-hour rainfall records using frequency analysis, producing depths of 104 mm and 114 mm for the 50- and 100-year return periods, respectively. Four road–stream crossings were identified, draining watershed areas of 0.99–15.71 km² with design discharges of 13.1–111 m³/s. Mesh sensitivity analysis identified a 10 m resolution as optimal, balancing computational efficiency and simulation accuracy. Flood inundation results showed significant variations in depth and velocity between return periods, providing the basis for culvert sizing and scour protection. Box culverts ranging from 2.13 × 1.22 m to 3.66 × 3.05 m satisfied the allowable headwater-to-rise ratio of 1.5 for the 100-year event, while outlet velocities exceeding 3.0 m/s required riprap apron protection. The proposed open-data framework provides a transferable, cost-effective approach for culvert design in ungauged, data-scarce watersheds.

1. Introduction

Road transportation networks are increasingly vulnerable to flood-related damage, particularly across rapidly urbanizing developing regions. A fundamental component of roadway construction is the hydraulic design of drainage structures, most notably culverts at intersections with natural waterways. These engineered elements must safely convey stormwater beneath roadways, preventing washouts, ensuring traffic safety, and preserving long-term structural integrity; their cumulative cost can represent a significant fraction of total highway project expenditure[1]. Inadequate hydraulic design invariably leads to premature failure, with documented consequences including catastrophic washouts, elevated maintenance costs, and loss of life[2].

Recent studies consistently demonstrate that integrating GIS, hydrologic, and hydraulic models significantly improve flood prediction and culvert design accuracy in both humid and arid environments. Applications employing combinations of WMS, HEC-HMS, and HEC-RAS have shown reliable performance in estimating runoff, inundation depth, and hydraulic characteristics, even where hydrological observations are scarce.

Accurate estimation of the design discharges the peak flow rate a culvert must accommodate for a specified return period is the cornerstone of effective culvert design. This necessitates integrated hydrologic and hydraulic analysis within the contributing catchment to characterize the flow regime[3]. Whereas such analyses were historically conducted through manual calculations that often-propagated errors, the emergence of computational hydraulic models and Geographic Information Systems (GIS) has substantially improved design accuracy and efficiency[4]. The return period (also termed the recurrence interval) of a design rainfall event is a critical statistical concept that quantifies the average

time between hydrological extremes, and its selection balances infrastructure construction and maintenance costs against expected flood damage[5, 6].

Flood mapping provides valuable information on flow characteristics and peak discharge; however, this exercise becomes particularly challenging for small or ungauged catchments where the absence of historical discharge records precludes conventional model calibration[7, 8]. The application of GIS has substantially facilitated the preparation of spatial data for hydrologic and hydraulic analyses, and several studies have demonstrated its utility in culvert design[2–4, 9]. The Hydrologic Engineering Center River Analysis System (HEC-RAS) is among the most widely validated hydraulic modelling platforms, capable of simulating one-dimensional and two-dimensional unsteady flow in floodplains with demonstrable skill[10]. Sensitivity analyses have established that flood area and volume are 1.5 to 2 times more sensitive to variations in channel roughness than to cross-section spacing[11], underscoring the importance of careful parameter specification. Kang[12] further demonstrated that basing culvert design on the critical storm duration rather than a fixed-duration design storm yields more hydraulically appropriate configurations.

Recent integrated modelling studies have reinforced the value of coupling GIS, hydrologic, and hydraulic tools. Applications combining GIS, HEC-HMS, and HEC-RAS in forested watersheds have documented that even moderate 25-year events can produce average inundation depths of 1.8–2.0 m with velocities reaching 0.7 m/s[13, 14]. In arid regions typified by flash floods and sparse data, integrated WMS–HEC-RAS approaches have successfully addressed challenges posed by rapid urbanization and complex topography[15, 16]. Al-Zahrani[17] demonstrated that WMS functions as an efficient GIS-based interface for automated watershed delineation, parameter extraction, and input preparation for HEC-HMS and HEC-RAS, materially improving simulation accuracy in data-scarce environments. Hussein[1] subsequently employed WMS to simulate the extreme rainfall event in the UAE, confirming the platform's value for flood prediction in arid, rapidly urbanizing settings. Comparative analyses using HEC-RAS, SWMM, and national design handbooks have further concluded that GIS-based hydraulic models are preferable for assessing environmental impacts near residential areas[3].

Despite these methodological advances, the Kurdistan Region of Iraq remains conspicuously underserved in the literature. No study has yet applied a fully coupled WMS– HEC-RAS 2D Culvert Master framework to culvert siting in the region, and design engineers continue to rely on simplified empirical methods that are ill-suited to the semi-arid, topographically complex terrain and the general absence of gauged discharge records[9].

The aim of This study addresses that gap through a case study in New Zakho City, where a new highway under construction intersects multiple natural drainage channels. The specific objectives are: (i) to automate watershed delineation and design flow estimation using WMS 11.1; (ii) to simulate flood inundation extents and hydraulic characteristics for 50 and 100-year return periods using HEC-RAS 2D; (iii) to conduct a systematic mesh sensitivity analysis to optimize computational resolution; and (iv) to

size and verify box culvert configurations using Bentley Culvert Master, with full hydraulic consistency across all three modelling stages. To the best of the authors' knowledge, this represents the first integrated application of these three coupled tools specifically for culvert siting in an ungauged basin within the Kurdistan Region of Iraq. The framework relies exclusively on freely available geospatial datasets and is therefore directly transferable to other data-scarce regions worldwide.

2. Materials and Methods

2.1 Study Area

New Zakho City is in the Duhok Governorate of the Kurdistan Region of Iraq, between latitudes 37°00'–37°22'N and longitudes 42°22'–43°11'E (Fig. 1). The study area covers approximately 1,454 km² and ranges in elevation from about 350 to 2,250 m above sea level, resulting in a diverse landscape that includes mountainous terrain, rolling hills, alluvial plains, and deeply incised valleys. This topographic variability strongly influences watershed characteristics, surface runoff generation, and drainage patterns. Owing to its strategic location at the tri-border junction of Iraq, Turkey, and Syria, Zakho serves as an important agricultural, commercial, and transportation hub within the region[9].

The climate is characterized as semi-arid, with rainfall concentrated primarily during the winter months, leading to highly variable runoff responses and occasional flash floods. The combination of complex topography, ephemeral drainage channels, and limited hydrological monitoring presents significant challenges for hydraulic infrastructure design. The selected study site is located along a newly constructed highway in the southern part of New Zakho City, developed as part of the Zakho Development Master Plan. The highway intersects several natural drainage channels, necessitating the design of appropriately sized culverts to ensure safe stormwater conveyance, minimize flood risk, and maintain long-term road stability.

Figure 1 Location map of the study area.

2.2 Data Collection

A comprehensive topographic dataset for the catchment was assembled from two complementary sources. A detailed topographic survey of the development footprint was conducted using differential GPS technology; the surveyed point cloud was interpolated into a high-resolution digital elevation model (DEM) and subsequently merged with the freely available DEM with 30-m precision of the broader contributing catchment to produce a seamless terrain model spanning the full modelling domain. Land-use and land-cover maps, stream network data, and soil classification layers specific to the Zakho area were integrated into the modelling framework. Rainfall data comprising maximum 24-hour depths were compiled from the Zakho meteorological station for a 22-year record (1999–2021). Historical flood discharge data are unavailable in the study area, precluding conventional model calibration; consequently, physically based default parameters from the literature and regional analogues were adopted throughout.

The integrated framework adopts a three-stage sequential workflow in which outputs from each stage serve as primary boundary inputs to the subsequent stage (Fig. 2). Stage 1 (watershed-scale analysis) employs WMS 11.1 for DEM-based watershed delineation, hydrologic parameter extraction, and design storm derivation. Stage 2 (reach-scale hydraulic simulation) employs HEC-RAS 2D (v6.4.1) to simulate rainfall–runoff generation and flood propagation across the 10 m finite-element mesh, yielding spatially distributed water surface elevations, inundation extents, and flow velocities for each return period. Stage 3 (structure-scale design) employs Bentley Culvert Master to size and verify box culvert configurations, receiving 100-year peak discharges and tailwater levels directly from Stage 2. This tight coupling in contrast to the loosely coupled or single tool approaches commonly reported in the literature[2, 3, 18] minimizes the inter-stage propagation of modelling error and ensures hydraulic consistency across all three design phases.

Figure 2 Methodology flowchart illustrates the three-stage integrated framework.

2.3 Pre-processing and Mesh Generation

All spatial datasets were projected to the Universal Transverse Mercator (UTM) Zone 38N coordinate system to ensure spatial consistency. The merged DEM was subjected to quality-control procedures to identify and correct elevation anomalies, void-fill artefacts, and terrain discontinuities at the survey-SRTM boundary; previous studies have demonstrated that such corrections materially improve HEC-RAS simulation performance[19]. The corrected DEM was imported into WMS for automated flow-direction analysis, stream network extraction, and sub-catchment delineation.

The modelling domain was discretized into a 2D finite-element mesh using HEC-RAS's terrain-following mesh generation utility. A mesh sensitivity analysis was conducted at three resolutions 20 m, 15 m, and 10 m by comparing predicted flood extents, peak water surface elevations, and computational runtimes. The 10 m mesh was selected as optimal, consistent with published guidance that mesh refinements below approximately 10 m yield diminishing returns in accuracy relative to the substantial increase in computational cost[20]. The 10 m mesh also aligned with the effective resolution of the terrain data, ensuring that mesh refinement did not introduce spurious numerical artefacts from sub-grid terrain variability.

2.4 Hydrologic Parameter Definition

Infiltration and runoff generation were parameterized using the SCS Curve Number (CN) method[6], with CN values assigned on the basis of the classified land-cover map and the corresponding NRCS hydrologic soil group. Manning's roughness coefficient for overland flow was set to n = 0.05, a value widely adopted for mixed floodplain conditions and consistent with recommendations in comparable regional studies[21]. Because historical flood records are unavailable for the study area, formal calibration could not be performed; the sensitivity of key parameters was therefore assessed by reference to published uncertainty ranges, and the adopted values are within those bounds.

2.5 Hydraulic Modeling

Two-dimensional flood simulations were executed in HEC-RAS using rainfall-on-grid as the primary inflow boundary condition, enabling direct simulation of catchment scale rainfall–runoff generation and flood propagation within a single modelling environment. Both the diffusion wave (DW) and full shallow water equation (SWE) solvers were initially applied to the same design event; comparison of the resultant flood extents and hydraulic characteristics showed negligible differences (< 2% in peak water surface elevation), consistent with published findings[4]. The computationally efficient DW solver was therefore adopted for all production runs.

A normal depth boundary condition was specified at each downstream outlet to represent free outflow conditions. Simulations employed a dynamic time stepping scheme with an initial time of 4 s and allowable Courant numbers of 0.4–0.9 to ensure numerical stability across a range of flow conditions. All simulations were run for a 24-hour storm duration plus a 6-hour recession period to capture peak hydraulic conditions and the subsequent drawdown phase.

A recognized limitation of 2D grid-based flood modelling is the occurrence of localized 'cupping' artefacts in topographically complex areas, wherein water surface interpolation between adjacent computational cells produces isolated inundated zones that are not hydraulically connected to the main flow network. To mitigate erroneous hazard attribution, flood risk interpretation in this study was restricted to the principal drainage channels and physically connected inundation zones, with isolated shallow depressions explicitly excluded from the design analysis.

2.6 Design Input and criteria

Design inputs supplied to Culvert Master for each of the four road stream crossings comprised: (i) 100-year peak discharge from the HEC-RAS 2D simulation; (ii) tailwater depth extracted from the 2D water-surface profile at the downstream barrel face; (iii) roadway embankment geometry (fill height and crest elevation from highway design drawings); (iv) culvert barrel length estimated from the embankment cross section width; and (v) natural streambed slope measured from the corrected 10 m DEM. Box culvert configurations were selected for all four sites due to their structural efficiency and suitability for the large design discharges encountered. Interior Manning's n was set to 0.013 for smooth precast concrete[21], and a square-edged headwall with wingwalls at 30° to the barrel axis was adopted as the standard inlet configuration (inlet loss coefficient K = 0.5)[22, 23].

Two primary performance criteria governed iterative sizing: (i) the headwater to rise ratio (HW/D) must not exceed 1.5, consistent with FHWA Hydraulic Design Series No. 5 guidelines[22]; and (ii) mean outlet velocity must not exceed 3.0 m/s for unprotected native soil channels, with Class II or Class III riprap apron protection specified where this threshold was exceeded. Culvert Master outputs were independently verified, achieving agreement within 4% at all four crossings. Tailwater levels used as downstream boundary conditions were extracted directly from the HEC-RAS 2D water surface profiles for the 100-year event, ensuring full hydraulic consistency across all three modelling stages.

3. Results and Discussion

3.1 Rainfall Frequency Analysis and Design Hyetographs

Table 1 presents the 22-year annual maximum 24-hour rainfall record for Zakho station, spanning water years 1999–2000 through 2020–2021. The series has a mean of 53 mm and a standard deviation of 19 mm. The single highest recorded depth (106 mm in 2010–2011) is 2.8 standard deviations above the mean, indicating a rare but physically plausible extreme. Frequency analysis yielded 24-hour design depths of 104 mm (50-year) and 114 mm (100-year). The relatively modest difference between these two return-period depths (Δ = 10 mm, ~ 10%) reflects the moderate coefficient of variation (CV = 0.36) of the annual maxima series, which implies a comparatively flat frequency curve in the upper tail.

Table 1
Annual maximum 24-hour rainfall depths at Zakho meteorological station (1999–2021).

No.	Year	Max 24-hr Rainfall (mm)	No.	Year	Max 24-hr Rainfall (mm)
1	1999–2000	70	12	2010–2011	106
2	2000–2001	46	13	2011–2012	36
3	2001–2002	71	14	2012–2013	56
4	2002–2003	55	15	2013–2014	93
5	2003–2004	42	16	2014–2015	46
6	2004–2005	41	17	2015–2016	58
7	2005–2006	44	18	2016–2017	36
8	2006–2007	53	19	2017–2018	30
9	2007–2008	29	20	2018–2019	62
10	2008–2009	47	21	2019–2020	66
11	2009–2010	49	22	2020–2021	34

3.2 Watershed Delineation and Hydrological Characterization

Automated watershed delineation in WMS, driven by the corrected 10 m DEM, identified four sub-catchments contributing to the four road–stream intersections within the 27 km² study domain (Figs. 3–7). The delineated watershed areas are 0.99, 8.71, 11.83, and 15.71 km², respectively. The pronounced range a factor of nearly sixteen between the smallest and largest catchments reflects the complex, converging drainage network of the area and has direct implications for culvert sizing.

Curve Number values, assigned based on local soil classification and land-cover data, vary across sub-catchments in response to differences in soil hydraulic conductivity and the degree of imperviousness. These spatial variations, rather than watershed area alone, drive the disproportionate specific runoff rates observed in the larger catchments.

Figure 3 Locations of natural streams with the proposed road.

Figure 4 The boundaries of these catchments.

Figure 5 Discretization and delineation of the catchment areas.

Figure 6 Delineated watersheds and streams inside the study area.

Figure 7 Boundary and CN of the watersheds.

3.3 Design Discharges

Table 2 presents the resulting 100-year and 50-year peak discharges for each sub-catchment. The 100-year peak flows range from 13.1 m^3/s (Watershed 1, 0.99 km^2) to 111 m^3/s (Watershed 4, 15.71 km^2). The 50-to-100-year discharge ratios are 0.88, 0.87, 0.87, and 0.86 for Watersheds 1 through 4, respectively values that converge with increasing catchment area, consistent with the greater attenuation of runoff extremes in larger basins. The increasing trend in discharge with larger watershed areas and longer return periods underscores the importance of incorporating hydrological variability into culvert design to ensure long term flood resilience and infrastructure safety.

Table 2
Peak design discharges for the four sub-catchments.

No.	Watershed Area (km^2)	Intersection ID	Q_{50} (m^3/s)	Q_{100} (m^3/s)
1	0.99	W1	11.5	13.1
2	8.71	W2	72.5	83.2
3	11.83	W3	84.1	96.5
4	15.71	W4	96.0	111.0

3.4 Flood Inundation Mapping

The 2D HEC-RAS simulations produced spatially distributed water surface elevation, inundation extent, flow depth, and flow velocity grids for both the 50 and 100-year return periods (Figs. 8 and 9). The flood inundation maps confirm that the deepest inundation occurs along the primary channel reaches and low lying flow-convergence zones, while peripheral areas exhibit shallow ponding with limited hydraulic hazard. Inundation depths and velocities increase systematically with watershed size and return period, underscoring the imperative to individually characterize each sub-catchment rather than apply area-scaling relationships.

The flow velocity distributions indicate that the highest velocities are concentrated in the main channel reaches particularly at hydraulically contracted cross sections where confined flow accelerates. Floodplain areas exhibit characteristically low velocities, consistent with the dominance of overland flow and temporary ponding processes. This spatial velocity heterogeneity has direct implications for culvert design: high velocity zones at culvert outlets impose erosion and scour risks on the downstream channel, whereas low velocity zones are primarily associated with sediment deposition and potential blockage. These results underscore the importance of accounting for both velocity and discharge when determining culvert dimensions and protective measures such as scour control.

Furthermore, localized ‘cupping’ artefacts were identified in several topographically complex cells. These were systematically excluded from the design analysis; the final flood hazard interpretation was restricted to hydraulically connected inundation zones confirmed by visual inspection of the computed water surface profiles.

Figure 8 The results of flow depth in the watershed.

Figure 9 Distribution of flow velocity in the watershed.

3.5 Culvert Hydraulic Design

Table 3 and Fig. 10 summaries the final culvert dimensions and hydraulic performance metrics for all four road stream crossings. Box culvert configurations were selected at all sites owing to their structural efficiency, favorable hydraulic coefficients, and suitability for the large design discharges encountered.

Table 3
Final culvert design dimensions and hydraulic performance summary (100-year event).

No.	Q_{100} (m^3/s)	Box Cell (m×m)	Cells	HW/D	HW (m)	V_out (m/s)	Control	Protection
1	13.1	2.13×1.22	2	1.35	1.65	2.7	Inlet	None
2	83.2	3.05×2.44	3	1.38	3.37	3.4	Outlet	Class II riprap
3	96.5	3.05×3.05	3	1.42	4.33	3.6	Outlet	Class II riprap
4	111.0	3.66×3.05	3	1.43	4.36	3.9	Outlet	Class III riprap

HW/D = headwater-to-rise ratio (permissible limit: 1.5); V_out = mean outlet velocity (permissible limit for unprotected native soil: 3.0 m/s).

Culvert 1 (Q_{100} = 13.1 m^3/s) converged on a twin cell 2.13 m × 1.22 m box section under inlet control, yielding HW/D = 1.35 and an outlet velocity of 2.7 m/s comfortably within both performance thresholds. No downstream protection was required. Culverts 2 and 3, serving the intermediate sized watersheds with 100-year discharges of 83.2 and 96.5 m^3/s, required triple-cell configurations. The larger tailwater

depths at these sites, extracted from the HEC-RAS 2D water-surface profiles, shifted both crossings into outlet control; computed HW/D ratios of 1.38 and 1.42 remained within the permissible limit, while outlet velocities of 3.4 and 3.6 m/s necessitated Class II riprap apron protection extending 2.0 m downstream. Culvert 4, associated with the largest catchment (15.71 km², Q_{100} = 111 m³/s), required a triple cell 3.66 m × 3.05 m section under outlet control (HW/D = 1.43). Given the outlet velocity of 3.9 m/s, a more robust Class III riprap apron of minimum 0.6 m thickness and 3.0 m length was specified.

A notable outcome of the design phase is the nonlinear relationship between watershed area and required culvert cross sectional area. Although Watershed 4 is approximately sixteen times larger than Watershed 1, the total barrel flow area of Culvert 4 (3 × 3.66 × 3.05 = 33.5 m²) is only approximately 6.4 times that of Culvert 1 (2 × 2.13 × 1.22 = 5.2 m²). This disproportion arises from the combined influence of higher Curve Numbers, steeper slopes, and lower infiltration capacity in the larger sub catchments, which generate disproportionately elevated specific runoff rates per unit area. The finding reinforces the critical importance of individual sub catchment characterization rather than simple area scaling when sizing drainage infrastructure in topographically diverse, semi-arid basins.

All Culvert Master outputs were independently verified against FHWA nomograph hand calculations[22], with agreement within 4% for all four crossings, confirming the reliability of the software generated results. The consistent use of HEC-RAS 2D derived tailwater levels as downstream boundary conditions for Culvert Master ensured full hydraulic continuity across the three modelling stages a methodological distinction from loosely coupled approaches in which inter stage boundary conditions are estimated independently.

Figure 10 Culvert design results from Culvert Master for a 100-year flood return period for the watersheds (A, B, C, and D).

4. Conclusion

This study developed and demonstrated a tightly integrated three stage hydrologic–hydraulic modeling for culvert siting at road–waterway intersections in ungauged basins, using New Zakho City, Kurdistan Region of Iraq, as a case study. The following principal conclusions are drawn:

1. The coupled WMS–HEC-RAS – 2D Bentley Culvert Master workflow produced hydraulically defensible culvert configurations without recourse to calibrated discharge records, relying exclusively on freely available terrain and land-cover datasets.
2. Frequency analysis of the 22-year maximum 24-hour rainfall record yielded design depths of 104 mm (T = 50 year) and 114 mm (T = 100 year). The resultant 100-year peak discharges ranged from 13.1 m³/s (Watershed 1, 0.99 km²) to 111 m³/s (Watershed 4, 15.71 km²), confirming a strong but nonlinear relationship between catchment area and peak discharge governed by sub-catchment specific curve numbers, slopes, and infiltration characteristics.

3. A systematic mesh sensitivity analysis identified 10 m resolution as the optimal balance between spatial accuracy and computational cost, consistent with published guidance.
4. The areal extent of inundation did not vary dramatically between the 50 and 100-year events however, flow depth and velocity distributions differed substantially across return periods and sub-catchments. These findings demonstrate that flood inundation mapping should be an integral, rather than supplementary, component of culvert design in topographically complex basins.
5. The proposed framework demonstrates that reliable culvert siting and sizing can be achieved using open-access geospatial datasets, making it particularly suitable for ungauged watersheds in developing regions.
6. Box culvert configurations ranging from twin cell 2.13 m × 1.22 m to triple cell 3.66 m × 3.05 m sections were designed to satisfy a maximum permissible HW/D ratio of 1.5 under the 100-year event.
7. The proposed workflow provides a practical, transferable framework for resilient culvert siting and hydraulic design in ungauged, data-scarce basins, reducing reliance on empirical methods while supporting more reliable infrastructure planning.

The primary limitation of this study is the absence of gauged discharge records for formal model calibration and validation, an unavoidable constraint common to ungauged-basin applications. For future work the framework should also be extended to incorporate climate change projections for updated design storm derivation, given the documented intensification of extreme precipitation events across the Kurdistan Region and the broader Middle East under projected warming scenarios.

Declarations

Author contributions

Nawbahar F. Mustafa: Conceptualization, methodology, data curation, formal analysis, visualization, writing—original draft.

Anwar H. Dawood, Hydraulic analysis, writing—review and editing.

Luqman S. Othman: Review and editing, Kawa Z. Abdulraman: Analysis and review.

Nadhir Al-Ansari: visualization, Supervision and review.

Data Availability Statement:

Data is available by request from corresponding author.

Conflicts of Interest:

“The authors declare no conflicts of interest.”

Funding:

This research did not receive any funding

Ethical approval:

This work has no effect on humans or animals and all work and images are properly used.

Consent to participate and publish:

All authors agreed to publish this version of the paper

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Figures

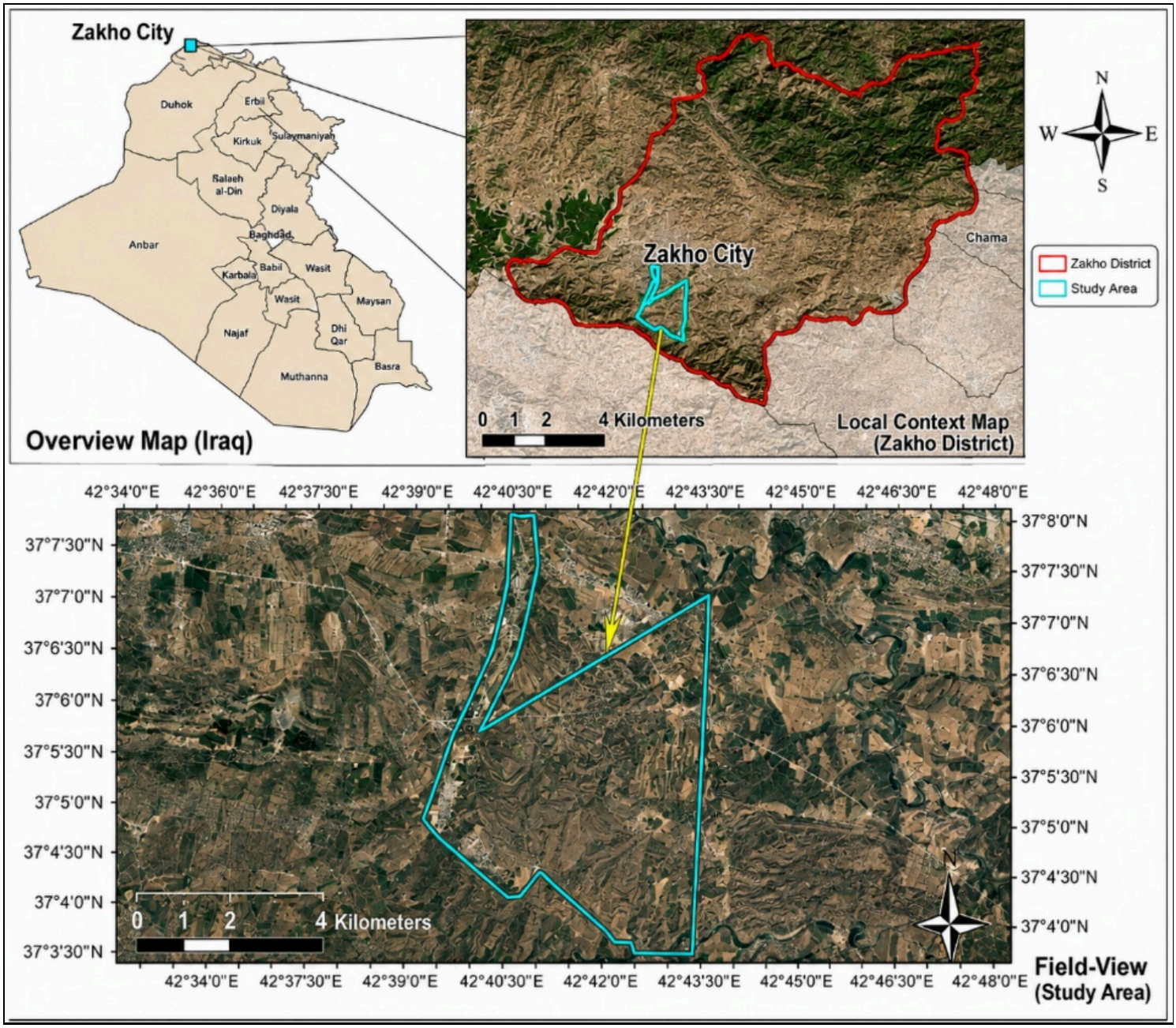


Figure 1

Location map of the study area.

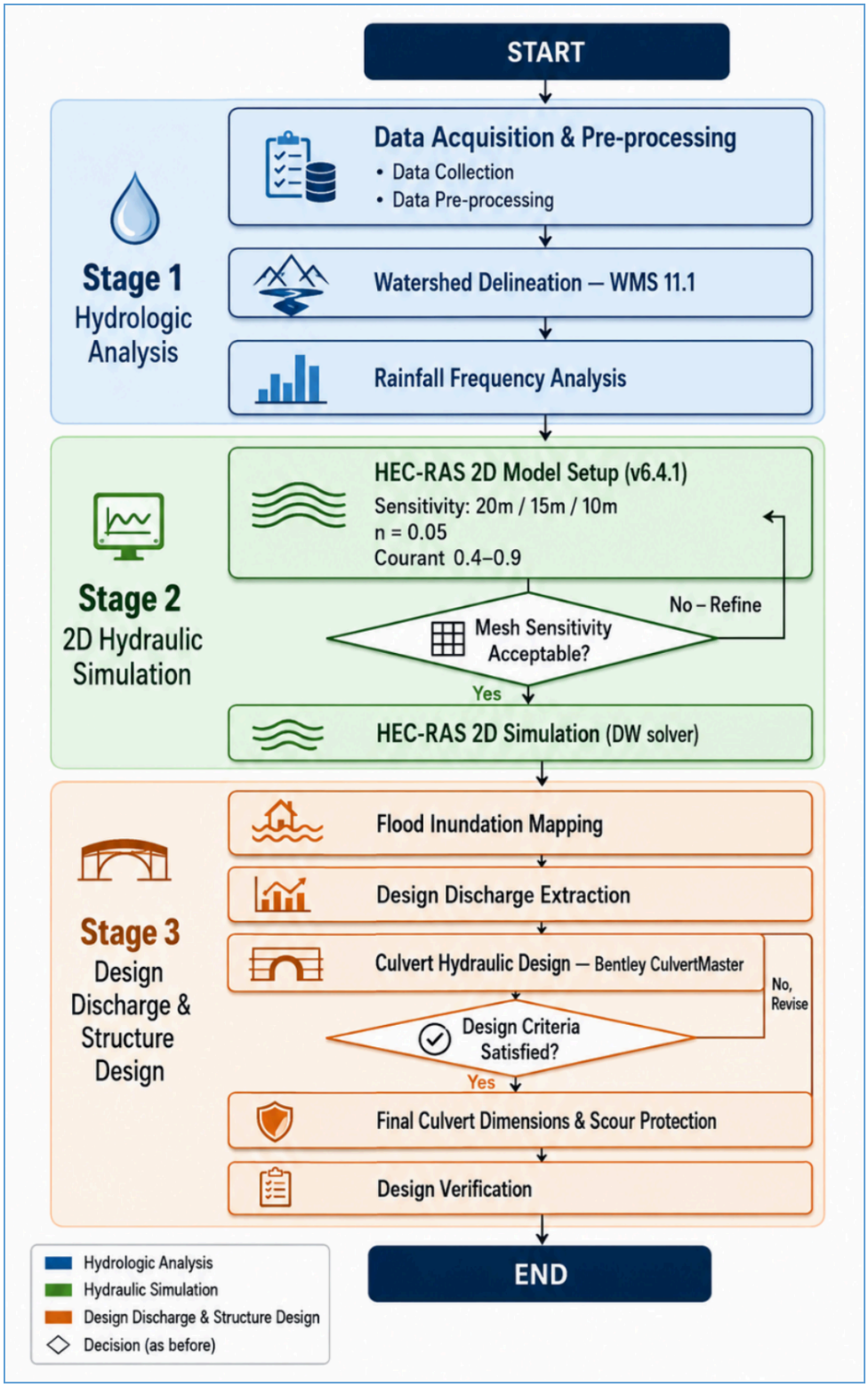


Figure 2

Methodology flowchart illustrates the three-stage integrated framework.

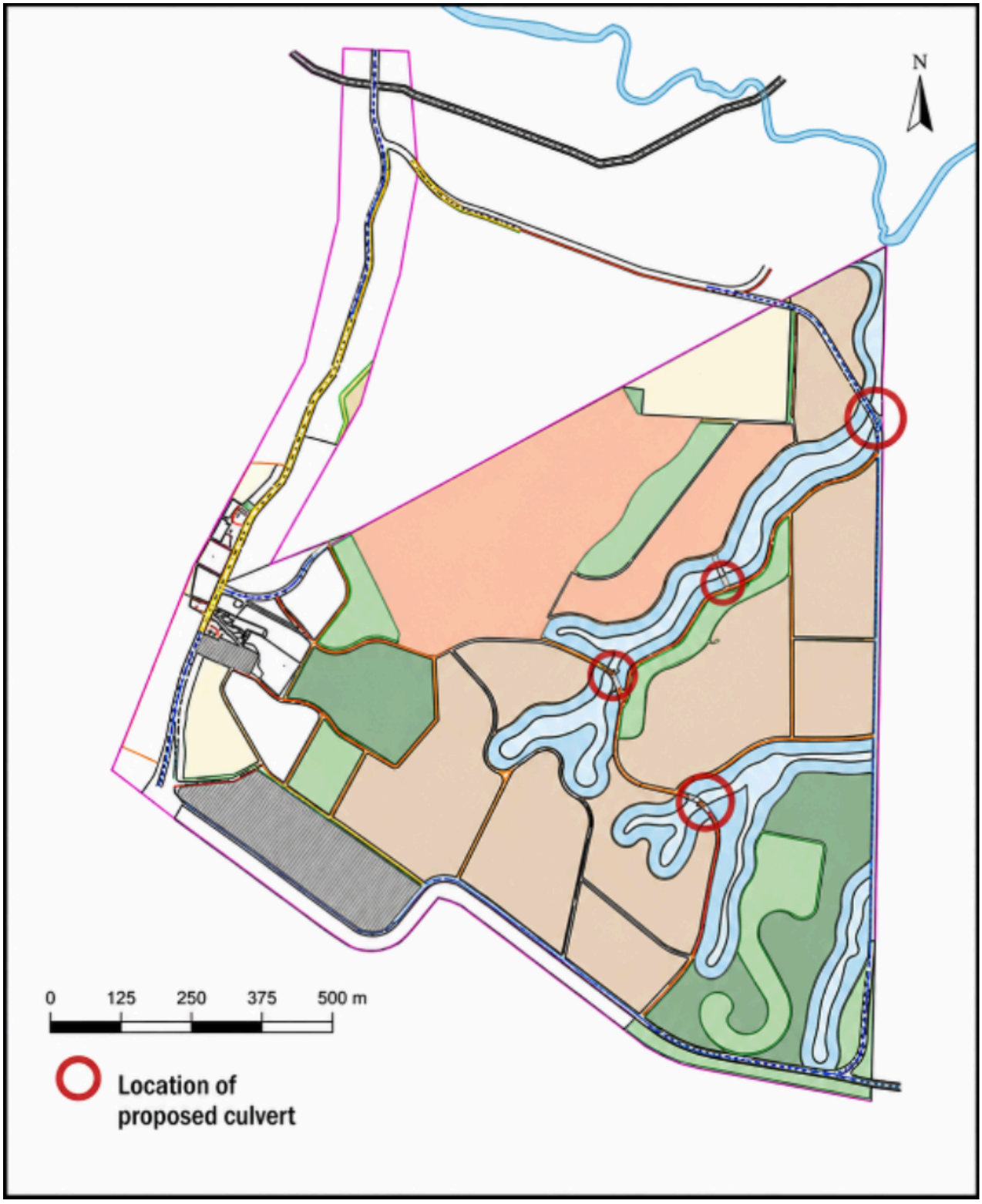


Figure 3

Locations of natural streams with the proposed road.

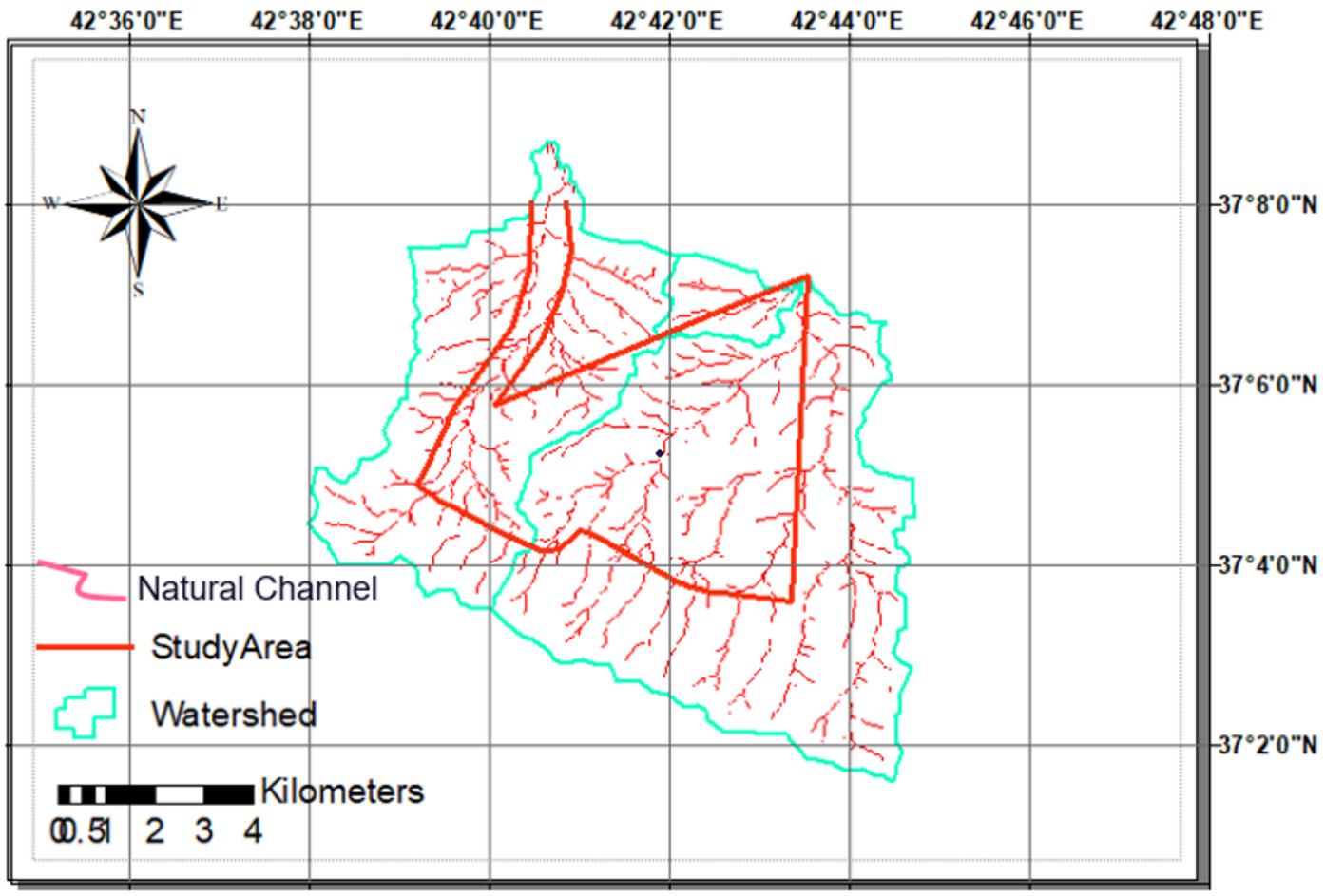


Figure 4

The boundaries of these catchments.

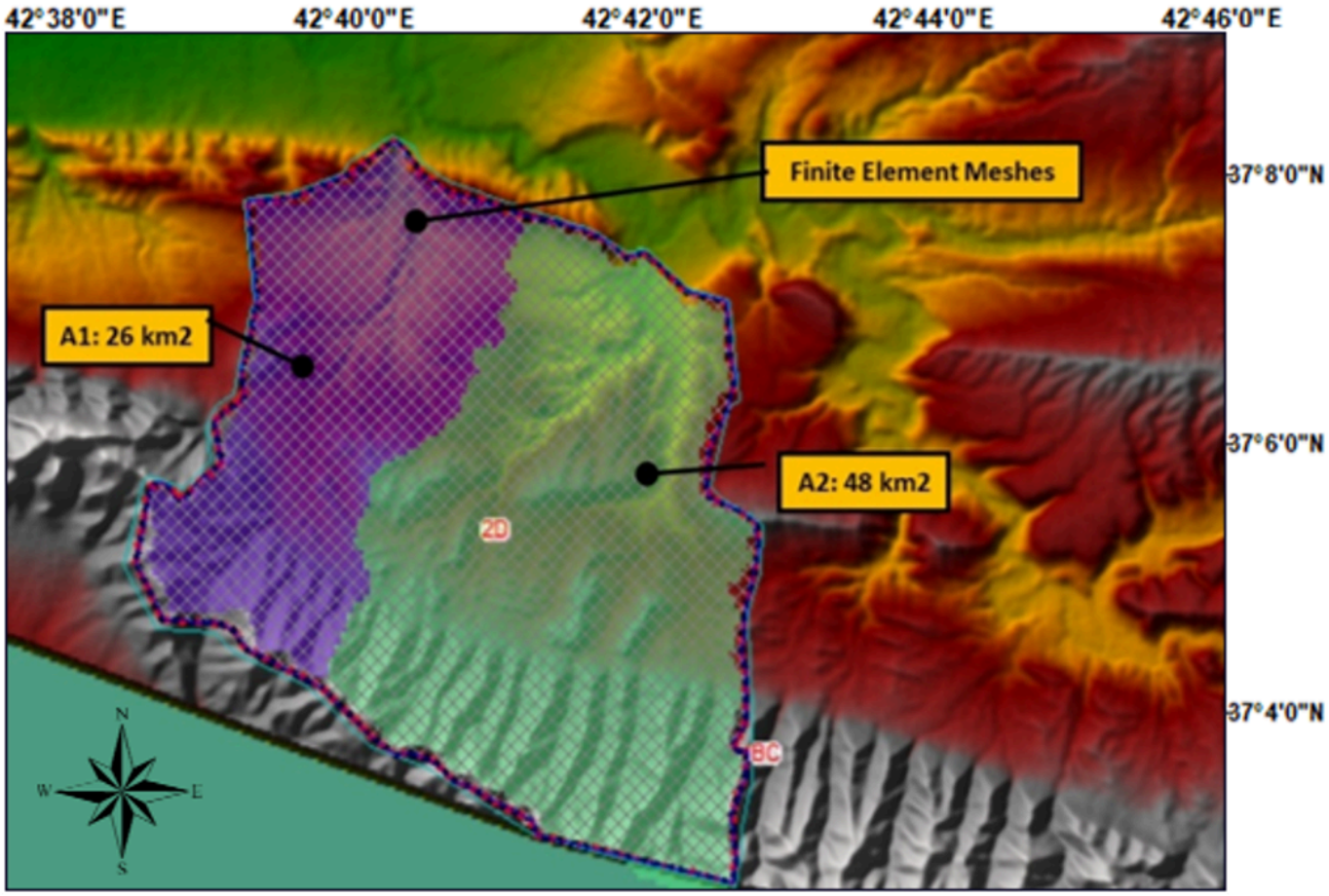


Figure 5

Discretization and delineation of the catchment areas.

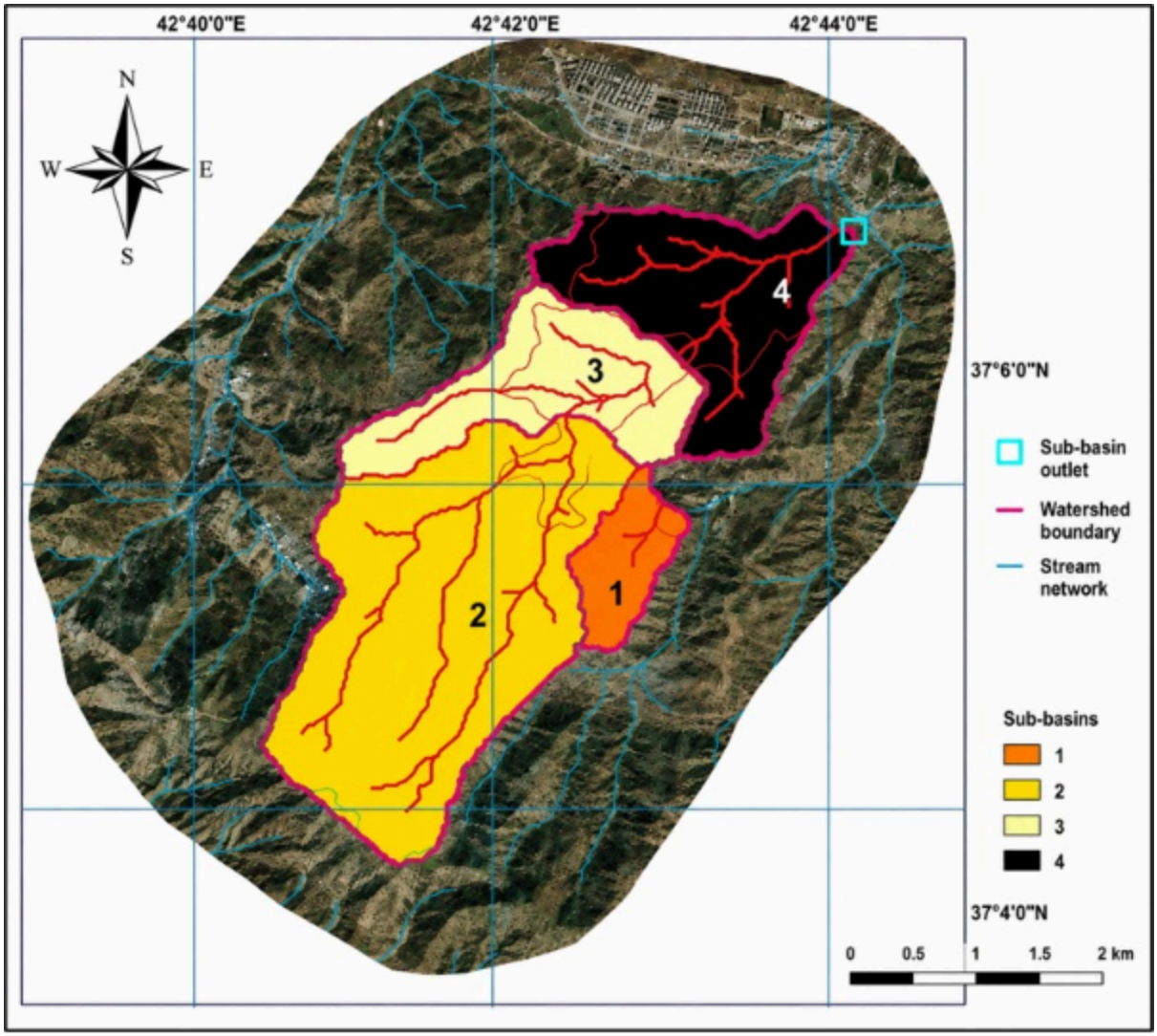


Figure 6

Delineated watersheds and streams inside the study area.

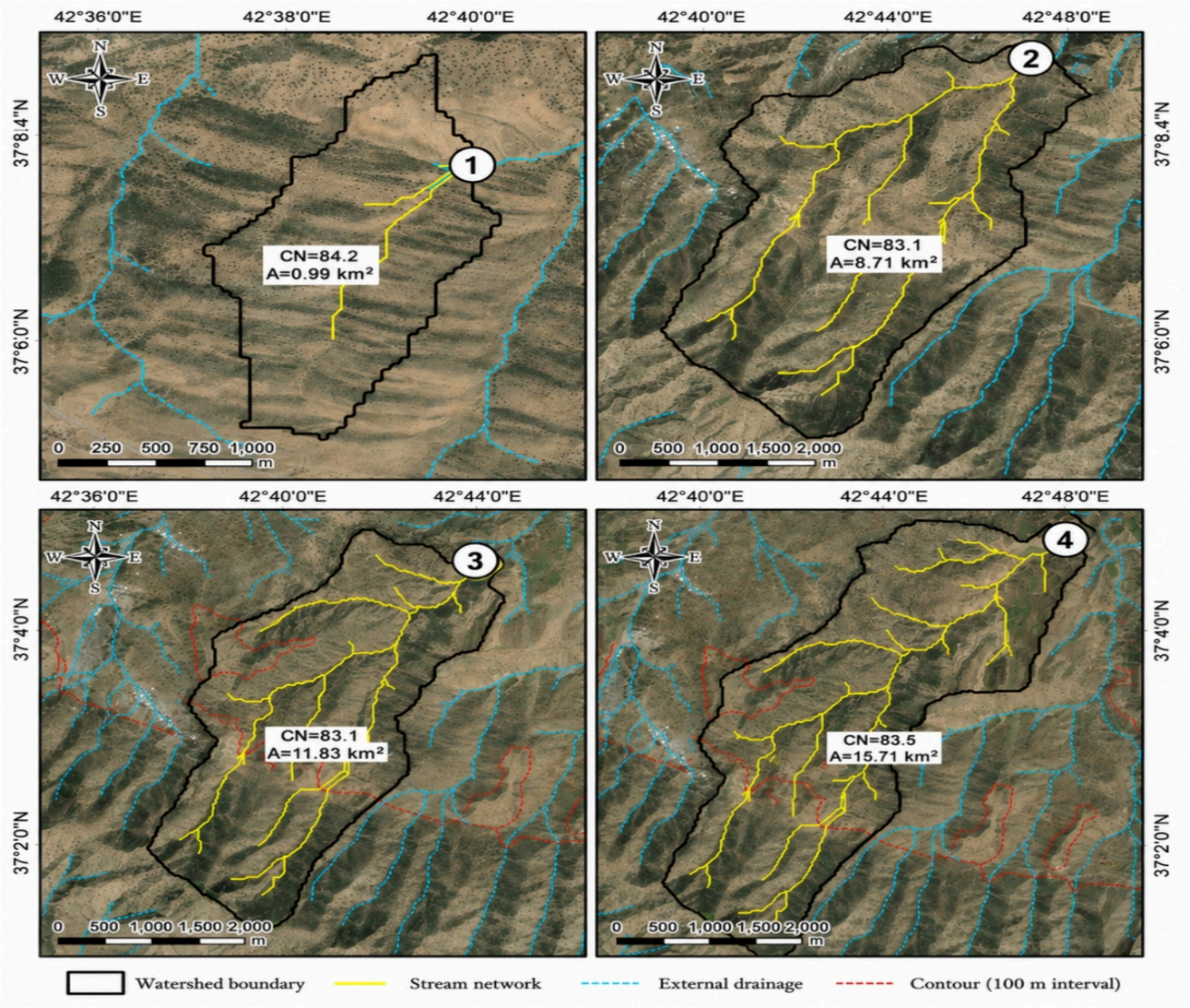


Figure 7

Boundary and CN of the watersheds.

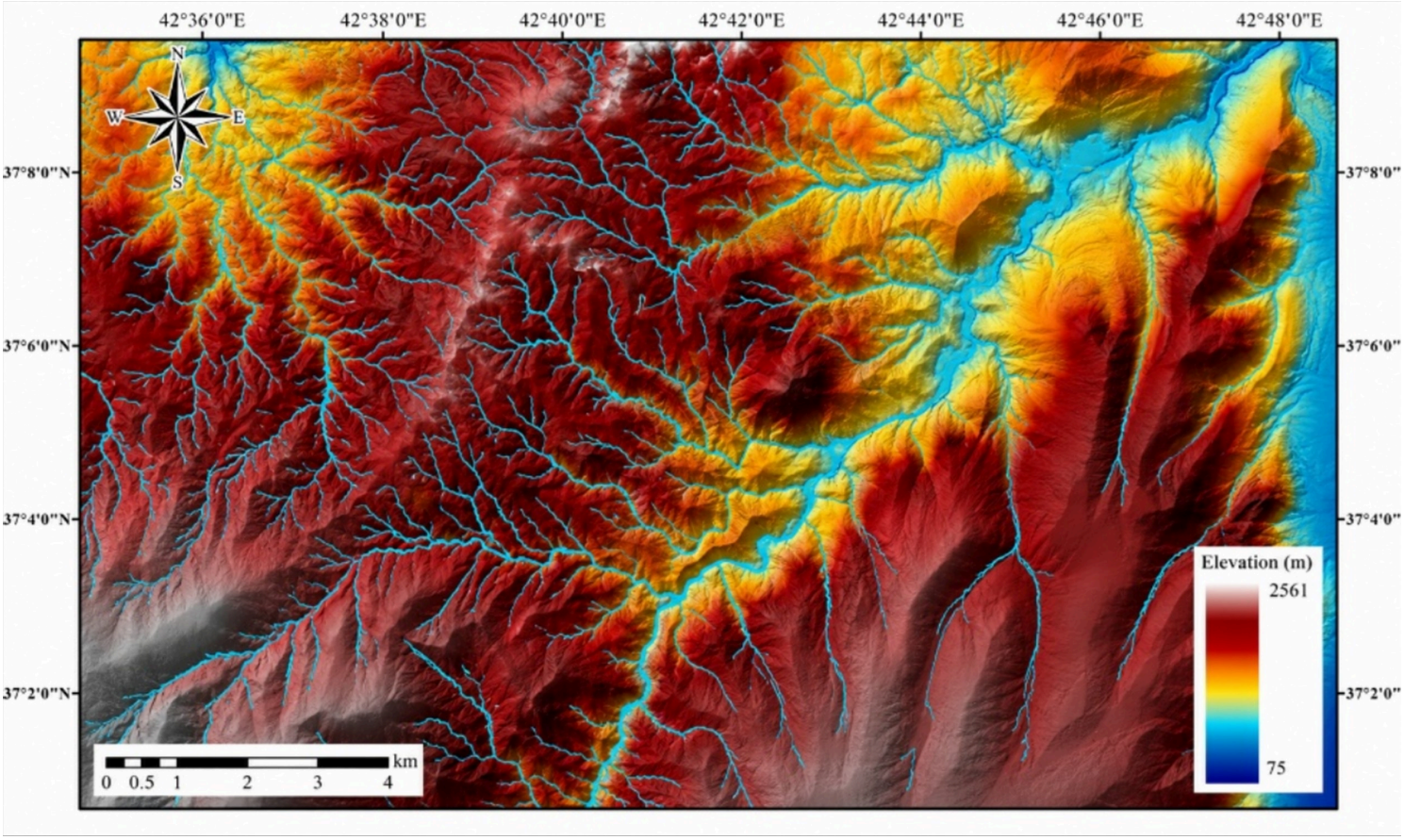


Figure 8

The results of flow depth in the watershed.

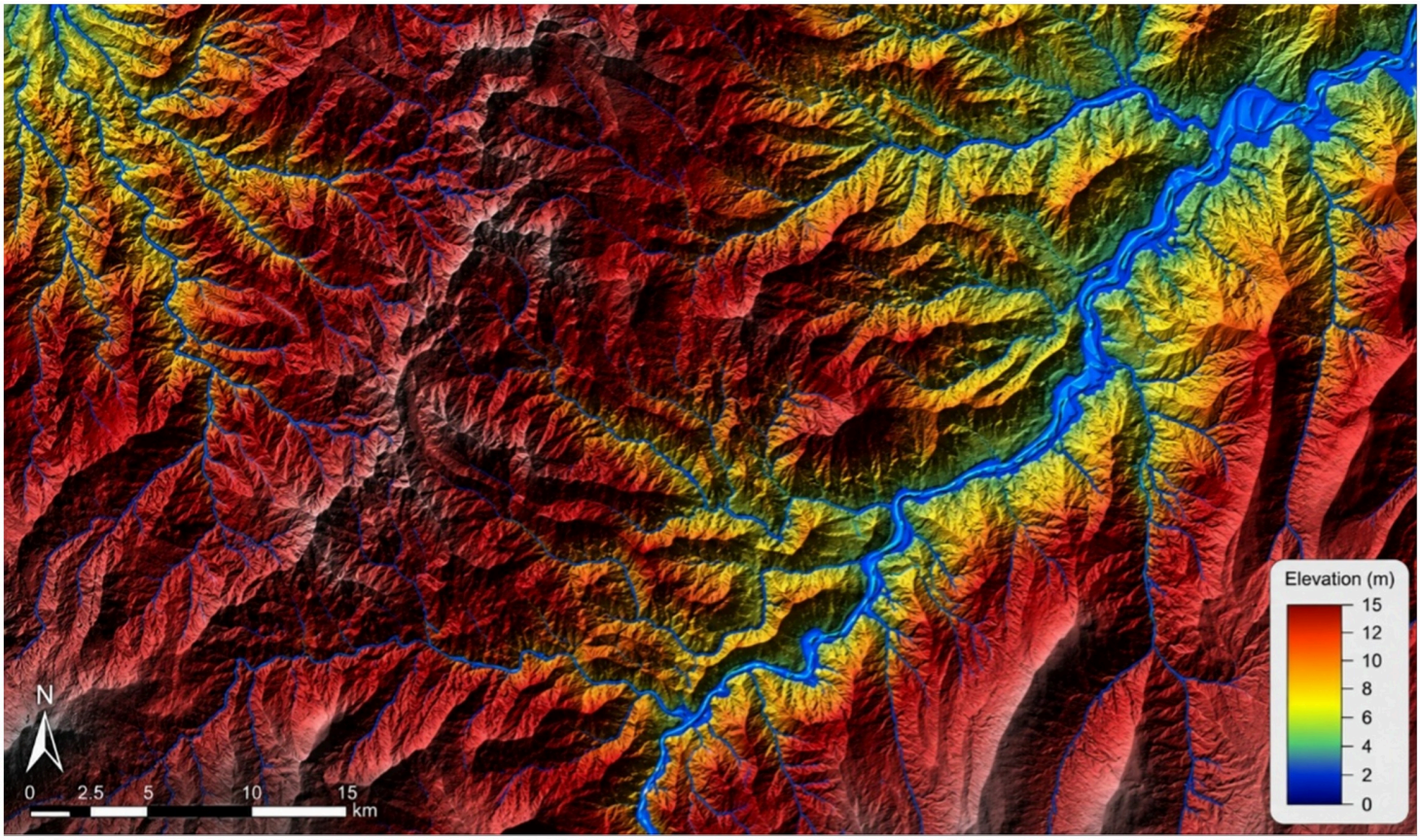


Figure 9

Distribution of flow velocity in the watershed.

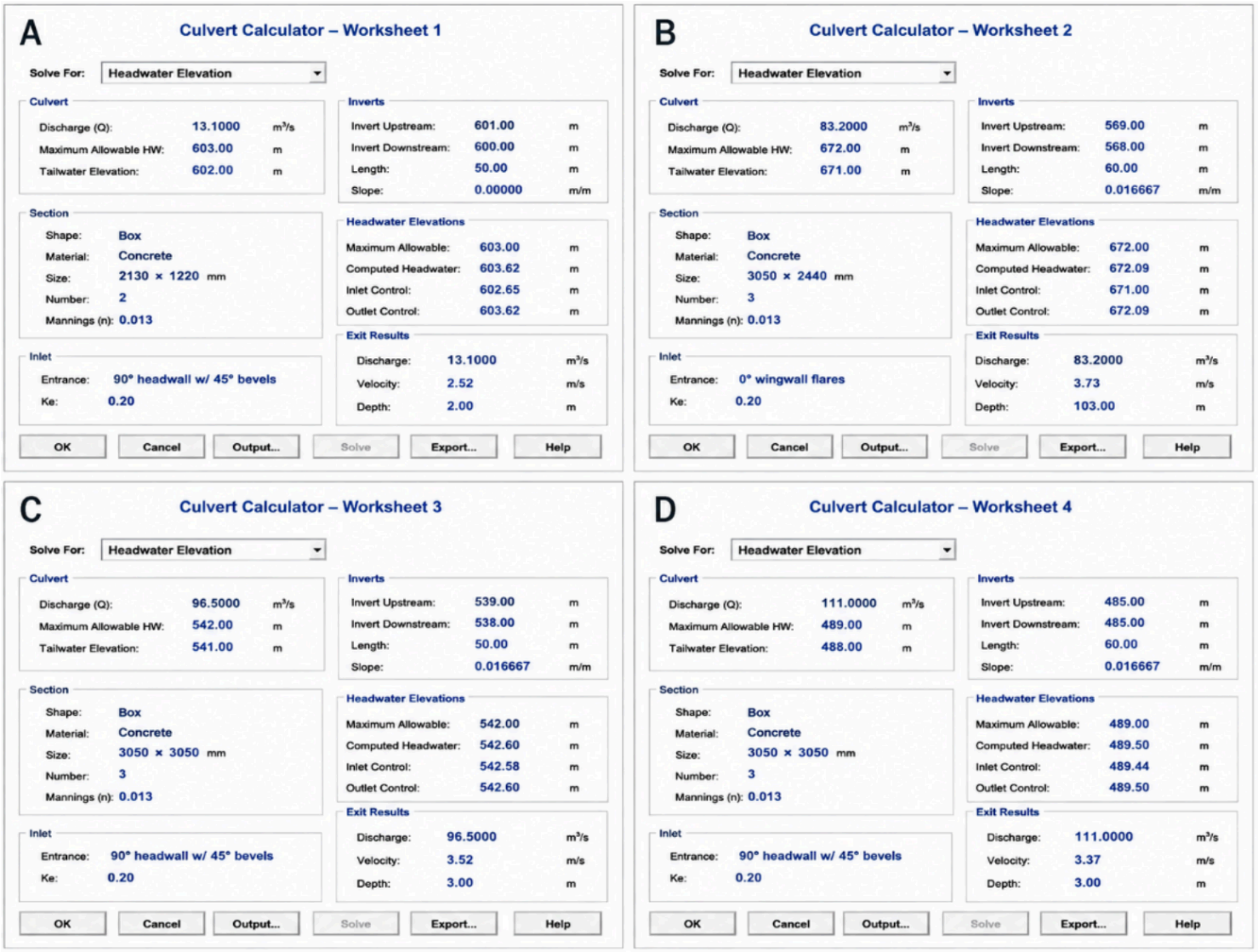


Figure 10

Culvert design results from Culvert Master for a 100-year flood return period for the watersheds (A, B, C, and D).