Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 80 (1999)

Artikel: New Zealand case studies of scour at bridge foundations

Autor: Coleman, Stephen / Melville, Bruce

DOI: https://doi.org/10.5169/seals-60778

New Zealand Case Studies of Scour at Bridge Foundations

Stephen COLEMAN
Lecturer
Civil and Resource Engineering
The University of Auckland
Auckland
New Zealand

Stephen Coleman received the PhD degree from The University of Auckland in 1992, winning the international Lorenz G. Straub Award in 1991. He then worked as a consulting engineer before returning to The University of Auckland. His specialist areas of research and consulting expertise include river mechanics; bridge scour; and overtopping of embankment dams.

Bruce MELVILLE
Associate Professor
Civil and Resource Engineering
The University of Auckland
Auckland
New Zealand

Bruce Melville received the PhD degree from The University of Auckland in 1975. Following five years hydraulic design work in UK, the Middle East, S.E. Asia and NZ, he returned to an academic position. His research interests are focused on various aspects of fluvial sediment transport including bridge scour. He is Associate-Editor of the Journal of Hydraulic Engineering, ASCE.

SUMMARY
Three case studies of scour at New Zealand bridges are presented to illustrate examples of components of scour which need to be considered in bridge foundation design. The Ashburton River Road Bridge was threatened by scour arising from a combination of progressive degradation, local scour, and debris accumulation effects. The Bulls Road Bridge failed principally as the result of a combination of progressive degradation, scour at a bend, local scour for oblique flows, debris accumulation effects, and lateral loading for the failed pier. The Waitangitaona River Road Bridge failed as the result of the combination of scour at the convergence of flow paths (scour at a confluence), contraction scour, local scour for oblique flows, and debris accumulation effects. Various forms of measures proposed and adopted for remediation and prevention of scour in these cases are presented, where these measures include modified pier designs, riprap aprons, riprap weirs, and riprap-lined guidebanks.

1 INTRODUCTION

Sediment movement or erosion around bridge foundations, referred to as bridge scour, can result in damage to the bridge structure. The types of scour that can occur at a bridge crossing are typically referred to as ***general scour***, ***contraction scour*** and ***local scour***. ***Progressive degradation*** or ***aggradation***, ***thalweg effects***, ***scour at a bend***, ***scour at a confluence***, and ***bed-form migration effects*** are all components of general scour which act to influence riverbed elevations. General scour components influencing the planform of the river include ***shifts in channel bends***, ***meander migration***, ***shifts in braids or anabranches*** within the channel, and also ***lateral bank erosion due to channel widening***. Whereas general scour occurs irrespective of the existence of the bridge, contraction and local scour arise principally due to the presence of the bridge, contraction scour occurring as a result of the flow being constricted by the bridge foundations. Local scour is caused by the direct interference of the bridge foundations (abutments and piers) with the flow. The effects of ***debris rafting*** at a bridge site further magnify any erosion around the foundations and also any lateral and vertical forces on the bridge due to debris and sediment loads. At a particular bridge crossing, any or all of the different types of scour may occur simultaneously, the total scour being the sum of the individual scour components.

Three cases of bridge scour damage that have occurred within New Zealand are presented herein. In each case, the bridge and the site are described, the damage process is outlined, and any adopted remedial measures are discussed. The cases presented have been selected to highlight both the various vertical scour components occurring for bridge sites, and also a range of appropriate remedial measures for endangered bridge structures.

2 ASHBURTON RIVER ROAD BRIDGE

This road bridge over the Ashburton River is a 340 m long, two lane, reinforced concrete structure that was built in 1931. The bridge comprises 31 slab-type piers (Figure 1), about 25 of which lie within the active river channel. Each pier is founded on seven 400 mm reinforced concrete octagonal piles, these piles having been driven to a relatively uniform depth of between 6.5 m and 6.7 m below the underside of the pile caps. The original average bed level was measured at 0.2 m below the underside of the pile caps.

The Ashburton River in the vicinity of the bridge site is about 280 m wide, is straight, uniform in slope and width, and is bounded by trees and straight stopbanks. The channel is braided and there is evidence of active bed movement. The bed material is well-graded gravel. The similar amount of debris caught at each of the piers confirms that flood flows are uniform across the channel.

Over the life of the bridge, various stopbanking, river-clearing works and gravel-extraction works have taken place over extensive lengths of the river, upstream and downstream of the bridge. The extraction of gravel from the river has been and continues to be controlled.

General scour exacerbated by gravel extraction from the river upstream and downstream of the bridge has resulted in a gradual lowering of the bed level at the bridge site (Figure 1). The average bed level lowered approximately 0.9 m from 1937 to 1994, each of the piers within the river channel having exposed piles. In 1978, the average bed level was measured at a level of 1.7 m below the underside of the pile cap. The bridge at this time was concluded to be extremely vulnerable to scour damage by pier undermining arising from the progressive degradation of the bed. Concern for the stability of the bridge was accentuated by the shallowness of the foundation piles and also debris rafts forming at the site increasing the potential for local scour pier undermining and bridge damage or failure in significant floods.

Rock aprons were constructed in 1979 around each of the piers within the active channel (Figure 1). Each apron measures 5 m wide, 15 m long and 1.6 m thick. Rock riprap of a median size d_{50} of 0.5 m was used to construct the aprons, with the upper surface of each apron located beneath the riverbed surface and approximately 2 m below the base of the pile cap. Aprons act to reduce the scour potential at piers both by armouring the bed against local scour due to local hydraulic vortices, and also by protecting against general scour by dropping at the apron extremities as this scour develops. Aprons nevertheless cannot provide total assurance against scouring, particularly for ongoing general degradation. Inspection in 1994 indicated the rock apron to be exposed at one pier only. Bed levels in the river channel continue to be regularly monitored.

The bridge has to date performed satisfactorily in the large floods over its life, even with the bed lower than its present level. Possible additional future scour damage mitigation options which have been identified include underpinning of the piers and the construction of a weir immediately downstream of the bridge, with the weir crest level no lower than the underside of the pier caps. The usual form of weir adopted is a rock riprap structure comprising a crest and downstream apron. Such weirs have been used very successfully in the West Coast and Canterbury areas of New Zealand to control degrading river channels. At the Ashburton River Road Bridge site, the weir would significantly affect fish migration, riverbed levels, and stopbank effectiveness upstream of the bridge, and also bed levels immediately downstream of the weir, which would degrade.

3 BULLS ROAD BRIDGE

This two-lane structure over the Rangitikei River opened in 1949 and consisted of 17 spans of 26.8 m length, with two end spans of 20 m length. Alternate spans had a cantilever suspended span in mid-section. Each pier was a reinforced concrete slab type and was founded on two rows of six vertical 0.4 m octagonal reinforced concrete piles. The piles had been difficult to drive, anecdotal evidence suggesting that a number of piles had fractured upon being driven. The piles at Piers P and Q extended approximately 8.75-9.5 m below the base of the pile cap. The piles were founded in a shingle (gravel) surface stratum, extending to about 4.7 m below the base of the pile cap, underlain by a thin papa (mudstone) layer of about 0.9 m thickness, and then a layer of a fine black sand.

In 1947, the river was significantly braided (Figure 2), with numerous channels upstream of the bridge site. Bed levels were relatively uniform over the width of the cross-section. With gravel extraction occurring from 1949 upstream and downstream of the bridge site, the number of braided channel branches reduced to 1-2 main channels meandering within the wider channel (Figure 2). The new flow channels were deeper and narrower than the previous channels. Such developments arising from gravel extraction from a river can be forecasted based on qualitative methodologies describing channel development. River control works were developed over the period from 1949 to 1973 to protect the outsides of the meander bends upstream of the bridge. A high terrace that is relatively erosion-resistant defines the true-left bank of the wider channel. The river meanders reinforced by the bend protection works immediately upstream of the bridge resulted in the main flow channel deflecting off the true-left terrace just upstream of the bridge and passing underneath the end of the bridge at an angle of about 55^{o} to the bridge centreline in the early 1970s (Figure 2).

In conjunction with the gravel extraction, mean and minimum bed levels at the bridge site fell relatively steadily over the period from 1945 to 1972. Anecdotal evidence based on cross-section measurements indicates the existence of about 6 m depth of scour at Pier Q about one month prior to failure. With the deepening of the main flow channel under the bridge, an old timber pier was exposed immediately downstream of Pier Q in 1971-1972 (Figure 3).

On June 15, 1973, failure of the bridge occurred on the falling limb of a flood hydrograph estimated as approximating an annual flood event. The flood flows oblique to Pier Q (Figure 3) are estimated to have eroded through the thin papa layer and exposed the underlying fine river sand to the flow. The oblique angle of the flows to the slab-type pier and the increase in material erodibility would then have increased the depth and rate of development of the local scour at this pier. Flow between Pier Q and the exposed debris-enlarged old timber pier immediately downstream (Figure 3) would also have induced increased local flow velocities around Pier Q, further increasing local scour rates and depths. The reduced fixity at the base of the piles as the scour developed, combined with the flows oblique to the pier exerting lateral pressures on the exposed piles, resulted in hinging occurring both at the base of the piles and also at the underside of the pile cap for Pier Q (Figure 3). The collapse of Pier Q together with the end of the bridge deck connecting Piers P and Q also caused rotation of Pier P, with the suspended spans adjacent to Piers P and Q falling into the river (Figure 3).

Each of Piers P and Q was replaced by a single column of 1.83 m diameter supported by a cylinder of 2.13 m diameter. The respective cylinders for Piers P and Q extend to 17.1 m and 18.9 m below the level of the base of the old pile cap. Piers R, S and H to O were also underpinned by 2 cylinders of 1.7 m diameter spaced at 12.95 m centres, the cylinders extending down to 17.1 m below the level of the underside of the original pile cap. To control bed levels, strict control has also been introduced on gravel extraction. It is of interest to note that from 1992 to 1997, the main river channel passed principally between Piers J-M (Figure 2).

4 WAITANGITAONA RIVER ROAD BRIDGE

This bridge across the Waitangitaona River is a 149 m long (6 spans of 24.7 m) two-lane carriageway, which was built in 1968 to replace a bridge that needed to be raised owing to channel aggradation (resulting from a major slip upstream in the 1920s). The new bridge was supported by reinforced concrete abutments and slab piers, each pier being supported by a row of nine 380 mm square blunt-ended prestressed concrete piles, all but the central pile being raked at 1:8 (H:V) along the line of the pier. The bridge was initially designed for a scour depth of 1.8 m below minimum bed level, piles for the piers being driven to 9.3 m below minimum bed level. The channel bed is composed of surface gravels and cobbles with relatively minor amounts of sands and silts, and a substrata of medium to dense gravels.

The bridge, which spans a channel forming a sudden contraction for flood flows, is located at the exit from a wide channel containing a braided river at low flows (Figure 4). A shift and shortening in the river course downstream of the bridge resulted in rapid bed degradation at the bridge site during construction of the bridge in 1967. This degradation and later flooding resulted in piles extending to only 6.8 m below the minimum bed level. The angle of flow attack on the slab piers also increased at this time. The channel, which was relatively flat between the five central piers, was subsequently essentially stable in alignment and cross-section.

On 12 March, 1982, a flood occurred for which the peak flow was estimated to be 700 m^3/s (approximately 82% of the design flood flow of 850 m^3/s). The bridge had previously withstood a flood flow in June 1967 estimated at 700 m^3/s. Debris, including trees up to 20 m long and 1.2 m in diameter, were noted in the flow and adjacent to the piers as the flood receded, the skewed approach flows holding the debris against the sides of the exposed piles of the piers during the flood event.

For the 1982 flood, it appears that the two approach embankments concentrated flow between Piers D and F at an angle of about 30^o to the face of Pier E (Figure 4). The flood flows at a skewed angle to the pier, exacerbated by debris accumulation effects, resulted in increased lateral loading and increased scour depths for Pier E. Pier E was consequently scoured out, with the associated loss of the two adjoining deck spans (Figure 4). The remaining sections of the bridge were undamaged. Scour holes at the piers were noted to be infilled as the flood receded.

New guidebanks extending into the channel were subsequently established through the bridge site (Figure 4). A 1.5 m thick riprap layer at a slope of 2:1 (H:V), extended by a horizontal riprap toe apron 2 m thick with its upper surface at the existing bed level, was used to protect existing piers located within the new guidebanks. For these piers, the existing piles were also supplemented with three steel H-piles at each end of the pile cap, these additional piles extending to 13 m below the design bed level. The riprap utilised consisted of rocks of $d_{50} = 750$ mm. Each pier remaining in the revised stream channel was underpinned with two cylinders of 1.5 m diameter at 11 m centres, each cylinder connected to the existing pier via a 1.0 m octagonal column above the design bed level. The cylinder foundations were driven to 13 m below the bed. The existing piled foundations and mass concrete skirt for each of these piers in the revised stream channel were removed along with the base of the slab pier to 2m above the design flood levels, this level allowing some clearance to minimise any debris accumulations at the piers.

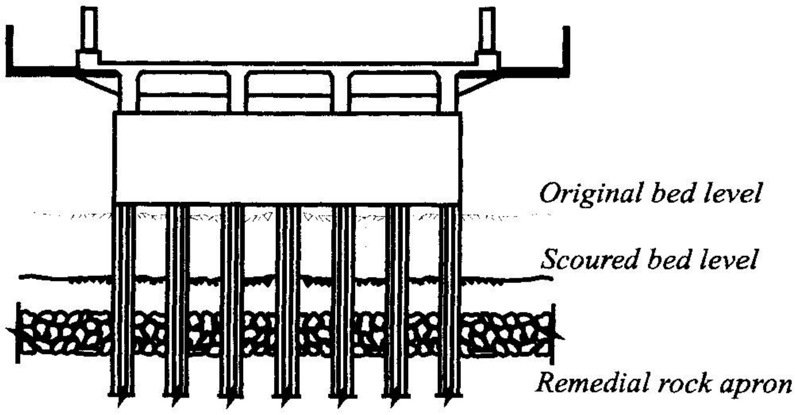

Figure 1 Typical Side Elevation of a Foundation and Remedial Works for the Ashburton River Road Bridge

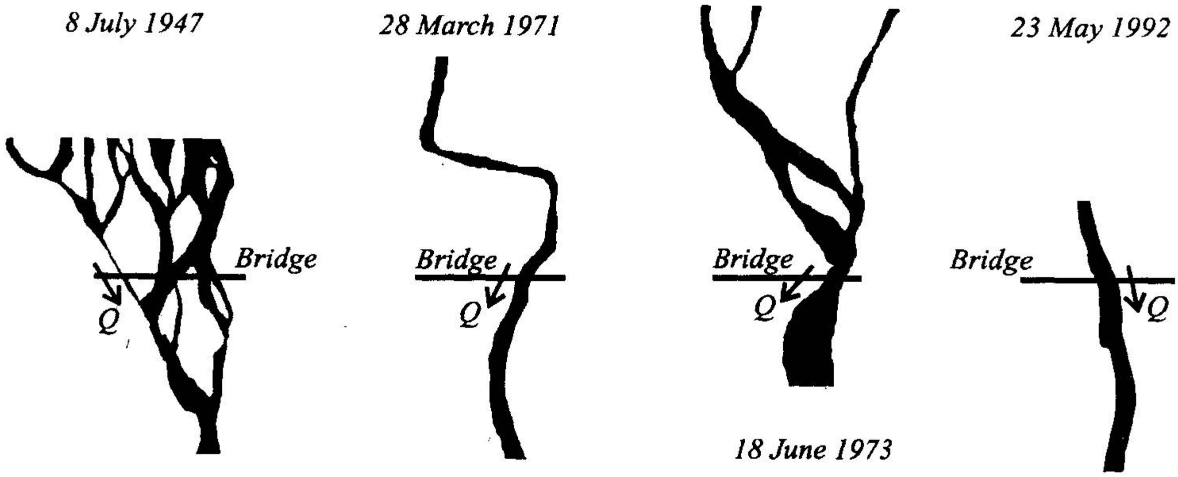

Figure 2 Development of the Principal Flow Channels of the Rangitikei River at the Bulls Road Bridge

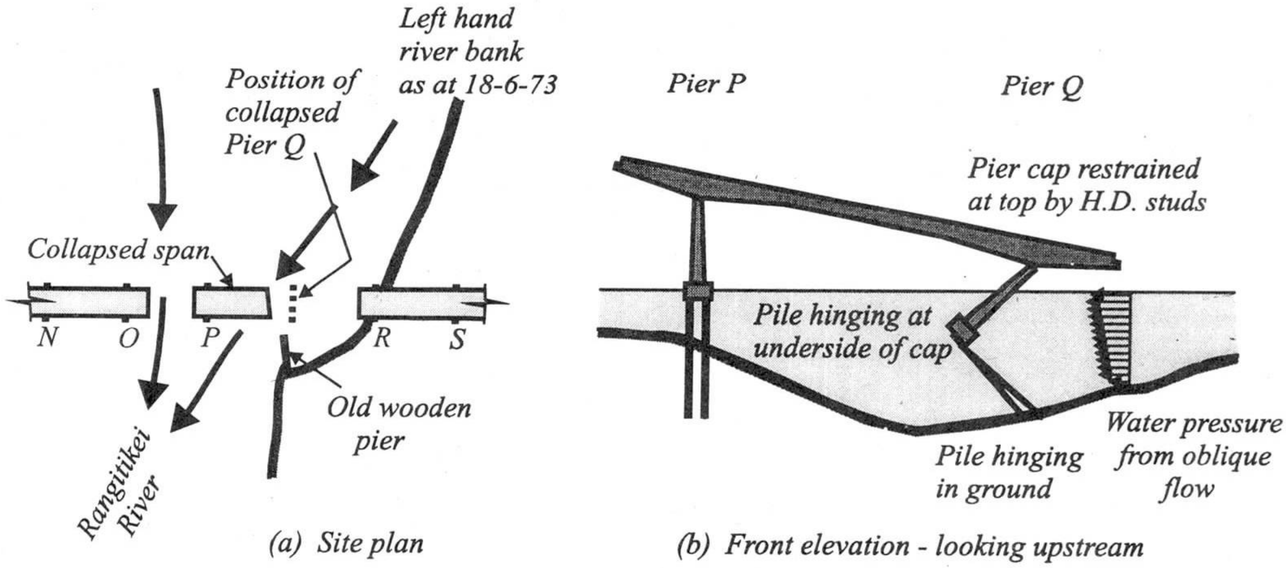

Figure 3 The Bulls Road Bridge Failure

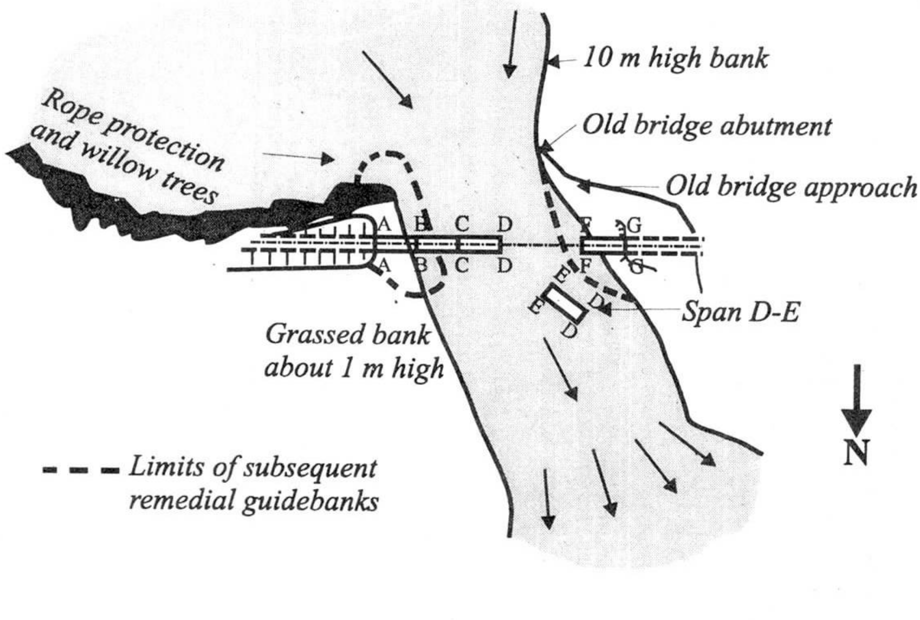

Figure 4 Schematic Site Plan of the Waitangitaona River Road Bridge Failure