A STUDY ON RISK ASSESSMENT OF SCOUR VULNERABLE BRIDGES

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ABSTRACT

A STUDY ON RISK ASSESSMENT OF SCOUR VULNERABLE BRIDGES

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Many river bridges fail or are seriously damaged due to excessive local scouring around piers and abutments. To protect a bridge from scour-induced failure, it should be designed properly against excessive scouring and its scour criticality should be checked regularly throughout the service life to take prompt action. The Federal Highway Administration of United States (FHWA) developed a program, HYRISK, as a basis for evaluation of existing scour failure risk of a bridge. It provides implementation of a risk-based model, which is used to calculate the annual risk of scour failure of a bridge or series of bridges in monetary values. A case study is carried out for a bridge crossing Fol Creek in Black Sea Region (close to Vakfıkebir), for the illustration of this software. Besides, hydraulic analysis and scour depth computations of the bridge are carried out via HEC-RAS program. Also, a study is carried out to recommend scour countermeasures that can be applied to the aforementioned bridge.

Keywords: Risk assessment, local scour, bridge piers, abutments, scour countermeasures, HYRISK, HEC-RAS

ÖZ

OYULMA EĞİLİMLİ KÖPRÜLERDE RİSK DEĞERLENDİRMESİ ÜZERİNE BİR ÇALIŞMA

Apaydın, Meriç

Yüksek Lisans, İnşaat Mühendisliği Bölümü

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Çok sayıda nehir köprüsü orta ve kenar ayaklar etrafındaki aşırı yerel oyulmalar nedeniyle yıkılmakta veya ciddi hasar görmektedir. Bir köprüyü oyulma nedenli yıkılmaya karşı korumak için köprü oyulmaya karşı uygun tasarlanmalı; sonra gerekli korumaları yapabilmek için köprünün oyulma kritikliği servis ömrü içerisinde düzenli olarak kontrol edilmelidir. Bir köprünün mevcut oyulma riskini değerlendirmede bir temel oluşturması amacıyla Amerika Birleşik Devletleri, Federal Karayolu İdaresi (FHWA) HYRISK yazılımını geliştirmiştir. Bu yazılım, bir veya bir dizi köprünün oyulma nedeniyle yıllık yıkılma riskini parasal olarak hesaplamakta kullanılan risk tahmin modelinin uygulanmasını sağlamaktadır. Risk tahmin modeli ve HYRISK yazılımının gösterimi için, Karadeniz Bölgesi'ndeki (Vakfıkebir civarı) Fol Deresi üzerinde bulunan mevcut bir köprü ile örnek uygulama yapılmıştır. Ayrıca, bu köprünün hidrolik analizi ve oyulma derinliği hesapları HEC-RAS yazılımı ile gerçekleştirilmiştir. Ayrıca, bahsi geçen köprüye uygulanabilecek oyulma önleyici düzenlemeler üzerine bir çalışma yapılmıştır.

Anahtar Kelimeler: Risk değerlendirmesi, yerel oyulma, köprü ayağı, kenar ayak, oyulma önleyici düzenlemeler, HYRISK, HEC-RAS

In the memory of my very special grandmother Naşide Selamoğlu

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LIST OF SYMBOLS AND ABBREVIATIONS

a	: Pier width
A = ADT	: Average daily traffic
ACB	: Articulated concrete block system
B_1	: Present value benefit accounting for loss of life
B_2	: Present value benefit precluding for loss of life
C_E	: Present value of expected rebuilding cost
C_f	: Cost of failure, including injury and loss of life
C_R	: Current rebuilding cost
C_1	: Unit rebuilding cost
C_2	: Unit cost of running vehicle
C_3	: Unit value of time per adult
C_4	: Unit value of time for truck
d	: Duration of detour
D	: Detour length
D_d	: Dimensionless depth
D_f	: Depth of flow corresponding to Q_f
D_i	: Normal depth of a corresponding Q_i
D_L	: Costs associated with loss of life
D_m	: Distance from bridge upstream to meander impact point
D_{r50}	: Median riprap size diameter
D_{50}	: Median grain size diameter
D_{90}	: Diameter of grain of which 90% is finer
DSİ	: General Directorate of State Hydraulic Works
EİE	: Electrical Power Resources Survey and Development Administration
F	: Failure
Fr	: Froude number
Fr_1	: Froude number just upstream of pier
Fr_a	: Froude number just upstream of abutment

g	: Gravitational acceleration
H,V	: Horizontal and vertical value of side inclinations
i	: Discount rate
K	: Risk adjustment factor
K_1	: Bridge type factor
K_2	: Foundation type factor
K_{a1}	: Coefficient for abutment shape
K_{a2}	: Coefficient for angle of embankment to flow
K_p	: Coefficient for pier nose shape
K_{p1}	: Correction factor for pier nose shape
K_{p2}	: Correction factor for angle of attack of flow
K_{p3}	: Correction factor for bed condition
K_{p4}	: Correction factor for armoring
KGM	: General Directorate of Highways
KHGM	: General Directorate of Rural Services
L	: Remaining useful life of bridge
L_a	: Projected abutment length
L_b	: Bridge length
L'	: Length of active flow obstructed by embankment
M	: Emergency cost multiplier
M_P	: Present value given annual cost multiplier
NBI	: National Bridge Inventory
O	: Occupancy rate
OT	: Overtopping frequency
P	: Annual probability of scour failure
P_A	: Annual probability of failure without protection
P_L	: Probability of failure over the expected life of bridge
P_L'	: Probability of failure over the extended life of the protected bridge
P_{tr}	: Trial probability of scour failure
Q	: Discharge
Q_f	: Full flow discharge
Q_i	: Discharge corresponding to a return period

R	: Annual scour failure risk
R_h	: Hydraulic radius
RP	: Return period protection desired
S	: Average detour speed
S_f	: Energy grade line slope
S.C.	: Scour Criticality
SV	: Scour vulnerability
T = ADTT	: Average daily truck traffic
T_r	: Return period of a discharge
u_p	: Velocity just upstream of pier
W	: Bridge deck width
X_{90}	: 90^{th} percentile mean time to scour failure
y_a	: Average flow depth
y_{a1}	: Approach flow depth
y_1	: Flow depth just upstream of pier
y_s	: Scour depth
Z_{min}	: Minimum channel bed elevation
Z_w	: Water surface elevation
α	: Approach flow angle to bridge
Δ	: Relative density
γ	: Specific weight of water
ρ_w	: Water density
τ_0	: Bed shear stress
τ_c	: Critical shear stress at the incipient motion

CHAPTER 1

INTRODUCTION

1.1 Statement of the Problem

Water is one of the most powerful natural resource which is sometimes on the side of people and sometimes against them. Structures on and/or around water should be designed properly for the sake of safety of people and environment. River bridges are one of those structures serving in contact with water and also standing against water.

Many river bridges fail or are extremely damaged due to floods. Most common reason of failure is excessive local scouring during high floods. Excessive local scour occurs around bridge piers and abutments as a result of removal of bed material due to severe flow patterns surrounding the foundations (Yanmaz and Selamoğlu, 2010). Since excessive scouring leads to considerable riverbed degradation, bridges should be designed to resist such unfavorable effects. In addition to proper design, bridges should be monitored periodically and existing scour criticality of them should be evaluated. Scour criticality of a bridge is assessed according to the level of scour with respect to the footing elevation of that bridge. A bridge is said to be scour critical if the final eroded bed level around the bridge foundation reaches the upper elevation of its footing (See Figure 1.1) (Pearson et al.,2002).

According to scour criticality of bridges, countermeasures should be installed if a bridge is scour critical (Özdemir, 2003 and Yanmaz and Özdemir, 2004). Scour countermeasures are structural units to mitigate the adverse effects of scouring on the stability of bridge components and bridge as a whole. To sum up, it is a must to evaluate scour vulnerability of a bridge, consider possible results of scouring, then select and design suitable countermeasures against scouring for the sake of bridge safety and life around the bridge.

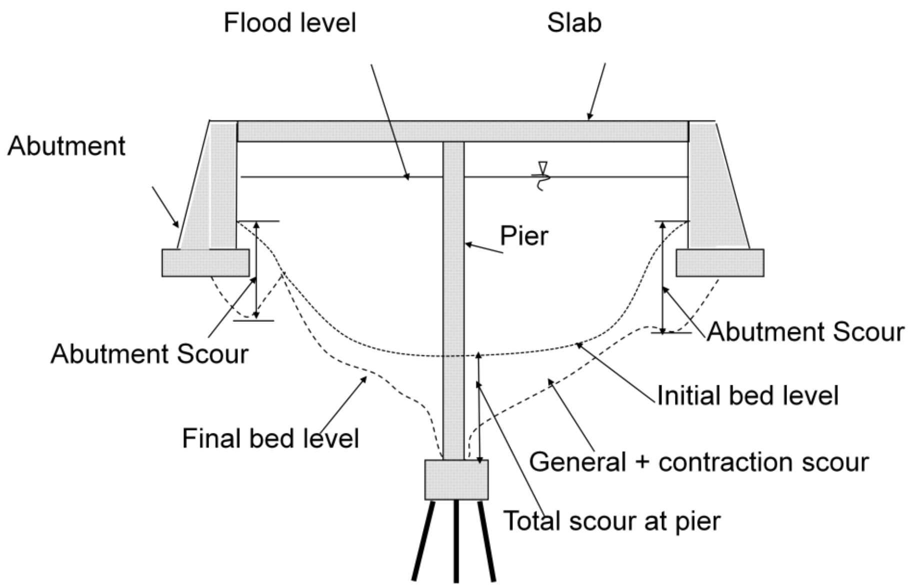


Figure 1.1 A definition sketch for local scour around bridge piers and abutments (Yanmaz A. M., 2002)

1.2 Objectives of the Study

Main objective of this study is to apply the software HYRISK for determining the annual failure risk of a scour vulnerable bridge. In determination of the failure risk, several stability parameters are studied together with scour criticality aspects of bridges. Scour depth calculations are performed to get support for assessing the bridge scour criticality. To illustrate the assessment of scour vulnerability and determination of failure risk of a bridge, a case study is carried out for an existing bridge in Turkey. In addition to those evaluations mentioned above, suitable countermeasures are defined to protect the aforesaid bridge against scour-induced failure.

1.3 Description of the Thesis

This thesis is composed of five chapters. Contents of the chapters are as follows:

An introduction to the thesis is made and the objectives of the study are highlighted in Chapter 1.

In Chapter 2, the methodology of the study is explained and HYRISK software is introduced. Governing equations and tables used in the study are presented in this chapter.

The case study is described in Chapter 3. Results of the analyses are also given in this chapter.

Chapter 4 discusses the results obtained in the case study.

In Chapter 5, a summary of the thesis is presented. Also several recommendations for future works are made.

In the Appendix A, the outputs of HEC-RAS program are tabulated.

CHAPTER 2

EVALUATION OF RISK OF SCOUR VULNERABLE BRIDGES

2.1 General

Scouring is the most common reason for bridge damage or failure. Excessive scouring occurs due to erosive effects of water flow, which would lead to removal of material from the stream bed and bank, and around infrastructural elements of bridges.

This study is mostly concentrated on the probabilistic approach to scour risk evaluation; therefore the mechanism of local scour is not given. Interested readers are advised to refer to Yanmaz and Altınbilek (1991), Melville and Coleman (2000), Yanmaz (2002), Yanmaz (2006), and Yanmaz and Köse (2009).

Since scouring is the most severe damage or failure reason for river bridges, investigating the potential of scour failure of bridges should be made carefully. Due to complexity of river flow pattern, analyzing the real scouring phenomenon around piers and abutments is relatively difficult. Because of unknown nature of various parameters, uncertainties would arise, which should be treated and incorporated in the model (Yanmaz and Selamoğlu, 2010). As a preliminary approach, probabilistic scour risk evaluation would be quite helpful to evaluate the scour criticality of a bridge. This approach and its methodology will be explained in the subsequent sections. Such an approach is implemented using a software program, HYRISK (Pearson et al., 2002).

2.2 Scour Risk Evaluation

For evaluating the risk of scour failure of a bridge, a risk-based model is developed and documented in the Federal Highway Administration (FHWA) report in 1994, i.e. Report No. FHWA-RD-92-030 "Strategies for Managing Unknown Bridge Foundations". FHWA coded the software program HYRISK for the implementation of this risk-based model (Pearson et al., 2002).

The risk-based model allows determining the annual risk of scour failure of a bridge. The risk is obtained by multiplying annual probability of scour failure of a bridge and economic losses related to the bridge failure. It means that the risk of a bridge failure is estimated in monetary terms. Obtaining the results in cost terms provides convenience to those who intend to improve safety of infrastructural systems both in rural and urban areas. Therefore, assessment of scour criticality of bridges is an integral part of such evaluations. Also the model is helpful to categorize a series of scour-critical bridges according to degree of deficiency. It is, therefore, possible to rank those bridges such that priority is given to the most susceptible ones.

2.2.1 The Risk-Based Model

In the risk-based model, data related to bridge and economic factors are used. A database called National Bridge Inventory (NBI) was generated by FHWA to collect and store information pertinent to bridges in the United States. Data, which are recorded in the database, are modified and/or updated based on periodical field inspections. The NBI parameters used in the risk-based model are tabulated in Table 2.1.

Table 2.1 NBI parameters used in the risk-based model (Pearson et al., 2002)

NBI Parameters
Bypass length
Functional classification
Year built
Average daily traffic
Type of service
Type of span
Structure length
Deck width
Waterway adequacy
Average daily truck traffic
Scour-critical bridges (criticality level)

A brief information on the terms used in this model clarifies the approach. In case of a bridge failure, the bridge is closed and a bypass (detour) route is determined. Bypass length is the additional distance traveled while detouring. Functional classification of a bridge is determined according to its location and character of the roadway. Year of construction of the bridge is another parameter, which accounts for aging effects. Average daily traffic (ADT) is the annual average daily traffic load on the inventory route. Type of service is determined both for under the bridge and over the bridge, whether they are highway, railroad, waterway, or another service type. Type of span is described as both the material used in the construction of the bridge and type of its design. Structure length is the length of the roadway supported on the bridge sub-structure. Deck width is out-to-out width of the bridge slab. Waterway adequacy is related to whether the waterway opening under the bridge is sufficient for no overtopping or not. Average daily truck traffic (ADTT) is the annual average daily truck traffic load on the inventory route, and is indicated as a percentage of ADT. Finally, in scour-critical bridges, current scour vulnerability of a bridge is indicated, based on scour field inspection or scour evaluation study, if available (Pearson et al., 2002).

As mentioned before, annual scour failure risk, *R*, is the product of the expected losses due to scour failure and the annual probability of scour failure of a bridge. Expected losses are the summation of three different cost items associated with failure. The risk-based model can be given simply in Equation (2.1) as follows (Pearson et al., 2002):

$$R = KP\left[(Rebuilding\ Cost) + (Running\ Cost) + (Time\ Cost)\right] \tag{2.1}$$

Here, *Rebuilding Cost* is the cost of replacing the bridge in case of a scour-induced failure. *Running Cost* is the additional costs associated with vehicles running while detouring during the rebuilding period. *Time Cost* is the money loss of trucks and people in vehicles while detouring.

Equation (2.1) can be written in detail as in Equation (2.2):

$$R = KP\left[(C_1 W L_b M) + (C_2 DAd) + \left(C_3 O\left(1 - \frac{T}{100}\right) + C_4 \frac{T}{100}\right)\frac{DAd}{S}\right] \tag{2.2}$$

in which, R = annual scour failure risk ($/year), K = risk adjustment factor = $K_1.K_2$; K_1 = bridge type factor, K_2 = foundation type factor, P = annual probability of scour failure (1/year), C_1 = unit rebuilding cost ($/m^2), W = bridge deck width (m), L_b = bridge length (m), M = cost multiplier to replace bridge after scour failure, C_2 = unit cost of running vehicle ($/km.vehicle), D = detour length (km), A = average daily traffic (vehicle/day), d = duration of detouring (days), C_3 = unit value of time per adult ($/hr.adult), O = occupancy rate (adults/vehicle), T = average daily truck traffic (%), C_4 = unit value of time for truck ($/hr.vehicle), and S = average detour speed (km/hr) (Pearson et al., 2002).

Using Equation (2.3), a first estimation of annual probability of scour failure is obtained as a function of overtopping frequency and scour criticality (Pearson et al., 2002).

$$P_{tr} = (F \mid (OT\ and\ SV)) = \sum P(D_d \mid OT)P(F \mid SV\ and\ D_d) \tag{2.3}$$

where, P_{tr} = trial probability of scour failure, F = failure, OT = overtopping frequency, SV = scour vulnerability, and D_d = dimensionless depth. Overtopping frequency and scour vulnerability are explained in the next two sections.

Duration of detouring (d) and cost multiplier (M) in Equation (2.2) are both specified according to ADT. M accounts for the emergency of rebuilding of a bridge based on the importance of the inventory route. Rebuilding costs will be multiplied by M. The relation between ADT, d, and M is given in Table 2.2. As ADT value of a route on bridge increases, importance of bridge pronounces. Therefore, the bridge should be replaced in shorter durations for higher ADT values. If a bridge is intended to be replaced in shorter durations than planned for nonemergency condition, the cost of replacement will increase. Multiple shift operations may be needed in that case, and this will result in increased cost of workmanship and higher operating cost of construction equipment. Therefore, the rebuilding cost of a bridge should be increased by a specified multiplier, to reflect the emergency of the condition.

Table 2.2 Average daily traffic versus detour duration and cost multiplier (Pearson et al., 2002)

ADT (veh/day)	**Detour Duration (days)**	**M**
< 100	1095	1.0
< 500	731	1.1
< 1000	548	1.25
< 5000	365	1.5
5000 ≤ A ≤ 10000	183	2.0

2.2.1.1 Overtopping Frequency

Overtopping frequency is estimated according to the waterway adequacy and functional classification of a bridge. Overtopping frequency grades and corresponding return periods and annual overtopping probabilities for full-flow condition are presented in Table 2.3. Annual overtopping probabilities are taken as approximately the reciprocals of the average return periods in the proposed intervals. Some regression equations are proposed for various discharges corresponding to different return periods using available hydrologic data (Fletcher et al., 1977). In fact, such discharges should be obtained for every basin exhibiting different characteristics than those considered in Fletcher et al. (1977). In this thesis, a flow-frequency analysis conducted for the study area is directly used in the execution of the software.

Table 2.3 Overtopping frequency grades for full-flow (Pearson et al., 2002)

Overtopping Frequency	**Return Period (years)**	**Annual Overtopping Probability**
None	Never	Never
Remote	> 100	0.01
Slight	11 – 100	0.02
Occasional	3 – 10	0.2
Frequent	< 3	0.5

Using the dimensionless discharge and depth ratio relation given in Equation (2.4), annual overtopping probabilities corresponding to various overtopping frequency grades and dimensionless depth ratios are calculated and presented in Table 2.4 (Pearson et al., 2002). Considering an overtopping frequency, the row-wise summation of the probabilities corresponding to different dimensionless depth ratios is 1.0. These probabilities tend to increase with respect to flow depth for occasional and frequent overtopping frequency grades. Such a tendency is also observed for slight and remote overtopping frequency grades for up to 75% fullness. The last column of Table 2.4 presents overtopping probabilities for depth ratio greater than 1.0. This case may represent pressure flow conditions, i.e. the flow depth exceeds the bottom elevation of the girder but the bridge is not overtopped.

$$\frac{D_i}{D_f} = \left[\frac{Q_i}{Q_f} \right]^{0.6} \tag{2.4}$$

Here, Q_i = instantaneous discharge, D_i = normal depth of a corresponding Q_i (from Manning's equation), Q_f = full flow discharge, and D_f = depth of flow corresponding to Q_f.

Table 2.4 Annual overtopping probabilities for various grades and dimensionless depth ratios (Pearson et al., 2002)

Overtopping Frequency	**Dimensionless Depth Ratio**				
	0 – 0.25	**0.25 – 0.50**	**0.50 – 0.75**	**0.75 – 1.0**	**> 1.0**
Remote	0.12	0.48	0.31	0.08	0.01
Slight	0.12	0.34	0.43	0.09	0.02
Occasional	0.07	0.13	0.25	0.35	0.2
Frequent	0.04	0.08	0.15	0.23	0.5

2.2.1.2 Scour Vulnerability

Estimation of scour vulnerability of a bridge is generally based on field inspections, and it can be graded according to Table 2.5. Scour vulnerability can also be estimated as a function of two NBI parameters; channel and substructure conditions. These two parameters can be graded from excellent to critical conditions, and these grades give an idea about the scour criticality of a bridge. Channel condition of a bridge can be determined by "channel stability indicators", which will be explained in the next section.

Annual probability of scour failure regarding the scour vulnerability of a bridge and the dimensionless depth ratios, which are explained before, is also given in Table 2.5. Since degree of scouring is directly proportional to the bed shear stress, and hence flow depths, this probability tends to increase with respect to dimensionless depth ratio for a particular scour vulnerability condition.

Table 2.5 Scour vulnerability grades and dimensionless depth ratios (Pearson et al., 2002)

Scour Vulnerability	**Dimensionless Depth Ratio**				
	0 – 0.25	**0.25 – 0.50**	**0.50 – 0.75**	**0.75 – 1.0**	**> 1.0**
0 (Bridge failure)	1	1	1	1	1
1 (Bridge closed)	1	1	1	1	1
2 (Extremely vulnerable)	0.25	0.4	0.55	0.7	0.88
3 (Unstable foundations)	0.14	0.2	0.3	0.45	0.65
4 (Stable, action required)	0.06	0.1	0.15	0.26	0.41
5 (Stable, limited life)	0.002	0.002	0.002	0.03	0.1
6, U (Unassessed/Unknown)	0.1	0.15	0.225	0.355	0.53
7 (Countermeasure installed)	0.1	0.15	0.225	0.355	0.53
8 (Very good condition)	0.002	0.002	0.002	0.01	0.05
9 (Excellent condition)	0.002	0.002	0.002	0.002	0.01

The numbers given in the scour vulnerability column in Table 2.5 are the codes of vulnerability grades which will be used in the HYRISK implementation. Those scour vulnerability codes and corresponding explanations are given below:

* *Bridge is failed (0) – Bridge is closed to traffic (1):* in these conditions bridge is failed or failure is imminent, respectively. Bridge is closed to traffic in both conditions.
* *Bridge is scour-critical (2):* extensive scour has occurred at bridge piers and abutments.

  Bridge is scour-critical (3): bridge foundations are unstable.

  In these two conditions, scour level, y_s, is below footings or within the limits of footings with a minimal chance (See Figure 2.1 a).

* *Bridge foundations are stable (4):* although the foundations are stable, still action is required for protection.

  Bridge foundations are stable (5): the foundations are stable but still bridge has limited life.

  In these two conditions, scour level is within the limits of footings (See Figure 2.1 b).
* *Unassessed foundation (6):* bridge is not evaluated for scouring.
* *Countermeasures installed (7):* countermeasures have been installed for overcoming the excessive scouring effect. Bridge still retains its scour criticality although the countermeasures are installed.
* *Bridge foundations are stable (8):* bridge is in very good condition against scouring.

  Bridge foundations are stable (9): bridge is in excellent condition against scouring.

  In the last two conditions, scour level is above the top of the footings (See Figure 2.1 c).

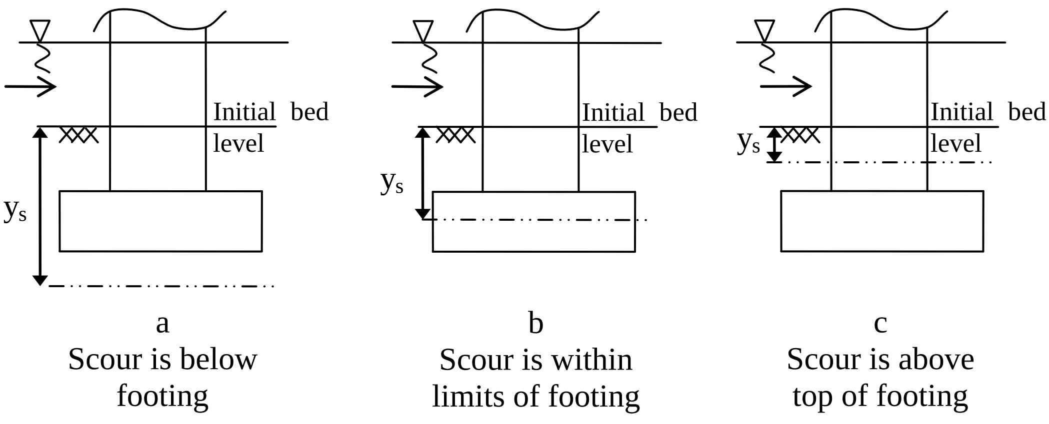


Figure 2.1 Scour levels with respect to bridge footings

Combining Tables 2.4 and 2.5 with Equation (2.3), annual failure probability of a bridge regarding scour vulnerability and overtopping frequency can be obtained, which is given in Table 2.6. Annual failure probability increases as overtopping frequency grade changes from remote to frequent for a particular scour vulnerability condition. It may be stated that the proposed failure probabilities given in Table 2.6 are defined by the program developers. Therefore, this program gives mainly a preliminary qualitative information for sites having different characteristics than those included in NBI database.

Table 2.6 Annual failure probability of a bridge (Pearson et al., 2002)

Scour Vulnerability	**Overtopping Frequency**			
	Remote	**Slight**	**Occasional**	**Frequent**
0 (Bridge failure)	1	1	1	1
1 (Bridge closed)	1	1	1	1
2 (Extremely vulnerable)	0.4573	0.4831	0.628	0.7255
3 (Unstable foundations)	0.2483	0.2673	0.3983	0.4951
4 (Stable, action required)	0.1266	0.1373	0.2277	0.2977
5 (Stable, limited life)	0.00522	0.00648	0.0314	0.05744
6, U (Unassessed/Unknown)	0.18745	0.2023	0.313	0.3964
7 (Countermeasure installed)	0.18745	0.2023	0.313	0.3964
8 (Very good condition)	0.00312	0.00368	0.0144	0.02784
9 (Excellent condition)	0.00208	0.00216	0.0036	0.006

As mentioned before, the obtained probability in Table 2.6 is a first estimation for the annual scour failure probability (trial probability). The current age of the bridge is a check for this probability. Assuming that binomial distribution is suitable for the annual failure probability, the probability is modified by Equation (2.5) (Pearson et al., 2002):

$$X_{90} = \frac{\log(1-0.90)}{\log(1-P_{tr})} \tag{2.5}$$

in which, X_{90} = 90th percentile mean time to scour failure and P_{tr} = trial probability of scour failure. In circumstances, where the current age of the bridge is greater than X_{90}, P_{tr} needs to be modified. The required modification can be assessed by applying the age of the bridge as X_{90} in the above equation. Once new P_{tr} is obtained, it is used in Equation (2.2).

2.2.2 Channel Stability Indicators

As mentioned in the previous section, scour vulnerability of a bridge can be assessed by the aid of the information about the nearby channel characteristics and substructure condition of the bridge concerned. Substructure condition includes the physical condition of the bridge components, such as piers, abutments, footings and other structural members. Observing the probable deteriorations, cracks in these members and grading their condition is not complicated at such a level of study. A methodology developed by Yanmaz et al. (2007) can be used to assess the existing level of a bridge with respect to conditions of main body, earth retaining, and serviceability components. This methodology is based on a grading system according to in-situ measurements and observations. In another study conducted by Caner et al. (2008), the remaining lifetime of existing bridges were examined. An empirical equation was derived based on extensive field surveys conducted at various bridge sites using the methodology developed by Yanmaz et al. (2007). However, assessment of the channel condition is a relatively complex task. That is why dividing it to sub-parameters provides convenience.

Johnson et al. (1999) proposed a method for rapid assessment of channel stability. In this method, there are 13 stability indicators having different weights. These indicators are graded from 1 – 12, such as excellent (1 – 3), good (4 – 6), fair (7 – 9), and poor (10 – 12). These grades are multiplied with their corresponding weights and an overall rating is obtained for the channel stability by the summation of 13 weighted grades. Yanmaz et al. (2007) also used this method for assessing the current hydraulic conditions of the channel in close vicinity of the bridge concerned. The stability indicators and their descriptions for the corresponding grade ranges are presented in Table 2.7, in which H and V are horizontal and vertical values of side inclinations, respectively, τ_0 is bed shear stress, τ_c is critical bed shear stress leading to incipient motion at the bed, α is the angle between approach flow and pier axis, and D_m is the bridge or culvert distance from meander impact point. In Table 2.8, weights of the indicators are shown, and finally in Table 2.9, overall rating ranges of channel stability are presented.

Table 2.7 Stability indicators, descriptions, and ratings (Johnson et al., 1999)

	Ratings			
Stability indicator	**Excellent (1–3)**	**Good (4–6)**	**Fair (7–9)**	**Poor (10–12)**
1. Bank soil texture and coherence	Clay and silty clay; cohesive material.	Clay loam to sandy clay loam.	Sandy clay to sandy loam.	Loamy sand to sand; non-cohesive material.
2. Average bank slope angle (Pfankuch 1978)	Bank slopes <3H:1V (18° or 33%) on both sides.	Bank slopes up to 2H:1V (27° or 50%) on one or occasionally both banks.	Bank slopes to 1.7H:1V (31° or 60%) common on one or both banks.	Bank slopes over 60% common on one or both banks.
3. Vegetative bank protection (Pfankuch 1978; Thorne et al. 1996)	Wide band of woody vegetation with at least 90% density and cover. Primarily hard wood, leafy, deciduous trees with mature, healthy, and diverse vegetation located on the bank. Woody vegetation oriented vertically.	Medium band of woody vegetation with 70-90% plant density and cover. A majority of hard wood, leafy, deciduous trees with maturing, diverse vegetation located on the bank. Woody vegetation oriented 80–90° from horizontal with minimal root exposure.	Small band of woody vegetation with 50–70% plant density and cover. A majority of soft wood, piney, coniferous trees with young or old vegetation lacking in diversity located on or near the top of bank. Woody vegetation oriented at 70–80° from horizontal often with evident root exposure.	Woody vegetation band may vary depending on age and health with less than 50% plant density and cover. Primarily soft wood, piney, coniferous trees with very young, old and dying, and/or mono-stand vegetation located off of the bank. Woody vegetation oriented at less than 70° from horizontal with extensive root exposure.

Table 2.7 continued

	Ratings			
Stability indicator	**Excellent (1–3)**	**Good (4–6)**	**Fair (7–9)**	**Poor (10–12)**
4. Bank cutting (Pfankuch 1978)	Little or none evident. Infrequent raw banks less than 15 cm high generally.	Some intermittently along channel bends and at prominent constrictions. Raw banks may be up to 30 cm.	Significant and frequent. Cuts 30–60 cm high. Root mat overhangs.	Almost continuous cuts, some over 60 cm high. Undercutting, sod-root overhangs, and side failures frequent.
5. Mass wasting or bank failure (Pfankuch 1978)	No or little evidence of potential or very small amounts of mass wasting. Uniform channel width over the entire reach.	Evidence of infrequent and/or minor mass wasting. Mostly healed over with vegetation. Relatively constant channel width and minimal scalloping of banks.	Evidence of frequent and/or significant occurrences of mass wasting that can be aggravated by higher flows, which may cause undercutting and mass wasting of unstable banks. Channel width quite irregular and scalloping of banks is evident.	Frequent and extensive mass wasting. The potential for bank failure, as evidenced by tension cracks, massive undercuttings, and bank slumping, is considerable. Channel width is highly irregular and banks are scalloped.
6. Bar development (Lagasse et al. 1995)	Bars are mature, narrow relative to stream width at low flow, well vegetated, and composed of coarse gravel to cobbles.	Bars may have vegetation and/or be composed of coarse gravel to cobbles, but minimal recent growth of bar evident by lack of vegetation on portions of the bar.	Bar widths tend to be wide and composed of newly deposited coarse sand to small cobbles and/or may be sparsely vegetated.	Bar widths are generally greater than 1/2 the stream width at low flow. Bars are composed of extensive deposits of fine particles up to coarse gravel with little to no vegetation.

Table 2.7 continued

	Ratings			
Stability indicator	**Excellent (1–3)**	**Good (4–6)**	**Fair (7–9)**	**Poor (10–12)**
7. Debris jam potential (Pfankuch 1978)	Debris or potential for debris in channel is negligible.	Small amounts of debris present. Small jams could be formed.	Noticeable accumulation of all sizes. Moderate downstream debris jam potential possible.	Moderate to heavy accumulations of various size debris present. Debris jam potential significant.
8. Obstructions, flow deflectors, and sediment traps (Pfankuch 1978)	Rare or not present.	Present, causing cross currents and minor bank and bottom erosion.	Moderately frequent and occasionally unstable obstructions, cause noticeable erosion of the channel. Considerable sediment accumulation behind obstructions.	Frequent and often unstable causing a continual shift of sediment and flow. Traps are easily filled causing channel to migrate and/or widen.
9. Channel bed material consolidation and armoring (Pfankuch 1978)	Assorted sizes tightly packed, overlapping, and possibly imbricated. Most material >4 mm.	Moderately packed with some overlapping. Very small amounts of material <4 mm.	Loose assortment with no apparent overlap. Small to medium amounts of material <4 mm.	Very loose assortment with no packing. Large amounts of material <4 mm.

Table 2.7 continued

	Ratings			
Stability indicator	**Excellent (1–3)**	**Good (4–6)**	**Fair (7–9)**	**Poor (10–12)**
10. Shear stress ratio	$\tau_0 / \tau_c < 1.0$ generally.	$1.0 \leq \tau_0 / \tau_c < 1.5$	$1.5 \leq \tau_0 / \tau_c < 2.5$	$\tau_0 / \tau_c \geq 2.5$
11. High flow angle of approach to bridge or culvert (Simon and Downs 1995)	$0° \leq \alpha \leq 5°$	$5° < \alpha \leq 10°$	$10° < \alpha \leq 30°$	$\alpha > 30°$
12. Bridge or culvert distance from meander impact point (Simon and Downs 1995)	$D_m > 35$ m	$20 < D_m \leq 35$ m	$10 < D_m \leq 20$ m	$0 < D_m \leq 10$ m
13. Percentage of channel constriction (Simon and Downs 1995)	0–5%	6–25%	26–50%	>50%

Table 2.8 Weights of the stability indicators (Johnson et al., 1999)

Stability Indicator	**Weight**
1. Bank soil texture and coherence	0.6
2. Average bank slope angle	0.6
3. Vegetative bank protection	0.8
4. Bank cutting	0.4
5. Mass wasting or bank failure	0.8
6. Bar development	0.6
7. Debris jam potential	0.2
8. Obstructions, deflectors, and sediment traps	0.2
9. Bed material consolidation and armoring	0.8
10. Shear stress ratios	1.0
11. High flow angle of approach to bridge	0.8
12. Distance from meander impact point	0.8
13. Percentage of channel constriction	0.8

Table 2.9 Overall rating ranges (Johnson et al., 1999)

Description	**Rating (R')**
Excellent	$R' < 32$
Good	$32 \le R' < 55$
Fair	$55 \le R' < 78$
Poor	$R' \ge 78$

2.2.3 HYRISK Methodology

For the implementation of the risk-based model, HYRISK Version 2.0 (Pearson et al., 2002) is used in this thesis. The software is quick and reliable to estimate annual risk of scour failure of a bridge or many bridges at the same time. As mentioned before, NBI database stores the information of bridges in United States. Information of a bridge in another country is not available in the database. Therefore, use of this approach outside the USA needs special care about the definitions and default and/or characteristic parameters used in the model. An “NBI

Data" window, shown in Figure 2.2, exists in the program and all NBI parameters related to a bridge are entered there. Based on the details of a study, either NBI parameters of many different bridges, or different combinations of parameters of one bridge could be entered using this window. Studying a single bridge with different parameter combinations may help to assess scour-induced risk level of that bridge, whereas analyzing different bridges provides a ranking for the severity of their scour vulnerabilities.

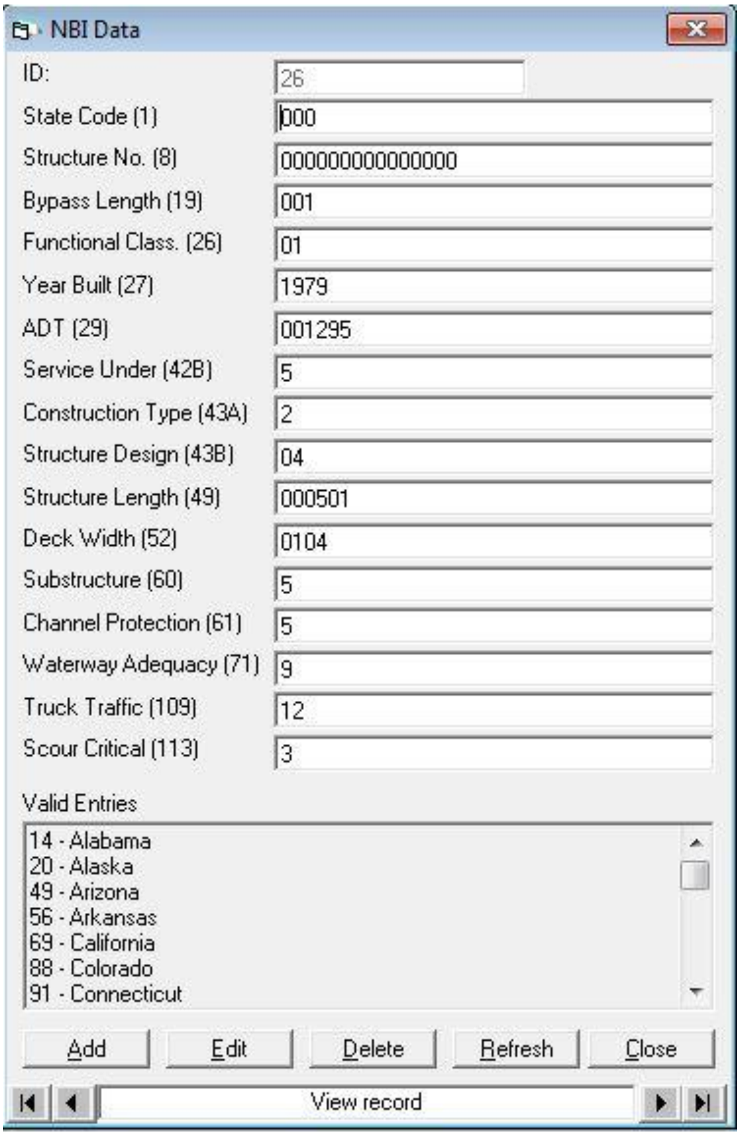


Figure 2.2 NBI data window

In NBI Data window, some parameters are entered directly. For example, bypass length is entered as its actual value in kilometers, or the construction year is directly written in "Year built" area. However, functional classification of bridge, service type under bridge, construction design type, and material used in construction, substructure condition, channel condition, waterway adequacy, and scour vulnerability have definitions rather than numerical values. Various definitions exist for each parameter and each definition has a corresponding code for quick implementation of the model (See Pearson et al., 2002). Therefore, those definitions are coded accordingly in this window.

As soon as all required NBI date is entered, various analysis assumptions, such as current year (in which the analysis is carried out), confidence limit for the calculations of expected age via Equation 2.5, which is initially set to 90% as a default in the program, K_1 and K_2 factors, average speed in the bypass route, occupancy rate, and C_1, C_2, C_3, and C_4 unit costs, are defined in another window so called "Basic Assumptions". By this way, the necessary data for the analysis are completed and the analysis can be run. The result of a single bridge is presented in a window as it is shown in Figure 2.3. Also the results for multiple bridges or multiple cases are tabulated and can be viewed together in the program.

In the analysis result window, annual scour failure probability (P), annual risk of scour failure (R), and costs associated with scour failure of a bridge are presented. Obtaining scour failure risk in monetary values provides a better understanding for the criticality of bridge scour. By this way, annual risk taken by doing nothing to scour-critical bridge can be compared with costs associated with scour failure of a bridge and also cost of countermeasures that can be applied to the bridge and the bridge site. This will be helpful for selecting the feasible countermeasure for a scour-critical bridge.

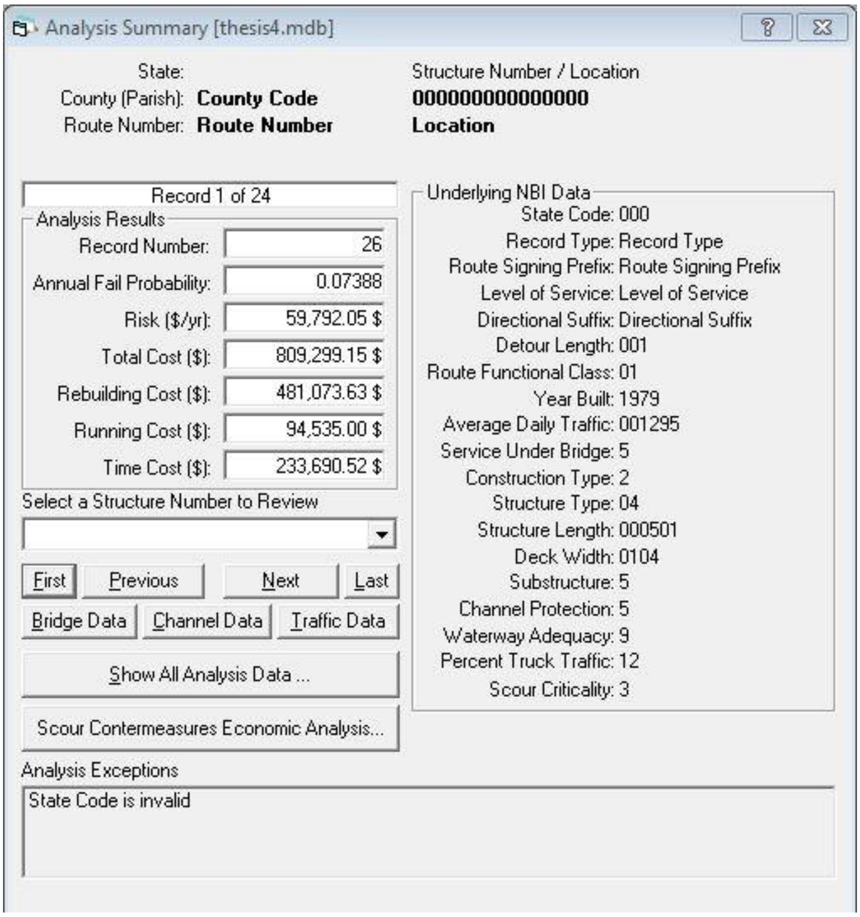


Figure 2.3 Analysis result window for a single bridge

2.3 Evaluation of Scour Countermeasures

2.3.1 General

In the absence of extensive and specific data pertinent to bridges and/or the bridge(s) under consideration, the results of HYRISK model can be used to clarify which bridges represent the greatest annual expected loss due to failure or heavy damage due to scour. On the contrary, the model itself is not capable to answer the main concern of the bridge owners: How much is reasonable to spend on scour countermeasures to protect a bridge with a known, finite life before scheduled replacement?

The Sour Countermeasures Calculator in the HYRISK model (Pearson et al., 2002) can be used to answer this question. This question can only be answered if particular information about a particular bridge site is available. In line with this argument, the calculator performs its calculations on a single bridge rather than a “set” of bridges as does the basic HYRISK model.

The Scour Countermeasures Calculator is mainly used to evaluate the economic feasibility of available scour countermeasures at a particular bridge site. The analysis used allows better accounting for the costs associated with loss of life and, knowing the service life of the structure, the time value of money.

The calculator performs its analysis using the results of the basic HYRISK analysis. However, it is possible to use the calculator itself without analysis results of HYRISK in case if real required input data is available so as to perform the economical analysis.

The analysis results are then reviewed, refined, and modified in a seven-step process, which are defined in the next section.

2.3.2 Scour Countermeasures Calculator

The steps followed in the Scour Countermeasures Calculator are as follows:

1) *Step 1: Describe the Bridge*

In this first step, the data entered to HYRISK model to calculate the annual failure probability of the bridge under consideration is initially displayed on the screen presented in Figure 2.4. The user should change this information if it differs from what is known about the bridge and further; the basic assumptions (i.e. rebuild cost, vehicle running cost, etc.) used in the HYRISK methodology can be modified if required.

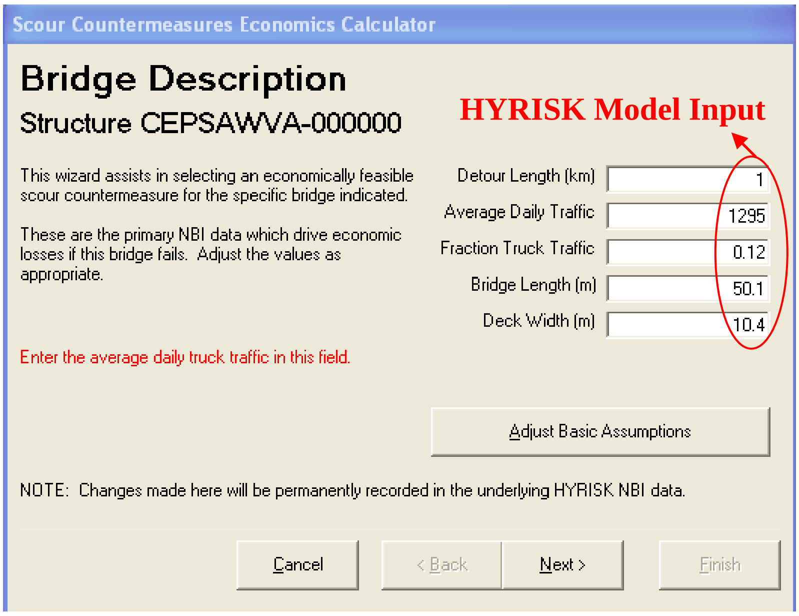


Figure 2.4 Setting bridge description

It is important to note that the changes made at this step will automatically be recorded in the active HYRISK database, which means that in the next run of the database new values for annual failure probability, rebuilt cost, etc. will be calculated.

2) *Step 2: Set Cost Multipliers*

As in Step 1, the data entered to HYRISK model to calculate the annual failure probability of the bridge under consideration is initially displayed on the screen shown in Figure 2.5 and if more appropriate values than those offered by HYRISK are known, it should be entered at this stage.

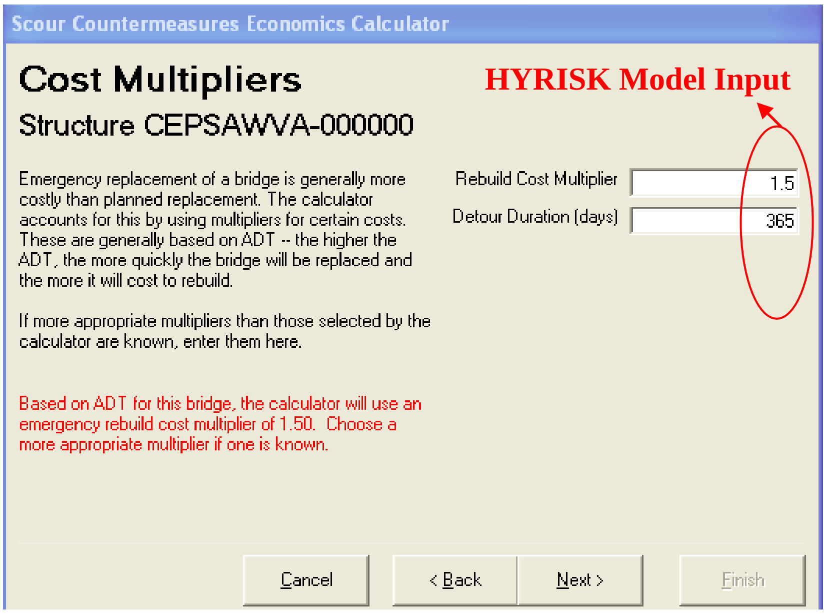


Figure 2.5 Setting cost multipliers

3) *Step 3: Quantify Economic Assumptions*

The economic assumptions to be used during scour countermeasures economic analysis are entered at this step. The costs associated with loss of life will be the product of the values set in the first three fields on the screen presented in Figure 2.6. The National Bridge Inventory data contains no information which can be used to derive this cost, so, if it is to be accounted for, it must be quantified. One reasonable approach is to estimate amounts to be awarded (or settled for) as the result of legal action. In the case study of this thesis, loss of life is ignored, therefore, the costs associated with loss of life is out of consideration.

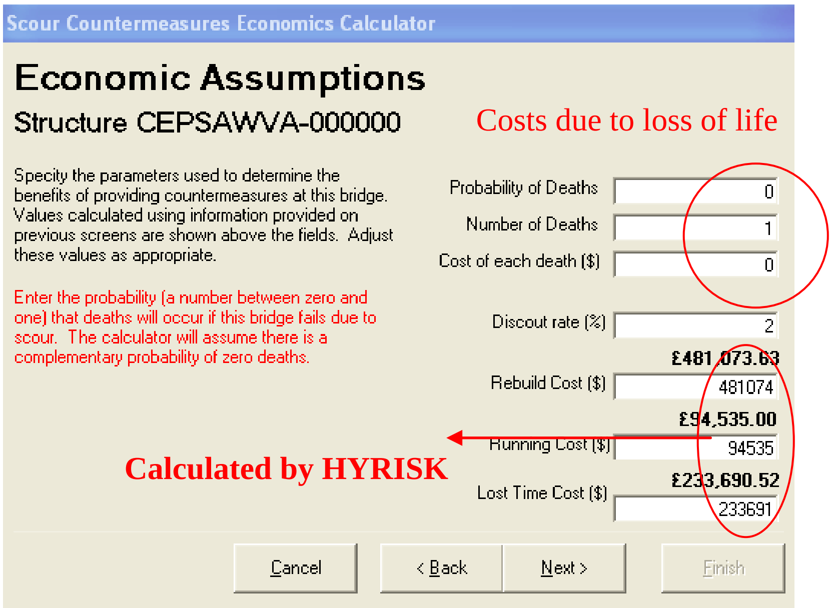


Figure 2.6 Setting economic assumptions

The discount rate is the time value of money (interest) minus inflation during the remaining life of the bridge. Owing to the fact that, US$ is used as a currency of all cost calculations, a discount rate of 2%, which is commonly accepted in Turkish engineering practice, is considered.

The displayed rebuild costs, running costs, and time lost costs are those calculated by the HYRISK methodology. In case if required, the alternate cost values can be entered.

4) *Step 4: Set the Annual Probability of Failure*

Initially, the annual probability of failure due to scour is set to that estimated by HYRISK, however, as in the previous steps; the values may be adjusted in the screen shown in Figure 2.7.

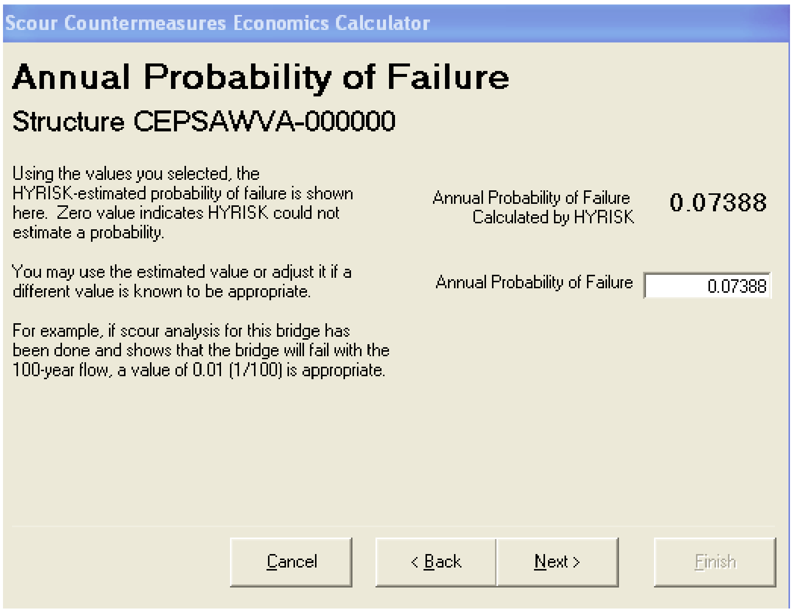


Figure 2.7 Setting annual probability of failure

5) *Step 5: Specify a Remaining Useful Life for the Bridge*

In this step, either the remaining useful life for the bridge or the lifetime probability of failure is specified. For the latter, the remaining useful lifetime will be calculated by the program automatically.

Using the annual probability of failure specified on the screen given in Figure 2.7, the probability of failure during that lifetime is calculated using the following relationships (Pearson et al., 2002):

$$P_L = 1-(1-P_A)^L \tag{2.6}$$

$$L = \frac{\log(1-P_L)}{\log(1-P_A)} \tag{2.7}$$

where, P_L = probability of failure over the expected life of the bridge, P_A = annual probability of failure calculated by HYRISK or supplied by the modeller, and L = remaining useful life of bridge (year).

Using the equation(s) given above, the calculator automatically generates both a table and a graph showing the probabilities of failure at five-year intervals up to 100 years, as given in Figure 2.8.

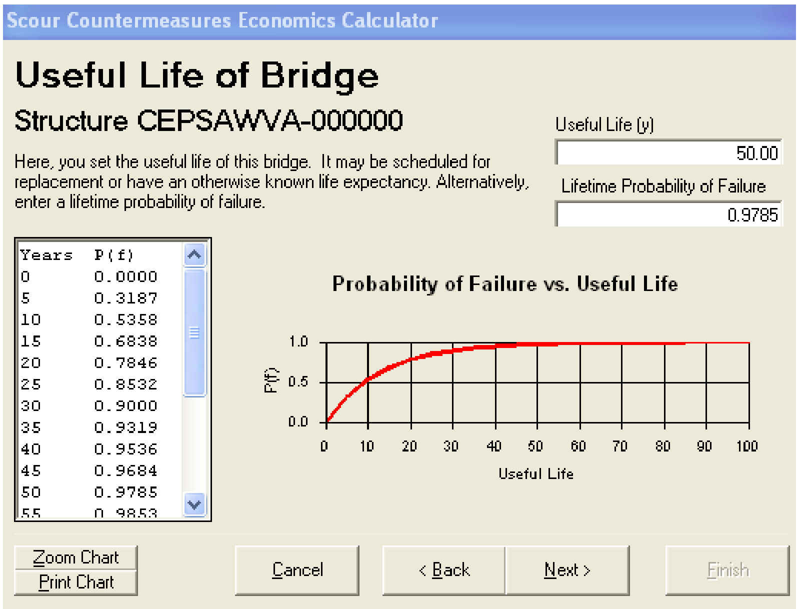


Figure 2.8 Setting useful life of bridge

6) *Step 6: Economic Risks*

A reasonable measure of resources appropriate for protection of a particular bridge is the present value benefit of any countermeasure contemplated.

Using the information specified in the previous steps, the present values of the economic risks of scour failure for the bridge under consideration are shown in Figure 2.9. For each return period, two values are shown; one accounts for loss-of-life costs while the other precludes them.

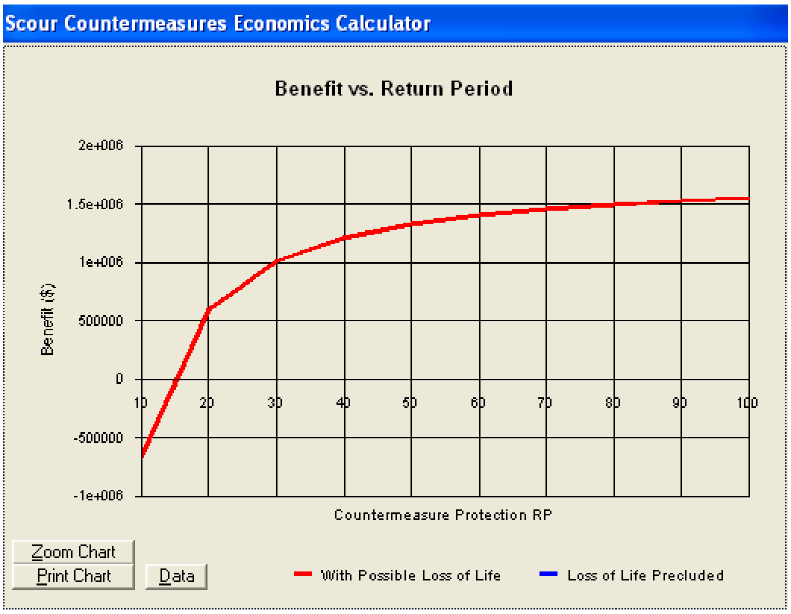


Figure 2.9 Presentation of economic risks

The following relationships are used to perform these calculations (Pearson et al., 2002):

$$C_E = \frac{C_R}{(1+i)^L} \tag{2.8}$$

where, C_E = present value of expected rebuilding cost ($), C_R = current rebuilding cost ($), i = discount rate (%), and L = remaining useful life of bridge (year); and

$$M_p = \frac{(1+i)^L - 1}{i(1+i)^L} \tag{2.9}$$

where, M_p = present value given annual cost multiplier; and

$$B_1 = \left(M_p \left(P_A C_f - \frac{C_f}{RP} \right) \right) + C_E (P_L - P_L') \tag{2.10}$$

where, B_1 = present value benefit accounting for loss of life ($), C_f = cost of failure, including injury and loss of life ($), P_A = annual probability of failure without protection, RP = return period protection desired (year), and P_L' = probability of failure over the extended life of the protected bridge; and

$$B_2 = \left(M_p \left(P_A C_f - \frac{C_f - D_L}{RP} \right) \right) + C_E \left(P_L - P_L' \right) \tag{2.10}$$

where, B_2 = present value benefit precluding for loss of life ($) and D_L = costs associated with loss of life ($).

The last step of scour countermeasures economic calculator is used to evaluate the net benefit and hence benefit/cost ratio of each user-defined scour countermeasure. The NBI Data do not store any information to derive costs of scour countermeasures. Therefore, these costs shall be calculated and entered manually to the program.

The methodology described in this chapter is applied to a case study, which includes further details of last step of scour countermeasure calculator, and it is presented in Chapter 3.

CHAPTER 3

CASE STUDY

The objective of this thesis is to investigate the failure risk of river bridges and evaluate the feasibility of installing scour countermeasures. For this purpose, a case study is carried out for the demonstration of this evaluation. Fol-1 Bridge is selected for this study, which is a highway bridge crossing Fol Creek.

3.1 Fol Creek Basin and Flood Frequency Analysis

Fol Creek Basin is located in the Black Sea Region which is a flood prone zone in Turkey. Bahadırlı (No. 2228) flow gauging station, with a catchment area of 191.4 km^2, operated by Electrical Power Resources Survey and Development Administration (EİE) since 1960 is located near the outlet of the basin. The station is still in operation, and the maximum flood estimated at this station is 412 m^3/s in 1990. The 1990 flood caused extensive damage to the settlement in close vicinity of Fol Creek, as seen in Figure 3.1. Using a frequency analysis method named L-moments, Bilen (1999) carried out a uni-variate flood frequency analysis in which annual maximum flows were considered as a series. Bi-variate flood frequency analysis of Fol Creek was also carried out by Yanmaz and Günindi (2006) and Yanmaz et al. (2008) in which annual peak discharges and flood volumes were used in the frequency analysis using multi-variate probability density functions. Since uni-variate frequency analysis results are also covered by discharge ranges of the multi-variate analyses, the analysis conducted by Bilen (1999) is accepted to be representative for the basin and is used in this study. Out of 12 different probability distributions, Wakeby distribution was selected as the suitable one. The results of the flood frequency analysis for Wakeby distribution is presented in Table 3.1, in which T_r is return period and Q_i is the discharge corresponding to this return period.

Figure 3.1 Highway is damaged due to flood (Tuna, 2008)

Table 3.1 Flood frequency analysis for Fol Creek Basin (Bilen, 1999)

Q_i	Return Period, T_r (years)	Discharge (m^3/s)
Q_2	2	56.7
Q_5	5	93.3
Q_{10}	10	117.3
Q_{25}	25	145.2
Q_{50}	50	163.8
Q_{60}	60	168.4
Q_{70}	70	172.2
Q_{80}	80	175.4
Q_{90}	90	178.1
Q_{100}	100	180.6
Q_{150}	150	189.6
Q_{200}	200	195.6
Q_{300}	300	203.7
Q_{400}	400	209.1
Q_{500}	500	213.1

In a study carried out by Tuna (2008), it is estimated that the maximum discharge during the passage of 1990 flood is 246 m^3/s. He obtained the maximum annual flows synthetically by Mockus method and he calculated the return period of this discharge in a range of 10-25 years, where Q_{100} discharge is 498 m^3/s. At this point, it should be stated that the results of frequency analysis conducted using different approaches are subject to high levels of uncertainties.

Another important point is that Bahadırlı station is close to the outlet of the basin and the Fol-1 Bridge is almost at 1.5 km upstream of this station. Within this distance, additional surface water flow might have joined to the main channel from the sides. Therefore, the maximum flow occurred at the bridge site may be less than the maximum flow expected at the station. However, the annual maximum flows accepted at the station are used in the analyses carried out for the Fol-1 Bridge.

3.2 Determination of Water Surface Profiles

Water surface profile computations are needed to check bridge waterway adequacy and to decide on degree of bed and bank protection facilities. For the water surface profile calculations of Fol Creek, HEC-RAS software (Version 3.1.3 released in 2005) (Brunner, 2002) is used in this thesis. HEC-RAS is a well-known hydraulic analysis software, which is capable of one dimensional steady flow calculations, unsteady flow simulations, sediment transport calculations, and scour analysis (Brunner, 2002). Sinuosity of Fol Creek, which is the ratio of thalweg length to valley length of a river reach under consideration, is calculated as 1.08 using the information presented in Bilen (1999). According to its sinuosity, the study reach along Fol Creek is almost straight, so HEC-RAS was used with confidence. To determine the water surface profiles, cross-sections of Fol Creek are needed, which are obtained from Bilen (1999) and some of them are modified based on the field inspections carried out by the author. Furthermore, characteristics of the bridge are necessary inputs for water surface profile calculations. The cross-section data of Fol-1 Bridge are also obtained from Coşkun (1994) and Bilen (1999). Detailed information of Fol-1 Bridge will be given in Section 3.3.

Water surface profiles of Fol Creek are calculated by implementing a mixed regime steady flow analysis in the software. Calculations are made using energy equation and momentum equation, which is necessary for mixed flow regime calculations (Bilen, 1999). For each discharge given in Table 3.1, the corresponding water surface profile of Fol Creek is calculated and the analysis results for just upstream of the bridge are tabulated in the Appendix A. Characteristic input data are also provided in the Appendix A.

3.3 Description of Fol-1 Bridge

The selected bridge for this study, Fol-1 Bridge, was constructed in 1979. The bridge is located on Route 61-77, which is the road between Vakfıkebir and Tonya counties in Trabzon (See Figure 3.2). The route is indicated as green line and the bridge is highlighted inside the blue oval in Figure 3.3. Actual location of the bridge is 1+442 km upstream from the centre of Vakfıkebir (along the roadway) and 1+600 km upstream from the outlet of the Fol Creek Basin (along the river) (Coşkun, 1994), (Yanmaz and Selamoğlu, 2010).

Figure 3.2 A view of the study bridge from downstream

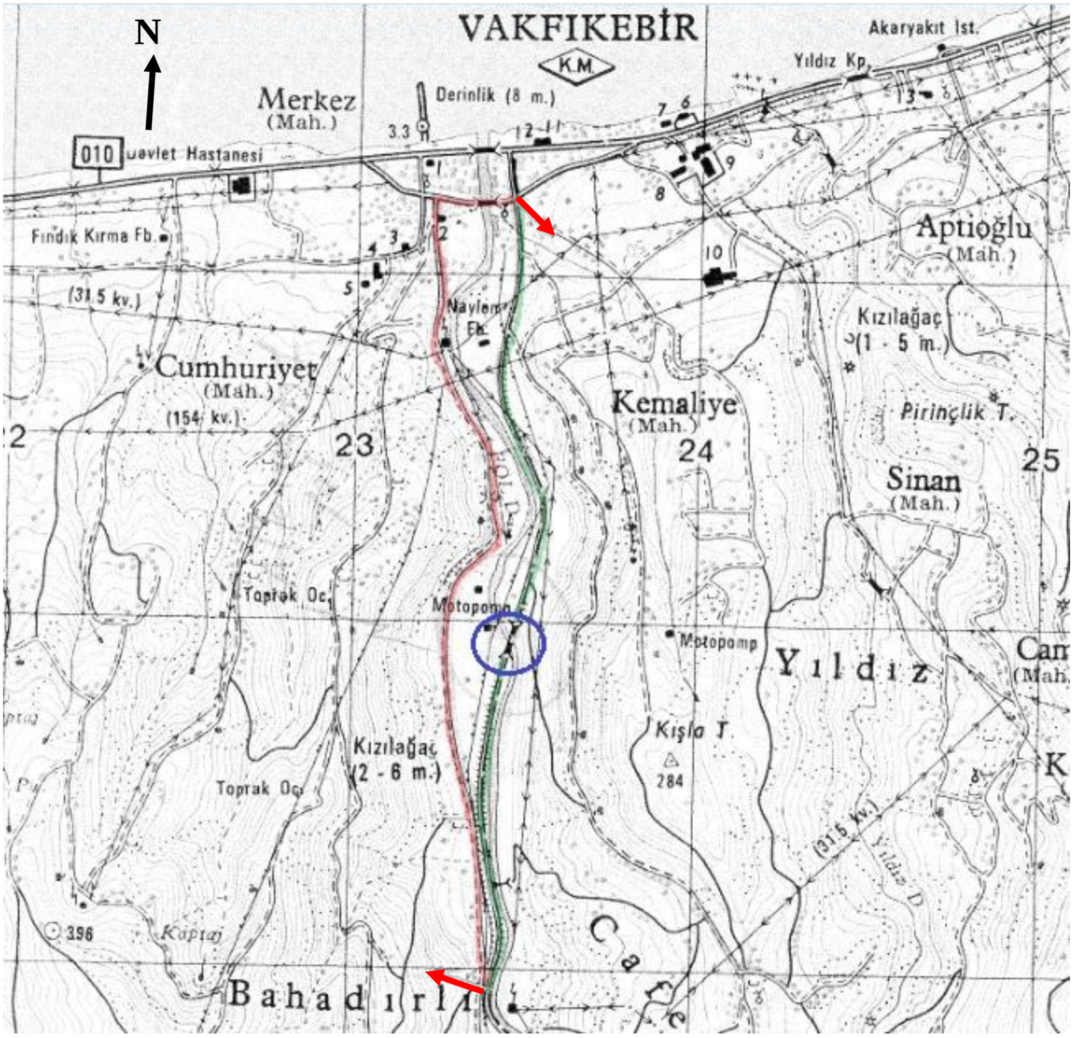


Figure 3.3 Map of the case study area

Fol-1 is a simple spanned and concrete tee beam bridge. The bridge slab is 50.1 m long and 10.40 m wide. The bridge is supported by vertical wall abutments on both sides and intermediary piers as shown in Figure 3.4, which presents the longitudinal section of it. There are two groups of cylindrical piers in between the abutments (See Figure 3.5). Each pier group is composed of 4 cylindrical piers having 1 m diameter (See Figure 3.6). The bridge deck is 58° skewed to the flow, whereas the piers and abutments are aligned with the flow (See Figure 3.7).

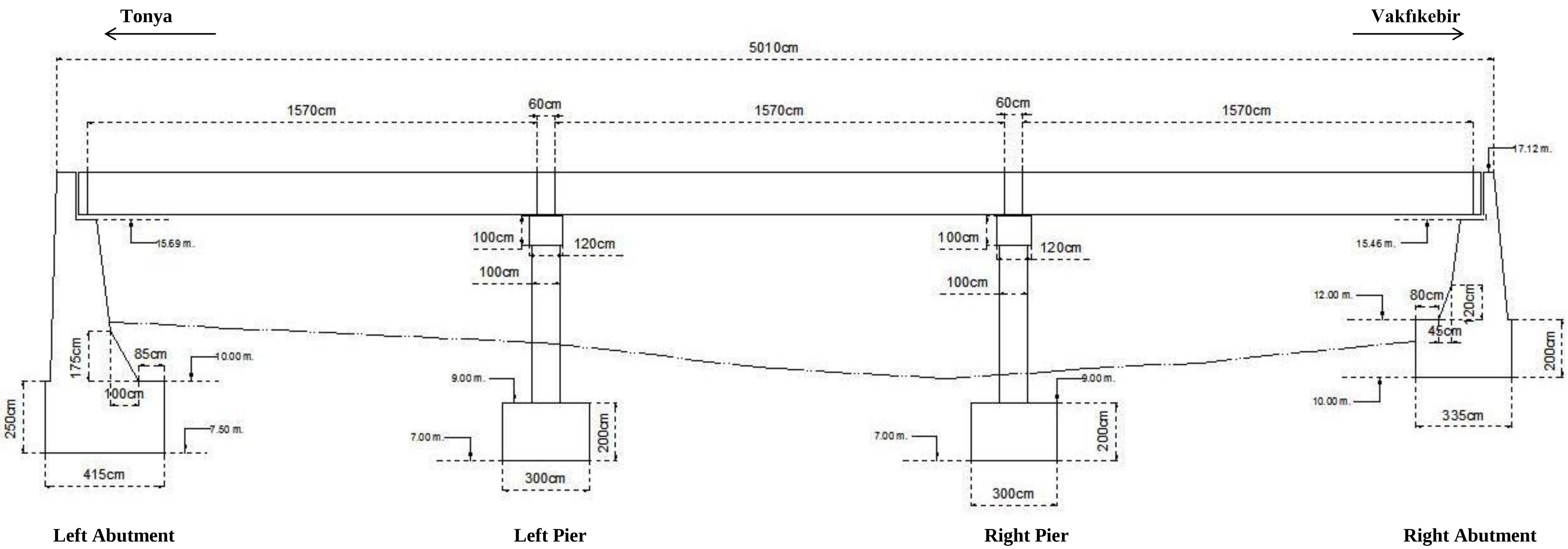


Figure 3.4 Longitudinal section of the bridge (from upstream)

Figure 3.5 General view of Fol-1 Bridge

Figure 3.6 Right piers and abutment (Vakfikebir side)

Figure 3.7 Bridge deck skewed to flow

3.4 Field Trip

To obtain information about the basin, bridge, and the bridge site, a field trip was made on 16.10.2008. Field inspections were made with the assistance of the head of 221st Department Directorate of State Hydraulic Works (DSİ) in Trabzon. Besides, a meeting was held with the head of Department of Bridges in 10th Local Administration of General Directorate of Highways (KGM) in Trabzon to obtain information about the characteristics of the bridge and inventory route. That information is given in Section 3.5.1. In the field trip, it is observed that there is long term bed degradation compared to bed elevations obtained from Coşkun (1994). Excessive debris accumulation between piers is conspicuous and is presented in Figure 3.8.

Figure 3.8 Excessive debris accumulations between piers

In addition, inspections on the channel stability indicators were made in the field trip. The information gathered is given in Section 3.5.2.2. Observations made on the bridge site are also used in scour countermeasure determinations. In this thesis, analyses are carried out in 2009, and it is assumed that the observations made in the field trip are still valid in 2009.

3.5 HYRISK Parameters of the Case Study

In the risk-based model, NBI parameters are needed as mentioned in Section 2.2.1. Those parameters and their definitions for this study are given in the subsequent sections. Then, analysis assumptions will be given. The definitions and corresponding codings of those parameters are based on Pearson et al. (2002).

3.5.1 NBI Parameters Related to Fol-1 Bridge

* *Functional classification:* functional classification of the inventory route, which is Route 61-77 in the study, is defined as “rural principle arterial – interstate” based on the specifications of FHWA (1989). Coded as 01.
* *Type of service:* service type on the bridge is highway and under the bridge is waterway. Coded as 1 and 5, respectively.
* *Average daily traffic:* previous years’ ADT values are taken from the 10th Local Administration of General Directorate of Highways (KGM) in Trabzon in 2008. By making linear extrapolation, the ADT value in 2009 is estimated as 1295 vehicles/day. Coded as 001295.
* *Average daily truck traffic:* based on the information taken from KGM, ADTT is estimated to be 158 trucks/day, which is 12% of ADT. Coded as 12.
* *Construction type:* this parameter stands for the material used in the construction, which is concrete continuous for Fol-1 Bridge. Coded as 2.
* *Structure design:* design type of this bridge is tee beam. Coded as 04.
* *Structure length:* this is the length of bridge deck and is 50.1 m, as mentioned before. Coded as 000501.
* *Deck width:* width of the bridge deck is 10.40 m. Coded as 0104.
* *Bypass length:* in case of bridge closure, a bypass route is determined, which is a roadway parallel to Route 61-77 and passing through villages between Vakfıkebir and Tonya. Bypass route is indicated as the red line in Figure 3.3 and detouring is between the red arrows on the map. The additional length is measured as 325 m, from 1/25000 scaled map of the study area. But in HYRISK, the minimum acceptable bypass length is 1 km, therefore bypass length for this study is assumed as 1 km. Coded as 001.

3.5.2 Substructure and Channel Condition, Waterway Adequacy, and Scour Criticality of Fol-1 Bridge

Four NBI parameters that are not mentioned in the previous section; substructure and channel condition, waterway adequacy, and scour criticality are explained in this section.

3.5.2.1 Substructure Condition

Physical condition of the structural components of a bridge may be estimated by field inspections and monitoring, which was handled by visual inspections in the field trip. Although some minor cracks and deterioration have been observed, the components of the bridge were still sound. Therefore, the substructure components are said to be in fair condition, which is coded as 5 (Pearson et al., 2002).

3.5.2.2 Channel Condition

Assessment of the channel stability condition is carried out using channel stability indicators. Based on the observations during the field trip, using Table 2.7, the indicators for Fol-1 Bridge site are determined:

1. According to the study carried out by Coşkun (1994), the bed material is coarse sand with a median grain size of D_{50} = 0.75 mm. Possible deviations from this value, as a result of progression of annual floods, were ignored and this median size was used in the computations of the critical bed shear stress and scour depths at bridge elements.
2. Average bank slope is 2.5H:1V on Vakfıkebir side where it is almost 15H:1V (See Figure 3.2) on Tonya side.
3. There is a medium band of woody vegetation (See Figure 3.9).
4. Some bank cutting exists, less on Tonya side, more on Vakfıkebir side.
5. Mass wasting is observed and channel width is quite irregular (See Figure 3.10 for the upstream of the bridge).
6. Some bar development is observed without vegetation.

7. High debris accumulation exists, especially between piers (See Figure 3.8).
8. Obstructions and flow deflectors cause potential minor bank erosion.
9. Channel bed material is loose and almost all material size is less than 4 mm according to grain size distribution obtained from Coşkun (1994). The assumption made in item 1 is also applicable for grain size distribution.
10. The critical bed shear stress, τ_c, for D_{50} < 1 mm can be obtained from Equation (3.1) (Yanmaz, 2002).

$$\tau_c = \rho_w \left[0.0115 + 0.0125 D_{50}^{1.4}\right]^2 \qquad (3.1)$$

where, τ_c = critical bed shear stress (Pa), ρ_w = water density (kg/m^3), and D_{50} = median grain size (mm). With this information, τ_c is calculated as 0.394 Pa, which is relatively smaller than the bed shear stresses occuring under various discharges which are to be discussed in Section 3.8.1. Shear stress ratio is, therefore, much greater than 2.5.

11. Bridge piers and abutments are almost aligned with the approach flow.
12. Upstream of the bridge is very close to a meander impact point as presented in Figure 3.9.
13. Channel is constricted about 50% in close vicinity to the bridge.

The indicators defined above are rated using Table 2.7 and an overall rating is obtained for the channel stability. Ratings are presented in Table 3.2. Weighted ratings are obtained as the product of rating of each indicator and the corresponding weight, which is tabulated in Table 2.8.

Table 3.2 Rating of stability indicators in close vicinity to Fol-1 Bridge

No	Stability Indicator	Rating	Weighted Rating
1	Bank soil texture and coherence	11	6.6
2	Average bank slope angle	5	3.0
3	Vegetative bank protection	4	3.2
4	Bank cutting	6	2.4
5	Mass wasting or bank failure	7	5.6
6	Bar development	5	3.0
7	Debris jam potential	9	1.8
8	Obstructions, flow deflectors, and sediment traps	5	1.0
9	Channel bed material consolidation and armoring	11	8.8
10	Shear stress ratio	12	12
11	High flow angle of approach to bridge or culvert	2	1.6
12	Bridge or culvert distance from meander impact point	12	9.6
13	Percentage of channel constriction	9	7.2
		OVERALL RATING	65.8

The overall rating is obtained as 65.8 for the channel stability condition. Using Table 2.9, this rating can be described as fair. In HYRISK, fair condition is explained as “the primary structural components are sound but may have minor section loss, cracking, spalling, or scour” (Pearson et al., 2002). Furthermore, bank protection is eroded and embankment has damage. This condition is coded as 5 in HYRISK.

Figure 3.9 A view of the bridge site from its upstream

It is indicated that almost all of the bed material is smaller than 4 mm in diameter. However, very coarse material is observed at the bridge site as presented in Figure 3.10. The photograph presented in the figure is taken during the field trip, which was made in the low-flow season of Fol Creek. In high flow season, upstream bed and bank material is transported through downstream of the river. Then, in low-flow season, temporary aggradation coarse upstream material occurs, whereas the fine upstream material continuously transported. As flow conditions change, irregularly accumulated coarse material is transported. Therefore, the coarse material at the bridge site is expected to be transported in the next high-flow season.

Figure 3.10 Mass wasting and aggradation in the upstream of Fol-1 Bridge

3.5.2.3 Waterway Adequacy

According to water surface profile calculations tabulated in Appendix A, overtopping is not expected at the bridge even for Q_{500} discharge (213.1 m^3/s). Also based on the information obtained from 10th Local Administration of General Directorate of Highways, no overtopping was observed at Fol-1 Bridge since it was constructed. As mentioned before, the maximum discharge recorded at Bahadırlı Station is 412 m^3/s, which is much greater than Q_{500} discharge. Even at this maximum discharge, no overtopping was observed. Therefore, the overtopping frequency is said to be remote with a return period greater than 100 years (See Table 2.3).

Waterway adequacy is coded according to overtopping frequency and functional classification together. Remote chance of overtopping frequency and rural principle arterial – interstate roadway is coded as 9 in HYRISK. In this study, although overtopping frequency of Fol-1 Bridge is obviously remote, to be able to observe the effect of waterway adequacy on failure risk, slight and occasional chance of overtopping are also considered. The expected effect of waterway adequacy will be discussed in Section 3.6.1. Together with the functional classification of Route 61-77, slight and occasional chance of overtopping result in waterway adequacy codes 8 and 4, respectively.

3.5.2.4 Scour Criticality

Scour criticality of a bridge is the most important parameter for the assessment of the risk of scour failure. As mentioned before, channel and substructure conditions of a bridge are quite assistive for evaluating the scour criticality. However, in this study, as indicated in the previous section, risk of the scour failure of Fol-1 Bridge is studied for different combinations of waterway adequacy and scour criticality. As presented in Table 2.6, scour criticality codes are ranging between 0 – 9. Considering the conditions of Fol-1 Bridge, the analysis is carried out for scour criticality codes: 2, 3, 4, 5, 7, and 8. In Section 3.6.1, why some intermediary codes are discarded will be discussed, and effect of scour criticality on failure risk will be evaluated.

3.5.3 Analysis Assumptions

As indicated before, together with the NBI parameters, some assumptions which are included in the risk-based model (Equation (2.2)) are determined for the case study. These assumptions and their explanations are listed below:

* *Current year:* the year when the analysis is carried out. All types of analyses are carried out in 2009.
* *Risk adjustment factor (K):* $K = K_1.K_2$

  K_1, bridge type factor is 1.0 for simple spanned bridges. K_2, foundation type factor has a default value of 1.0 (Pearson et al., 2002). Therefore, K is obtained as 1.0.
* *Detour speed (S):* based on the information gathered from KGM, allowable speed in the bypass route is 50 km/hr.
* *Occupancy rate (O):* occupancy rate is assumed as 3 adults/vehicle based on

the information gathered from KGM.

* *Unit rebuilding cost (C_1):* cost of rebuilding the same bridge is obtained by a detailed calculation using 2009 unit costs and presented in Section 3.5.3.1. Unit rebuilding cost is calculated as 615.53 \$/m^2.
* *Unit cost of running vehicle (C_2):* this cost is obtained by estimating the average fuel consumption and depreciation of the vehicle (Yanmaz and Selamoğlu, 2010). It is assumed as 0.20 \$/km.
* *Unit cost of time per adult and truck (C_3 and C_4):* value of time for people and for trucks for commercial purposes is uncertain for the study area. Therefore, the default values proposed by HYRISK are used in this study, which reflect average values for the United States. C_3 is assumed as 8 \$/hr and C_4 is assumed as 30 \$/hr. However, a comprehensive survey is needed in future to obtain more realistic values according to region-specific conditions in Turkey.

Together with these assumptions listed above, detour duration (d) and cost multiplier (M) are determined for the analysis. Based on Table 2.2, for ADT of 1295 vehicle/day, d = 365 days and M = 1.5. This means that in case of bridge failure, the bridge should be replaced in 365 days, and this replacement would result in 50% higher rebuilding cost.

3.5.3.1 Unit Rebuilding Cost (C_1) Calculations

In case of failure of bridge or irreversible damage to bridge, it should be replaced. Unit rebuilding cost in risk calculations is found by calculating the cost of constructing the same bridge at the same location. A detailed study has been carried out with the assistance of bridge design engineers in Yüksel Domaniç Engineering Limited Company, to obtain the rebuilding cost of Fol-1 Bridge. Calculations are based on the fact that if the bridge is failed, exactly the same bridge will be replaced there. Items related to construction of a bridge are defined, and their dimensions are obtained from the longitudinal section of the bridge shown in Figure 3.4. Also, some necessary information on these dimensions is gathered from the headquarters of

KGM in Ankara. Calculations are made using 2009 unit prices of the items, which are defined by KGM. An exchange rate of US$/TL of 1.50 is considered.

In Tables 3.3 and 3.4, calculations for the major bridge components, such as superstructure (deck), cap on top of the piers, piers, abutments, and foundation (footing) under the piers are considered. Multiplying the area, length/height, and quantity of the component, a total volume of concrete is obtained. Weight of reinforcement per cubic meter of concrete used in these components is also presented. Concrete volume and reinforcement weight of the items are multiplied with their corresponding unit prices of installation. Summation of these costs gives the total cost of concrete and reinforcement work in these items. The total costs are presented in Table 3.4.

Table 3.3 Concrete and reinforcement quantities used in the major bridge components

Item	**Cross-section/ Plan (*)Area (m^2)**	**Length/ Height (*) (m)**	**Quantity**	**Volume (m^3)**	**Reinforcement (kg/m^3)**
Superstructure	5.100	50.1	-	255.51	99.25
Cap	1.200	13.4	2	32.16	78.40
Pier	0.785	4.6	8	28.90	78.40
Abutment	-	-	-	358.00	117.70
Foundation(*)	36.420	2	2	145.68	39.25

Table 3.4 Costs of concrete and reinforcement work in major bridge components

Item	**Concrete Volume (m^3)**	**Pose No.**	**Unit Price ($)**	**Reinf. Weight (tons)**	**Pose No.**	**Unit Price ($)**	**Total Cost ($)**
Superstructure	255.51	16.133/K-1	129.79	20.44	23.015/K	299.37	39,282.59
Cap	32.16	16.133/K-2	161.87	2.57	23.015/K	299.37	5,975.22
Pier	28.90	16.133/K-2	161.87	2.31	23.015/K	299.37	5,369.68
Abutment	358.00	16.133/K-2	161.87	42.96	23.015/K	299.37	70,811.60
Foundation	145.68	16.133/K-2	161.87	11.65	23.015/K	299.37	27,069.35

In addition to concrete and reinforcement work costs tabulated above, calculations for cost of materials, such as railing, expansion joint, pavement, deck water proofing, elastomeric bearing are made. Also costs associated with transportation of cement and reinforcement, as well as excavation for footings, which is assumed to be approximately double of the foundation volume considering certain excavation slope inclination, are calculated by multiplying their amounts with corresponding unit prices. These items and their costs are given in Table 3.5.

Table 3.5 Costs of materials in bridge construction

Item	**Amount**	**Unit**	**Pose No.**	**Unit Price ($)**	**Total Cost ($)**
Railing	9.00	tons	23.176/K	2,288.60	20,597.40
Expansion Joint	21.00	m	28.010/K	312.27	6,557.61
Pavement	521.04	m^2	4475	25.95	13,520.99
Cement Supply	266.58	tons	3000	86.00	22,925.88
Deck Water Proofing	521.04	m^2	3650	10.15	5,290.12
Elastomeric Bearing	45.00	d^3	3805	50.41	2,268.32
Foundation Excavation	341.55	m^3	14.124	54.49	18,610.28
Reinforcement Supply	79.93	tons	3790	508.00	40,604.44

Total costs presented in Tables 3.4 and 3.5 are added up and total cost is obtained as $278,883. The cost becomes $320,716 by addition of 15% contingencies. Dividing this total cost by the bridge deck area, which is 521.04 m^2, unit rebuilding cost, C_1 is found as 615.53 $\$/m^2$.

3.6 Implementation of the Risk-Based Model for the Case Study

All necessary NBI data and analysis assumptions are gathered. At this stage, the risk of scour failure of Fol-1 Bridge is analyzed via HYRISK software. The results of the analysis are presented in Table 3.6.

Rebuilding cost of the bridge is calculated from the software as $481,074. In the previous section, total rebuilding cost is calculated as $320,716. This difference is due to effect of emergency cost multiplier (*M*), since rebuilding cost is multiplied with M = 1.5 based on the ADT value of Fol-1 Bridge. Additional running cost in detouring is $94,535 and additional time cost for people and trucks together is $233,690. Additional time cost is minimized in the case study, since allowable speed is used in the model, rather than average speed. Summation of these costs give the total cost associated with the bridge failure and is found as $809,299.

Table 3.6 Risk of scour failure of Fol-1 Bridge

Waterway Adequacy	**Scour Criticality**	**Trial Failure Probability**	**Annual Failure Probability**	**Risk ($/yr)**	**Expected Age (yr)**
4	2	0.628	0.074	59,792	2.3
4	3	0.398	0.074	59,792	4.5
4	4	0.228	0.074	59,792	8.9
4	5	0.031	0.031	25,412	72.2
4	7	0.313	0.074	59,792	6.1
4	8	0.014	0.014	11,654	> 100
8	2	0.483	0.074	59,792	3.5
8	3	0.267	0.074	59,792	7.4
8	4	0.137	0.074	59,792	15.6
8	5	0.006	0.006	5,244	> 100
8	7	0.202	0.074	59,792	10.2
8	8	0.004	0.004	2,978	> 100
9	2	0.457	0.074	59,792	3.8
9	3	0.248	0.074	59,792	8.1
9	4	0.127	0.074	59,792	17.0
9	5	0.005	0.005	4,225	>100
9	7	0.187	0.074	59,792	11.1
9	8	0.003	0.003	2,525	> 100

In Table 3.6, the analysis results are given for 18 combinations of waterway adequacy and scour criticality. Trial failure probabilities are obtained from

Table 2.6. Expected age of the bridge is calculated by Equation (2.5). In Table 3.6, “> 100” stands for very long expected ages. In the conditions where expected age is less than 30, which is the actual age of bridge, the trial failure probability is modified to obtain annual failure probability. Otherwise, annual failure probability remains the same as trial failure probability. The annual failure risk is calculated using modified annual failure probability, which is calculated by setting the X_{90} age to the actual bridge age, 30 years in Equation (2.5). Since the actual bridge age is used in the equation, the annual failure probability obtained is constant, and it is 0.074. That is why the risk does not change at certain scour criticality ratings where the expected bridge age is less than the actual bridge age, although their trial failure probabilities vary. Further discussions for the obtained results are given in Chapter 4.

3.6.1 Sensitivity Analysis

To evaluate the effect of waterway adequacy on risk computation, various waterway adequacy codings are considered. In fact, such evaluations should not be carried out considering only the present circumstances. Possible alterations in channel conditions may occur in the future especially during or after the passage of severe floods, and mis-use of the river. Therefore, it may be possible to have various coding levels for waterway adequacy. That is why different waterway adequacies are considered in the analysis. Codings corresponding to remote, slight, and occasional chance of overtopping are considered. Frequent chance of overtopping leads to same results as occasional chance gives, therefore it is not necessary to present in the analysis.

Similarly, scour criticality of the bridge may change in the future, may be because of long term degradation, short term irregular aggradation around the bridge, which may alter characteristics of sediment-laden flow, nearby channel mining activities, etc. Therefore possible criticality codings are analyzed in the study. If the bridge is closed to traffic or will be closed in future, HYRISK does not calculate a failure risk since the bridge will be replaced immediately. Therefore, codes 0 and 1 are discarded. Code 6 stands for unknown/unassessed foundations, which is an invalid condition for Fol-1 Bridge, so it is excluded from the analysis. Scour is observed around the study bridge foundations, so the bridge is somehow scour vulnerable. Even if scour countermeasures are installed, excellent condition of scour criticality (coded as 9) cannot be reached in the future, and it is excluded from the analysis. Although the bridge has not failed, it still holds a probability of failure because of possible aforementioned alterations in its vicinity. Therefore, implementation of proper scour countermeasures would decrease failure risk.

As presented in Table 3.6, for some scour criticality levels, risk of failure changes as waterway adequacy changes. In Figure 3.11, the relation between waterway adequacy and failure risk is presented for different scour criticality, S.C., levels.

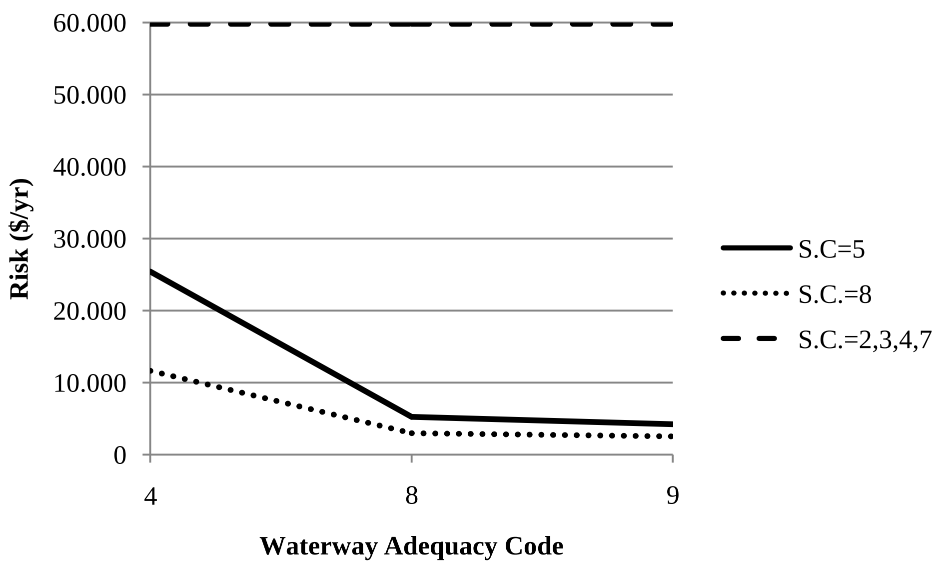


Figure 3.11 Risk versus waterway adequacy for different scour criticalities

For scour criticality codes 2, 3, 4, and 7, risk is constant and is $59,792 per year. Rate of decrease in risk of failure between waterway adequacy codings 8 and 9 is almost equal for scour criticality ratings 5 and 8. However, the mentioned rate between waterway adequacies 4 and 8 is considerably different for presented scour criticalities. It is deduced that, for good condition of scour criticality, effect of waterway adequacy on failure risk is relatively small.

3.7 Scour Calculations

Risk of scour failure of Fol-1 Bridge is calculated for various scour criticality values above. Although scouring is a complicated mechanism due to complexity of the river flow, acceptable scour determination can be made using HEC-RAS software. Hydraulic design part of the software is utilized for bridge scour calculations. In this study, the effect of contraction scour is ignored.

1) *Pier scour:* for the pier scour computations, additional information about the bridge site should be determined. Pier shape is indicated as group of cylinders. In the computations, equivalent pier length is used, which is the total length of four piers for zero angle of attack (Richardson and Davis, 2001). Together with median grain size, diameter of grain of which 90% is finer (D_{90}) is needed, and it is 2.38 mm in close vicinity of Fol-1 Bridge (Coşkun, 1994). Pier scour computations are made using both CSU and Froehlich equations. Since CSU equation gave higher results than Froehlich, it is selected as the governing equation for pier scour computations. It is given by:

$$\frac{y_s}{y_1} = 2.0 K_{p1} K_{p2} K_{p3} K_{p4} \left(\frac{a}{y_1} \right)^{0.65} Fr_1^{0.43} \tag{3.2}$$

where, y_s = scour depth (m), y_1 = flow depth just upstream of the pier (m), K_{p1} = correction factor for pier nose shape, K_{p2} = correction factor for angle of attack of flow, K_{p3} = correction factor for bed condition, K_{p4} = correction factor for bed armoring, a = pier width (m), and Fr_1 = Froude number just upstream of pier. The value of K_{p1} is 1.0 for cylindrical shape, whereas K_{p2} is 1.0 for zero angle of attack. Furthermore, K_{p3} is 1.1 for plane bed to medium dunes, and K_{p4} is 1 for the grain size around bridge (Richardson and Davis, 2001). The values of y_1 and Fr_1 are automatically gathered from water surface profile calculations, and then y_s is computed from Equation (3.2). Pier scour computation results are presented in Table 3.7.

2) *Abutment scour:* local scour around abutments can be computed using Froehlich or HIRE equations. There is a limitation for the use of HIRE equation; the equation is applicable for $L_a/y_{a1} > 25$, where L_a = projected abutment length perpendicular to flow direction and y_{a1} = approach flow depth. In Fol-1 Bridge case, this limitation is not satisfied, therefore Froehlich equation, given in Equation (3.3), is used in the computations:

$$\frac{y_s}{y_a} = 2.27 K_{a1} K_{a2} \left(\frac{L'}{y_a} \right)^{0.43} Fr_a^{0.61} + 1 \tag{3.3}$$

in which, y_s = scour depth (m), y_a = average flow depth on floodplain (m), K_{a1} = coefficient for abutment shape, K_{a2} = coefficient for angle of embankment to flow, L' = length of active flow obstructed by embankment (m), and Fr_a = Froude number just upstream of abutment. K_{a1} is 1.0 for vertical wall and K_{a2} is 1.0 for 90° angle of approach flow with the abutment axis. The values of y_a and Fr_a are automatically calculated for each flow profile and then y_s is computed using Equation (3.3). Abutment scour computation results are also presented in Table 3.7.

Table 3.7 Scour depths around bridge piers and abutments

Q_i	Scour Depth, y_s (m)		
	Left Pier (Tonya side)	**Right Pier (Vakfıkebir side)**	**Right Abutment (Vakfıkebir side)**
Q_2	-	2.68	-
Q_5	-	2.93	-
Q_{10}	-	3.06	-
Q_{25}	0.94	3.18	-
Q_{50}	1.16	3.26	-
Q_{60}	1.23	3.27	-
Q_{70}	1.29	3.29	-
Q_{80}	1.33	3.30	0.02
Q_{90}	1.36	3.31	0.02
Q_{100}	1.39	3.32	0.31
Q_{150}	1.49	3.35	0.37
Q_{200}	1.54	3.37	0.54
Q_{300}	1.62	3.40	0.61
Q_{400}	1.66	3.42	0.70
Q_{500}	1.69	3.43	0.77

In the computations, scouring is not observed even under Q_{500}, at the left abutment, which is at Tonya side of Fol-1 Bridge. Also there is no scouring under the left pier and the right abutment for the first three and seven discharge profiles, respectively. The maximum scour depth for the left pier, which is 1.69 m for Q_{500}, even will not endanger the footing of the pier (See Figure 3.4). Scouring is most critical under the right pier and scour depths are severe. Detailed discussion on bridge scouring is given in Chapter 4 and scour-related results obtained from HEC-RAS for the right pier are tabulated in Appendix A.

3.8 Scour Countermeasures for Fol-1 Bridge

To mitigate the severe results of excessive scouring, scour countermeasures can be installed around bridge foundations. According to Richardson and Davis (2001), a suitable countermeasure should be installed when the scour criticality code of a bridge is equal to or less than 3. Discussions on the scour criticality of Fol-1 Bridge will be given in Section 4.2. Briefly, scour criticality of the bridge is 2, 3, and 4 corresponding to different flood profiles and scour criticality of the bridge is originated from the right pier group. Therefore scour countermeasures should be considered for this bridge and they should be designed only for the right pier.

A case study on bridge scour countermeasures has been conducted by Özdemir (2003). He studied design methods of various countermeasures against local scour around bridge piers and abutments, stream bed degradation, meander migration, and erosion of banks. Rock riprap, grout filled mattresses or bags, gabion boxes, articulated concrete block system (ACB), and concrete armor units, such as A-Jacks and Toskanes are proposed as pier scour countermeasures. Riprap is natural rock installed around the pier or footing under the pier. Grout filled bags are bags made of canvas and filled with grout of Portland cement. These bags are produced in specific dimensions and are placed around piers. Gabion boxes are interconnected wire mesh boxes filled with rock of suitable size, which may be acquired in site of installation. ACB and concrete armor units are pre-cast concrete units to protect bridge from local scouring, which may be used in case of unavailability of riprap of sufficient size. The advantage of concrete armor units is the interlocking of units providing stability to them. However, they are expensive.

For the pier scour in Fol-1 Bridge, riprap, grout filled bags, and gabion boxes are selected as scour countermeasures from view points of local site conditions, ease in installation, and workmanship. Riprap is the most common countermeasure among the alternatives mentioned, due to availability of material and installation convenience. A photograph of riprap around a bridge pier is given in Figure 3.12. Grout filled bags are used in case of absence of riprap close to the bridge site. Although they are not used in Turkish practice to date, grout filled bags specifically considered as a feasible alternative because of ease of installation, less production time, and less cost. An application of grout filled bags around bridge abutment is presented in Figure 3.13. Gabion boxes are generally used in rivers where bed shear stress is high and when there is no riprap in large diameters available in site, or it is economically infeasible to use such large riprap. An example of gabion box use around bridge piers is presented in Figure 3.14.

Figure 3.12 An application of riprap around bridge piers (Newsline, 2007)

Figure 3.13 An application of grout filled bags around bridge abutment (Lagasse, 2002)

Figure 3.14 Use of gabion boxes around bridge piers
(Red Star Wire Mesh Factory, 2006)

For Fol-1 Bridge, scour countermeasure computations are carried out according to the procedure followed by Özdemir (2003). Calculations of riprap, grout filled bags, and gabion boxes are carried out and presented in the subsequent sections. In the calculations, 1 US Dollar is taken as equal to 1.5 Turkish Liras and 2009 unit prices are used. Also, the total costs of these countermeasures include 15% contingencies.

3.8.1 Riprap Calculations

As a scour countermeasure, riprap is mostly installed on a geotextile filter cloth. The geotextile is necessary to prevent erosion of the fine material below the riprap layer (GISHydro, 2004). In the calculations, first of all, riprap size to be used should be selected. However, there is no universally accepted single criterion for the design discharge of riprap size computation. It normally changes according to the location of the bridge, traffic intensity, importance of the structure, etc. that is why various alternatives were considered for riprap size for $Q_i \geq Q_{50}$ up to Q_{500} (See Table 3.8). Median riprap size diameter is calculated according to HEC-23 criteria and presented below (Lagasse et al., 2001):

$$D_{r50} = 0.692\frac{(K_p u_p)^2}{2g\Delta} \tag{3.4}$$

in which, D_{r50} = median riprap size diameter (m), K_p = coefficient for pier nose shape, which is 1.5 for round nose shape, u_p = velocity just upstream of pier (m/s), g = gravitational acceleration (m/s^2), and Δ = relative density, which is 1.65 for rock riprap. For the discharges Q_{50} to Q_{500}, u_p values for right pier are obtained by HEC-RAS and then using Equation (3.4), D_{r50} values are calculated and presented in Table 3.8.

Table 3.8 Median riprap size calculations

Q_i	u_p (m/s)	D_{r50} (m)
Q_{50}	7.21	2.50
Q_{60}	7.27	2.54
Q_{70}	7.32	2.58
Q_{80}	7.36	2.61
Q_{90}	7.39	2.63
Q_{100}	7.43	2.66
Q_{150}	7.54	2.73
Q_{200}	7.62	2.79
Q_{300}	7.72	2.87
Q_{400}	7.78	2.91
Q_{500}	7.84	2.96

As tabulated above, necessary riprap diameter is too high. Using such large ripraps would be economically infeasible. Moreover, ripraps with such diameters are almost impossible to find in vicinity of the bridge. At this stage, bed shear stress of the river should be calculated to evaluate whether using smaller ripraps would be sufficient to protect the bridge from excessive scouring. Bed shear stress is calculated for Q_{100} and Q_{500} discharges using Equation (3.5).

$$\tau_0 = \gamma R_h S_f \tag{3.5}$$

where, τ_0 = bed shear stress (Pa), γ = specific weight of water (N/m^3), R_h = hydraulic radius, S_f = energy grade line slope. From Equation 3.5, τ_0 is calculated as 483 Pa for Q_{100} and as 516 Pa for Q_{500}. Since bed shear stress of the river is too high, smaller size ripraps cannot resist the force of water flow. Moreover, use of concrete blocks in the channel bed is appropriate up to $\tau_0 \approx 60$ kgf/m^2 (588 Pa) and gabion boxes are appropriate up to $\tau_0 \approx 150$ kgf/m^2 (1472 Pa) (Erkek and Ağıralioğlu, 2010). Therefore, riprap is not an appropriate alternative for scour countermeasures to be used at Fol-1 Bridge.

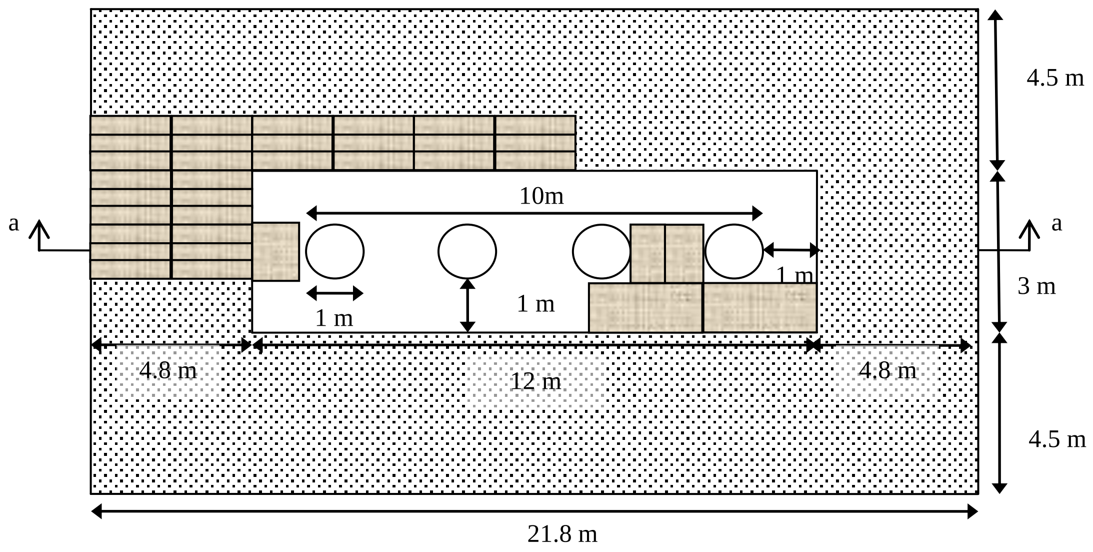
3.8.2 Grout Filled Bag Calculations

Grout filled bags are made of canvas bags and filled with grout. Unit prices for the grout filled bag placement items against pier scouring are listed in Table 3.9. The authorities defining those unit prices are also indicated in the table below.

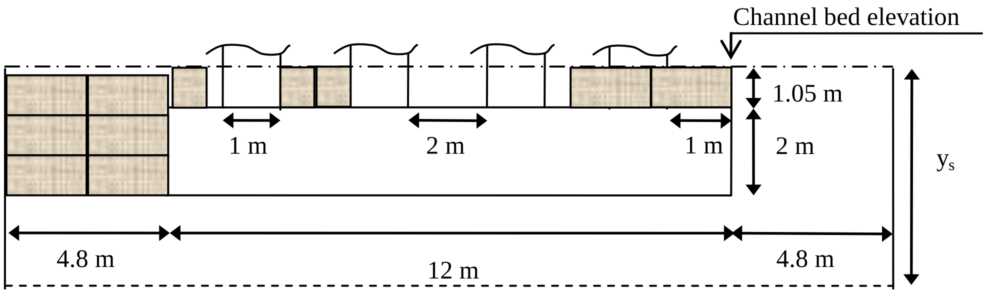
Table 3.9 Unit prices for grout filled bag placement (Özdemir, 2003 and Birimfiyat.com, 2010)

Item No.	Pose No.	Explanation	Unit	Unit Price ($)
1	14.100 (KGM)	Excavation of any type of soil around bridges manually except rocks	m^3	9.353
2	07.006/14 (DSİ)	Transportation of excavation (2 km)	ton	1.053
3	10.022/K (KGM)	Cost of grout	m^3	9.027
4	Market price	Canvas	m^2	1.062

Grout filled bags are placed around the right pier, and they are shown with canvas hatch in Figure 3.15. The bags are placed all around the footing from channel bed to bottom elevation of the footing (dotted in Figure 3.15) and on top of the footing from channel bed elevation to top elevation of the footing. Dimensions of the bags used are 1.2x0.9x0.3 m and 2.4x0.9x0.3 m (length, width, thickness), which are indicated as “A” and “B”, respectively (Özdemir, 2003). The area of grout filled bag installation is shown in Figure 3.15 in which y_s is the depth of scour. Number of bags in depth and area are calculated for around and on top of the footing as presented in Table 3.10.



a) Plan view



b) Section a-a

Figure 3.15 Details of grout filled bag placement

Total volume of excavation is calculated as the summation of volume of grout filled bag placement on top of and around the footing. On top of the footing, area of excavation is 32.86 m^2 and depth of soil is 1.05 m. Therefore, the volume on top of the footing to be excavated is 34.50 m^3. Around the footing, area of excavation for the bags is 223.2 m^2. The volume is then calculated by multiplying this area by total depth of soil to be excavated, which is 3.05 m, and it is found to be 680.76 m^2. Therefore total volume of excavation is 715.26 m^3. Grout volume is equal to the total volume of bags used and canvas area is the total surface area of bags used.

Table 3.10 Cost calculation for grout filled bag placement

Item	Unit	Quantity
# of B bags in depth (around)	piece	3
# of B bags in area (around)	piece	310
# of bags in depth (above)	piece	3
# of A bags in area (above)	piece	8
# of B bags in area (above)	piece	10
Total # of A bags	piece	24
Total # of B bags	piece	960
Excavation volume	m^3	715.26
Grout volume	m^3	629.86
Canvas area	m^2	6,130.08
Item 1 cost	$	6,689.83
Item 2 cost	$	753.17
Item 3 cost	$	5,685.71
Item 4 cost	$	6,510.14
Total cost	$	19,638
Total cost with contingencies	$	22,584

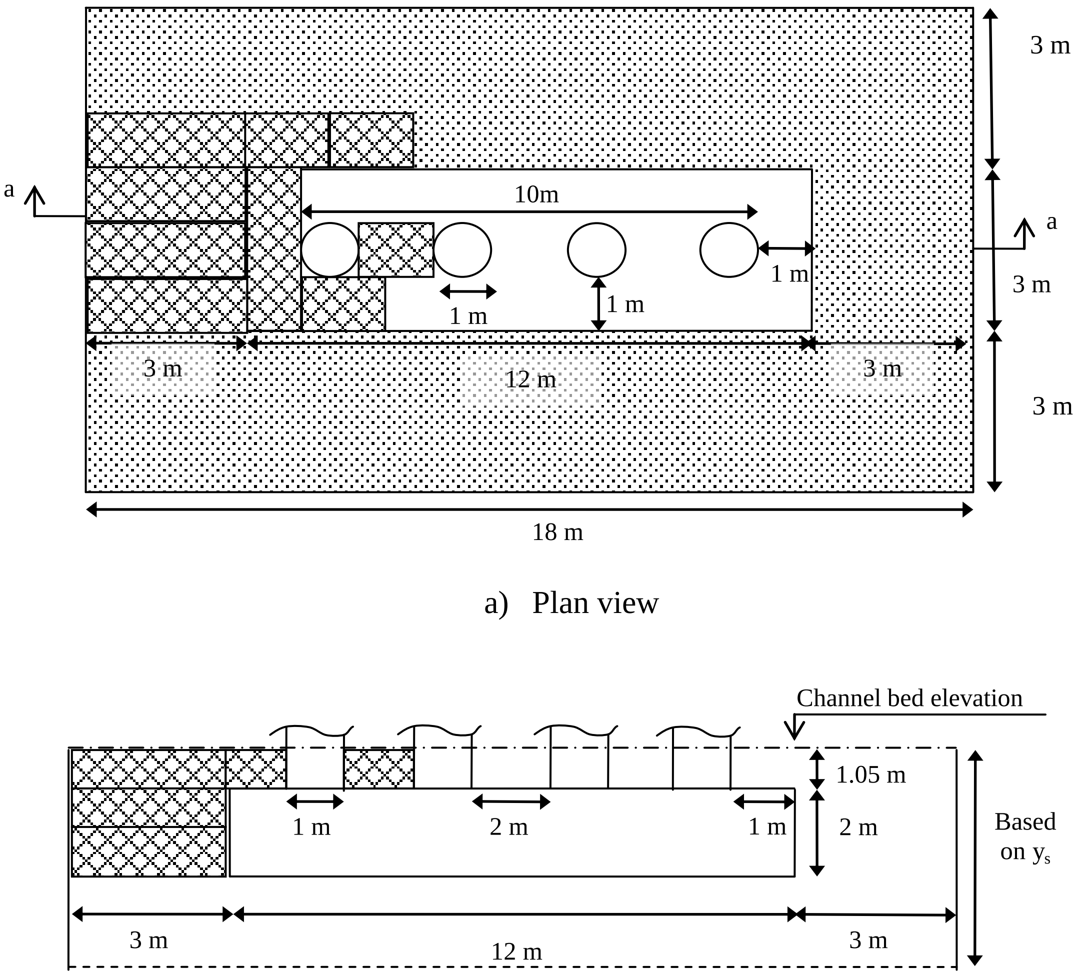
3.8.3 Gabion Box Calculations

Gabion boxes are filled with riprap and they are placed over a geotextile filter cloth on soil. In Table 3.11, unit prices of items related to gabion box installation and the authorities defining those unit prices are presented. Construction with gabion box includes the cost of wire mesh and riprap together with its cost of placement.

Table 3.11 Unit prices for gabion box placement (Dursun, 2008 and Birimfiyat.com)

Item No.	Position No.	Explanation	Unit	Unit Price ($)
1	14.100 (KGM)	Excavation of any type of soil around bridges manually except rocks	m^3	9.353
2	07.006/14 (DSİ)	Transportation of excavation (2 km)	ton	1.053
3A	23.KH/5-2 (KHGM)	Construction with gabion box of 2.0x1.0x1.0 m size	piece	109.610
3B	23.KH/5-4 (KHGM)	Construction with gabion box of 3.0x1.0x1.0 m size	piece	168.270
4	07.006/35 (DSİ)	Transportation of riprap (11 km)	m^3	7.853
5	Market price	Placement of geotextile	m^2	3.589

Two different sizes of gabion boxes are used in the study, which are 2.0x1.0x1.0 m and 3.0x1.0x1.0 m (length, width, thickness) (Dursun, 2008). These boxes are indicated as “A” and “B”, respectively. Gabion boxes are placed around the right pier and they are shown with diamond hatch in Figure 3.16. For the extent of the gabion box placement, riprap placement criteria, which is mentioned in Özdemir (2003) is followed. Similar to the grout filled bags, the boxes are placed around (dotted area in Figure 3.16) and on top of the footing. The dimensions of gabion box installation area are shown below, in which y_s is the depth of scour. Number of boxes in depth and area are calculated, and they are presented in Table 3.12.



a) Plan view

b) Section a-a

Figure 3.16 Details of gabion box placement

Similar to grout filled bag calculations, total volume of excavation is calculated. On top of the footing, it is 34.50 m^3. Around the footing, area of excavation is 126 m^2. The volume is then calculated by multiplying this area by depth of soil from channel bed to bottom of footing, which is 3.05 m. Therefore, excavation volume around the footing is found to be 384.30 m^3, and the total excavation volume is 418.80 m^3. The area of excavation around the footing is equal to the area of geotextile used. Volume of riprap needed is calculated as the total volume of gabion boxes. Total cost of gabion box placement is presented in Table 3.12.

Table 3.12 Cost calculation for gabion box placement

Item	Unit	Quantity
# of A boxes in depth (around)	piece	3
# of A boxes in area (around)	piece	54
# of B boxes in depth (around)	piece	3
# of B boxes in area (around)	piece	6
# of A boxes (above)	piece	13
# of B boxes (above)	piece	2
Total # of A boxes	piece	175
Total # of B boxes	piece	20
Excavation volume	m^3	418.80
Riprap volume	m^3	410.00
Geotextile area	m^2	126.00
Item 1 cost	$	3,917.04
Item 2 cost	$	441.00
Item 3 cost	$	22,547.15
Item 4 cost	$	3,219.73
Item 5 cost	$	452.21
Total cost	$	30,577
Total cost with contingencies	$	35,164

A summary of the results obtained from the calculations for scour countermeasures is presented in Table 3.13. Comparing only the costs of countermeasures, gabion box is found to be a more expensive alternative.

Table 3.13 Comparison table for costs of scour countermeasures

Scour Countermeasure	Cost ($)
Grout filled bags	22,584
Gabion boxes	35,164

3.9 Economic Analysis of Scour Countermeasures

In this section, the seventh step of Scour Countermeasures Calculator, which is mentioned in Section 2.3.2, is carried out. In this last step, up to seven scour countermeasures which may be appropriate at the bridge site can be specified in order to calculate the net benefit and benefit cost ratio of each countermeasure proposed.

In the Step 5 of Scour Countermeasures Calculator, remaining useful life of the bridge is defined. In OECD countries, average service life of bridges is 80 years (Caner et al., 2007). Therefore, the remaining life of Fol-1 Bridge is 50 years, since it is a 30-year-old bridge. In Step 7, three alternatives are discussed: doing nothing to the bridge, installation of grout filled bags and gabion boxes. Entering the costs of these alternatives and their return period of protection, adjusted lifetime failure probability of Fol-1 Bridge, net benefit, and benefit/cost ratio of each alternative are calculated as presented in Figure 3.17. With the proposed scour countermeasures, it is aimed to protect the bridge against return period of floods equal to at least reciprocal of the annual failure probability. In the case study, reciprocal of the annual failure probability (i.e. 1/0.07388) corresponds to a return period of approximately 14 years. Any scour countermeasure with an assumed return period of protection greater than this return period will eventually decrease the annual failure probability of the bridge. This annual probability is referred as adjusted lifetime failure probability and is calculated with Equation (3.6).

$$P_L' = 1 - \left(1 - \frac{1}{RP}\right)^L \tag{3.6}$$

where, P_L' = probability of failure over the extended life of the protected bridge, RP = return period protection desired (year), and L = remaining useful life of bridge (year).

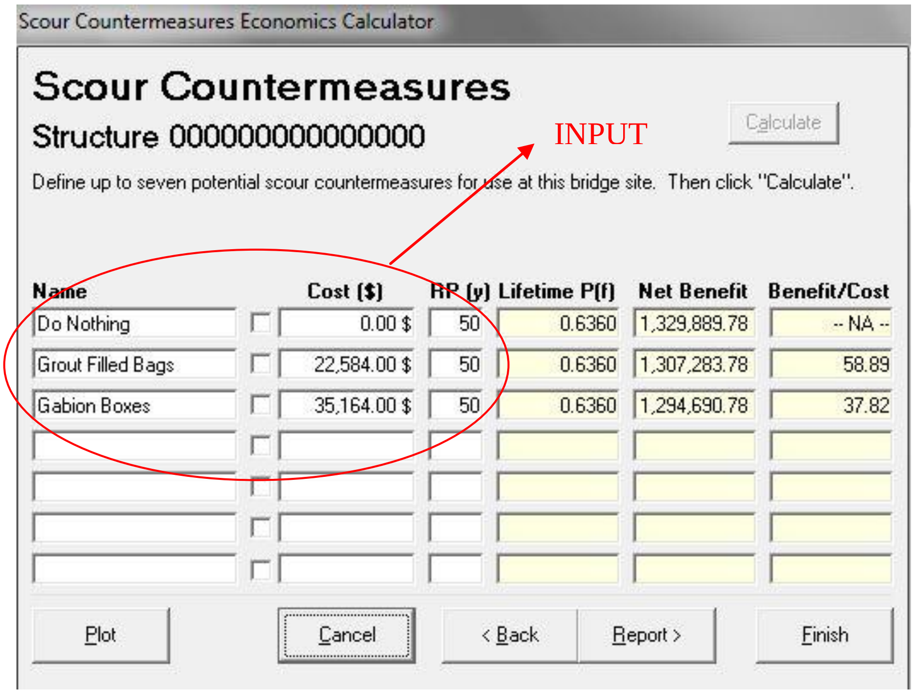


Figure 3.17 Benefit calculations of countermeasures

A graphical comparison of the proposed countermeasures is provided in the HYRISK software. The graphical comparison of the countermeasures proposed for Fol-1 Bridge is presented in Figure 3.18.

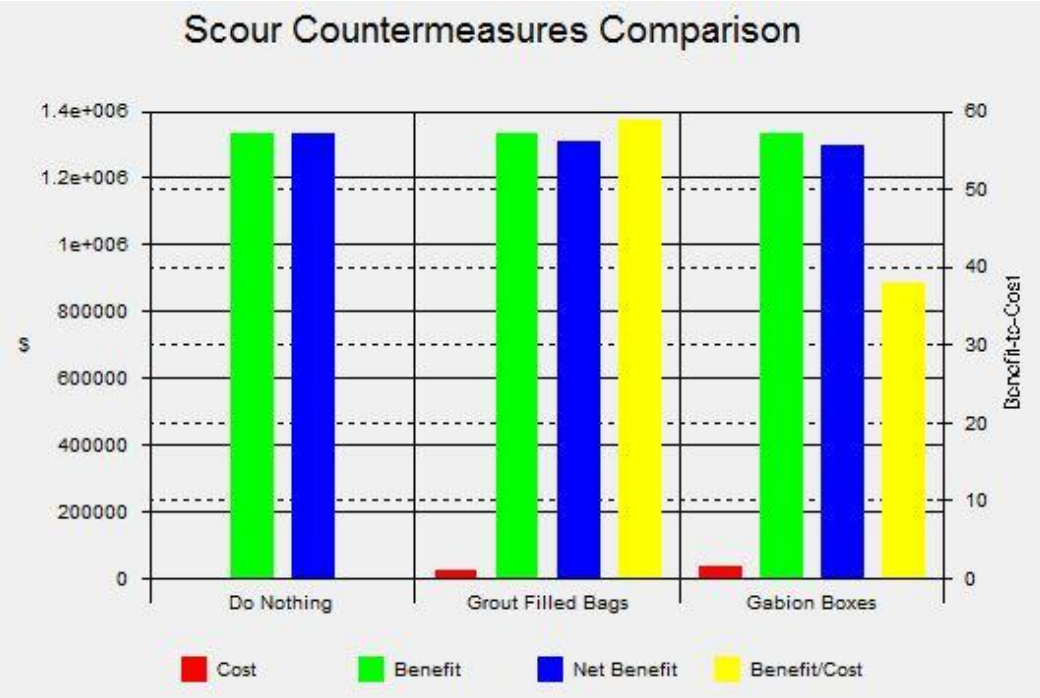


Figure 3.18 Economical comparison of the proposed scour countermeasures

Also, the report of the proposed countermeasure analysis is generated in the model, and the report for Fol-1 Bridge is given in Figure 3.19.

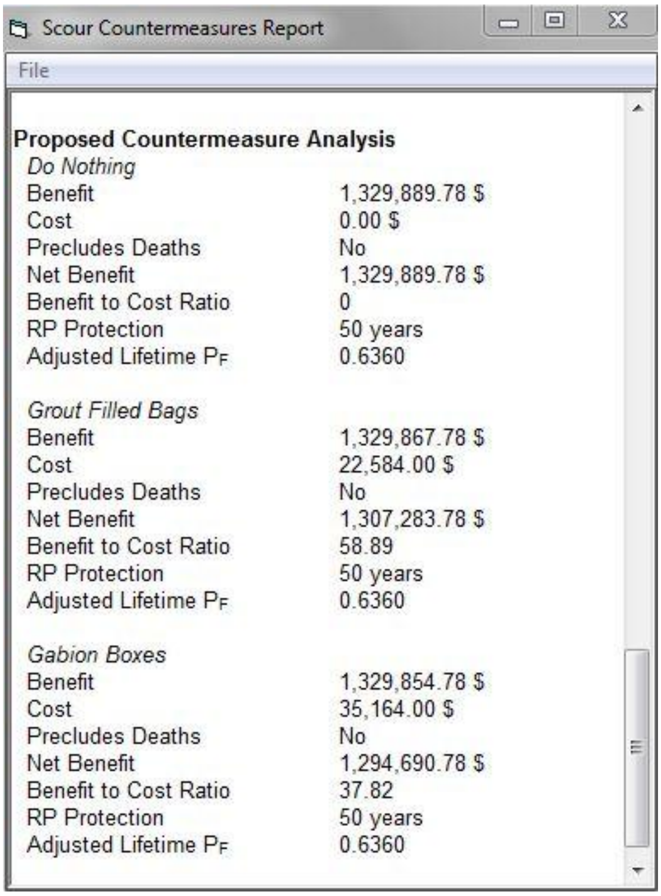


Figure 3.19 Scour countermeasure analysis report

Considering the net benefit and benefit/cost ratio of the countermeasures together, it is reasonable to select grout filled bags for Fol-1 Bridge. A discussion on the selection of scour countermeasure will be provided in Chapter 4.

CHAPTER 4

DISCUSSION OF THE RESULTS

4.1 Introduction

The objective of this thesis is to evaluate the scour failure risk of Fol-1 Bridge. A case study is carried out for the demonstration of this evaluation. Scour failure risk calculations are carried out for Fol-1 Bridge using HYRISK software. In addition to scour failure risk calculations, local scour computations are conducted and costs of applicable scour countermeasures for the bridge site are determined. Results of these calculations are presented in the previous chapter. In this chapter, evaluation of the risk of the bridge is made and discussions are given for the case study.

4.2 Evaluation of Scour Failure Risk

As it is discussed in Chapter 3, waterway adequacy of Fol-1 Bridge is coded as 9, which means that overtopping chance is remote corresponding to return periods greater than 100 years. This decision is based on the fact that no overtopping is observed even under the maximum discharge recorded in the basin, i.e. 412 m^3/s. This discharge almost doubles the discharge corresponding to 500-year return period.

Scour criticality rating can be estimated based on the channel and substructure conditions. However, making an evaluation using HEC-RAS program is a more realistic approach. As briefly explained in Section 3.7, scour depths and their levels corresponding to footings differ in each abutment and pier. Discharges considered and levels of scouring are summarized in Table 4.1.

Table 4.1 Scour levels at piers and abutments of Fol-1 Bridge

Q_i	Scour Level		
	Left Pier	**Right Pier**	**Right Abutment**
Q_2	No scour	Within limits of footing	No scour
Q_5	No scour	Within limits of footing	No scour
Q_{10}	No scour	Within limits of footing	No scour
Q_{25}	Above footing	Below footing	No scour
Q_{50}	Above footing	Below footing	No scour
Q_{60}	Above footing	Below footing	No scour
Q_{70}	Above footing	Below footing	No scour
Q_{80}	Above footing	Below footing	Within limits of footing
Q_{90}	Above footing	Below footing	Within limits of footing
Q_{100}	Above footing	Below footing	Within limits of footing
Q_{150}	Above footing	Below footing	Within limits of footing
Q_{200}	Above footing	Below footing	Within limits of footing
Q_{300}	Above footing	Below footing	Within limits of footing
Q_{400}	Above footing	Below footing	Within limits of footing
Q_{500}	Above footing	Below footing	Within limits of footing

Scour levels explained in Table 4.1 are presented in Figure 2.1. Scour criticality codings corresponding to these scour levels are given in Table 4.2 which are explained previously in Section 2.2.1.2.

Table 4.2 Scour criticality codes for piers and abutments of Fol-1 Bridge

Q_i	Scour Criticality Codes		
	Left Pier	**Right Pier**	**Right Abutment**
Q_2	9	3 - 4	9
Q_5	9	3 - 4	9
Q_{10}	9	3 - 4	9
Q_{25}	8	2 - 3	9
Q_{50}	8	2 - 3	9
Q_{60}	8	2 - 3	9
Q_{70}	8	2 - 3	9
Q_{80}	8	2 - 3	5
Q_{90}	8	2 - 3	5
Q_{100}	8	2 - 3	5
Q_{150}	8	2 - 3	5
Q_{200}	8	2 - 3	5
Q_{300}	8	2 - 3	5
Q_{400}	8	2 - 3	5
Q_{500}	8	2 - 3	5

Instead of evaluating the scour criticalities of piers and abutments individually, the entire bridge should be considered. As a whole, the worst condition for scour criticality is taken into consideration. This approach yields to a result that, scour criticality of Fol-1 Bridge is coded as 3 - 4 for Q_2, Q_5, and Q_{10} profiles where it is coded as 2 - 3 for Q_{25} and greater discharges. According to Table 3.6, for waterway adequacy coding 9 and scour criticality codings 2, 3, and 4, the annual risk of scour failure of the bridge is $59,792. Compared to rebuilding cost of the bridge ($481,074) it is a high risk to take annually, if no necessary action is done. Therefore, it is reasonable to consider appropriate local scour countermeasures for Fol-1 Bridge.

4.3 Evaluation of Scour Countermeasures

Appropriate pier scour countermeasures for Fol-1 Bridge are selected as grout filled bag and gabion box, and necessary cost calculations are carried out in Section 3.8. After installing a countermeasure, scour criticality code of a bridge can be upgraded to a code of 8 or 7, meaning that the bridge is in good condition or scour countermeasures are installed, respectively (Richardson and Davis, 2001). Upgrading the code to 7 makes no difference in the annual scour failure risk, since this code means that necessary action has been done for the bridge but still there exist the risk of failure. If the code is upgraded to 8, the risk decreases to 2,525 $/year.

Since Fol Creek is frequently exposed to high flows, both grout filled bags and gabion boxes are designed to protect the bridge from excessive scouring throughout its remaining useful life, which is 50 years. Therefore, the upgraded scour criticality ratings are predicted as 8 for both countermeasures. Considering their effects on the scour criticality rating together with their costs, net benefits, and benefit/cost ratios, grout filled bags is accepted to be the economically feasible countermeasure. However, in Turkish practice, there is no use of grout filled bags to date. Although the benefit/cost ratio of the gabion boxes is less than that of grout filled bags, they still have a high ratio. Furthermore, gabion boxes and mattresses are used in river training practices in Turkey. Therefore, gabion box alternative is selected as a reasonable scour countermeasure for Fol-1 Bridge.

As given in Table 3.13, the cost of gabion box is \$35,164 whereas, for scour criticality codes of 2, 3, 4 and 7 the annual risk of failure is \$59,792. In line with this figures, it is evident that with a total investment of \$35,164 for gabion box installation (which is likely to be spent once in whole economic life of the bridge), which is approximately 60% of the annual risk of failure, the scour criticality of the bridge can be upgraded to 8 and consequently the risk decreases to 2,525 \$/year. In other words, with an overall investment of \$35,164, the annual risk of failure can be reduced approximately 95%, i.e. from \$59,792 to \$2,525. Another considerable fact of such investment is that once the scour criticality is upgraded to 8, the annual failure probability is reduced from 0.074 corresponding to a return period of 14 years (Return period=1/annual failure probability) to 0.003 with a return period of 333 years.

In this study, although the appropriate local scour countermeasures are designed properly and they both may mitigate the severe effects on the bridge stability, there exist long-term bed degradation problem in close vicinity of the bridge. Therefore, long-term bed degradation of the bridge should be studied and degradation countermeasures should also be considered carefully. Check-dams and bank training facilities are suitable countermeasures against degradation. Derivatives of these countermeasures are reasonable to consider for coping with the severe results of bed degradation (Lagasse et al., 2001). Bank protection at the bridge site has not been considered specifically in this study as it is assumed to be an integral part of the bank protection facility to be implemented along the whole reach of Fol Creek. A pioneering study was carried out in Turkey by Yanmaz and Coşkun (1995) to emphasize the effect of hydrologic and hydraulic aspects on design of river bridges. In fact, various combinations of bank protection facilities with their economic analyses are studied for Fol Creek by Yanmaz and Bilen (2000). Suitability of an upstream flood detention dam on Fol Creek was also studied by Yanmaz and Günindi (2006).

CHAPTER 5

CONCLUSIONS

5.1 Summary and Conclusions

Excessive local scouring around bridge piers and abutments may lead to stability problems or damage on bridge, or even failure of it. Therefore, scouring and the failure risk of a bridge associated with scouring have to be studied. This study is carried out to present the determination of scour failure risk of a river bridge via a risk-based model, using HYRISK software. The methodology of the study is discussed and a case study is performed for Fol-1 Bridge crossing Fol Creek to exemplify the objective of the thesis. Water surface profile and scour depth calculations are performed using the HEC-RAS software. Also a study on suitable scour countermeasures for the bridge is carried out for mitigation of unfavorable results of excessive scouring. In the light of these studies, the following conclusions are reached:

1) With the availability of necessary information, the annual risk of scour failure of a bridge can be calculated using HYRISK. However, one has to be careful in assigning relevant input parameters, which should be compatible to the local site conditions and the commonly accepted standards used in the country concerned.
2) A detailed study on channel and substructure conditions, waterway adequacy, and scour criticality leads to more realistic results of risk calculations. Channel stability indicators proposed by Lagasse et al. (2001) and Johnson et al. (1999) provide confidence in determining the channel condition around bridge site. Using hydraulic design of HEC-RAS software for scour depth calculations is convenient for scour criticality estimation.
3) Suitable scour countermeasures should be assessed for mitigating the

excessive scour around bridge foundations. Selection of the appropriate countermeasure is based on both its applicability on the bridge site and its economical feasibility. The risk of scour failure obtained in the calculations is quite guiding for the selection. Bank protection at the bridge site has not been considered specifically in this study as it is assumed to be an integral part of the bank protection facility to be implemented along the whole reach of Fol Creek.

4) Fol-1 Bridge is 30-year-old, it has resisted to 1990 flash flood and no overtopping is observed during this event. Although the bridge is still serviceable, supplementing proper scour countermeasures would be of worth since it is subject to various sources of uncertainties, e.g. uncertainties associated with flow computations and distribution of flow in the channel reach and possible future alterations in river resistance. These uncertainties should force an engineer to consider the protection of the bridge. Besides, in case of a bridge failure, many people may die or injured. It is unethical for an engineer to discard the probability of loss of life. Therefore, it is reasonable and even compulsory to protect the bridge with suitable scour countermeasure(s).
5) In the case study, scour criticality of Fol-1 Bridge is found to be 2, 3, and 4 for various flood profiles, which lead to an annual failure probability of 0.074 and an annual risk of scour failure of $59,792. The right pier of the bridge is exposed to extreme local scouring. For the computed discharges corresponding to big return periods, i.e. 100 and 500 years, the size of the riprap was obtained quite large which is not economical. Therefore, grout filled bags and gabion boxes were found to be suitable to be used in the study area. Cost calculations of these countermeasures and a benefit/cost study are carried for the selection of the appropriate countermeasure. Although installation of grout filled bags have higher benefit/cost ratio than gabion boxes, the extensive use of gabion boxes in Turkish practice predominates the selection criteria. Therefore, gabion boxes are selected for the case study area.

5.2 Recommendations

This study has been carried out to present a method for scour failure risk assessment which is developed and being used in the United States. A case study is conducted on a pilot bridge to present the applicability of this method for bridges in Turkey. For the analyses in the study, necessary information is gathered from the previous studies, General Directorate of Highways, State Hydraulic Works, and inspections made in the field trip to the bridge site.

Although a comprehensive study has been conducted to obtain information on bridge and bridge site, still there are some missing data for the bridge and local conditions both in the study area and in Turkey in general. Lacking data and recommendations for overcoming these problems are simply as follows:

1) There is no database for bridges in Turkey. For each study, the necessary information should be gathered from various authorities, which leads to loss of time and effort. A national database, similar to NBI, should be developed for Turkey. The database should include the cross-sectional data, dimensions and structural classifications of the bridge, functional classification, traffic information of the roadway on/under the bridge, and relevant hydraulic and hydrologic information.

   Also, the stability of the channel and structure together with its scour criticality should be defined. Considering these items, bridges should be inspected periodically using the methodology proposed by Yanmaz et al. (2007) and the database should be updated periodically when necessary.
2) Obtaining current information of old bridges in Turkey is very difficult. A special attention should be attracted to this problem and information of old bridges should be updated by detailed field inspections. Approximate remaining lifetime of existing bridges should be computed using the procedure described by Caner et al. (2008) such that series of bridges are ranked according to the structural and hydraulic deficiencies. Therefore, priority may be given to those bridges to reinforce them with the right techniques at the right time.

3) The awareness for the concept of importance of time loss and the resultant inconveniences should be established in Turkey. A study on the cost of time for people and commercial trucks should be conducted considering the socio-economic conditions of different regions in Turkey.

A national based study on bridges together with overcoming the problems stated above will provide convenience to those structures and the surrounding properties.

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APPENDIX A

HEC-RAS OUTPUTS FOR STEADY FLOW SIMULATION OF FOL CREEK AND SCOUR CALCULATIONS OF FOL-1 BRIDGE

Steady flow simulation of Fol Creek is carried out via HEC-RAS software. In the execution of the program, the input data are taken as: Manning's roughness coefficient, which is 0.069 for bank and 0.033 for channel, contraction and expansion coefficients, which are 0.1 and 0.3 for natural sections and 0.3 and 0.5 for bridge section, respectively, normal depth both at upstream and downstream ends as a boundary condition for mixed flow, and average bed slope of 0.008 (Bilen, 1999). Also, scour depth calculations are carried out using HEC-RAS software. For fifteen flood profiles corresponding to different return periods, analyses are made and the results of the analyses are presented in Table A.1, in which Q_i, Q, Z_{min}, Z_w, u_p, and y_s are discharge corresponding to a return period, discharge, minimum channel bed elevation, water surface elevation, velocity just upstream of right pier, and depth of scour around right pier of Fol-1 Bridge, respectively.

Table A.1 Results of steady flow analysis and scour calculations

Q_i	Q (m^3/s)	Z_{min} (m)	Z_w (m)	u_p (m/s)	y_s (m)
Q_2	56.7	9.95	10.86	5.37	2.68
Q_5	93.3	9.95	11.07	6.14	2.93
Q_{10}	117.3	9.95	11.19	6.51	3.06
Q_{25}	145.2	9.95	11.31	6.92	3.18
Q_{50}	163.8	9.95	11.37	7.21	3.26
Q_{60}	168.4	9.95	11.39	7.27	3.27
Q_{70}	172.2	9.95	11.40	7.32	3.29
Q_{80}	175.4	9.95	11.41	7.36	3.30
Q_{90}	178.1	9.95	11.42	7.39	3.31
Q_{100}	180.6	9.95	11.43	7.43	3.32
Q_{150}	189.6	9.95	11.46	7.54	3.35
Q_{200}	195.6	9.95	11.48	7.62	3.37
Q_{300}	203.7	9.95	11.50	7.72	3.40
Q_{400}	209.1	9.95	11.52	7.78	3.42
Q_{500}	213.1	9.95	11.53	7.84	3.43