

Bridge Abutment Scour in Compound River Channels

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ABSTRACT

Most previous laboratory studies of local scour at bridge abutments were performed in rectangular channels in which the distributions of flow velocity and bed shear stress were considered uniform in the transverse direction. In reality however, bridge abutments are usually located in the floodplain zone of rivers where velocity and shear stress distributions are directly affected by the lateral momentum transfer. The influence of channel geometry and lateral momentum transfer in compound flow field on scouring phenomenon, however, has not been fully investigated and understood as yet. This paper presents the results of an experimental study performed to investigate the impact of both sediment size and lateral momentum transfer on local scour at abutments terminating in the floodplain of a compound channel. It is shown that, by accounting for lateral momentum transfer at small floodplain/main channel depth ratios ($y_a/H < 0.3$), estimates of maximum local scour depth are increased by up to 30%. In relation to the sediment size, earlier studies of scouring around circular bridge piers proposed a limit for the relative size of sediment (pier diameter/median size of bed material) beyond which the sediment size has no effect on the equilibrium scour depth (Ettema, 1980; Chiew, 1984). The results of the current laboratory studies, however, indicated that the limit established for circular bridge piers might not be appropriate for the abutment case installed in the floodplain zones; further studies are required to draw a more general conclusion regarding the effects of relative grain size in the abutment case.

Keywords: Scouring , Floodplain , Momentum transfer , Compound channel , Bridge, Abutment , Pier

INTRODUCTION

In spite of improved construction methods and materials and advanced techniques of analysis and design, the list of the failed bridges due to excessive scour around piers and abutments continues to lengthen. Some bridge failure records indicate that about 60% of failures can be attributed to scour and other related hydraulic conditions (Shirole

and Holt, 1991). Although some 35 formulae have been proposed for predicting maximum local scour at bridge piers (Copp *et al.*, 1988), only a few equations have been proposed for the case of bridge abutments. Moreover, most equations for predicting abutment scour are based on laboratory data compiled for model abutments installed in rectangular flumes (Ahmad, 1953; Garde *et al.*, 1961; Gill, 1972; Rajaratnam and Nwachukwu, 1983; Kwan,

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1988; and Kandasamy, 1989). While this arrangement is appropriate for the case of bridge abutments projecting into a river's Main Channel (MC), it is generally inappropriate when abutments terminate in the Flood Plain (FP) where significant variations in both velocity distribution and boundary roughness can be expected.

Local Scour Classification

Introducing a bridge abutment, or any other kind of flow obstructing structure into a watercourse, dramatically changes the flow pattern of the stream in the vicinity of the structure. Highly three-dimensional and rotational in nature, the generated flow structure locally increases the bed shear stress to a value higher than the critical one and initiates the scour process. The temporal development of the scour hole can be expressed as:

$$\frac{dV}{dt} = (Q_s)_o - (Q_s)_i \quad (1)$$

in which V = volume of scour hole, t = time, and $(Q_s)_o$ and $(Q_s)_i$ are respectively the volumetric rate of sediment going out of and coming into the scour hole.

According to the conditions of bed material movement, the local scour process is generally classified as clear-water scour or live-bed scour (Laursen, 1952; Raudkivi and Sutherland, 1981). When the approach flow velocity of a stream does not exceed a critical value, above which bed particles move, no general bed material movement is observed. In such flow conditions $(Q_s)_i = 0$ and no bed material enters the scour hole, therefore, the local scour process is classified as clear-water scour.

Live-bed scour, however, occurs in streams having general bed movement. In this case, the bed shear stress is higher than the critical value at every point of the streambed. Therefore, the removed particles from the scour hole are replenished by the moving

bed materials, i.e. $(Q_s)_o \geq (Q_s)_i > 0$. Since no such replacement for the removed material takes place in a clear-water condition, the scour hole is generally deeper in clear-water conditions than in live-bed conditions. Accordingly, the clear-water scour depth is recommended for design purposes (Melville, 1992; Melville and Sutherland, 1988).

Effects of Channel Geometry

Watercourse cross-sectional geometry has long been recognized as an important parameter that governs the pattern of velocity distribution, boundary shear stress, momentum transfer, and secondary circulations in open channel flows (Townsend, 1968; Myers, 1991). Because of the high resistance coefficients associated with the flood plain zones of rivers, flows in the FP are generally much slower than those in the main channel. At small FP depths, the difference between MC and FP velocities initiates a strong Lateral Momentum Transfer (LMT) in the form of banks of vortices having their vertical axes along the MC/FP junction regions (Figure 1). These vortices act as a mechanism for transferring momentum between the MC and FP zones by their continuous emergence and decay (Sellin, 1964). As a consequence of LMT, the MC flow velocity and discharge decrease immediately above the bankfull depth, while the corresponding FP values increase.

Myers and Elsayw's (1975) study of the flow interaction phenomenon quantified the

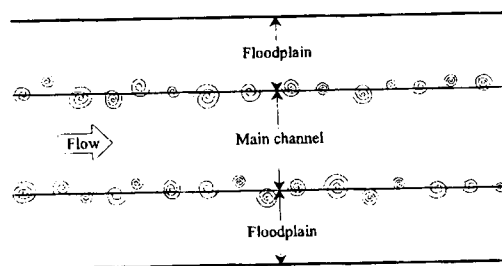


Figure 1. Banks of vortices at the MC/FP junctions

impact of LMT on both the value and distribution of boundary shear stress in the MC and FP of an asymmetric compound-shaped channel. They compared observed boundary shear stress distributions obtained under interacting (i.e. combined MC/FP flow) conditions with those for non-interacting (isolated FP flow) conditions. For their shallowest FP depth, they observed 260% and 200% increases respectively in maximum and average FP shear stresses (Figure 2). Also, in other studies by Myers (1977) it was indicated that the apparent shear force acting

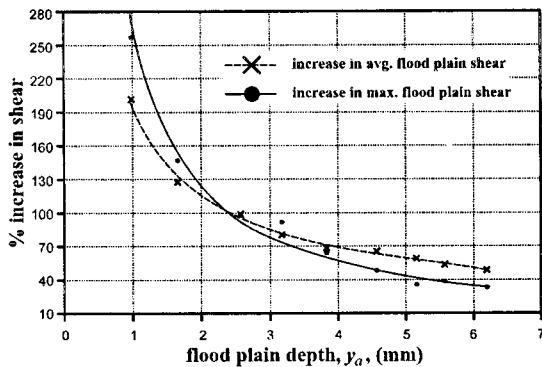


Figure 2. Increase in average and maximum floodplain shear due to MC/FP interaction, expressed as percentage of isolated floodplain value (Myers and Elsayy, 1975).

on the vertical plane of MC/FP junction amounts to 25% of the weight component of the MC and 200% of that of the FP. Clearly, in severely compound flow fields, LMT significantly impacts both the value and distribution of boundary shear stress. Therefore, to improve the understanding of the mechanism of local scour in FP zones it is necessary to properly account for LMT in these situations. This may hopefully lead to a more realistic estimation of maximum scour in such circumstances.

While extensive literature has been published on the subject of local scour at bridge piers and to a lesser extent at bridge abutments, since most of these studies were performed

in rectangular-shaped laboratory channels, none address the issue of LMT and its impact on the local scouring process. Melville and Parola (1995) categorized the abutment scour into three cases (Figure 3) and point to the need for further research to investigate the impact of channel shape and sediment size and gradation effects on the local scouring process at abutments located in compound river channels. By considering an imaginary boundary at the MC/FP junction, Melville (1995) regarded the scouring process at abutments terminating in the FP of a compound channel (case III(a), Figure 3) as being similar to that at abutments located

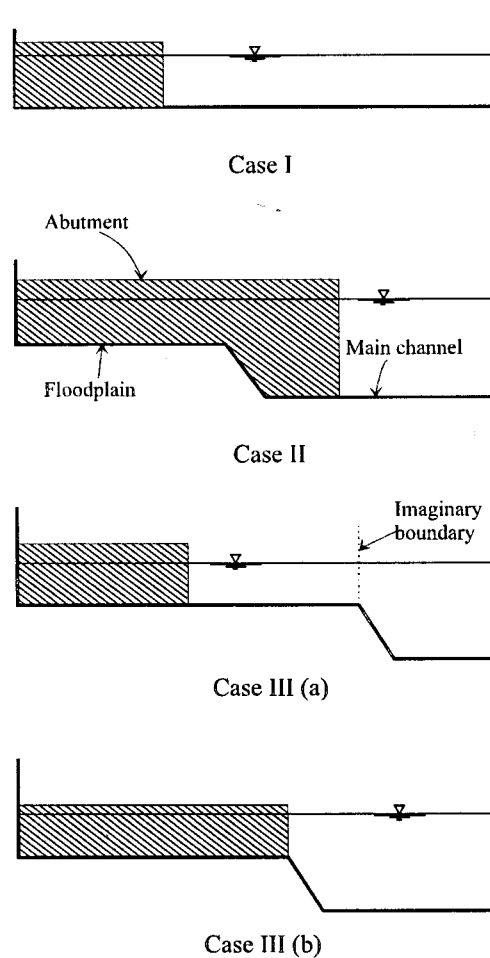


Figure 3. Bridge abutment cases (Melville and Parola, 1995)



in a simple rectangular channel (case I, Figure 3). However, he stressed on the need for further investigations for such cases by stating that “the concept of using an imaginary flow boundary for case III(a) needs to be checked experimentally”.

Effect of Relative Size of Sediment

The effects of sediment characteristics on the equilibrium scour depth, y_{se} , has been the subject of studies by, among others, Shen *et al.* (1966), Ettema (1980), and Chiew (1984). Considering other researchers' piers data in addition to their own data, Shen *et al.* (1966) came to the conclusion that the maximum scour depth is independent of sediment size when $D_{50} < 0.52$ mm.

The effects of sediment size and gradation on the maximum scour depth at bridge piers installed in rectangular channels were studied by Ettema (1980). Based on laboratory data compiled for clear water conditions (no general bed material movement), different pier sizes, b , and sediment sizes in the range $0.24 < D_{50} < 7.8$ mm, Ettema established a graphical relationship between the relative size of the pier, b/D_{50} and the relative equilibrium scour depth y_{se}/b . With non-ripple-forming sediment his collected data indicate that sediment size has no influence on y_{se}/b as long as $b/D_{50} > 25$.

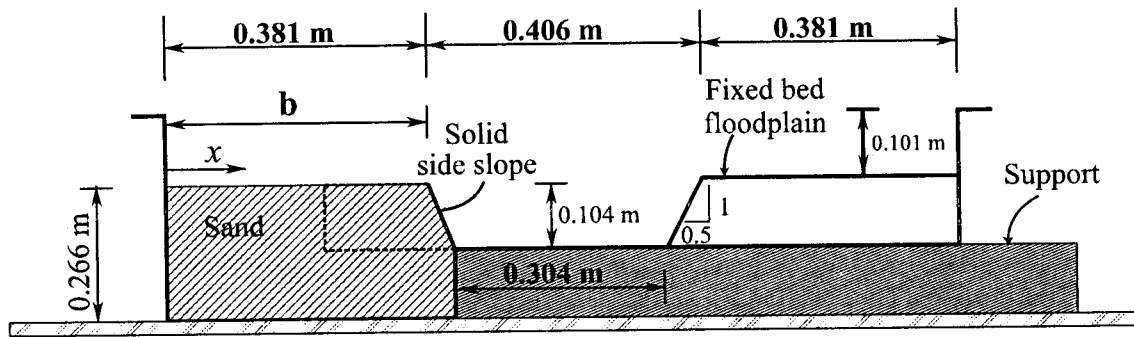
Chiew (1984) extended Ettema's work to observe the effects of pier and sediment sizes on the equilibrium scour depth under live-bed conditions (general downstream movement of bed material exists). Using the same parameters as Ettema's, Chiew showed that for live-bed conditions the relationship between y_{sa} (average equilibrium scour depth) and b/D_{50} follows the same trend as it does for the clear-water condition. Therefore he concluded that for the bridge pier case, in both clear-water and live-bed conditions, y_{sa}/b might be considered independent of sediment

size as long as the relative pier size $b/D_{50} > 50$.

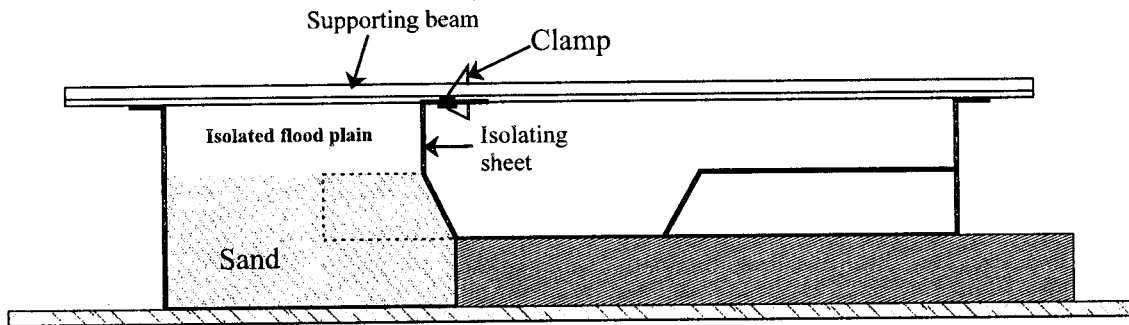
In the abutment case, since $L/D_{50} < 50$ is unlikely to be encountered in both laboratory and field conditions on the one hand, and on the other there is a lack of data to evaluate the effects of the relative sediment size, engineers usually rely on Ettema's (bridge pier) results when considering abutments (Melville, 1992).

MATERIALS AND METHODS

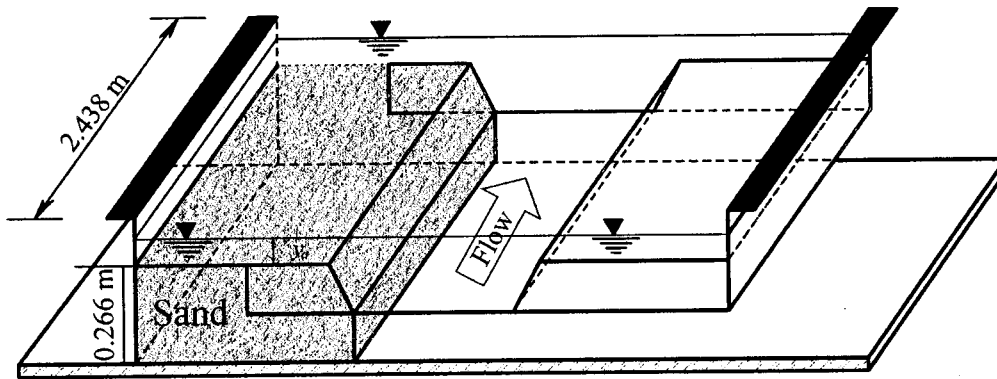
The experimental program was performed in a 12.20 m-long \times 1.20 m-wide compound channel located in the Hydraulics Laboratory of the Civil Engineering Department, University of Ottawa. The channel cross-section comprises a trapezoidal-shaped MC (bottom width=0.30 m; side slope=0.5 H:1V) located between two rectangular-shaped FPs (each 0.38 m wide), Figure 4a. The longitudinal slope of the channel is fully adjustable. The 2.43 m-long test section had a specially-designed (recessed) FP compartment to accommodate the model abutments and a sand test bed. The remaining FP sections of the channel were artificially roughened with wire mesh, using wire diameter of 0.5 mm built into 3.17 mm square-patterned openings. The interaction phenomenon had been studied by isolating the MC flow from that of the FP by many researchers (Myers and Elsayy, 1975; Myers, 1977, Rajaratnam and Ahmadi, 1979). The same method was adopted here, i.e. the non-interacting conditions were established by isolating the MC flow from that of the FP by means of a 0.5 mm thick metal sheet 0.202 m wide and 2.438 m long, which was formed in an L-shape and installed over the test section junction line separating the MC and the left FP zone (Figure 4b). Figure 4c shows a perspective view of the test section.



a. Cross-section through the test section



b- Schematic of the flow-isolating sheet

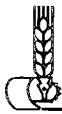


c- Perspective view of the test section

Figure 4. The test section

Since all experiments were to be run at or close to the threshold condition of bed material movement, the ratio $u./u_{.c} \approx 1.00$, where $u_{.}$ = local shear velocity and $u_{.c}$ = critical shear velocity (associated with D_{50} size). Shields

diagram (see Vanoni, 1975) is the criterion for developing the necessary threshold relationship between the representative grain size (D_{50}), the sediment Reynolds Number ($R_{.s}$) and the dimensionless shear stress parameter



(τ_c). The relationship takes the form:

$$D_s = \left[\frac{(R_{e*} \nu)^2 \rho}{\tau_* (\gamma_s - \gamma)} \right]^{1/3} \quad (2)$$

where D_s =uniform grain size, ν =fluid kinematic viscosity, ρ =fluid density, γ_s and γ = specific weight of sediment particle and fluid, respectively.

From Shields analysis, the critical shear stress (τ_c) associated with the selected D_{50} may be expressed as:

$$\tau_c = \tau_* (\gamma_s - \gamma) D_{50} \quad (3)$$

Based on Equations 2 and 3, a computer program was written to calculate the D_{50} size required to satisfy the threshold condition for a specific flow depth y_a and bed slope S_o . Sample relationships obtained in this manner for 3 different levels of S_o and for a near-threshold condition (i.e. $u./u_{*c}=0.95$), are presented in Figure 5. Once the hydraulic boundary conditions for a particular set of experiments are specified, this program facilitates selection of the appropriate D_{50} size. Following this procedure, two different grades of sand whose characteristics are summarized in table 1 were selected for the experimental program. Since the geometric standard deviations, σ_g of both sand samples are smaller than 1.5, the samples are classified as uniformly graded sand.

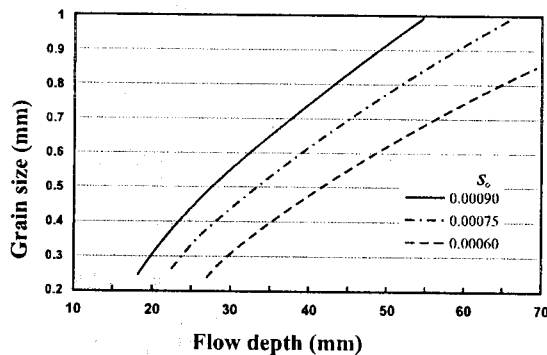


Figure 5. y_a - S_o - D_{50} -relationship for establishing near- threshold condition

Abutments with overall lengths of 90, 150 and 210 mm and of four different shapes: namely, (i) Vertical-Wall (VW), (ii) Semi-Circular (SC), (iii) Wing-Wall (WW), and (iv) Spill-Through (ST) were investigated (Figure 6). The models were constructed from solid plexiglass blocks and a side slope of 1(H):1.175(V) was adopted for the WW-and ST-abutments.

Table 1. Characteristics of the sand used in the study.

sand	D_{84} (mm)	D_{50} (mm)	D_{16} (mm)	σ_g ---	angle of repose ϕ° ---	ρ_s/ρ ---
1	0.794	0.7	0.586	1.16	31	2.65
2	0.530	0.500	0.416	1.13	31	2.65

A mini-propeller meter, capable of detecting mean velocities as low as 0.025 m/s with a maximum error of $\pm 1.5\%$, was employed for measuring velocity distributions. Measurements were recorded using a data acquisition system specifically designed for the purpose. Water surface profiles along the channel were monitored via twelve manometers connected to tapping points located along the MC. To achieve near-uniform flow conditions, the flume tailgate was adjusted until the water surface slope closely matched the preset channel bed slope. A duration of 5 hrs was selected for the tests. Long-period tests (140 hr duration) indicated that scour depths at 5 hr were approximately 60% of the equilibrium scour depth, y_{se} for the $D_{50}=0.5$ mm sand and 65% for the $D_{50}=0.7$ mm one.

RESULTS

Shear Velocity Ratios

Local shear velocity ratios, $u./u_{*c}$ ($u.$ and u_{*c} =local and critical shear velocities, respectively) at the end of the 90 mm-, 150 mm-, and 210 mm-long model abutments were

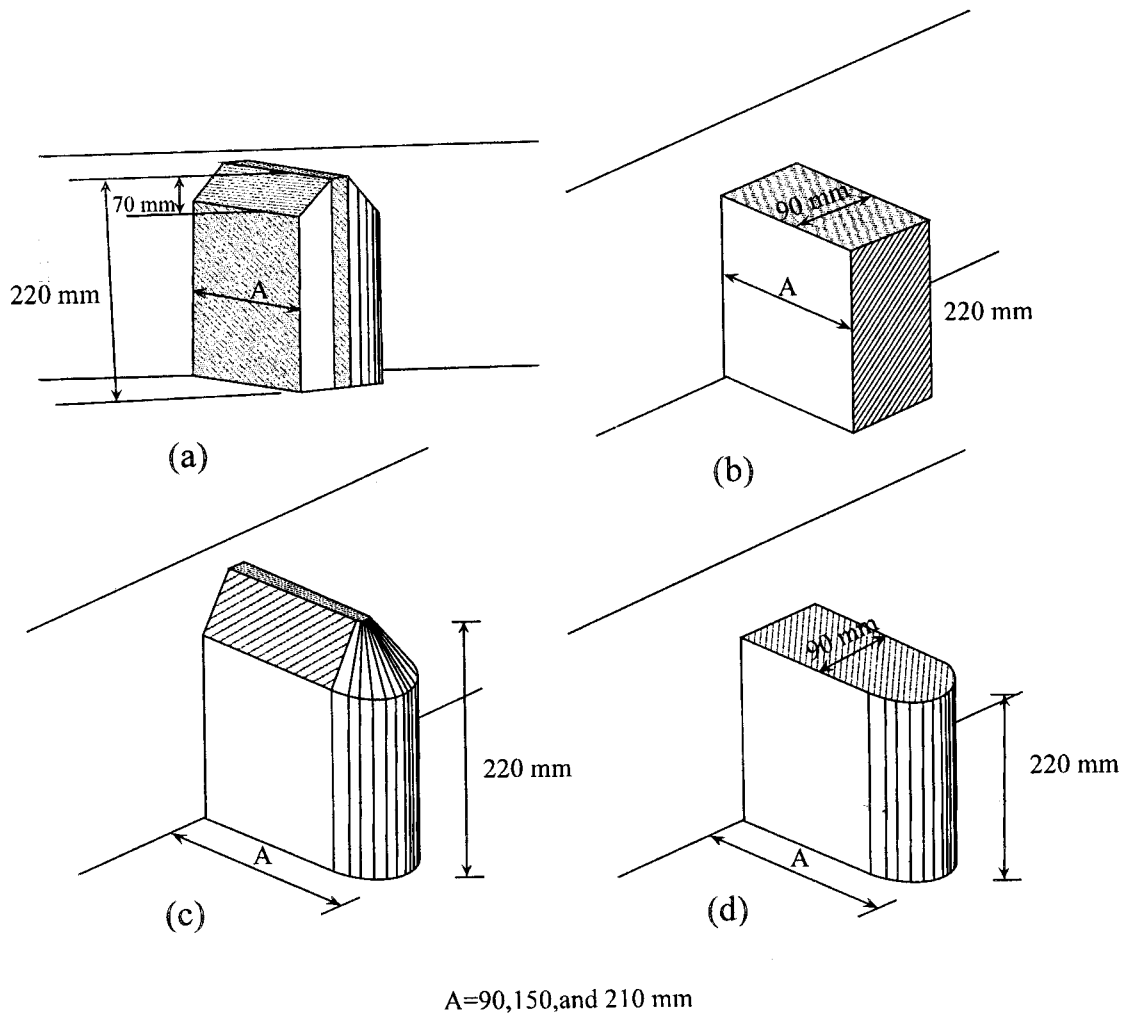


Figure 6. Abutment shapes tested: (a) wing-wall, (b) vertical-wall, (c) spill-through, and (d) semi-circular

determined based on comparisons between observed (vertical) velocity profiles and the theoretical (log-law) distributions. Figure 7 shows a sample comparison for non-interacting flow conditions for $D_{50}=0.5$ mm and $y_a/H=0.24$ (y_a and $H=FP$ and total flow depths, respectively). The shear velocity ratios for non-interacting flow conditions are presented in Figure 8. The figure shows that u_s/u_{*c} increases as the MC/FP junction is approached.

Effect of LMT on Scour Depth

To examine the effects of isolating the FP flow component from the MC flow on the local scouring process, the data for interacting conditions are compared with those for non-interacting ones. The data for different abutment shapes and $y_a/H=0.21$ and 0.24 indicate that non-interacting conditions produce 15 to 30% smaller y_{sc}/y_a than that for interacting conditions.

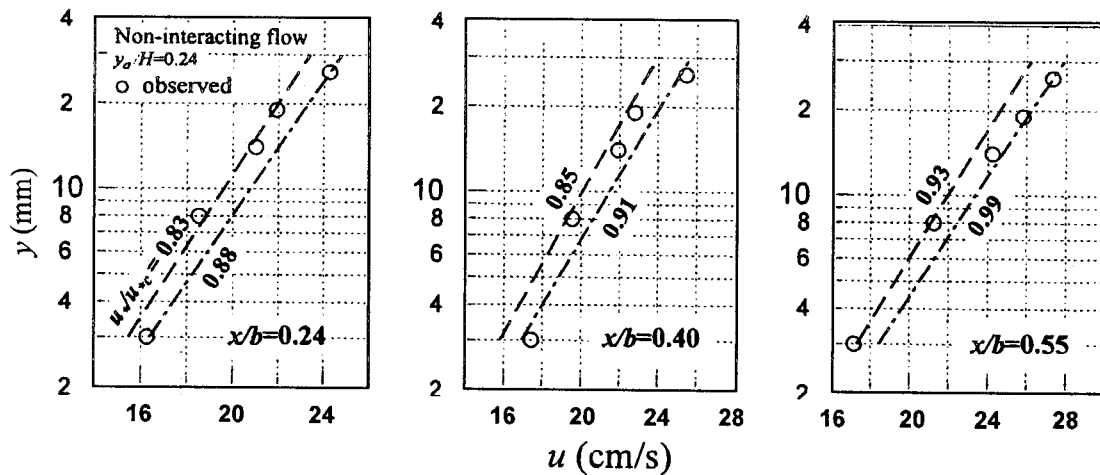


Figure 7. Observed and theoretical vertical velocity profiles

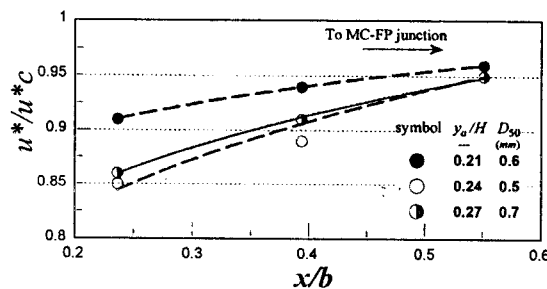


Figure 8. u_*/u_{*c} in non-interacting FP flows

Isolating the MC flow from the FP flow eliminates LMT and decreases flow velocity in the region close to the MC/FP junction. This in turn decreases the shear velocity ratio in the FP. Assuming a linear relationship between y_{se} and u_*/u_{*c} in the range $0.5 < u_*/u_{*c} < 1$, the data were such modified as to account for the fact $u_*/u_{*c} < 1$. The modified y_{se}/y_a for non-interacting conditions still show 5 to 15% smaller values than for interacting conditions. The differences between y_{se}/y_a for interacting and non-interacting conditions were between 10 to 35% for $D_{50} = 0.7$ mm and $y_a/H = 0.27$. The non-interacting data for $D_{50} = 0.7$ mm, after being modified for the shear velocity ratio, are still smaller than the interacting data. Isolating the FP flow from that of the MC affected the 90 mm-abutments the most.

The data for VW-, ST-, WW-, and SC-abutment shapes are presented in Figs. 9.a to 9.d respectively. The variations in y_{se}/y_a follow similar trends for all abutment shapes. Non-interacting conditions, however, decreased the scour depth at VW-abutments the most (Figure 9.a). Also, the figures show that the relative decrease in y_{se}/y_a for non-interacting conditions is higher at shorter abutments than at longer ones. For instance, the decrease in y_{se}/y_a at 90 mm-SC abutments due to non-interacting conditions amounted to 30% while for 210 mm-SC abutments 15% decrease in y_{se}/y_a was observed.

A comparison of the interacting and non-interacting data reveals that isolating the MC flow from the FP flow reduces scour depth. Under non-interacting conditions the 210 mm-abutments obstruct 55% of the FP width. In such a case one would expect to observe an increase in the scour depth, because of the scour component that would normally occur as a result of the relatively high degree of flow contraction. Yet, the resulting scour depths are substantially smaller for non-interacting conditions than for interacting conditions for the 210 mm-abutments. Because flows in the FP are generally slower than those in the main channel, one would expect smaller

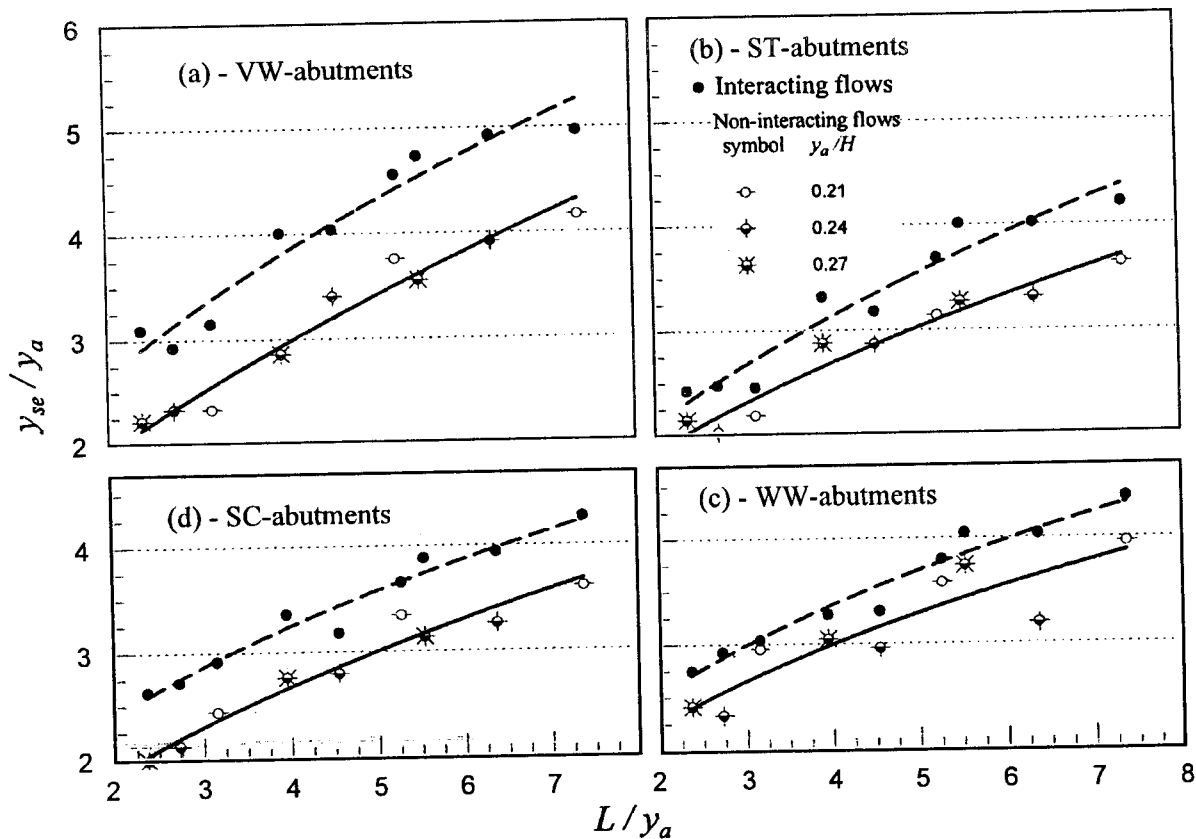


Figure 9. Maximum local scour for interacting and non-interacting flows

scour depths in the FP zones. However, the presence of a strong LMT at small FP depths increases the scour depth in the FP zones.

Relative Size of Sediment

The sediment size and gradation are substantial factors in accurately estimating the magnitude of the scour depth at abutments. A comparison between the $D_{50}=0.7$ mm and $D_{50}=0.5$ mm results is possible if the data are non-dimensionalized by a length scale. The effect of sediment size can be examined if scour depth and sediment size are non-dimensionalized by abutment length, L . The non-dimensionalized data for interacting conditions are presented in Figure 10. This figure shows the variations of y_{se}/L with L/D_{50} for different abutment shapes and

sediment sizes. Figure 10a shows the results for SC-abutments of $L=90, 150,$ and 210 mm, for both $D_{50}=0.5$ and 0.7 mm. The data indicate that, for a given abutment length, decreasing D_{50} decreases y_{se}/L . The rate of reduction in the scour depth, however, decreases with an increase in L . Figure 10 suggests that there should be a peak of relative scour depth for $L/D_{50} \gg 50$. The conclusion is valid for the applied sediment sizes and further investigations are required to draw a more general conclusion regarding the maximum relative scour depth and the range beyond which the relative sediment size has no influence on the relative scour depth for the abutment case. The results of the current study are in concordance with Ettema's (1980) data for $D_{50} < 0.7$ mm. Although Ettema concluded that no general

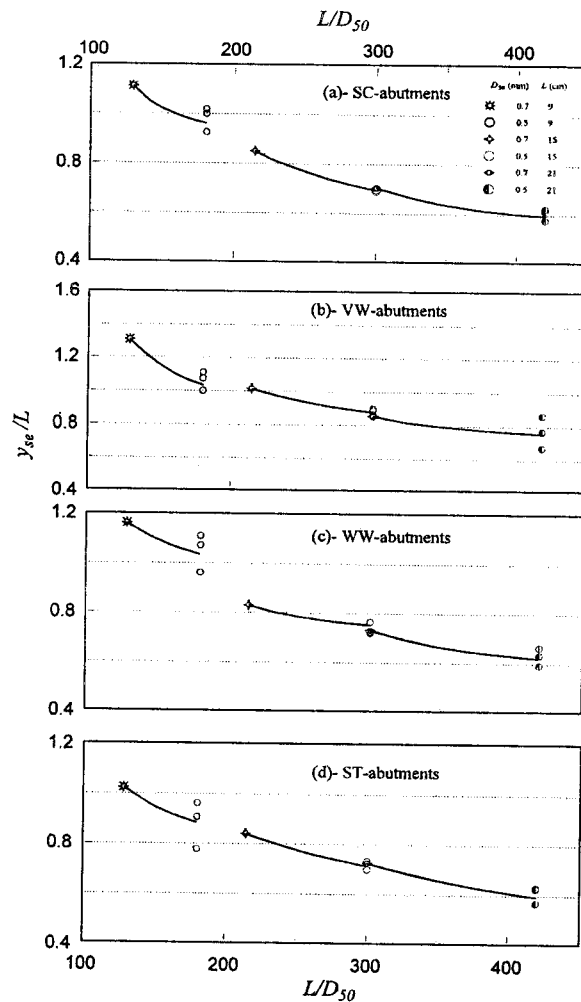


Figure 10. Influence of sediment size on scour depth

trend was observed for this range of D_{50} , careful attention to his data indicates that the trend (increasing the sediment size increases the relative scour depth) could be observed for any pier size utilized in his study.

Comparison of the results obtained using two sediment sizes indicates that, for the case of abutments, D_{50} affects scour depth even when $L/D_{50} \gg 50$. Accordingly, the limit set by Ettema for circular bridge piers might not be appropriate for the abutment case. Further studies, however, are required to draw a more general conclusion regarding the effects of sediment size for the case of bridge abutments.

CONCLUSIONS

Most of the earlier experimental studies of abutment scour were performed in channels of rectangular cross-section. While this approach is appropriate for abutments located in a river's MC, it is inappropriate for abutments terminating in the FP zones, particularly if LMT effects are strong. Although slower flows in the FP zones should result in shallower scour depths, the LMT tends to increase the scour depth in those regions. Our study data indicate that, for the case of abutments terminating in the FP regions, under conditions of strong flow interaction, LMT effects can produce a 15-30% increase in local scour depth. Therefore, design relationships for predicting maximum scour depth should account for LMT in these instances.

Comparison of the results obtained using two sediment sizes indicates that, for the case of abutments, D_{50} affects scour depth even when $L/D_{50} \gg 50$. Accordingly, the limit set by Ettema for circular bridge piers might not be appropriate for the abutment case. Further studies, however, are required to draw a more general conclusion regarding the effects of sediment size for the case of bridge abutments.

REFERENCES

1. Ahmad, M. 1953. Experiments on Design and Behaviour of spur Dikes. Proc. Int. Hydr. Conv. Minneapolis, MN: 145-159.
2. Chiew, W.M. 1984. Local Scour at Bridge Piers. Report No. 355, School of Engineering, University of Auckland, Auckland, New Zealand.
3. Copp, H.D., Johnson, J.P., and McIntosh, J.L. 1988. Prediction Methods for Local Scour at Intermediate Bridge Piers. *Transp. Res. Rec. National Research Council*, Washington, D.C. No. 1201: 46-53.
4. Ettema, R. 1980. Scour at Bridge Piers.

- Report No. 216, School of Engineering, University of Auckland, Auckland, New Zealand.
5. Garde, R.G., Subramanya, K., and Nambudripad, K.D. 1961. Study of Scour Around Spur-dikes. *J. Hydr. Div., ASCE*, **87(HY6)**: 23-37.
 6. Gill, M.A. 1972. Erosion of Sand Bed Around Spur-dikes. *J. Hydr. Div., ASCE*, **98(HY9)**: 1587-1602.
 7. Kandasamy, J.K. 1989. Abutment Scour. Report No. 458, School of Engineering, University of Auckland, Auckland, New Zealand.
 8. Kwan, T.F. 1988. A Study of Abutment Scour. Report No. 451, School of Engineering, University of Auckland, Auckland, New Zealand.
 9. Laursen, E.M. 1952. Observation on the Nature of Scour. Proc. Fifth Hydr. Conf. Bulletin 34, University of Iowa, Iowa city, Iowa: 179-197.
 10. Melville, B.W. 1992. Local Scour at Bridge Abutments. *J. Hydr. Engrg., ASCE*, **118(4)**: 615-631.
 11. Melville, B.W. 1995. Bridge Abutment Scour in Compound Channels. *J. Hydr. Engrg., ASCE*, **121(12)**: 863-868.
 12. Melville, B.W., and Sutherland, A.J. 1988. Design Method for Local Scour at Bridge Piers. *J. Hydr. Engrg., ASCE*, **114(10)**: 1210-1226.
 13. Melville, B.W., and Parola, A. 1995. The Need for Additional Abutment Scour Research. Proc. 1st. Int. Conf. on Water Resources Engrg. *ASCE*, San Antonio, TX: 1239-1243.
 14. Myers, W.R.C. 1991. Influence of Geometry on Discharge Capacity of Open Channels. *J. Hydr. Engrg., ASCE*, **117(5)**: 676-680.
 15. Myers, R.C., and Elsayy, E.M. 1975. Boundary Shear in Channel with Floodplain. *J. Hydr. Div., ASCE*, **101(7)**: 933-946.
 16. Myers, W.R.C. 1977. Momentum Transfer in Compound Channel. *J. Hydr. Res., LAHR*, **16(2)**: 139-150.
 17. Rajaratnam, N., and Nwachukwu, B.A. 1983. Flow near Groin-like Structures. *J. Hydr. Engrg., ASCE*, **109(3)**: 463-480.
 18. Rajaratnam, N., and Ahmadi, R.M. 1979. Interaction Between Main Channel and Floodplain Flows. *J. Hydr. Div., ASCE*, **105(HY5)**: 573-588.
 19. Raudkivi, A.J., and Sutherland, A.J. 1981. Scour at Bridge Crossings, Bulletin No. 54, Road Research Unit, National Roads Board, New Zealand.
 20. Sellin, R.H.J. 1964. A Laboratory Investigation into the Interaction Between the Flow in the Channel of a River and that over its Flood plain. *La Houille Blanche*, **7**: 793-801.
 21. Shen, H.W., Schneider, V.R., and Karaki, S.S. 1966. Mechanics of Local Scour. Report No. CER66HWS22, Civil Engineering Department, Colorado State University, Fort Collins, CO.
 22. Shirole, A.M., and Holt, R.C. 1991. Planning for a Comprehensive Bridge Safety Assurance Program. *Transp. Res. Rec.* No. 1290: National Research Council, Washington, D.C., 39-50.
 23. Townsend, D.R. 1968. An Investigation of Turbulence Characteristics in a River Model of Complex Cross Section. *Proc. Inst. Civil Engineers (London)*, **40**: 155-175.

آبشستگی پایه‌های جناحی پل‌ها در رودخانه‌های با مقطع مرکب

چکیده

در گذشته اغلب مطالعات آزمایشگاهی پدیده آبشستگی اطراف پایه‌های جناحی پل‌ها در فلوم‌های با مقطع مستطیلی ساده صورت گرفته است. یکنواختی توزیع سرعت جریان و تنش برشی بستر در مقاطع عرضی، از مشخصه‌های بارز جریان در این مطالعات بوده است. این گونه مطالعات نمایانگر استقرار پایه‌های جناحی در مجرای اصلی رودخانه می‌باشد. در حالی که در عمل معمولاً پایه‌های جناحی پل‌ها در ناحیه میداین غرقاب رودخانه‌ها قرار می‌گیرند، جایی که انتقال جانبی اندازه حرکت سرعت جریان و تنش برشی بستر اثر مستقیم دارد. اما هنوز اثر این عامل و تأثیر شکل مقطع بر روی پدیده آبشستگی در مجاری با مقطع مرکب به طور جامع مورد بررسی قرار نگرفته است. در این تحقیق، مطالعه آزمایشگاهی اثر اندازه نسبی ذرات و اثر انتقال جانبی اندازه حرکت بر میزان آبشستگی اطراف پایه‌های جناحی که در میدان غرقاب قرار گرفته‌اند، صورت گرفت. عمق آبشستگی که تحت تأثیر انتقال جانبی اندازه حرکت به دست آمده با عمقی که بدون آن مشاهده شده است، مورد مقایسه قرار گرفته‌اند. در نسبت‌های کوچک اعماق جریان در میدان غرقاب به کل عمق جریان ($\frac{y_a}{H} \leq 0.3$) انتقال جانبی اندازه حرکت شدید می‌باشد. اندازه‌گیری‌های عمق آبشستگی در اطراف پایه‌های جناحی در این حدود از نسبت اعماق جریان، نشان داد که انتقال جانبی اندازه حرکت می‌تواند میزان آبشستگی را تا حدود ۳۰٪ افزایش دهد. این نتیجه نشان می‌دهد که نادیده گرفتن اثر انتقال جانبی اندازه حرکت در برآوردهای مهندسی می‌تواند به نتایج نادرستی منجر شود. از این رو برای برآورد صحیح میزان آبشستگی در چنین موقعیت‌هایی باید به گونه‌ای اثر این عامل اعمال شود. همچنین مطالعات Ettema, 1980; Chiew, 1984 در مورد اثر اندازه نسبی ذرات (نسبت قطر پایه به قطر ذرات) بر میزان آبشستگی منجر به تعیین حدی برای نسبت فوق شده است که ورای آن حد اندازه نسبی ذرات تأثیری بر میزان آبشستگی ندارد. این مطالعات عمدتاً برای پایه‌های میانی با سطح مقطع دایره‌ای صورت گرفته است. به دلیل کمبود اطلاعات در مورد اثر نسبی ذرات برای پایه‌های جناحی، در حال حاضر طراحان از حد وضع شده برای پایه‌های مستدیر میانی استفاده می‌کنند. اما نتایج تحقیق فعلی نشان می‌دهد که کاربرد حد مذکور برای پایه‌های جناحی مناسب نیست، رسیدن به یک نتیجه قطعی مستلزم مطالعات تفصیلی بیشتری می‌باشد.