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EROSION OF PLUNGE POOLS DOWNSTREAM OF DAMS DUE TO THE ACTION OF FREE-TRAJECTORY JETS

BY

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Erosion of plunge pools downstream of dams due to the action of free-trajectory jets

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Discharging flood water downstream of dams in the form of free-trajectory jets has become increasingly popular in recent years as a means of energy dissipation. On occasions, however, unforeseen and dangerous erosion of scour holes has occurred leading to costly remedial works. In this Paper the case histories of scour developments on selected prototypes are reviewed, summarizing features common to those case histories where scour has been of particular interest. The practical aspects of the hydraulic model testing of this form of scour are also discussed. By analysing sets of data from both model and prototype plunge pool developments, the Author presents formulae for calculating the probable depths of erosion under free jets, both on models and prototypes.

Notation

D	depth of scour from the tail-water level to the base of the scour hole, m
d	mean particle size of the eroded bed material, m
g	acceleration due to gravity
H	head drop from reservoir level to tail-water level, m
h	tail-water depth downstream of the scour hole, m
K	coefficient
q	discharge per unit width of the jet at the point of impact with the downstream tail water, m^2/s
v, w, x, y, z	exponentials for g, h, q, H and d respectively.

Introduction

The discharge of flood waters downstream of dams is always associated with the need to dissipate the hydraulic energy of the flow. One of the most common ways of accomplishing this in recent years has been by the use of free-trajectory jet dissipators such as flip buckets, ski jumps and free overfalls. The structures required are relatively compact and inexpensive as they simply guide the flow, directing the jet to some point of impact downstream, well clear of the dam and other permanent works. At the point of impact the jet will dissipate energy by excavating a scour hole in the river bed.

2. It is the erosive characteristic of free-trajectory jets about which least appears to be known, particularly in quantitative terms. On occasions the erosion of scour holes or plunge pools has taken place in an uncontrolled, unforeseen and dangerous manner. Furthermore, owing to the scale of the works frequently involved, remedial measures have been costly.

3. The Author carried out a survey of prototype scour developments, which had formed as a result of free-trajectory jet action, both to form a body of scour

Written discussion closes 16 July 1984; for further details see p. (ii).

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data and to examine those cases where remedial measures had been needed.[1] The case histories of six prototypes, where particularly interesting scour developments occurred, are presented.

Alder Dam (USA, 1945)[2]

4. The 100 m high Alder Dam has a 2265 m^3/s capacity spillway situated on its left flank. This terminates in a flip bucket which discharges flood waters on to an area of blocky andesite. The maximum recorded spillway discharge up until 1953 was 566 m^3/s.

5. Authorization for the Alder Dam power station was delayed due to the Second World War, and the subsequent extensive use of the spillway during the period 1945–1952 resulted in the erosion of a plunge pool approximately 30 m × 45 m × 24 m deep at the base of the hill below the bucket. The plunge pool was separated from the river proper by a narrow barrier of rock. Deterioration of this barrier, and the rapid regressive erosion of a fault zone leading back from the plunge pool to the bucket, obliged the owners to carry out remedial work. Furthermore, the deposition of the eroded material into the river downstream increased tail-water levels at the power station with an associated loss in power output.

6. Remedial work was carried out in 1952. The plunge pool was stabilized by grouting and by the use of anchor bars, the wide fault zone leading back to the bucket was backfilled with concrete and the barrier between the plunge pool and river also grouted and anchored. A weir was built across the barrier to add to stabilization and to maintain water levels in the plunge pool. About 6000 m^3 of mass and reinforced concrete were used in the repairs.

Nacimiento Dam (USA, 1957)[3,4]

7. The 82 m high Nacimiento embankment dam features a spillway and flip bucket on the left embankment. The spillway can accommodate a flow of 1472 m^3/s from an ungated crest supplemented by 142 m^3/s from two low-level gated intakes. It is sited along a ridge of sandstone which is flanked by mudstones and siltstones and which continues downstream of the flip bucket.

8. In February 1969 a flood occurred which reached a maximum depth over the crest of 1·07 m. In addition the low-level gates jammed open. The resulting flow along the chute was projected on to the sandstone ridge downstream from where lateral deflexion caused considerable erosion of the neighbouring mudstones and siltstones. On the right-hand side, continued erosion could have undermined the main embankment. The situation after the flood is shown on Fig. 1. Remedial works included:

(*a*) demolishing the existing flip bucket, extending the spillway chute by 83 m and constructing a new flip bucket at the downstream end
(*b*) reconstruction of fill either side of the extended chute with suitable material incorporating drainage layers and slope erosion protection
(*c*) excavation of a new stilling pool downstream to provide partial energy dissipation and guide the flow to the natural channel further downstream.

Picote Dam (Portugal, 1958)[5–7]

9. The river section of the 100 m high Picote arch dam houses flood-relief gates with a discharge capacity of 11 000 m^3/s. A tapering chute and flip bucket concen-

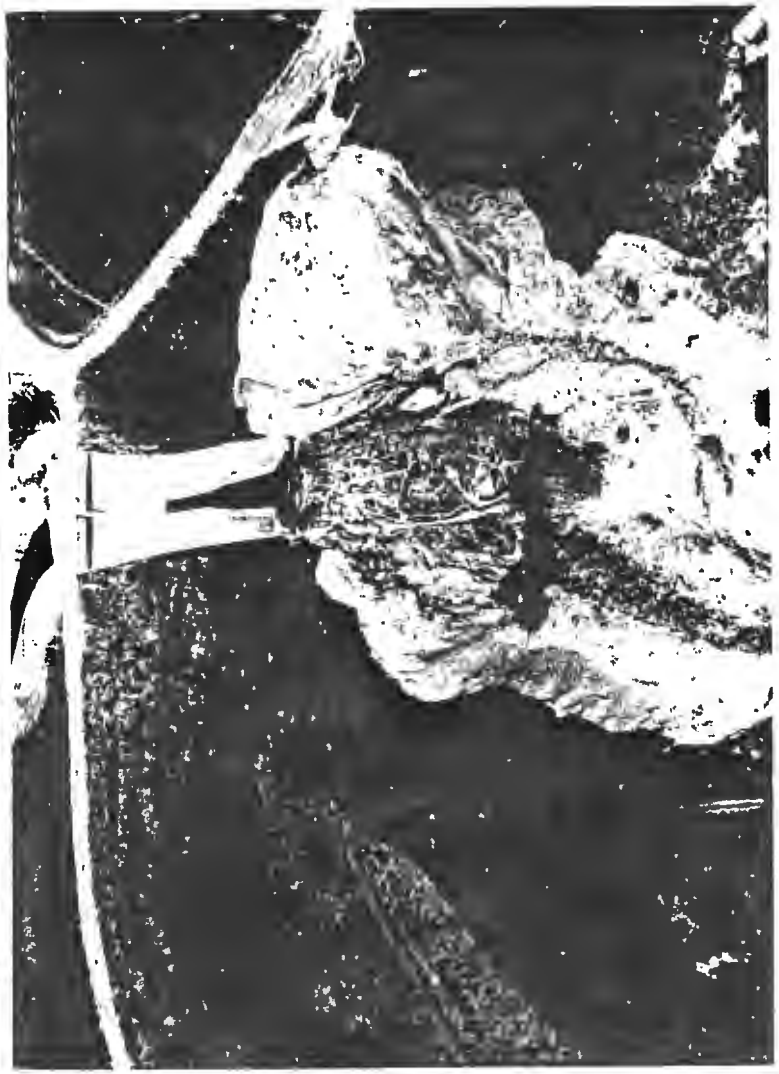

Fig. 1. The scour hole downstream of the Nacimiento dam spillway after the flood of February 1969

trate the flow to fit the narrow canyon downstream which is formed in excellent granite.

10. After a flood in 1962, a pit 20 m deep had been formed in the granite causing a 15 m high bar of eroded material downstream. The bar caused a significant reduction in head at the power station and an associated loss in output.

11. Remedial works included extending an existing diversion tunnel so that flows could be made to bypass the plunge pool and bar. Costs involved included the construction of the tunnel, the repairs in the river, the removal of the bar, the value of the electricity lost during the repair work (when no generation could take place), and the value of the reduction in output while the bar was in place. Not all these costs have been evaluated but the cost of tunnel construction alone was of the order of US $1 million.

Grand Rapids generating station (Canada, 1962)[8,9]

12. The 4000 m^3/s capacity spillway associated with the Grand Rapids generating station was completed in 1962. Immediately downstream of the gated crest a vertical curve projects the flow on to the local bedrock. This comprises an upper 3–5 m thick layer of massive, hard limestone, overlying a weaker zone which is highly jointed and stratified.

13. During the first four years of operation, the spillway was operated for much longer periods than had originally been anticipated due to delays in commissioning the power station turbines. Furthermore, gate operations were uneven, subjecting the rock to concentrated and asymmetrical flows. Following the removal of the upper sound layer, erosion of the lower, soft, layers proceeded rapidly. This undermined further blocks of the upper strata which fell into the plunge pool, acted abrasively against the softer layers, and further accelerated the scour.

14. The depth of scour, after the first four years of operation, was 30% more than that anticipated for the complete life of the structure. By the end of August 1967, it was decided that the scour had progressed to 'an extent which gave concern for the future safety of the works'.[9]

15. Remedial works involved extending the chute 20 m to the upstream face of the scour hole. At this point a dentated sill directed flows towards the downstream edge of the scour hole, protecting the bottom from further erosion. The scheme was implemented as the US $1 million estimated cost was significantly less than that for other alternatives.

Kariba Dam (Zambia/Zimbabwe, 1962)[10]

16. When it was first filled in 1962, the 130 m high Kariba arch dam impounded the largest man-made lake in the world. Flood openings discharge free jets to a point immediately downstream of the dam where the rock is sound gneiss.

17. When Kariba was designed in 1955 two power stations were planned, a 600 MW south bank station followed immediately by a 900 MW north bank station. The water extraction so produced would have reduced spillway usage to only once in five years.

18. By 1967 it became apparent that there would be a considerable delay in completing the second power station and the Consultants reviewed the effects of continued regular spillway usage. From 1962 to 1967 a plunge pool 50 m deep had been excavated downstream of the dam and almost 400 000 m^3 of rock removed, although bar formation downstream of the plunge pool had not been a problem.

19. The scour hole was closely monitored to ensure that its development did not threaten the stability of the banks and some underwater sealing of seams was undertaken on a regular basis.

20. A more immediate threat to the stability of surface material proved to be the spray associated with discharges from the flood gates. Spilling often took place for several months at a time and it was estimated that the disseminated spray effects arising out of the spilling cloud could amount to as much as 100 mm/day over a large part of the abutments.

21. As a result of the plunge pool development and surface bank slides, and in view of the continuing delay in the construction of the north bank power station, the 1967 Consultants' review recommended the construction of a bypass spillway to reduce the discharge requirements on the main flood openings. Preliminary estimates for such a bypass were of the order of US $4 million although much of this would have been recoverable in that the works could have subsequently been incorporated as part of any further power development.

22. In the event a decision was taken to proceed with the second power station and the bypass spillway was not built. The plunge pool continued to be monitored annually, with local repairs effected as necessary, and the banks were protected with extensive stone pitching.

23. The action of the jet and profile of the scour hole, as surveyed in September 1981, is shown in Fig. 2.

Tarbela Dam (Pakistan, 1975)[10–13]

24. The 143 m high Tarbela dam is one of the largest embankment dams in the world. It is sited on the River Indus in the Himalayan foothills of northern Pakis-

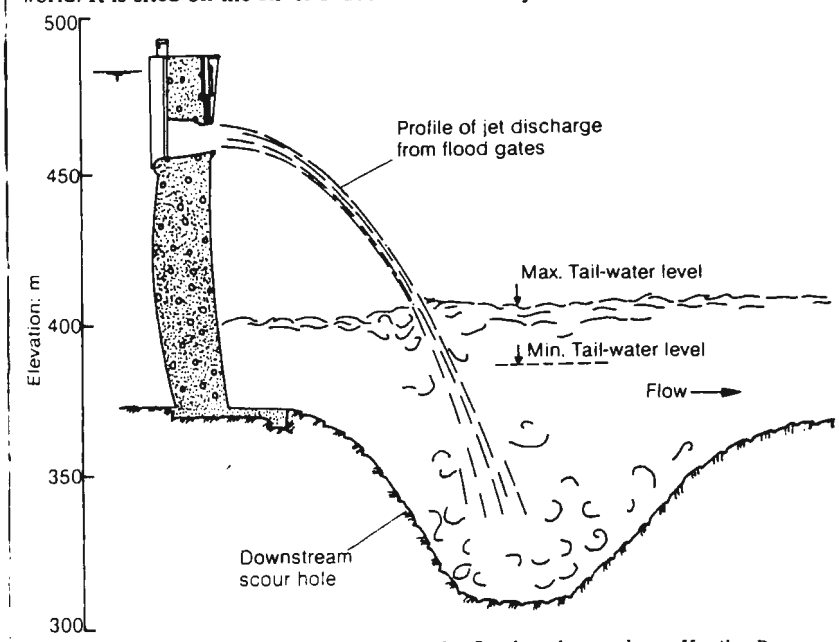

Fig. 2. The profile of the jet issuing from the flood outlet works at Kariba Dam and the extent of the downstream scour hole as at September 1981

Fig. 3. The scour hole downstream of the Tarbela Dam, service spillway at the end of July 1976

tan. Reservoir filling is rapid and each year considerable quantities of water are passed downstream via two spillways. These have a combined design capacity of 42 200 m^3/s.

25. The auxiliary spillway chute is much shorter than the service spillway chute, and the plunge pool that has developed downstream of its flip bucket is therefore much closer to the main dam than the one downstream of the service spillway. For this reason it was always intended that the service spillway would form the principal flood relief works with the auxiliary chute used for occasional relief and back-up.

26. The geology in the regions of the two plunge pools is similar in that both are sited in areas of weak siliceous limestone coupled with limestone interbedded with phyllites. The service spillway pool has the added complication of a dark band of hard igneous rock running obliquely across its downstream end.

27. Spilling via the service spillway commenced in August 1975. Lateral erosion of the plunge pool occurred, especially on the right-hand side, and developed dramatically on 12 June 1976 with a partial collapse of the steep slope bordering the pool. Approximately 400 000 m^3 of material were displaced. The development of the scour hole by the end of June 1976 is shown in Fig. 3.

28. As erosion continued and ground relaxation occurred, fears were expressed that the bucket structure might collapse into the plunge pool or be undermined by the return currents on the right-hand side of the pool. There was also considerable uncertainty about what limits the erosion would reach.

29. A programme of remedial works was embarked upon which lasted for several years. Work carried out included:

(*a*) stabilization of high slopes around the plunge pool to avoid major ground collapses
(*b*) post-tensioning the bucket structure into the rock immediately upstream, to ensure stability in the event of undermining
(*c*) lowering the level of the igneous intrusion at the downstream end of the plunge pool in the hope of reducing the return currents on the sides of the pool
(*d*) lining the sides of the pool with massive 'walls' of rolled concrete incorporating drainage galleries and stressed anchors.

30. Repair work to the service spillway and regular inspection during flood periods led to a greater use of the auxiliary spillway than had otherwise been envisaged. Erosion progressed in a similar way to that at the service spillway plunge pool, although without any dramatic or sudden rock collapses. In view of the uncertainties about the development of the erosion at the service spillway, it was decided to treat the auxiliary spillway plunge pool with similar caution.

31. The cost of repairs to the service spillway and its plunge pool have been in excess of US $120 million. Remedial works at the auxiliary spillway have cost in excess of US $90 million.

Summary of prototype data

32. A brief summary of prototype experience with trajectory jet plunge pool erosion is presented in Table 1. Table 2 considers the geology and repair costs of those plunge pools where remedial works have been necessary. These tables include the detailed case histories already presented. Comparing the details of the prototype erosions, various points emerge.

33. First, the costs involved in repair work can be considerable; this is inevitable given the size of some of the works. Also, there are often secondary costs. In the cases of both Alder Dam and Picote Dam the rubble bar which formed downstream of the scour hole raised tail-water levels and caused a corresponding loss in turbine power output.

34. Another correlation related to power production is that between Alder, Grand Rapids and Kariba dams. In each case the spillways were used far more than originally intended owing to delays in power station construction or commissioning.

35. Interestingly, four of the case histories tabulated mentioned gate operations in conjunction with scour. The scour at Nacimiento Dam occurred when the lower gates were jammed open, although as the main crest was ungated, some spillway flow would have occurred in any case. The excessive and rapid scour at Grand Rapids Dam was blamed, in part, on asymmetrical gate operations which subjected the plunge pool to unnecessarily concentrated flows. At Jaguara Dam similar asymmetrical gate operations led to rapid scour and restrictions on future gate operations, while at Ghandi Sagar Dam uneven scour downstream due to some gates being inoperable was corrected later by selective releases in unscoured regions. The last two cases above were not included among those where remedial work was necessary.

36. Geologically it can be seen from Table 2 that unacceptable scour is by no means limited to soft rocks such as shales, limestones and sandstones. In many of the cases quoted such scour has occurred in conjunction with hard igneous rocks such as andesite, granite and gneiss. Often rocks weighing several hundred tonnes

Table 1. A summary of prototype experience with trajectory jet plunge pool erosion

Dam	Year	Country	Remedial works	Comments
Ricobayo[14,15]	1933	Spain	Yes	Bucket splitters required to reduce plunge pool erosion
Bartlett[3]	1939	USA	Yes	Erosion downstream of bucket required extensive concrete protection to rock on two separate occasions
Alder[16]	1945	USA	Yes	Delays in power station construction led to heavy spillway usage and, in turn, considerable plunge pool erosion
Hirakud[17–19]	1956	India	—	Listed here as prototype scour data available; no repairs as of 1966
Maithon[17,18,20]	1957	India	—	Listed here as prototype scour data available; no repairs as of 1966
Ottenstein[21–23]	1957	Austria	Yes	Minor erosion of plunge pool
Nacimiento[3,4]	1957	USA	Yes	Uncontrolled erosion of plunge pool and banks led to extensive repair and to the elongation of the existing spillway chute
Picote[5–7]	1958	Portugal	Yes	Plunge pool erosion caused a rubble bar downstream of power outlet which raised tail-water levels locally and reduced power output; a bypass tunnel was constructed, the bar removed and plunge pool repaired
Panchet Hill[17,18,24]	1959	India	—	Listed here as prototype data available; no repairs as of 1966
Ghandi Sagar[17,18,21]	1960	India	—	Listed here as prototype data available; no repairs as of 1966
Grand Rapids[8,9]	1962	Canada	Yes	Delay in commissioning of power station led to heavy spillway usage and severe plunge pool erosion. The spillway works were re-modelled and extended
Kariba[10]	1962	Zambia/ Zimbabwe	Yes	Delays in power station construction caused prolonged and heavy spillway usage; this led to rapid plunge pool scour and bank instability due to spray

Table 1.— *continued*

Dam	Year	Country	Remedial works	Comments
Brazeau[25,26]	1969	Canada	—	Little used and no repairs necessary as of 1981; included here as prototype scour data available
Jaguara[27]	1971	Brazil	—	Rapid plunge pool erosion due to uncontrolled spillway operation; no repairs needed as of 1981, but restrictions placed on spillway gate operations
Cabora Bassa[28]	1974	Mozambique	—	Included here as prototype scour data available; regular scour pool monitoring has taken place though no repairs have been needed as of 1981
Tarbela[10–13]	1975	Pakistan	Yes	Prolonged use of the high head spillways, with plunge pools in areas of weak rock, led to extensive lateral erosion; considerable amounts of remedial works undertaken

Table 2. Plunge pools where remedial works have been carried out—summary of geology and indications of cost

Dam	Remedial work costs at present day worth, US $ million	Plunge pool geology
Ricobayo	Considerable	Sound igneous rock
Bartlett	Moderate	Granite
Alder	Moderate (6000 m^3 of concrete)	Blocky andesite
Ottenstein	Minor	Granite
Nacimiento	6·0	Sandstone, mudstone and siltstone
Picote	4·0 (tunnel costs only)	Granite
Grand Rapids	4·0	Limestone and dolomite
Kariba	Moderate	Gneiss
Tarbela (Service chute)	> 120·0	Siliceous limestone and limestone with phyllites
(Auxiliary chute)	> 90·0	

have been displaced by the flow.

37. Lastly, in three cases problems appear to be linked with variations of rock strength within the plunge pool area. On Nacimiento Dam a sandstone ridge in the centre of the plunge pool deflected the jet, causing the preferential erosion in the mudstones and the siltstones either side. At Grand Rapids, scour in the soft lower levels of the plunge pool was accelerated by abrasion due to blocks from the harder strata above. At Tarbela Dam it has been argued that the rate of scour within the service spillway plunge pool was enhanced by a band of igneous rock downstream effectively containing the flow, also that the differentially large amount of scour on the right side of the pool was caused by the oblique orientation of the band.

Hydraulic model testing of plunge pool scour

38. In every case encountered by the Author the scaling law used for the model investigation of plunge pool erosion was the Froude law. It is gravity forces which predominate in governing the formation of, and force from, free-trajectory jets, and also gravity forces which the water has to overcome in ejecting eroded material from the scour hole. One implication of Froude law scaling is the requirement for a minimum size of model bed material. The surface of a prototype scour hole in rock will be rough and the flow at this boundary fully turbulent. In order for this condition to be reproduced on the model, a minimum size bed material of about 3 mm should be used.[29,30] A smaller material will be affected by viscous forces within the boundary layer and scaling will therefore be effected by the Reynolds number.

39. The most common problem in modelling scour downstream of trajectory buckets is representing the geology of the site at a model scale. A typical approach is to examine the rock on site and to estimate, from joint and fissure patterns and from the strength of the rock, the individual block size to which the rock will break down. This is then represented on the model by an equivalent size of gravel. The gravel is often left as a non-cohesive, loose material to give a 'worst-case'. Such tests give a reasonably close approximation of the likely depths of scour which will occur on the prototype, though they greatly over-estimate the extent of scour horizontally, the loose gravel being unable to stand at the near vertical slopes which can be expected from rock.

40. Various methods have been tried to overcome this. One series of model tests into scour downstream of Massira Dam[31] used small concrete cubes and blocks as a model bed material. This was based on the work of Martins[32] who considered that closely packed blocks were the best way of simulating fissured rock at a model scale. As the blocks were stacked uniformly, with their faces touching, the edges of scour holes could stand vertically.

41. In practical terms, carefully re-stacking a bed of loose blocks after each test is very time consuming, while the equal size of block needed to make re-stacking practicable does not correspond to the varying and irregular sizes which occur on prototypes. Therefore an alternative, and more usual, way of representing rock on a model is to bind loose gravel together with a material, or materials, which give it cohesion. Such a material must be sufficiently open to allow internal hydrodynamic pressures to develop in the same way as would be possible within a fissured rock mass. The Kariba Dam model tests[33] tried various 'binders' such as clay, cement, grease and paraffin wax. However, a common problem was that the cohesive properties changed with time; cement hardened while paraffin mixtures gradually disintegrated under water. The material finally selected was a compacted mixture of gravel (71%), clay (21%) and water (8%). The Author has been involved with the successful use of a similar mix on the model testing of scour[34] and found that vertical sides to scour holes can be easily obtained while depths of scour in the

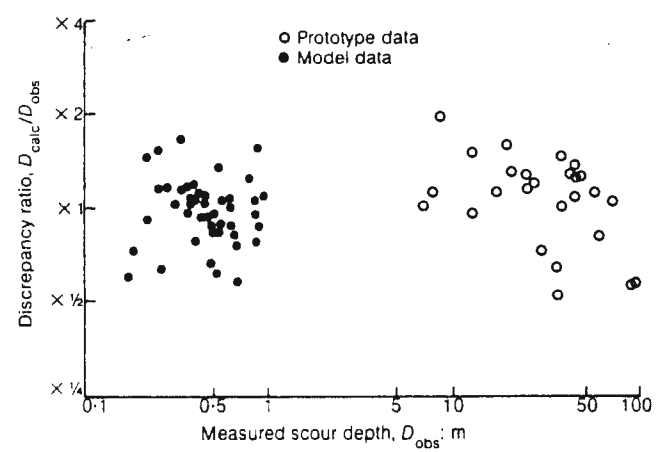

Fig. 4. Comparison of predicted and observed scour depths using the Author's comprehensive formula

cohesive material compare closely with those using only loose gravel. This indicates that although the binder is producing a more realistic geometry to the scour hole, it is not affecting the principal erosion process immediately under the jet.

42. A disadvantage with clay-bound gravel is that the testing water rapidly becomes opaque. It has to be separated from the supply to the rest of the laboratory and makes observation of the erosion process difficult. Johnson[35] has suggested an alternative mixture consisting of gravel (69·6%), chalk powder (11·6%), cement (0·25%) and water (18·55%). A similar material consisting of crushed gravel and sand (79·8%) chalk powder (13%), cement (1·2%) and water (6%) was used on the El Cajon Dam model tests.[36] Tests were carried out 4 h after the material had been mixed and placed.

43. It is worth noting a calibration technique for model bed materials which has been used in the cases of several dams such as Picote, Kariba and Tarbela. Following the first few years of prototype operation, the prototype flows which have occurred are repeated on a model. Various bed materials are tested and the one which most closely simulates the scour patterns and depths which have occurred on the prototype is used in further tests to assess the probable maximum extent of prototype scour.

Calculation of plunge pool scour depths

44. It has been proposed by many workers[37–40] that the depth of scour D under a jet is a function of unit flow q, head drop from reservoir level to tail-water level H and the particle size of the bed material d. It has also been suggested[32] that it is a function of tail-water depth h. If the acceleration due to gravity g is included to give a dimensional balance, the general form of such a relationship can be assumed as:

$$D = K\,\frac{q^x H^y h^w}{g^v d^z}$$

45. In order to examine this, the Author assembled 26 sets of data from the prototype scour developments listed in Table 1 and supplemented these with a further 47 sets of data from various hydraulic model tests of scour holes.[1] The model depths of scour ranged from 0·071 m to 1·175 m with head drops ranging from 0·325 m to 2·150 m. The prototype depths of scour ranged from 6·70 m to 90·0 m with head drops ranging from 15·82 m to 109·0 m.

46. Depths of scour were calculated for the measured values of q, H, h and d, and compared with those values of scour depth which had actually occurred. The values of the coefficient K and exponentials w, x, y and z were varied in turn to optimize the accuracy of the results and the value for the exponential v selected to complete a dimensional balance. For models it was found that the optimal values were $K = 3{\cdot}27$, $v = 0{\cdot}30$, $w = 0{\cdot}15$, $x = 0{\cdot}60$, $y = 0{\cdot}05$ and $z = 0{\cdot}10$. Thus for models the expression for scour depth becomes

$$D = 3{\cdot}27\,\frac{q^{0{\cdot}60} H^{0{\cdot}05} h^{0{\cdot}15}}{g^{0{\cdot}30} d^{0{\cdot}10}}$$

This expression has both a dimensional and Froude law balance. It can therefore be scaled according to the Froude law without error. When considering the ratios of calculated to measured scour depths, the above formula gave an average of 1·00 and a coefficient of variation of 25·4%. However, in spite of the formula's satisfactory nature with regard to model result accuracy, dimensional balance and Froude scaling error, it was found that it did not give the best results for prototype scour.

47. In order for the formula to function acceptably for both models and prototypes the following values were found to be necessary:

$$K = 6{\cdot}42 - 3{\cdot}10\,H^{0{\cdot}10}$$

$$v = 0{\cdot}30$$

$$w = 0{\cdot}15$$

$$x = 0{\cdot}60 - H/300$$

$$y = 0{\cdot}05 + H/200$$

$z = 0{\cdot}10$ (with an assumed constant value for d of 0·25 m for prototypes).

These revised values for K, x and y produce a formula which gives, for models, an average ratio of calculated to measured scour depths of 1·01 and a coefficient of variation of 25·3%. For prototypes the ratios of calculated to measured scour depths had an average value of 1·07 and a coefficient of variation of 30·1%. The coefficient of variation for prototypes is kept to a minimum by the modifications to x and y. The mean values for prototype scour compared with those for models are obtained by the modification of K. The need to modify K, x and y from those values which gave the best formula for models, suggests that the relationship between scour depths in model and prototype plunge pools is a function of factors more complex than Froude law scaling alone. The increased air entrainment in prototype flows compared with those on models is one such probable factor.

48. Figure 4 illustrates the accuracy of both model and prototype data points, processed using the above comprehensive formula. The plot follows the form used by White *et al.*[41] in their analyses of the accuracy of formulae for a similar scour-related topic.

49. It should be noted that the coefficient of variation for prototype results is only slightly worse than for models. This is in spite of prototype flood flows being of a much more variable and transitory nature than the controlled conditions possible with models. This would suggest both that prototype scour development is rapid and that the concept of an ultimate depth for prototype scour is valid for all practical purposes.

Acknowledgements

50. The review of prototype and model data was carried out in part fulfilment of a Master of Philosophy Degree at The City University, London, and the Author would like to thank Professor P. O. Wolf and Dr K. Arumugam for their help and guidance. The scour formulae above were developed by the Author subsequent to this work but followed a detailed review of the accuracy of formulae by others also carried out as part of the same degree course and which is the subject of a separate paper.[42]

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