Failure Mechanisms of Riprap Layer Around Bridge Piers

By

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ABSTRACT

By summarizing research efforts on using riprap as a pier countermeasure over the previous decade, the paper highlights the deficiencies of riprap in arresting pier scour. To this end, five failure mechanisms are identified. They are shear failure, winnowing failure, edge failure, bedform-induced failure and bed-degradation induced failure. Each failure mechanism can singly or, more likely, combine to cause the eventual breakdown of the riprap layer. The study shows that a riprap layer is most vulnerable when subjected to bedform-induced failure, which is dominant under a condition where large dunes are present. This failure mechanism leads to embedment of the riprap layer, rendering it ineffective in arresting scour. It is postulated that a pragmatic solution is to adopt innovative idea, such as seepage induction, to reduce the size of the migrating dunes during high flows past the riprap layer.

INTRODUCTION

Much damage to bridges at river crossings can be attributed to scouring at bridge foundations. Citing examples of 143 bridge failures dating as far back as 1847, Smith (1976) reported that 66 failures were scour related. Using data collected in the USA, Shirole and Holt (1991) stated that 60% of the 1,000 plus bridges that failed could be attributed to scour. In the widely cited bridge failure that occurred in New York and New England in 1987, 17 bridges were either damaged or destroyed by scour (Richardson and Davis 1995). Melville and Coleman (2000) reported that at least one serious bridge failure each year on the average could be attributed to scour in New Zealand. All these data point towards an important relationship between bridge failure and scour at bridge foundation.

The threat of pier scour on the integrity of a bridge has been known for many years. The classical works on bridge pier scour was that by Chabert and Engeldinger (1956) and Laursen and Toch (1956). These early works and many others published in that era are summarized in the state-of-the art paper initiated by the IAHR Section on Fluvial Hydraulics. The result of the labor of the task

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force was published in Breusers et al. (1977). It is interesting to note that in the 42-pages paper, which comprises nine sections; only one section (Section 7) with approximately two full pages was devoted to pier scour protection. Three different types of scour countermeasures were discussed therein: (1) foundation caisson; (2) additional structures placed upstream: piles; and (3) riprap mats. The concerted effort of scour research undertaken in New Zealand over the past three decades (Melville and Coleman 2000) also revealed the lack of emphasis placed on the study of scour countermeasures. Citing the thirty-seven published reports between 1975-1994 from the New Zealand efforts, it is significant to note that only one was on riprap protection on pier scour.

Despite the apparent lack of research effort on methods to reduce pier scour, engineers throughout the ages have used many ingenious methods in an attempt to arrest pier scour, thereby safe-guarding the bridge from failure during floods. Engineering methods that were devised to deal with pier scour problems are not sophisticated. They can be categorized generally into two distinctive groups: flow-altering and armoring countermeasures. The function of the first group of pier scour protection method is the provision of a means to reduce the power of the eroding agents; i.e., the downflow and horseshoe vortex, which are primarily responsible for causing scour at the pier. The flow-altering group involves one of the following three methods:

(a) Installation of some form of structures upstream of the pier (e.g., sacrificial piles or sills);
(b) Modification of pier shape; and
(c) Provision of scour reduction devices on the pier (e.g., collars, plates or slots).

The function of the second group of scour countermeasure is to provide a physical barrier against scouring. The barrier is designed to withstand the eroding power of the local flow field around the bridge pier. In practice, it is in the form of large, heavy units that are not easily eroded. Parker et al. (1998) and Melville and Coleman (2000) provide excellent descriptions of examples of these two methods used as pier scour countermeasure.

When comparing the use of these two groups of countermeasures in the field, it is clear that the latter (i.e., armoring countermeasure) is used much more extensively in both hydraulic and coastal engineering. Use of a flow-altering countermeasure is limited; even testing of such devices in the laboratory is not extensive and is exclusively conducted under clear-water conditions. On the other hand, use of an armoring countermeasure is comparatively much more prevalent, especially the use of riprap. Engels (1929) dated the possible use of riprap at bridge sites as far back as 1893. Besides using riprap as a means to reduce pier scour, there are other alternative armoring devices such as cabled-tied blocks, reno mattresses, gabion mattresses, concrete-filled mats and bags, concrete apron, dolos, tetrapods, etc. Some of these methods, such as dolos and tetrapods, are "borrowed" from coastal engineering practices. Their success as a pier scour countermeasure is as yet uncertain.

Although armoring countermeasures, especially riprap, have been used as a means to protect bridge piers against scour for more than a century, research efforts to-date inadequately examine how riprap behaves around bridge piers. Many earlier design recommendations (Neill 1973) were "loaned" from riprap practices used on riverbanks in alluvial rivers. No specific research has been conducted to explore riprap in the 3-dimensional flow field around piers. In recent years, this deficiency has begun to be addressed by growing research efforts. The main objective of this paper is to summarize knowledge gained from these recent research efforts. Highlighted herein are the deficiencies in riprap as a method for protecting bridge piers from scour. The paper also recommends further research into new and innovative ideas so as to render riprap a more reliable countermeasure in arresting scour.

FAILURE BEHAVIOR OF RIPRAP AROUND BRIDGE PIERS

The earlier studies on using riprap as a scour countermeasure at bridge piers were mainly conducted under clear-water conditions, which pertain to a stage of flow where no bedload sediment transport occurs in a channel. In other words, the undisturbed mean flow velocity is less than the critical velocity for bed sediment entrainment. Examples of some of these studies are Bonasoundas (1973), Neill (1973), Posey (1974), Hjorth (1975), Breusers et al. (1977), Richardson and Davies (1995) and Chiew (1995).

Based on a study conducted under clear-water conditions, Chiew (1995) identified three failure mechanisms that may destabilize riprap stones and lead to the breakdown of a riprap layer:

(a) Shear failure – where the riprap stones are entrained by the local flow field around the pier;
(b) Winnowing failure – where the finer underlying bed material is eroded through the voids or interstices of the coarser riprap stones under the action of turbulence and seepage; and
(c) Edge failure – where bed scouring at the periphery of the riprap layer undermines the armor stones.

Whilst the above mechanisms were originally identified as the main cause of failure for riprap layer around a bridge pier under clear-water conditions, it was subsequently confirmed (Chiew and Lim, 2000; Lauchlan and Melville, 2001) that they are also present under live-bed conditions. However, there is an additional failure mechanism associated with riprap protection that is unique in live-bed conditions where bedforms, such as dunes are present. Lim (1998) and

Chiew and Lim (2000) called it *bed feature destabilization,* and it was later referred to as *bedform undermining* by Melville and Coleman (2000). This mode of failure mechanism, which is called *bedform-induced failure* in this paper, was found to be the dominant mode of failure under conditions where large bed features are present in the flow.

All the four failure mechanisms have been observed in live-bed conditions when the undisturbed mean bed level of the river remains unchanged; i.e., in the absence of general scour of the alluvial bed. When the channel bed degrades, which can be caused by changes to the land use of the river catchment or more specifically due to direct human intervention such as excessive streambed mining and dam starvation, an additional mode of failure mechanism may destabilize the riprap. This type of failure may be called *bed-degradation induced failure*. As far as the author is aware, there have been only two studies on this failure mechanism to-date (Lauchlan and Melville 2002, and Chiew 2002). There are, however, distinct differences between the two studies and they are discussed in a latter section of the paper.

Shear Failure

Shear failure of a riprap layer around a bridge pier refers to the entrainment of the riprap stones by the local flow field. This occurrence simply means that the stones are not large or heavy enough to withstand the downflow and horseshoe vortex associated with the pier-scour mechanism for the given flow condition. Among all the five failure mechanisms outlined above, this is the easiest to conceptualize as one can clearly see the entrainment of individual stones in a laboratory experiment. For this reason, it is the most studied of all the failure mechanisms. To-date, there are a host of formulas available for use by bridge engineers seeking to find the appropriate riprap stone size. Lauchlan (1999), in her doctoral dissertation, compiled a list of these published equations and compared their performance. Some of these equations and their comparisons, which are also shown in Melville and Coleman (2000), are reproduced in Table 1 and Figure 1, respectively.

It must be pointed out that some of the 12 equations listed in Table 1 were derived semi-empirically, such as the equations proposed by Breusers and Raudkivi (1991) and Chiew (1995), whereas the others are purely empirically fitted equations, such as those by Bonasoundas (1973), Quazi and Peterson (1973) and Parola (1993). With the latter type, it is important to recognize the definition of failure - the criterion pre-determined by the researchers to assess whether a riprap layer has failed or not. In his work, Parola (1993) spelt out his definition of the critical condition, which is defined as "exposure of a portion of the painted gravel (stones in the middle of a 3-layer thick riprap layer), without removal of the painted layer over a period of about 30 minutes". It can be inferred from this definition that Parola considered the riprap layer has begun to fail when one of the stones on the first layer of his riprap is entrained within a 30-minute duration. Conversely, the definition of failure in Croad (1997) refers to the condition where the riprap layer is completely disintegrated. Which of these two definitions accurately depicts actual failure is a matter yet to be determined. There is still no consensus amongst researchers as to the precise definition of failure of the riprap layer in laboratory studies. As a result, care is needed when comparing the results obtained from these equations.

Some of the equations listed in Table 1 were not originally designed for use in a bridge pier. Farraday and Charlton's (1983) equation is in fact the equation proposed by Maynord (1978) for bank or streambed erosion. This type of equation is no different from the design curve cited in Neill (1973); both of which were not originally intended for use in pier scour condition. Additionally, the commonly cited HEC 18 equation by Richardson and Davies (1995) was in fact a modified form of the Isbash equation, which again was not formulated specifically for bridge pier application. This predicament likewise applies to the equation proposed by Breusers et al. (1977), who also used the Isbash equation. Whether one could really use these equations designed for streambed and bank erosion and apply them in a pier-scour condition is debatable. The difference between the flow field responsible for erosion in both these conditions is unmistakable, and there is really no reason whatsoever that they should be transferable.

Notwithstanding the difficulties outlined above, these equations have formed a basis from which engineers and researchers in the field of pier scour protection would continue to explore new ways to arrive at a more reliable method for sizing riprap. The designers, on the other hand, would have to be satisfied with them until such time that a better set of equation becomes available.

A comparison of these 12 equations was done by Lauchlan (1999), and is reproduced in Figure 1. Although researchers in the area of the pure sciences may be appall with the lack of consistency of the data, researchers in sediment transport area is not unfamiliar with such variations. In fact, when compared with other sediment transport predictive equations, the bulk of the equations do give reasonably consistent results.

Notwithstanding the drawbacks of the equations discussed above, engineers are not overly concerned with this form of erosion – shear failure of the riprap stone. Generally, this mechanism seems not to be the controlling failure mechanism as larger stones than are necessary can always be used to avert this failure mechanism.

Winnowing Failure

In an alluvial channel, the process of winnowing occurs when a sediment bed comprising a layer of coarse particles overlay a bed of finer sediments. It refers to the erosion of finer sediment particles through the voids between the coarser ones. The extent of winnowing is dependent on the magnitude and type of flow, and the relative size of the overlying coarse and underlying fine sediment particles. Chiew (1995) described some experiments conducted in a 2-dimensional flow where winnowing is the dominant mode of erosion. In a 3-dimensional flow field, as is the case of flow around a bridge pier, winnowing is even more conducive because of the high level of turbulence. This is especially apparent if there are less than three layers of coarse stones making up the riprap layer. Experimental observations (Chiew 1995, Chiew and Lim 2000, Lauchlan and Melville 2001) have clearly revealed the importance of winnowing on the degradation of a riprap layer. Under such a condition, the riprap stone is large enough to resist shear failure, but the underlying finer sediments were entrained through the process of winnowing. Published experimental results (Chiew 1995 in clear-water conditions; Chiew and Lim; 2000 and Lauchlan and Melville; 2001 in live-bed conditions) show the importance of winnowing failure when assessing the performance of a riprap layer. Degradation of the riprap layer can be significant even though the riprap stones themselves may remain intact. Chiew and Lim (2000) called this type of failure *embedment failure* and it can have dire consequences on the performance of the riprap layer. To this end, they have published a figure, which is reproduced in this paper as Figure 2 to demarcate embedment failure (caused primarily by winnowing failure) from total disintegration failure (caused primarily by shear failure).

Edge Failure

The third type of failure associated with riprap protection around bridge pier is edge failure. This occurs at the periphery of the coarse riprap stones and fine bed sediment interface. Chiew (1995) first documented this type of failure, and attributed it to the increase in shear stress on the finer sediment bed as water flows from the coarse to the fine bed boundary. It is not uncommon in flume experiments involving a rigid-loose bed interface. At such a location, a scour hole inevitably forms on the downstream erodible bed.

With riprap protection around a bridge pier, edge failure can occur when the coarse riprap stones remain intact; i.e., they can resist shear failure. With the intact riprap layer in place, the downstream finer sediment particles are entrained by the flow because of their lower critical shear stress. When this happens, a local depression forms, exposing the larger riprap stones. The stones at the edge thus roll or slide into the depression, as illustrated in Figure 3. Chiew (1995) observed the occurrence of edge erosion at the downstream end of the riprap layer. Such erosion takes place at low velocity ratio where shear and winnowing erosion are normally absent, and constitutes the first sign of failure of the riprap layer. With the erosion of some of the larger riprap stones due to edge erosion, shear erosion and subsequently winnowing erosion may set in. He described total disintegration or embedment of the riprap layer with the aid of a series of six photographs, beginning with edge failure at the downstream of the riprap layer. He claimed that how the riprap layer would progress beyond edge failure was very much dependent on its aerial coverage and thickness. This aspect of scour research is still not thoroughly investigated.

Edge failure was also observed at the upstream end of the bridge pier. Describing this type of occurrence, Lauchlan and Melville (2002) stated that "as the scour (at the upstream location) progresses deeper, edge stones in the riprap layer begin to experience undercutting, causing them to subside into the scour region. With the edge stones removed the inner riprap is exposed. These stones gradually subside as sand is winnowed from beneath them".

Edge failure, whether it occurs at the upstream or downstream end of the bridge pier, serves as an important function in weakening the riprap layer. By itself, edge failure will not render the demise of the riprap layer. Its role is indirect in that it -

(a) Exposes the larger stones in the coarse-fine sediment interface, rendering it more susceptible to erosion; and
(b) Reduces the thickness of the riprap layer, thereby promoting winnowing failure.

In summary, edge failure hastens the eventual demise of the riprap layer as it enhances either winnowing or shear failure. The former leads to embedment while the latter enhances disintegration of the riprap layer.

Bedform-Induced Failure

Shear failure, winnowing failure and edge failure occur under both clear-water and live-bed conditions. Bedform-induced failure, however, only occurs in live-bed conditions. It is especially prevalent where large dunes are present on the approaching bed. It is the dominant failure mode and can easily overshadow the other three types of failure.

Lim and Chiew (1997) first described how the propagating bedforms interact with the riprap stones. The mechanism of bedform-induced failure was described in more detail by Chiew and Lim (2000) in which they stated that "bed-feature destabilization (or bedform-induced failure) is prompted by the fluctuation of the bed level due to the propagation of bed features (ripples and dunes) past the pier. The fluctuating bed level causes the riprap stones to lose support, and therefore stability. When the troughs of these dunes arrive at the riprap layer, high level of turbulence is generated at the reattached zone of the flow over the dune crest. This combines with the flow field that formed at the pier to pluck and erode additional riprap stones from the degraded riprap layer. Once these stones are eroded, the degraded layer becomes thinner and is further loosened, which gives an impetus for winnowing, resulting in the embedment of the riprap layer".

Chiew and Lim (2000) have also devised an experiment to distinguish bedform-induced failure from the other three types of failure. It includes two tests which were both conducted under transition flat bed conditions at a velocity ratio, $U/U_c = 4.5$. All the other conditions for both the tests were identical except that in the first test, the flow velocity was increased as quickly as practicable so that transition flat bed was formed immediately after the onset of the test. The objective of such a procedure was to avoid the formation of ripples or dunes on the approaching bed so as to negate any bedform-induced failure at the riprap layer before the test was conducted. With the second test, velocity was increased incrementally to allow ripples and dunes to form prior to the final test with $U/U_c = 4.5$.

Figure 4 shows the results of the two tests, and it depicts the temporal scour depth fluctuation at $U/U_c = 4.5$ where transition flat bed was present on the approaching bed. The horizontal lines in Figures 4(a) and 4(b) represent the depth of the riprap layer – the level at which the riprap layer has degraded. The results show that the levels of riprap degradation, d_{rp} for Test 1 and 2 are 133 mm and 176 mm, respectively. In Test 1, the scour depth expectedly fluctuates with time and it is due to avalanches of the sediment particles on the upstream slope of the scour hole. It shows that the riprap layer has clearly degraded to the same level as the maximum scour depth. On the other hand, d_{rp} associated with Test 2 is deeper than the observed maximum scour depth at $U/U_c = 4.5$. The reason for the deeper d_{rp} is because of the antecedent condition where ripples and dunes were allowed to form before the later test was conducted. The differential of 43 mm (difference between 176 and 133 mm) is caused by bedform-induced failure.

The importance of bedform-induced failure at riprap layers is also highlighted in Parker et al. (1998) and Lauchlan and Melville (2001). In the latter study, the authors concluded that destabilization of a riprap layer by the progression of bedforms past the pier is the dominant failure mode under live-bed conditions, whereas shear, winnowing and edge failures play a secondary role.

Research studies published to-date are unanimous that bedform-induced failure is of prime importance in affecting riprap layer around bridge piers. This failure mechanism highlights the reality that, when considering riprap stability around bridge pier, it is just not sufficient to consider the ability of the stone itself to resist erosion; i.e., shear failure alone, as was done by early researchers when considering riprap stability under clear-water conditions. In a nutshell, the stability of a riprap stone depends not only on its ability to resist shear, but also on the firmness of its foundation.

Bed-Degradation Induced Failure

The fifth and final type of failure affecting riprap failure at bridge piers is bed-degradation induced failure. Unlike the other four, this type of failure will only occur under a condition when general scour is present. Under this condition, there is a net loss of sediment over the reach of the alluvial channel at which the bridge is founded.

Lauchlan and Melville (2002) used a specially designed flume to investigate this failure mechanism. It consists of a sediment recess in which a vertically adjustable table was installed. This set-up was originally used by Chin (1985) to study streambed armoring. A unique feature of the set-up is that the shear stress of the sediment bed could be kept constant throughout the experiment. This is because the adjustable table allows the sediment bed to be raised such that it is subjected to a constant shear stress. While such a set-up reduces the effort in conducting the tests immensely, because no sediment needs to be placed upstream of the test section, it does not replicate the actual field condition on two counts. First, the shear stress in a degrading channel under a steady flow condition is not a constant but decreases with time. At the limiting condition, the shear stress approaches the critical shear stress of the bed sediment, at which time bed degradation ceases. At this juncture, the riprap layer is exposed to a much lower shear stress than that simulated in the test conducted by Lauchlan and Melville (2002). Second, bedform-induced failure is altogether absent in their study because the upstream bed in their test is rigid and no approaching bedform is present.

Chiew (2002) did not have the benefit of an adjustable table in his flume, and had to resort to a much more labor-intensive study. Even with the help of (and thanks to) research students, only three sets of experiments were conducted in an 8-months period. The study is reported in a separate paper in this conference and only the essence is presented here. At the commencement of his study, the bed sediment was eroded causing general scour. At the same time, edge and to a limited extent, shear and winnowing failure were observed at the riprap layer. With time, dunes were formed on the approaching bed. When they approached the bridge pier, a certain degree of bed degradation was observed as bedform-induced failure set in. This, however, took place at the same time that the entire bed was degrading, resulting in a reduction of the approach shear stress. With time, two important features resulted:

(1) The approach flow shear stress reduces so much that the riprap stones become effectively more stable as compared with those before general scour has taken place; and
(2) The decreasing mobility of the bed sediment reduces the size of the migrating bedforms. At the limiting condition were no inflow of sediment particles was provided, an immobile plane bed was formed.

The cumulative effect of these two features is the formation of a stable mound consisting of riprap stones around the bridge pier, as is shown in Figure 5. Chiew (2002) further showed that the riprap mound might be susceptible to failure when it is subjected to another flood accompanied by large dunes.

The observations of Chiew (2002) are very different from that reported in Lauchlan and Melville (2002). In the latter's study, the riprap mound was not observed, primarily because shear stress was kept constant throughout their experiment. As the bed (artificially) degraded with the raising bed, the riprap stones increasingly protruded above the general bed level, causing them to have a reduced critical shear stress (Chin and Chiew 1993). The result is that they eventually are entrained, or subjected to shear failure. Based on such an observation, Lauchlan and Melville (2002) concluded that "riprap installed as local scour protection around bridge piers is inadequate for protecting the pier in rivers where significant degradation of the channel bed is likely to occur. The overall degradation of the sediment bed causes the riprap stones to subside into the degrading bed. After substantial degradation the riprap layer disintegrates and can no longer prevent local scouring around the pier". This conclusion may not be so in the field if the shear stress is reduced as a consequence of bed degradation.

Notwithstanding this concern, general scour does cause riprap stones (in the form of a mound) to protrude above the bed level, resulting in a reduced critical shear stress needed to entrain the stones. If the stones are subjected to subsequent floods, especially with large approaching dunes, they are very likely to be entrained, causing the riprap mound to disintegrate.

In summary, bed-degradation induced failure constitutes another facet of riprap failure at bridge piers. Under a steady flow condition with an unchanged downstream control point, general bed degradation results in a reduction in bed shear stress, causing an apparent stable riprap mound to form. This gives a subtle impression that the riprap layer, albeit having a modified form, can function as planned in arresting scour. Research shows that subsequently floods, especially if large dunes accompany them, can cause total disintegration of the riprap mound.

FUTURE STUDY

Arising from the knowledge gained from research studies conducted over the last 10 years, we are now in a better position to assess how a riprap layer around bridge piers fails. The early emphasis on searching for an equation to size riprap stones for a particular application had only addressed the issue on shear failure. While an appropriately sized stone is very important in riprap design, the question on coverage and thickness of the riprap layer is still not well understood. A carefully planned research study in this area is still lacking. Moreover, the other four failure mechanisms outlined in this paper are not overcome merely through the use of large stones. Lim and Chiew (2001) present a series of experimental results, showing that riprap layers with different geometrical properties would fail if subjected to high flows accompanied by large dunes.

Published results so far appear to suggest that the critical factor in augmenting riprap failure is large dunes that accompany high flows. Dunes have a twofold influences:

(a) Reattachment of the main flow over the dune crest as it attacks the riprap layer; and
(b) Undermining of the riprap layer as the dune troughs propagate past the pier.

To a large extent, both these influences can be negated if one were able to reduce their height as dunes migrate past the bridge pier. One way is to explore innovative methods to reduce the size of propagating bedforms past the pier. With this, it is surmise that the threat of bedform-induced failure of the riprap layer can be significantly reduced, and thereby increasing the reliability of the riprap as a pier-scour countermeasure.

To this end, the present writer has conducted some very preliminary studies on seepage effect on dune geometry. These data are encouraging in that the dune geometry does appear to change as bedforms translate past a region subjected to upward seepage or suction. Previous studies on both seepage and suction (Cheng and Chiew, 1998; 1999 and Chen and Chiew, 2001) have shown a close correlation between such flows on the turbulence characteristics of the main flow field. This, in turn, affects the sediment transport behavior of the river. It, therefore, would not be inappropriate to surmise that either seepage or suction could have a positive influence in reducing the size of the dunes. Figure 6 shows the preliminary results of the effect of relative seepage velocity, v_s/U on the dune steepness, h/λ, where v_s = seepage velocity in which a positive v_s denotes upward seepage and vice-versa; U = undisturbed mean flow velocity; h = dune height; and λ = wavelength of dune. These data appear to show that the dune steepness increases and decreases with suction and upward seepage, respectively. If we are able to exploit this, the problem engineers faced in designing for an effective riprap as a pier-scour countermeasure may be overcome. At this juncture, it is by no means clear whether the seepage idea would lead to a realistic and practical solution to overcoming bedform-induced failure. Additional research is necessary to explore its applicability.

CONCLUSIONS

A riprap layer around bridge piers is subjected to five failure mechanisms. They are shear failure, winnowing failure, edge failure, bedform-induced failure and bed-degradation induced failure. Each of these failure mechanisms plays a role in causing the eventual breakdown of the riprap layer. The first three failures occur in both clear-water and live-bed conditions, whereas the latter two only occur in live-bed conditions. Moreover, bed-degradation induced failure will only take place in a river where general scour occurs. The other four failure mechanisms may occur in either a non-degrading or degrading channel.

It is important to note that a riprap layer may fail either in the form of embedment or total disintegration. The former mode is closely related to winnowing and bedform-induced failure whereas the latter is cause by shear failure. Whether a riprap layer will embed or totally disintegrate depends not only on the size of the riprap layer, but also on its aerial coverage and thickness. The latter two parameters are as yet not adequately researched on. Under a live-bed condition where large dunes are present, embedment failure is the principal mode of failure because the dune trough has the tendency to undermine the riprap layer. Edge failure serves to initiate failure especially under conditions of comparatively low flow velocity. An interesting failure mechanism is bed-degradation induced failure, which can cause the formation of a riprap mound around the bridge pier. When subjected to subsequent floods with large dunes, it may also fail.

Recent studies show the vulnerability of a riprap layer when subject to high flows accompanied by large dunes. Bedform-induced failure can be so dominant that it readily causes riprap layer embedment, rendering it completely ineffective in arresting pier scour. It is postulated that a solution to increase riprap reliability is to reduce the size of the propagating bedforms. Innovative ideas, such as using seepage to reduce dune geometry, are necessary to effect a pragmatic method that can curtail bedform-induced failure so that a more reliably riprap layer can be built.

REFERENCES

1. Bonasoundas, M., 1973, “Flow Structures and problems at Circular Bridge Piers”, Report No. 28, Oskar V. Miller Inst., Tech. Univ. Munich, Germany.
2. Breusers, H. N. C., Nicollet, G., and Shen, H. W., 1977, “Local Scour around Cylindrical Piers”, Journal of Hydraulic Research, IAHR, Vol. 15, No. 3, pp. 211-252.
3. Breusers, H. N. C., and Raudkivi, A. J., 1991, “Scouring”, IAHR Hydraulic Structures Design Manual 2, A. A. Balkema, Rotterdam, The Netherlands.
4. Chabert, J. and Engeldinger, P., 1956, “Etude des Affouillements autour des Piles des Ponts”, *Laboratoire National d’Hydraulique*, Chatou, France.
5. Chen, X., and Chiew, Y. M., 2001, "Bed Shear Stress in Open Channel Flow with Bed Suction" Proceedings of the XXIX IAHR Congress, Theme D, Vol. 1, Beijing, pp. 276-281.
6. Cheng, N. S., and Chiew, Y. M., 1998, “Turbulent Open-Channel Flow with Upward Seepage” Journal of Hydraulic Research, IAHR, Vol. 36, No. 3, pp. 415-431.
7. Cheng, N. S., and Chiew, Y. M., 1999, “Incipient Sediment Motion with Upward Seepage" Journal of Hydraulic Research, IAHR, Vol. 37, No. 5, pp. 665-681.
8. Chiew, Y. M., 1995, "Mechanics of Riprap Failure at Bridge Piers", Journal of Hydraulic Engineering, ASCE, Vol. 121, No. 9, pp. 635-643.
9. Chiew, Y. M., 2002, “Riprap Protection around Bridge Piers in a Degrading Channel”, to be published in Proc. First International Conference on Scour of Foundations ICSF-1, College Station, Texas, November 17-20, 2002.
10. Chiew, Y. M., and Lim, F. H., 2000, "Failure Behavior of Riprap Layer at Bridge Piers under Live-Bed Conditions", Journal of Hydraulic Engineering, ASCE, Vol. 126, No. 1, pp. 43-55.
11. Chin, C. O., 1985, “Stream Bed Armouring”, Report No. 403, School of Engineering, University of Auckland, Auckland, New Zealand.
12. Chin, C. O., and Chiew, Y. M., 1993, "Effect of Bed Surface Structures on Particle Stability" Journal of Waterway, Port, Coastal, and Ocean Engineering, ASCE, vol. 119, no. 3, pp. 231-242.
13. Croad, R. N., 1997, “Protection from Scour of Bridge Piers using Riprap”, Transit New Zealand Research Report No. PR3-0071, Transit NZ, Wellington, New Zealand, 77pp.
14. Engels, H., 1929, “Experiments Pertaining to the Protection of Bridge Piers against Undermining”, Hydraulics Laboratory Practice, Chapter V, ASME, N.Y., USA.
15. Farraday, R. V., and Charlton, F. G., 1983, “Hydraulic Factors in Bridge Design”, Hydraulic Research Station, Wallingford, England, 102pp.

16. Hjorth, P., 1975, “Studies on the Nature of Local Scour”, Bulletin, Series A, No. 46, Department of water Resources Engineering, Lund Institute of Technology, University of Lund, 191pp.
17. Lauchlan, C. S., 1999, “Countermeasure for Pier Scour”, pH.D Thesis, The University of Auckland, Auckland, New Zealand.
18. Lauchlan, C. S., and Melville, B. W., 2001, “Riprap Protection at Bridge Piers”, Journal of Hydraulic Engineering, ASCE, Vol. 127, No. 5, pp. 412-418.
19. Lauchlan, C. S., and Melville, B. W., 2002, “Effect of Bed Degradation on Riprap Protection at Bridge Piers”, Proceedings of the Institution of Civil Engineers, Water and Maritime Engineering, Vol. 154, Issue 1, pp. 19-27.
20. Laursen, E. M., and Toch, A., 1956, “Scour around Bridge Piers and Abutments”, Bulletin No. 4, Iowa Highway Research Board.
21. Lim, F. H., 1998, “Riprap Protection and Its Failure Mechanisms”, A thesis submitted to the School of Civil and Structural Engineering, Nanyang Technological University, Singapore in fulfillment of the requirement for the degree of Doctor of Philosophy.
22. Lim, F. H. and Chiew, Y. M., 1997, “Failure Behavior of Riprap Layer around Bridge Pier”, Proceedings of the XXVII IAHR Congress, Theme A - Managing Water: Coping with Scarcity and Abundance, San Francisco, pp. 184-189.
23. Lim, F. H. and Chiew, Y. M., 2001, “Parametric Study of Riprap Failure around Bridge Piers”, Journal of Hydraulic Research, IAHR, Vol. 39, No. 1, pp. 61-72.
24. Maynord, S. T., 1978, “Practical Riprap Design”, Misc. Paper H-78-7, U.S. Army Engineer Waterway Experiment Station, Vicksburg, Mississippi.
25. Melville, B. W., and Coleman, S. E., 2000, “Bridge Scour”, Water Resources Publication, LLC, CO., USA.
26. Neill, C. R., 1973, “Guide to Bridge Hydraulics”, Roads and Transportation Association of Canada, University of Toronto Press, Toronto, Canada, 191pp.
27. Parker, G., Toro-Escobar, C., and Voigt, R. L., 1998, "Countermeasures to Protect Bridge Piers from Scour." Final Report prepared for National Highway Research Program, Transportation Research Board, National Research Council, University of Minnesota, Minneapolis, December, 360pp.
28. Parola, A. C., 1993, “Stability of Riprap at Bridge Piers”, Journal of Hydraulic Engineering, ASCE, Vol. 119, No. 10, pp. 1080-1093.
29. Posey, C. J., 1974, “Tests of Scour Protection for Bridge Piers”, Journal of Hydraulic Division, ASCE, Vol. 100, No. 12, pp. 1773-1783.
30. Quasi, M. E., and Peterson, A. W., 1973, “A Method of Bridge Pier Riprap Design”, Proceedings, First Canadian Hydraulic Conferences, Edmonton, Canada, pp. 96-106.

31. Richardson, E. V., and Davies, S. R., 1995, "Evaluating Scour at Bridges", 3rd Edition, HEC 18, U. S. Department of Transportation, Federal Highway Administration.
32. Shirole, A. M., and Holt, R. C., 1991, “Planning for a Comprehensive Bridge Safety Assurance Program”, Transp. Res. Rec. No. 1290, Transportation Research Board, Washington D.C., 39-50.
33. Smith, D. F., 1976, “Bridge Failures”, Proceedings of the Institution of Civil Engineers, Vol. 60, No. 1, pp. 367-383.

Table 1. Equation for sizing riprap at bridge piers (Melville and Coleman 2000)

Reference	Original Equation	Equation used for Comparison	Symbols
Bonasoundas (1973)	$d_r(cm) = 6 - 3.3V + 4V^2$		Apply to stones with $S_s = 2.65$; V = mean approach velocity (m/s)
Quazi and Peterson (1973)	$N_{sc} = 1.14\left(\frac{d_r}{y}\right)^{-0.2}$	$\frac{d_r}{y} = \frac{0.85}{(S_s-1)^{1.25}} Fr^{2.5}$	N_{sc} = critical Stability Number = $V^2/[g(S_s-1)d_r]$; Fr = Froude number = $V/(gy)^{0.5}$
Breusers et al. (1977)	$V = 0.42\sqrt{2g(S_s-1)d_r}$	$\frac{d_r}{y} = \frac{2.83}{(S_s-1)} Fr^2$	y = undisturbed approach flow depth
Farraday and Charlton[2] (1983)	$\frac{d_r}{y} = CFr^3$	$\frac{d_r}{y} = 0.547Fr^3$	C = coefficient and is dependent on the bank slope and the factor of safety, SF, selected. For a flat slope, C = 0.22, 0.25 and 0.28 for SF = 1, 1.5 and 2, respectively. Moreover, there is a velocity multiplier depending on whether the pier is sited in a straight channel (1.25); bends (1.5) or nose of groins or guide banks (2.0).
Parola and Jones (1989)	$\frac{d_r}{y} = \frac{C^*}{(S_s-1)} Fr^2$	$\frac{d_r}{y} = \frac{C^*}{(S_s-1)} Fr^2$	C^* = coefficient for pier shape = 1.0 (rectangular), 0.61 (round-nose)
Breusers and Raudkivi (1991)	$V = 4.8\sqrt{S_s - 1}d_r^{1/3} y^{1/6}$	$\frac{d_r}{y} = \frac{0.278}{(S_s-1)^{1.5}} Fr^3$	
Austroads (1994)	$\frac{d_r}{y} = \frac{0.58K_pK_v}{(S_s-1)} Fr^2$	$\frac{d_r}{y} = \frac{0.58K_pK_v}{(S_s-1)} Fr^2$	K_p = pier shape factor = 2.25 (round nose), 2.89 (rectangular) K_v = velocity factor, varying from 0.81 for a pier near the bank of a straight channel to 2.89 for a pier at the outside bend in the main channel

[2] For the formula proposed in Faraday and Charlton (1983), Melville and Coleman (2000) used a coefficient of 0.547. This is obtained by assuming a flat bed and a factor of safety of 2.0, yielding C = 0.28. They assume the pier is sited on a straight river reach, thus giving a velocity multiplier of 1.25. As a result of this the coefficient becomes $0.28*1.25^3 = 0.547$.

Reference	Original Equation	Equation used for Comparison	Symbols
Richardson and Davies (1995)	$d_r = \frac{0.692(f_1 f_2 V)^2}{(S_s - 1)2g}$	$\frac{d_r}{y} = \frac{0.346 f_1^2 f_2^2}{(S_s - 1)} Fr^2$	f_1 = pier shape factor = 1.5 (round nose), 1.7 (rectangular) f_2 = factor ranging from 0.9 for a pier near the bank of a straight reach to 1.7 for a pier in the main current at a bend
Chiew (1995)	$d_r = \frac{0.168}{\sqrt{y}} \left(\frac{V}{U_* \sqrt{(S_s - 1)g}} \right)^3$	$\frac{d_r}{y} = \frac{0.168}{(S_s - 1)^{1.5} U_*^3} Fr^3 \; U_* = \frac{0.3}{K_d K_y}$ K_d = sediment size factor; K_y = flow depth factor	$K_y = 0.783 \left(\frac{y}{b} \right)^{0.322} - 0.106 \quad 0 \le \frac{y}{b} < 3$ $K_y = 1 \quad \frac{y}{b} \ge 3$ $K_d = 0.398 \ln\left(\frac{b}{d_r} \right) - 0.034 \left[\ln\left(\frac{b}{d_r} \right) \right]^2 \quad 1 \le \frac{b}{d_r} < 50$ $K_d = 1 \quad \frac{b}{d_r} \ge 50$
Parola (1993)	Rectangular: $N_{sc} = 0.8 \quad 20 < \frac{b}{d_r} < 33$ $N_{sc} = 1.0 \quad 7 < \frac{b}{d_r} < 14$ $N_{sc} = 1.2 \quad 4 < \frac{b}{d_r} < 7$ *Aligned Round-nose:* $N_{sc} = 1.4$	$\frac{d_r}{y} = \frac{f_1 f_3}{(S_s - 1)} Fr^2$	f_1 = pier shape factor = 1.0 (rectangular), 0.71 (round-nose if aligned) f_3 = pier size factor $f_3 = 0.83 \quad 4 < \frac{b}{d_r} < 7$ $f_3 = 1.0 \quad 7 < \frac{b}{d_r} < 14$ $f_3 = 1.25 \quad 20 < \frac{b}{d_r} < 33$

Reference	Original Equation	Equation used for Comparison	Symbols
Croad (1997)	$\frac{V}{A\sqrt{(S_s-1)gd_r}} = 1.16\left(\frac{y}{d_r}-2\right)^{1/6}$ $d_r = 17\ d_{50}$	$\frac{d_r}{y}\sqrt{\left(1-\frac{2d_r}{y}\right)} = \frac{0.641}{A^3(S_s-1)^{1.5}}Fr^3$ *use larger of computed d_r or 17 d_{50}*	A = acceleration factor = 0.45 (circular and slab piers), = 0.35 (square and sharp edge piers). Equation given for factor of safety = 1.25.
Lauchlan (1999)	$\frac{d_r}{y} = 0.3SF\left(1-\frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	$\frac{d_r}{y} = 0.3SF\left(1-\frac{Y_r}{y}\right)^{2.75} Fr^{1.2}$	SF = factor of safety, with a minimum recommended value = 1.1 Y_r = placement depth below bed level

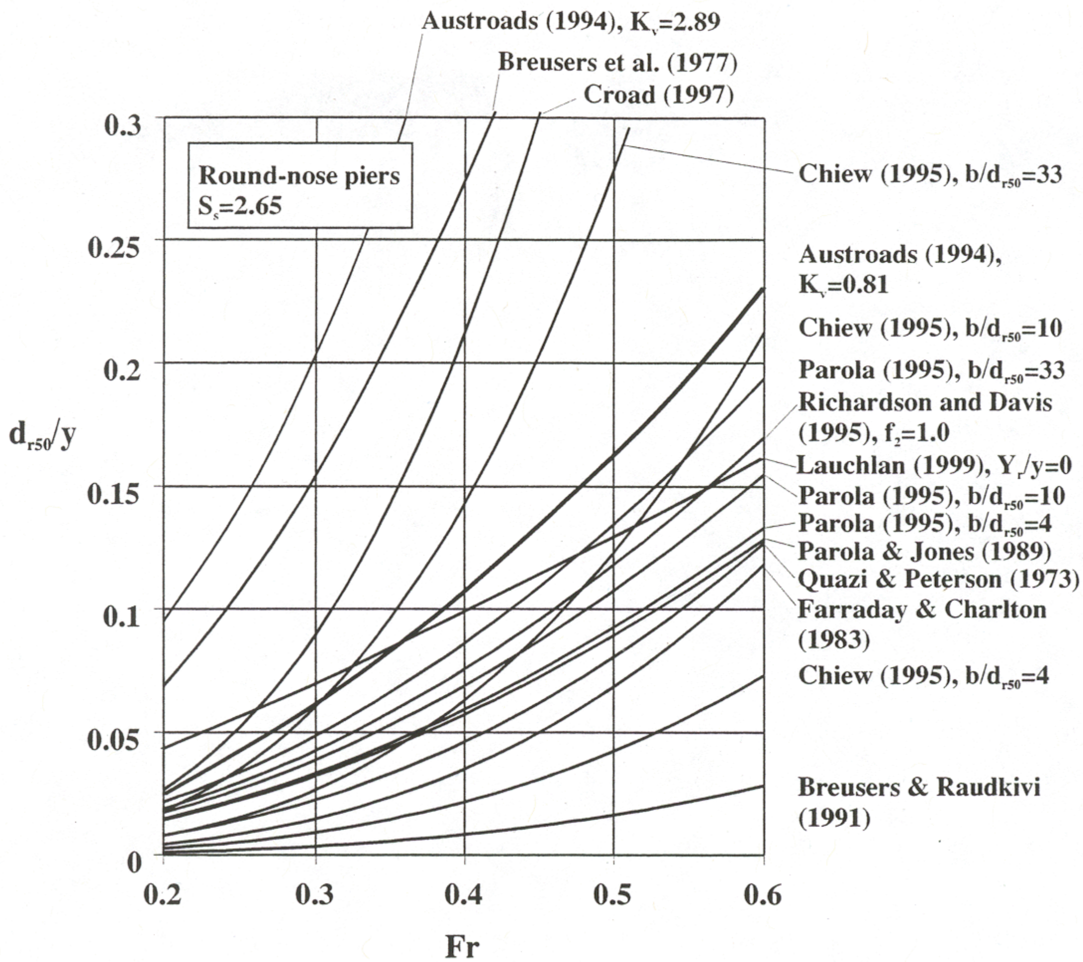

Figure 1. Comparison of Equations for Sizing Riprap at Bridge Piers
(Melville and Coleman 2000)

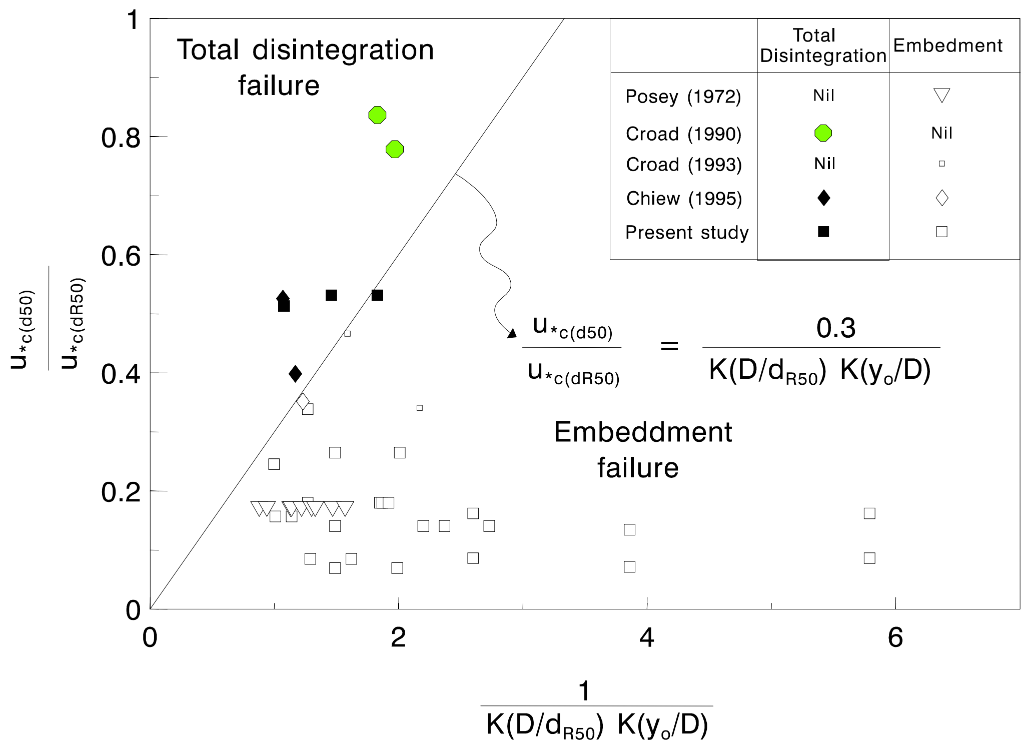

Figure 2. Classification of Failure Mode of riprap Layer (Chiew and Lim 2000)

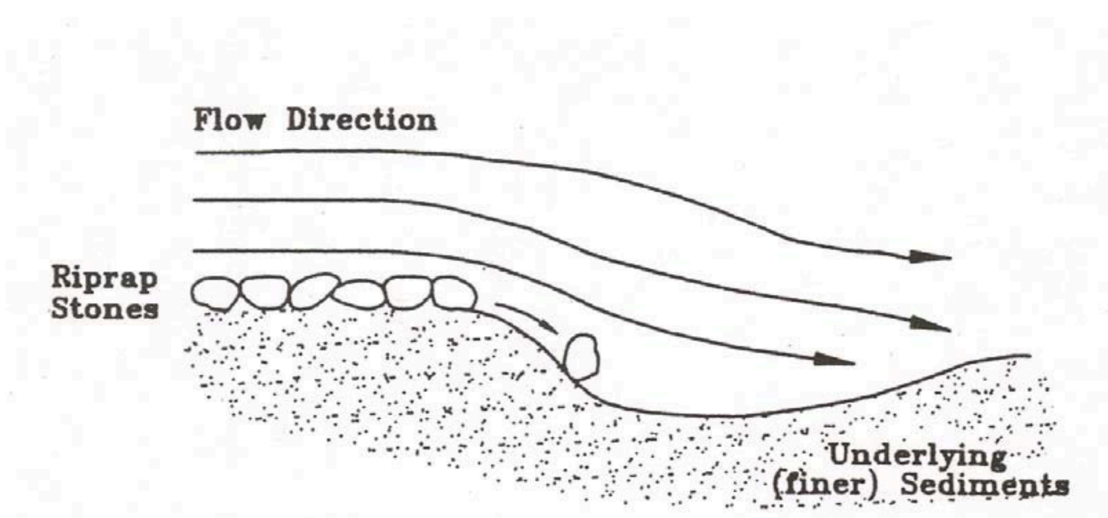

Figure 3. Schematic Illustration of Edge Failure (Chiew 1995)

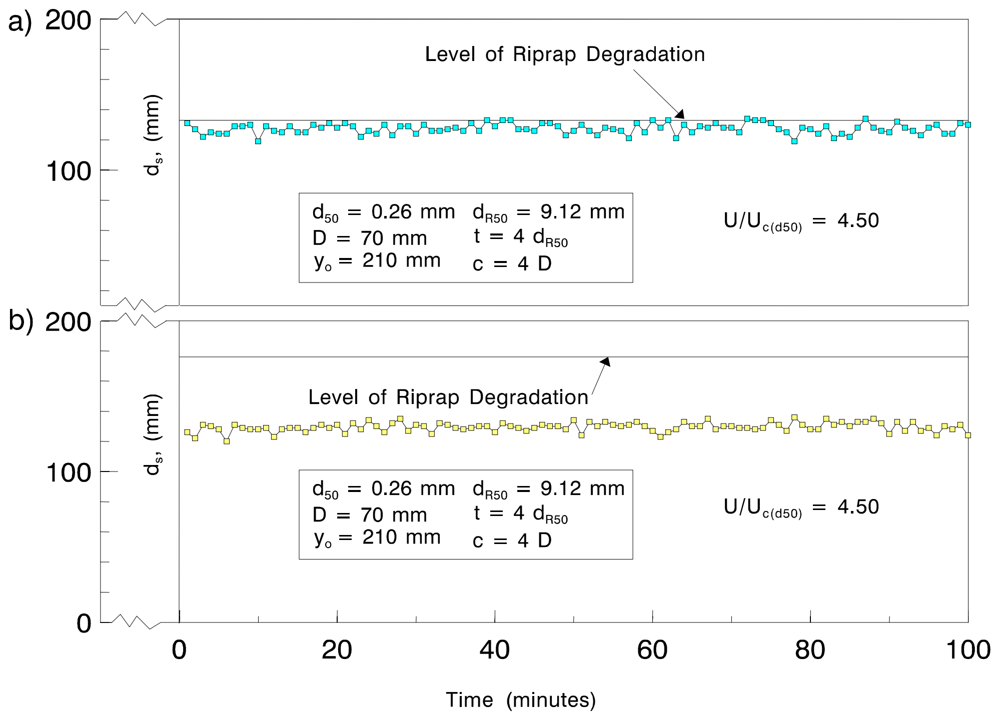

Figure 4. Temporal Variation of Scour Depth in Transition Flat Bed Regime with Different Antecedent Bedforms: (a) Immobile Plane Bed; (b) Immobile Plane Bed, Ripples and Dunes (Chiew and Lim 2000)

Figure 5. Formation of riprap mound around bridge pier in a degrading bed (Flow from right to left)

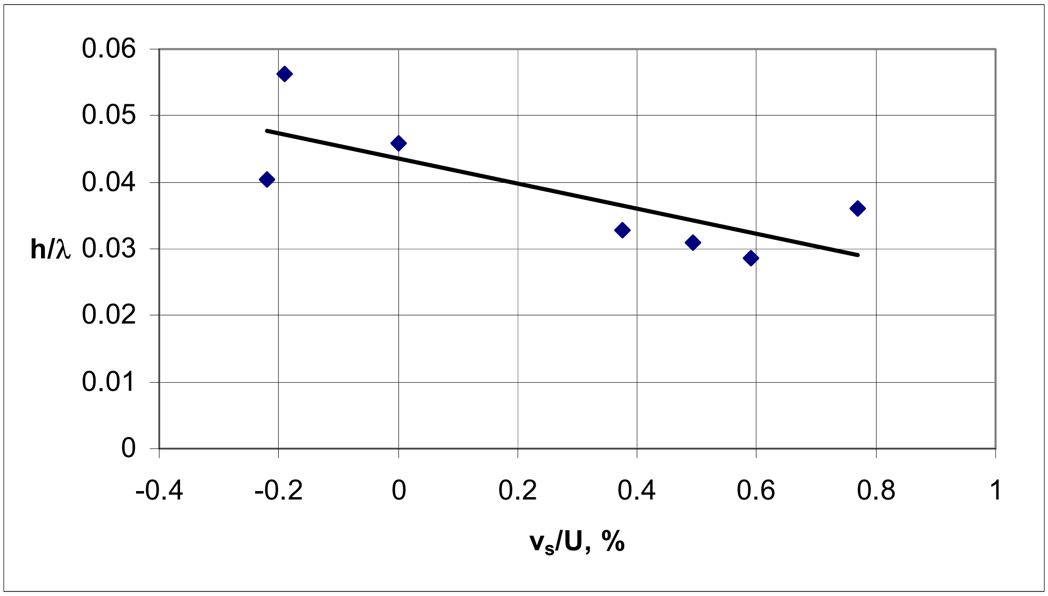

Figure 6. Effect of relative seepage velocity on dune steepness