State-of-the-art of Pier Scour Prediction for Design Application

Choi, Gye Woon* / Ahn, Sang Jin** / Kang, Kwan Won***

ABSTRACT/ Scour at bridge pier is a complicated three-dimensional problem involving interaction of fluid force on movable aid nonuniformly distributed sand grains. Although several analytical solution approaches, experimental research and field investigations for scour at piers have been conducted, no comprehensive and universally acceptable solution is so far available. Even though many methods and equations for predicting scour at piers are available in the literature, hydraulic and/or bridge design engineers are often at a loss over which method or equation is applicable for the specific bridge sites. To provide better understanding about scour phenomena and better predicting of scour at piers, intensive research is conducted through comprehensive review of published literature. Based on the research the state-of-the-art of pier scour prediction for design application is provided as a design guide for practicing engineers in this field. Recommendations for applying aggradation and degradation, contraction scour, and local scour prediction methods or equations are suggested. It is hoped that this paper may provide good information for the prediction of scour at piers.

1. Introduction

Construction of bridge crossings in river channels could cause significant global and local change in the geomorphology of the river systems. Generally, the geomorphological features of river channels are closely related to hydraulics responses in river systems. To hydraulic and/or bridge design engineers proper predictions of these hydraulic and geomorphological responses are essential for the design of suitable bridge crossings. These responses can be indicated as short-term and long-term changes of the rivers and their tributaries. These responses include the impact on environmental factors, the aesthetic changes of the river environment, and scour effects at the bridge crossings. Especially, the scour effects, mainly on bridge piers and abutments, have been the major research topic of many investigators

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* Research Associate, Eng. Res. Center, CSU, U. S. A.

** Prof. Dept. of Civil Eng. Chungbuk National Univ.

*** Prof. Dept. of Civil Eng. Inha Univ.

throughout the world. Major research efforts have been directed to the studies of scour mechanisms influence of scouring parameters, and proper ways to predict maximum or mechanisms influence of scouring parameters, and proper ways to predict maximum or equilibrium scour depths. In reality, even though numerous investigations through analytical approaches, experimental research and field data analysis have been conducted, no comprehensive and universally accepted prediction techniques are so far available because of the complex hydraulic and geomorphologic phenomena in the rivers. From the theoretical view point the scour, especially scour at piers, is a complicated three dimensional problem involving interaction of fluid forces on movable, cohesionless aid nonuniformly distributed sand grains.

Although there are no comprehensive and universally accepted solutions for scour prediction, reasonably accurate prediction of maximum or equilibrium scour depth is still important to hydraulic or bridge design engineers for proper financial and safety considerations. At the present time more accurate scour prediction can only be achieved through better understanding of scour phenomena for the specific design sites and selection of appropriate prediction method for the situation. Inaccurate predictions of scour depth could cause even bridge failures. Bridge failures could result in traffic delays, losses of lives as well as financial losses to recover the bridges. In fact, many bridge failures recorded in the past are attributed directly to the undermining of piers or abutments by scour(Jarrett and Boyle, 1986).

As a guidance for predicting scour at piers for hydraulic and bridge design engineers, the state-of-the-art for scour prediction at piers is presented through extensive literature review which covers over 50 publications related to the scour at bridge piers. This paper comprises the general descriptions about scour at piers and recommendations for the selection of proper scour prediction methods. The general descriptions include classification of scours , scouring mechanisms, and effects of scouring parameters, etc. Because of the lack of general agreement among the researchers on maximum or equilibrium scour depth prediction methods, it is not a simple task to select appropriate scour prediction methods for the specific bridge sites.

2. Scour Classifications

Scour can be defined as the enlargement of a channel cross-section through the removal of materials composing the channel boundary due to fluid forces. This phenomenon comprises the erosive actions of running water, excavation of channel boundary and transportation of materials eroded from the channel cross-sections of rivers. Therefore, different bed materials have different rates of scour. Loose granular soils are rapidly eroded under fluid forces acting on the bed, while cohesive or cemented soils are more scour-resistant. However, ultimate scour depths in cohesive or cemented soils are usually deeper than those in sand bed rivers.

Generally, pier scour can be classified by scour pattern and existence of sediment supplied to the scour hole from upstream. First, pier scours may be classified into aggradation and degradation, contraction scour, and local scour by scour pattern. (Richardson, 1987, Chee, 1982, Ettema, 1980, etc.) The aggradation and degradation are due to changes of controls, hydraulic characteristics, and geometry in the river and/or upper watershed. Aggradation involves the deposition of material eroded from upper sections of a river, whereas degradation involves the lowering of a river bed.

Contraction scour is defined as the bed change resulting from the constriction of the waterway due to natural or artificial channel geometry changes. This is mainly by the decrease in channel width, either naturally or by artificial structures. The decrease in channel width causes decrease of channel cross-section area and increase flow velocity of which results in contraction scour. Contraction scour can also be caused by short term (daily, weekly, seasonally, or yearly) changes in downstream water surface elevations that control backwater and the flow velocity through the bridge openings.

Local scour is defined as the bed change in consequence of the interference of flow by piers, abutments, spurs, and embankments obstructing the flow. These obstructions accelerate the motion of fluids and create vortices that remove bed material around them. Generally, scour depth from local scours are much larger than these from the other two scour types. However, if there are major changes in stream conditions, such as construction of a large dam upstream or downstream of the bridge, or severe straightening of the stream, long-term bed elevation changes can be the largest contributor to the total scour.

Second, depending upon existing of sediment supplied to the scour hole from upstream, pier scour may be classified into clear-water and live-bed scours. (Richardson, 1990, Raudkivi and Ettema, 1983, Jain and Fisher, 1980, etc.) This classification is used mainly for local scour rather than aggradation and degradation, or contraction scour. This classification can be well explained using the mathematical expression below.

$$q_s = q_{s1} - q_{s2} \tag{1}$$

where, q_s is the rate of scour in volume per unit time, q_{s1} is the rate of sediment transport out of the scour hole in volume per unit time, and q_{s2} is the rate of sediment transport supplied to the scour hole in volume per unit time. The clear water scour occurs when sediment is removed from the scour hole without replenishment. The clear water scour can be expressed, as $q_s = q_{s1}$; that is, q_{s2} is zero. The live-bed scour occurs when the scour hole is continuously supplied with material carried from the upper river bed. Mathematically, both q_{s1} and q_{s2} are not zero. Generally, live-bed scour occurs in the fine bed material river, and clear water scour occurs in the coarse bed material river having low stream power. However, in the coarse bed material river having high stream power live-bed material rivers. Generally, maximum clear water scour is about 10 percent greater than the maximum live-bed scour in depth. The scour, when the rate of sediment transport out of the scour hole becomes equal to the rate of sediment supplied to the scour hole, is defined as equilibrium scour. Equilibrium scour is expressed as $q_{s1} = q_{s2}$.

3. Viewpoints on Development and Analysis of Scouring

3.1 Aggradation and Degradation

Aggradation and degradation can be examined from the viewpoints of geomorphologists and hydraulic engineers. Leopold et. al. (1964) provide the following geomorphological viewpoint.

"With the rise in stage accompanying flood passage through a river reach, there is an increase in velocity and shear stress on the bed. As a result the channel bed tends to scour during high flow. Because sediment is being contributed from upstream, as the shear decreases with the fall of stage the sediment tends to be deposited on the bed or the bed fills. Channel scour and fill are words used to define sedimentation during relatively short periods of time, whereas the terms degradation and aggradation apply to similar processes that occur over a longer period of time. Scour and fill involve times measured in minutes, hours, days, perhaps even seasons, where, aggradation and degradation apply to persistent mean changes over period of time measured in years."

Contrast with this, Simons and Senturk(1977) provide the following engineering viewpoint.

"A river is stable when the geometry of a cross section is constant in time. If the bottom level increases in elevation the streambed is aggrading. If the bottom level decreases in elevation the streambed is subject to degradation."

The case studies of histories of gradation changes indicate that most changes are due to human activities, and that significant gradation changes normally take place with the first significant flow event. These gradation changes can take place in a few days, weeks, or months depending on the duration of the flow event. Generally, it is during the first formation of a river when the river system is most unstable, and at that time the river will make a significant gradation change or adjustment in location. These gradation changes or adjustments may then continue at a slower rate over periods of time measured in years. Because such long-term changes may be caused either by natural processes or by human activities, the engineers should assess the future changes in the river system based on the present state of the river and watershed.

Analysis of long term river bed elevation changes must be made using the principals of river mechanics in the context of a fluvial system analysis. Such analysis of a fluvial system requires the consideration of all influences upon the bridge crossing, such as runoff from the watershed to the channel (hydrology), the sediment delivery to the channel (erosion), the sediment transport capacity of the channel (hydraulics), and the response of the channel to these factors (geomorphology and river mechanics). Because largest impacts of long term changes result from human activities, analysis requires a study of the past history of the river and human activities on it present water use and stream control activities, and finally concerned policies and management plans of agencies.

3.2 Contraction Scour

Contraction scour occurs when the flow area of a stream is decreased by a natural and/or artificial constriction. With the decrease in flow area there is an increase in average velocity and bed shear stress. Accordingly, there is an increase in stream power because of increasing shear stress. Finally, more bed material is transported through the contracted reach than it is transported into the reach. The increase in transport of bed material lowers the bed elevation. It is important to note that as the bed elevation is lowered due to scour, the flow area increases, and the velocity and shear stress decreases until equilibrium. Parameters that cause contraction scour, are natural river constriction, long approaches over the flood plain to the bridge, ice formation or jams, berm forming along the banks by sediment deposits, island or bar formations upstream or downstream of the bridge opening, and the growth of vegetation in the channel or flood plain.

To determine the magnitude of contraction scour from a variable backwater two considerations, which are determining of there will be variable backwater, and determining the magnitude of contraction scour for this condition, are required. If the pier is located on or close to a bend the scour will be concentrated on the outer part of the bend. Because there may be deposition on the inner portion of the bend, further concentrating is expected. Accordingly, more scour at the outer part of the bend occurs. Also at bends, the thalweg(the part of the river where the flow rate or velocity is largest) will shift toward the center of the stream as the flow increases. This can increase scour and may cause the non-uniform distribution of the scour in the bridge opening.

3.3 Local Scour

Local scour occurs due to the action of vortex systems induced by obstructions to the flow. This scour can occur in conjunction with the absence of aggradation, degradation, or contraction scour. Local scour does not normally occur over the entire channel width. The basic mechanism causing local scour is the vortex of fluid resulting from the pile up of water on the upstream edge and subsequent acceleration of flow around the nose of the pier. The action of the vortices is to erode bed materials away from the base region. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth is increased, the strength of the vortices is reduced. Accordingly, the transport rate is reduced and equilibrium is established.

The mechanism of local scour and influence of parameters at local scour are explained in the following sections.

4. Mechanism of Local Scour

Some theoretical approaches to analytical treatment of local scour have been developed from the viewpoint of jet impact, Secondly currents, Bernoulli equation, and so on. The limitation of these approaches is that they use simplifying assumptions of a very complex problem. The theories have not been able to describe the mechanics of scour, adequately and have not been found feasible for practical applications. The generally accepted mechanism of local scour is based upon vortex systems.

The analysis of vortex systems must begin with the predominant aspects of the flow. The dominant feature of the flow in the neigthborhood of the pier is the large scale eddy structure, that is, the vortex systems. Depending on the type of pier and free stream conditions, the eddy structure can be composed of all, any or, none of three basic systems, which are horseshoe, wake and trailing vortex systems.

If the pressure field induced by the pier is sufficiently strong, it causes a three dimensional separation of the boundary layer which, in turn, rolls up ahead of the pier to form the horseshoe vortex system. It is important to note that no vorticity is created by the pier in this instance. That is, the horseshoe vortex begins and ends on the channel bottom or walls.

Wake vortex system is generated by the pier itself contrary to the case of the horseshoe vortex system. The wake vortex system is formed by the rolling up of the unstable shear layers generated at the surface of the pier.

The trailing vortex system usually occurs only on pier completely submerged in the fluid. It is composed of one or more discrete vortices attached to the top of the three dimensional pier and exist between two surfaces meeting at a corner such as at the top of the pier.

The vortex development at a cylindrical pier is shown in Fig. 1.

4.1 Horseshoe Vortex System

The horseshoe vortex is related to the vertically downwards flow in front of the pier. Melville(1975) made the following observations concerning the development of the horseshoe vortex system with the formation of the scour hole around a cylindrical pier;

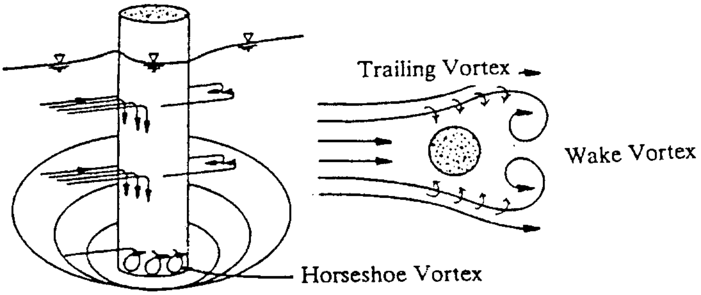

Fig. 1 Vortex development at a Cylindrical Pier

1. The horseshoe vortex is initially small in cross section and comparatively weak as is the associated downflow occurring in the plan of stagnation.
2. With the formation of the scour hole, the vortex rapidly grows in size and strength as additional fluid attains a downwards component and the strength of the downflow increases. The downflow acts somewhat like a vertical jet in eroding the bed.
3. The horseshoe vortex moves down into the developing scour hole and expands as the hole enlarges. During this initial period of scour activity the erosive power of the downflow ahead of the cylinder becomes greater as that of the accelerated flow around the base of the cylinder becomes less. Eventually, the scour becomes large enough to contain the complete vortex.
4. As the scour hole enlarges, circulation associated with the horseshoe vortex increases due to its expending cross section, but at a decreasing rate. The rate of increase is controlled by the quantity of fluid supplied to the vortex via the downflow ahead of the cylinder, which in turn is determined by the magnitude of the velocity of the approach flow.
5. Contours of bed shear stress mean flow magnitudes and directions, and turbulence intensities on the bed of the scour hole remain remarkably similar throughout the development of the scour hole after its initial formation. This is a direct consequence of the similarity of shape of the scour hole which is apparent during its growth.

As a measurement of the ability of the horseshoe vortex to scour, the strength of the horseshoe vortex can be expressed using circulation in the certain control volume (Ettema, 1980; Shen, et al., 1969; Schneider, 1968; Roper, 1967; Roper et al., 1967). The control volume is shown in Fig. 2.

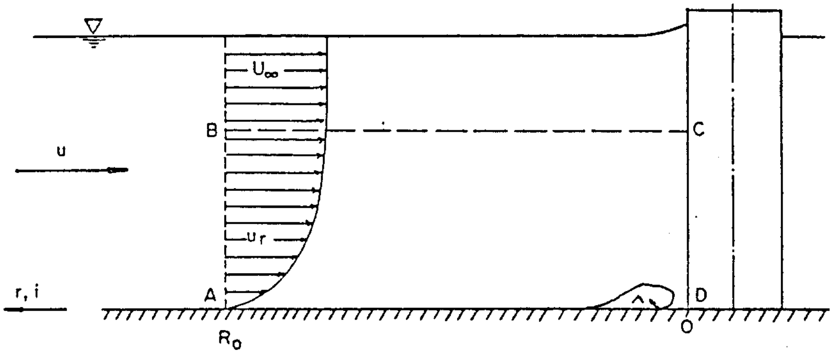

Fig. 2 Control Volume for Horseshoe Vortex

The circulation Γ about ABCD is:

$$\Gamma = \oint u.ds = \int\int \Omega.dA \tag{2}$$

Where, Ω is $\nabla \times u$, u is velocity vector, *ds* is distance along ABCD, and dA is an infinitesimal element area of the surface ABCD. By applying the no-slip requirement to the flow at the faces AD and CD and specifying that AB be placed far enough upstream so that the only component of flow at that point is in this r direction, Eq. 2 can be reduced to:

$$\int_B^C u.dr = \int\int_{ABCD} \Omega.dA \tag{3}$$

It is important to point out that the above simplification is possible regardless of the shape of the bed. If a is the half-width of the pier, then

$$u(r) = U_\infty \left[\frac{r}{(r^2 + a^2)^{1/2}} \right] \tag{4}$$

In Eq. 4, U_∞ is the velocity at point B. This can be replaced as follows:

$$\int_{ABCD} \Omega.dA = U_\infty R_o - \frac{aR_o}{a + R_o} U_\infty \tag{5}$$

Where, R_o is distance from the pier to face AB. It is important to note that the initial term of Eq. 5 is the circulation which would exist about a hypothetical control volume ABCD and would not present in the pier. Therefore,

$$\Delta\Gamma = -\frac{aR_o}{a + R_o} U_\infty \tag{6}$$

In case of Ro ⟩⟩ a, Eq. 6 becomes as follows:

$$\Delta\Gamma = -aU_\infty \tag{7}$$

This is a strength of circulation which causes the horseshoe vortex.

4.2 Wake Vortex System

The stagnation pressure causes sideward acceleration of flow past the obstacle. The separation inducing by sideward acceleration of flow creates the wake with the wake vortex system at the interfaces to the main stream. Melville(1975) postulated, from observations of dye traces, that the arms of the horseshoe vortex extending around the flanks of a pier oscillated laterally and vertically at the same frequences as the shedding of the wake vortices. For a sharp-nosed pier with a relatively weak horseshoe vortex system, the wake vortex system can become the dominant scour mechanism. However, when this occurs the local scour develops to lesser depths than when the horseshoe vortex and the downflow form the dominant scour mechanism(Ettema, 1980).

Fig. 3 shows schematic formation of reversed flow, separation and wake by an adverse pressure gradient(AGARD, 1978).

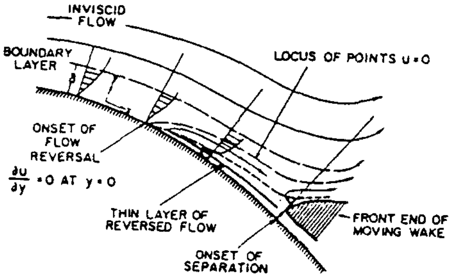

Fig. 3 Formation of reversed flow, separation and wake

The reverse motion of the fluid particles at the edge of the separation region causes a wake vortex flow in the separated regions. The wake vortices in the flow less than 40 of Reynolds number are independent of time. In the flow more than 40 of Reynolds number the vortices are shed periodically from alternate sides of pier having symmetrical or asymmetrical characteristics. The strength of the vortices in the wake system varies greatly depending on pier shape and fluid velocity. A streamlined pier will create a relatively weak while a bluff body produces a very strong one. The wake vortex system acts somewhat like a vacuum cleaner to pick up the bed material. It is then carried downstream by the eddies (John, et al., 1980; Keefer, et al., 1980; Roper, 1967; Batchelor, 1967; Duncan et al., 1950).

4.3 Trailing Vortex

As mentioned previous chapter, the trailing vortex system forms in the wake region due to the pressure difference between the pier top and sides with a tendency of flowing from the lower surface to the upper surface. It can be also occurred anywhere two surfaces at different pressures meet at sharp corners. Therefore, without pier width enough to induce pressure difference, the trailing vortex system cannot be indicated (John, et al., 1980; Keefer, et al., 1980; Roper, 1967; Batchelor, 1967; Duncan et al., 1950). The expressions of trailing vortex system can be shown as based upon the continuity concept and pressure distribution concept. The equation based upon continuity concept can be expressed as Eq. 8 with assuming axial-symmetry and full development in the flow direction (Roper, 1967).

$$\frac{\partial \zeta}{\partial t}+\left(\frac{\partial}{\partial r}+\frac{1}{r}\right)\{R\Omega^2(\Psi_1(\lambda)-\Psi_2(\lambda))\}=\left(\frac{\partial^2}{\partial r^2}+\frac{1}{r}\frac{\partial}{\partial r}\right)\zeta \tag{8}$$

where, ζ is vorticity component in flow direction, Ω is core vorticity, ψ is stream function, $\lambda=\gamma/R(t)$ is radius ratio, γ is radius in core of a vortex and R is core radius of a vortex. The equation based upon the pressure distribution can be expressed as Eq. 9 (Duncan, et. al., 1960).

$$\Gamma=\int_{x_1}^{x_2}\frac{p_1-p_2}{\rho v_0}dx \tag{9}$$

where, Γ is circulation, $\chi_1 - \chi_2$ is distance difference, $P_1 - P_2$ is pressure difference, and υ_0 is approaching velocity. These vortices or circulation cause the trailing vortex.

4.4 Summary

Although the vortex system approach is strong in the theoretical sense because it tells much about fluid mechanics of the scour, the equations based upon the vortex system are seldom used in the practical application. More generally accepted equation is based upon the dimensional approach which is done on the most significant parameters shown in the following section. Those equations were usually validated by experimental tests or field observation data.

5. Influence of Parameters at Local Scour

The basic mechanism causing local scour at a pier is the formation of a vortex at the base. The vortex is influenced by following parameters which are fluid characteristics, stream flow variables, stream bed material variables and pier variables (Richardson, 1990; Molinas et al., 1989; Melville, 1984; Breusers, et al., 1977; Laursen, 1952; etc.).

5.1 Flow Depth

There are several references in the literature stating that the scour depth is affected by the flow depth relative to the pier width or diameter. From the experimental results the influence of this parameter can be neglected in case of $\frac{y}{b} > 3$.

That is, as the water depth increases the scour depth becomes almost independent of depth of flow.

5.2 Pier Size

The width of pier has a direct affect on the depth of scour. With an increase in pier width, the velocity of flow in the bridge opening is increased to maintain continuity of mass. Consequently, there is an increase in scour depth. Generally, the length of a pier has no appreciable affect on scour depth as long as the pier is parallel with the flow. However, if the pier is at an angle to the flow, the length has a large affect.

5.3 Velocity

For the case of clear water scour, the scour depth increases almost linearly with velocity, with a slow approach to the maximum scour depth. For scour with sediment motion the scour depth is not a function of velocity. The depth of live-bed scour fluctuates around the equilibrium depth. Generally, it is known that the scour depth is increased with an increase in Froude number in subcritical flow.

5. 4 Sediment Size

In the steady state flow during the long period time, the sediment size does not affect to the depth of local scour. Especially, for larger grain size the ultimate or maximum scour is unaffected by the grain size. However, the time that it may take for these materials to be removed depends upon the size of the material. The time that it may take to reach ultimate scour may be very large in the coarse sediment. Therefore, there may be sediment size effect in the short time flood.

5. 5 Sediment Gradation

Raudkivi and Ettema(1977) stated that sediment gradation has an influence on the maximum depth of clear water scour and presented a relationship between the maximum clear water scour depth and the geometric standard deviation of sediment grading. For the live-bed scour the sediment gradation has little effect to the maximum scour depth. However, the time reached to the maximum scour depth is depending upon the sediment gradation.

5. 6 Shape of the pier

The upstream pier shape has a strong influence on the scour depth but the downstream pier shape has a munimum effect if the pier is aligned with the flow. The scour at elliptic or lenticular shape pier is expected a 20 to 30 % reduction than that in rectangular pier. In the semicircular shape pier, about 10 % reduction of scour is expected (Laursen, 1962).

5. 7 Angle of Attack

For pier shapes other than circular, the depth of local scour depends upon the alignment of the pier with flow. The local scour depth is related to the projected width of the pier, and this width increases with the angle of attack of flow. If pier is at an angle to the flow, the projected width increases. In the same incident angle, doubling of the projected width can increase scour depth by 33 percent. With an increasing angle of attack, the location of maximum scour depth moves along the exposed side of the pier from the front to the rear end of the pier.

5. 8 Bed Material Density

Some authors have carried out experiments with various bed material densities under the same conditions. The conclusion was that the scour depth increases with decreasing bed material density.

5. 9 Flow Duration

The ultimate scour depth will be reached only during floods of a sufficiently long duration.

For very short floods time may be important, because maximum scour will occur on the receding flood. At this stage, the river bed has been lowered to its lowest level, and with decreasing flow the general sediment transport is already reduced so that clear water scour conditions prevail. Here the rate of scour development can have an important influence on the maximum scour depth.

6. Recommendation for Design Application

6.1 General

Generally, the hydraulic engineer cooperates with the bridge design engineer in three principal areas: in deciding an bridge opening to pass some design discharge without producing adverse backwater conditions, in evaluating scour conditions to aid in establishing foundation depths, and in evaluating the need for designing bank protection and/or flow control structures.

In fact, the decision of an bridge opening, and the evaluation of the need of bank protection and flow control structures are necessary in the planning stage. In the stage of designing pier foundations, ground elevation at the pier location and physical properties of the bed material are two major considerations to bridge design engineers. Gradation changes by scouring at a site will not significantly affect the properties of the supporting material, but they will influence the bed elevation and thus affect the bearing capacity of a foundation located at some elevation below the surface. Long-term aggradation and degradation, contraction scour, and local scour will influence the depth at which a foundation should be placed.

Grade changes by aggradation and degradation, and/or long-term contraction scour influence the design of pier foundation in two ways. First, they alter the normal base elevation of the channel bed. Since local scour or short-term contraction scour computations predict a depth of scour below this normal bed elevation, ignoring the long-term grade change in an analysis to determine an adequate depth for pier foundations could result in the use of an inadequate base elevation for the foundations. Long-term grade change also influences the normal hydraulic characteristics of the river, such as slope, velocity, flow depth, degree of constriction at the bridge, bed roughness and so on. Since these parameters are the inputs to local scour depths, changes in their magnitudes will influence expected local scour depths. Aggradation at a site influences the hydraulic characteristics so as to increase computed local scour depths. Degradation, on the other hand, influences the hydraulic characteristics at a crossing so as to reduce the computed local scour; however, although the computations will show less scour, its importance as a hazard is increased.

Therefore, it is necessary to predict the total scour rather than just local scour. Many hydraulic or bridge design engineers misunderstand the local scour as only scour which is considered to predict the lowest elevation at bridge pier.

6.2 Aggradation and Degradation

Aggradation and degradation are usually obtained through satisfying the sediment continuity equation using estimated sediment transport rates at each reach. This process involves the following five basic steps, which are setting up the model, evaluating the water surface and energy grade line profiles for current geometric conditions, estimating sediment transport rates within each reach using sediment transport equations, applying the continuity equation for sediment transport over segments of the river to determine depth of aggradation or degradation, and computing the new bed profile.

Understanding the basic concepts of sediment transport is essential for estimating sediment transport rates. Sediment transport theory is presented in numerous texts or reports, such as Brown et al. (1981), Simons and Senturk(1977) and so on. Sediment particles are transported by one of a combination of the bed(surface creep), moving in short steps but periodically coming to rest on the bed(saltation), and supported by the surrounding fluid during its entire motion (suspension). Sediments that move as surface creep or saltation, and are supported by the bed, are called bed load. Sediments that are suspended and supported by the flow are called suspended load. The sum of the bed load and suspended load is the total bed material load. That part of the sediment load, which is smaller than representative bed material supplied from upstream sources, is termed wash-load. Generally, it is assumed that bed material load size is equal to or greater than 0.0625 mm, the division point between sand and silt. The sum of bed material load and wash load becomes the total sediment load. Since the sediment transport equations can merely determine the bed material load, total

Table 1 Applicability of aggradation and degradation equations.

Material	Method	Wash Load	Bed Material Load		
			Suspended	Bed Load	Total Load
Wash Load	Direct Measurement	x	x		
	Estimated from river response	x	x		
Sand	Einstein		x	x	
	Modified Einstein	x	x	x	x
	Colby	Measure or Estimate	x	x	x
	Schoklitsch			x	
	Bagnold		x	x	x
	Toffaletti		x		x
	Laursen			x	x
	Shen and Hung				x
	Yang				x
	Ackers and White		x	x	x
Gravel and Cobble	Schoklitsch			x	
	Meyer-Peter, Muller			x	

sediment load can be predicted only if the wash load is estimated by measurement, empirically, or by analytical relations.
The frequently used equations for aggradation and degradation prediction are given in Table 1.

Because there is no universal equation which can be used under all conditions, Yang(Molinas et al., 1989) suggested following recommendations of selection of equations for bridge design engineers.

1. Determine the type of field data available or measurable within the time, budget, and manpower limitations.
2. Examine all the equations and select those with measured values of independent variables determined from step 1.
3. Compare the field situation and the limitations of equations selected in step 2. If more than one equation can be used, calculate the rate of aggradation and/or degradation by these equations, and compare the results.
4. Decide which equations can best agree with the measured sediment load and use these equations to estimate the rate of aggradation and/or degradation at those flow conditions when actual measurements are not available.
5. In the absence of measured sediment load for comparison, the following equations should be considered:
   a. Use Meyer-Peter and Muller(1948) equation when the bed material is coarser than 5 mm.
   b. Use Einstein's(1950) procedure when bed load is a significant portion of the total load.
   c. Use Toffaletti(1968) equation for large sand-bed rivers.
   d. Use Colby's(1964) equation for rivers with depth less than 10 feet.
   e. Use Shen and Hung's(1972) equation for laboratory flumes and very small rivers.
   f. Use Yang's(1973) sand equation for sand bed laboratory flumes and natural rivers with wash load excluded. Use Yan's(1984) gravel equation for gravel transportation when the bed material is between 2 and 10 mm.
   g. Use Ackers and White's(1973) or Engelundand Hansen's(1972) equation for subcritical flow condition in the lower flow regime.
   h. Use Laursen's(1958) equation for laboratory flumes and shallow rivers with fine sand or coarse silt.
   i. A regime or regression equation can be applied to a river only if the flow and sediment conditions are similar to that from where the equation was derived.

The least scattered curve without systematic deviation from a one-to-one correlation between dependent and independent variables should be selected as the sediment rating curve for the station.

6.3 Contraction Scour

Contraction scour is usually caused by different bridge site conditions. four possible cases of contraction scour are diagramed in Fig. 4., and are as follows (Richardson, et al., 1990; FHWA, 1988):

1. Case 1: Overbank flow on a flood plain forced back to the main river by the approaches to the bridge. In this case the bridge and/or river width is narrower than the normal river width.
2. Case 2: The normal river channel width narrowed because of the bridge itself, or by the bridge site being on a narrower reach of the river.
3. Case 3: A relief bridge in the overbank area with little or no bed material transport in the overbank area.
4. Case 4: A relief bridge over a secondary river in the overbank area.

There are a few equations for estimating the magnitude of contraction scour. Those equations are based on laboratory studies with limited field data. Currently, Laursen's equations only are generally used to predict contraction scour in the world. Eq. 10 (laursen, 1958) are used for case 1, case 2 and case 4.

$$\frac{y_c}{y} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{\frac{6(2+e)}{7(3+e)}} \left(\frac{n_2}{n_1}\right)^{\frac{(6e)}{7(3+e)}} \qquad (10)$$

where, $d_s = y_c - y$ is contraction scour depth, y is average flow depth in normal section, y_c is average flow depth in contracted section, Q_t is flow rate in the contracted section, Q_c is flow rate in the normal section, n_1 is Manning's n for normal section, n_2 is Manning's n for contracted section, e is transport coefficient, W_1 is river width in the normal section, and W_2 is river width in the contracted section.

Laursen's simplified equation, Eq. 11, (Laursen, 1980) is used for case 3.

$$\frac{y_c}{y} = \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{v^2}{120 y^{1/3} D_{50}^{2/3}}\right)^{3/7} \qquad (11)$$

where, v is average flow velocity (ft/sec) and D_{50} is median diameter of the bed material (feet).

6.4 Local Scour

As mentioned in the previous chapter, three approaches are used to predict the depth of local scour at piers. The analytical approaches, which are mainly based on vortex theory, are encouraging, but little predicting equations are developed because of the complexity of the scour phenomena.

Most popular predicting equations are based on the experimental approaches. These approaches are very often supported by dimensional analysis emphasizing one or more nondimensional parameters. Typically used dimensional analysis are conducted as following steps. First, the maximum or equilibrium scour depth can be written as a function of a number of parameters.

$$d_s = F(y, b, U, D_{50}, \sigma_g, \phi, \alpha, \gamma_s, t, g, \rho, \nu) \tag{12}$$

where, U is mean approach velocity, σ_g is shape factor, T_s is submerged weighted weight of the sediment, t is duration, g is the acceleration of gravity, ρ is water density, and ν is kinematic viscosity. Second, non-dimensional function can be obtained with neglecting viscosity effect.

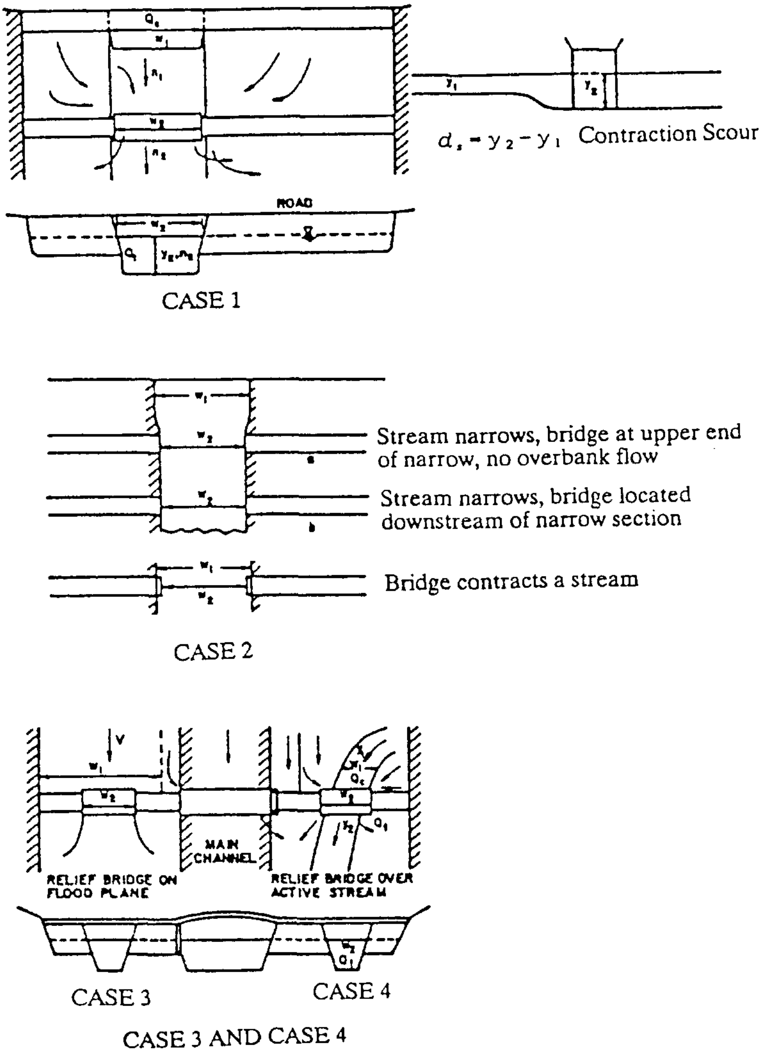

Fig. 4 Four possible cases of contraction scour

$$\frac{d_s}{b}=F\left(\frac{y}{b},\frac{U}{\sqrt{gy}},\frac{D_{50}}{y},\sigma_g,\phi,\alpha,\frac{Ut}{b},\frac{\rho V^2}{\gamma_s D_{50}}\right) \tag{13}$$

Third, with constant density assumption and elimination of redundants, Eq. 13 can be rewritten as:

$$\frac{d_s}{b}=F\left(\frac{y}{b},\frac{U}{\sqrt{gy}},\frac{D_{50}}{y},\sigma_g,\phi,\alpha\right) \tag{14}$$

Fourth, ther existing equations suggests that the dependence of scour depth on sediment gradation is negligible. If the effect of σ_g is neglected, Eq. 14 becomes:

$$\frac{d_s}{b}=F\left(\frac{y}{b},F_r,\frac{D_{50}}{y},\phi,\alpha\right) \tag{15}$$

Fifth, referring to the structure of most of the developed models, the functional relationship might be expressed as:

$$\frac{d_s}{b}=\kappa K_1 K_2\left(\frac{y}{b}\right)^a (F_r)^b\left(\frac{D_{50}}{y}\right)^c \tag{16}$$

where, κ is proportional constant, K_1 is shape factor, K_2 is alignment factor, and a, b, c are exponents.

To analyze scour, the engineer should evaluate his problem and select the equations which can be best suits in the applied cases in his judgement. It may be necessary to use more than one equation and then use engineering judgement in determining the local scour depth. Usually, the maximum clear water scour depth is about 10 % greater than live-bed scour. However, there is no need to increase the scour depths because the equations predict the maximum scour depths. When armoring occurs, the coarser bed material will tend to remain in place or quickly redeposit so as to form a layer of riprap-like armor in the scour holes, thus limiting further scour for a particular discharge. This armoring effect can decrease scour hole depths which were predicted to occur based on equations developed for sand or other fine material rivers. When larger flow conditions occur the armor layer can be broken and the scour hole deepened until either a new armor layer is developed or the maximum scour as given by the sand bed equations is reached.

However, unfortunately knowledge of how to predict the decrease in scour hole depth, when there are large particles in the bed material, is lacking. Researches in New Zealand by Raudkivi(1986) and in Washington State(Copp et al., 1988) gives a bases for calculating the decrease in scour depth by armoring but their equations need field validation.

Sterling Jones(1984) compared several pier scour equations using field data. He found out that some equations predict scour depths had good agreement with field data, but the others did not. In "Interim Procedures for Evaluating scour at Bridges"(FHWA, 1988), only one equation (CSU's) is recommended to predict the depth of proper scour at piers. However, the following five equations are generally accepted to predict scour at piers (Richardson et al., 1990; Molinas, et al 1989; FHWA, 1988, and so on).

1. Colorado State University's (CSU) equation (Richardson et. al., 1975, 1988)
2. Jain and Fisher's equation (Jain and Fisher, 1980; Jain, 1981)
3. Laursen's equation (Laursen, 1958; Laursen, 1980)
4. Froehlich's equation (Froehlich, 1988)
5. University of Aukland's (UAK) equation (Copp, et. al., 1988)
6. In case none of the existing aggradation and/or degradation equations can give satisfactory results, use the existing data collected from a river station and plot sediment load or concentration against water discharge, velocity, slope, depth, shear stress and stream power.

7. Summary and Conclusions

Scour, which is the enlargement of flow section by the removal of material composing the boundary through the flowing of fluid at piers, consists of three components of aggradation and degradation, contraction scour, and local scour. Therefore, to predict scour depth at piers it is necessary that the depth predicted by local scour should be superimposed on the depths by aggradation and degradation, and/or contraction scour. The dominant feature of the flow in the neighborhood of the pier is the large scale vortex systems which are horseshoe, wake and trailing vortex systems. The aggradation and degradation are influenced by human activities, land use changes, and natural processes. The contraction scour is influenced by natural or artificial river contractions, and the local scour is influenced by flow characteristics, river flow variables, river bed materials, and pier variables.

There are many predicting equations for the estimation of depths of scour at piers. Because only some equations predicting scour depths have good agreement with field data, it is necessary the engineer should evaluate his problem and select the equations which can be best suits in the sites in his judgement. In general, existing equations are only applicable to conditions similar to those for which they were obtained. Although no universally accepted prediction equation can selected, more comprehensively used methods and/or equations are recommended as an aid for predicting pier scour.

References

1. AGARD, (1978) AGARD Lecture Series No. 94, Three Dimensional and Unsteady Separation at High Reynolds Numbers, London, England.
2. Baker, R. E., (1986) Local Scour at Bridge Piers in Non-Uniform Sediment, Report No. 402, Dept. of Civil Engineering, Univ. Of Auckland, Auckland, New Zealand.

3. Baker, C. J., (1981) New Design Equations for Scour around Bridge Piers, J. Of the Hydraulics Division, ASCE, Vol. 107, PP. 507-511.
4. Batchelor, G. K., (1967) An Introduction to Fluid Dynamics, Cambridge University Press, Cambridge, England.
5. Breusers, H. N. C., Nicollet, G., Shen, H. W., (1977) Local Scour around Cylindrical Piers, J. Of Hydraulic Research, pp. 221-252.
6. Brown, S. A., McQuivey, R. S. And Keefer, T. N., (1981) Stream Channel Degradation and Aggradation: Analysis of Impacts to Highway Crossings, Report No. FHWA/RD-80/159, FHWA, Washington, D. C.
7. Chee, R. K. W., (1982) Live-Bed Scour at Bridge Piers, School of Engineering Report No. 290, Dept. of Civil Engineering, Univ. Of Auckland, Auckland, New Zealand.
8. Copp, H. D., Johnson, J. P. And McIntosh, J. L., (1988) Prediction Methods for Determination of Anticipated Depths of Local Pier Scour at Intermediate Bridge Piers, Paper Presented at the 68 Annual, TRB meeting, Washington D. C.
9. Duncan, W. J., Thom, A. S. And Young, A. D., (1960) An Elementary Treatise on the Mechanics of Fluids, Edward Arnold(Publishers)Ltd., London, England.
10. Ettema, R., (1980) Scour at Bridge Piers, Report No. 216, Dept. of Civil Engineering, Univ. Of Auckland, Auckland, New Zealand.
11. FHWA, (1989) Scour at Bridges, Hydraulic Engineering Circular, No. 18, Washington, D. C.
12. FHWA, (1988) Interim Procedures for Evaluating Scour at Bridges, U. S. Dept. of Transportation, FHWA(Federal Highway Administration), Washington, D. C.
13. Froehlich, D. C., (1988) Analysis of On-Site Measurements of Scour at Piers, Proceedings, ASCE National Hyd. Eng. Conference, Colorado Springs, Co.
14. Jain, S. C., (1981) Maximum Clear-Water Scour Around Circular Piers, J. Of the Hydraulics Division, ASCE, Vol. 107, pp. 611-626.
15. Jain, S. C., Fisher, E. F., (1980) Scour around Bridge Piers at High Flow Velocities, J. Of Hydraulic Engineering, ASCE, Vol. 104, pp. 1827-1842.
16. Jarrett, R. D. And Boyle, J. M., (1986) Pilot Study for Collection of Bridge-scour Data, U. S. Geological Survey, Water-Resources Investigations Report 86-4030, Denver, Co.
17. John, J. E. A., Haberman, W. L., (1980) Introduction to Fluid Mechanics, Prentice Hall, Inc., EngleWood Cliffs, New Jersey.
18. Keefer, T. N., McQuivey, R. S. And Simons, D. B., (1980) Stream Channel Degradation and Aggradation: Causes and Consequences to Highways, Report No. FHWA/RD-80/038, FHWA, Washington, D. C.
19. Laursen, E. M., (1980) Predicting Scour at Bridge Piers and Abutments, General report No. 3, Engineering Experiment Station, Univ. Of Arizona, Tucson, Arizona.

20. Larsen, E. M., (1962) Scour at Bridge Crossings, Transactions, ASCE, Vol. 127, pp. 166-209.
21. Laursen, E. M., (1958) The Total Sediment Load of Stream, J. Of Hydraulics Division, ASCE, Vol. 84, pp. 1530. 1-1530. 36.
22. Laursen, E. M., (1958) Scourc at Bridge Crossings, Bulletin No. 8, Iowa Highway Research Board, Iowa State Highway Commission, Iowa.
23. Laursen, E. M., (1952) Observations on the Nature of Scour, Proceedings 5th Hydraulics Conference, State Unversity of Iowa, pp. 179-197.
24. Leopold, L. B., (1964) Fluvial Processes in Geomorphology, W. H. Freeman and Company, London, England.
25. Melville, B. W., (1984) Live Bed Scour at Bridge Piers, J. Of Hydraulic Engineering, ASCE, Vol. 110, PP. 1234-1247.
26. Melville, B. W., (1975) Local Scour at Bridge Sites, Report No. 117, School of Engineering, Univ. Of Auckland, Auckland, New Zealand.
27. Milne-Thomson, L. M., (1950) Theoretical Hydrodynamics, Second Edition, The Macmillan company, New York.
28. Molinas, A., Choi, G. W. And Greager, B. A., (1989) User's Manual for BRI-STRS, Hydrau-Tech Engineering & Software, Fort Collins, Co.
29. Raudkivi, A. J., (1986) Functional Trends of Scour at Brigde Piers, J. Of Hydraulic Engineering, ASCE, Vol. 112, pp. 1-13.
30. Raudkivi, A. J., Ettema and R., (1983) Clear Water Scour at Cylindrical Piers, J. Of Hydraulic Engineering, ASCE, Vol. 109, pp. 338-350.
31. Raudkivi, A. J. And Ettema, R., (1977) Effect of Sediment Gradation on Clear-Water Scour, J. Of Hydraulic Engineering, ASCE, Vol. 103
32. Richardson E. V. And Richardson, J. R., (1990) Bridge Scour, Draft, Colorado State University, Fort Collins, Co.
33. Richardson, E. V., Simons, D., Julien, P. Y., (1988) Highways in the River Environment, Second Ed., U. S. Department of Transportation, FHWA, Fort Colllins, Co.
34. Richardson, E. V., (1987) Scour at Bridges, FHWA, Technical Advisory, Fort Collins, Co.
35. Richardson, E. V. et. al., (1975) Highways in the River Environment, First Ed., U. S. Dept. of Transportation, FHWA, Fort Collins, Co.
36. Roper, A. T., (1967) A Cylinder in a Turbulent Shear Layer, Ph. D. Dissertation, Colorado State Univ., Fort Collins, Co.
37. Roper, A. T., Schneider, V. R. And Shen, H. W., (1967) Analytical Approach to Local Scour, Proceedings XII Congress of IAHR, Fort Collins, Co. pp. 151-161.
38. Schneider, V. R., (1968) Mechanics of Local Scour, Ph. D. Dissertation, Colorado State Univ., Fort Collins, Co.

39. Shen, H. W., Schneider, V. R., Karaki, S., (1969) Local Scour Around Bridge Piers, J. of the Hydraulics Divison, ASCE, pp. 1919-1940.

40. Simons, D. B. And Senturk, F., (1977) Sediment Transport Technology, Water Resources Publications, Fort Collins, Co.

41. Sterling Jones, J., (1984) Comparison of Prediction Equations for Bridge Pier and Abutment Scour, TRR 950, Second Bridge Engineering Conference, Volume 2, Washington D.C. pp. 202-209.