Bridge Scour

Basic Mechanisms and Predictive Formulas

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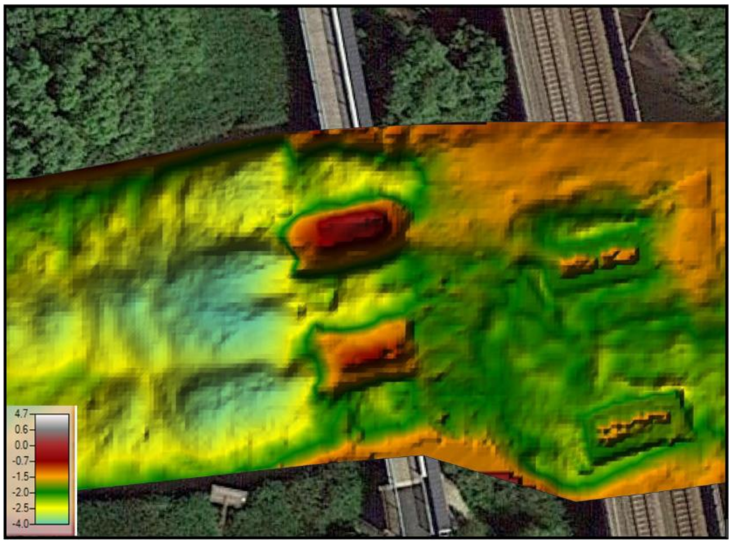

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Summary

This report aims at presenting basic knowledge on bridge scour and the processes governing its evolution as well as summarizing the most common formulas used to calculate scour depth at bridges. Design procedures concerning bridge scour in several different countries are also discussed, including United States, Australia, and the United Kingdom. The situation in Sweden with regard to bridge scour is briefly reviewed and several case studies are presented where marked scour holes have been detected at bridges. Two cases of bridge failures in Sweden are included where local scour was the main reason for the collapse.

Bridge scour is typically separated into pier, abutment, and contraction scour, where each mechanism is controlled by different physics and governing parameters. Each type of bridge scour is discussed separately in the report with sections on basic mechanisms, governing parameters, common predictive formulas, and concluding remarks.

The report also includes a brief summary on the expected influence of climate change on bridge scour. Larger and more intense rainfalls in the future imply larger flows in the rivers with increased bridge scour as a result.

The report deals only with scour induced by bridges; other types of scour, such as general scour due to longitudinal transport gradients in the river, scour related to secondary flows in river bends, or scour downstream hard bottom, are not discussed. Most of the formulas included to estimate bridge scour are valid for friction material and only a few examples are given that are applicable to cohesive sediment, mainly related to recommended design procedures from different countries. Also, the objective of bridge scour analysis is often to estimate the maximum scour depth, occurring at equilibrium conditions under a certain flow, implying that most of the formulas are valid for such conditions.

Contents

1. Introduction

1.1 Background

A bridge can fail due to several causes, such as earthquakes, wind, and flooding. Among these factors, flooding inducing bridge scour is worldwide the most common reason for bridge failure (Kattell and Eriksson, 1998; Melville and Coleman, 2000). About 60% of the bridge failures implying total bridge collapse in the United States since 1950 have been related to the scour of bridge foundations (Shirhole and Holt, 1991).

Numerous numbers of bridges have been damaged or destroyed by floods in the United States and other parts of the world over the years. The Colorado Department of Transportation (CDOT) estimated that several state highway bridges were destroyed and many were seriously damaged by floods in the year of 2013 (see https://www.codot.gov/). Another example of bridge failure due to scour is the Wairoa River Bridge failure during Cyclone Bola in 1988 that severed the link between the two areas of Wairoa Town, dramatically increasing the motorized traffic travel time across the river and resulting in significant community costs in addition to the cost of bridge replacement.

Works Consultancy Services (1990) found that the capital costs of temporary facilities required after the failure and before completion of a new bridge (*e.g.*, bridges to carry services and pedestrians, upgrading of an alternative route for diverted vehicle traffic) were 50% greater than the costs of replacing the failed bridge and its approaches. In Nepal, due to the degradation of bed materials during the 2014 flooding, the foundation of the highway bridge over the Tinau River was seriously exposed (Shrestha, 2015). These are just a few examples of bridge failures; however, there are numerous numbers of incidents like these in the past that encouraged engineers and scientists to investigate the issue of bridge scour.

Bridge piers obstruct the flow of floodwaters, causing an increase in velocity and the development of vortices. The unique flow field with the higher velocity can seriously damage bridge foundations. Thus, if the depth of the foundation is not deep enough, the likelihood of bridge failure becomes higher. The damage to and failure of bridges caused by scour are problems of great significance all over the world. In spite of significant investment in bridge scour research, bridges still fail due to scour. It is believed that this is partially a consequence of inadequacies in both design criteria adopted for older bridges and also the present state of knowledge about some aspects of bridge hydraulics and scouring. It is also due to the lack of convenient and appropriate availability of the results of the past scour research to practitioners. A summary of present state of knowledge on bridge scour is needed. A very important reason of this failure is also the lack of knowledge of the scour that is taking place over the years due to insufficient regular survey of the river bed condition around the bridge piers.

Many investigations have been performed since the late 1950s to understand the flow and the erosion mechanisms around bridge piers and to estimate the scour depth. However, the complexities of the three-dimensional (3D) separated flow, its interaction with the transport of sediment and the changing mobile boundary, present great difficulties in analyzing the problem theoretically. Hence, the early investigations concentrated mostly on scour estimation based on dimensional analysis and data correlation of small-scale laboratory experiments (Breusers *et al*., 1977; Raudkivi 1991). The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil properties, streamflow conditions, and bridge designs commonly encountered.

Much research has been carried out on bridge pier scour; however, even now in modern times, pier and abutment undermining due to scouring and riverbed erosion has been widely recognized as being the main cause for bridge damage and failure (Melville and Coleman, 2000; Richardson and Davis, 2001).

1.2 Objectives

It should be stressed that many bridges in Sweden and moreover scour conditions around the piers and abutments of those bridges have never been evaluated to date. It is evident from several bridge failures all over the world that the study of scour around the piers and abutments is very important to avoid any forthcoming disastrous situation, especially in view of possible changes in the flow conditions due to climate change. In fact the amount of money incurred after any accidental bridge failure to restore the previous condition is huge. Hence, a proper understanding of the bridge pier and abutment scour will help us to take prior precautions and avoid any disastrous situation related to bridge failure. Most of the bridge failures around the world occurred during severe floods. The river flow during floods is unsteady and it gives rise to complex flow conditions around the structures, which causes enhanced scour. Moreover, due to climate change the storm characteristics may change significantly. Under such changing climatic conditions, as well as storm characteristics, it is natural that the flood hydrograph of a stream will also change, giving rise to changing unsteady flows in the rivers.

The overall aim of the present study is to review basic knowledge on local scour at bridges and methods to quantitatively estimate this scour, including design procedures in a number of countries that have extensive experiences from bridge scour. In addition, an assessment of the significance of local scour at bridges in Sweden is made with some discussion on expected changes related to the flow conditions resulting from climate change. More specifically, the detailed objectives include:

- to elucidate the basic processes of bridge pier and abutment scour as well as structural problems related to these phenomena
- to determine the main factors that affect bridge scour and their specific properties
- to discuss empirical equations for scour estimation that may be suitable for Swedish conditions
- to evaluate how climate change may influence scour in the future for Swedish conditions
- to review design procedures employed in countries with major local scour problems

1.3 Procedure and report overview

This report is based on a comprehensive review of mainly international literature on bridge scour focusing on research publications, but also reporting on design practices and procedures in selected countries with extensive experience from problems with bridge scour. The literature in Sweden is rather limited on this subject and problems with bridge scour has not been reported very frequently, but the available Swedish material is referred to as well. One reason for the rare cases of bridge scour reported in Sweden is the lack of detailed measurements; however, in recent years, high-quality bathymetric surveys have been performed in a number of rivers that have revealed distinct cases of bridge scour. After a brief background to the problem of bridge scour and an introduction of basic concepts, the report starts out with some general examples of bridge scour problems also illustrated by several case studies from Sweden.

The literature review includes the main processes causing bridge scour that is the starting point for identifying the factors determining bridge scour and its evolution in time. Also, the most common equations to calculate bridge scour under a variety of conditions are presented. The scour induced by a bridge is typically divided into three different mechanisms: (1) pier scour; (2) abutment scour; and (3) contraction scour. These mechanisms are discussed in detail in separate chapters, in each giving an overview of the phenomenon, presenting the most common equations for quantitative estimates, and providing some concluding remarks. Finally, the procedures recommended by various government bodies are presented in a special chapter, where guidelines from United States, Australia, and United Kingdom are reviewed. Also, guidelines from Sweden are discussed, although they are much less comprehensive and not employed to the same extent as in the other countries mentioned.

Towards the end of the report a brief discussion is provided on how a changing climate can affect bridge scour, which may occur directly through changing flow conditions and groundwater levels. Other, more indirect factors, would be different ice conditions and material transport from the river catchment that affect debris flow. Larger obstacles transported in the river (e.g., ice sheets and wood material) could get stuck at the bridge, causing blockage, which reduces the flow cross-sectional area and increases the velocity resulting in increased bridge scour.

1.4 Limitations

The report deals only with scour induced by bridges; other types of scour, such as general scour due to longitudinal transport gradients in the river, scour related to secondary flows in river bends, or scour downstream hard bottom, are not discussed. Most of the formulas included to estimate bridge scour are valid for friction material and only a few examples are given that are applicable to cohesive sediment, mainly related to recommended design procedures from different countries. Also, the objective of bridge scour analysis is often to estimate the maximum scour depth, occurring at equilibrium conditions under a certain flow, implying that most of the formulas are valid for such conditions. However, in some formulas the time evolution of the scour is described; this could be significant in the construction phase when equilibrium will not be achieved and this evolution should be considered.

Only the scour in the vicinity of the bridge structure is considered and downstream effects are typically not elaborated on. Such effects could be significant, especially related to contraction scour, causing undercutting of banks and slope stability problems. Certain material is prone to sliding, which could be triggered by slope steepening from local scour. No review of scour protection methods was performed in the study; there is a rich literature available on this topic. Finally, the selection of design conditions to estimate bridge scour based on the characteristics of the river flow is only briefly discussed in connection with the design guidelines from different countries. Deriving such conditions require calculations belonging to the field of river hydraulics.

2. Bridge scour processes and problems

2.1 Basic mechanisms

Scour at bridge crossings is the result of the erosive action of flowing water, when it has the strength to excavate and carry away material from around bridge piers and bridge abutments. Scour depth is the lowering of the river bed level and is a measure of the tendency to expose bridge foundations. Scour at bridge crossings is usually the result of the joint effects of three different scour processes (general scour, contraction scour, and local scour at piers and abutments) that may occur either independently or simultaneously, whose different origin suggests a different estimate of each individual scour contribution.

Three types of scour affect bridges:

1) Local scour is the removal of sediment from around bridge piers or abutments (piers are the pillars supporting a bridge; abutments are the supports at each end of a bridge). Water flowing past a pier or an abutment may scoop out holes in the sediment; these holes are known as scour holes.

2) Contraction scour is the removal of sediment from the bottom and sides of the river related to changes in the flow cross-sectional area. Contraction scour is caused by an increase in speed of the water as it moves through a bridge opening that is narrower than the natural river channel.

3) Degradational scour is the general removal of sediment from the river bottom by the flow of the river. This sediment removal and resultant lowering of the river bottom is a natural process not related to the presence of the bridge, but may still remove large amounts of sediment over time.

A schematic diagram of different types of scour that takes place at a bridge is shown in Figure 1.

2.1.1 Causes for scour

Stream channel instability resulting in river erosion and changing angles-of-attack can contribute to bridge scour. Debris can also have a substantial impact on bridge scour in several ways. A build-up of material can reduce the size of the waterway under a bridge causing contraction scour in the channel. A build-up of debris on the abutment can increase the obstruction area and increase local scour. Debris can deflect the water flow, changing the angle of attack, increasing local scour. Debris might also shift the entire channel around the bridge causing increased water flow and scour in another location. During flooding, the foundations of a bridge and abutments might suffer damage.

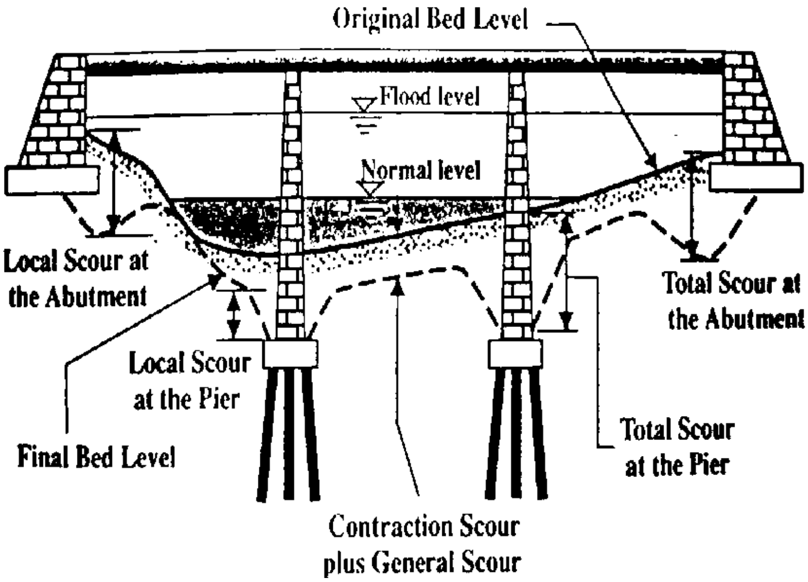


Figure 1. Schematic diagram of the types of scour that can occur at a bridge site (Melville and Coleman, 2000)

2.1.2 Flow and scour at bridge piers

At bridge sites, localized scour in the vicinity of piers poses a challenging problem to the hydraulic engineers. Failure of bridges due to scour at pier foundations is a common occurrence. The obstruction of the flowing stream by a bridge pier causes a three-dimensional separation of flow forming a vortex flow field around the pier (Dey *et al.*, 1995; Dey, 1995). To be more specific, the flow separates at the upstream face of the pier as it travels by the side of the pier, creating a vortex trail, termed horseshoe vortex, which moves downstream. As a result, local scour takes place around the pier due to the removal of bed sediments. The scour at bridge piers has been studied extensively by various researchers. Review of the important experiments and field studies was given by (Melville and Coleman, 2000; Breusers *et al.*, 1977; Raudkivi, 1991; Dargahi, 1990; Dey, 1997; Richardson and Davis, 2001; Sumer and Fredsoe, 2002, Das *et al.*, 2013).

2.1.3 Flow field around bridge piers

The flow field around a pier is coupled with a complex three-dimensional separation of the approaching flow upstream and a periodical vortex shedding downstream of the pier. The complexity increases with the development of the scour hole. The existence of the pier in the flowing stream induces a downward negative pressure gradient normal to the approaching flow. The boundary layer at the pier upstream gets through this pressure gradient set up by the pier, separating the flow and forming the horseshoe vortex. In addition, the other flow components developed occur downstream, including a wake vortex and a bow wave. Figure 2 shows the different components of flow field around a pier.

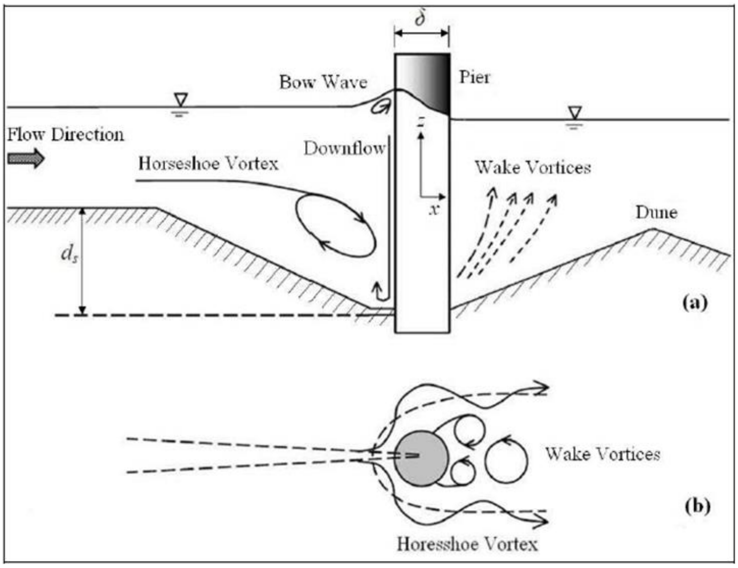


Figure 2. Flow structure at cylindrical pier: a) vertical cross-section b) horizontal cross-section (Das et al., 2013)

Thus, the following types of vortexes develop at bridge piers:

- **Horseshoe vortex:** The velocity experiences a reversal within the scour hole in the vertical direction forming a horseshoe vortex (Figure 2). Viewed from the top, this vortex system has the characteristic shape of a horseshoe and, thus, is called a horseshoe vortex.

- **Wake vortex:** The velocity experiences a reversal within the scour hole along the horizontal water surface forming a wake vortex (Figure 2). Wake vortices form at the downstream side of piers and are the result of flow separation at the sides of the pier. The wake vortices dissipate as they move downstream.

2.1.4 Different types of scour

Unidirectional flow in an open channel becomes three-dimensional when encountering a protruding vertical obstacle. The flow field at a rectangular abutment, embedded vertically in an erodible bed, is intricate and the intricacy increases with the development of scour hole due to vortex flow as a result of three-dimensional flow separation. Although the flow field at piers has been extensively explored, the flow field at abutments has received less attention. The primary vortex, which is analogous to the horseshoe vortex at bridge piers, is primarily responsible for scour hole development at bridge abutments. The downflow component associated with the primary vortex is the main scour agent.

The primary vortex and downflow are confined mostly within the scour hole beneath the original bed level. The flow patterns and maximum downflow are relatively unaffected by changes in approach flow depth. These findings are similar to those for pier scour. The inner core of the primary vortex occupies about 17% of the scour hole area and contains up to 78% of the total circulation of the flow. The primary vortex is elliptical in shape with an inner core region approximating that of a forced vortex and an outer core region approximating that of a free vortex. A secondary vortex with counter rotational direction to that of the primary vortex occurs next to the primary vortex. The secondary vortex is believed to have the effect of limiting the erosive capacity of the primary vortex (Kwan and Melville, 1994). The flow pattern around a bridge abutment is shown in Figures 3 and 4.

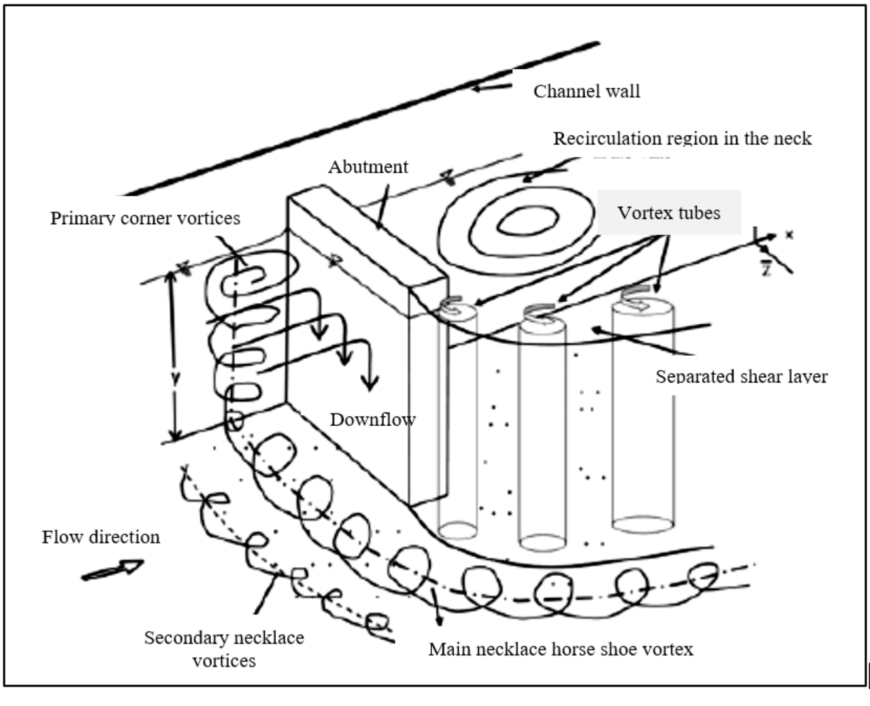


Figure 3. Flow structures around a vertical-wall abutment (Koken and Constantinescu, 2008a, b)

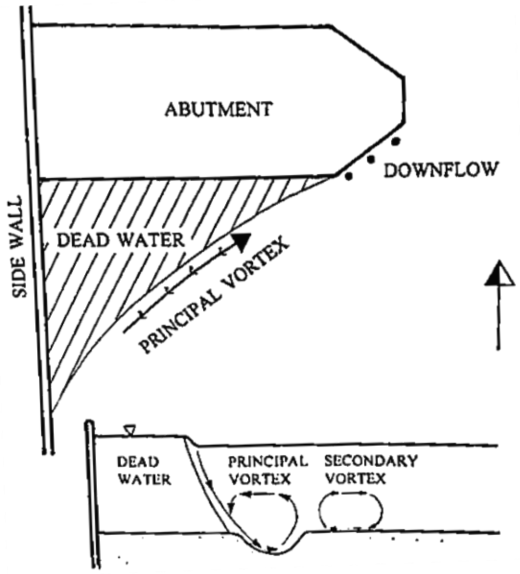


Figure 4. Schematic illustrations of the flow past an abutment (Raudkivi, 1998)

2.2 Example of scour problems

Bridge failure due to scour around piers and abutments is a common phenomenon throughout the world. There are many such incidents that have happened and caused huge economic loss in a number of countries. For example, about 60% of bridge failures leading to total bridge collapse in the United States since 1950 have been related to the scour of bridge foundations. As previously mentioned, the Colorado Department of Transportation (CDOT) estimated that several state highway bridges were destroyed and many were seriously damaged by floods in the year of 2013 (https://www.codot.gov/).

An event took place early 1987 in Wales, UK, where a passenger train fell into the River Towy near Llandeilo. The accident was caused by the Glanrhyd Bridge being partially washed away by the swollen river (Railway Inspectorate, 1990). The Schoharie Creek Bridge was a New York State Thruway bridge over the Schoharie Creek near Fort Hunter and the Mohawk River in New York State. On April, 1987 it collapsed due to bridge scour at the foundations after a record rainfall (Croyle, 2019). In the year 2013, CPR Bonnybrook Bridge, Calgary, Alberta, steel railroad bridge collapsed partially due to scouring from flood event of the Bow River (CBC, 2013). These are just a few examples of bridge failures; however, there are numerous numbers of incidents like these in the past that have encouraged engineers and scientists to investigate the problem of local scour.

Table 1. Percentage of collapsed bridges over time (Imhof, 2004)

Causes of bridge collapse	All bridges (237)	Before 1900 (35)	1900-1940 (27)	1941-1990 (117)	1991-2004 (58)
Natural hazards	40	31	37	37	50
Limited knowledge	9	14	30	7	1
Design error	5	9	0	4	5
Overloading	14	26	4	14	14
Impact	25	17	29	30	19
Human error	3	0	0	2	7
Vandalism	1	3	0	0	2
Deterioration	3	0	0	6	2

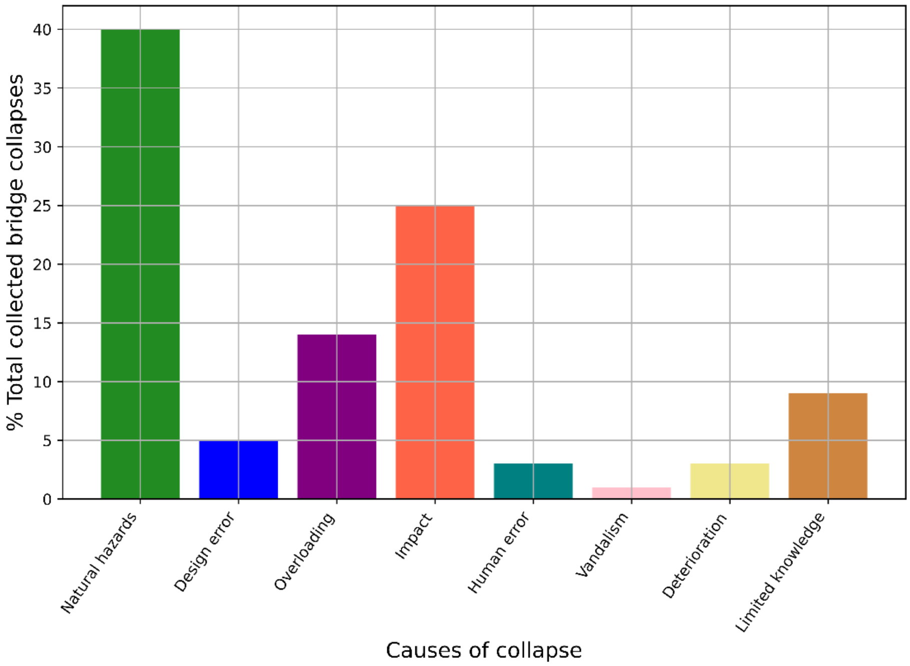


Figure 5. Main causes of bridge collapse (Imhof, 2004)

According to a comprehensive collection of bridge failure data worldwide gathered by Imhof (2004) (Table 1), natural hazard is the main cause of bridge collapse (Figure 5) and among the natural hazard listed causes, flooding or scour is responsible worldwide for around 60% of the collapses (Figure 6).

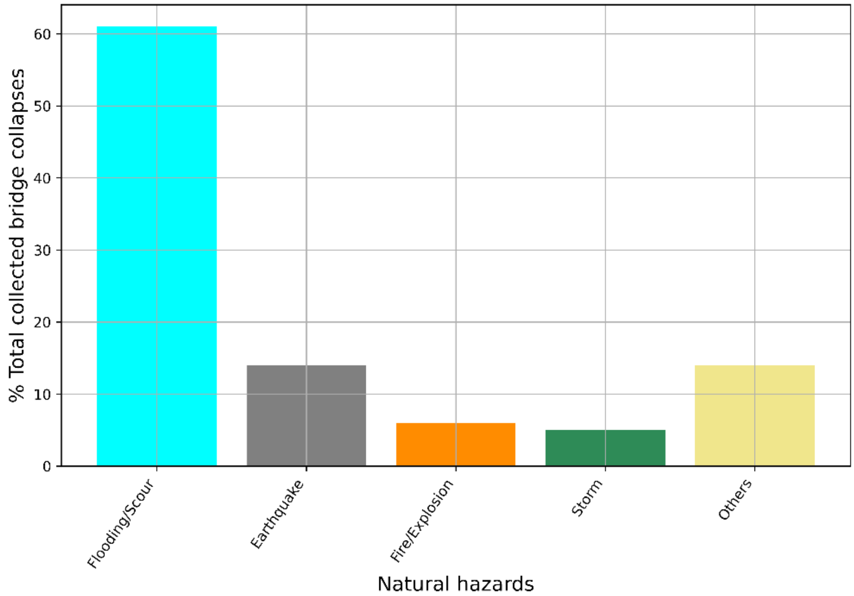


Figure 6. Different natural hazards causing bridge collapse (Imhof, 2004)

Bridge scour has been a subject of interest and importance to people from the time of the earliest civilizations. In modern times, one of the first systematic studies of bridge pier scour was undertaken in Germany at the Zeuner laboratory by Engels in the early 20th century. Later, a significant study of scour at piers and scour countermeasure was performed at the Chatou laboratory by Chabert and Engeldinger (1956). Their excellent data were the first to illustrate the important distinction between clear-water and live-bed scour. At about the same time, Laursen and others published results from comprehensive investigations of bridge scour at the Iowa Institute of Hydraulic Research (Laursen and Toch, 1956; Laursen, 1958; Laursen, 1962; Laursen, 1963). Perhaps the most important contribution of the Iowa studies was the concept of adapting solution to scour in the long rectangular contraction to the case of local scour at piers and abutments (Straub, 1935).

Other earlier scour studies in United States of note were conducted at Colorado State University by Shen *et al*. (1966), Shen *et al*. (1969) on pier scour and (Liu *et al*., 1961) on scour at abutments. Other researchers such as Garde *et al*. (1961), Gill (1972), Gill (1981), Lacey (1930), Blench (1969) also made important contributions to the knowledge of scour at abutments. Numerous studies have been conducted with the purpose of predicting scour and various equations have been developed (Laursen and Toch, 1956; Liu *et al.*, 1961; Shen *et al.*, 1969; Breusers *et al.*, 1977; Jain and Fischer, 1979; Raudkivi and Ettema, 1983; Melville and Sutherland, 1988; Froehlich, 1989; Melville, 1992; Abed and Gasser, 1993; Richardson and Richardson, 1994; Lim, 1997 and Heza *et al.*, 2007). To test the accuracy of the developed bridge scour equations, comparative studies have been conducted by many researchers (Jones, 1984; Johnson, 1995; Mueller, 1996 and Landers and Mueller, 1996).

Although the general dynamics of bridge scouring is known and several studies are available in the literature for interpreting the scour process and predicting the maximum scour depth, over the past decades a number of cases with bridge damages that occurred during river floods has shaken the scientific community and spurred engineers and researchers to improve scour prediction models and to renew scour measurement techniques.

2.3 Swedish situation

2.3.1 Background

At present, the knowledge of bridge scour in Sweden is limited and no major research has been conducted during the last decades (Hjorth, 1972, 1974, 1975). The work by Hjorth (1975) resulted in a Ph.D. thesis involving studies on the nature of local scour that focused on pier scour and scour around pipelines, not including abutment or contraction scour. Bridge piers of different shapes were investigated and only the conditions for the initially, undisturbed bed was studied (i.e., no scour hole development). The details of the flow field were measured in the laboratory and compared with analytical solutions. About a decade later, a literature review on bridge pier scour was carried out by Dargahi (1982), who also recorded two cases of bridge failures in Sweden due to local scour (briefly discussed in the following). The report summarized different methods of scour protection. Based on the review, Dargahi (1982) provided some suggestions for the most suitable formulas to estimate the scour depth as well as for erosion protection design against pier scour.

No national overview has been performed that summarizes existing problems related to bridge scour; there are many bridges in Sweden and a significant number of them are quite old and have possibly experienced scour. In recent years, primarily in connection with studies on flooding, many rivers have been surveyed in detail with regard to the bathymetry. This has in several cases revealed distinct scour holes in the vicinity of bridges indicating that this is a common problem. Although relatively few cases so far have been recorded where bridge scour has caused structural collapse, it is not unusual to observe structural damage that can be attributed to bridge scour. In the following, three case studies are presented as examples where marked bridge scour have been observed in Swedish rivers. A brief overview of the site and general conditions together with the observed scour problems and their causes are provided. In general, there is a need to make a more comprehensive survey of the bridge scour in Sweden in order to establish the scale and causes of this problem, particularly in view of possible changes in the climate.

In Sweden, about 21,000 bridges are managed by the Swedish Transport Administration ("Trafikverket", in Swedish) of which 17,000 are road bridges and 4,000 railroad bridges. Many of them span bodies of flowing water, such as rivers, streams, and estuaries, where construction elements are in contact with the water. If the bed material is erosive, scour is likely to occur and considerations are needed to assess the risk of scour hole development that may cause structural damage. However, the problem of bridge scour is of potential interest to several other stakeholders, not only to Trafikverket. Examples of such stakeholders in Sweden that may have an interest in bridge scour problems are local municipalities, county boards, government agencies, consultant companies, and construction companies.

The present Swedish guidelines for scour analysis and design was developed by the Swedish Road Administration ("Vägverket", in Swedish) in the 1980's. This government agency became a part of Trafikverket in 2010, but the report is still the main document regarding bridge scour and methods for erosion protection (Vägverket, 1987). The report also discusses propeller erosion as well as erosion of road banks due to surface waves. Although the Swedish guidelines are not as comprehensive as, and rather more simplistic than, the guidelines discussed from other countries, a summary of the former are also included in the chapter about design procedures in the present report. The guidelines in Vägverket (1987) have subsequently been referred to in other reports from Trafikverket (2011a, 2011b) as well as by Carlsson and Persson (2006). In assessing bridge scour in a risk-based approach, the reports Trafikverket (2017, 2019) contain valuable information of general applicability.

2.3.2 Bridge scour in Rönne å, Ängelholm

Rönne å (river) runs through south Sweden and is the second longest river (approximately 85 km) in the most southern province Skåne (Scania); see Figure 7. The river has a total catchment area of close to 2,000 km^2, originating in a major lake (Ringsjön) and discharging its water at a coastal outlet in Skälderviken Bay. The average flow in the river at the outlet is about 23 m^3/s, whereas the estimated flows for return periods of 50 and 100 years are about 200 and 220 m^3/s, respectively, based on a fitted Gumbel distribution to data from the Swedish Meteorological and Hydrological Institute (SMHI, 2021a); for more information, see Inamdeen (2020) and Inamdeen *et al*. (2021). The flow data were simulated with the S-Hype hydrological model covering 40 years of data (1981-2020); the model has been validated with measurements. Ängelholm Municipality is located at the most downstream part of Rönne å, close to the outlet.

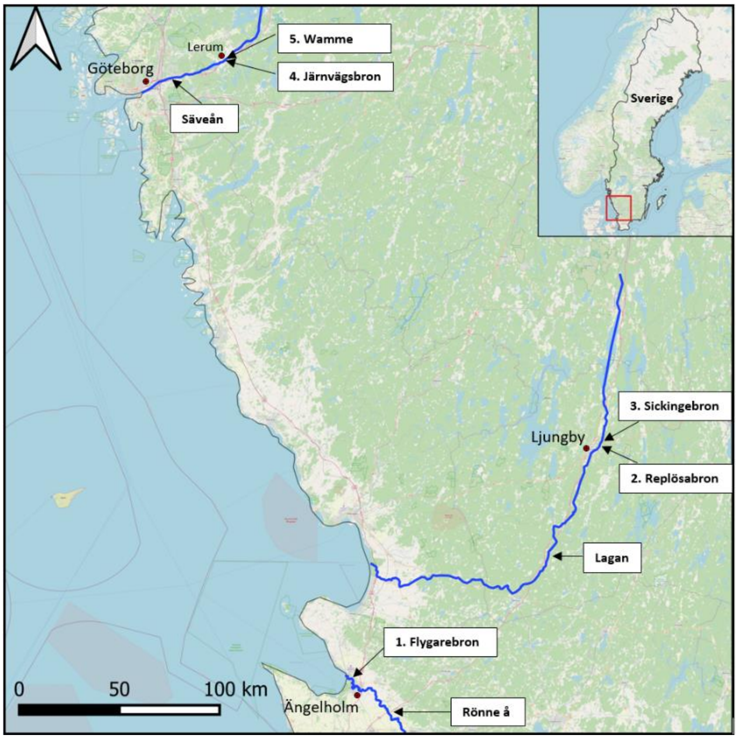


Figure 7. Overview map of south Sweden with study locations displayed, including names of bridges

During a bathymetric investigation performed by MarCon Teknik AB (MTE, 2020), the lower 12 km of the river was surveyed in detail using a combination of multi-beam echo sounding and LiDAR techniques. Analysis of the bathymetric data revealed pronounced scour holes at a large number of locations having different magnitude and origin. Most of the holes were due to natural scour, where meandering bends and hard bottom are responsible for the induced secondary flows that locally increase sediment transport and cause erosion. However, in some cases local scour developed related to the presence of a bridge, often together with other mechanisms such as hard bottom scour.

About 1.5 km upstream from the outlet, pronounced scour was detected downstream two closely located bridges, where one is a railroad bridge that is an important link in the transport system along the Swedish west coast (Skälderviksbron; see Figure 8a) and the other a walk and bicycle bridge (Flygarebron; see Figure 8b). Figure 9 shows a close-up of the two bridges, the bathymetric map of the bottom downstream Flygarebron, and cross sections along and across the river that display the bed level with regard to the undisturbed bottom. The maximum depth of the scour hole is about 4 m, whereas the corresponding vertical distance from the average depth of the undisturbed river bed is 1.5 m. The length (along the river) and the width (across the river) of the hole is 18 m and 20 m, respectively.

Figure 8. Pictures of (a) Skälderviksbron and (b) Flygarebron located in Rönne å where local scour has been observed (from BaTMan, 2021)

The scour hole downstream Flygarebron is a result of pier scour influenced by the presence of hard bottom. Sediment sampling showed that in the bottom of the scour hole fine material was present (e.g., silt and clay), see Inamdeen and Larson (2021). At upstream sampling points under the two bridges hard or armored bottom was detected and no useful sampling could be performed. In Figure 9 two square-shaped piers may be observed that belong to Flygarebron, although the bridge in total have five piers, where a few piers support the span over a smaller side channel (not seen in the figure). Skälderviksbron is also built on five piers, but they are slenderer and because of the exposed hard bottom no pronounced scour holes can be observed near these bridge piers. The interaction between the two bridges with regard to the flow and the bed conditions, including hard bottom and armoring, make the scour situation quite complex at the site. As can be seen from Figure 9, the marked scour hole developed downstream two of the bridge piers, somewhat asymmetric, with a small ridge in the middle of the hole extending along the river.

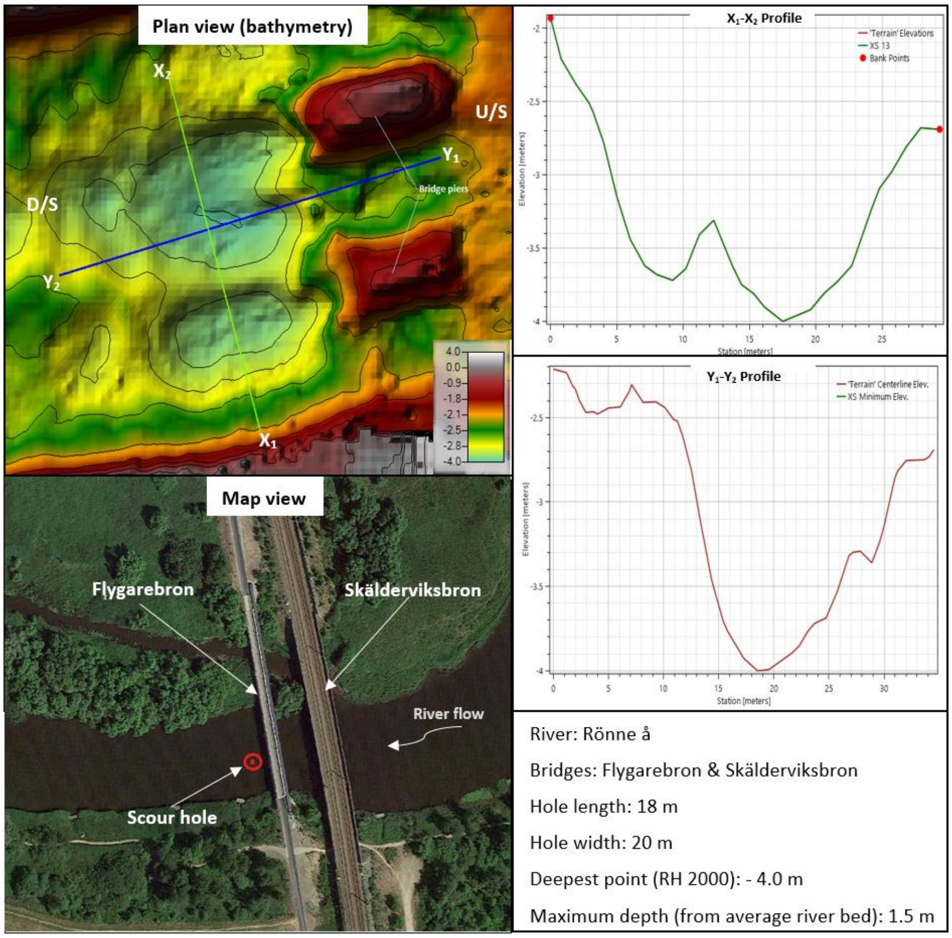


Figure 9. Plan view of the two bridges Skälderviksbron and Flygarebron in Rönne å, a bathymetric map of the bottom downstream Flygarebron showing a scour hole, and cross sections along and across the river displaying the bed level with regard to the undisturbed bottom

2.3.3 Bridge scour in Säveån, Lerum

Säveån discharges its water into Göta älv in the central part of Göteborg, the second largest city in Sweden that is located on the west coast (see Figure 7). The river has a length of about 130 km and originates in Lake Säven. In the downstream part, the river runs through an area with glacial and postglacial clay deposits sensitive to erosion, which can cause steepening of the river bed and banks that induces slides of the banks and adjacent areas (SGI, 2017). The Municipality of Lerum is located about 20 km upstream the river outlet and it is particularly exposed to erosion and bank stability problems (Svantesson, 2017; COWI, 2018). Several of the bridges in this area are suffering from local scour, which causes increased bank and bed slopes with the possibility of slides.

The size of the catchment contributing to the flow in Säveån at Lerum is 1340 km^2 and the average flow in the river about 18 m^3/s (SMHI, 2021a). By fitting a Gumbel distribution to data from SMHI the estimated flows for return periods of 50 and 100 years are about 95 and 105 m^3/s, respectively. The flow data was generated through simulations with the S-Hype model for the same period as for Rönne å.

Figure 10. Pictures of (a) Järnvägsbron (across Säveån) and (b) Wamme bro located in the Säveån where local scour has been observed (from BaTMan, 2021)

Bathymetric surveys commissioned by the municipality and performed by MarCon Teknik AB (Karlsson, 2021) revealed several pronounced scour holes downstream bridges. For example, such holes were detected downstream the bridges Järnvägsbron (over Säveån; see Figure 10a) and Wamme bro (see Figure 10b). Järnvägsbron is a critical link in the railroad system between Gothenburg and Stockholm, whereas Wamme bro, built in the 1850's has great historical and cultural value. Slides have been recorded in the areas adjacent to the bridges, the latest one in 1981 (SGI, 2017). A large number of geotechnical investigations have been performed to determine the risk of slides and to design appropriate measures to prevent mass failure. This includes different types of erosion protection in the river itself. However, no particular studies have been carried out with regard to local erosion related to the bridges.

The scour hole observed downstream Järnvägsbron is approximately 1.5 m below the undisturbed bed with a length and width of 30 m and 10 m, respectively (see Figure 11). The hole is most likely the result of a combination of contraction scour from the bridge together with bend scour. Wamme bro has a more pronounced effect on the downstream river bottom, where the maximum scour depth is 3.6 m from the average river bed. The scour hole has a more symmetric shape than at Järnvägsbron with a length of 30 m and a width of 20 m (see Figure 12). The scour at Wamme bro is a result of pier scour together with contraction scour; the flow cross section is quite restricted at bridge.

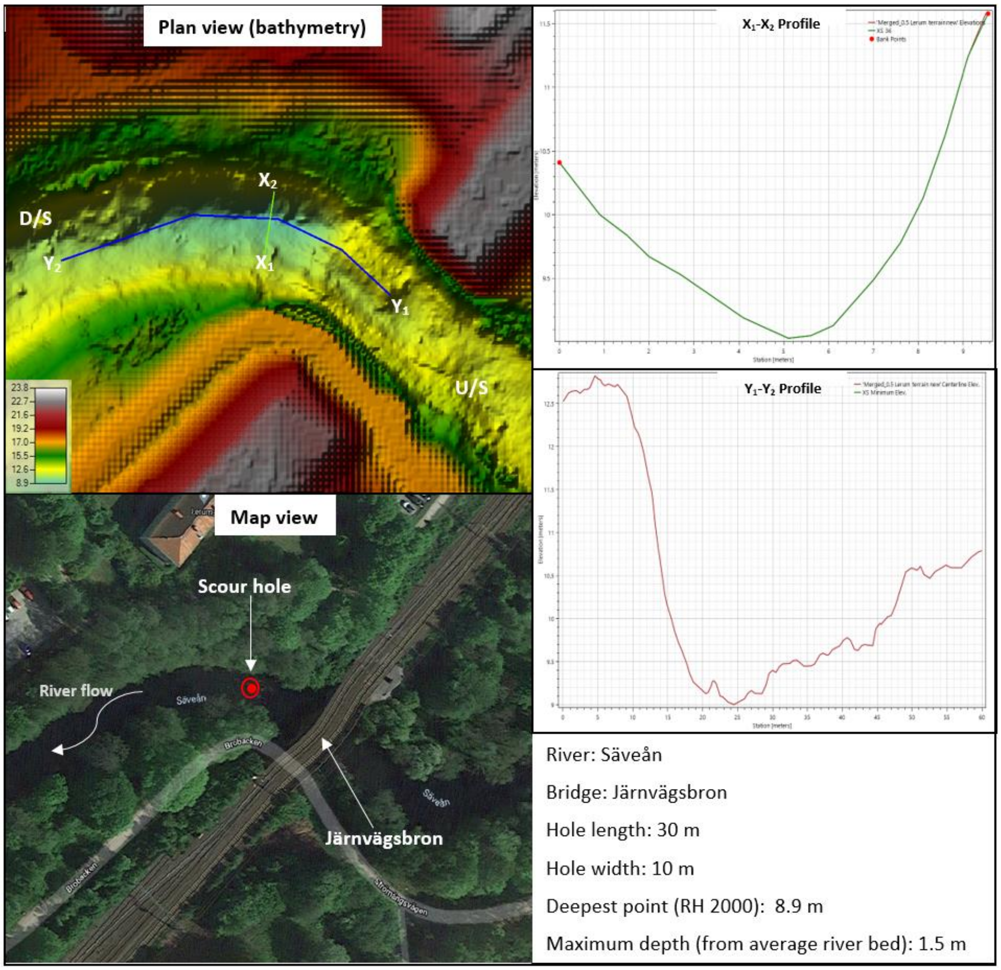


Figure 11. Plan view of the bridge Järnvägsbron in the river Säveån, a bathymetric map of the bottom downstream the bridge showing a scour hole, and cross sections along and across the river displaying the bed level with regard to the undisturbed bottom

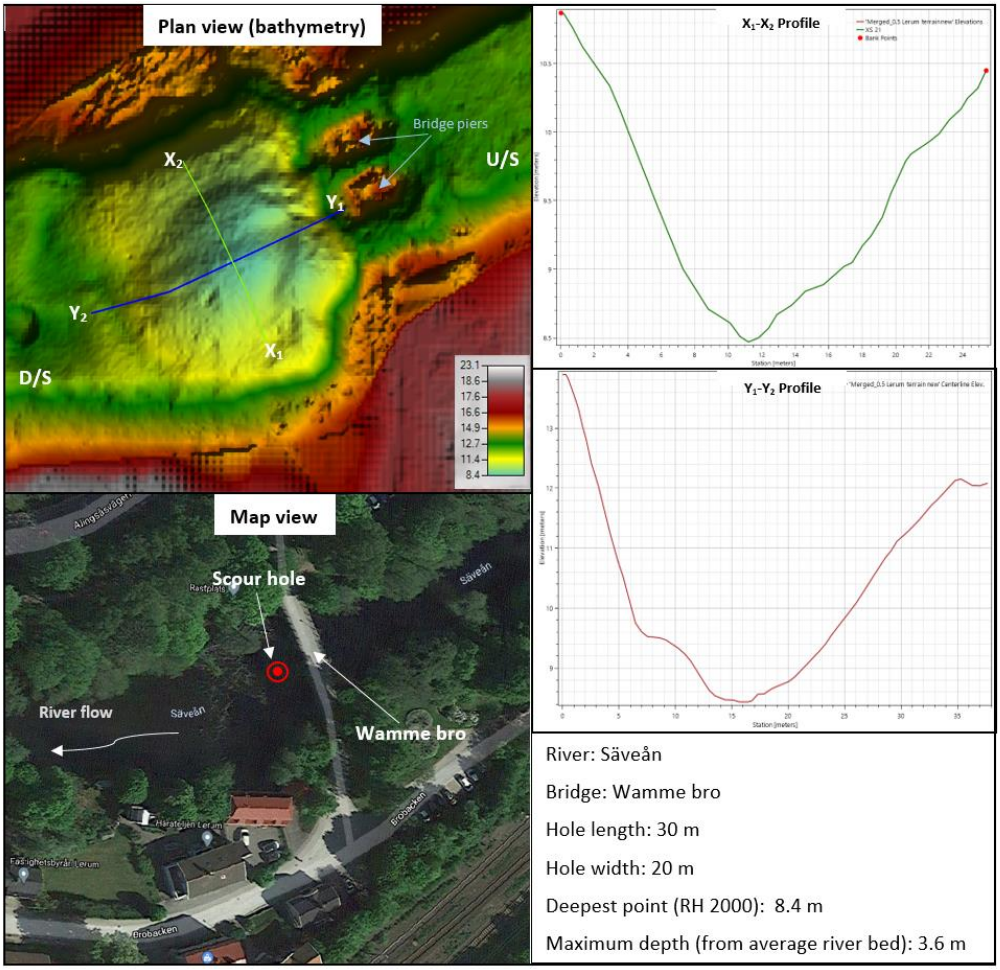


Figure 12. Plan view of the bridge Wamme bro in the river Säveån, a bathymetric map of the bottom downstream the bridge showing a scour hole, and cross sections along and across the river displaying the bed level with regard to the undisturbed bottom

2.3.4 Bridge scour in Lagan, Ljungby

Lagan is one of the major rivers discharging its water on the Swedish west coast near the city of Laholm. The river is about 250 km long and originates in the lake Tahesjön. The municipality of Ljungby is located approximately halfway between the starting point of the river and the outlet to the sea (see Figure 7). Erosion along the river banks of Lagan has been observed in Ljungby as well as local scour at some bridges. The catchment area of the river at Ljungby is about 3000 km^2 and the mean flow 35 m^3/s. Based on a fitted Gumbel distribution to the data from SMHI (2021a), the flows for return periods of 50 and 100 years are about 140 and 150 m^3/s, respectively (time series covering the same period as for Rönne å).

Partly with regard to the former problem, Clinton (2012) performed a bathymetric survey along a 6-km stretch of the river at Ljungby. Geotechnical analysis of the river bed was also performed together with investigations of the bed conditions in general (e.g., indications of former slides, presence of sand banks and different types of debris). The surveying was carried out with a multi-beam echo sounder; however, since the craft used needed a draft of more than 1 m, the shallow areas could not be surveyed as well as an area downstream a hydropower plant, where the flow conditions were too turbulent. Most of the material in the area consists of sandy, silty moraine, whereas the river mainly flows in an alluvial formation deposited during the latest ice age (Clinton, 2012).

Along the studied area there are eight bridges, of which the two most northerly located bridges are Sickingebron and Replösabron (see Figures 13a and 13b, respectively). Downstream both these bridges pronounced scour holes have been observed, most likely related to contraction scour induced by the bridge and the presence of hard or armored bottom upstream the hole. At Sickingebron the depth of the scour hole is about 6 m below the undisturbed river bed with a width of 30 m and a length of 50 m (see Figure 14). For Sickingebron, bend effects may also be important, affecting the flow direction through the bridge, making the scour hole asymmetric. Downstream Replösabron the scour hole has a depth of 2.5 m with regard to the adjacent undisturbed bed and the width is 30 m and the length 40 m (see Figure 15). Limited information was available to assess the geotechnical conditions at the sites and how they may influence the scour hole development.

Figure13. Pictures of (a) Sickingebron (from BaTMan, 2021) and (b) Replösabron (photo by Clemens Klante, LTH) located in the river Lagan where local scour has been observed

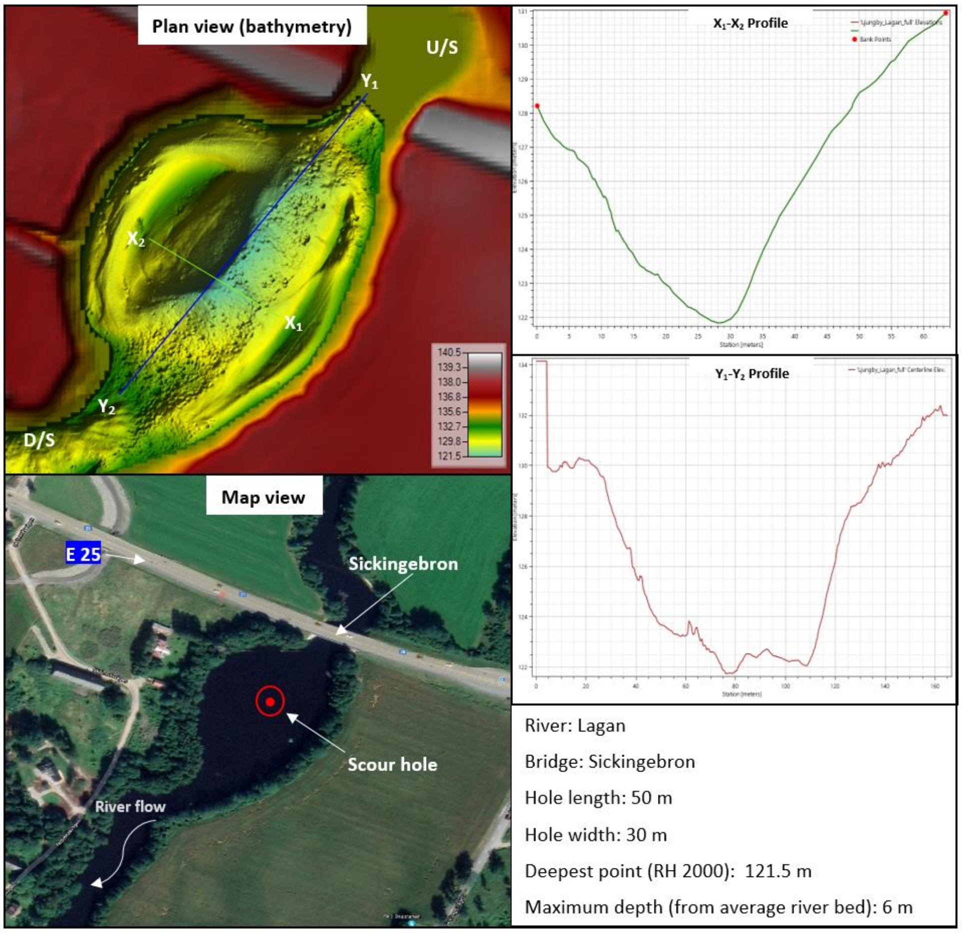


Figure 14. Plan view of the bridge Sickingebron in the river Lagan, a bathymetric map of the bottom downstream the bridge showing a scour hole, and cross sections along and across the river displaying the bed level with regard to the undisturbed bottom

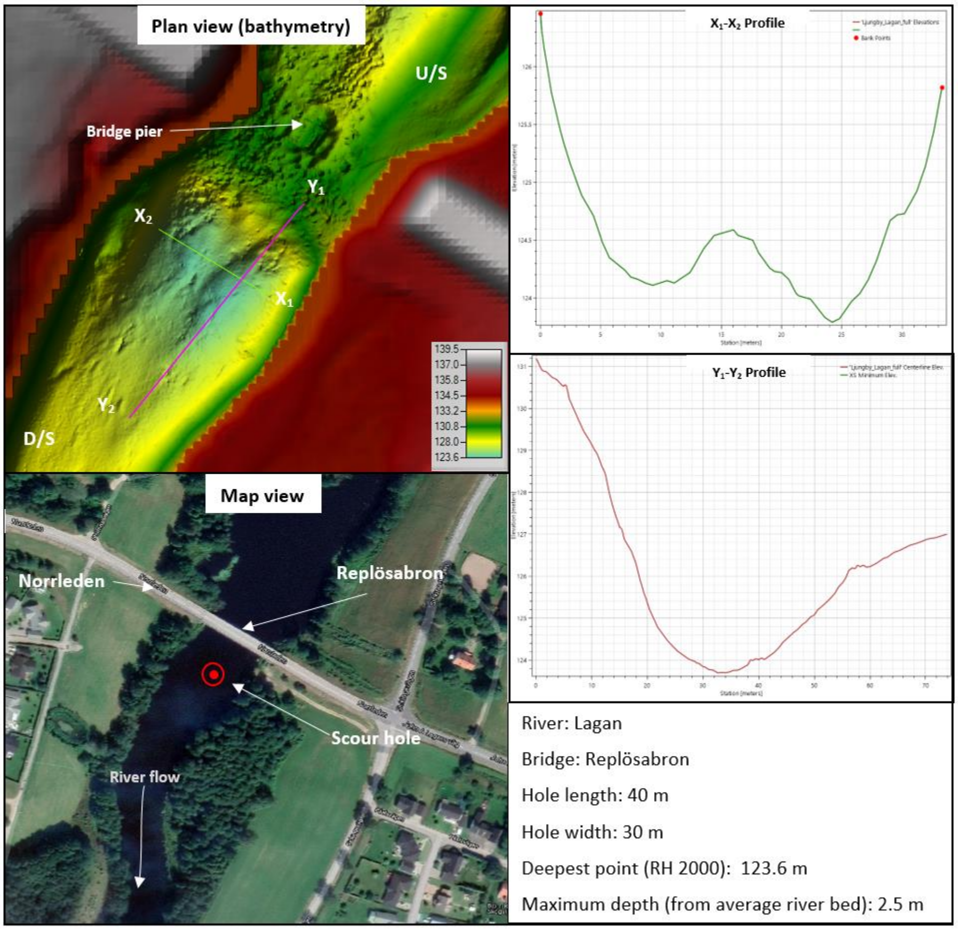


Figure 15. Plan view of the bridge Replösabron in the river Lagan, a bathymetric map of the bottom downstream the bridge showing a scour hole, and cross sections along and across the river displaying the bed level with regard to the undisturbed bottom

2.3.5 Bridge failures in Sweden due to bridge scour

No detailed, comprehensive investigation of bridge scour has been performed in Sweden to quantify the problem of local scour and the damage it has caused on bridges. The few cases reported here (previous sections) indicate that scour holes are common downstream bridges and in some cases damages such as cracks in wing walls have been observed (Inamdeen, 2020). However, there are a few cases of bridge failures in Sweden due to local scour that was presented in the review by Dargahi (1982). Two cases presented in this report is briefly reviewed; all material is compiled from Dargahi (1982), who received it from the Swedish National Road Administration.

During the final stage of the construction of a bridge across the Österdal River (May 1979), located about 500 km northwest of Stockholm, high flows developed that scoured one of the bridge piers causing failure of the deck. Inspection by a diver after the event revealed a scour hole with a maximum depth of 2 m. The main reasons for the pronounced scour and failure were attributed to (1) reduction of the effective flow area by about 40% due to extension of the abutments; (2) inadequate scour protection not fulfilling the recommendations; and (3) the sheet piles around the pier during the construction increased the area exposed to the flow. Although the bridge was designed for much higher flows than those occurring during the failure, the reduction of the flow area resulted in velocities sufficient to cause scouring. Figure 16 illustrates photos of the bridge over Österdal River after the failure due to scour.

Figure 16. Failure of the bridge over Österdal River during construction due to scour around one of the piers (from Dargahi, 1982)

Figure 17. Failure of the bridge over Lainio River due to scour (from Dargahi, 1982)

The bridge over Lainio River in the most northern part of the Sweden suffered serious damage in connection with ice jamming in May 1973 (see Figure 17). The scouring of the river bed caused the middle pier of the bridge to settle about 1.7 m. No special scour protection was placed around the bridge since the native sediment was judged to be sufficiently stable. Inspection after the damage revealed a deepening of the river bed by 3 m near the western abutment, which is markedly protruding into the river. During the event producing the scour, the flow was large, but not extreme; however, breakup of the ice cover resulted in blockage of the flow passing the river, causing backwater effects and large velocities downstream the bridge.

3. Bridge pier scour

3.1 Overview

Scour at hydraulic structures is an important process that alters the sediment-flow equilibrium and have consequences for the ecosystems at the cross-sectional scale, flood vulnerability in the vicinity of the structure, and the local geomorphological characteristics. Thus, it is important that engineers look at the governing processes and provide estimates of scour depth and its evolution at structures in order to predict such impacts at the local scale.

The main aspects to be taken into account when analyzing bridge pier scour may be summarized:

- **Processes**: to better understand the dynamics triggering pier scour, including an analysis of the type of scour occurring at bridge piers, the most influencing factors, and failure mechanisms
- **Measurements**: to compile available data, possibly to collect new data; acquiring relevant scour data is one of the main difficulties faced in real world practice
- **Estimates**: to review different approaches the scientific literature offer for the estimation of the maximum local scour depth

Scour at bridge crossings is the result of the erosive action of flowing water, when it has the strength to excavate and carry away material from around bridge piers and bridge abutments (Richardson and Davis, 2001). Scour depth is the lowering of the river bed level and is a measure of the tendency to expose bridge foundations (Melville and Coleman, 2000). Although scour process mechanisms are well established, quantifying the magnitude of scour at bridge crossing is not an easy task, due not only to the complexity of the cyclic nature of the phenomenon, but also because bridge geometry, river channel morphology, and hydrologic regime are different and unique for each particular bridge. As previously pointed out, scour at bridge crossings is usually the result of the joint effects of three different scour processes (general scour, contraction scour and local scour at piers and abutments) that may occur either independently or simultaneously, whose different origin suggests a different method to estimate each individual scour contribution.

A channel with a mobile bed is usually exposed to general scour, which takes place independently of the presence of the bridge and is due to streambed elevation changes in the reach where the bridge is located. The main causes of general scour are to induce aggradation or degradation of the bed channel. This scour can be a result of either natural phenomena, such as channel straightening, climate changes, and land activities (landslides, mudflows), or due to human activities, such as land-use changes (deforestation, urbanization), dam and reservoir construction, river bed material mining, and channel alterations.

In the presence of bridge crossings, additional scour – known as *local scour* - is induced by the local change in cross-sectional geometry due to the presence of the bridge (Graf, 1998; Richardson and Davis, 2001). Local scour usually results from the joint effects of contraction scour, due to the flow velocity increase associated with the reduction of channel cross section, and the pier and abutment scour, due to the (local) alterations of the flow field induced by piers and abutments (Graf, 1998).

Factors that may influence scour depth and scour rate vary according to the type of scour process. Geomorphic characteristic of the catchment and river bed characteristics are the main causes of general scour; the type of vegetation, rainfall regime, riverbed sediment climatic factors determine the water and sediment transport rate at the bridge reach; channel cross-sectional shape, bridge location, and valley setting are riverbed characteristics that may determine the bed channel tendency towards degradation (Melville and Coleman, 2000).

Man-induced structures, such as dams and reservoirs, or human activities, such as bed gravel mining, are also responsible for general scour. Italy, for example, has reported streambed gravel mining as the number one cause of general scour that has increased significantly after the second World War and that has not been properly controlled, causing exacerbated bed channel level lowering.

Contraction scour is mainly influenced by the magnitude of the cross-sectional width restriction due to bridge piers and abutments in the channel, causing the flow contraction at the bridge site, and by flow debris accumulation. Floating woody debris being transported by the flow may accumulate at bridge piers and abutment, partially or, in some cases, totally clogging the bridge opening. The potential for debris accumulation at bridge foundations is strongly related to catchment characteristics and to the type of vegetation in the catchment.

Moreover, vegetation accumulated at bridge foundations exposes to scour a larger area around piers and is one of the main causes of local scour around pier foundations. The main factors in assessing local scour around piers are bridge geometry: the shape, type, and length of the piers; bridge location in relation to bed channel; and alignment of piers with flow direction.

Local scour may be influenced by the armoring phenomenon, due to the different mobility of non-homogeneous bed soils: as scour develops in time, fine-particles at the bed surface are carried away and, when the flow is not able to remove all sizes of widely-graded bed sediments, the coarser materials may create an armor layer, protecting channel bed from the flow erosive action. However, if the armor layer is not stable, when flows able to exceed the mobility threshold of the coarser material occur, the underlying riverbed material is highly exposed to erosion and deep scour hole are expect to occur.

3.1.1 Parameters Influencing Scour Depth at Piers

Scour at piers is influenced by various parameters (Breusers *et al*. 1977), which may be grouped as follows:

- Parameters related to the pier: Size, shape, spacing, number, and orientation with respect to the approaching flow direction.
- Parameters related to the bed sediment: Median grain size, particle size distribution, mass density, angle of repose, and cohesiveness.
- Parameters related to the approaching flow condition: Approaching flow velocities, approaching flow depth, shear velocity, and roughness.
- Parameters related to the fluid: Mass density, viscosity, gravitational acceleration, and temperature (may not be important in scour problems).
- Parameters related to the time: Time of scouring for an evolving scour hole.
- Parameters related to the unsteadiness of the flow: Passage of a flood wave in rivers and waves in marine environment.

The relationship showing the influence of various parameters on the equilibrium scour depth d_s at piers can be given in functional form as:

$$d_s = f_1(U_1, y_0, \rho, \rho_s, \nu, b, d_{50}, \sigma_g, t)$$

where b = pier width; and t = time of scour. The dependency of scour depth on various parameters and their definition, as well as the general notation, are given in the following subsections.

3.1.2 Clear-Water and Live-Bed Scour

Local and contraction scour depend on the balance between streambed erosion and sediment deposition. To this end two different scour regimes have been defined, namely clear-water scour and live-bed scour (Graf, 1998; Melville and Chiew, 1999; Richardson and Davis, 2001). In the former case no sediments are delivered by the river or the bed material is transported in suspension through the scour hole at less than the capacity of the flow. In the latter case an interaction exists between sediment transport and scour processes, due to bed material being transported from the upstream reach into the crossing.

Live-bed scour shows a cyclic nature: the scour hole that develops during the rising stage of a flood refills (totally or partially) during the falling stage (Richardson and Davis, 2001). It follows that in live-bed conditions the presence of sediment loads leads to smaller scour depths than in clear-water conditions. Moreover, a different evolution of the hole scouring is expected: in clear water, the scour depth increases slowly and tends to reach a stable solution; in live bed conditions, the scour depth increases rapidly and, due to the interaction between erosion and deposition, it tends to fluctuate around an equilibrium value (Figure 18) (Raudkivi and Ettema, 1983; 2000; Richardson and Davis, 2001).

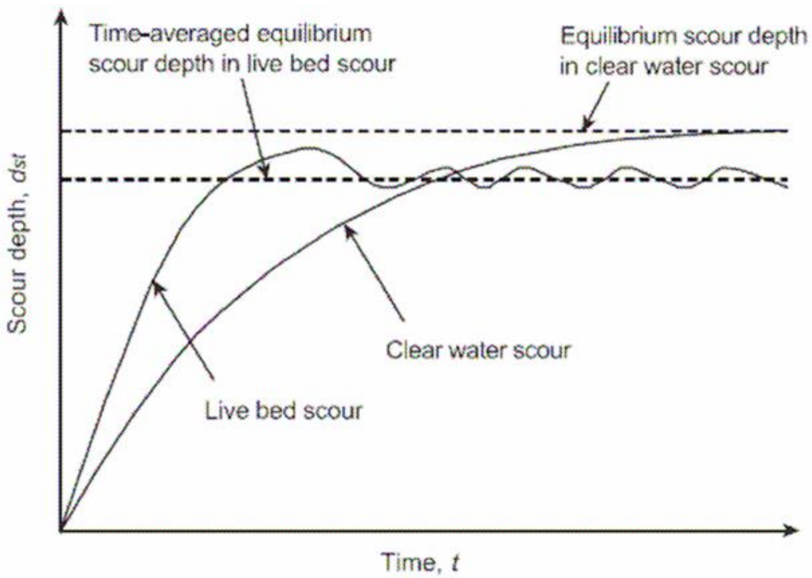


Figure 18. Time evolution of scour depth in clear-water and live-bed conditions (Chabert and Engeldinger,1956; Raudkivi and Ettema, 1983)

Laboratory research has dominated the field of local scour at bridge piers. Such research is limited by the range of hydraulic conditions typically tested and is conducted primarily under steady-flow conditions with uniform bed material. Relations and predictive equations developed from laboratory research have not been adequately verified by the use of field data.

Research into local scour problems have involved attempts to obtain a correlation between scour depths and some other chosen parameter or parameters. Such correlations usually resulted from model study investigations. Although the results obtained in this way may have been entirely adequate for the particular problem defined by the researcher's choice of parameters, attempts to generalize the results to prototype scour problems have not been successful. This is because scour in the field is generally orders of magnitude more complicated than its laboratory counterpart. Thus, it has been found necessary to over-simplify the phenomenon in order to obtain a laboratory model for research purposes. The alternatives are either a much more elaborate model investigation or the direct collection of prototype data.

3.2 Predictive formulas

Local pier scour has been a popular topic of study for many laboratory researchers. A large number of equations have been proposed for predicting the depth of scour at a bridge pier (McIntosh, 1989). Most local scour equations are based on research in laboratory flumes with non-cohesive, uniform bed material, and with limited verification of the results with field data. In evaluating and applying scour-prediction equations, it is valuable to know the limitations of the equations, the conditions for which they were developed, how the underlying data were interpreted, and the methods used to develop the equations. Such information about the most common equations has previously been published in Landers and Mueller (1996), Mueller (1996) and Pritsivelis (1999).

Analysis of bridge scour field data is more complicated than analysis of laboratory data because in the field all explanatory variables have the potential to vary at the same time. In the laboratory, all explanatory variables can be held constant and a specific variable systematically changed to study its effect. These controlled laboratory investigations, however, may not adequately describe the variability and interaction of variables present in natural conditions. In the field, all variables can change and interact; the effect of individual variables cannot be easily isolated. Although variables and dimensionless parameters from field data can be compared with laboratory data, the effect of all variables is present in the field data. For example, in a comparison of the effects of velocity, the field data will also include the effects of flow depth, bed material properties, pier shape, and pier size.

The depth of scour in non-uniform sediments is often less than the depth of scour in uniform sediments. In uniform sediments, the energy is sufficient to transport the material (live bed) or it is insufficient to transport the material (clear water). In non-uniform sediments, the energy of the flow may only be sufficient to transport some material, allowing the coarser material to armor the bed. Armoring can occur in both the approach section (reducing the sediment transport to the scour hole) and in the scour hole (limiting the depth of the hole). The combination of armoring that occurs depends on the energy available for transport in the approach, the energy available for transport at the pier, and the gradation of the bed material. If the armoring occurs in the approach and no armoring occurs in the scour hole, the scour can be deeper than for identical conditions in uniform sediments, because the armoring of the approach has reduced the sediment supply to the scour hole. If armoring of the scour hole occurs, the depth of scour is likely to be less than that for uniform sediments regardless of the transport condition at the approach (live bed or clear water). The traditional classification of live bed and clear water conditions is insufficient to describe the conditions that may occur in non-uniform bed material.

The scientific literature has provided, over the past decades, a number of laboratory-derived equations for estimation of the depth of local scour at bridge piers. These empirical methods are mostly derived analyzing experimental data, which often are discordant, being the experiments related to several site dependent parameters, such as the geometric characteristics of the pier and the cross section and hydrological regime of the flow at the bridge site. Most of these formulations express the final scour depth as a function of the flow characteristics (mean flow velocity at the approach section, water depth), flow properties (mass density, and cinematic viscosity of the fluid), stream bed material properties (mean particle diameter, mass density), and bridge geometry (shape and dimension of the pier, angle of attack of the flow). The equations, published references, and the equation names for the 32 equations discussed in this report are presented in Table 2.

Table 2. Predictive formulas to estimate pier scour depth

Equation name and reference	Equation	Eq^n No.
Ahmad (Ahmad, 1953)	$y_s = KV_0^{2/3} y_0^{2/3} - y_0$	1
Arkansas (Southard, 1992)	$y_s = 0.827 D_{50}^{-0.117} V_0^{0.684} e^{0.476(c_1)}$	2
Blench-Inglis I	$y_s = 1.8 b^{0.25} y_0^{0.75} - y_0$	3

(Blench, 1962)		
Blench-Inglis II (Blench, 1962)	$y_s = 1.53b^{0.25}V_0^{0.5}y_0^{0.5}D_{50}^{-0.125} - y_0$	4
Breusers (Breusers, 1965)	$y_s = 1.4b$	5
Breusers-Hancu (Breusers *et al*., 1977)	$y_s = bf\left(K_1K_2 2\tan h\left(\frac{y_0}{b}\right)\right)$	6
Chitale (Chitale, 1962)	$y_s = y_0\left(-5.49F_0^2 + 6.65F_0 - 0.51\right)$	7
Froehlich (Froehlich, 1989)	$y_s = 0.32\phi g^{-0.1}V_0^{0.2}y_0^{0.36}b^{0.62}D_{50}^{-0.08}$	8
Froehlich Design (Froehlich, 1989)	$y_s = 0.32\phi g^{-0.1}V_0^{0.2}y_0^{0.36}b^{0.62}D_{50}^{-0.08} + b$	9
HEC 18 (Richardson *et al*., 1993)	$y_s = 2.0K_1K_2K_3g^{-0.215}y_0^{0.135}b^{0.65}V_0^{0.43}$	10
HEC 18-K4 (Richardson-Davis, 1995)	$y_s = 2.0K_1K_2K_3K_4g^{-0.215}y_0^{0.135}b^{0.65}V_0^{0.43}$ $K_4 = \left(1 - 0.89\left(1 - V_R\right)^2\right)^{0.5}$ $V_R = \frac{V_0 - V_c'}{V_{c90} - V_c'}$	11

HEC 18-K4Mo (Molinas, 2004)	$y_s = 2.0K_1K_2K_3K_4K_i g^{-0.215} y_0^{0.135} b^{0.65} V_0^{0.43}$ $K_4 = 1.25 + \sqrt[3]{\frac{D_{CFM}}{D_{50}}}\left(\frac{V_0 - V_i}{V_{CM} - V_i}\right)^{0.60} ln\left(\left(\frac{V_0 - V_i}{V_{CM} - V_i}\right) + 0.5\right)$ $K_i = \left(1 - \frac{V_i}{V_0}\right)^{0.45}$ $V_{cm} = 6.625 D_{CFM}^{1/3} y_0^{1/6}$ $V_i = 2.65 D_{35}^{1/3} y_0^{1/6}$ $D_{CFM} = \frac{D_{85} + 2D_{90} + 2D_{95} + D_{99}}{6}$	12
HEC 18-K4Mu (Mueller, 1996)	$y_s = 2.0K_1K_2K_3K_4 g^{-0.215} y_0^{0.135} b^{0.65} V_0^{0.43}$ $K_4 = 0.4\left(\frac{V_0 - V_c'}{V_c - V_{c95}'}\right)^{0.15}$	13
Briaud *et al.* (2011)	$y_s = 2.2K_1K_2 b^{0.65}\left(\frac{2.6V_o - V_c}{\sqrt{g}}\right)^{0.7}$	14
Inglis-Poona I (Inglis, 1949)	$y_s = 1.7b^{0.22} V_0^{0.52} y_0^{0.52} - y_0$	15
Inglis-Poona II (Inglis, 1949)	$y_s = 1.73b^{0.22} y_0^{0.78} - y_0$	16
Larras (Larras, 1963)	$y_s = 1.42K_{s2} b^{0.75}$	17
Laursen I (Laursen, 1962)	$y_s = 1.5b^{0.7} y_0^{0.3}$	18
Laursen II (Laursen, 1962)	$\frac{b}{y_0} = 5.5\left(\frac{y_s}{y_0}\right)\left(\left[\left(\frac{1}{11.5}\right)\left(\frac{y_s}{y_0}\right) + 1\right]^{1.70} - 1\right)$	19
Laursen-Callander (Melville, 1975)	$y_s = 1.11 y_0^{0.5} b^{0.5}$	20

Melville and Sutherland (Melville and Sutherland, 1988)	$y_s = K_I K_d K_y K_{\alpha L} K_s b$	21
Melville and Coleman (Melville and Coleman, 2000)	$y_s = K_{yB} K_I K_d K_s K_\theta K_G K_t$	22
Kirby et al. (2015)	$y_s = S_f K_v K_{yf} K_{\alpha L} K_s b$	23
Mississipi (Wilson, 1995)	$y_s = 0.9 b_e^{0.6} y_0^{0.4}$	24
Molinas (Molinas, 2004)	$y_s = 0.99 K_1 K_2 K_3 K_4 \left(\frac{V_0 - V_i}{V_{cm} - V_i}\right)^{0.55} b^{0.66} y_0^{0.17}$ $K_4 = 1.25 + \sqrt[3]{\frac{D_{CFM}}{D_{50}}} \left(\frac{V_0 - V_i}{V_{cm} - V_i}\right)^{0.60} ln\left(\left(\frac{V_0 - V_i}{V_{cm} - V_i}\right) + 0.5\right)$	25
Shen ** (Shen *et al.*, 1969)	$y_s = 0.00073 \nu^{-0.619} V_0^{0.619} b^{0.619}$	26
Shen-Maza (Shen *et al.*, 1969)	$y_s = 11.0 g^{-1} V_0^2$ for $F_p \leq 0.2$ $y_s = 3.4 g^{-0.33} b^{0.67} V_0^{0.67}$ for $F_p > 0.2$	27
Sheppard-Melville method (Sheppard *et al.*, 2014)	$\frac{y_s}{a^*} = 2.5 f_1 f_2 f_3$ for $0.4 \leq \frac{V_0}{V_c} \leq 1.0$ $\frac{y_s}{a^*} = f_1 \left[2.2 \left(\frac{\frac{V_o}{V_c} - 1}{\frac{V_{LP}}{V_c} - 1} \right) + 2.5 f_3 \left(\frac{\frac{V_{LP}}{V_c} - \frac{V_o}{V_c}}{\frac{V_{LP}}{V_c} - 1} \right) \right]$, for $1 < \frac{V_0}{V_c} \leq \frac{V_{LP}}{V_c}$	28

	$$\frac{y_s}{a^*} = 2.2 f_1 \quad \text{for } \frac{V_0}{V_c} > \frac{V_{LP}}{V_c}$$ $$f_1 = \tanh\left[\left(\frac{y_o}{a^*}\right)^{0.4}\right], \quad f_2 = \left\{1 - 1.2\left[\ln\left(\frac{V_o}{V_c}\right)\right]^2\right\}$$ $$f_3 = \left[\frac{\left(\frac{a^*}{D_{50}}\right)}{0.4\left(\frac{a^*}{D_{50}}\right)^{1.2} + 10.6\left(\frac{a^*}{D_{50}}\right)^{-0.13}}\right]$$ $$V_{LP1} = 5V_c \ , \ V_{LP2} = 0.6\sqrt{g y_0}$$ $$V_{LP} = V_{LP1} \quad \text{for } V_{LP1} \geq V_{LP2}$$ $$V_{LP} = V_{LP2} \quad \text{for } V_{LP1} < V_{LP2}$$ $$a^* = K_s b_e$$	
Sheppard-Alkhalidi (Sheppard, 2001 as reported by Alkhalidi, 2001)	$$y_s = b1.5k\left[2.5\left(\frac{V_0}{V_c}\right) - 1\right] \quad \text{for } 0.4 \leq \frac{V_0}{V_c} \leq 1.0$$ $$y_s = bc_2\left(\frac{V_{LP} - V_0}{V_c}\right) + c_3 \quad \text{for } 1 < \frac{V_0}{V_c} \leq \frac{V_{LP}}{V_c}$$ $$y_s = b2.4\tan h\left(\frac{y_0}{b}\right) \quad \text{for } \frac{V_0}{V_c} > \frac{V_{LP}}{V_c}$$	29
c_2	Is a coefficient defined by Sheppard as $$\frac{k - 2.4\tan h\left(1.775\left(\frac{y_0}{b}\right)^{0.618}\right)}{\left(\frac{V_{LP}}{V_c} - 1\right)}$$ Where: tanh is the hyperbolic tangent function	30
c_3	Is a coefficient defined by Sheppard as $$2.4\tanh\left(1.775\left(\frac{y_0}{b}\right)^{0.618}\right)$$	31
k	Is a coefficient defined by Sheppard as	32

	$$\frac{1.1\tanh\left(1.775\left(\frac{y_0}{b}\right)^{0.618}\right)}{\left(-0.025+0.233\exp\left(0.334\log_{10}\left(\frac{b}{D_{50}}\right)\right)+\frac{3.098}{\exp\left(2.411\log_{10}\left(\frac{b}{D_{50}}\right)\right)}\right)}$$	

**Units are English units in feet

where,

y_s	=	Depth of scour (m)
V_0	=	Approach velocity (m/s)
y_0	=	Approach depth (m)
b	=	Pier width (m)
b_e	=	Effective pier width defined as b cosα + L sinα
$D_{16}, D_{50}, D_{84}, D_{95}$	=	Sediment size (mm)
D_{CFM}	=	an average of the coarse grain sizes used by Molinas (2004)
σ_g	=	Gradation coefficient
S	=	Slope (m/m)
F_0	=	Flow Froude Number defined as $V_0/(gy_0)^{0.5}$.
F_p	=	Froude number defined as $V_0/(gb)^{0.5}$.
ϕ	=	Pier shape factor in Froehlich's equation.
g	=	Acceleration due to gravity.
V_R	=	Velocity intensity term used by Richardson and Davis (2001).
V_c	=	Critical (incipient-transport) velocity for the D_{50} size particle.
V_{c90}	=	Critical (incipient-transport) velocity for the D_{90} size particle.
V_c'	=	Approach velocity corresponding to critical velocity and incipient scour of the D_{50} in the accelerated flow region at the pier.
V_{c95}'	=	Approach velocity corresponding to critical velocity and incipient scour of the D_{95} in the accelerated flow region at the pier.
V_i	=	Approach velocity corresponding to critical velocity and incipient scour in the accelerated flow region at the pier defined by Molinas (2004).

V_{CM} = Critical (incipient-transport) velocity for the coarse size fraction defined by Molinas (2004).

V_{LP} = Live bed peak velocity defined by Sheppard (2001)

ν = Kinematic viscosity in Shen's equation (ft^2/sec).

K = Multiplying factor that varies from 1.3 to 2.

K_1 = a coefficient based on the shape of the pier nose, defined as 1.1 for square-nose piers, 1.0 for circular- or round-nosed piers, 0.9 of sharp-nosed piers, and 1.0 for a group of cylinders.

K_2 = a coefficient to correct for the skew of the pier to the approach flow, defined as (cos α + (L/b) sin α) 0.65.

K_3 = a coefficient to correct for the channel bed condition, defined as 1.1 except when medium to large dunes are present, and then it can range from 1.2 to 1.3.

K_4 = a coefficient to correct for bed material size and gradation.

K_4 (Eq. 12) = Coefficient derived by Molinas (2004).

K_4 (Eq. 13) = Coefficient derived by Mueller (1996).

K_i = Coefficient to correct the HEC 18 equation for sediment size by Molinas (2004).

K_d = Coefficient to correct for sediment size by Melville and Sutherland (1988).

K_I = Coefficient to correct for flow intensity defined by Melville and Sutherland (1988).

K_s = Coefficient to correct for pier shape defined by Melville and Sutherland (1988).

K_y = Coefficient to correct for flow depth defined by Melville and Sutherland (1988).

$K_{\alpha L}$ = Coefficient to correct for flow alignment defined by Melville and Sutherland (1988).

K_{yb} = Coefficient to depth-size ratio for piers defined by Melville and Coleman (2000).

K_θ = Coefficient to pier alignment defined by Melville and Coleman (2000).

K_G = Coefficient to channel geometry defined by Melville and Coleman (2000).

K_t = Coefficient to time defined by Melville and Coleman (2000).

K_{yf} = Depth factor mentioned by Kirby *et al*. (2015).

K_v = Velocity factor mentioned by Kirby *et al*. (2015).

S_f = Safety factor mentioned by Kirby *et al.* (2015).

c_1 = the pier location code in the Arkansas pier scour equation, c_1 = 0 for main channel piers and c_1 = 1 for piers on the banks of the main channel or on the floodplain.

3.2.1 Discussion of Equations

Laboratory experiments are designed to isolate specific scour processes; thus, the resulting equations may not account for complex and dynamic field conditions. Some field conditions that effect scour are undefined in the selected equations and assumptions are required to apply the equations. The flow in the field is assumed to be steady state and uniform to allow the application of laboratory-based equations to predict scour at bridges. All equations presented here are used to estimate scour for both live bed and clear water conditions. Many equations do not include corrections for pier shape, or they include corrections for only a few pier shapes. Pile groups are classified as round-nose or circular piers for equations that do not specify a shape correction for pile groups (Richardson et. al., 1993). In equations such as equations **1 (Ahmad)**, **2 (Arkansas)**, **3 (Blench-Inglis I)**, **4 (Blench-Inglis II)**, **5 (Breusers)**, **7 (Chitale)**, **10 (HEC 18)**, **15 (Inglis-Poona I)**, **16 (Inglis-Poona II)**, **24 (Mississipi)** and **26 (Shen)** (Ahmad, 1953; Southard, 1992; Blench, 1962; Breusers, 1965; Chitale, 1962; Richardson *et al.*, 1993; Inglis, 1949; Wilson, 1995 and Shen *et al*., 1969) the procedures to correct for pier shape, and corrections were not applied in the evaluation. In equation **17** (**Larras**) (Larras, 1963) specifies only square-nose and circular pier shapes, sharp-nose piers and pile groups are classified as circular piers in this evaluation.

However, due to the lack of field data and to the complexity of the scour phenomenon, these formulations have several limitations as:

a) they are derived for simple solid pier foundations with limited attention to scour depth developing in the case of pile groups and pile groups and pile caps (Salim and Jones, 1996)
b) the laboratory representations of the stream reach involving bridge scour is often realized by means of straight, typically rectangular laboratory flumes
c) these formulations assume steady flow conditions
d) the bed material is assumed to be non-cohesive

The application of these, typically conservative formulas leads to an overestimation of the scour hole (Melville and Coleman, 2000; Richardson and Davis, 2001). This implies an overestimation of bridge foundations that results in an economic waste during bridge design. As shown in Table 2, some of these equations have the velocity as a variable, either as the mean flow velocity or through the Froude number; some of them, however, are independent from flow velocity. As shown in the study by Jones (1984), these commonly used equations may provide, for the same case study, really different scour estimates, due to the variability of parameters involved in these equations. This observation should suggest bridge engineers to carefully select the methods to employ when evaluating bridge scour vulnerability, according to the case study characteristics, and apply several different methods to be able to critically compare scour depth values obtained by these equations.

Scientists from Texas A&M University have introduced an equation to predict pier scour for cohesive sediments. Furthermore, they developed a method called SRICOS-EFA to estimate scour depth based on erodibility characteristics of different soils (Briaud *et al*., 2011).

3.2.2 Application of equations in the field for scour evaluation

It is clear from the above discussion that all the equations presented are laboratory based equations and all of them have some limitations. However, these equations can be checked with field data to find out the most appropriate equation for the particular condition. Hence, field data of several bridge piers of a region may be selected and these equations employed to find out which gives the best result. If required, a modified equation may be developed for the particular region or place, to provide guidelines to the engineer in a design situation. Similar works have been carried out in several other countries to formulate design criteria for the bridge pier construction.

Selection of field data

The following guide lines may be of use when interpreting or collecting field data on local scour:

- Laboratory research indicates that for a pier skewed to the flow, maximum scour can occur along the sides of the pier rather than at the nose (Laursen and Toch, 1956). Therefore data should be collected along the sides of the piers, where the pier is skewed to the flow and also where the flow is not aligned with the pier.
- If there are measurements along the upstream and downstream edges of the bridge, only the maximum depth of scour should be used.

- Debris accumulations on the piers have an unknown effect on local scour and often make measurements of the maximum scour impossible. Hence, measurement on bridge pier accumulation should be avoided.
- The time required for scour to reach its maximum depth in cohesive material is considerably longer than in non-cohesive material (Richardson and Davis, 1995); therefore, observations of scour in cohesive material may be avoided or carefully interpreted.

The hydraulic parameters measured should be the conditions that caused the measured depth of scour. It is difficult to exactly associate hydraulics with a depth of scour because of the temporal development of the scour hole. It was rationalized that if the scour hole can be reasonably associated with the reported hydraulic conditions, the velocity at the pier must be competent to erode the bed material. Gao *et al.* (1992) published the following equation to compute the critical approach velocity that results in transport of the bed material at the pier based on the critical velocity for incipient transport of the bed material:

$$V_c' = 0.645\left(\frac{D_{50}}{b}\right)^{0.053} V_c \qquad (33)$$

where,

V_c' is the approach velocity corresponding to critical velocity and incipient scour in the accelerated flow region at the pier;
D_{50} is the mean grain size of the bed material;
b is the pier width
V_c is the critical (incipient-transport) velocity for the D_{50} size particle.

Equation 33 was used with Neill's formulation of the critical velocity equation (Neill, 1973):

$$V_c = \theta^{1/2} K_u 31.08 y_0^{1/6} D_{50}^{1/3} \qquad (34)$$

where,

θ is the Shield's parameter;
K_u is 1.0 for SI units and 1.81 for customary English units;
y_0 is the depth of flow; and
D_{50} is the median grain size.

to compute the critical approach velocity (V_c') for transport of the D_{50} grain size at the pier. All measurements having an approach velocity (V_0) less than the critical approach velocity for transport at the pier (V_c') should be removed. The appropriate value for the critical Shield's parameter, θ, has been a topic of considerable research and discussion, with no conclusive answer. Miller *et al.* (1977) and Buffington and Montgomery (1977) compiled and analyzed all available data on incipient sediment transport. Both investigations found scatter in the data caused by inconsistencies in the definition of incipient motion, the experimental method, the experimental facility, and the type of bed material. According to these studies, the critical Shield's parameter may vary from 0.02 to 0.086 with a common average value for gravel of about 0.046 Buffington and Montgomery (1977). Miller *et al.* (1977) presented a method based on Inman (1949) that relates grain size to the critical shear velocity; this method is only valid for water at a temperature of 20 °C and for bed material with a specific gravity of 2.65. The method was presented graphically, but has been reduced to equations for the critical Shield's parameter by Mueller (1996):

$$\theta = 0.0019 D_{50}^{-0.384} \qquad \text{or } D_{50} < 0.0009 \text{ (m)}$$
$$\theta = 0.0942 D_{50}^{0.175} \qquad \text{for } 0.0009 \text{ (m)} < D_{50} < 0.020 \text{ (m)}$$
$$\theta = 0.047 \qquad \text{for } D_{50} > 0.020 \text{ (m)}$$

The method is easily applied, provides for variation in the critical Shield's parameter for smaller grain sizes, and is within the range of variation defined by previous research; therefore, this method may be used to evaluate the critical Shield's parameter needed to estimate the critical velocity for incipient sediment transport.

3.3 Conclusions

Scour at hydraulic structures is one of the main issues engineers have to face at various stages of design, operation, and maintenance. Scour at bridge piers is the main cause for bridge failure and might represent a potential threat to the civil population. The scientific community has made significant advances in understanding the scour process dynamics and has explored different approaches to estimate the maximum expected scour depth at bridge piers. These advances provide tools for supporting engineers in the design phase of adequate bridge foundations. Various researchers have proposed many pier scour equations, but few have accurately and conservatively predicted the scour observed in the field. Most equations are based on scaled laboratory experiments that did not account for the complexity of the field conditions.

However, the numerous empirically derived equations available to predict pier scour are easy to use and are still widely applied in the engineering practice, but they seem to over- or underestimate the scour depth when they are employed outside the range of applicability for which they are derived in the laboratory experiments. Preferably, equations should be compared with field data and conditions so that the most appropriate equation for a particular region and flow conditions can be selected in applications of the field engineers.

4. Abutment scour

4.1 Overview

Failure of bridges due to local scour has motivated many investigators to explore the causes of scouring and to predict the maximum scour depth at abutments. Failure of bridges due to scour at their foundations, which consist of abutments and piers, is a common occurrence. A study of the US Federal Highway Administration in 1973 concluded that of 383 bridge failures, 25% involved pier damage and 72% involved abutment damage (Richardson *et al.*, 1993). In a report submitted to the National Roads Board of New Zealand, Sutherland (1986) pointed out that of 108 bridge failures recorded, 29 were attributed to abutment scour during 1960–1984. In another report of the Department of Scientific and Industrial Research (DSIR) of New Zealand, Macky (1990) mentioned that about 50% of total expenditure was made towards bridge damage repairing and maintenance, out of which 70% was spent towards repairing abutment scour. Localized scour around abutments is a common occurrence and poses a challenging problem to the hydraulic engineers due to its detrimental effect to the foundations of abutments. Abutments are located at either end of the bridge, help to transmit the weight of the bridge including traffic to the foundation bed. On the other hand, piers are located within the bridge span. Based on the supply of sediment by the approaching flow, the localized scour can be classified in two ways: clear water scour and live bed scour (Dey, 1997).

The flow field around abutments embedded vertically in a bottom consisting of sediment is complex in detail, involving separation of flow that develops three-dimensional vortex patterns; and the complexity increases with the development of the scour hole. The flow field around piers has been well researched, e.g., Hjorth (1975), Melville (1975), Melville and Raudkivi (1977), Dey (1995), Dey *et al*. (1995) and Graf and Istiaro (2002). However, research on the flow field around abutments has been very limited. Kwan (1989) and Kwan and Melville (1994) observed the three-dimensional flow field in a scour hole around a wing-wall abutment. They concluded that a primary vortex, being similar to the horseshoe vortex around piers along with the downflow, is the primary cause of scouring at abutments. Rajaratnam and Nwachukwu (1983), and Ahmed and Rajaratnam (2000) investigated the flow fields at groins and abutments, respectively, placed on a planar or unscoured bed.

4.1.1 Physics of Vortex Flow

Using the preceding findings, the physics of the flow in the scour hole around an abutment can be explained. The main characteristic feature of the flow around an abutment is a relatively large primary vortex flow and skewed flow velocity distributions on the side of the abutment. A primary vortex, which is almost similar to a ground roller downstream of a dune crest, is formed inside the scour hole due to the flow separation at the upstream edge of the scour hole, known to be the line of separation; thus, the scour hole acts as a zone of separation (Barbhuiya, 2003). The downflow is developed due to the downward negative stagnation pressure gradient of the non-uniform approaching flow velocity (maximum at the free surface and zero at the bed) adjacent to the vertical face of the abutment upstream and is pushed up by the vortex. The process of flow separation can be explained using the concept of limiting streamline (Maskell, 1955). The two limiting streamlines along the original bed upstream (due to the approaching flow) and the sloping bed (due to the reversed flow) merge at the edge of the scour hole, forming a separated streamline.

Thus, a surface of separation is produced in the form of an envelope by the separated streamlines. In this process, the approaching flow curves down into the scour hole and rolls to form a primary vortex, which migrates downstream by the side of the abutment. The migration of the primary vortex by side of the abutment towards the downstream in the scour hole (which is a triangular section) may partially be compared with the passage of helicoidal flow in a right-angled triangular bend of a channel (Rozovksii, 1961). However, unlike the flow in the right-angled triangular channel bends, the vertical velocity component is substantially higher due to a strong three-dimensional vortex flow inside the scour hole associated with a downflow in front of the abutment. Downstream, the flow characteristic becomes reversed (towards the abutment). This is due to the flow separation at the abutment side, which produces a shedding of wakes downstream of the abutment.

4.1.2 Parameters related to scour at abutments

Parameters involved in the scour phenomenon at abutments can be grouped as follows:

(1) Geometry of the channel: width, cross-sectional shape, and slope.
(2) Geometry of the abutment: size, shape, orientation with respect to main flow and surface condition.
(3) Bed sediment properties: median size, grain size distribution, mass density, angle of repose and cohesiveness.
(4) Fluid (water) properties: density, viscosity, gravitational acceleration and temperature.
(5) Approaching flow condition: mean flow velocity, flow depth, shear velocity and roughness.

(6) Temporal evolution of the scour hole (can be taken as an additional parameter for an evolving scour hole).

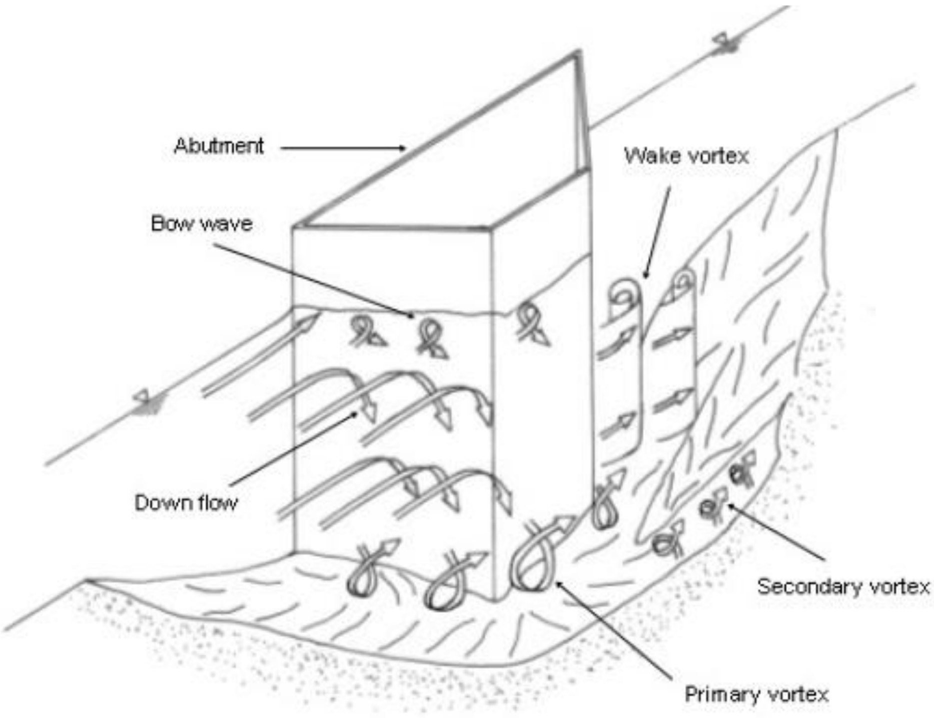


Figure 19. Schematic diagram of flow field at an abutment (after Kwan 1988)

4.1.3 Influence of parameters on scour depth

Approaching flow velocity

The effect of the approaching flow velocity V_0 is incorporated in the scour predicting formulas in the form of a flow Froude number F_0 or the shear velocity u_*. Garde *et al.* (1961), Zaghloul and McCorquodale (1975), Zaghloul (1983), Rajaratnam and Nwachkwu (1983), and Froehlich (1989) included the flow Froude number in their analyses. Garde *et al*. (1961) concluded that the flow Froude number for the normal channel flow adequately represents the effect of the approaching flow velocity on the maximum scour depth. Kandasamy (1989) showed that the scour depth increases with an increase in flow depth due to incorporation of the flow Froude number.

It is generally recognized that the shear velocity u_* is an important parameter not only in distinguishing clear-water scour from live-bed scour, but also in representing the erosive power of the flowing stream for a given sediment size.

Clear-water scour occurs for an approaching flow velocity up to the critical velocity V_c for bed sediments, that is $V_0/V_c = 1$; whereas live-bed scour occurs when $V_0/V_c > 1$. For non-uniform sediments, Melville and Sutherland (1988) defined an armor velocity V_a, which marks the transition from clear-water to livebed conditions for sediment-transporting flow and is equivalent to V_c for uniform sediments. Thus, for non-uniform sediments, live-bed conditions prevail when $V/V_c > 1$. However, if $V/V_c < 1$, armoring of the bed occurs as scouring proceeds and clear-water conditions exist.

It is recognized that under clear-water conditions, the maximum scour depth occurs when $V/V_c = 1$; this scour depth is called the threshold peak. For $V/V_c > 1$, which is under livebed conditions, scour depth initially decreases with an increase in approaching flow velocity, reaching a minimum value, and then increases again towards a second maximum. The second maximum occurs at about the transitional flatbed stage of sediment transport on the channel bed and is termed live-bed peak.

Approaching flow depth

According to Laursen (1952), the approaching flow depth y_0 is an important factor to determine scour depth. Experimental results of Gill (1972), Wong (1982), Tey (1984), and Kandasamy (1989) indicate that for a constant value on the shear velocity ratio $u_* = u_{*c}$ ($u_{*c} =$ critical shear velocity for sediment particles), the maximum scour depth increases with an increase in the approaching flow depth. It is also observed that the maximum scour depth increases at a decreasing rate with an increase in approaching flow depth. According to Kandasamy (1989), for shallow flow depths, the scour depth increases proportionally with y_0, but is independent of l (transverse length or protrusion length of abutment). On the other hand, for intermediate flow depths, the scour depth depends on both y_0 and l. Melville (1992) distinguished between short and long abutments, and concluded that for short abutments ($l/y_0 \geq 1$), the scour depth is independent of flow depth, and for long abutments ($l/y_0 \geq 25$), the scour depth is dependent on flow depth. However, most abutments are neither long nor short; as a result the scour depth is influenced by both y_0 and l. Dey and Barbhuiya (2004a) reported that for smaller flow depths, the equilibrium scour depth increases significantly with an increase in y_0, whereas for higher flow depths, the equilibrium scour depth is independent of the flow depth. There is consensus that the maximum scour depth increases at a decreasing rate with an increase in the approaching flow depth and that there exists a limiting depth corresponding to which the maximum scour depth is independent of the flow depth.

Abutment length, contraction ratio and opening ratio

Abutment length and contraction ratio have extensively been used in formulating the maximum scour depth at abutments. The inverse of the opening ratio is termed contraction ratio.

Kandasamy (1989) pointed out that if the length of the abutment is increased, the opening ratio decreases, and the effect on scour depth of such a change can be ascribed to both a decrease in contraction ratio and an increase in abutment length. Garde *et al.* (1961), Gill (1972), Zaghloul and McCorquodale (1975), and Rajaratnam and Nwachukwu (1983) used contraction ratio in their analyses. Neill (1973) argued that the use of contraction ratio as a scaling parameter cannot be justified in the case of a short abutment projecting into a very wide channel, and that contraction might be regarded as a secondary influence. Extending the same argument, Neill (1973) concluded that it is logically fallacious to express the results primarily in terms of contraction ratio. Cunha (1975) found that for flow without continuous sediment motion, scour depth does not depend on the contraction ratio and is only affected by local phenomena. A similar conclusion was also drawn by Liu *et al.* (1961) for abutments and Laursen (1963) for bridge piers and abutments.

Laursen (1963), Neill (1973), Cunha (1975), Wong (1982), Tey (1984), Kandasamy (1989), Melville (1992), and Cardoso and Bettess (1999) advocated that scour is a local phenomenon and is independent of the contraction ratio. Accordingly, abutment length has been proposed as a scaling parameter. The convincing argument is that as long as the scour hole does not extend to the opposite bank of the stream or the flume-wall, with other conditions being the same, maximum scour depth at an abutment of a fixed length is the same irrespective of the flume width or stream. A conciliatory approach would be to use contraction ratio and abutment length when the extent of the scour hole is affected and unaffected by the opposite bank or the flume-wall respectively.

Size and gradation of sediments

Characteristics of the bed sediments are derived from particle size distribution curves. The two most commonly used parameters are median sediment diameter d_{50} and geometric standard deviation $\sigma_g \left[= \left(d_{84}/d_{16} \right)^{0.5} \right]$ of particle size distribution, which is a measure of the uniformity of the bed sediments. Laursen and Toch (1956) and Ahmad (1953) stated that the maximum scour depth is independent of the sediment size. Blench (1957), Garde *et al.* (1961), and Gill (1972) reported that sediment size has an influence on the maximum scour depth. Laursen (1960) found that maximum scour depth is affected by sediment size under clear-water scour, but not under live-bed scour. Results of Gill (1972) for two sediment sizes (d_{50} = 1.52 mm and 0.914 mm) indicate that for the same value of $\tau_0/\tau_c < 1$ ($\tau_0 =$ bed shear stress of approaching flow), scour depth is greater with coarse sediments than with fine sediments, where τ_c = critical shear stress for sediment particles. However, for the same value of absolute bed shear stress in the approaching flow, fine sediments produce greater scour depth. For wing-wall, spill-through, and semicircular abutments, Wong (1982) found that scour depth increases with increase in bed sediment size for a constant value of τ_0/τ_c , which is close to unity.

For live-bed scour in uniform sediments, the amount of sediment being transported by the approaching flow as bed load into the scour hole and that being picked-up from the scour hole at an abutment are the same at equilibrium conditions. Since sediment size does not have any effect on the existing balance of sediment continuity, the equilibrium scour depth is unaffected by change in sediment size.

Ahmad (1953), Garde *et al.* (1961) and Gill (1972) found that the rate of scour is different for different bed sediments. According to them, fine sediments are scoured at a faster rate than course sediments. Ramu (1964) observed that for the same sediment size, a change in sediment gradation σ_g affects the equilibrium scour depth. Ahmad (1953) asserted that equilibrium scour depth depends on the sediment gradation σ_g . Ettema (1980) and Raudkivi and Ettema (1983) found that the maximum clear-water equilibrium scour depth y_s at a bridge pier depends on the sediment grading σ_g .

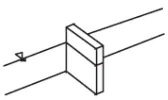
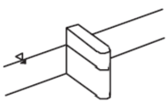
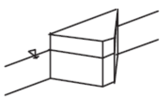
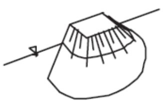
According to Dey and Barbhuiya (2004b), the effect of sediment gradation on scour depth is pronounced for non-uniform sediments, which reduce scour depth significantly due to the formation of armour layers in scour holes. Dey and Barbhuiya (2004b) conducted experiments to study the effects of thin armour layers on scour depth at abutments. They concluded that the scour depth at an abutment with an armour layer in clear-water scour condition under limiting stability of surface particles (approaching flow velocity nearly equaling critical velocity for threshold motion of surface particles) is always greater than that without an armour layer for the same bed sediments.

Abutment shape

The shape of the abutment plays an important role for the equilibrium scour depth. Streamlined bodies, such as semi-circular (SC), spill-through (ST), and wing-wall (WW) abutments, produce vortices of feeble strength, whereas blunt obstructions, for example vertical-wall abutments, are capable of producing strong turbulent vortices. Thus, a relatively large scour depth may be observed at a blunt obstruction. From laboratory experimental data, Laursen and Toch (1956), Liu *et al.* (1961), Garde *et al.* (1961), and Wong (1982) concluded that vertical wall abutments produce greater scour depth in comparison with spill-through and wing-wall abutments.

Melville (1992, 1995, and 1997) used a shape factor K_s to account for the effect of the shape of abutments on equilibrium scour depth. Commonly used abutment shapes and corresponding values of the shape factor are furnished in Table 3. The vertical plate is the simplest shape of an abutment and is, therefore, used as reference. For spill-through abutments, the abutment length is taken as the length at mid-depth in the flow. The shape factors given in Table 3 are derived from laboratory experimental data of Gill (1972), Wong (1982), Tey (1984), Kwan (1984, 1988), Kandasamy (1989), and Dongol (1994). However, Melville (1992) asserted that the importance of the abutment shape diminishes when the abutment becomes longer. Thus, for $l/y_0 \geq 10$, an adjusted shape factor K_s^* (= adjusted abutment shape factor) is recommended. The adjusted shape factor K_s^* varies linearly between the value of K_s at $l/y_0 = 10$ and unity at $l/y_0 = 25$.

Table 3. Abutment shape factors (Barbhuiya and Dey 2004)

Abutment model	Abutment shape	Shape factor, K_s
	Vertical-wall	1·00
	Semicircular ended	0·75
	45° wing-wall	0·75
	Spill-through with slope horizontal : vertical 0·5 : 1 1 : 1 1·5 : 1	 0·60 0·50 0·45

Abutment alignment
The angle of approaching flow with respect to the abutment alignment, termed angle of attack, significantly influences scour depth. It was experimentally studied by Laursen and Toch (1956), Garde *et al.* (1961), Zaghloul (1983), and Kwan (1984). Garde *et al.* (1961) reported that for the same flow, sediment, and abutment conditions, the maximum scour depth is greatest for a spur dike with an inclination of 90^0. For all other inclinations the scour depth is smaller; similar observations were made by Kwan (1984). Zaghloul (1983) reported that the greatest equilibrium scour depth is seen for an upstream spur dike inclination, and the smallest when the spur dike is inclined downstream. The magnitude of the equilibrium scour depth at a spur dike placed normal to the flow is in between the magnitude of scour depths at spur dikes inclined upstream and downstream. Melville (1992) included the effect of abutment alignment incorporating the alignment factor K_θ in the design equations. The values on K_θ given in Table 4 were derived by the envelope curve method from the data of Ahmad (1953), Laursen (1958), Sastry (1962), Zaghloul (1983), Kwan (1984), and Kandasamy (1985). Melville (1992) recommended that the alignment factor should be applied only to longer abutments ($l/y_0 \geq 3$). Alignment effects are negligible for short abutments ($l/y_0 \leq 1$) having $K_\theta = 1$. For abutment lengths between these two limits, a value of K_θ obtained from the linear interpolation has been recommended.

Table 4. Flow alignment factor for different angles of attack (Barbhuiya and Dey 2004)

θ_a (deg)	30	60	90	120	150
K_θ	0·90	0·97	1·00	1·06	1·08

Channel geometry
Cross sections of rivers may have different shapes depending on the geographical location, the characteristics of sediments through which they pass, and the characteristics of its catchment area. Normally, in hilly regions, the cross-section of a river is parabolic with steep side slope, whereas in the plains, its cross section is compound, with floodplains and a main channel. Thus, for bridges in the hilly region, abutments are founded in the main channel. On the other hand, for bridges in the plains, all abutments including the approach embankment may terminate in the floodplain or may extend into the main channel. Richardson and Richardson (1998) argued that experimental results in rectangular flumes do not accurately reflect the abutment scour process in compound channels.

Froehlich (1989) considered the channel geometry effect in calculating the approaching flow Froude number. He calculated the number based on the average velocity and the depth in the area obstructed by the embankment and the abutment at the approaching flow cross-section. A systematic investigation of the effect of channel geometry on scour depth at an abutment located in a compound channel, comprising floodplains and main channel, was done by Melville and Ettema (1993) and Melville (1995). These studies are limited to the case of an abutment spanning the floodplains and extending into the main channel. The effect of the channel geometry on scour depth is represented by a multiplying factor K_G, which is defined as the ratio of the scour depth at an abutment sited in a compound channel to a scour depth at an abutment sited in the corresponding rectangular channel of the same overall width as that of the compound channel and the same depth as that of the main channel of the compound section. In general, K_G depends on the size, shape, roughness of the main channel and floodplains, and the abutment length with respect to the floodplain width.

However, they found that the time to reach equilibrium scour, when the scour hole extends into the main channel, is shorter than that to reach equilibrium scour, when the scour hole is confined to the floodplain.

4.2 Predictive formulas

Equations for estimating scour depth at abutments are classified into the three following categories:

(1) Regime approach relating the scour depth to the increased discharge intensity
(2) Empirical approach using dimensional analysis of the main parameters causing scour
(3) Analytical or semi-empirical approach

4.2.1 Regime approach

In the regime approach, scour depth is related to the discharge at the section under consideration. Using Lacey's regime formula, the CBI (1949) proposed the following relationship to estimate the maximum scour depth at obstructions like spur dikes or abutments (in FPS units),

$$y_s + y_0 = 0.47k_1\left(Q/f_1\right)^{1/3} \qquad (35)$$

where y_s is in mm) and k_1 is a coefficient that depends on the type of obstruction (= 1 to 3.5). Based on laboratory experimental model studies, Ahmad (1953) suggested a relationship to estimate the maximum scour depth at spur-dikes as,

$$y_s + y_0 = k_2 q^{2/3} \quad (36)$$

where k_2 is a constant depending on the flow intensity and the angle of inclination of the spur dike.

The above equations have an inherent drawback for estimating the scour depth. The regime concept, originating from the analysis of general scour in live-bed conditions, was extended to local scour at spur dikes and abutments on the basis of observations. However, although the local scour caused by the change in flow pattern at the obstruction is fundamentally different from that of general scour, the equations indicate that the scour depth is a function of flow intensity only. Thus, the sediment characteristics, rate of scour, and mode of sediment transport are not considered.

4.2.2 Empirical approach

In the empirical approach, parameters involved in the abutment scour are correlated through dimensional analyses. Using regression analyses of the experimental data, equations for the scour depth at abutments have been developed:

$$y_s/y_0 = 1.1\left(\frac{l}{y_0}\right)^{0.4} F_0^{0.33} + 0.3, \text{ for spill-through abutments} \quad (37)$$

$$y_s/y_0 = 2.15\left(\frac{l}{y_0}\right)^{0.4} F_0^{0.33} + 0.3, \text{ for vertical-wall abutments} \quad (38)$$

Garde *et al.* (1961) developed a relationship for the scour depth at spur dikes under live-bed scour:

$$\left(y_s + y_0\right)/y_0 = \left(K_5/\alpha\right)F_0^n \quad (39)$$

In continuation of their work, Garde *et al.* (1963) modified the above equation due to inherent difficulties in determining the drag coefficient C_D and relating K_5 to C_D. The modified equation is given by:

$$\left(y_s + y_0\right)/y_0 = 4\eta_1\eta_2\eta_3\left(F_0^n/\alpha\right) \quad (40)$$

Starting from the equation for a long contraction and using experimental data, Gill (1972) derived the following generalized equation for scour depth at spur dikes:

$$\frac{y_s + y_0}{y_0} = 8.375\left(\frac{d_{50}}{y_0}\right)^{0.25}\left(\frac{B}{B-l}\right)^{6/7}\left[\left(\frac{B}{B-l}\right)^{1/n}\left(1-\frac{\tau_c}{\tau_0}\right)+\frac{\tau_c}{\tau_0}\right]^{-3/7} \tag{41}$$

In the above equation, τ_o is assumed to be equal to τ_c in live-bed scour. Gill (1972) further extended the above equation for three different flow conditions as follows:

$$\frac{y_s + y_0}{y_0} = 8.375\left(\frac{d_{50}}{y_0}\right)^{0.25}\left(\frac{B}{B-l}\right)^{6/7}\left(\frac{\tau_c}{\tau_0}\right)^{3/7}, \tag{42}$$

for clear-water scour condition $\left(\tau_c/\tau_0 > 0\right)$

$$\frac{y_s + y_0}{y_0} = 8.375\left(\frac{d_{50}}{y_0}\right)^{0.25}\left(\frac{B}{B-l}\right)^{(6/7)-(3/7n)}, \tag{43}$$

for high sediment transport condition $\left(\tau_c/\tau_0 > 0\right)$

$$\frac{y_s + y_0}{y_0} = 8.375\left(\frac{d_{50}}{y_0}\right)^{0.25}\left(\frac{B}{B-l}\right)^{(6/7)}, \tag{44}$$

for maximum scour depth d_s condition $\left(\tau_c/\tau_0 \approx 0\right)$,

In the above equation, n varies from 1.5 to 3.

Zaghloul and McCorquodale (1975) presented the following equation that includes the effect of angle of inclination of a spur dike with respect to the main flow:

$$y_s/y_0 = 2.62F_0^{2/3}\left(\alpha\theta_a\right)^{-0.043} \tag{45}$$

Froehlich (1989) analysed the scour data of different researchers using statistical method and developed the following equations for clear-water and live-bed scour depths:

$$\frac{y_s}{y_0} = 0.78K_s K_\theta\left(\frac{l}{y_0}\right)^{0.63} F_0^{1.16}\left(\frac{y_0}{d_{50}}\right)^{0.43}\sigma_g^{-1.87}+1, \tag{46}$$

for clear water scour

$$\frac{y_s}{y_0} = 2.27 K_s K_\theta \left(\frac{l}{y_0}\right)^{0.43} F_0^{0.61} + 1, \qquad \text{(47) for live bed scour}$$

The coefficient of the abutment alignment K_θ is $(\theta_a/90)^{0.13}$. The addition of 1 at the right hand side of the equations results in large overestimation of the scour depths, especially for large approaching flow depths.

Strum and Janjua (1994) conducted experiments in a flume with a fixed-bed main channel and a movable-bed floodplain, where the abutment terminated. Using dimensional analysis and least-square regression analysis, they derived the following equation of clear-water scour depth at abutments in floodplains:

$$y_s/y_0 = 7.7\left(F_0/MF_{0c} - 0.35\right) \qquad (48)$$

Melville (1992, 1995, and 1997) proposed a design method to estimate the scour depth at abutments based on empirical relationships containing different factors or coefficients. Each factor or coefficient represents the effect of flow depth, abutment size, flow intensity, sediment characteristics, abutment shape, abutment alignment, and channel geometry on scour depth. The proposed equation is:

$$y_s = K_{hl} K_I K_d K_s K_\theta K_G \qquad (49)$$

He argued that for short abutments $(l/y_0 \le 1)$, the scour depth scales with the abutment length, whereas for long abutments $(l/y_0 \ge 25)$ the scour depth scales with the flow depth. For all other abutments $(1 < l/y_0 < 25)$, the scour depth is proportional to $(y_0 l)^{0.5}$. Thus, according to Melville (1992), the coefficient accounting for the flow depth and abutment size is given by:

$$K_{hl} = 2l, \qquad \text{for } l/y_0 \le 1 \qquad (50)$$

$$K_{hl} = 2\left(y_0 l\right)^{0.5}, \qquad \text{for } 1 < l/y_0 < 25 \qquad (51)$$

$$K_{hl} = 10 y_0, \qquad \text{for } l/y_0 \ge 25 \qquad (52)$$

Melville and Sutherland (1988) presented a method for accounting sediment gradation effects including armor velocity V_a. The value of V_a is calculated to be $0.8V_{cn}$. The flow intensity factor, K_I for uniform and non-uniform sediments is given by:

$$K_I = \left[V_0 - \left(V_a - V_c\right)\right] / V_c\,, \quad \text{for } \left[V_0 - \left(V_a - V_c\right)\right] / V_c < 1 \quad (53)$$

$$K_I = 1, \qquad \text{for} \left[V_0 - \left(V_a - V_c\right)\right] / V_c \geq 1 \quad (54)$$

The critical velocities V_c and V_{cn} can be determined from the logarithmic velocity distribution as:

$$V_c / u_{*c} = 5.75 \log\left(5.53\, y_0 / d_{50}\right) \qquad (55)$$

$$V_{cn} / u_{*cn} = 5.75 \log\left(5.53\, y_0 / d_{50a}\right) \qquad (56)$$

The sediment size effect depend on the value of l/d_{50} , given by:

$$K_d = 0.57 \log\left(2.24\, l/d_{50}\right), \quad \text{for } l/d_{50} \leq 25 \qquad (57)$$

$$K_d = 1, \qquad \text{for } l/d_{50} > 25 \qquad (58)$$

The abutment shape factor is assumed to be 1 for vertical-wall abutments and 0.75 for wing-wall abutments. Spill-through abutments are assigned values of $K_s = 0.6,\ 0.5$, and 0.45 for 0.5 : 1 (horizontal : vertical), 1 : 1 and 1.5 : 1 side slopes, respectively. These values on the shape factor apply only to shorter abutments $\left(l/y_0 = 10\right)$. Shape effects were found to be unimportant for long abutments, and hence, $K_s = 1$ for $l/y_0 \geq 25$. For abutment lengths $10 < l/y_0 < 25$, a linear interpolation has been proposed. Thus, the adjusted shape factor K_s^* for intermediate abutments is:

$$K_s^* = K_s + 0.667\left(1 - K_s\right)\left[0.1\left(l/h\right) - 1\right] \qquad (59)$$

In the above, shape factor K_s is referred to the case $l/y_0 \leq 10$.

The value of the abutment alignment factor is $K_\theta = 1$ for an abutment aligned across the flow, that is, an angle of alignment $\theta_a = 90^\circ$. For $\theta_a < 90^\circ$, the abutment is pointed downstream and vice versa. For alignment angles $\theta_a = 30^\circ, 60^\circ, 120^\circ$ and 150°, the values of $K_\theta = 0.9, 0.97, 1.06$ and 1.08, respectively. Melville (1992) recommended that the alignment factor can be applied only to longer abutments $\left(l/y_0 \geq 3\right)$. Alignment effects are negligible for short abutments, and hence, $K_\theta = 1$ for $l/y_0 \leq 1$. For abutment lengths $1 < l/y_0 < 3$, a linear interpolation has been recommended. Thus, adjusted alignment factor K_θ^* for intermediate abutments is:

$$K_\theta^* = K_\theta + \left(1 - K_\theta\right)\left[1.5 - 0.5\left(l/y_0\right)\right] \qquad (60)$$

Channel geometry factor K_G is defined as the ratio of the scour depth at a given abutment sited in the compound channel to that at the same abutment sited in a corresponding rectangular channel of the same overall width as that of the compound channel and the same depth as that of the main channel of the compound section. Melville and Ettema (1993) put forward the following equation for K_G:

$$K_G = \left\{1 - \left(l^*/l\right)\left[1 - N/N^*\left(y_0^*/y_0\right)^{5/3}\right]\right\}^{1/2} \qquad (61)$$

Kothyari and Ranga Raju (2001) defined an analogous pier, with a size such that scour depth at the pier is same as that at the given spur dike or abutment under similar hydraulic conditions. The size of the analogous pier is related to the parameters that influence the drag due to the flow past the spur dike or abutment and pier, and given by Kothyari an Ranga Raju (2001) is:

$$b_s/K_s b_d = 0.074\left(l/b_d\right)^{2.7}\left(V_0/\sqrt{g d_{50} \Delta\rho_s/\rho}\right) + 0.46 \qquad (62)$$

Scour depth at abutments can then be calculated using the relationships for estimation of pier scour given by Kothyari *et al.* (1992a, b) with the size of the analogous pier taken as the pier width.

Based on the field data of scour at the end of spurs in the Mississippi River, Richardson *et al.* (2001) proposed the following equation (recognized as HIRE equation) to estimate scour depth at an abutment for live-bed scour:

$$y_s/y_0 = 7.27 K_s K_\theta F_0^{0.33} \qquad (63)$$

Dey and Barbhuiya (2004b) put forward an equation for clear-water scour depth at short abutments as follows:

$$y_s/l = 5.16K_s\left(y_0/l\right)^{0.18}\left(V_c\Big/\sqrt{\Delta gl}\right)^{0.26} \qquad (64)$$

4.2.3 Analytical or semi-empirical approach

Laursen (1960, 1963) developed semi-empirical scour depth relationships for bridge abutments treating the abutment scour as a limiting case of scour through a long flow constriction. His proposed relationships for clear-water and live-bed scour depths at vertical-wall abutments is:

$$l/y_0 = 2.75\left(y_s/y_0\right)\left[\left(\left(y_s/\hat{d}_l y_0\right)+1\right)^{1.7}-1\right], \text{ for live-bed scour} \quad (65)$$

$$l/y_0 = 2.75\left(y_s/y_0\right)\left\{\left[\left[y_s\Big/\left(\hat{d}_l y_0\right)+1\right]^{7/6}\Big/\left(\tau_0/\tau_c\right)^{0.5}\right]-1\right\}, \text{ for clear-water scour} \quad (66)$$

Laursen (1960, 1963) assumed that $\hat{d}_l = 12$ and 11.5 for clear-water and live-bed scour conditions, respectively.

Based on the flow continuity equation, scour geometry, and a generalized power-law formula for flow resistance in alluvial channels, Lim (1997) presented a semi-empirical formula for the equilibrium clear-water scour depth at an abutment as:

$$y_s/y_0 = K_s\left(0.9X-2\right) \qquad (67)$$

where, $X = \theta_c^{-0.375}F_d^{0.75}\left(d_{50}/y_o\right)^{0.25}\left[0.9\left(l/y_0\right)^{0.5}+1\right]$ and $K_s = 1$ for vertical-wall abutment; for other shapes, the values of K_s are as given by Melville (1992). The above equation is valid up to $X = 2.22$.

Following the approach of Lim (1997), Lim and Cheng (1998) introduced a semi-empirical equation for the time-averaged equilibrium live-bed scour at vertical-wall abutments as:

$$\left(1+\frac{y_s}{2y_0}\right)^{4/3}=\left[1+1.2\sqrt{1/y_0}\right]\Big/\left[\frac{u_{*c}^2}{u_*^2}+\left(\frac{l\tan\phi_s}{y_s}\right)^{2/3}\left(1-\frac{u_{*c}^2}{u_*^2}\right)\right]^{1/2} \qquad (68)$$

Kandasamy and Melville (1998) developed a relationship for maximum scour depth at piers and abutments aligned perpendicular to the flow,

$$y_s = K_s K_6 y_0^n l^{1-n} \qquad (69)$$

In the above, $K_6 = 5$ and $n = 1$ for $y_0/l \le 0.04$, $K_6 = 1$ and $n = 0.5$ for $0.04 < y_0/l < 1$, and $K_6 = 1$ and $n = 0$ for $h/l > 1$

Other notations used are:

B	=	flume width
b_d	=	width of cylindrical pier experiencing the same drag as that on abutment
b_s	=	width of analogous pier;
d_{50a}	=	$d_{\max} = 1.8$
$\hat{d}_l$	=	ratio of scour depth at abutment to scour depth in equivalent long contraction
f_1	=	Lacey's slit factor, $1.76d^{0.5}$
F_{0c}	=	$V_c/(gy_0)^{0.5}$, approaching flow Froude number corresponding to critical velocity
$k_{1,2}, K_{5,6}$	=	coefficients
K_d	=	particle size factor
K_G	=	channel geometry factor
K_{hl}	=	flow depth – abutment length factor
K_I	=	flow intensity factor
K_s	=	abutment shape factor
K_s^*	=	adjusted abutment shape factor
K_θ	=	abutment alignment factor
K_θ^*	=	adjusted abutment alignment factor
l	=	transverse length or protrusion length of abutment

M	=	discharge contraction ratio, defined as the ratio of the discharge at approaching section through the opening width to the total discharge
n	=	exponents depending on bed sediment size
q	=	discharge intensity
Q	=	discharge
s	=	relative density of sediment particles (ρ_s/ρ)
u_*	=	shear velocity
u_{*c}	=	critical shear velocity for sediment particles
V_a	=	armor velocity
V_{cn}	=	critical velocity for armour particle size d_{50a}
α	=	$1-l/B$, opening ratio
$\eta_1\eta_2\eta_3$	=	coefficients
θ_a	=	angle of attack
τ_0	=	bed shear stress of approaching flow
τ_c	=	critical shear stress of sediment particles
Δ	=	$s-1$
ρ	=	mass density of water
ρ_s	=	mass density of sand
ϕ	=	side slope angle of scour hole

4.3 Conclusions

Although there have been marked achievements on estimating scour near spur dikes in the previous decades, large tasks remain for future works. All the theoretical and experimental work on local scour near the different spur dikes studied, as well as in-depth past studies, showed a significant need for better understanding of the problem. From the point of view of large-scale distortion of the models, internal flow characteristics do not truly characterize prototype spur dike scouring as per the experimental studies conducted in the laboratory. Moreover, from the literature, the partial knowledge of the exact scour mechanism and the effect of different parameters on scour depth phenomenon has been unveiled. Generally, studies are conducted for uniform sediments and at the upper stage, i.e., hilly rivers, natural riverbed sediments are absent; shielding of beds occurs because of high flow velocity and natural sorting of bed sediments by the steep slopes. Therefore, exploration of scour near spur dikes in non-uniform sediment beds and with a shielded layered bed is essential. The requirement of such studies along with field studies on at least large-scale models is necessary to make progress on scouring predictions.

5. Contraction Scour

5.1 Overview

The reduction in the width of river or channel, to minimize the cost of the structures that are built across, is known as river contraction or channel contraction. Bridges, barrages, weirs, and cross-drainage works are the common structures constructed across the rivers at which the river width is reduced. Also, cofferdams and end dump channel constriction used for the maintenance of the riverbanks are other examples of channel contraction. The reduction in the flow area of the channel increases the velocity of flow in the contracted zone of the channel. As a result, the bed shear stress induced by the flow increases considerably causing the scour of sediment bed within the channel contraction. Such localized scour in the contracted zone of the channel is called contraction scour.

In the categories of scour, general scour refers to the haphazard removal of sediment from the channel bed by the high discharge of water irrespective of the location of riverine structure, whereas local scour is the localized removal of streambed material by erosive action of flowing water around obstructive structures, i.e., piers and abutments. By definition, contraction scour falls in the category of local scour, which occurs due to bridge pier and abutment that cause flow acceleration surrounding them. Due to flow acceleration near the contracted zone, bed shear stress also increases in this zone. As soon as this bed shear stress surpasses the critical shear stress, the scour process will start. Straub (1934) initiated the study of scour in long contractions and proposed a simplified one-dimensional theory. His investigation was later extended and modified by Ashida (1963), Laursen (1963), Komura (1966), Gill (1981), and Webby (1984). Lim (1993) put forward an empirical equation of maximum equilibrium scour depth in long contractions under clear-water and live-bed scour conditions. In most of the aforementioned studies, the investigations were undertaken only on sediment beds.

Contraction may be short or long, depending on the ratio of the longitudinal length of the abutment (L_a) to the channel width (B). Different researchers have selected varying ratios for defining contraction as short or long contraction, e.g., Raikar (2004) has suggested that a long contraction should have a ratio greater than or equal to 1. Channel contractions are designated as long or short based on the ratio of the length of contraction L_a to the approaching channel width (B). As per Komura (1966), a channel contraction is considered long when $L_a/B > 1$, whereas according to Webby (1984) the ratio $L_a/B > 2$.

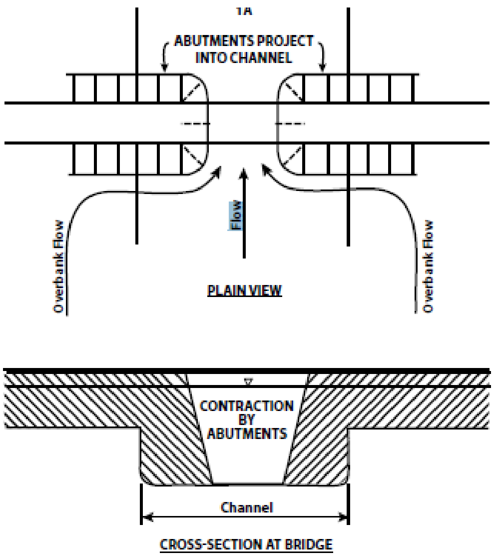


Figure 20. Schematic diagram of bridge contraction (Arneson et al., 2012)

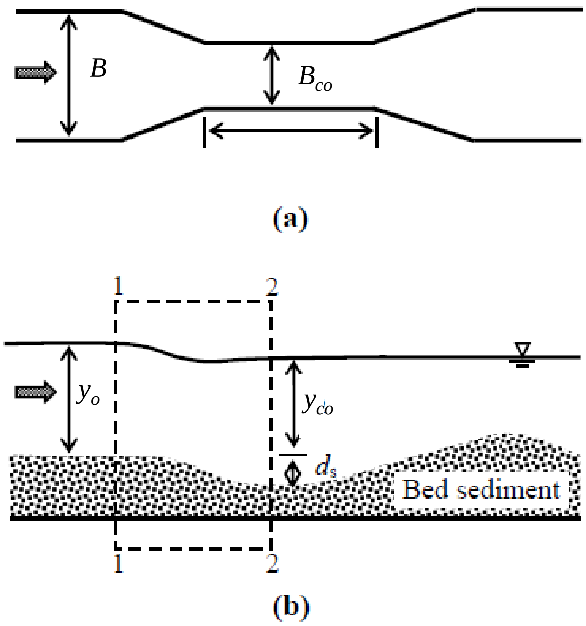


Figure 21. Schematic view of a channel contraction at equilibrium scour condition; (a) top view and (b) side view (Dey and Raikar, 2005)

Detailed analysis of scour depth variation in different studies reports that:

- The scour process takes place swiftly during its initial stage, but after a certain time has passed, the scour rate becomes very small.

- It can be seen that the maximum scour depth always occurs at the upstream abutment nose.
- It is observed that the maximum scour depth increases with increase in contraction ratio. Contraction ratio is the total transverse length of abutment (b_a) to the channel width (B), where the total transverse length of abutment is the sum of transverse lengths of each side abutment. For a fixed value of velocity, the maximum scour depth increases with the contraction ratio. Also, for a constant value of contraction ratio, the maximum scour depth increases with an increase in the approach mean velocity.

Parameters that affect the scour depth at an abutment due to contraction of the flow can be listed as the geometry, transverse length of the abutment, sediment, approach flow properties, and time of scour. Temporally varying scour depth (d_s) around an abutment for the contraction case can hence be expressed as:

$$d_s = f(d_{50}, \sigma, \rho_s, U_c, \rho, v, U_0, y_0, L_a, l_a, b_a) \qquad (70)$$

In Equation (70), the flow parameters are (ρ, v, U_0, y_0), the abutment geometry and contraction parameters are (L_a, l_a, b_a), and the sediment parameters are (d_{50}, σ, ρ_s, U_c), where d_{50} is median diameter of sediment and σ is respective geometric standard deviation, ρ_s is sediment density, U_c is critical mean velocity, ρ is fluid density, v is kinematic viscosity of fluid, U is approach mean velocity, y_0 is approach flow depth, l_a is transverse length of each side abutment, L_a is the longitudinal length of abutment, b_a is total transverse length of abutment, i.e., the contraction width, and d_s is the scour depth.

For sediment–fluid interaction, the parameters g, ρ_s and ρ should not appear as self-governing parameters; a better description is $[(\rho_s/\rho) - 1]g$. Equation (70) can then be written in non-dimensional form,

$$d_s/b = f\left(F_{d_{50}}, y_0/l_a, b_a/B\right) \qquad (71)$$

where $F_{d_{50}}$ is the densimetric Froude number, defined as $F_{d_{50}} = U_0 \big/ \left[\left\{\left(\rho_s/\rho\right) - 1\right\} g d_{50}\right]^{0.5}$, and b_a is the total transverse length, defined as $b_a = 2l_a$.

5.2 Predictive formulas

A few previously suggested analytical equations are available for maximum equilibrium scour depth (d_{sm}) computation near bridge abutments in contraction scour conditions, i.e., Laursen (1963), Gill (1981), Lim (1993), and Singh (2020), as given below.

Laursen (1963):

$$\frac{d_{sm}}{y_0}+1=\left(\frac{b_a}{B}\right)^{-0.875} \qquad (72)$$

Gill (1981):

$$\frac{d_{sm}}{y_0}+1=1.58\left(\frac{b_a}{B}\right)^{-0.875} \qquad (73)$$

Lim (1993):

$$\frac{d_{sm}}{y_0}+1=1.85\left(F_{d_{50}}\right)^{0.75}\left(\frac{b_a}{B}\right)^{-0.75}\left(\frac{d_{50}}{By_0}\right)^{0.25} \qquad (74)$$

Singh (2020):

$$\frac{d_{sm}}{R_L}=0.1\left(F_{d_{50}}\right)^{0.6}\left(\frac{b_a}{B}\right)^{-1.1} \qquad (75)$$

where $R_L=\left(l_a^2 y_0\right)^{1/3}$.

In case of a situation as shown in Figure 20, the scour near the abutments in contraction scour can be determined by equations 72 - 75. However, for a long contraction (Figure 21) the scour due to contraction in sand and gravel bed channels may be evaluated using equation (76). Dey (2005) proposed an analytical model to evaluate the equilibrium scour depth according to,

$$d_s=y_{co}-y_0+\frac{U_{co}^2}{2g}-\frac{U_0^2}{2g} \qquad (76)$$

where:

d_s	=	Equilibrium Scour depth
y_o	=	Flow depth in normal channel
y_{co}	=	Flow depth in contraction zone
U_o	=	Flow velocity in normal channel
U_{co}	=	Flow velocity in contraction zone

Dey (2005) also proposed an empirical equation to evaluate the equilibrium scour depth using,

$$\frac{d_s}{y_0} = 0.368 F_{1e}^{0.55} \left(\frac{d_{50}}{y_0}\right)^{-0.19} \left(\frac{B_{co}}{B}\right)^{-1.26} \qquad (77)$$

where:

B = Channel width

B_{co} = Channel width in contraction zone

F_{1e} = Excess approaching flow Froude number, $U_{1e} / \left(\Delta g y_0\right)^{0.5}$

U_{1e} = excess approaching flow velocity

The FHWA HEC 18 method includes a modified version of the Laursen (1960, 1963) live-bed and clear-water contraction scour equations for non-cohesive soils, and uses the Briaud *et al*. (2011) equation for cohesive soils.

The modified version of Laursen (1960) live-bed scour formula is:

$$d_s = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1} y_o - y_{co} \qquad (78)$$

Where:

Q_1 = Flow in the upstream channel transporting sediments, (m^3/s).

Q_2 = Flow in the contracted channel, (m^3/s).

W_1 = Bottom width of the upstream main channel that is transporting bed material (m).

W_2 = Bottom width of the main channel in contracted section minus pier widths (m).

K_1 = Exponent for mode of bed material transport (calculated from table below)

$V^* = (g\ y_1\ S_1)^{1/2}$ shear velocity in the upstream approach section (m/s).

ω = Fall velocity of bed material based on D_{50} and temperature (T) (m/s). (based on the graph below)

g = Acceleration of gravity (m/s^2).

S_1 = Slope of the energy grade line of main channel (m/m).

V* /ω	K1	Mode of bed material transport
< 0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

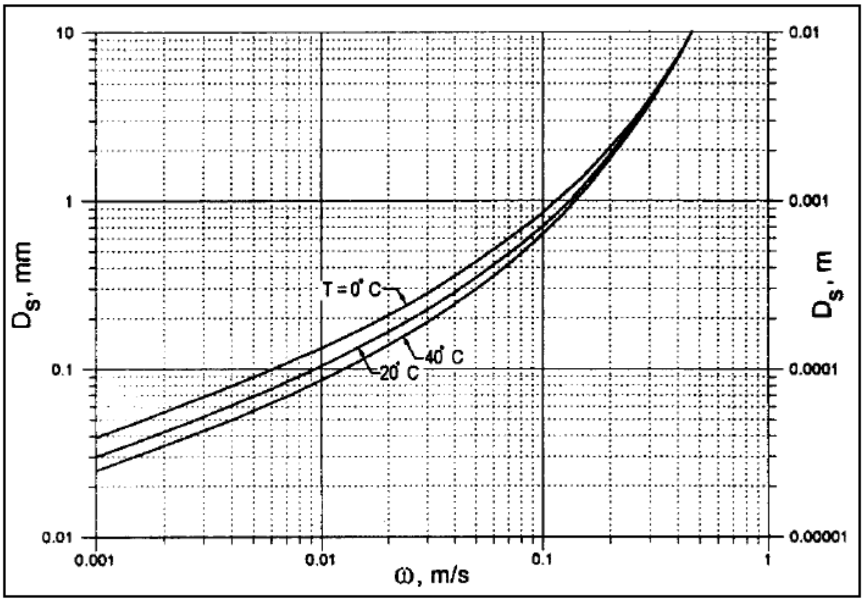


Figure 22. Fall velocity of sand-sized particles with relationship of water temperature (Arneson et al., 2012)

The modified version of the Laursen (1963) clear-water scour formula is:

$$d_s = \left(\frac{K_u Q_2^{\ 2}}{D_m^{\ 2/3} W_2^{\ 2}} \right)^{\frac{3}{7}} - y_{co} \qquad (79)$$

where:

D_m = (1.25 *D_{50}) Diameter of the smallest non-transportable particle in the bed material in the contracted section (m).

D_{50} = Median diameter of the bed material (m).

K_u = 0.025 for metric units.

From Briaud *et al*. (2011):

$$d_s = 0.94 y_0 \left(\frac{1.83 U_{co}}{\sqrt{g y_o}} - \frac{\sqrt{\frac{\tau_c}{\rho_w}}}{g n y_o^{1/3}} \right) \tag{80}$$

where:

τ_c = Critical shear stress (N/m^2).

n = Manning's value.

ρ_w = Density of water (kg/m^3).

5.3 Conclusions

The physical mechanisms of the bridge scour process are very complex. Furthermore, the variability of the site conditions and the potential interaction of the various components of scour make predicting the scour depth using general formulas based on the assumption of a very long contraction, uniform and non-uniform sediment a difficult problem. For example, the contraction scour in the field actually develops in an abrupt contraction with a non-uniform sediment bed. Contraction scour is very dynamic and constantly adjusting to the incoming sediment load. Some general observations are:

- The scour depth increases with an increase in approaching flow depth at lower flow depths, but it becomes unaffected by the approaching flow depth at higher flow depths.
- The scour depth increases with a decrease in contracted width of channel.
- The non-uniform sediments reduce scour depth to a great extent due to the formation of the armor layer within the scour hole.

6. Design procedures

Evaluation of bridge scour risks is one of the most important steps in bridge design and inspection to safeguard bridges within their design life. In general, many countries follow their own manuals or standards for bridge scour analysis, but they include many similarities in procedures and steps. For many countries, increasing climate change effects associated with large river flows would enhance the importance of bridge scour evaluation at a detailed level. This chapter summarizes the design procedures of bridge scour analysis practiced in the USA, Australia, and UK.

6.1 USA

In the United States, the Federal Highway Administration (FHWA) has issued three Hydraulic Engineer Circulars (HEC) to provide proper guidance for comprehensive scour evaluation at different stages. The bridge scour analysis steps and procedures outlined in the following circulars are strictly adhered by many state transportation agencies in the United States.

1. HEC 18 Evaluating scour at bridges
2. HEC 20 Stream stability at highway structures
3. HEC 23 Bridge scour and stream instability countermeasures

Figure 23 provides a brief explanation of the activities involved (not complete procedure) and the relationship between the three circulars. The procedures are normally undertaken for both new and existing bridges. Through the above documents, FHWA stresses to practitioners the requirement of a multi-disciplinary approach involving hydraulic, structural, and geotechnical engineering disciplines to evaluate the potential scour risk for newly design as well as constructed bridges. FHWA insists to assess scour vulnerability of every water structure during all design stages, also in the inspection stage with a period after construction, to take adequate measures for minimizing structural damages. In the USA, the evaluation of scour at existing bridges are conducted through the National Bridge Inspection Program of FHWA (Arneson *et al.*, 2012) based on the guidelines outlined in HEC 18.

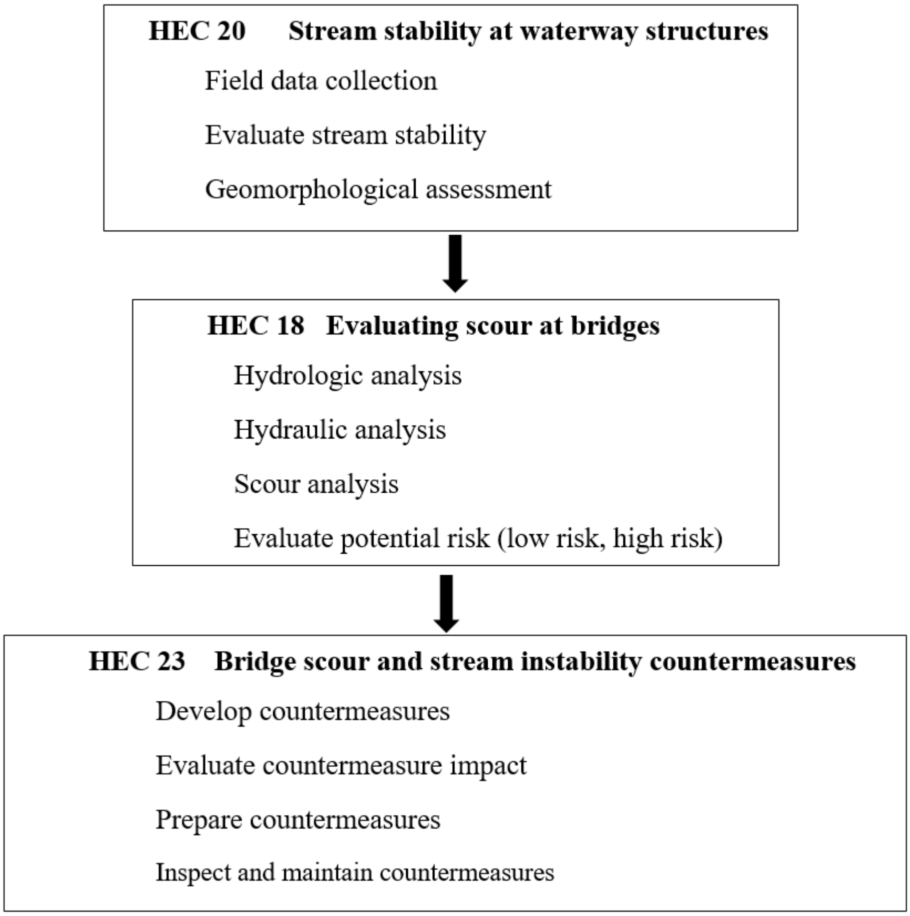


Figure 23. The flow chart including summarized steps for complete scour analysis and countermeasures (Arneson et al., 2012)

6.1.1 HEC 18 method

HEC 18 is a detailed document including comprehensive knowledge and a guide for the design and evaluation of bridges regarding bridge scour. The new version (fifth edition) of HEC 18 was released in 2012 by replacing the previous version issued in 2001. The revised version includes many novel approaches and outcomes from new research projects conducted by various agencies in the USA (Arneson *et al.*, 2012).

According to HEC 18 recommendations, the general bridge design for potential scour can be performed by an eight-step procedure. The first four steps determine a method to estimate total potential scour, and the later four steps evaluate the bridge foundation for scour risk to produce a better hydraulic design with prudent countermeasures, if needed to prevent bridge failures due to a pronounced scour risk. Therefore, this report summarizes the first four steps with general information as follows.

Step 1. As mentioned in previous chapters, hydraulic characteristics of the flow involves are highly important parameters to be considered for bridge scour estimation that directly influences the magnitude of the scour depth. Therefore, to determine a suitable scour design flood is the most important first step in the analysis. Through a risk-based approach, FHWA recommends minimum scour design flood based on a hydraulic design flood as shown in Table 5. The scour design flood is always higher than the corresponding hydraulic design flood to ensure the safety of bridges for a higher level of scour, when a flood exceeds the hydraulic design flood. Generally, in step 8, FHWA reevaluates the whole foundation design for a scour design check flood to make sure the design reaches a minimum safety factor 1.0. Otherwise, necessary modifications in the foundation design to reach safety requirements are required. However, if there is an overtopping flood that makes a greater impact on the bridge and less than a scour design flood, then that flood would be used as scour design flood without considering the listed values in Table 5. (Arneson *et al.,* 2012).

Table 5. The design flood for scour analysis which outlined in HEC 18 (Arneson et al., 2012)

Hydraulic Design Flood Frequency	Scour Design Flood Frequency	Scour Design Check Flood Frequency
Q_{10}	Q_{25}	Q_{50}
Q_{25}	Q_{50}	Q_{100}
Q_{50}	Q_{100}	Q_{200}
Q_{100}	Q_{200}	Q_{500}

Step 2. In this step, the designer develops necessary hydraulic characteristics (e.g., water depth, flow width, and velocity) corresponding to scour design floods through a one or two-dimensional hydraulic model. It is necessary to study the complete hydraulic conditions at the forecasted or existing bridge site. In the USA, the HEC-RAS software developed by the U.S. Army Corps of Engineers (USACE) is widely used for the hydraulic analysis. Furthermore, HEC-RAS includes an in-built bridge scour analysis option based on the HEC 18 method outlined in the fourth edition (2001). Therefore, the analysis is

limited to cohesion-less particles and the hydraulic properties can be acquired through a one-dimensional hydraulic model (Brunner, 2016).

Step 3. This is the step to estimate the total potential scour based on hydraulic conditions developed in previous steps. This step includes collecting scour analysis variables (hydraulic data, geotechnical data, and structural data), determining the effect of long-term degradation or aggradation, estimating potential contraction, pier, and abutment scour depth, determining foundation depths for abutments, and evaluating estimated scour depths based on limitations in the calculation methods.

The contraction scour for non-cohesive soils is estimated using a modified version of the Laursen (1960, 1963) live-bed (equation #78) and clear-water (equation #79) contraction scour equations; for cohesive soils the scour is estimated by the Briaud *et al*. (2011) equation (equation #80) as listed in chapter 5.

The pier scour for non-cohesive soils is estimated using the HEC 18 pier equation (equation #10) for both live-bed and clear-water conditions; for cohesive soils it is estimated by the Briaud *et al*. (2011) equation (equation #14) as listed in chapter 3. Furthermore, HEC 18 includes many other modified equations to address the complexity of different pier arrangements, debris accumulation, and weathered rock foundations.

The abutment scour is estimated using the Froehlich live-bed equation (equation #47) or the HIRE equation (equation #63), as listed in chapter 4. Besides, HEC 18 includes the NCHRP 24-20 approach for abutment scour estimation. This approach is an outcome of the National Cooperative Research Program (NCHRP) funded by FHWA in 2010 (Ettema *et al*., 2010).

Step 4. In this step, the estimated and adjusted scour depths in the step 3 will be plotted across the cross-section of the channel and flood plain for further evaluation, also being the basis for the foundation design.

According to FHWA, the above steps can be employed to do scour evaluation for existing bridges as well. However, it may require knowledge of many different aspects to implement a Plan of Actions (PoA) for bridges exposed to critical scour. A scour countermeasure design flood is used to check the suitability of the designed countermeasures.

6.1.2 Other methods

Apart from HEC 18, there are a few other methods for scour estimation that are used in the USA for certain applications:

- SRICOS-EFA Method

The SRICOS-EFA (Scour Rate in Cohesive Soil-Erosion Function Apparatus) method was developed by Briaud and his colleagues in 1990 (Briaud *et al*., 2011). The method includes time effects and thus gives an estimation of scour rates- This is the major advantage of this method apart from being applicable in cohesive soils.

- Simplified SRICOS Method
  Briaud *et al*. (2009) developed this simplified method due to typical constraints regarding in-situ sampling and laboratory testing. The soil characteristics and erodibility for a particular site are obtained from pre-classified charts based on earlier research.

- The FDOT Method
  The method was developed by the Florida Department of Transportation (FDOT) in 2005 for pier scour estimation.

- ABSCOUR Method
  The method was developed by Maryland State Highway Administration (MD SHA) in 2007 and is similar to the HEC 18 method. In comparison, this method is using the MDSHA proposed equations to estimate abutment scour based on research and development by Chang and Davis (1999).

6.2 Australia

In Australia, the general bridge scour design practices follow the Austroads guide. Austroads is a non-profit collective body that includes the Australia and New Zealand transport agencies, and it delivers proper guidance to construct sustainable transport infrastructures in both countries through inclusive research projects. The complete bridge infrastructure guidelines are issued through “Austroads Guide to Bridge Technology”, which is divided into 8 parts. The bridge scour design procedure is included in “Guide to Bridge Technology Part 8: Hydraulic Design of Waterway Structures”, and the current edition 2.1 was published in 2019. Though part 8 (hereafter reference as Austroads scour guide (2019)) provides information about bridge scour design, it requires complementary guidance from other chapters, since the bridge scour design constitutes a multidisciplinary approach, as discussed before.

6.2.1 Austroads scour guide (2019)

The scour guide includes foundation design of bridges for scour, scour estimation procedure, design of scour countermeasures, and monitoring and evaluation of potential scour at existing bridges. Overall, the bridge scour design procedures are very similar to the method outlined in HEC 18 of FHWA, USA. Therefore, under this section we briefly discuss the differences in the Austroads scour guide compared to the HEC 18 scour manual.

According to the Austroads scour guide (2019) it is important to select relevant floods for bridge scour design. It suggests that the peak flood conditions do not always produce the highest velocity and shear stress in the bridge sections. In general, the overtopping flood is the most crucial flood that generates severe scour conditions at bridges. Therefore, it is important to find the overtopping flood at the bridge site for every bridge scour design situation.

For new bridges, the foundation is designed based on a scour estimation calculated for hydraulic design floods. Furthermore, the designed foundation should be evaluated for scour representing the Ultimate Limit State (ULS). The ULS is known for a 2000-year return flood. If an overtopping flood condition that prevails and it is less than the 2000-year return flood, then the particular overtopping flood will be used to evaluate the foundation design against potential scour without considering ULS. If the overtopping flood is critical and less than hydraulic design flood, then the overtopping flood will be used to design the bridge foundation irrespective of the design evaluation procedure. For existing bridges, the scour countermeasures for bridges exposed to critical scour will be designed for the critical flood (overtopping flood) or the 50- or 100-year return flood (Austroads, 2019).

The Austroads scour guide (2019) includes an important procedure for designing abutment protection. Australian agencies follow this common approach, because of shortcomings to validate the abutment scour estimation techniques and its typical overestimation. The abutment protection will be either rock riprap or guide banks. This protection will be designed based on considering estimated contraction scour, local pier scour, and abutment scour depth, which can induce pressure on the abutment foundation. In the end, a reasonable scour depth will be determined to design abutment protection by evaluating predicted scour depths. The scour estimation is performed using the waterway design flood or overtopping flood when it is less than the waterway design flood. However, the manual urges that the protection design needs to be evaluated based on the engineering judgment from the hydraulic, geotechnical, and structural disciplines.

The method of scour estimation in Austroads scour guide (2019) agrees to a large degree with the HEC 18 method, including similar equations and alternative approaches. The hydraulic characteristics can be determined using a one- or two-dimensional hydraulic model for representing the hydraulic conditions that are employed for relevant flood scenarios. The magnitude of the scour depth is independently estimated for contraction, pier, and abutment scour. If the contraction scour becomes significant, the local scour should be estimated based on recalculated hydraulic characteristics after including the effect of contraction scour.

Mean velocity method

The mean velocity method is another important concept included in the Austro scour guide (2019) to evaluate the contraction scour obtained by proposed equations (similar to HEC 18 method). At first, the mean velocity of the hydraulic design flood in the unrestricted main channel is calculated by assuming that this flood will initiate contraction scour at the restricted section. In practice, according to different transport agencies, the hydraulic design flood could be the 50- or 100-year return flood. Next, an average contraction depth at the bridge section is estimated corresponding to the calculated mean velocity through the contracted bridge section, based on the continuity law. Finally, the contraction scour depths estimated both from the equations and the mean velocity method will be compared to make the final assessment (Austroads, 2019).

6.2.2 Bridge Scour Manual of Queensland Department of Transport and Main Roads

The Department of Transport and Main Roads of State of Queensland has issued the second edition of the bridge scour manual in January of 2019. This document is a supplement to Austroads bridge scour guide, edition 2.0 released in 2018. This manual was technically reviewed by the New Zealand scientist Bruce Melville. He has been engaged in extensive research regarding bridge scour in the past. This supplement provides some additions and amendments to the Austroads guide by carefully evaluating it. However, most of the addition and amendments were related to HEC 18 scour manual.

The supplement guide recognizes that use of a two-dimensional model to obtain hydraulic characteristics is appropriate. Meanwhile, it emphasizes the benefit of Computational Fluid Dynamics (CFD) models to calculate bed shear stress around bridge piers and abutments. For design approach, the guide highly recommends to conduct an initial assessment before estimate scour depth. This assessment is for evaluating field conditions around the bridge site concerning the scour risk by considering geomorphological and geotechnical conditions. Furthermore, the guide includes the Melville and Coleman (2000) equation (equation #22) for pier scour depth estimation as an alternative method. The recommended procedure for complete scour assessment is given below in Figure 24.

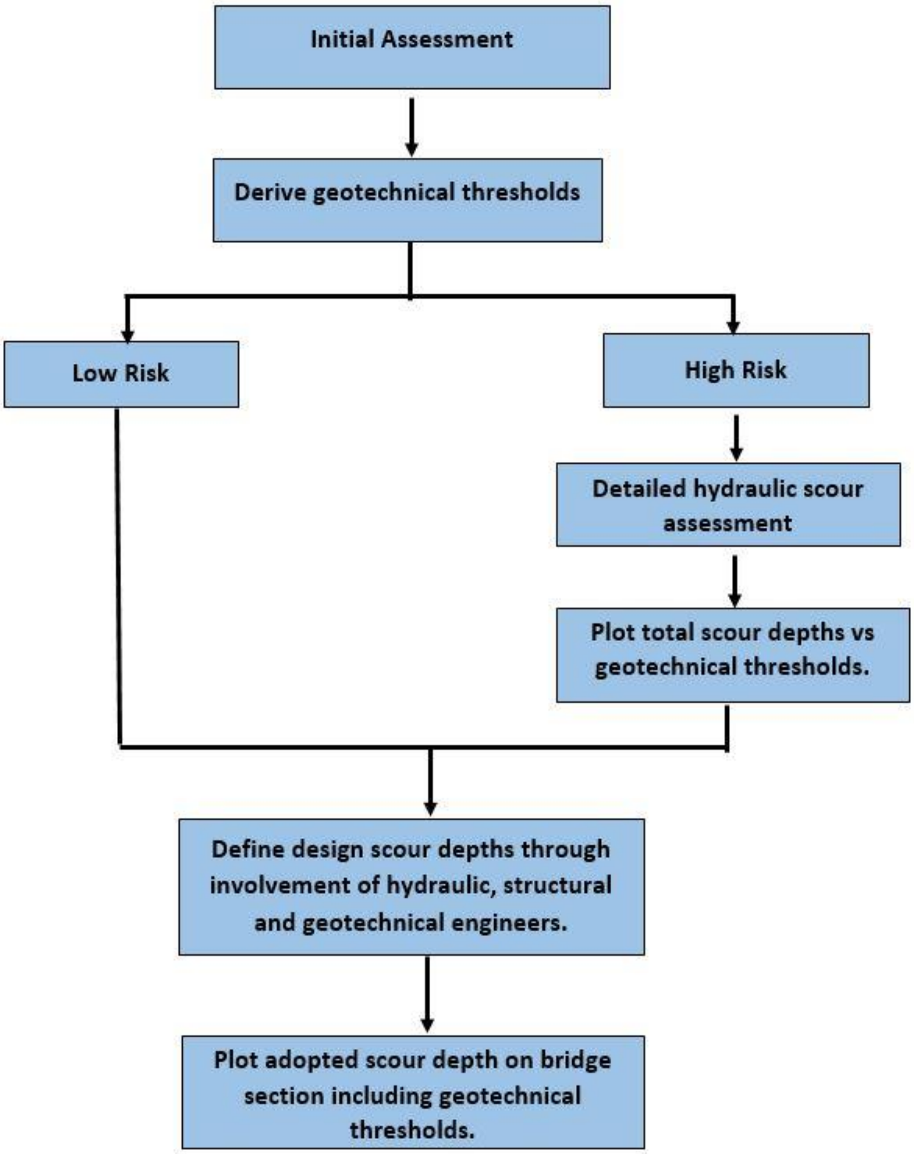


Figure 24. The recommended scour assessment methodology in the Queensland bridge scour manual (2019)

6.3 United Kingdom (UK)

In the UK, the scour assessment is being a historical topic involving many stakeholders and authorities. The Construction Industry Research and Information Association (CIRIA) developed and published a comprehensive manual for scour assessment through a research project in 2015. The manual is known as "Manual on scour at bridges and other hydraulic structures, second edition" (hereafter reference as CIRIA C742). The CIRIA is a non-profit and a neutral professional body formed by representing clients, contractors, consultants, public sectors, and academia across the UK boarders. The body works to enhance performances of the construction industry through better collaboration and transferring knowledge among sectors.

6.3.1 CIRIA C742 (2015) manual

The comprehensive manual provides systematic guidance to identify scour potential corresponding to river morphology, hydraulic, and geotechnical characteristics. Further, it includes a detailed scour risk assessment for new and existing structures, scour estimation methods, and scour mitigation measures. The systematic methodologies and tools for scour risk management are the unique and highlighted chapters in the CIRIA C742 manual. Therefore, under this section we briefly discuss the bridge scour risk management strategy presented in CIRIA C742. Table 6 summarizes some of the important steps for the scour risk management cycle outlined in CIRIA C742.

Table 6. The important steps for scour risk management cycle outlined in CIRIA C742 manual (Kirby et al., 2015)

	Anticipation	**Assessment**	**Prevention**
Objective	To identify the hydraulic structures which could have potential scour risk, and make a common database system for further assessment.	To identify and quantify the risks, evaluate the consequences, and support for the development of comprehensive mitigation measures and plans.	To implement proper scour prevention measures based on comprehensive assessment results.

Sub steps	❖ **Screening** This step determines every structure over the waterway and categorizes them as low risk or potentially at risk structures based on scour history, watercourse type, asset location, structure design, and consequences of failure. The potentially at risk structures are involved for further assessment.	❖ **Initial assessment** This step uses to identify scour risk at the initial level and prioritize it for detailed assessment based on its vulnerability. This step helps to analyze available data and identify other requirements for detailed investigations. ❖ **Survey and Inspection** This step uses to assess bed profile around close proximity of the structures through topographic survey, an inspection of structure and waterway through site inspection, structural inspection, and underwater inspection to identify scour related detritions or defects.	❖ **Scour mitigation measures** This step develops suitable scour mitigation measures. In general, the scour mitigation measures can be divided into scour reduction measures, structural measures, and scour protection measures. The selection of the method will be based on holistic outcomes of previous steps.
	❖ **Asset register** The data management system is used to capture and store the structures, and existing conditions of the structures in a sophisticated way by using GIS, online geological mapping or computer programs.	❖ **Detailed assessment** This step estimates potential natural, contraction scour and local scour against relevant (designed) hydraulic conditions. Further, the designed or existing foundation depth will be evaluated against potential scour depth to identify suitable scour mitigation measures.	❖ **Debris management** This step determines the scour enhancement due to the accumulation of debris at hydraulic structures. Further, it needs to be a proper system for removal and disposal of debris by considering

	This step provides an efficient way to access and use stored data for prioritize actions.	❖ **Re-assessment** This step utilizes the dynamic of scour risk over a period of time due to changing hydraulic and geomorphological changes. Therefore, a proper system need to repeat assessment over a certain period of time based on specific site conditions.	environmental effects.
		❖ **Monitoring** This step uses for continuous monitoring of the enhancement of scour depth at structures by using suitable monitoring devices. Though the monitoring is not a solution for scour risk, which can be used to detect critical risk for take decision on replacement works or installation of protection measures. There are some occasions that give favor for installing a monitoring device.	❖ **Economic appraisal** This step guides decision-makers to adopt the correct investment for scour prevention measures based on monetary and non-monetary costs in the long run. The most common economic appraisal methods are cost-effectiveness analysis, cost-benefit analysis, and multi-criteria analysis.
		❖ **Environmental Assessment** This step helps to recognize the surrounding ecosystem from a broad perspective. Though some measures have good positive socio-economic effects	

		sometimes which may have negative effects on the existing ecosystem. Therefore, identify a sustainable solution for scour protection works by considering environmental impacts is very important. There is a strong recommendation to carry out an Environmental Impact Assessment (EIA) before implementing scour mitigation works.	

According to CIRIA C742, a 100- or 200-year return design flood will be used for scour estimations in the UK practices. However, the manual suggests hydraulic structure owners to perform their own risk assessment to select a suitable design flood. Furthermore, it acknowledges that the foundation should be designed with scour protection to withstand the complete potential scour risk for the design flood. The UK Network Rail uses a 200-year return flow for designing new structures, and by considering climate change they recommend to check the stability of the structure for a peak flow by adding 20% to the design flood (Kirby *et al.,* 2015).

The method of bridge scour estimation in CIRIA C742 (2015) has a similar approach to the HEC 18 method to predict contraction scour. For pier scour estimation, it recommends a non-dimensional equation (equation #23) based on previous research. Since the presented equation and the Sheppard-Melville method, equation #28 (also known as NCHRP Project 24-32 (Ettema *et al.,* 2011) produce almost similar results, CIRIA C742 recommends the Sheppard-Melville method as well to predict scour depth at piers. Also, the equation from Melville and Coleman (2000) (equation #22) is presented in the guide as an alternative option to compare estimated pier scour results with. For abutment scour estimation, the manual recommends to use Melville (1997) equation (equation #49) as listed chapter 4. CIRIA C742 also emphasizes the importance of multi-disciplinary approaches to scour assessment, similar to other manuals.

6.3.2 Other standards in UK

Apart from CIRIA C742 (2015) guide in the UK, several agencies have their own approaches and protocols towards scour assessment based on commissioned studies. Among them, the Highways Agency, Network Rail, Railway Safety and Standard Board (RSSB), Environment Agency, TSO, and Canal and River Trust (CRT) are well-known authorities for their many studies regarding scour design and mitigation procedures (Kirby *et al*., 2015).

6.4 Sweden

As mentioned in Section 2.3, the guidelines in Sweden for estimating and designing against local scour around bridges are not very comprehensive and rather simplistic. In the reports by Trafikverket (2011a, 2011b), it is simply stated that the foundation of a bridge should be protected against erosion that may cause damage. If the mean water velocity is less than 2.0 m/s, then the guidelines presented in Vägverket (1987) should be followed; in cases where the mean velocity is higher than this, then a special investigation is required for the analysis and design. In Trafikverket (2017, 2019) it was noted that erosion protection at bridges has typically been designed for flows corresponding return periods between 50 and 100 years.

The procedure to estimate bridge scour discussed in Vägverket (1987) basically only deals with pier scour, whereas contraction and abutment scour is not included. However, there is a simple rule of thumb given that can be used, if the construction of the bridge protrudes into the water; this addresses in a simple manner the two latter types of scour. This rule states that the remaining river cross section erodes as much as the structure reduces the original cross-sectional area. It qualitatively captures some aspects of the formulas in Section 5.2 (see also Schiereck, 2001), where an increase in velocity due to the contraction will generate a deepening of the river bed until equilibrium prevails. However, in most cases this procedure is too primitive for reliable estimates of scour related to a reduced cross-sectional area.

The method recommended by Vägverket (1987) for pier scour is similar to the one presented by Breuser *et al*. (1977), although some coefficient values are given in graphical form and not as a mathematical function. The maximum equilibrium scour depth is a function of the ratio between the undisturbed depth in the river and the width of the bridge pier as given graphically. Also, two correction factors are introduced, one with regard to the orientation of the bridge pier towards the main flow direction in the river and the other with regard to the shape of the pier nose. The equation presented in Vägverket (1987) is mainly valid for a rectangular pier shape and no effects of the sediment properties are included. Schiereck (2001) discusses a more general version of the equation that relaxes these limitations. If the foundation for the bridge pier is wider than the pier itself and emerges above the river bed, a method is proposed to compute a modified, representative width based on an assumption about the velocity distribution through the water column (Vägverket, 1987).

7. Climate change impact on bridge scour

It is expected that climate change will cause extreme weather conditions more frequently in the future compared to the past (IPCC, 2021; IPCC, 2014; Alfieri *et al*., 2015; WMO, 2017; Sköld Gustafsson *et al*., 2021). Changing conditions may also include other climate-related phenomena such as sea level rise, increase in atmospheric temperature, increase in intensity and frequency of precipitation, strong winds and storms, drought, and wild fires. Among the above-mentioned changes, increase in intensity, duration, and frequency of precipitation and rain storms have the largest direct impact on bridge scour by increasing flows in rivers. Nasr *et al*. (2019) indicated bridge scour as one of the main climate change risks related to bridge structures, resulting from increasing water depths and velocities in rivers. The increased discharge will most likely significantly change hydraulic conditions in rivers, where the effects are closely related to the existing river geometry. According to the river capacity, sometimes rather small changes in the flow can significantly alter the turbulent conditions in a river. In addition to rainfall, large amounts of melting snow within a short period of time can cause floods in rivers, especially in northern Europe, leading to higher mean velocities and enhanced turbulent conditions around bridge structures. Eagle *et al*. (2021) studied changes in the overall river morphology due to repeated peak flows and general scour. These morphological changes can indirectly affect scour at structures over longer period of time. In recent years, there are many rivers around the world experiencing extreme floods (see Floodlist, 2021). Many studies suggest that the risk of river floods will increase significantly in the future across Europe and worldwide (Alfieri *et al*., 2015; Christodoulou and Demirel, 2017; Alfieri *et al*., 2017).

As mentioned in chapter 6, identifying a suitable design flood and its return period is a primary step in bridge scour assessment. Climate change impact on future flows brings significant uncertainties to the selection of design floods for scour assessment (Dikanski *et al*., 2018; Yang and Frangopol, 2019). Many widely used scour manuals still do not include climate change impacts; however, UK based Network Rail and TSO adopt a rule of thumb to increase peak flows by 20% after estimating design flood frequencies in their scour assessment procedure (Kirby *et al.,* 2015). It is reasonable to include climate change effects when considering the long service life of bridges, required to maintain functionality during their entire lifespan. According to Nemry and Demirel (2012), around 20% of the bridges in Europe exhibit risks of bridge scour during the period 2041-2100 due to increases in peak flows related to climate change. Figure 25 shows the forecasted increased bridge scour risk across Europe in terms of the increase in the 100-year return flow for the periods 2040-2070 and 2070-2100.

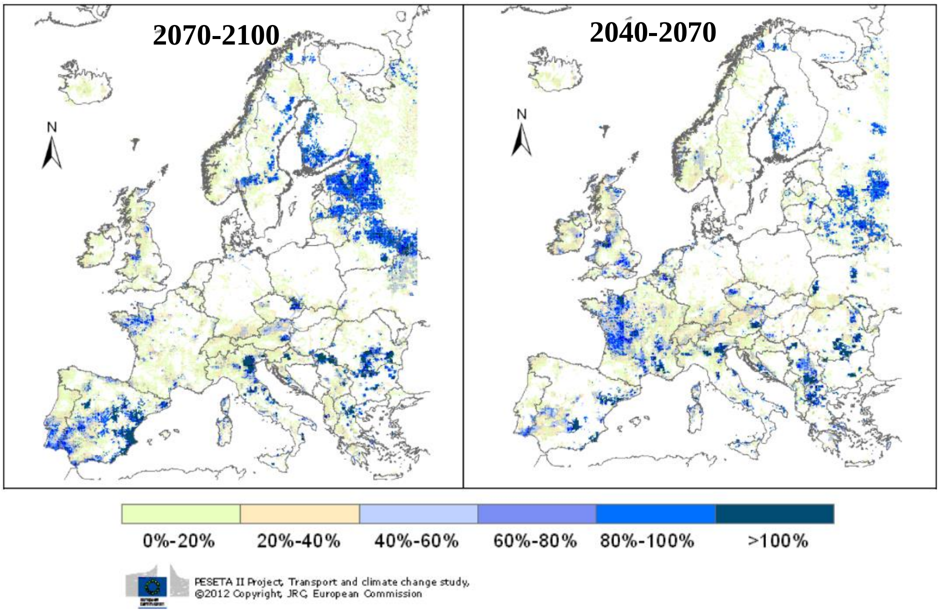


Figure 25. Predicted increase in bridge scour risks for Europe based on the increase in the 100-year return flow (Nemry and Demirel, 2012)

Apart from intensity, an increased frequency in peak flows implies significant impact on scour depth evolution. As explained in Figure 18 in Chapter 3, scour depth increases gradually during an event with a peak flow; later backfilling of the scour hole can take place during lower flows based on the sediment transport conditions in the river. Therefore, regularly occurring peak flow events will change the scour evolution cycle and further enhance scouring during short time intervals.

SMHI has been working with climate change impact using in-depth analysis of historical observations together with forecasts of future climate parameters. Furthermore, SMHI has undertaken climate risk analysis in Sweden, especially focusing on heavy rainfalls in river catchments resulting in high flows in water courses. SMHI reports that the climate has been become warmer and wetter in Sweden compared to a reference period from 1961 to 1990 (SMHI, 2021b). According to climate projections, Sweden will experience torrential rainfalls more often in the future and the intensity of a 10-year rain will increase by 25% until end of this century (Olsson and Foster, 2013). As shown in the examples discussed in Chapter 2, many cities in Sweden are situated close to rivers. Thus, in addition to the runoff from the river catchment, the runoff from urban areas may bring additional flow to the river that is significant and can possibly cause flash floods. Continuous urbanization will increase the runoff to rivers, which may result in larger impact on bridge structures during short time periods.

Also, because of extreme weather events with heavy rainfall and strong winds, catchments can generate large amounts of wood debris (e.g., tree trunks and limbs) that follows with the water flow in the river. The problem is expected to be more severe where rivers flow through forested catchments that have steep slopes. Wood debris can easily accumulate at bridges, reducing the flow area which causes backwater effects and increased downstream velocities with enhanced scour (see Figure 26). Such flow obstruction will also generate extra turbulence around bridge structures, again inducing more pronounced scour (Arneson *et al*., 2012). Expected future increase of such problems led Lagasse *et al*. (2010) to conduct a comprehensive study about the effects of debris on bridge scour.

Figure 26. Example of debris accumulating at bridge piers (Lagasse et al., 2010)

Other adverse effects of climate change relevant for bridge scour are simultaneous occurrence of multiple climate hazards and cascading effects of multiple hazards. Several recent studies have been carried out on multiple climate hazard in Sweden and Europe (Johansson *et al*., 2021; Sköld

Gustafsson *et al*., 2021; Forzieri *et al*., 2016). There are several combinations of climate hazards that can enhance bridge scour risks, directly or indirectly. For example, increasing temperature in the summer can affect the conditions of cloud formation and hence induce intensive summer rainfalls (Berg *et al*., 2013). Storms with heavy precipitation can generate large river discharges and wood debris from surrounding bank failures and landslides. Furthermore, after the occurrence of wild fires there is an increased risk of flash floods due to changes in soil permeability and surface runoff conditions; the risk is particularly high for steep-slope terrain that can generate large amounts of debris flow downhill (Thomas *et al*., 2021). Therefore, the occurrence of wildfire around steep floodplains can increase bridge scour risks with regard to wood debris accumulation at bridge structures.

8. Conclusions

Bridge scour is a difficult topic involving complex flows around a structure and over a movable bed where strong interaction takes place between the flow and the sediment. In general, bridge scour is divided into pier, abutment, and contraction scour. For the former two types of scour, it is primarily secondary flows induced by the structure that cause the scouring leading to increased sediment transport around structural elements (e.g., piers and abutments). In case of contraction scour, a reduction in the cross-sectional area increases the mean velocity, which in turn induces increased sediment transport and associated gradients that result in local erosion (scour).

Since the flow and sediment transport conditions are challenging to simulate in numerical models, analysis and design of bridge scour are typically based on empirical formulas determined from laboratory data (field data are scarce). There are large uncertainties in these formulas, so they have to be employed under careful considerations. However, the analysis and design procedures, as well as the recommended calculation formulas, presented in governmental codes developed in different countries, are normally sufficiently accurate to obtain a first, general idea of possible scour problems. In this respect, the US design code proposed by the Federal Highway Administration known as HEC 18, constitutes a robust and reliable approach to investigate bridge scour.

Some design codes from other countries, for example, Australia and UK, use a bit different procedures, but too a large extent the same equations are used. In Sweden, the design guidelines are brief and limited to certain types of pier scour. Thus, to improve and generalize these guidelines, preferably parts of the US code could be used in design and analysis regarding bridge scour.

The expected influence from climate change on bridge scour is through larger and more intense rainfalls in the future that imply larger flows in the rivers. This in turn will cause increased bridge scour, both concerning pier, abutment, and contraction scour. In Sweden a more thorough investigation is needed of the implications of increased river flows on bridge scour. Such an investigation would require the collection of field data, bathymetric analysis, and simulation of river flows and bridge scour under different scenarios.

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