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MITIGATION OF SCOUR FAILURE RISK OF A RIVER BRIDGE LOCATED IN AN UNGAUGED BASIN

HÜSEYİN AKAY^{*}

Civil Engineering Department, Gazi University, Ankara, Turkey

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Abstract. In this study, scour failure risk of the Catalzeytin Bridge located in the Western Black Sea Basin, Turkey, was assessed for possible future flood events and appropriate scour countermeasures were considered based on economic and constructability considerations. Waterway adequacy in the spans of the bridge and scour criticality around bridge foundations considered for risk calculations in HYRISK were estimated by hydrological and hydraulic analyses of the watershed and stream. Since the watershed of the bridge is ungauged. geomorphological instantaneous unit hydrograph concept was adopted to estimate the peak discharges with various return periods to be used in hydraulic modelling. Monte Carlo simulation results indicated that most of the simulated peak discharges were in the 95% confidence interval. Hydraulic model results from HECRAS indicated that waterway adequacy and scour criticality were critical for discharges with 200 and 500-year return periods. Scour failure risk of the Catalzeytin Bridge was classified as high and it was proposed to reduce the risk by constructing partially grouted riprap as the most feasible alternative that would consequently increase the expected lifespan of the bridge. Following this methodology, river bridges may be prioritized based on the risk analysis.

Keywords: GIUH, HYRISK, scour criticality, ungauged basin, waterway adequacy, Western Black Sea Basin.

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^{*} Corresponding author. E-mail: akayhsyn@gmail.com

Hüseyin AKAY (ORCID ID 0000-0002-9714-4590)

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Introduction

Scour criticality and waterway adequacy are the key input parameters to assess scour failure risk and expected age of a bridge computed using HYRISK software developed by the Federal Highway Administration (FHwA). Johnson and Whittington (2011) assessed cost vulnerability for Level 2 assessment of a river bridge in terms of stability of stream, channel condition, waterway adequacy and scour criticality. Hence, scour criticality and waterway adequacy items used in HYRISK must be rigorously determined and scour countermeasures must be designed, accordingly. However, waterway adequacy in the spans of a river bridge and scour criticality around bridge foundations may not be examined adequately, particularly for old bridges, due to the lack of stream data and gauging stations. In Turkey, specifically river bridges located on steep streams, which collect water from small watersheds, suffer from the lack of stream discharge data due to the limited number of gauging stations. In recent years, an increasing number of river bridges in the northern part of Turkey with steep-slope, small and ungauged watersheds have been damaged or have failed due to the flash floods as a result of changes in watershed characteristics and precipitation pattern. Furthermore, possible changes in climatic and environmental factors, river or land use in the watershed may alter the safety of river bridges in time, thereby increasing the need for performing non-stationary ensemble models.

As floods threaten the serviceability of river bridges, their safety must be assured under different flood events with varying return periods (T_r) . Hence, scour criticality and waterway adequacy in the spans of a bridge under various flood conditions must be examined using the available data and scientific approaches. Possible future flood discharges for ungauged basins can be estimated by transferring, regionalizing hydrologic parameters from gauged neighbouring basins or remote sensing and field measurement techniques. However, it is not recommended to use transferred or spatially distributed hydrologic parameters in the computation of design discharge as the spatial heterogeneities in the considered watershed may not be captured (Akay, Baduna Koçyiğit, & Yanmaz, 2018; Baduna Koçyiğit & Akay 2018; Baduna Koçyiğit, Akay, & Yanmaz, 2017; Zhang et al., 2013). Unit hydrograph of a watershed can also be estimated using morphometric parameters, which can directly reflect the characteristics of a watershed. For this, geomorphological instantaneous unit hydrograph (GIUH) concept is considered to be successful and has been validated for many basins all over the world (Ghumman, Al-Salamah, Alsaleem, & Haider, 2017; Kumar,

Mitigation of Scour Failure Risk of a River Bridge Located in an Ungauged Basin

Chatterjee, Singh, Lohani, & Kumar, 2007; Rodriguez-Iturbe & Valdes, 1979; Sahoo, Chatterjee, Raghuwanshi, Singh, & Kumar, 2006). GIUHensemble hydrologic models, whose estimated results are validated using the data of nearby gauged basins, may contribute to further understanding of the response of an ungauged basin.

Scour failure risk can be mitigated by constructing an appropriate countermeasure at the point before the bed lowers towards the footings. Johnson and Niezgoda (2004) proposed appropriate countermeasures suggesting the risk-based priority number method based on failure modes and effects analysis. Lagasse *et al.* (2007) also introduced detailed design procedures of various scour countermeasures. Yanmaz and Apaydin (2012) assessed scour failure risks under possible future conditions, designed some scour countermeasures and selected the most appropriate and feasible one.

In this study, scour failure risk assessment of the Çatalzeytin Bridge, whose watershed is ungauged, was performed using HYRISK software. Necessary peak discharges with different return periods were transformed from effective precipitation using GIUH approach. Then, Monte Carlo simulation was used to test whether those values were in the 95% confidence interval. Scour criticality and waterway adequacy were estimated by HECRAS using estimated peak discharges. Some of the input parameters used in HYRISK were estimated in the course of hydrologic and hydraulic analyses, and field trips. Finally, various types of scour countermeasures were designed and economically assessed.

1. Materials and methods

1.1. Study area

The Çatalzeytin Bridge located in the Western Black Sea Basin is the only route that connects two counties. It was opened to traffic in 1963. It was designed as Gerber beam with two piers skewed to the flow. In 1988, one of the piers of the bridge was damaged due to flood and then was reconditioned (Akay & Baduna Koçyiğit, 2020; Akay, Baduna Kocyigit, & Yanmaz, 2019). However, in recent years, the width of the stream bed was significantly reduced to expand a route parallel to the stream. So, stream levees were constructed at both banks of the stream. This is thought to increase the risk that the Çatalzeytin Bridge might face during flooding. During this study, the Çatalzeytin Bridge was inspected ten times in three years at different flow conditions. During inspections, it was noticed that excessive scour around bridge foundations occurred and the tops of bridge

THE BALTIC JOURNAL OF ROAD AND BRIDGE ENGINEERING 2021/16(1)

foundations were visible. Furthermore, some inputs used in risk analysis, such as channel protection and substructure conditions, were recorded.

The Akçay Stream, which flows through the bridge, collects water from 372.4 km² drainage area and drains water into Black Sea, just downstream of the bridge. The watershed has dominantly forested, steep mountains with maximum, median and minimum altitudes of 2005 m, 1150 m, and 7 m, respectively, with an average slope of 45%. The watershed may take heavy rainfall in a year with heavy snow, especially at high altitudes. Curve number of the watershed was estimated as 79.3 considering the land cover/use and hydrologic soil group distributions in the watershed. Considering the curve number, it can be pointed out that large amount of that precipitation transforms into runoff.

1.2. Methodology

Methodology followed in this study is demonstrated in the flowchart given in Figure 1. The main steps of the methodology include GIUH ensemble hydrologic modelling, Monte Carlo simulation and uncertainty



Figure 1. Flowchart of the methodology to assess the scour failure risk of the Çatalzeytin Bridge

Mitigation of Scour Failure Risk of a River Bridge Located in an Ungauged Basin

assessment of GIUH parameters, hydraulic modelling, scour failure risk, and countermeasure design. Details of the main steps are given in the following sections.

1.3. Hydrologic modelling

Since there are no available stream gauging stations on the Akçay Stream and the frequency analysis of the stream cannot be directly determined, hydrologic analysis has been performed to estimate the peak flow discharge for different return periods. Hydrologic parameters for ungauged basins could be directly transferred, regionalized from the gauged neighbouring basins or determined by remote-sensing and field measurements. However, hydrologic parameters used in these hydrological processes may not be efficiently estimated and those methods may not reflect the topographic heterogeneities in the ungauged basins. Hence, a reliable method for the estimation of hydrological processes should be adopted in order to estimate the flood hydrographs.

hydrologic processes Analysis of was conducted using meteorological and basin model components. Precipitation data of intensity duration frequency (IDF) curve with return periods of 2, 5, 10, 25, 50, 100, 200 and 500 years observed at the Catalzeytin Meteorological Observation Station were used in the meteorological model. In the basin model, excess rainfall and transformation of that rainfall to runoff were modelled using SCS-CN method and unit hydrograph approach, respectively. The SCS-CN method can estimate all kinds of losses and effective rainfall (ER) in the watershed, as given in Eqs. (1) and (2):

$$ER(t) = \frac{(P - I_a)^2}{P - I_a + S_s},$$
(1)

$$S_{\rm s} = \frac{25\,400}{CN} - 254,\tag{2}$$

where I_a – initial abstraction; P – precipitation; S_s – potential maximum retention in soil, mm; and CN – curve number of the watershed. I_a value depends on the moisture of the soil and can generally be defined as 0.2 times of S_s assuming the soil is in dry conditions.

Direct runoff hydrographs corresponding to any return periods were transformed from *ER* values using Nash's geomorphological instantaneous hydrograph (GIUH) concept, since it was found to perform very good in estimation of direct runoff hydrograph at close watershed of the Akçay Basin (Akay, 2019). GIUH ordinates were calculated considering the geomorphological characteristics of the watershed THE BALTIC JOURNAL OF ROAD AND BRIDGE ENGINEERING 2021/16(1)

and the following procedure was followed to estimate the watershed characteristics.

Digital elevation model (DEM) and flow accumulation (Fac), with 10 m resolution of the watershed of the Catalzevtin Bridge sited at the outlet of the watershed, were generated in ArcGIS 10.1 using the 1:25000 scaled topographic maps. Stream network was extracted by taking the threshold value of Fac as 100 since it was determined to be sensitive for prioritization of the watershed with regard to flash floods (Baduna Kocviğit & Akay, 2018). Stream orders were designated using Strahler (1957) method, as shown in (Figure 2). After discrete streams having the same order that were greater than one were merged manually, the weighted dominant morphometric parameters such as length ratio, bifurcation ratio and area ratio parameters were computed and used for calculation of GIUH ordinates. Design storm hydrographs and flood peak discharges with return periods of 2, 5, 10, 25, 50, 100, 200, and 500 years, which may potentially flow through the Catalzeytin Bridge, were determined to estimate the waterway adequacy and scour criticality around bridge foundations.



Figure 2. Stream orders of the watershed of the Çatalzeytin Bridge

1.4. Nash's instantaneous unit hydrograph

The Nash's instantaneous unit hydrograph concept assuming twoparameter gamma distribution (Γ) considers *n* numbers of successive linear reservoirs having the same storage coefficients (*k*), as given in Eq. (3).

Mitigation of Scour Failure Risk of a River Bridge Located in an Ungauged Basin

$$u(t) = \left(\frac{t}{k}\right)^{n-1} \frac{\mathrm{e}^{-t/k}}{k\Gamma},\tag{3}$$

where u(t) – ordinate of the hydrograph at time t, h^{-1} ; and Γ – gamma function. Rosso (1984) proposed Eqs. (4) and (5) to estimate simply k and n values of a watershed without solving non-linear iterations. Moreover, Γ values may be approximated in terms of n by Eq. (6) proposed by Nemes (2010).

$$n = 3.29 \left(\frac{R_{\rm b}}{R_{\rm A}}\right)^{0.78} R_{\rm l}^{0.07}; \tag{4}$$

$$k = 0.70 \left(\frac{R_{\rm A}}{R_{\rm b}R_{\rm l}}\right)^{0.48} \frac{L_{\Omega}}{\nu};\tag{5}$$

$$\Gamma = \left(\frac{n}{e}\right)^n \sqrt{\frac{2\pi}{n}} \left(1 + \frac{1}{12n^2 - 0.1}\right)^n,$$
(6)

where R_b – bifurcation ratio; R_A – area ratio; R_1 – length ratio; L_{Ω} – main channel length, km; v – velocity of peak flood discharge (Q_p), m/s; and e – natural logarithm constant. R_b , R_A , R_I , and L_{Ω} values of the watershed of the Çatalzeytin Bridge were estimated by ArcGIS 10.1, while n, k and Γ values were computed from Eqs. (4)–(6).

There are various approaches for estimation of the v value that considers watershed characteristics and excess rainfall depths (Chen et al., 2019; Ghumman et al., 2017; Gupta, Waymire, & Wang, 1980, Hosseini, Mahjouri, & Riahi, 2016; Rodriguez-Iturbe, Gonzalez-Sanabria, & Bras, 1982; Sahoo et al., 2006). Sahoo et al. (2006) proposed a stepfunction relationship between excess rainfall intensity and flow velocity. Kumar et al. (2007) adopted the peak velocity as 2.5 m/s based on the field measurements in their study. Hosseini et al. (2016) adopted the main channel length and watershed slope, while Ghumman et al. (2017) considered the excess rainfall, drainage area and kinematic wave parameter of the channel to estimate the expected peak velocity. On the other hand, Chen et al. (2019) proposed a new method to estimate the dynamic-factor peak velocity considering the extraction of geomorphologic characteristics of the watershed using random forest technique. In most of the similar studies, the peak velocity was calculated as 2.0–2.5 m/s, so in this study the peak velocity was taken as 2.5 m/s, as recommended by Kumar et al. (2007).

Calculated parameters to estimate the ordinates of GIUH of the watershed of the Çatalzeytin Bridge are given in Table 1. Since parameters given in Table 1 may have uncertainties, those uncertainties might directly affect the model outputs. So, Monte Carlo simulation,

Table 1. Computed parameters to estimate GIUH ordinates

Rı	R _b	R _A	L _Ω	v	n	k	Г	СN
4.74	2.05	4.95	21.92	2.5	3.35	4.44	2.82	79.3

which is one of the stochastic approaches, was implemented coupled with the hydrologic model to investigate the uncertainty and factor analysis of the impact of each parameter on model outputs and the usability of specified parameters.

1.5. Monte Carlo simulations

Monte Carlo simulations coupled with the hydrological processes are based on the random number generation of effective parameters of GIUH and implementation of the hydrological model. Monte Carlo simulations were performed to examine the uncertainty assessment and factor analysis of the impact of each generated parameter on the calculated peak discharges. Hence, random variables such as $R_{\rm h}$, $R_{\rm A}$, $R_{\rm l}$, L_{Ω} , v, and CN which have effects on the model outputs were generated with 1000 runs assuming normal distribution due to its simplicity. $R_{\rm h}$, $R_{\rm A}$, $R_{\rm h}$ and L_{Ω} values are sensitive to the scale effect of stream network caused by the threshold value of Fac and vary in a narrow range. CN value of the watershed may be affected by the initial moisture of the soil and errors in the land use/cover and soil maps. v value is the velocity of the peak discharge as a consequence of excess rainfall and watershed characteristics (Sahoo et al., 2006). v value may vary in a large range depending on the local channel properties. The calculated parameters given in Table 1 were assumed as averages of the generated random variables. Coefficient of variance (COV) was assigned the value of 0.1 for v parameter, while COV values of other parameters were assigned the value of 0.05, as mentioned before. Although COV for CN was taken at a low value, random CN values varied in a large range, which might describe the soil moisture conditions at different saturation levels that reflect the real case condition. It should be noted that a COV value of simulated peak discharges is an important indicator of uncertainty. As the COV values of simulated results increase, uncertainty also increases.

The ordinates of the GIUH were first calculated and then the peak discharges were simulated using generated random parameter values for each run and $T_{\rm r}$. Moreover, 95% upper and lower limits of confidence interval of the simulated peak discharges for different $T_{\rm r}$ were determined. Computed peak discharges using average values of parameters corresponding to $T_{\rm r}$ were tested by simulation

Q _p	T _r , years	2	5	10	25	50	100	200	500
	Average (Table 1)	31.3	71.9	107.1	160.4	207.0	259.2	370.0	520.8
	95% upper limit	33.8	74.9	110.4	164.2	210.7	262.8	373.5	524.7
	95% lower limit	32.1	72.1	106.8	159.7	205.6	257.0	366.4	516.0
	COV	0.44	0.31	0.26	0.22	0.20	0.18	0.15	0.13

Table 2. Computed peak discharges from average and generated values, confidence intervals and COV values of peak discharges for 2, 5, 10, 25, 50, 100, 200 and 500-year return periods

of parameters obtained from Monte Carlo results (Table 2). Peak discharges computed from the average values were found to be in the 95% confidence interval with the results obtained from Monte Carlo simulations except for the discharges of T_r of 2 and 5 years. COV values computed from Monte Carlo simulations decreased as T_r increased. This is an important indicator that COV of computed peak discharges vary in a large range which means a decrease in uncertainty as T_r increases. This is an important finding, since peak discharges with higher T_r values may be critical for assessment of waterway adequacy and scour criticality. Moreover, it can be concluded that peak discharges for T_r of 2 and 5 years were not critical in terms of waterway adequacy and scour criticality, peak discharges computed from average parameter values were found to be reasonable to be used in the hydraulic analysis.

Factor analysis based on the eigenvalues of parameters extracted by principal component analysis in SPSS was also conducted using the



Figure 3. Factor analysis results indicating the weights of parameters obtained from cross-correlation analysis

generated GIUH parameters to investigate the contribution of each parameter. It is noted that the contributions of parameters become close varying between 15–19%, since each parameter has initial eigenvalues close to each other. Weights of each parameter were simplified by using the cross-correlation analysis of parameters obtained from factor analysis adopted by Malik *et al.* (2019). Figure 3 represents parameter weights in percent from factor analysis. R_1 had the maximum weight, while *CN* had the minimum weight.

1.6. Hydraulic modelling

In this study, hydraulic computations were performed by HECRAS model which is a free software developed by US Army Corps of Engineers, Hydrologic Engineering Centre. HECRAS is used to analyse both one- and two-dimensional hydraulic calculations including steady and unsteady flow simulations, sediment transport computations, and water quality analysis (HEC, 2010). All computations performed by HECRAS require entry of geometric data of the channel and flow either manually or importing. Geometric input data, such as stream cross-sections at desired stations, location and dimensions of control structures, and roughness values at cross-section, as well as flow data at boundary conditions can be entered.

Inspection studies enabled surveying of the alignment and crosssections of the stream, and geometric properties of the Catalzeytin Bridge were used in hydraulic computations. Flood peak discharges obtained from hydrologic modelling coupled with Monte Carlo simulation and factor analysis for different T_r were also used in hydraulic computations of steady state flow data. Running the model, water surface profile along the cross-sections of reach and in the spans of the bridge was computed. Moreover, scouring at bridge piers and abutments in terms of dimensions of pier, abutment, and bed material size and conditions was computed by Colorado State University (CSU) and Froehlich equations, respectively. According to the hydraulic model results, bridge was found to be submerged for peak discharge having T_r of 200 years and more. Moreover, calculations showed that the piers of the Catalzeytin Bridge were unstable due to the excessive scouring with a peak discharge having $T_{\rm r}$ of 200 years and more. Waterway adequacy in the spans and scour criticality around bridge foundations that was used to estimate the scour failure risk of Catalzeytin Bridge were estimated by combining the hydraulic model results and scour risk assessment criteria defined by Pearson et al. (2002).

Mitigation of Scour Failure Risk of a River Bridge Located in an Ungauged Basin

1.7. Scour failure risk assessment

HYRISK software developed by FHwA was used for scour failure risk assessment of the Çatalzeytin Bridge for possible future peak discharges. HYRISK implements a qualitative risk analysis of scour induced failure that is the product of failure probability and its consequences (Pearson *et al.*, 2002). Failure probability in US dollars is considered as a function of overtopping frequency and scour criticality, while the consequences of failure are considered as the summation of rebuilding and running of the bridge, and time costs. The economic feasibility of any available scour countermeasures at a particular bridge site can be evaluated based on the cost analysis associated with loss of life and, knowing the service life of the structure, the time value of money, in the scour countermeasures calculator provided in HYRISK (Pearson *et al.*, 2002). So, the cost-benefit relation can be analysed for any countermeasure structure. Annual scour risk (*R*) of a river bridge in US dollars, which is composed of rebuilding, running and time costs, can be computed by Eq. (7).

$$R = KP_{\rm A} \left[C_1 W L_{\rm b} M + C_2 D A d + \left[C_3 O \left(1 - \frac{T}{100} \right) + C_4 \frac{T}{100} \right] \frac{D A d}{S} \right],$$
(7)

where *K* –risk adjustment factor; P_A – annual probability of scour failure, 1/year; C_1 – unit rebuilding cost, US\$/m²; *W* – bridge deck width, m; L_b – bridge length, m; *M* – cost multiplier to replace bridge after the scour failure; C_2 – unit cost of running vehicle, US\$/km; *D* – detour length, km; *A* – average daily traffic, vehicles per day; *d* – duration of detouring, days; C_3 – unit value of time per adult, US\$/h; *O* –occupancy rate, adults per vehicle; *T* – average daily truck traffic of *A*, %; C_4 – unit value of time per truck, US\$/h; and *S* – average detour speed, km/h (Pearson *et al.*, 2002).

Some input parameters, such as the year of built, functional class of the bridge, bypass length, average daily traffic (*A*), truck traffic per cent of *A*, service type, construction type, structural design, deck width, and structural length, were provided by the authorities that are in charge of the construction, maintenance, and running of highways and bridges. Current conditions of the substructure and channel protection were inspected during field studies. Inspections at different time intervals showed that excessive scour, which occurred around bridge foundations, could be visually observed. Since levees at both sides of the channel were constructed, it was concluded that the banks were protected. Hence, the substructure was coded as 4, which indicated poor conditions, and channel protection was coded as 8. As mentioned before, one of the most important parameters P_A is a function of waterway adequacy in the spans of the bridge, as well as the scour criticality around the foundations. Hydraulic modelling simulation results as the product of hydrologic modelling coupled with Monte Carlo analysis and factor analysis of GIUH parameters, enabled to estimate at which level water may approach to the bridge girders and scouring depth at bridge foundation. Waterway adequacy was coded in HYRISK according to both the hydraulic modelling simulation results and the National Bridge Inventory (NBI) Item 71 descriptions. Similarly, scour criticality was coded according to the scour calculations performed by HECRAS and the stability of bridge foundation due to scouring described in NBI Item 113 (Pearson *et al.*, 2002). So, P_A was calculated accordingly.

For some parameters and coefficients, default values stored in the software recommended by AASHTO were used for calculation of R, especially for the components of time cost parameters. The 90th percentile mean time to scour failure (X_{90}), which is the expected remaining service life of a river bridge, can be approximated by Eq. (8).

$$X_{90} = \frac{\log(1 - 0.90)}{\log(1 - P_{\rm TR})},\tag{8}$$

where P_{TR} represents trial probability of scour failure assuming binomial distribution (Pearson *et al.*, 2002). Table 3 presents the codes of waterway adequacy and scour criticality for various T_{r} , and thus, the calculated scour failure risk and expected ages, accordingly.

Rebuilding, running, and time costs for the Çatalzeytin Bridge were independent of T_r , waterway adequacy, and scour criticality, and were found to be US\$ 329 858, US\$ 1 320 543 and US\$ 2 518 540, respectively. Annual scour failure risk *R* for T_r of 2 years was US\$ 14 580 per year, while

T _r , years	Waterway adequacy	Scour criticality	P_{TR}	P _A	<i>R</i> , US\$ per year	Expected age, years
2	9	5	0.005	0.005	14 580	440
5	9	4	0.127	0.041	114 523	17.0
10	9	4	0.127	0.041	114 523	17.0
25	9	4	0.127	0.041	114 523	17.0
50	8	4	0.137	0.041	114 523	15.6
100	5	4	0.228	0.041	114 523	8.9
200	3	3	0.398	0.041	114 523	4.5
500	2	3	0.495	0.041	114 523	3.4

Table 3. Scour failure risks and expected ages of the Çatalzeytin Bridge for various T_r

Mitigation of Scour Failure Risk of a River Bridge Located in an Ungauged Basin

R was US\$ 114 523 per year for the rest of T_r , since P_A did not change with respect to T_r . However, the change in P_{TR} inversely altered the expected age of the river bridge. The expected lifespan is 17 years, if a 5-year recurrence flood passes through the spans of the Çatalzeytin Bridge. However, the expected lifespan of the Çatalzeytin Bridge is 3.4 years is case of a flood with 500-year return period that causes significant delay in traffic and serious damage to the infrastructure. Annual scour failure risks of bridges in the USA are classified as acceptable risk at US\$ 1000 per year, medium risk at US\$ 10 000 per year, and unacceptable risk at US\$ 100 000 per year (Briaud, Gardoni, & Yao, 2012). Hence, an appropriate countermeasure against scouring should be installed around bridge foundations of the Çatalzeytin Bridge to prevent serious damage or failure risk is estimated as high for the Çatalzeytin Bridge.

1.8. Risk mitigation and countermeasure design

Calculations showed that piers of the Catalzeytin Bridge were unstable when peak discharge of the flood with 200 years and more recurrence peak discharge passed through the bridge. In case of installing scour countermeasure around foundations of the Catalzeytin Bridge, scour criticality was coded as 8 in HYRISK indicating that the foundations were stable (Pearson et al., 2002; Yanmaz & Apaydin, 2012). Thus, annual scour failure risks for 200 and 500-year recurrence floods then decreased to US\$ 40 222 per year and US\$ 77 762 per year and expected ages increased to 158.7 and 81.6 years, respectively. Moreover, Yanmaz and Apaydin (2012) compared the benefits of the bridge countermeasures, such as partially grouted riprap, grout filled bag and gabion box. In this study, when the riprap was considered as a countermeasure, the depth of the riprap under the subgrade level was found to be very large and piers were unstable. So, partially grouted riprap, grout filled bag and gabion box were considered to mitigate scour failure risk.

1.9. Partially grouted riprap

As rocks are glued by grouts, rock classification, amount and grout volume should be determined in case of installation of partially grouted riprap (Lagasse *et al.*, 2007). Since the median size of 6 cm of the classical riprap was acceptable, Class I with median riprap size of 16 cm, void ratio of 50%, porosity of 35% and specific gravity of 2.65 were selected for design of partially grouted riprap. As the tops of pier foundations of the Çatalzeytin Bridge were visible, 2 m soil was

Work package	Unit	Unit price, US\$	Quantity	Cost, US\$
Excavation	m³	7.2000	126.00	907.20
Transportation of excavation	t	0.6415	333.50	213.94
Grout	m³	45.0736	22.05	993.87
Riprap	m³	5.9962	81.90	491.09
Transportation of riprap	m³	1.1830	126.00	149.06
Geotextile and its placement	m ²	0.9340	38.00	35.49
Total cost				2790.66
Total cost with contingencies				3209.26

Table 4. Unit prices and costs of partially grouted riprap

excavated from the channel bed to the bottom elevation of the pier foundation. Thus, 1.5 m was excavated, which was greater than two times of pier width of 1.2 m, and covered by soil from each outer edge of the circumference of the bridge pier foundations (Lagasse *et al.*, 2007). Partially grouted riprap was placed to the excavated lands. Excavation and manufacturing of partially grouted riprap were performed according to these requirements. Geotextile filter was placed extending two thirds of the distance where riprap was stretched. Grout volume was determined by multiplication of the excavation volume, void ratio and porosity. Riprap volume is the volume of the solid rock soil. 126 m³ soil samples were excavated and transported. 38 m² geotextile was placed under partially grouted riprap. 81.90 m³ riprap and 22.05 m³ grout were needed to construct partially grouted riprap. The cost of each item of partially grouted riprap and contingencies of the total cost are given in Table 4. Contingency rate for all countermeasures was considered as 15% of the total cost (Yanmaz & Apaydın, 2012).

1.10. Grout filled bag

Canvas bags filled with grout are used to armour the channel in the vicinity of bridge crossing (Yanmaz & Apaydın, 2012). Canvas bags with dimensions of 1.2 m × 0.9 m × 0.3 m (length, width and thickness) as suggested by Ozdemir (2003) were used. For this, 1.8 m of soil depth between the lower foundation level and the top level of canvas bags and 2.4 m of soil depth around the circumference of each edge of pier foundations were excavated and transported. 212.55 m³ soil was excavated and 212.55 m³ grout was filled in 656 canvas bags and placed into the excavated lands. Costs of each item to be installed for grout filled bags are given in Table 5.

Work package	Unit	Unit price, US\$	Quantity	Cost, US\$
Excavation	m³	7.2000	212.55	1530.36
Transportation of excavation	t	0.6415	563.26	361.34
Grout	m³	45.0736	212.55	9580.39
Canvas	m²	1.0283	2243.52	2307.02
Total cost				13 779.10
Total cost with contingencies				15 845.97

Table 5. Unit prices and costs of grout filled bags

1.11. Gabion box

Gabion boxes can be used in many fields including scour countermeasures. Riprap with graded sizes is placed into gabion boxes with varying permeable mesh sizes and the channel bed is protected by the armouring effect. In the study, 2 m of soil depth between the lower foundation level and the channel bed and 2 m of soil depth around the circumference of the pier foundations were excavated. Geotextile was placed under the gabion boxes, its dimensions were 2.0 m × 1.0 m × 1.0 m and 2.0 m × 1.0 m × 0.5 m, where necessary, and filled with graded riprap. 184 m³ of soil was excavated and transported. 92 m² geotextile was placed under the gabion boxes. 80 of 2.0 m × 1.0 m × 1.0 m and 24 of 2.0 m × 1.0 m × 0.5 m gabion boxes were filled in with 184 m³ graded riprap. Costs of each item to be installed for the gabion box are given in Table 6.

Work package	Unit	Unit price, US\$	Quantity	Cost, US\$
Excavation	m³	7.2000	184.00	1324.80
Transportation of excavation	t	0.6415	487.60	312.80
2.0 m × 1.0 m × 1.0 m gabion box	piece	35.4868	80	2838.94
2.0 m × 1.0 m × 0.5 m gabion box	piece	28.4868	24	683.68
Transportation of riprap	m³	1.1830	184.00	217.68
Geotextile and its placement	m²	0.9340	92.00	85.92
Total cost				5463.83
Total cost with contingencies				6283.40

Table 6. Unit prices and costs of the gabion box

2. Results and discussions

Since US\$ 114 520 per year annual scour risk was classified as high and the Çatalzeytin Bridge would continue to be in service for the next 50 years or more, three different scour countermeasures for the Çatalzeytin Bridge were analysed for 200 and 500-year recurrence floods (Pearson *et al.*, 2002). These were partially grouted riprap, grout filled bags and gabion box with 15% contingencies costing US\$ 3209.26, US\$ 15 845.97 and US\$ 6283.40, respectively. According to the HYRISK results, as the scour criticality code was updated as 8, the expected ages of the Çatalzeytin Bridge increased and the annual scour risk decreased. The corresponding category was changed after installation of the appropriate scour countermeasure. Net benefits of the construction of partially grouted riprap, grout filled bags and gabion box for the 50-year return period were calculated as US\$ 2 724 241, US\$ 2 711 604, US\$ 2 721 167, respectively, and partially grouted riprap was found to be the most feasible alternative among other countermeasure options.

Conclusions

In this study, scour failure risk of the Çatalzeytin Bridge located in the Western Black Sea Basin was investigated for flash floods with 2, 5, 10, 25, 50, 100, 200 and 500-year return periods. The Akçay Creek flowing through the Catalzeytin Bridge collects water from an ungauged basin. So, GIUH concept was used to estimate the peak discharges of the aforementioned return periods conducting analysis with some morphometric parameters computed for this concept. Next, the values of the computed morphometric parameters and curve number were taken as average values and then Monte Carlo simulation was used to generate 1000 random variables assuming a normal distribution for them. The hydrologic analysis indicated that although the peak discharges for T_r of 2 and 5 years were not in the limits of the 95% confidence interval and not critical for scour risk assessment of the Catalzeytin Bridge for those $T_{\rm r}$ values, using computed peak discharges for the whole $T_{\rm r}$ in hydraulic modelling was found to be reasonable. According to the HECRAS results for T_r of 200 and 500 years during flooding, the Catalzeytin Bridge was found to be submerged and scour criticality around bridge foundations was unstable. Since within this methodology it was significant to adopt 95% lower and upper limits of peak discharges in hydraulic modelling, it would be beneficial to evaluate the sensitivity of both waterway adequacy and scour criticality of the Catalzeytin Bridge in scour failure risk assessment. Scour failure risk was computed coding the relevant

Mitigation of Scour Failure Risk of a River Bridge Located in an Ungauged Basin

input in HYRISK for various T_r . Risk was classified as high, so the scour countermeasure structure that would decrease the scour failure risk and increase the expected age was suggested. Conducting the economic analysis of three scour countermeasure structures, partially grouted riprap was found to be the most feasible one. It is believed that using this methodology, prioritization of river bridges, which are susceptible to flash floods and in which stream discharge values cannot be gauged, may be conducted regarding the results of economic analysis and considering the available budget of a responsible authority. Moreover, it is recommended to improve this methodology to account for the changing environmental, climatic, policy of run-of-river, and other factors in future.

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NOTATIONS

Variables and functions

- A average daily traffic;
- CN curve number;
- C_1 unit rebuilding cost;
- C_3 unit value of time per adult;
- C₂ unit cost of running vehicle;
- C₄ unit value of time per truck;
- *D* detour length;
- *d* duration of detouring;
- ER effective rainfall;
- e natural logarithm constant;
- *I*_a initial abstraction;
- K risk adjustment factor;
- k storage coefficient;
- $L_{\rm b}$ bridge length;
- L_{Ω} length of main channel;
- *M* cost multiplier to replace bridge after scour failure;
- *n* number of successive linear reservoirs;
- *O* occupancy rate;
- *P* precipitation;
- $P_{\rm A}$ annual probability of scour failure;

THE BALTIC JOURNAL OF ROAD AND BRIDGE ENGINEERING 2021/16(1)

 $P_{\rm TR}$ – trial probability of scour failure;

R – annual scour risk;

 $R_{\rm A}$ – area ratio;

 $R_{\rm b}$ – bifurcation ratio;

 R_1 – length ratio;

S – average detour speed;

 $S_{\rm s}$ – potential maximum retention in soil;

T – average daily traffic;

t – time;

u – ordinates of the hydrograph;

v – velocity of peak flood discharge;

W – bridge deck width;

 X_{90} – 90th percentile mean time to scour failure;

 Γ – gamma function;

a – explanation of the first variable;

b – explanation of the second variable;

c – explanation of the third variable, etc.

ABBREVIATIONS

COV - coefficient of variance;

Fac – flow accumulation;

FHwA – Federal Highway Administration;

 $T_{\rm r}$ – time of return period;

US\$ – US Dollar.