DAM FOUNDATION EROSION

SURVEY OF LITERATURE

AUGUST 6, 1993

BY

- HYDRAULICS BRANCH, RESEARCH & LABORATORY SERVICES DIVISION, US BUREAU OF RECLAMATION
- HYDRAULICS PROGRAM, DEPARTMENT OF CIVIL ENGINEERING, COLORADO STATE UNIVERSITY
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This report fulfills the requirements for Task Agreement: Literature Survey - Erosion Characteristics of Dam Foundations under Master Agreement No. Z-19-2-196-91 between PG&E and Reclamation. A Bureau of Reclamation R-series report will follow.

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PAP file

D-3570

AUG 13 1993

Mr. Kiran K. Adhya
Senior Civil Engineer
Pacific Gas & Electric Company
One California Street, Room 1736B
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Subject: Submittal of Final Report - Dam Foundation Erosion, Survey of Literature Dated August 6, 1993 - Master Agreement No. Z-19-2-196-91 (Hydraulic Research, Cooperative Agreement)

Dear Mr. Adhya:

Enclosed are five copies of the final report entitled "Dam Foundation Erosion - Survey of Literature" compiled by Rod Wittler of my staff. Submittal of this report completes the Task: Literature Survey - Erosion Characteristics of Dam Foundations signed December 30, 1992, under the Master Agreement No. Z-19-2-196-91 between Reclamation and Pacific Gas & Electric Company.

We value our client's comments on the services we provide and are committed to providing quality products. Your assistance with completion of the attached Client Feedback Form will provide a means to encourage and/or correct areas of our service that need special attention.

We have appreciated the opportunity of working with you and your staff on this task and look forward to continued cooperation on the Dam Foundation Erosion Research Project.

Sincerely,

STANLEY L. PONCE

Stanley L.Ponce
Chief, Research and
Laboratory Services Division

Enclosures
(On file in D-3751)
Copies to persons on attached sheet.

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DAM FOUNDATION EROSION

SURVEY OF LITERATURE[1]

INTRODUCTION

Regulatory agencies require owners of dams to periodically relicense their dams. The owners must demonstrate the safety of the dam during a Probable Maximum Flood (PMF) as part of the dam licensing process. A dam will overtop when the inflow exceeds the storage capacity of the reservoir and flow capacity of existing spillways and outlet works. Additional reservoir storage, spillway capacity, or a plan for overtopping of the dam usually corrects this dam safety deficiency. Designing for dam overtopping requires a stability review of the structure under overtopping conditions. Potential erosion of the dam foundation and abutments is one factor to consider in a stability analysis. Current erosion prediction formulas do not track erosion as a function of time, and have limited application in hard-rock or cohesive foundation materials. A predictive tool for estimating the progressive erosion of dam foundations is necessary. This survey of literature is one of two documents planned for a research study on the characteristics of dam foundation erosion. The second document is a Pre-Test report.

Survey of Literature

The survey of literature provides background on the subject and a basis for identifying areas of erosion prediction that need improvement. Predicting and modeling dam foundation erosion is the objective of a proposal for a joint research project. The parties in the research proposal are Pacific Gas and Electric Company (PG&E), Colorado State University (CSU), and the US Bureau of Reclamation (USBR).

Pre-Test Report

A Pre-Test report based upon the information presented in this survey of literature will follow. The purpose of the Pre-Test report is to define the physical processes of dam foundation erosion. The Pre-Test report will present plans for studying and modeling the physical processes of dam foundation erosion such as hydrology, jet characteristics, erosion, transport, and hydraulics. The Pre-Test report will include the scaled and prototype model designs, instrumentation designs, variables, and the schedule for the remainder of the project.

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[1]Figures, equations and tables referenced in quoted text that are not included in this report are indicated by an *.

Background

Dam overtopping occurs in two distinctly different situations. The first is water overtopping the dam and forming a free trajectory jet impinging at the foundation. The second is water flowing over the downstream face of the dam. A free trajectory jet is typical of steep faced concrete dams, while a supported jet on the downstream face is typical of embankment dams.

The format of this survey is narrative. The narration discusses each citation and the conclusions of the author. Discussion focuses on the relation of the reference to dam foundation erosion and where appropriate the limitations, advantages, and disadvantages of formulas, model studies, prototype studies, and prediction methodologies.

The survey is a compilation of many references. The survey cites and discusses a portion of the references. The survey includes uncited references for the sake of completeness. Following the uncited references is a table that includes authors abstracts from both cited and uncited references. Not all references have an authors abstract.

The contributors to this survey include Rodney Wittler and Brent Mefford of the Hydraulics Branch, US Bureau of Reclamation, Dr. George Annandale of HDR Engineering Inc., Prof. James Ruff, Prof. Steven Abt, and Julio Kuroiwa, of the Hydraulics Program, Colorado State University. Note that portions of this survey are extracted from a recent Reclamation report produced by Harza Engineering discussing Roosevelt Dam plunge pool design (48).

SUMMARY

Prediction of dam foundation erosion is an extremely complex problem. However, a great body of work exists shedding light on many facets of the problem. This survey attempts to highlight the most important work in the areas of overtopping, plunge pool scour, and culvert scour. Since overtopping erosion is similar to erosion in a plunge pool, or scour below a culvert, the subjects combine to provide a wide variety of prediction formulae and techniques.

Upon review of the literature cited in this report, a wealth of erosion data and erosion prediction algorithms are available to the designer. However, the technical approaches fall short of what is needed to tackle the problem of estimating erosion of dam foundation materials subjected to overtopping flow. Perhaps the limits of current erosion prediction methods and the need for better methods were best stated by two respected researchers, Mason and Spurr.

Spurr (57) summarizes the problems with current foundation erosion prediction schemes:

Thus the discusser contends that time must be considered together with the unique hydraulic and geological processes existing at each site, the amount of surplus energy contained by a given jet at impaction over and above the threshold resistance of the bedrock to scour, the shape of the sides of the scour hole, and the size of the downstream bar in order to determine the plunge pool's maturity before any meaningful comparison can be made. This is specifically true when predicting scour in any plunge pool which does not respond as though its bed material were essentially non-cohesive.

Mason writes,

The quantification of plunge pool scour rates, in conjunction with varying rock types is an area where much useful research remains to be done."

This survey focuses on the following factors: erosion, jet aeration, angle of impingement, and erosion extents. Erosion is dependent upon geotechnical and geomechanical properties of the foundation materials. The literature contains many formulas for estimating the ultimate depth of erosion. The majority of the formulas neglect the progressive nature of the erosion and instead concentrate upon ultimate scour depth. Most formulas also ignore aeration of the jet and the jet impingement angle.

The best formulas for estimating plunging jet scour, probably the formulas by Mason, have very narrow application. Mason acknowledges that accuracy greater than 70% is not readily attainable. Case studies reported herein show wide variance in the accuracy of the predicted and actual depths of scour. The reasons for the lack of accuracy are model specific formulas, site specific application, fragmented results from multiple studies, and the factors of geology and cohesive material properties. Current formulas have only a rudimentary capability of predicting progressive erosion, that is why the extents vary with time. This capability is crucial for predicting erosion due to multiple overtopping events, each likely less than the PMF.

This survey documents three techniques for evaluating erosion due to an impinging jet:

- Techniques that predict the erodibility of earth material
- Techniques that estimate the depth of erosion, and
- Techniques that attempt to relate space and time in estimating erosion.

Techniques For Predicting the Erodibility of Earth Materials

Methods by Cameron (13), van Schalkwyk (66) Barton (106) and Kirsten (35) result in estimates of the erodibility of earth material. Earth material erodes when the erosive power of

the impinging water exceeds the ability of the material to resist erosion. Erodibility is therefore a threshold condition.

Techniques For Estimating the Extents of Erosion

Mason and Arumugam (41) list 31 methods of calculating scour depth, and divide these into five groups. The first group, 17 equations, relates scour depth to discharge, head drop and characteristic particle size. The second group, 2 equations, adds the impact of tailwater depth. The third group, three equations, is empirical in nature, and relates estimated scour depth to jet dimensions and characteristics. The fourth group, eight equations, developed by six Russian authors, relate scour depth to drop height, particle diameter, tailwater depth and the angle with which the falling stream enters the downstream area. The fifth group, one equation, is for equations that include a time parameter. Mason (42) also deals with the impacts of air entrainment on plunge pool scour in a later publication. The equations and associated coefficients of Mason and Arumugam (41) and Mason (42) are currently state of the art.

Culvert scour relates closely to plunge pool scour. The primary differentiation is the energy level and the angle of the flow at the point of impingement. Jets generally impinge in plunge pools at an angle nearer to vertical than horizontal, while flow from culverts generally impinges at an angle close to horizontal. Similar development of parameters and scour prediction methods are available for culvert scour. Found in the culvert scour literature is much of the work addressing erosion of cohesive materials and the importance of material properties.

Techniques For Estimating the Progressive Extents of Erosion

The method recorded in the paper of Mason and Arumugam (41) relates the depth of scour to time. The few attempts, made with the use of an explicit equation, to account for the progression of erosion as a function of time have not been too successful. However, many authors postulate the importance of erosion progression and case histories of Kariba dam (81), Tarbela dam (39)(77)(116), Guri dam (23), and Seven Mile dam (58) support this postulation.

PART I EROSION PREDICTION TECHNOLOGY

This first part of this survey concentrates on formulas and methods of predicting erosion and erosion extents. The foremost author in this area is Mason. His writings are extensive and he commands a great deal of respect in this area of expertise. Part II concentrates on classification systems of the materials being eroded. The purpose of the second part is to provide a basis for correlating a material index to hydraulics and erosion.

Techniques for Estimating Depth of Erosion or Scour

The terms erosion and scour are interchangeable throughout this survey. There are two situations where a considerable body of literature exists on scour depth from an impinging jet. These two situations are plunge pools and culvert outlets.

Plunge Pool Scour

The best known and simplest equation is from Reclamation's "Design of Small Dams" (21), which recommends use of the Veronese (67) equation.

$$Y_S = 1.32\ H^{0.225}\ q^{0.54}$$

Y_S = depth of scour below tailwater (in feet).
H = effective head (in feet).
q = unit discharge (in ft^3/s-ft).

This equation is intended to produce limiting scour depth for relatively fine granular tailrace material. Veronese developed the equation based upon scour tests similar to those by Scimemi (52):

> *When formulas such as Equation* 2 cannot be used for the design of hydraulic structures, at locations where the tailrace is subject to scour, and where it is not possible to reduce the model material to the relative scale of the prototype material, observations to determine the maximum scour, or scouring limit, can be made by another experimental method. This consists in making successive observations on the model with smaller and smaller grain sizes without regard to the grading of the prototype material. Trials must be begun with rather large gravel and then repeated with successively smaller gravel. The scour will be deeper and deeper with each reduction in grain size until a limiting depth is soon reached.*

This method should lead to good results for any type of hydraulic structure, without necessarily conforming to any reduced, in scale, of the prototype material, and of its grading curve (if available).

The Veronese equation was obviously intended for vertical, or near vertical falling jets, such as might occur at a natural waterfall or an over fall spillway near the top of a high arch dam. Another common application of the plunge pool energy dissipater occurs with the flip-bucket type spillway, where the jet angle of entry into tailwater is much flatter than vertical. Coleman (17) found that based on physical models, using granular, non-cohesive tailrace material, the Veronese equation was still applicable with the addition of sin α to the equation, as shown on Figure* 1. Coleman concluded:

"Correlation of model results with the Veronese equation show reasonable agreement for both magnitude and location of maximum scour, if the angle of jet entry into tailwater can be predicted. The Veronese prediction of scour depth should be measured in the direction of the jet entry. This technique can be expected to apply where tailrace material is relatively fine and uniform. In the prototype, varying quality of geologic material and bedding planes make prediction much more uncertain."

Schoklitsch (22) produced an equation similar in form to Veronese, except that the size of tailrace material was introduced, as:

$$h_s = \frac{3.15}{D_n^{0.32}} H^{0.2} q^{0.57}$$

in which the depth of scour h_s (in feet) is measured from the water surface over the scour hole to the bottom of the scour hole; D_n is the diameter of the bed material (in millimeters) such that only 10 percent is coarser; H is the height of drop (in feet) measured from upstream to downstream water surfaces; and q is the discharge (in ft^3/s per foot length of crest of drop structure).

Thomas (61) made model studies for comparison with Schoklitsch and found that:

"Of particular importance, is the fact that the Schoklitsch equation gives no consideration to the variable of time. Therefore, the research reported herein shows that it is possible for the depth of scour as evaluated by Schoklitsch to be exceeded if sufficient time is given. Despite this important limitation, it compares favorably with the data taken in which time was considered as a variable. This comparison is given in Table 1*

where it may be seen that the agreement is good for the smaller depths of scour. For the greater depths of scour and more active scouring conditions, however, the equation of Schoklitsch predicts a scour depth only half as great as that which actually occurred. This comparison clearly demonstrates the need for considering the influence of time upon the depth of scour."

Mason (41) compared the accuracy of 25 formulas for predicting maximum scour depth, based on 47 sets of data from models and 26 sets of data from prototypes. Results are shown on Mason's Table 3.

Table 3. - Statistical Analysis of Results

Group	Equation	x_c Model	V% Model	x_c Prototype	V% Prototype
I	Schoklitsch	1.0887	34.93	0.5796	42.98
	Veronese (A)	0.8114	39.32	0.2697	41.63
	Veronese (B)	1.1250	31.90	1.4171	41.63
	Eggenburger	4.0154	39.38	6.6812	35.13
	Hartung	2.4735	35.68	3.8987	39.83
	Franke	4.0332	41.54	7.7815	37.68
	Damle (A)	0.4438	38.38	1.2724	33.16
	Damle (B)	0.3698	38.41	1.0596	33.16
	Damle (C)	0.2465	38.36	0.7064	33.16
	Chee and Padiyar	1.1878	30.21	2.3093	46.41
	Bisaz and Tschopp	1.4665	38.60	1.8553	40.60
	Chee and Kung	1.2004	29.79	1.4472	43.22
	Martins (B)	0.7520	29.48	0.8538	47.19
	Taraimovich	0.2689	32.28	0.8644	44.23
	Machado	1.1706	34.29	1.3406	38.31
	SOFRELEC	1.1529	29.47	1.3091	47.19
	INCYTH	0.9320	33.29	1.0010	40.19
II	Jaeger	1.0020	30.62	1.3973	39.00
	Martins (A)	1.3051	26.21	0.5743	46.51
III	Cola	1.6715	46.47	2.4174	86.04
IV	Mikhalev	2.6828	66.16	1.6742	150.95
	Rubinstein	1.5890	40.43	4.1149	100.55
	Mirtskhulava	0.9485	57.82	2.2371	66.98
V	Thomas	1.6100	41.89	7.6896	326.79

x_c = mean error D calculated/D measured; V = coefficient of variation.

From this analysis, Mason concluded that:

"The mean values of $\overline{x_o}$ and V for each formula are given in Table 3 for both models and prototypes. It is clear that in all Group I cases, the coefficient lies between about 30% and 50% with, as might be expected, a tendency towards higher values on the prototypes. Clearly from the results, the lowest coefficients of variation, and thus the best forms of equation are those by Martins - (B) (and thus SORFELEC) and Chee and Kung (15) for models and Damle for prototypes. Interestingly, the best model formulas are noticeably bad for prototypes.

The results of the analyses of Jaeger's and Martins -(A) Group II formulas are good at model scales, with Martins - (A) formula potentially better than any of those from Group I. For prototypes, however, both are relatively poor.

The Group III formulas are the two simple formulas of Cola and Davis and Sorenson. The results show that neither have much to commend them.

The three Russian formulas, taken as Group IV, for all their complexity, gave generally poorer results than those of any preceding group. At a prototype scale, none of the Group IV formulas would be of any real value.

The Group V equation by Thomas gave coefficients of variation for the results of 41.89% for models, but 326.79% for prototypes; the latter being principally due to one dramatically high overestimation of prototype scour. The high prototype result cannot be ignored, however, as the same scour data was processed by all the other formulas without producing such an anomaly."

Dimensional graphs for predicting scour hole dimensions caused by jets were developed by Rajaratnam (48), including maximum scour depth. Mason did not include this author in his comparison of formulas because no explicit formulas were given. Mason (41) then developed his own equation to fit the same data sets used to test the formulae of Table 3, as follows:

"The proceeding discussions lead to the form of expression

$$D = K\frac{q^x H^y}{d^z} \tag{18}$$

as being the most satisfactory for estimating the depth of scour under a jet while retaining the benefit of simplicity. It is also clear, however, that the inclusion of tailwater depth h as an additional factor may enhance accuracy. It can be seen from substituting the results of analyses using Jaeger's formula (32) into Figs. 1 and 2 that the exponentials used by Jaeger would not alone have given the potential accuracy which his formula achieved. It can only be assumed that the improvement stemmed from his inclusion of tailwater depth as an additional factor. A new formula was, therefore, developed by Mason incorporating h. The acceleration due to gravity, g, was also incorporated making possible a dimensional balance and giving the form of expression

$$D = K\frac{q^x H^y h^w}{g^v d^z} \tag{19}$$

This basic formula was used to process the same sets of data as before, and the appropriate values of x, y, w and z were obtained by varying each in turn to minimize the coefficient of variation of the results. The value of v was then chosen to give an overall dimensional balance, and K was chosen to give a mean value for $\overline{x}_c$ close to unity. Using this approach for the model data only yielded the equation

$$D = \frac{3.27 q^{0.60} H^{0.05} h^{0.15}}{g^{0.30} d^{0.10}} \tag{20}$$

In this expression d_m was used rather than d_{90} .

It was particularly pleasing to discover that the above formula also gave a Froude number balance so that

$$1.5x + y + w - z = 1.0 \tag{21}$$

For model data, the formula gave an $\overline{x}_c$ value of 1.00, and a coefficient of variation of 25.4%. As might be expected from the earlier work, however, the formula did not give very good results for prototype data. The previous studies had shown the need to vary the exponentials x and y to give low coefficients of variation for both model and prototype results. In order to maintain a Froude number balance this was done so that the variation in the value of x always equaled two-thirds of the variation in the value of y. Various expressions were tried; the ones finally selected were those which produced the best optimization for the coefficients of variation for both model and prototype data. In order to obtain acceptable $\overline{x}_c$ values for both models and prototypes, however, it also proved necessary to use a variable value for K. This was obtained by an essentially trial and error process. For a complete model and prototype expression it was found that: $K = (6.42 - 3.10H^{0.10})$; $V = 0.30$; $W = 0.15$; $x = (0.60 - H/300)$; $y = (0.15 - H/200)$; and $z = 0.10$ (with an assumed constant value for d of 0.25 m for prototypes).

These values produced coefficients of variation for model results of 25.3% and for prototypes 30.1%, and corresponding values for $\overline{x}_c$ of 1.01 and 1.07, respectively."

Mason (41) further concluded that his proposed equation could be applied broadly, as:

"It can be applied within the ranges of head drop H from 0.325 to 2.150 m for models, and 15.82 to 109.0 m for prototypes: These represent the data range from which the formula was derived. For the same reason, it can be applied to the cohesive and non-cohesive granular model bed materials with mean particle sizes d_m from 0.001 to 0.028 m. In the case of prototype bed rocks, it can be applied to all normally encountered rocks

from sandstone, shale and silt stone through to quartzite, gneiss and granite, by assuming a mean, equivalent particle size d_m of 0.25 m. It is not recommended that any allowance be made for either jet impact angle or chute friction loss, as the studies produced some inconsistency between the model and prototype results of such allowances.

The accuracy which can be expected from the use of Equation 19 can be assessed by the coefficients of variation of approximately 25% and 30% and values of $\bar{x}_c$ of 1.01 and 1.07 obtained for model and prototype data respectively."

Mason (42) made further model studies in an attempt to determine the effect of air concentration on scour depth. His studies produced a new equation based on the following:

"A series of two-dimensional laboratory tests were carried out by the writer in which the percentage of air entrained in the plunge pool could be varied to assess its effect on scour, along with the effects of q and H. It was found that in the absence of entrained air, the scour depth was solely dependent on q and independent on H. It was reasoned that the scour may in fact be dependent on only q and the percentage of entrained air.

It was further concluded that the degree of aeration produced by previous authors' models may have affected previously proposed (exponents for q and H). Since aeration is not only a function of head drop, but of other factors like jet thickness, internal turbulence, and impact angle, it can be seen that not allowing for, or measuring aeration as a separate parameter would have produced variable results.

*The results of the tests were explained in terms of a characteristic force on the particles of bed material where plunge pool depth varies with flow, and with the percentage of entrained air needed to maintain that force. The equations produced were developed to give a three-dimensional scour approximation. This estimate was shown to reasonably fit a body of data from model tests of dam plunge pools. It was further shown to give a reasonable upper bound to a body of prototype scour measurements. It is concluded that plunge-pool scour depth can be calculated from the Equation**

$$D = \frac{3.39 q^{0.60} (1+\beta)^{0.30} h^{0.16}}{g^{0.30} d^{0.60}} \qquad (16)$$

where β is calculated from Equation 2

The expression for β was quoted earlier in Mason (42), and introduced as follows:

"The most relevant expression for sheet or rectangular jets, typical of spillway releases, is that proposed by Ervine (24). He related the volumetric air to water ratio β in the plunge pool to the jet fall height H, the jet thickness t at impact, the jet impact

velocity v, and the minimum jet velocity V_e required to entrain air. The expression he proposed was

$$\beta = 0.13(1 - V_e / v)(H / t)^{0.446} \tag{2}$$

This is not unlike some expressions proposed for circular jets (van de Sande (64); Bin (8). Equation 2 was therefore used to establish how much air to introduce into the plunge pool in order to simulate how much air a given jet, falling a height H, would entrain naturally. The value taken for V_e was 1.1 m/s (Ervine (24)).

It is concluded that Equation 16 represents a new form of expression for calculating scour under jets, where air entrainment replaces the use of H as a factor defining the scour process. The expression seems to be applicable to models and prototypes and in the case of the former is at least as accurate as, and probably more accurate than, other expressions developed to date (Mason and Arumugam (41)). It is also concluded that any further work on similar scour formulas should incorporate any allowance for air entrainment if the results of such work are not to be essentially flawed.

Equation 16 can be approximated by the writer's earlier expression:

$$D = \frac{3.27 q^{0.60} H^{0.05} h^{0.15}}{g^{0.30} d^{0.10}} \tag{17}$$

where it is now assumed that the factor $H^{0.05}$ in fact equals an approximation to the effect of air entrainment in the plunge pool.

One last interesting conclusion from this study is that air entrainment on prototypes may not be significantly different from that encountered on reasonably sized models, given that there may be an upper limit on β of around 2-3 and that such figures can easily be approached on models."

Chee and Padiyar (14) made model studies to determine scour depths resulting from 30° flip buckets, using uniform granular tailrace materials. An additional, and potentially more useful part of their study was the development of a generalized scour hole geometry. The result was as follows:

"The shape of the hole in plan is symmetrical about the longitudinal axis with the deepest point located downstream of the geometrical center. To define the horizontal and vertical dimensions of the hole, the deepest point is taken as the origin with radius vectors originating from it. The

reference axis is the zero degree radius vector point in the upstream direction (Figure 3).*

The scour depth at any position measured from the surface water level along any radius vector is related to D_m and the ratio r/R. By curve fitting, the following equation was derived (units in ft):

$$D = \frac{D_m}{(1 + r/R)^{0.92}} \tag{7A}$$

where:

D = scour depth at any point along a radius vector
D_m = maximum scour depth as given by Equation (1) (ref. 8)*
r = distance at any point along a radius vector
R = length of the radius vector (measured to the rim of the hole)

Equation (7A) is applicable to any radius vector and fixes the vertical dimensions of the hole.

To determine the horizontal coordinate, the length of the radius vector (R) is required. Many groupings of the significant variables have been attempted. The most successful correlation is given by Equation (7B) (units in ft):

$$\frac{R_\theta}{B} = C[(D_m - h)/h]^x[(H/d)]^y \tag{7B}$$

where R_θ = length of radius vector at the angle θ measured to the rim of the hole

B = width of spillway flip bucket
D_m = maximum scour depth as given by Equation (1) (ref. 8)*
h = downstream surface water depth referred to the original bed level
H = head drop from water level upstream of spillway to water level at scour hole
d = mean bed material size

The values of the coefficient C and the exponents x and y are given in Table 1. Equation (7A) and (7B) will determine the coordinates of the scour hole from which the contours can be drawn."*

Acres Consulting Service, (1), studied various prototype scour data, as given in their Table 5.1. The basis of their equation is unit discharge and head. Acres concludes the resulting equation predicts scour depth with sufficient accuracy for preliminary level designs.

$$D(ft) = q^x H^y$$
$$H < 100; x = .36, y = .38$$
$$H > 100; x = .65, y = 0.32$$

Table 5.1. - Prototype Scour Data

Name of spillway	Country	Unit discharge (ft^3/s)	Head (ft)	Observed scour depth (ft)	References
Maithon	India	368	114.0	40.0	(9)
Panchet Hill	India	270	107.0	26.5	(9)
Hirakud	India	780	105.0	60.0	(9)
Gandhi Sagar	India	460	158.0	64.0	(9)
Mandira	India	230	41.0	64.0	(9)
Tilaiya	India	39	80.0	34.0	(9)
Brazeau	Canada	33	98.4	29.5	(19)
Assekinski	Fergan	28	5.9	8.2	(26)
Hoschtedt	Germany	19	6.2	7.8	(26)
Beznau	Switzerland	183	20.7	46.9	(26)
Unknown	Unknown	646	24.0	53.2	(26)
Konovingo	United States	344	85.3	36.1	(26)
Unknown	Unknown	151	29.5	21.0	(18)
Unknown	Unknown	151	29.5	21.0	(18)
Unknown	Unknown	646	55.8	55.8	(22)
Overflow dam	Unknown	517	62.3	78.7	(18)
Unknown	Unknown	754	62.3	105.0	(18)
Kariba	Rhodesia	754	328.0	230.0	(24)
Akosombo	Ghana	?	?	137.8	(25)
Grand Rapids	Canada	975	61.0	90.0	(1)
Konolopoga	Unknown	?	37.4	15.7	(17)

Bormann and Julien (1991) derived an equilibrium scour equation based on the concepts of jet diffusion and particle stability in scour holes. Jet trajectory, jet diffusion, and particle stability were taken into account and the parameters in which they depend on were identified. The equilibrium scour depth Ds is calculated from:

$$D_s = \left[\left(\frac{\tau \sin \Phi}{\sin(\Phi + \alpha) B(\tau_s - \tau) g} \right)^{0.8} \frac{C_d^{\,2} Y_o^{\,0.6} U_o^{\,1.6}}{d_s^{\,0.4}} \sin \beta' \right] - D_p$$

Where:

d_s = sediment size
g = gravitational acceleration
β = coefficient of friction relationship
C_d = jet diffusion coefficient
D_p = drop height of structure
D_s = equilibrium scour depth
U_o = jet velocity entering tailwater
Y_o = jet thickness entering tailwater
τ = specific weight of water

τ_s = specific weight of sediment
α = embankment angle
β' = jet angle near bed
Φ = submerged angle of repose of the granular material

The authors also concluded that the equation of Mason and Arumugam appears to be representative of previous investigations using vertical jets.

Plunge Pool Case Histories

Mason (40) summarized several prototype scour hole developments, as follows:

"Alder Dam (USA, 1945)

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.

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 x 45 m x 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.

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 back filled 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 reinforce concrete were used in the repairs.

Nacimiento Dam (USA, 1957)

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 silt stones and which continues downstream of the flip bucket.

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 neighboring mud stones and silt stones. 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 construction of 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)

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

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.

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)

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.

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.

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 works.

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)

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.

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 one in five years.

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.

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.

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.

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.

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.

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)

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 Pakistan. 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.

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.

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.

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.*

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.

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.

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.

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."

Mason (40) commented further:

"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.

Geologically it can be seen from Table 2 that unacceptable scour is by no means limited to soft rocks such as shale, limestone and sandstone. 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 tons have been displaced by the flow.*

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 center of the plunge pool deflected the jet, causing the preferential erosion in the mud stones and the silt stones either side. A 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."

Strontia Springs Diversion Dam

Strontia Springs Diversion Dam (17) is a 300 ft. high arch dam, with a crest overflow spillway. The spillway energy is intended to be dissipated by a pre-excavated plunge pool. The crest overflow spillway is designed to pass the project design flood of 90,000 ft^3/s (along with a fuse plug on the left abutment). The fuse plug is designed to trip at 12,000 ft^3/s, twice the 100 year flood (flood of record). The plunge pool was excavated 30 ft. below river bed, which removed the alluvium and boulders overlying the sound pre-Cambrian metamorphic gneiss bedrock. This depth was also considered sufficient for twice the flood of record, or 12,000 ft^3/s..

Since completion of construction (1982), spill has only occurred for a few hours at 1000 - 2000 ft^3/s, for demonstration purposes. Most releases are made through a battery of hollow jet valves near the base of the dam. Those valves have capacity of about 4000 ft^3/s, but have not operated beyond about 2000 ft^3/s. The resulting scour of the plunge pool is shown in Figure* 2. The scour, mostly from valve operation, has tended to lengthen the plunge pool in the downstream direction, while depositing in the original plunge pool bottom. The plunge pool bottom has remained intact, but has not yet been subjected to high flow.

The Guri Project

The Guri Project in Venezuela, EDELCA (23), was originally completed in 1968, and included a gated chute spillway with design capacity of 30,000 m^3/sec, under a head of about 90 meters. (See Figure* 1).

After 10 years of operation, with wet season spilling typically up to 12,000 m^3/sec each year, and with a record flood in 1976 of about 18,000 m^3/sec, severe erosion had occurred to the flip-buckets, but the downstream excavated river bed (sound granitic gneiss) had not shown significant erosion (Figure* 2). The erosion in the downstream river bed was generally 10-13 meters, much less than expected.

Seven Mile Dam

Another example of a plunge pool which is developing more slowly than expected because of excellent plunge pool rock is the Seven Mile Dam; an example quoted earlier by Spurr (58). Spurr attributed the low rate of development to low unit discharge and excellent rock characteristics. This example corroborates the tendency to overestimate prototype scour in good rock.

Other examples of rate of plunge pool development are given by Water Power (12). Figure* 1 gives rate of development of scour depth for Kariba, Picote, and Farchad. This Figure* demonstrates the usual nature of scour development, that is, initially more rapid, and gradually decreasing, or even becoming asymptotic. However, these curves would obviously depend somewhat on the discharge pattern. It could be expected that if an unusual flood occurred in a year following a series of years with normal floods, the maximum scour would suddenly increase.

An example of a plunge pool which developed rapidly, under moderate flow conditions was given by Cooke (19). The project is described as follows:

Akosombo

"Two spillways, each with 6 radial gates 38.5 ft wide by 40 ft high, handle 730,000 ft³/s with 13 ft surcharge above top of ages (53 ft over ogee crest). Density of flow is 1300 ft³/s/ft.

The plunge pool is 100 ft below normal tailwater. ENCL. 1 (8) shows 3 photos of the plunge pool for Spillway No. 2: before spilling, after several days at 70,000 ft³/s, (1966), and after 4 months at 4 central gates and 132,000 ft³/s. ENCL. 1 (9) includes 3 photos: 1968 operation at 330,000 ft³/s for #1 and #2, 1968 operation of Spillway #1 at 200,000 ft³/s, and spill lip of #1 in 1970.

The plunge pool for Spillway No. 2 is in weak rock, mudstone and shale, but the flip bucket is on competent quartzite. The lip of the flip bucket is 50 ft above normal tailwater. There is nominal surface erosion for the first 160 ft downstream from the flip bucket, then an excavated slope of about 1.5:1 to 2:1 to a depth of 140 ft, 100 ft being below tailwater. The pool is moderately higher than the width of the spillway chute. The jet excavates the weak rock, but the strong eddy currents do not erode the rock. The pool does not encroach on rock below a line of 2.5H:1V from the spillway bucket. There will be some enlargement of the pool in this weak rock with future operation, but the pool can be enlarged without consequences to the flip bucket or to stability of the spillway ridge."

The Akosombo spillway was subjected to one significant flood and the resulting scour hole is given in Enclosure 1 (6 & 7). The plunge pool rock is mudstone and shale, and eroded quickly. This again corroborates the concept that weak rock erodes quickly, while sound rock erodes slowly.

Tarbela

Tarbela is another example of a plunge pool in relatively weak rock, which eroded quickly. It also demonstrates the danger of a special geologic feature in the tailrace. The project is described by Lowe, Chow, and Luecker (39).

"The spillway design flood has an inflow peak of 50,200 m^3/s and a maximum outflow rate of 42,200 m^3/s. Site conditions dictated the use of two separate spillways to provide the required capacity. Both are located at low points on the left bank of the reservoir near the dam and both discharge into the same natural depression, the Dal Darra, which enters the Indus about 2.5 km below the dam, as shown in Fig. 1.*

The service spillway was originally conceived as the principal spillway, to be used exclusively to meet all requirements within its 18,400 m^3/s capacity. It consists of a gate-controlled ogee weir, a chute consisting of a 530 m section on a 1 percent slope followed by a 160 m section on a 2:1 slope, and a flip bucket.

Pertinent elevations are:

	1,550 ft (472.44m)
Normal full pool	*1,492 ft (454.76 m)*
Weir crest	*1,220 ft (371.86 m)*
Bucket lip	*1,140 ft minimum (347.5 m)*
Tailwater (roughly)	*1,170 ft (357 m) at 8,500 m^3/s*
	1,190 ft (363 m) at 18,000 m^3/s

The chute is 106.7 m wide. The bucket radius is 15.24 m and the lip angle is 35°.

The plunge pool area was excavated in rock to elev. 1,200 ft (365.8 m) with a bottom width of 128 m. Side slopes were 1:1, with berms at 15 m intervals in elevation. The berm at elev. 1,300 ft (396.2 m) on the right side was 46 m wide, and all others were 9 m wide. The maximum depth of cut to elevation 1,200 ft (365.8 m) was 110 m on the right-hand side and 80 m on the left-hand side. The approximate excavation limits are outlined in Fig. 3.*

The dominant geologic feature at the plunge pool site is an igneous intrusion crossing the exit to the Dal Darra and continuing along the right-hand side of the Dal Darra for 0.5 km downstream, where it obliquely crosses the channel as indicated in Fig. 1. At its crossing of the plunge pool exit the intrusion is 50 m thick and dips away from the pool at 60°. Its strike is at approximately 45° to the direction of the spillway center line. Bands of hand siliceous limestone occur on both sides of the igneous intrusion. Within the pool area, the rock consists of highly contorted and faulted beds of limestone and phyllite. Its resistance to water erosion, expressed as a probably minimum eroding velocity, has been variously estimated at between 3 and 6 m/s. One especially notable fault zone occurs*

near the downstream limit of the limestone and phyllite beds. Another is evident at the upstream edge of the scoured right-hand side embayment (Fig. 4). This latter fault continues diagonally across the chute foundation just upstream of the bucket.*

The service spillway has operated more than 10,000 h during the four-year period 1975-78. The highest discharge was 8,900 m^3/s. The total numbers of hours of operation at or above different discharges during various segments of the four-year period are shown by the duration curves of Fig. 2.*

Deepening of the plunge pool progressed rapidly during the early part of the 1975 season. In the first two-and-a-half weeks the bottom reached elevation 1,100 ft (335 m) over a broad area, with local scour 20 m lower on the right side of the pool adjacent to the siliceous limestone formation. Later in the year, caving of the right bank had occurred at the wide berm at elevation 1,300 ft (396.2 m) and had filled the local scour hole to the general level of the plunge pool bottom, which remained at elevation 1,100 ft (335 m). Surveys throughout 1976 showed the hole reappearing and disappearing from time to time as enlargement of the pool progressed through caving of the right bank downstream of the jet impact zone.

Erosion of the right-hand bank accelerated during 1976, beginning with a 400,000 m^3 slide on June 12. The discharge at the time was 6,500 m^3/s. Service spillway discharge for the remainder of the season was limited to 5,100 m^3/s and greater use was made of the auxiliary spillway.

Widening of the pool on the left-hand side also began early in 1975, in the area immediately upstream from the igneous intrusion. By the end of the year the cove at the location extended back 25 m from the original excavation line.

There was little or no lowering of the bed close to the bucket structure during 1975 and 1976 and no caving of the steep side slopes upstream from the jet impact zone. During this period, scour was generally symmetrical within the original pool limits, reaching elevation 1,070 ft (326 m) in October 1976, with deeper holes locally. The igneous rock and siliceous limestone, though not fully erosion-resistant, remained essentially intact with its top surface near elevation 1,200 ft (365.8 m) Bank erosion was notably greater on the right than on the left. By the end of 1976, caving along most of the right side covered the full height of the cut slope. Portions of the main jet, after penetrating the surface of the pool, were being deflected by the igneous rock band, with the larger proportion going toward the right because of the orientation of the band. In characteristic plunge pool fashion, the water level was raised in a boil forward of the jet and depressed beneath the jet. The elevation differential gave rise to currents in the upstream direction along the sides, but initially the flow was relatively small because of confinement by the banks along the paths between the raised and depressed areas.

As erosion and bank caving continued on the right side during 1977, the embayment on the right-hand side developed to the point where it provided a broad flow path between the raised and depressed areas. A strong clockwise peripheral circulation occurred in the embayment. The current, together with wave action, expanded the eroded zone upstream and toward the right. On entering the region of depressed water level beneath the jet, the circulating current continued in an arc that crossed the pool and turned downstream to follow the left bank until it entered the jet impact zone. The current scoured a depression in the bed along its path and thus became a dominant factor in the development of the pool during the latter part of 1976 and throughout 1977. The scoured depression along the flow path is referred to by project personnel as the "runnel". Bed contours mapped on August 7 and November 18, 1977, (Figs. 3 and 4) show two stages in the development of the runnel. Erosion at the upstream end of the embayment exposed portions of the underground protection wall for the bucket structure and caused about 1 cm of movement of the bucket to the right long the fault at that location.

Scour and bank sloughing also occurred on the left side but to a much lesser degree than on the right.

Materials picked up by the circulating current from the bed and from talus supplied by the caving right bank were ground down to coarse gravel sizes. Some was deposited at the central portion of the right-side embayment to form a banana-shaped island that projected above tailwater level when the spillway was not operating.

Design studies and hydraulic model tests were made during 1976 and 1977 to develop plans for controlling the erosion and stabilizing the banks at critical locations. During the 1976/77 low-flow season, the ridge formed by the igneous intrusion at the outlet was cut down to elevation 1,155 ft (352 m), just above the prevailing water level at the time. Further drilling and blasting were carried out to elev. 1,070 ft (326 m) to encourage erosion consistent with that of the adjoining softer rock and lessen the deflection of water into the embayment. High on the right-hand side unstable material was removed and slopes flattened which had been left nearly vertical by bank caving. A plan was developed for interim improvements that could be accomplished in the 7-month period available for construction between high-flow seasons. It included further drilling and blasting of the igneous intrusion; construction of a concrete groin at the right-hand side of the pool to deflect the circulating current away from the bucket structure; addition of a concrete-faced rollcrete buttress at the toe of the bucket structure and a rollcrete strut between the bucket and the right-hand side of the embayment; and, placement of a rollcrete dike across the cove on the left, plus concrete paving with passive anchors and drainage to stabilize the slope between the dike and the buttress. A plug of rollcrete with top at elev. 1,090 ft (332.2 m) was placed as shown on Fig. 5 between the nose of the groin and a ridge of more resistant limestone.

These works were constructed in the dewatered basin during the 1977/78 low-flow period.

The combination of groin, plug, and strut on the right-hand side had the desired effect of arresting development of the runnel which posed a threat of continued deepening in the crucial areas below and immediately to the right of the bucket structure. The clockwise flow in the embayment continued as expected and ended in a strong current that crossed the nose of the groin and plunged below the surface. The bed contours in Fig. 5 show the resulting scour after 5 weeks operation with flows up to 5,600 m^3/s.*

The pool was dewatered again in 1978/79 for further construction directed towards a permanent stabilization scheme that would assure the safety of the spillway for the foreseeable future. As finally resolved, the scheme was to provide for an essentially symmetrical pool with protection on both sides to below the estimated limit of future scour. A dike of rollcrete faced with concrete was to be constructed connecting the right-side groin with the hard rock near the pool outlet; protection of the rock slope would continue around the right-hand side of the mouth of the pool and a short distance along the Dal Darra.

Similar protection would be added to the left side. The first stage, completed in June 1979, consists of downward extension of the concrete protection on the pool side of the groin and along the left dike plus extension of the left-side protection to the end of the plunge pool and back along the right bank of the Dal Darra for 150 m.

Completion of the programme, consisting of further downward extension of existing protection and construction of the right wall and its extension along the Dal Darra, is scheduled for the 1979/80 low-flow season. The expected effect of these improvements is indicated by comparison of Figs. 6 and 7 which show scour patterns obtained at 11,300 m^3/s equivalent prototype discharge in a 1:50 scale model using 2 to 8 mm gravel as the bed material. Addition of the right wall in the model has shifted the main scour hole away from the nose of the groin and changed the highly-skewed overall scour pattern to a pattern that is essentially symmetrical. Where Fig. 6 shows a depression along the right side toward the outer end of the pool, Fig*. 7 shows a deposit of material. Though elevations along the left wall are generally lower in Fig*. 7, the deepest hole on that side, now located further upstream, has not until now, changed significantly in depth.*

As scour progressed in the lower reaches of the Dal Darra, the igneous intrusion became a control, limiting tailwater regression at the two spillways. During the past construction season its top was leveled and covered by a concrete slab at elev. 1,140 ft (347.5 m) and approach walls were constructed on both sides. A limited amount of rock bolting and dental concrete reinforcement was done to stabilize the steep downstream face. The probable life of this stabilized weir is uncertain, and lowering of

the tailwater by a possible 12 m has been anticipated in the fixing of protection depths for the spillway plunge pools. Under this assumption and guided by the scour observations made to date, together with model tests up to the full design discharge, the protection depth to be provided in the next year's construction is generally to elev. 930 ft (283.5 m) except that somewhat higher levels will prevail in the vicinity of the corner formed by the bucket buttress and right-hand side protection.

The other spillway, designated "auxiliary spillway", has been operated each year during shutdowns of the service spillway and to provide additional capacity during those periods when it seemed prudent to limit the flow into the service spillway plunge pool. In most respects, its plunge pool development and the protection works constructed and proposed are similar in principle to those described for the service spillway."

Ukai

Ukai (India) described by Thatte (60) is an example of significant plunge pool erosion which resulted from non-uniform operation of spillway gates and inadequate plunge-pool design. The non-uniform gate operation resulted from recommendations based on model studies; in order to accommodate severe geometry constraints in the tailrace. The spillway exit channel made a sharp bend to the right to return to the original river. Protective guide walls and a divide bund were meant to protect the main dam and powerhouse exit channel. Spillway flows were concentrated in the center, resulting in severe erosion of the moderately weathered rock. Protective grouting and rock bolting were recommended after 7 years of operation to protect the structures from possible undermining.

Equation-Prototype Comparisons

Harza (48) in their report to Reclamation also presented several comparisons of measured prototype plunge pool depths and calculated depths based upon Mason and Veronese. These comparisons reflect the significant scatter that can be expected from two of the most commonly applied methods to estimate plunge pool scour depth.

Tarbela

Maximum observed prototype scour for 8,900 m^3/s was checked with Veronese and Mason, as:
Veronese: $d_s = 58.4$ m
Mason: $D = 57.3$ m
Actual Prototype: $y = 47$ m.
Maximum in model for 11,320 m^3/s was as:
Veronese: $d = 66.4$ m
Mason: $D = 61.2$ m

Actual Model: $y = 58.0$ m

Conclusion is that both formulas give reasonable results, with Veronese somewhat more conservative.

Akosombo

Scour for Akosombo was computed for the maximum discharge of 132,000 ft^3/s (3,739 m^3/s), as:
Veronese: $d_s = 33$ m
Mason: $D = 24$ m
Actual: $y = 33$ m
Conclusion is that Veronese is identical to actual, while Mason under-predicts.

Seven-Mile Dam

Scour was computed using Veronese and Mason, based on maximum observed unit discharge of 70 m^3/s-m and head of 53 m as:
Veronese: $d_s = 46$ m
Mason: $D = 33$ m
Actual: $y = 18\text{-}23$ m
Conclusion is that both formulas over-predict, but Mason is closer to reality.

Guri

Scour was computed for the representative maximum discharge of 15,000 m^3/s, as:
Veronese: $d_s = 75$ m
Mason: $D = 87$ m
Actual: $y = 36$ m
Conclusion is that both formulas greatly over-estimate the scour which has actually occurred.

Kariba

Kariba is unusual in that the observed scour exceeds predictions of either Veronese or Mason. Based in a maximum spill of about 6,500 m^3/s prior to 1981, the computed scour was:
Veronese: $d_s = 79$ m
Mason: $D = 95$ m
Actual: $y = 100$ m $\pm$

It is interesting here to note that although the foundation rock is a sound gneiss, the scour is consistent with a relatively fine granular material. The only apparent difference between Kariba and Guri is the jet entry angle, Kariba approaching vertical, whereas Guri is probably effectively about 30° down from horizontal.

It is clear from the foregoing discussion and results, that the only reliable prediction which can be made is for terminal scour depth for a uniform granular tailrace material. However, this condition is almost never encountered in nature. It is more likely that special geologic features (non-uniform material) will produce a scour depth somewhat to greatly different from that

predicted by a given equation. For this reason, a physical model is almost always necessary to obtain a higher level of confidence in the scour depth and erosion pattern. This point is made very well in ICOLD Bulletin 58 (31), published in 1987; as follows:

> *"Phenomenological models are necessary in important cases, even with wide-valley sites, to give a better approximation of the shape and size of a freely developing scour-hole or to determine the protection required to inhibit development. The greatest difficulty resides in proper representation of the erodible materials, especially cohesive materials (non-homogeneous rock of varying weathering and hardness, which always contains a least some joints) and some means of verification of the model, e.g., by reproducing a known past event true to scale, would be needed.*
>
> *The use of sand or gravel of appropriate fineness for a complete model of the river channel and valley flanks at a wide-valley site will yield the maximum final size and shape of the scour hole. As the angle of repose varies with particle size, scale effects are important in this type of investigation. Maximum depth occurs at the point of impact of the jets; this is not usually a cause for worry in itself. With a bucket of appropriate geometry and throw, retrogressive erosion back from the impact point will be moderate; little or no protection (cut-off, large blocks) will be necessary to protect the bucket (especially for small flows when the jet does not jump).*
>
> *Back currents however are frequently the cause of excessive sideways extension of a fully developed scour hole. The pattern of currents change and may intensify as the hole grows. Cut-offs, boulders, concrete blocks, concrete or rock spurs, and various combinations thereof, can stop such growth. Useful design information on such protection (depth of cut-off, weight of blocks, lining batter, etc.) can be drawn from such tests.*
>
> *Prediction of the development of the scour hole in narrow-valley sites are (sic) much more hazardous. The ploy of determining final scour hole size with a mobile bed model ceases to apply, because the equilibrium profile would be reached after removal of a considerable quantity of material, liable to completely undermine large structures and produce serious blockages in the river downstream. Conditions where there is rock of exceptionally good quality that would be hardly affected by wear or damage despite prolonged attack by dynamic pressures, cavitation and abrasion are as rare as the large natural waterfalls whose development appears to be arrested on the human time scale. This has not prevented designers of an increasingly large number of spillways in narrow-valley sites from deliberately omitting any protection in the impact zone. The implied explanation is the low probability of any extended, heavy discharge whose inevitable consequence would be erosion of the channel and banks; they prefer to wait and see, on the argument that erosion will be slow enough for suitable protection to be provided before the situation becomes critical. This is a deliberate, "calculated" risk.*

At narrow-valley sites, realistic determination of the stable scour hole means that the model must reproduce the rock and its discontinuities, using a model material with the appropriate cohesion [67]. It has already been said that real progress in this area is still awaited. But the scale model is still a useful tool. Tests should begin with a bed material offering no resistance against scour, and subsequent runs made with a slightly cohesive material. If there are any well-defined barriers to erosion at the bottom, sides or upstream edge of the scour hole, they should be included in the model, preferably after the first test with cohesionless bed material. Flow conditions at the fixed sides can then be investigated (whether or not back-currents appear). Current velocities, pressures and dynamic pressure fluctuations are measured at significant points and yield valuable information on the erosion potential which the rock or lining must withstand. One can predict the likelihood of the unprotected rock remaining undamaged (the probability is usually very low) or the utility of a given type of lining material. The depth of standing water and the sideways extension of the hole are two essential parameters for the test.

If the hole is to be lined only at the sides (not at the bottom or downstream end), the concrete should extend downwards to a level just below the final depth of the scoured bed on the fixed-side model. The fixed sides representing the lining must be in place before subjecting the movable bed material to scour for the final depth to be a true estimate.

Measurement of velocities along the sides also helps in evaluating the risk of more or less coarse solids being entrained and resulting abrasion (especially if the bottom of the hole is not lined). This risk can be controlled by building vertical linings that are not so exposed to abrasion, but the problem may become more complex if these linings also serve to shore up the banks (deep anchors are necessary).

It is not easy to determine the protection necessary at the start of operation or becoming necessary later. The designer does not even have the imperfect tool of the mobile bed model, because the largest scour hole that can be accepted is usually far less than the size of the final hole in loose material. Laboratories offer models built wholly or partly with materials of low cohesion (sand and lime, sand and clay, sand/chalk and cement) claimed to represent rock and thus give information on the extension of erosion after a given time of operation. But the proper modeling of rocks of different hardnesses, and more importantly, of discontinuities, is unfortunately far from being a science. The preponderant role of discontinuities in such rocks (joints, fractures, faults) in the erosion process, because of their sensitivity to fluctuating dynamic pressures, means that the results from a model material must be approached with caution, unless it has been possible to calibrate the model after the first spilling has taken place, to predict ultimate development, as was the case at Kariba.

Engineering judgment therefore plays a major part in designing protection in narrow-valley sites. The guiding idea is that the energy must

be dissipated in a volume of water half as large again as the volume that would be required in a hydraulic jump for the same flow rate and head, and preferably larger. Finding this volume may mean excavating into the river channel and banks, as being preferable to uncontrolled scour by flow. It is however desirable that the scour hole should not be much wider than the nappe or jets, otherwise back-currents may form. A model with fixed sides will give valuable information on this point.

Means of stabilizing scour hole growth are reinforced-concrete linings and/or layers of large rocks or artificial blocks such as dolosse. Concrete linings are much more resistant than rock to damage from fluctuating dynamic pressures because of their greater uniformity. But in practice, they are nearly always confined to protecting areas where they have a good foundation and backing such as rock of moderate to very good quality, otherwise they have to be prohibitively thick.

Blocks are only suitable for horizontal or gently sloping surfaces where they would not be undercut, which greatly limits their field of application.

Protection may be required for the bottom and/or sides. Typical of bottom protection is the concrete apron, usually below arch dams. But there are many examples of such aprons being destroyed or seriously damaged right from the start, and one must look at the types and magnitudes of forces to which the apron is exposed.

In the impact zone, the dynamic pressure may be equal to the total reservoir head if there is no cushion of water to soften the load (Fig. 12), but it decreases rapidly father away so that the net effect is a concentrated load on the apron. Cola, Lencastre, Häusler, George and others [30, 56, 58, 79, 49] have attempted experimentally to find the relationships between steady pressure, head, discharge per unit width, jet angle with the apron, distance from point of impact and standing water depth. Some measurements from completed dams have also been reported. There is quite a high degree of scatter in the results although they all agree as to the importance of the depth of the water cushion on the apron (Häusler report 45 m for 100 m^3/s/m width for a height of 100 m).*

This dynamic pressure, however high should not raise an insurmountable structural design problem; the apron usually overlies rock and even if it has to span over a localized depression in the foundation, it is simply a matter of making it thicker, with more reinforcement.

Leaving aside the particular case of abrasive sediment load, the most dangerous factor is uplift pressure if a fissure in the rock, or the rock/concrete interface, communicates with the impact zone. This pressure propagates outward without loss, and where it is not counterbalanced by the falling jet above, it may act like a hydraulic jack capable of lifting the apron off its foundation and/or of displacing whole blocks of rock between natural joints.

This situation may be aggravated by fluctuating dynamic pressures caused by the high macro-turbulence in the impact zone [63]. Lencastre

[79] reports values 2.8 times higher than the average pressure in this zone if the layer of cushioning water is shallow (less than 11.4 times the jet thickness), but both the average pressure and the fluctuating pressure became negligible at the point of impact when the water depth was more than 15 times greater than the jet thickness, under his particular test conditions. More experimental research is needed. The influence of the height of the jet in particular does not seem to have been sufficiently investigated.

Thus the most important type of damage to a plunge pool is similar to what was described to explain the destruction of the floors of hydraulic jump stilling basins. The risk of vibration of the slabs and fatigue in the concrete and reinforcement under these alternating stresses is also significant except if there is a sufficient depth of water to substantially reduce the magnitude of the pressure and the amplitude of the fluctuations.

Concrete aprons are set at a relatively shallow level, precisely because their purpose is to prevent scour down to a stable depth where the steady pressure and pressure fluctuations would cease to be significant. Even if there is no cushion of standing water, they can successfully withstand prolonged spilling provided they are not exposed to uplift pressures. The same constructional arrangements as for stilling basin floors therefore apply: sealed joints, dowels and shear keys, and deep anchors to prevent vibration (and resist any residual uplift pressure) [14]. The slab thickness and tonnage or reinforcement and anchor steel must be more substantial at and around the point of impact; since this involves only a relatively limited area however, the extra cost is not prohibitive. Drainage can be provided at the rock/concrete interface provided the outlets are well away from the area of high pressure.

The extra protection provided by a submerged apron is only significant if the water cushion is deep enough. As already mentioned, model tests and full-scale measurements reported to date are still too incomplete to derive any general relationship between all the factors involved. It would appear that there is no appreciable reduction in the combined steady and fluctuating pressures if the depth of water is less than around 20 per cent of the head.

Side protection should be kept to the minimum necessary to prevent erosion by the current emerging from the scour hole and the inevitable back-currents. This means that the nappe or jets must not impinge on the banks where the depth of water is insufficient; cuts into the bank re often necessary.

The turbulence associated with back-currents and the flow leaving the scour hole is considerably less than in the impact zone. If the sides are far enough away, the fluctuating dynamic pressures on the side protection will be greatly attenuated. However, with vertical or battered concrete walls or linings, a relatively low pressure on the rear face may be enough to cause them to topple. A large footing area and rock anchors will provide

stability. A more difficult situation is where the side walls also shore up the bank. In any event, protection of the lower edge is the main issue. If side protection is combined with a concrete apron, the scour hole is almost fully contained and the stability of the sides and apron combined into the same problem. The opposite situation is where the scour hole is allowed to develop freely during spillage until the depth of water is sufficient for further deepening to proceed very slowly. In this case, it is wise for the side protection built at the same time as the dam to be several meters deeper than the final scour hole depth, which means awkward trenching work. In order to limit the depth of these walls, it is conceivable to try to slow down the rate of deepening of the scour hole by tipping in blocks once it has reached a certain depth; but this is a much less reliable solution and may often involve insurmountable practical problems.

Rock or concrete blocks for side protection are only feasible in the case of wide-valley sites mentioned at the beginning of this section. They are used to stabilize relatively gentle slopes inside the scour hole and up the banks to a comfortable distance from the impact zone."

Culvert Scour

A somewhat parallel and overlapping development of scour prediction relationships has evolved from the need to predict scour hole development at culvert outfalls. Much of this work draws on the influences and importance of soil properties.

The problem of excessive scouring in self-formed holes has been studied mainly during the last 50 years. However, the most important contributions, from a practical viewpoint to this problem have been made during the last 20 years. In 1952, Laursen (38) identified the following as basic principles:

- The rate of scour will equal the difference between the capacity for transport out of the scoured area and the rate of supply to that area.
- The rate of scour will decrease as the flow section is enlarged.
- There will be a limiting extent to the scour.
- The scour will be approached asymptotically.

Thomas (61) studied a freely falling jet of water and its resulting scour on a uniform gravel bed material. For this study a free over fall was constructed with a sudden drop in elevation from one horizontal bed to a lower horizontal bed. The tailwater depth was varied for different runs of the experiment. The geometric mean was held constant, but the standard deviation for the size was varied. The following conclusions were reached:

- The depth of scour continues to increase with a geometric progression of time.

- An increase in discharge causes a greater increase of depth of scour.
- A critical depth is reached at which either an increase or decrease in tailwater causes a decrease in the scour depth.

Two years later, Hallmark (28) studied the influence of particle size gradation at the base of a free over fall. Basically his research focused on armor plate design for avoiding excessive scour. Essentially, his conclusions were the following:

- A 50 % decrease in the standard deviation of the size distribution resulted in a 50% increase in depth of scour.
- Only a relatively small amount of armor plating material is necessary for a relatively large decrease in the rate of scour.
- The rate of scour decreases with a decrease in the size of the armor plate material while the armor plate material remains larger than the largest particle size of the bed material.
- The rate of scour decreases with an increase in the amount of armor plate placed in the scour hole.
- Graded armor plate material decreases the rate of scour more effectively than uniform material.

Opie (45) carried out a series of experiments to simulate scour at culvert outlets. He used flat, loose rocks as bed materials. He reported that the bed remained intact while the water filled the voids. Then, as the water rose, the scour hole "exploded" and very rapidly attained its final "stable" shape. The rate of scour, afterwards, depended upon the time. Differences in scour hole geometry due to the bed material shape were noticed, as well as quantities in material that remained in the bottom. When the material was rounded, relatively small material remained in the bottom. He also showed that there is a direct correlation between the scour hole dimensions.

Kuti (36) wrote a thesis about erosion of cohesive bed downstream of a spillway. In this study, he analyzed the relationship between parameters that are inherent to cohesive soils (such as plasticity, liquid limit, etc.) to scour. Plasticity is a valid soil index to compare the erodibility of two types of soil, provided that all conditions are equal. Furthermore, since electrochemical processes take place in clays, content of chemicals in water can determine erosion. So, percentage of clay and plasticity index are good indicators of erosion if the chemical content of water is known. In addition, he showed that void ratio can be a functional variable of rate of scouring in inactive clays (Those that have no ability to absorb water).

Also, in-place void ratio has direct influence on when scouring reaches an equilibrium condition.

Vanoni (65) described the parameters that influence erosion and deposition processes. Among these are:

- Temperature of the liquid: which affects viscosity of the water, and therefore affects concentration profile, fall velocity, etc..
- Particle shape: a shape factor, *SF* is defined as $SF = a/(bc)^{1/2}$, where a is the largest dimension, b and c are the largest dimensions perpendicular to a. (c< b< a) According to Vanoni, this affects the drag of a falling particle, and therefore the fall velocity.
- Concentration of solids: solids concentration affects the viscosity of the fluid, and therefore the fall velocity.

Mendoza-Cabrales (43) studied scour cavities produced by different discharges of varying duration through circular culverts onto a horizontal sand bed (d_{50}=1.86 mm, σ=1.33). Expressions describing depth, length, and volume of the scour hole as a function of time and of the discharge intensity, $DI = Qg^{-0.5}D^{-2.5}$, were developed.

An exponential decreasing type law was used for describing variation of depth as a function of time :

$$\frac{d_s}{d_{sm}} = 1 - e^{-at}$$

Where:

d_s = depth of the scour cavity below the elevation of the initial ground level,
d_{sm} = maximum scour depth
a= constant,
t= time from initiation of scour.

which has the double advantage:

At t=0, $d_s/d_{sm} = 0$

At $t \to \infty$, $d_s/d_{sm} = 1$

The equations describing the scour phenomenon with respect to time are:

$$\left(\frac{d_s}{d_{sm}}\right)_{NH} = 1 - e^{-(0.14t+0.90)}, t \rangle 0.5h(prototype\,time)$$

$$\left(\frac{d_s}{d_{sm}}\right)_{H} = 1 - e^{-(0.14t+0.74)}, t \rangle 0.5h(prototype\,time)$$

Where NH indicates non headwall condition and H indicates the headwall condition. The boundary condition was imposed because an extra term appears in the exponent of e indicating that at t=0 some scour occurs (d_s/d_{sm} constant).

Expressions for length and volume of scour were also developed:

$$\left(\frac{L_s}{L_{sm}}\right)_{NH} = 1 - e^{-(0.14t+0.47)}, t \rangle 0.5h(prototype\,time)$$

$$\left(\frac{L_s}{L_{sm}}\right)_{H} = 1 - e^{-(0.14t+0.41)}, t \rangle 0.5h(prototype\,time)$$

$$\left(\frac{V_s}{V_{sm}}\right)_{NH} = 1 - e^{-(0.18t-0.31)}, t \rangle 0.5h(prototype\,time)$$

$$\left(\frac{V_s}{V_{sm}}\right)_{H} = 1 - e^{-(0.18t-0.25)}, t \rangle 0.5h(prototype\,time)$$

In this study, Mendoza concluded that 80% of the scour occurred during the first 0.5 hr prototype time.

Kloberdanz (1982) carried out a series of tests, and using data previously taken by Ruff, Abt, and other researchers derived empirical relations for calculating scour in culverts produced in noncohesive material. Characteristics of the particles such as characteristics size and time relationships (similar to those used by Mendoza) were taken into account. Examples of design procedures were presented based on the derived relationships.

$$\frac{D_s}{D_{sm}} = 1 - e^{-(0.015t+0.96)}$$

$$\frac{W_s}{W_{sm}} = 1 - e^{-(0.016t+1.88)}$$

$$\frac{L_s}{L_{sm}} = 1 - e^{-(0.015t+0.47)}$$

$$\frac{V_s}{V_{sm}} = 1 - e^{-(0.015t+0.47)}$$

$$\frac{W_s}{D} = 7.79\frac{D_s}{D}$$

$$\frac{D_s}{D} = 0.28\left(\frac{V_s}{D^3}\right)^{0.41}$$

$$\frac{W_s}{D} = 2.23\left(\frac{V_s}{D^3}\right)^{0.42}$$

$$\frac{L_s}{D} = 6.99\left(\frac{V_s}{D^3}\right)$$

$$\frac{D_s}{D} = 1.25(D.I.)^{0.87}$$

$$\frac{W_s}{D} = 9.75(D.I.)^{0.87}$$

$$\frac{L_s}{D} = 13.49(D.I.)^{0.37}$$

$$\frac{V_s}{D^3} = 36.16(D.I.)^{2.08}$$

$$\frac{D_s}{D}\sigma_g^{\ 0.4} = 3.35\left[D.I.\left(\frac{d_{50}}{D}\right)^{0.2}\right]^{0.57}$$

Where:

D = culvert diameter
D_s = depth of scour hole
D_{sm} = maximum depth of scour hole
d_{50} = median grain size diameter
g = acceleration of gravity
L_s = length of scour hole
L_{sm} = maximum length of scour
Q = discharge
t = time
V_s = volume of scour hole

V_{sm} = maximum volume of scour hole
W_s = width of the scour hole
W_{sm} = maximum width of scour hole
g = geometry standard deviation of grain size = $(d84/d16)^{0.5}$
d_{84} = particle diameter of a material for which 84 percent of material is finer by weight
d_{16} = particle diameter of a material for which 16 percent of material is finer by weight

Abt (1) developed methodologies for calculating scour hole dimensions in cohesive soils that were impinged by a culvert. He basically studied sandy cohesive soils (SC) soils. He found out that the length, width, depth and volume of scour can be estimated in an SC soil type knowing any cavity dimension by using the following relationship:

$$\left.\begin{array}{c}\frac{d_s}{D}\\ \frac{V_s}{D^3}\\ \frac{L}{D}\\ \frac{W}{D}\end{array}\right\} = a\left(\frac{d_s}{D}\right)^b (t_0/t)^c$$

d_S= scour depth
D = culvert diameter.
L = maximum dimension of scour parallel to the culvert
W = maximum dimension perpendicular to L
V_S = Volume of scour
t_O = time in minutes < 1000
t = 1000 min.
a, b, c = constants, according to the unknown parameter.

On the other hand, scour hole dimensions for SC type soils can be estimated by knowing the culvert diameter and discharge, applying the discharge intensity relationships:

$$\left.\begin{array}{c}\frac{d_s}{D}\\ \frac{V_s}{D^3}\\ \frac{L}{D}\\ \frac{W}{D}\end{array}\right\} = a(Qg^{-0.5}D^{-2.5})^b (t_0/t)^c$$

where
Q= discharge
D= culvert diameter

A third method was proposed for obtaining the scour hole dimensions. For using this method, vane shear, strength, plasticity index and water content must be obtained. In addition, culvert outlet velocity must be known for computing the modified shear number. Cavity dimensions can be gotten from:

$$\left.\begin{array}{c}\frac{d_s}{D}\\ \frac{V_s}{D^3}\\ \frac{L}{D}\\ \frac{W}{D}\end{array}\right\} = a(1000\,\tau_c\rho^{-1}V^{-2})^b\,(t_0/t)^c$$

Where:
τ_c= shear stress
ρ= density of fluid
t_0= time of peak flow
V_s= volume of scour

Important observations were the following:

- Seventy percent of maximum depth, width and length occurs during the first 31 minutes. (45% of the volume scour occurs in the same time)
- The maximum depth of scour occurred at 0.35 L_m +- 0.05 L_m.
- Mounds formed downstream the scour hole were generally flat and 0.25D in height.
- It appears as if the scour cavity enlarged until the energy of the impinging jet was dissipated by the tailwater and scour pool.

Shaikh (54) studied noncohesive sands. He studied both graded and uniform sands with a mean diameter of 7.62 mm. In experiments with graded gravel it was observed that the fine and coarse particles were segregated. A portion of the coarse particles settled at the bottom of the scour hole while the fine particles were deposited at the side of the scour holes and around the edge of the mound". Other important conclusions were:

- Scour depth increased as a power function of discharge intensity $DI = Qg^{-0.5}D^{-2.5}$ in both uniform and graded sand.
- Ninety per cent of maximum scour occurred during the first 31.6 minutes of each test.
- Under similar conditions, scour in uniform gravel was 17 to 27% greater than in graded gravel.

Rajaratnam (48) conducted a series of experiments in sand beds impinged by circular turbulent water jets. Previously, he had used air jets. In a first report, he analyzed the effect of submerged vertical jets. Basically, he reported that the hole dimensions increase linearly with log t, and that an asymptotic state occurs. A dependence of the hole dimensions on a parameter later called the Erosion coefficient:

$$E = \frac{Fr_\rho}{H/d}$$

where
Fr_ρ= densimetric Froude number
H= distance from the nozzle to the initial bed elevation
d= Nozzle diameter

After the first report was written he carried out experiments with both submerged and unsubmerged jets, and later on he changed the jet position from a vertical to different oblique positions, using minimum tailwater depth. In a final report he concluded that the maximum depth as well as the distance from a gate that produces the jet are functions of mainly the densimetric Froude number. But the ratio of the maximum scour produced to the jet thickness is much less when used deep tailwater. In all cases the eroded bed profiles were found to be similar to each other.

Ruff (51) analyzed scour at culvert outlets in order to determine the dimensions of scour holes, and provide design criteria for the prediction of natural scour hole dimensions. Parameters taken into account in this study were:

-Discharge intensity: The shape of the scour hole is a function of the discharge intensity. For low discharges, scour holes were circular in shape and elongated as the discharge intensity exceeded one. It was also reported that as the Discharge Intensity increased the ultimate dimensions of the scour hole increased.

$$DI = \frac{Q}{\left(gD^5\right)^{1/2}}$$

-Bed material: Gradation effects. It was observed that the graded materials tended to armor the scour hole reducing the ultimate scour hole dimensions from those of more uniform materials. On the other hand, graded materials were observed to result in a scour hole of steeper slopes than those derived for uniform materials of similar mean diameters.

Particular attention was given to analyze the scour as a function of the plasticity of bed material. Given a fixed time (31 minutes) 80% of the maximum depth, width, and length was attained for noncohesive materials . For cohesive materials, 70% of the maximum depth, length and width was obtained for the same time.

For the cohesive soil, a series of empirical relationships were also formulated correlating the maximum dimensionless scour characteristics of d_{sm}/D, W_{sm}/D, L_{sm}/D and V_{Sm}/D^3 to the inverted shear number:

$$S_n = \frac{\rho_o V^2}{\tau_c}$$

where

t_c = Critical tractive shear stress
r_0 = Fluid density
v = Characteristic velocity

According to this report, scour measurements were taken at 31, 100 and 316 minutes for non cohesive materials. The measurements for cohesive materials extended to 1000 minutes. A comparison of the 316 minutes scour hole dimensions with the 1000 minutes scour dimensions indicated that although the duration of scour extended 216 percent longer, the scour hole dimensions increased in an average by 14 percent in depth, 7 percent in width, 16 percent in length and 46 percent in volume. The authors concluded that the scour mechanics seemed to approach a state of equilibrium.

Blaisdell (9) wrote a report about methods for analyzing scour at cantilever outlets. In this study he summarizes data that had been requested by Williams in 1962. These included basically cantilever outlets with excessive scour, that had been repaired or modified to control the extent of the scour, and those which had carried full pipe flow or at least 5 storms and the scour hole was stable. The following parameters were considered important in that research:

1. Pipe diameter
2. Discharge
3. Relative discharge: $\frac{Q}{\left(gD^5\right)^{1/2}}$
4. Flow time
5. Bed material classification, according to the Unified Soil Classification.

The latter parameter seems to be very important. Bratcher (11) reported that the limiting scour depth, and the volume of the scour hole are functions of three variables : discharge, total head, and the soil properties, particularly the plasticity of the soil. In a subsequent paper, Blaisdell and Anderson (10) reported that self-formed scour holes had performed satisfactorily, particularly where the soil is cohesive.

On the other hand, from the results presented in this report, clays of even low to medium plasticity seemed to lead to stable holes in the majority of cases. All of the parameters that were considered important were dimensionless:

- Discharge Intensity: $DI = Qg^{-0.5}D^{-2.5}$
- Distance of the cantilever invert above the tailwater level, (Z_p/D)
- Maximum depth of scour below the tailwater level Z_m, (Z_m/D)
-Distance from the cantilever exit to the maximum depth of scour, X_m/X_j
-Median diameter of the bed material: d_{50}, (d_{50}/D)
-Densimetric Froude number : $Fr_\rho = \dfrac{V_i}{\sqrt{\left(\rho_o/\rho - 1\right)gd_{50}}}$

where

ρ_o= density of the bed material
ρ= density of the fluid.
V_i= velocity at the pipe outlet
g= gravity acceleration.
d_{50}= particle size of which 50 % is finer by weight

The conclusions of this report were basically the following:

- Maximum depth of scour $Z_m/D = 5$, for typical cantilever outlets.
- The average distance from the pipe exit to the point of deepest scour X_m (the center of the scour hole) in terms of the free-falling and submerged jet trajectory length is $X_m/X_i = 1.3$
- Widening of the perimeter, together with the change in slope (formation of "beaches") is produced in laboratory, not in the field.

In a later report, Blaisdell and Anderson (10) developed equations that can be used to predict the time development and ultimate sizes of scour holes in natural noncohesive soils. These equations were developed for designing a pipe plunge pool energy dissipater.

Conclusions of this report can be summarized as follows:

- The ultimate limits of the scour hole can be approached asymptotically.

- Special care must be taken when selecting an intensity discharge. A maximum possible discharge is used. However, at a lesser discharge the jet may attack the upstream slope of the larger discharge plunge pool.
- Scour hole shape was approximated as an ellipse.
- Formulas for getting the scour hole axis coordinates were also developed as follows:

Shafai-Bajestan and Albertson (53) analyzed the stability of riprap below pipe outlets. A general relationship at the point of incipient motion was first developed, then the necessary coefficients were determined by an experimental study program. Basically the effects of riprap gradation on stability were studied.

- For uniform riprap having a minimum rock size considerably larger than the maximum size of bed material and thickness of less than $3D_{100}$, the removal of the bed material prior to any rock movement causes formation of a scour hole beneath the riprap layer that results in instability of the riprap layer.
- The effect of riprap gradation on sizing the riprap can be accomplished using the characteristic particle size. All data of uniform riprap and graded riprap can be correlated very well if the following particle size is used as the representative particle size.
- D_{30} particle size should be used for incipient motion.
- D_{90} particle size should be used for incipient failure.
- Penetration length(P_1) (the length of the path of the jet after it strikes the pool) should be used instead of the tailwater depth.

According to Shafai-Bajestan and Albertson, Blaisdell and Anderson's method overestimates the riprap size significantly.

Techniques for Relating Space and Time to Erosion

Several culvert scour investigators have incorporated time progression of scour into their analysis. However, Thomas (61) is the only reference to include time as a parameter in the estimation of plunge pool erosion. As presented by Mason and Arumugam (41), from Group V of their comparison Table 3, the incorporation of time did not clearly show better results than those of other approaches. In their discussion of the approach of Thomas, Mason and Arumugam state:

> *As early as 1950, Rouse commented that "scour was proportional to the geometrical progression of time and such a final equilibrium depth could not be expected." The results of experiments which considered the rate of scour rather than an ultimate depth were presenter by Doddiah and Thomas (22). Thomas, however, adapted his formulas by putting time*

equal to a sufficiently large number to give expressions for long-term, ultimate scour depth.

The Group V equation by Thomas gave coefficients of variation for the results of 41.89% for models, but 326.79% for prototypes; the latter being principally due to one dramatically high overestimation of prototype scour. The high prototype result cannot be ignored, however, as the same scour data was processed by all the other formulas without producing such an anomaly."

Although Mason, in the development of his erosion equation, did not attempt to include time as a parameter, there is considerable disagreement on the issue. Spurr (57) in his discussion of Mason's paper (41), disagreed that time may be ignored from a practical standpoint, as:

"The discusser disagrees with the authors' suggestion that time, as a parameter, may be ignored for all practical purposes. Any true comparison between prototypes, or between models and prototypes must relate to the same stage of plunge pool development. Although prototype spill duration to equilibrium scour are frequently just a few days, this is by no means always so. Thus the discusser contends that time must be considered together with the unique hydraulic and geological processes existing at each site, the amount of surplus energy contained by a given jet at impaction over and above the threshold resistance of the bedrock to scour (36), the shape of the sides of the scour hole, and the size of the downstream bar in order to determine the plunge pool's maturity before any meaningful comparison can be made. This is specifically true when predicting scour in any plunge pool which does not respond as though its bed material were essentially non-cohesive.

A useful illustration is given by the experience in the plunge pool at Seven Mile Dam (Canada) (Fig. 5). Fig*. 6 presents the corresponding spillway discharge-duration curve recorded for the dam's six years of operation. The maximum depth scoured by December 1984 was approximately 14 m below the initial bedrock level. The sides of the scour hole in the downstream direction are steep and the hole depth increases significantly every year. Fig*. 7 is a plot of the equilibrium scour depth against unit flow calculated using the authors' formula (23) for typical central two-gate discharges where: $k = 1.81$; $v = 0.30$; $w = 0.15$; $x = 0.42$; $y = 0.415$; $z = 0.10$ and $d \approx 1.0$ m (estimated from site). Since the relative differences between the changes in the reservoir and tailwater levels with discharges tend to be small, $H = 53$ m was assumed constant.*

The Seven Mile jet energy available to scour is low in comparison to other large dams while the resistance of the plunge pool bedrock to scour is high. Relating Fig. 6 to Fig*. 7, it is evident that the plunge pool is still very immature. Moreover it is not unreasonable to suppose that, if after six years of operation, the scour depth has only reached 14 m with the aid of*

blasting, anything like the natural maturity of the scour hole is unlikely during the expected life of the project."

Mason's response to Spurr's discussion regarding effect of time on scour depth was:

> *"One of the most important issues mentioned concerns the time taken for maximum scour to develop. The broad range of opinion on this in the literature is amply illustrated by Whittaker and Spurr. The former demonstrates that time can be effectively ignored, the latter demonstrates that it cannot be. Although neither the effects of time nor rock type could be readily detected by the analyses carried out in the paper, the writer suspects that in some cases, noticeably those featuring hard, massive rock, flow duration may be a significant factor. The quantification of plunge pool scour rates, in conjunction with varying rock types is an area where much useful research remains to be done."*

PART II GEOTECHNICAL AND GEOMECHANICAL TECHNOLOGY

This part presents geotechnical and geomechanical information related to the erosion phenomenon. In this sense, geotechnical refers to the micro material properties such as cohesion, plasticity, moisture content, and others. Geomechanical refers to the macro material properties, the interparticle forces and behavior evidenced in both granular and solid materials.

The section on Geotechnical Technology is primarily an extraction from texts on the subject. The purpose of the section is to serve as a reference to non-Geotechnical engineers working on this project. The Geomechanical Technology section presents methods of indexing earth materials. A geomechanical index is a quantitative means of describing widely varying properties and characteristics of earth materials.

Geotechnical Technology

Most of the material in this section is based upon textbook definitions of geotechnical properties. In order to investigate erosion in earth materials, a systematic method of characterizing the properties is essential. The definitions and equations present here are industry standards.

Definitions, Units, Equations

Plasticity: Plasticity is defined as the ability of a substance easily to undergo considerable shearing deformations without rupture (62). Plasticity is a colloidal property, because no mineral possesses plasticity unless it is reduced to a powder consisting of particles of colloidal sizes (59).

Atterberg developed a method of describing quantitatively the effect of varying water in the consistency of fine-grained soils. He established stages of soil consistency and defines definite but arbitrary limits for each. These are the so called Atterberg limits.

a. The liquid limit (LL) is defined as the water content at which a trapezoidal groove of specified shape, cut in moist soil held in a special cup, is closed after 25 taps on a hard rubber plate (56).

b. The plastic limit (PL) is the water content at which the soil begins to break apart and crumble when rolled by hand into threads 3 mm (1/8 inch) in diameter.(56)

c. The shrinkage limit (SL) is the water content at which the soil reaches its theoretical minimum volume as it dries out from a saturated condition.

A complete description of the procedures for determining the Atterberg limits can be obtained from the ASTM regulations or by Head (30).

In themselves the Atterberg limits mean little, but as indexes to the significant properties of a soil are very useful. The liquid limit has been found to be directly proportional to the compressibility of a soil. The difference between the liquid and plastic limits, termed as the plasticity index (PI), represents the range in water contents through which the soil is in plastic state.

The activity , A, is the ratio of plasticity index to percentage of clay sizes (finer than 0.002 mm)

$$A = \frac{P.I.}{(\% < 0.002mm)}$$

The water-plasticity ratio or liquidity index, I_L, relates the water content of the soil to the liquid and plastic limits.

$$I_L = \frac{\left(w/_{100\%}\right) - P.L.}{P.I.}$$

The tests are usually standardized on the portion of soil finer than 0.42 mm, passing No 40 sieve: fine sizes and smaller.

Cohesion is defined as capacity to resist shearing stresses (59). Soils in which the adsorbed water and particle attraction work together to produce a body which holds together and deforms plastically at varying water contents are known as cohesive soils or clays (56).

Shape (extracted from Introductory soil mechanics and foundations, by Sowers (56)) of particles is fully as important as the size in determining the engineering behavior of soil and clastic rock. Three classes of grain shape has been defined: bulky grains, flaky or scale like grains, and needle like grains.

Bulky grains: When the length, width, and thickness of the particles are of the same order of magnitude, the shape is bulky. Two aspects of the bulky shape are significant: the sphericity and the angularity or roundness. The sphericity describes the differences between length L,

width B, and thickness H. The equivalent diameter of the particle De is the diameter of the sphere of equal volume. Other parameters used for defining bulky shapes are the flatness, F and the elongation E. Angularity or roundness, R is a measure of the sharpness of the corners. It is defined as the ratio of the average radius of corners and edges to the radius of the maximum inscribed sphere. Due to the difficulty of getting this measure, particles are often describe qualitatively.

Flaky grains have very low sphericity (typically less than 0.01); they are thin but not necessarily elongated. The form from the mechanical weathering or disintegration of the micas, but the predominant flaky grains are the clay minerals. Small amounts of flaky mica can change the behavior of a predominantly bulky grain soil. The flakes act like springs separating the bulky grains and making the soil resilient and fluffy.

Needle like grains with elongated particles (E greater than 100) occur in some coral deposits and in the attapulgite clays. The particles are resilient and break easily under load.

Soils composed of bulky grains are capable of supporting heavy static loads. Vibration and shock, however, cause them to be displaced easily.. Soils composed of flaky grains tend to be compressible and deform easily under static load. They are relatively stable when subjected to shock and vibration. The presence of only a small percentage of flaky particles is required to change the character of the soil and to produce the typical behavior of a flaky material.

Stresses have been analyzed by scientists related to material properties. Otto Mohr devised a graphical procedure for solving the equations for shear and normal stress on a plane perpendicular to one principal plane and making an angle with the larger of the two principal planes. In soil mechanics experimentation, forces are applied in perpendicular planes; Mohr's procedure is used in most cases for getting shear stresses in the plane of failure.

One of the earliest methods for testing soil strength, used extensively today is direct shear. A sample of soil is placed in a rectangular box, the top half of which can slide over the bottom half. The lid of the box is free to move vertically, and to it is applied the normal load, Q. A shearing force, F, is applied to the top half of the box, shearing the sample along plane x-x

The most reliable shear stress for routine work is the triaxial direct stress. A cylindrical sample is used, typically with a diameter of 36 mm, 72 mm, or more and a length of a least twice the diameter. The sample is encased in a rubber membrane, with rigid pistons at both ends. It is placed inside a closed chamber and subjected to a confining pressure S3 on all sides or water pressure. An axial stress S1 is applied to the end of the sample by a piston. Either

the axial stress can be increased or the confining pressure decreased until the sample fails in shear along the diagonal plane or a number of planes. The Mohr circles of failure stresses for a series of such tests, using different values for S3 are plotted, and the Mohr envelope drawn tangent to them.

A special case of the triaxial shear is the unconfined compression test, in which S3=0. The important advantages of the method are the relative uniform stress distribution on the failure plane and the freedom of the soil to fail on the weakest surface. A major disadvantage is the elaborate equipment required.

A soil classification system is an arrangement of different soils into groups having similar properties. The purpose is to make it possible to estimate soil properties or capabilities by association with soils of the same class whose properties are known and to provide the engineer with an accurate method of soil description (56). The most common soil classification systems are:

- AASHTO (American Association of State Highways and Transportation Officials, 1945) This system divides all soils into three categories:

  Granular: with 35% or less passing a No 200 sieve (finer than 0.074 mm)
  Silt-clay: with more than 35% passing the No 200 sieve
  Organic soil

  The first two categories are subdivided further, depending on their gradation and plasticity characteristics. Some of the classes are subdivided to indicate differences in plasticity, but the subdivisions are not an essential part of the system. The classification is supplemented by the group index or GI :

  $$GI = 0.2\,a + 0.005ac + 0.01bd$$

  a= percentage passing the No 200 sieve greater than 35 and not exceeding 75; expressed as a whole number (0 to 40)

  b= percentage passing No 200 sieve greater than 15 and not exceeding 55; expressed as a whole number (0 to 40)

  c= that portion of the liquid limit greater than 40 and not exceeding 60; expressed as a whole number (0 to 20)

d= that portion of the plasticity index greater than 10 and not exceeding 30; expressed as a whole number (0 to 20)

- Unified Soil Classification system. In this system the soils are first divided into coarse-grained and fine-grained classes. The coarse- grained soils have over 50% by weight coarser than 0.075 mm (No 200 sieve). They are given the symbol G if more than half of the coarse particles by weight are coarser than 4.75 mm (No 4 sieve) and S if more than half are finer. The G or S is followed by a second letter that describes the gradation: W, well graded with little or no fines P, poorly graded, uniform, or gap-graded with little or no fines; M, containing clay or sand and clay. The fine-grained soils (over half finer than 0.075 mm) are divided into three groups: C, clays; M, silts and silty clays; and O, organic silts and clays. These symbols are followed by a second letter denoting the liquid limit or relative compressibility; L , a liquid limit less than 50; and H, a liquid limit exceeding 50.

- The Casagrande plasticity chart is the basis for dividing the fine-grained soils.

Core drilling: When a soil encounters a material so hard that its standard penetration resistance, N, exceeds 100 blows, further progress with soil-boring equipment is difficult and often impossible. This resistance is termed refusal. This indicates a very hard soil, a boulder, a cemented seam, am obstruction, or rock. (56)

Core drilling is used to penetrate such hard materials in order to determine whether refusal represents continuous hard formations, such as rock or simply a boulder or other obstruction underlain by softer materials. Holes 0.6 to 1.4m in diameter permit an engineer or geologist to examine the strata in place, but the cost of drilling is great. Small-diameter cores that are brought to the surface reveal the composition, soundness, and defects of the rock for great depth at a moderate cost (56).

Diamond drilling is the most common method for obtaining small-diameter cores. Although the detailed procedures must be adapted to the rock and its fracture patterns, the ASTM standard D-2113 is suited to a wide range of conditions.

Gradation and gradation parameters

Coarse-grained soils are grouped on the basis of the percentage of gravel, sand, fines, and the shape of the grain- size distribution curve. The coefficients of uniformity C_u and gradation C_g are used to judge the grain size distribution curve of coarse -grained soils. The empirical formulas for these coefficients are (63):

$$C_u = \frac{D_{60}}{D_{10}}$$

$$C_g = \frac{D_{30}^2}{D_{10}D_{60}}$$

Another measure of uniformity, frequently encountered in geologic work is the sorting coefficient, S_o. It is defined by the relation (56)

$$S_o = \sqrt{D_{75}/D_{25}}$$

Gradation is estimated by the same criterion as for the Unified Classification System. A well graded soil complies with the following requisites:

$$\frac{D_{60}}{D_{10}} > 4$$

$$1 < \frac{D_{30}^2}{D_{60}D_{10}} < 3$$

Cohesive Soils: Review and Summary

This is a brief review of previous research efforts on erosion of cohesive clays. This research shows the complexity of the erosion mechanism of cohesive soils. There are a large number of soil properties that influence the erodibility of cohesive soils. Table 3.3 (Shaikh (55)) presents a summary of the results of selected studies on the erosion of cohesive soils. Bulk soil parameters such as plasticity index and vane shear strength are not primary indices of erosion potential, but can be used as a means of identification. Parameters that describe the physicochemical behavior, such as sodium adsorption ratio, cation exchange capacity, salt concentration, and temperature give better indications of soil erodibility. Paaswell (1973) in a review of literature of cohesive soil erosion concluded that structural indices that reflect particle orientation, separation, previous stress history, and the strength and number of interparticle bonds need to be defined and related to the erodibility of a soil.

Table 3.3. Summary of the results of selected studies on erosion of cohesive soils.

Parameter	Investigator(s)	Testing Apparatus	Soil Type	Sample Preparation	Criterion of Erodibility	Conclusion, Order of Erodibility
Type of clay	Grissinger (1966)	Flume	Mixtures of commercial clays with a silt loam	Static compaction (unsaturated)	Erosion rate	Ca-mont. > Kaolinite > Illite > Na-mont
	Liou (1970)	Rotating disk	Pure clays	Static compaction	τ at which erosion stopped	At high water content: Na-mont. > Kaolinite At low water content: Kaolinite slaked
	Sherard et al. (1973)	Field study	Dispersive clays		Piping failure	Montmorillonite was the main clay in all dams which were damaged.
	Alizadeh (1974)	Rotating cylinder	Mixtures of commercial clays with silica flour	Saturated, consolidated from slurry	τ for zero erosion rate	At low SAR (flocculated condition): Kaolinite > Illite > Na-mont At high SAR (Dispersed condition): Na-mont. > Illite > Kaolinite
	Kandiah (1974)	Rotating Cylinder	Mixtures of commercial clays with Yolo loam silt	Saturated, consolidated from slurry	τ for zero erosion rate	SAR = 2.5: Kaolinite > Illite Na-mont. SAR = 48.0: Na-mont. > Illite > Kaolinite
	A. Shaikh et al. (1986)	Flume	Pure clay and ground silica	Static compaction	Erosion rate	Order of erodibility is Ca-montmorillonite > Kaolinite > Na-montmorillonite Erosion rate for Na-montmorillonite

Parameter	Investigator(s)	Testing Apparatus	Soil Type	Sample Preparation	Criterion of Erodibility	Conclusion, Order of Erodibility
Compaction water content	Grissinger (1966)	Flume	Mixture of commercial clays with a silt loam	Static compaction	Static compaction	For Grenada silt loam: low water content > high water content. For a 15% coarse Kaolinite - Grenada silt loam mixture: erosion rate follows a shaped curve
	Liou (1970)	Rotating disk	Pure clays	Static compaction	τ at which erosion stopped	Na-mont. at high water content Na-mont. at low water content
	Kandia and Arulanandan (1974)	Flume	Yolo loam at low SAR	Impact compaction	τ for zero erosion rate	At water content above optimum: Low water content > High water content At water content below optimum: $\tau_c = 0.0$
	Lewis and Schmidt (1977)	Pinhole	Silty clay	Kneading compaction	Erosion rate	Minimum amount of erosion occurred at a water content close to the plastic limit of the soil
	A. Shaikh et al. (1986)	Flume	Pure clay and ground silica	Static compaction	Erosion rate	Erosion rate of Kaolinite decreased with increasing water content.
Chemical composition of pore water	Grissinger (1966)	Flume	Mixture of commercial clays with a silt loam	Static compaction	Erosion rate	Ca-mont. > Na-mont.
	Sherard et al. (1973, 1976b)	Field study and pinhole test	Dispersive clay		Piping failure	Soils with pore fluid of flow salt concentration and high SAR was common in damaged dam

Parameter	Investigator(s)	Testing Apparatus	Soil Type	Sample Preparation	Criterion of Erodibility	Conclusion, Order of Erodibility
	Liou (1970)	Rotating disk	Pure Clay	Static compaction	τ at which erosion stopped	At high water content: Na-mont. + NaCl > Na-mont. + $CaCl_2$ At low water content: Na-mont. + NaCl > Na-mont. + $CaCl_2$ at the beginning Na-mont. + $CaCl_2$ > Na-mont. + NaCl (0.IN) after 90 min.
	Sargunam et al.	Rotating cylinder	Yolo loam	Saturated consolidated from slurry	Erosion rate and τ for zero erosion rate	Sodium soil > Calcium soil low salt conc. > high salt conc.
	Alizadeh (1974)	Rotating cylinder	Mixtures of commercial clays with silica flour	Saturated consolidated from slurry	τ for zero erosion rate	For a constant SAR: soil with low salt conc. > soil with high salt conc. For a constant salt concentration: soil with high SAR > soil with low SAR
Chemical composition of pore water	Kandish (1974)	Rotating cylinder	Mixtures of commercial clays & Yolo loam silt	Saturated consolidated from slurry	τ for zero erosion rate	For salt concentration = 20 meq/l soil with high SAR > soil with low SAR
	Raudkivi and Tan (1984)	Rotating cylinder	Commercial clays	Saturated consolidated from slurry	Erosion rate	The effects of salt concentration of NaCl and pH dominated shear stress
pH	Liou (1970)	Rotating disk	Kaolinite	Static compaction	τ at which erosion stopped	Soil with high pH > soil with low pH
	Kandiah (1974)	Rotating cylinder	Mixtures of commercial clays & Yolo loam silt	Saturated consolidated from slurry	τ for zero erosion rate	Soils with high pH > soils with low pH

Parameter	Investigator(s)	Testing Apparatus	Soil Type	Sample Preparation	Criterion of Erodibility	Conclusion, Order of Erodibility
	Kandiah (1974)	Rotating cylinder	Mixtures of commercial clays with Yolo loam silt	Saturated consolidated from slurry	τ for zero erosion rate	Soils with high pH > soils with low pH
Composition of eroding fluid	Liou (1970)	Rotating disk	Pure clays	Static compaction	τ at which erosion stopped	Domestic water > CCl_4
	Sherard et al. (1972)	Field study	Dispersive clay		Piping failure	Low salt concentration in eroding water > High salt concentration in eroding water
	Arulanandan et al. (1975) and Sargunam et al. (1973)	Rotating cylinder	Yolo loam	Saturated consolidated from slurry	τ for zero erosion rate	When concentration of eroding water was less than that of poor water, the soil was more erodible
Temperature	Grissinger (1966)	Flume	Mixed	Static compaction	Erosion rate	High temp. > low temp.
	Liou (1970)	Rotating disk	Pure clay	Static compaction	τ at which erosion stopped	High temp. > low temp.
	Christensen & Das (1973)	Tube	Mixed	Compacted	Erosion rate	High temp. > low temp.
	Kelly and Gularte (1981)	Water tunnel	Illitic clay	Remolded soft	Erosion rate	High temp. > low temp.
Dry density	Grissinger (1966)	Flume	Mixtures of commercial clays with a silt loam	Static compaction	Erosion rate	Soils with low density > soil with high density
	Liou (1970)	Rotating disk	Pure clays	Static compaction	τ at which erosion stopped	Na-mont. with low density > Na-mont. with high density
Plasticity Index (PI)	Smerdon and Beasley (1961)	Flume	Eleven Missouri soils	Soil was placed with compaction	τ for "general movement"	$\tau_c = 0.0034\ (PI)^{0.84}$

Parameter	Investigator(s)	Testing Apparatus	Soil Type	Sample Preparation	Criterion of Erodibility	Conclusion, Order of Erodibility
	Liou (1970)	Rotating disk	Pure clays	Static compaction	τ at which erosion stopped	Inverse correlation between plasticity index and τ_c
	Kamphuis and Hall (1983)	Flume-tunnel	Cohesive sediment	Consolidated from slurry	τ at which small pit marks appeared	Soils with low PI > soil with high PI
Vane Shear strength (S_v)	Dunn (1959)	Submerged vertical jet	Sandy to silty clay	Consolidated saturated	τ at which water becomes cloudy	$\tau_c = .001(S_v + 180)\tan(30 + 1.73\ PI)$ For PI = 5-16
	Liou (1970)	Rotating disk	Pure clays	Static compaction	τ at which erosion stopped	No changes is S_v with different chemical additives was observed. There was a correlation between and S_v for various water content
	Kamphuis and Hall (1983)	Flume-tunnel	Cohesive sediment	Consolidated from slurry	τ at which small pit marks appeared	$\tau_c = 3.8 + 0.55\ S_v/10^3$

Ca-mont. = Ca-montmorillonite
Na-mont. = No-montmorillonite
τ = tractive stress
τ_c = critical tractive stress

Geomechanical Technology

Geomechanical Resistance

The literature survey reveals that practical methods for determining the ability of earth material to resist erosion utilize geomechanical indexing techniques. These indexing techniques include the Erodibility Probability Index, Cameron (13), Kirsten's K, (35), van Schalkwyk's "Possibility of Movement" index (van Schalkwyk, (66), and Hanson (29).

The Erosion Probability Index of Cameron (13) is the sum of the Geotechnical Erosion Probability Index (EPI_g) and the Hydraulic Erodibility Probability Index (EPI_h). The value of EPI_g is determined by adding the rippability (E_r) and continuity parameters (E_c). The rippability parameters are based on the rippability rating chart of Weaver (69), whereas the continuity parameter accounts for vertical and lateral continuity. Cameron determined the Hydraulic Erodibility Probability Index by physical hydraulic model studies.

Van Schalkwyk's (66) "Possibility of Movement" Index is defined by the number of degrees of freedom of movement of a rock block out of the rock mass and the directions in which movement can take place. The degrees of freedom are defined as the 4 horizontal and 2 vertical mutually perpendicular directions, along which the block can theoretically move. A loose block on a horizontal plane have, e.g. 5 degrees of freedom.

Kirsten's K (35) Index is based on Barton's Q system (5), and was originally developed to determine the rippability of rock and other earth materials. The Index has subsequently been modified for determining the erodibility of earth materials, and was calibrated by making use of 100 field observations on the erodibility of unlined emergency spillways and by making use of published data on initiation of sediment motion of cohesionless material. The method was verified with data from the plunge pool of Bartlett Dam, Salt River Project, Arizona and appears to be reliable.

Hanson's (29) Jet Index is intended to rate the relative ability of soils to resist erosion; rock is not dealt with by his method. The Jet Index in essence utilizes a standardized hydraulic jet which is projected into the soil. The relative ability of the soil to resist erosion is determined by measuring the depth of scour per unit time.

Other indexing techniques which are used to classify rock, and which may be modified to express the relative ability of earth material to resist erosion, include the indexing techniques of Terzaghi (59), John (33), Coates (16), Deere and Miller (20), Wickham (214), Bieniawski (6), Bieniawski (7), Aufmuth (3), Barton (4), Weaver (69), and Laubscher (37). None of these

methods have, as far as could be established, previously been used to determine the relative ability of earth material to resist erosion.

A number of publications which deal with various aspects of the behavior of rock are presented in the list of publications. These publications can be used to gain understanding of the behavior of rock which is subjected to hydraulic attack by discharging water.

Predicting Erodibility of Earth Material Due to Hydraulic Action

The complexity and importance of hydraulics in determining the erodibility of rock in emergency spillways were recognized by Cameron (13), who proposed the Erosion Probability Index, EPI, for determining relative erodibility of rock. The Erosion Probability Index is the sum of the Geotechnical Erodibility Index and the Hydraulic Erodibility Index. The Hydraulic Erodibility Index must be determined from hydraulic model studies.

Powledge (47) reported the findings of a comprehensive study pertaining to the mechanics of overflow erosion on embankments. Although this study did not deal directly with erosion of dam foundations due to overtopping, it provides insight into the erosion processes in such cases. The authors conclude e.g. that the use of tractive stress to calculate erosion in such cases may be questionable. This appears to be true after surface discontinuities develop due to the process of erosion. Another important observation is that erosion is often initiated at the point of slope discontinuities. Once erosion at such locations has been initiated, the process is often completed by means of head cutting. Similar observations were made by Cameron (13), who concluded that the occurrence of erosion usually shows a strong correlation with geometrical features of spillway channels, such as nick points, gradients and channel drops.

The second set of observations pertaining to the process of erosion deal with the mechanism of hydraulic removal of earth material. Spurr (58) suggests that the phase difference between the turbulent hydraulic pressure fluctuations, that act at the bedrock surface and those in the bedrock itself, develop differential pressures within the fractured rock mass. These differential pressures cause rocks to move out of their positions of rest, where they are removed by the flowing water.

Spurr's hypothesis was confirmed by Reinius (50), who performed model tests to measure water pressures around a simulated rock block. He concluded that pressures caused by water propagation into cracks of the rock act on the bottom and sides of the rock, whereas pressure variations from the flowing water act mainly on the top surfaces of the rock fragments. He found that the resistance to uplift forces depends on the shape, weight of the rock blocks and

shear forces between adjoining blocks. Rocks with planes or joints dipping downstream, as well as rocks of poor quality with closely spaced, open joints are more prone to erosion than rocks with upstream dipping joints. He found that the turbulence in the water generated fluctuating forces which affected the joints and induced instability of the block.

Kirsten (35) and Annandale and Kirsten (2) suggest that the process of erosion is caused by fluctuating differential pressures over earth material units and show, by making use of research results by Fiorotto and Rinaldo (26), that the rate at which energy is dissipated is a good indicator of the magnitude of such pressure fluctuations. By using an indexing system to represent the relative ability of earth material to resist erosion, they conclude that the mass strength of earth material, block (or particle) size, interparticle strength, the relative shape of blocks (or particles) and the dip and strike of the earth material play dominant roles in determining the ability of earth material to resist erosion.

Research by Mohamed and McCorquodale (44) on the nature of short-term local scour indicate that local scour downstream of an apron with a swept-out hydraulic jump is shown to develop very rapidly, that is in less than 1% of the time to reach the ultimate scour depth. The short term scour, although not as deep as the ultimate scour, was found to occur much closer to the apron. This process might play and important role in initiating nick point erosion, and could be the reason for its apparent importance. The researchers observed that seven jump types occur during this dynamic process.

Discussions of the erodibility of earth material necessitates a knowledge of hydraulic forces. There are many studies that provide insight to the character of impinging jets, plunge pool energy dissipation, and dynamic pressure fluctuations. However, current knowledge of the hydraulics, aeration effects and spatial distribution of an impinging jet are limited to studies with a relatively narrow scope. Hydraulic studies that have been cited in relation to predicting erosion are discussed. Many other relevant studies are listed in the references.

Ervine and Falvey (25) deal with the behavior of turbulent water jets in the atmosphere and in plunge pools. Their paper is essentially a summary and combination of theory and observations, and can be used to estimate the erosive power of jets plunging into pools. However, in applying the technology in this paper it is important to ensure conservation of mass in the calculations. This requirement, although logical, is not specifically dealt with by the authors and may lead to erroneous results if omitted. The information provided in this paper is adequate to estimate the rate at which energy is dissipated as the jet plunges through the atmosphere and discharges into the water in the plunge pool. By estimating the rate at

which energy is dissipated in the plunge pool, it is possible to determine the relative magnitude of fluctuating, turbulent pressures.

Fiorotto and Rinaldo (26)(27) deal with the ability of hydraulic jumps to erode concrete slabs of aprons. The importance of these two papers is found in the ability of the authors to mathematically describe the relationship between hydraulic agitation and the resistance of the concrete slabs to erosion, and to successfully relate this detailed mathematical description with experimental observations. They measured differential pressure fluctuations which are caused by the action of an hydraulic jump, information which was used by Annandale and Kirsten (2) to relate the rate of energy dissipation to the relative magnitude of pressure fluctuations. The fundamental nature of Fiorotto and Rinaldo's research makes their findings suitable for further development and application in developing numerical techniques to predict the erodibility of dam foundations due to overtopping.

Palmerton (46) reports the findings of tests which were conducted to study head cutting mechanisms in emergency spillway channels. Head cutting is an important aspect of the erosion of dam foundations due to overtopping, as it often occurs as the next phase in the process which was initiated by nick point erosion. Palmerton (46) concludes that the partial vacuum beneath the nappe of the over falling water plays an important role in determining the relative magnitude of head cutting action. Head cutting action is associated with drop structure hydraulics, an analogy that has been used by Annandale and Kirsten (2) to derive equations to determine the erosive power of back rollers and to describe the erosive power of discharging water at nick points.

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ABSTRACTS OF SELECTED REFERENCES

Abdelkawi A.M., McCorquodale, J.A.	Simulation of the radial hydraulic jump	Journal of Hydraulic Research, Vol. 30, 1992, No. 2	1992	The radial hydraulic jump has been simulated using a strip integral mathematical model. The model is calibrated against the writers' experimental data. The strip integral method uses velocity shape functions to permit the partial integration of the equations of motion. A Gaussian velocity distribution is used in the mixing zone and the power law is used in the inner layer. The mathematical model includes the bed shear, turbulent shear, the potential core, entrained air, centrifugal force and turbulence pressure. The strip integral method has been applied to obtain a set of first order ordinary differential equations to describe the internal velocity distribution and water surface profile for a diverging radial hydraulic jump. These differential equations are solved by a fifth order Runge-Kutta method.
Abt, S.R.	Scour at culvert outlet in cohesive bed materials	Colorado State University - Dissertation	1980	The culvert has long been the primary means of conveying tributary drainage through roadway and construction embankments. As drainage waters discharge from the culvert and impinge upon the underlying material, degradation occurs to areas beneath and adjacent to the culvert outlet. In an attempt to predict, control and manage localized scour at culvert outlets, a series of investigations have been performed beginning in the 1940's. However, these scour studies were performed using cohesionless materials with the results extrapolated and applied to cohesive materials. This is the first study investigating scour at culvert outlets in a cohesive bed material. It was determined that scour and erosive indicators for cohesive materials differ from those of cohesionless materials. A series of empirical relationships were developed expressing the depth, length, width and volume of scour as a function of the discharge, culvert diameter, culvert outlet velocity, plasticity index, vane shear strength and soil water content. Scour cavity dimensions were correlated to the discharge intensity ($Q\ g^{-0.5}\ D^{-2.5}$) and the modified shear number ($\tau_c \rho^{-1} V^{-2}$). General observations concerning cavity formation, growth and stabilization were noted. Based upon these relationships, three design procedures for estimating the depth, length, width and volume of scour are presented.

Akhmedov, T.Kh.	Calculation of the depth of scour in rock downstream of a spillway.	Water Power and Dam Construction.	1988	Akhmedov divided the development of a scour hole into three stages, characterized by the relation between the hydrodynamic force P, the weight of the rock fragments in water G and the cohesive force between fragments and surrounding rock P_c. Phase I: $P > G + P_c$, rock scours Phase II: $P >= G + P_c$. scour decreases but energy still adequate to induce vibrations. Phase III: $P < G + P_c$, scour ceases. The author provides a graphical relationship between scour depth and time for Kariba, Picoti and Farchad Dams. By making use of the jet velocity and the permissible velocity for rock erosion, he proposes an equation which can be used to calculate the local scouring depth.

AUTHOR	TITLE	PUBLICATION	DATE	ABSTRACT
Annandale, G.W., Kirsten, H.A.D.	Determination of the erodibility of natural and engineered earth: Hydraulics	(final draft of manuscript prepared and ready for submittal)	1993	This paper is a companion to the paper of Kirsten, Moore and Annandale which presents the erodibility criterium that was found from the analysis of 100 field observations by the SCS pertaining to the erodibility of emergency spillways and published laboratory data pertaining to the initiation of sediment motion. The criterium was found by relating the relative ability of earth material to resist erosion to the erosive power of water. The conceptual model which was used to analyze the field and laboratory data is based on the concept of progressive dislodgement of earth material, a process which is initiated and maintained by fluctuating hydraulic pressures. The premise that the fluctuating hydraulic pressures are the primary cause of hydraulic erodibility is, in principle, confirmed by the findings of Fiorotto and Rinaldo (1992b). They concluded that the greater the fluctuation in hydraulic pressure, the higher the likelihood of erosion damage at hydraulic structures. The hydraulic pressure fluctuations are initiated by the turbulence of water discharging over or incident to a boundary. Equations representing the relative magnitude of the fluctuating turbulent pressures were therefore derived to represent the magnitude of the erosive power of water for use in the analysis of the field and laboratory data. The selection of the hydraulic parameter which was used to represent the relative magnitude of the erosive power of discharging water is justified by findings of further analysis of work by Fiorotto and Rinaldo (1992a). The outcome of this analysis confirms a strong relationship between the magnitude of the fluctuation of turbulent pressures and the rate at which energy is dissipated in turbulent discharging water. It was therefore decided to use estimates of the rate of energy dissipation in turbulent discharging water as a measure of the magnitude of its erosive power. The equation which were used in the analysis of the field and laboratory data to estimate the rate of energy dissipation at hydraulic jumps, headcutting situations, grade changes and in channel flow are presented in this paper.

Bandis, S. Lumsden, A.C. Barton, N.R.	Experimental Studies of Scale Effects on the Shear Behaviour of Rock Joints	Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol. 18, pp. 1-21 Pergamon Press Ltd.	1981	The effect of scale on the shear behaviour of joints is studied by performing direct shear tests on different sized replicas cast from various natural joint surfaces. The results show significant scale effects on both the shear strength and deformation characteristics. Scale effects are more pronounced in the case of rough, undulating joint types, whereas they are virtually absent for planar joints. The key factor is the involvement of different asperity sizes in controlling the peak behaviour of different lengths of joints. It is shown that as a result both the joint roughness coefficient (JRC) and the joint compression strength (JCS) reduce with increasing scale. The behaviour of multiple jointed masses with different joint spacing is also considered. It is found that despite unchanged roughness, jointed masses consisting of many small blocks have higher peak shear strength than jointed masses with larger joint spacing. These scale effects are related to the changing stiffness of a rock mass as the block size or joint spacing increases or decreases. Economic methods for obtaining scale-free estimates of shear strength are described.
Bandis, S.C. Barton, N.R. Christianson, M.	Application of a new numerical model of joint behavior to rock mechanics problems	Proceedings of the International Symposium on Fundamentals of Rock Joints/Bjorkliden	1985	A comprehensive numerical model of rock joint behavior has been developed, which enables discrete simulation of effective normal stress and scale dependent coupling of shear stress, shear displacement, dilation and closure. Changes in fluid conductivity are also modeled for both normal and shear deformation. The model is based on three key sets of input parameters, namely the joint roughness coefficient (JRC_0), the joint wall compression strength (JCS_0) and the initial conducting aperture (e_0). This paper presents a brief review of the modeling principles and some practical examples to demonstrate the potential for tackling various rock engineering problems, such a rock bolt design, grout take estimates, petroleum reservoir settlement, stress-path errors, and site characterization for nuclear waste disposal.

Bandis, S.C. Lindman, J. Barton, N.R.	Three-dimensional stress state and fracturing around cavities in overstressed weak rock	1st SRMJ International Congress on Rock Mechanics, Proceedings Vol. 2 Montreal	1987	Investigations of the stress-state and the failure modes of unlined cylindrical cavities in poor quality ground were conducted by utilizing instrumented 3-D physical models. Stress redistribution around an opening in overstressed porous material seems to be influenced by an associated compaction/strain-hardening material, shearing is the dominant failure mode. The failure planes develop in a pattern resembling logarithmic spiral slip lines. There are indications that a tensile stress field could develop locally under anisotropic stresses. Closed-form solutions, based on conventional failure criteria, show a moderate approximation to the physical reality.
Bandis, S.C. Vardakis, G.	Inability and stress transformations around underground excavations in highly stressed anisotropic media	Rock at Great Depth, Maury & Foumintraux (eds), Balkema, Rotterdam. ISBN 9061919	1989	The anisotropic behaviour of most geological masses has well-recognized implications upon design of underground excavations. Most of our experiences derive from near-surface structures and conventional stress levels. Accumulating evidence, however, indicates that the traditional inference of quasi-isotropic behaviour under high stresses can be misleading. This paper presents the results from physical model tests of circular openings simulated highly stressed formations with anisotropic mechanical characteristics.
Bandis, S.C. Lumsden, A.C. Barton, N.R.	Fundamentals of Rock Joint Deformation	Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol. 20, No. 6, pp. 249-268, 1983 Pergamon Press Ltd.	1983	This paper describes laboratory investigations of the deformation characteristics of rock joints under normal and shear loading. Normal deformability was studied by conducting loading/unloading and repeated load cycling tests on a wide variety of fresh and weathered joints in five different rock types. The data invariable showed non-linear behaviour, irrespective of the rock and joint type. A hyperbolic function is suggested to describe the stress-closure/opening curves of joints. Quantitative relations between normal deformability and relevant joint parameters (aperture, wall strength and roughness) are developed. Tentative conclusions on the changes in normal stiffness during shearing are also presented. The behaviour of dislocated (mismatching) joints is studied qualitatively and analytically. Shear deformability was studied by performing direct shear tests under normal stresses in the range of engineering interest. It is shown that behaviour in the pre-peak range is invariable non-linear depending on the joint type, and can be adequately described by easily measured parameters and hyperbolic functions.

Barton, N. Harvik, L. Christianson, M. Bandis, S.C. Makurat, A. Chryssanthakis, P. Vik, G.	Rock mechanics modeling of the ekofisk reservoir subsidence	27th Symposium on Rock Mechanics	1986	The large jointed chalk reservoir situated at 8 km depth in the North Sea's Ekofisk field is undergoing major compaction after nearly 15 years of oil and gas production. Approximately 150 km of the overlying sediments, mostly shales, are involved in the subsidence. A maximum central subsidence of nearly 3 meters, and a maximum present rate of 45 cm per year has set in motion numerous studies of the phenomenon. Non-linear finite element (FEM) and non-linear distinct element (DEM) analyses of the compaction and large scale subsidence were performed. Consistent differences between the continuum and slip on hypothetical bedding planes and sub vertical or vertical faults was allowed, which possible gives a more realistic simulation of the real processes of subsidence in such a large body of rock. Laboratory studies of the reservoir joints include roughness measurement, JRD and JCS characterization, direct shear tests while saturated in heated Ekofisk oil, and coupled closure-shear-flow tests with heated oil followed by heated sea water. Discontinuum modeling using cundall's UDEC was performed on representative jointed assemblies (two sets of steeply dipping conjugate joints) to investigate the effect of a major reduction of pore pressure within the deformable matrix and along each joint. It was found that the large shrinkage deformation of the matrix allowed joint shearing to occur despite the constraint of uniaxial strain. Relative mass bulking due to small but widely distributed joint shear possibly explains the observed maintenance of excellent productivity despite large vertical strains in the reservoir.
Barton, N. Bandis, S.	Review of predictive capabilities of JRC-JCS model in engineering practice	Rock Joints, Barton & Stephansson (eds) 1990 Balkema, Rotterdam,	1990	The database used in developing the Barton-Bandis joint model is reviewed. It is shown how tilt testing to obtain JRC is demonstrated, and relationships with J_r in the Q-system are developed. Constitutive modeling of shear stress-displacement, dilation and shear reversal are also described.

Barton, N. Bandis, S. Bakhtar, K.	Strength, Deformation and Conductivity Coupling of Rock Joints	Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol. 22, No. 2, pp. 121-140, 1985 Pergamon Press Ltd.	1985	Construction of dams, tunnels and slopes in jointed, water-bearing rock causes complex interactions between joint reformation and effective stress. Joint deformation can take the form of normal closure, opening, shear and dilation. The resulting changes of aperture can cause as much as three orders of magnitude change in conductivity at moderate compressive stress levels. Even the heavily stressed joints found in oil and gas reservoirs may also exhibit significant stress-dependent conductivity during depletion, and during water flood treatments. The magnitudes of the above processes are often strongly dependent on both the character and frequency of jointing. In this paper the results of many years of research on joint properties are synthesized in a coupled joint behaviour model. Methods of joint characterization are described for obtaining the necessary input data. The model simulates stress- and size-dependent coupling of shear stress, displacement, dilation and conductivity, and of normal stress, closure and conductivity. These processes are fundamental building blocks of rock mass behaviour. Model simulations are compared with experimental behaviour and numerous examples are given.
Barton, N. Lingle, R.	Rock Mass Characterization Methods For Nuclear Waste Repositories in Jointed Rock	ISRM Symposium / Aachen	1982	The planned isolation of nuclear waste in mined rock repositories poses unusual requirements for rock mass characterization. This paper describes recently developed block test methods for characterizing and quantifying the thermal, mechanical and hydraulic properties of rock masses. The heated block test, recently conducted in situ on an $8m^3$ block of jointed gneiss, provides normal stress and temperature-dependent data such as defortivity and dynamic elasticity modulus. Simpler tests conducted on singly jointed blocks or jointed drill core provide joint roughness data. This is incorporated in recently developed constitutive models which describe the coupling of normal displacement, shear displacement, shear strength, dilation and permeability.
Barton, N.	Estimating the Shear Strength of Rock Joints	Advances in Rock Mechanics Vol. 11, part A. 3rd Congress of the Int. Soc. for Rock Mechanics	1974	A new criterion for the peak shear strength of rock joints is summarized. It can be used both to fit experimental data and to predict it. The predicted strength is sensitive to the following: degree of surface roughness, compressive strength of the rock, degree of weathering, mineralogy, presence or absence of water.

Barton, N.	Model Studies of Very Large Underground Openings at Shallow Depth	ISRM Int. Congress on Rock Mechanics, Vol. 1, Monheux	1979	Thirteen carefully scaled physical models (scale 1:300) were used to study the two-dimensional deformation resulting from excavation of very large span openings in near-surface rock masses. The openings were excavated in stages up to final simulated spans of 50 meters. In some cases more than one opening was excavated in parallel. The models consisted of at lease 20,000 discrete blocks. Joint orientations and stress levels were varied, and some models were dynamically loaded to simulate strong earthquakes (peak horizontal acceleration 0.2 - 0.7g).
Barton, N. Lien, R. Lunde, J.	Engineering Classification of Rock Masses for the Design of Tunnel Support	Rock Mechanics 6, 189-236 by Springer-Verlag	1974	An analysis of some 200 tunnel case records has revealed a useful correlation between the amount and type of permanent support and the rock mass quality Q, with respect to tunnel stability. The numerical value of Q ranges from 0.001 (for exceptionally poor quality squeezing-ground) up to 1000 (for exceptionally good quality rock which is practically unjointed). The rock mass quality Q is a function of six parameters are as follows: the RQD index, the number of joint sets, the roughness of the weakest joints, the degree of alteration or filling along the weakest joints, and two further parameters which account for the rock load and water inflow. In combination these parameters represent the rock block-size, the interblock shear strength, and the active stress. The proposed classification is illustrated by means of field examples and selected case records. Detailed analysis of the rock mass quality and corresponding support practice has shown that suitable permanent support can be estimated for the whole spectrum of rock qualities. This estimate is based on the rock mass quality Q, the support pressure, and the dimensions and purpose of the excavation. The support pressure appears to be a function of Q, the joint roughness, and the number of joint sets. The latter two determine the dilatency and the degree of freedom of the rock mass. Detailed recommendations for support measures include various combinations of shotcrete, bolting, and cast concrete arches together with the appropriate bold spacings and lengths, and the requisite thickness of shotcrete or concrete. The boundary between self supporting tunnels and those requiring some form of permanent support can be determined from the rock mass quality Q.

Barton, N.	Rock Slope Performance as Revealed by a Physical Joint Model	Advances in Rock Mech. Rep. of Current Res. Vol. 11, part B. proc. 3rd congress of the Int. Society of Rock Mechanics 1-7	1974	"Two-dimensional" physical models (scale 1:500) were used to study the deformation and failure of slopes cut through jointed rock masses. The models were discontinuous each consisting of approximately 4000 discrete blocks. Three sets of joints were modeled with angles of dip 0°, 66° and 90°, using a specially designed machine that generates tension fractures. The models were consolidated and stressed by rotating the model plane from a horizontal to vertical position, while increasing the triangular distribution of horizontal stress. Slopes were excavated in the models after loading, in a attempt to simulate the excavation of slopes in jointed rock, while under realistic distributions of stress. Two models which were identically jointed are compared. One model was stressed horizontally to approximately half the self weight vertical stress gradient, while the other was stressed to twice the vertical stress gradient. The deformation occurring at the crest of the slopes as a result of stage by stage excavation appeared independent of the horizontal stress level. Contrary to the results of elastic analyses there was no evidence to suggest large stress gradients or failure initiation around the toe of the slopes. The stress distributions that are obtained from simple "depth of overburden" calculations appeared to be adequate for limit equilibrium analyses of the eventual slope failures.

Barton, N.	The Shear Strength of Rock and Rock Joints	Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol. 13, pp. 255-279, Pergamon Press Ltd.	1976	Rock joints exhibit a wide spectrum of shear strength under the low effective normal stress levels operating in most rock engineering problems. This is due to the strong influence of surface roughness and variable rock strength. Conversely, under the high effective normal stress levels of interest to tectonophysicists the shear strength spectrum of joints and artificially faults is narrow, despite the wide variation in the triaxial compression strength of rocks at fracture. In Part I of this review, empirical non-linear laws of friction and fracture are derived which explain this paradoxical behaviour and which can be used to predict or extrapolate shear strength data over the whole brittle range of behaviour. Under higher confining pressures the behaviour of rock ceases to be brittle as the brittle-ductile transition is reached. Expressions are derived which quantify this condition and explain the variable transition behaviour of rocks as dissimilar as limestone and shale. At still higher confining pressures the Mohr envelopes describing failure of intact rock eventually reach a point of zero gradient on crossing a certain line, defined here as the critical state line. This critical state is associated with a critical effective confining pressure for each rock. It appears that the dilation normally associated with the shearing of non-planar joints and faults may be completely suppressed if the applied stress reaches the level of the critical effective confining pressure. The empirical laws of friction and fracture were developed during a review of laboratory-scale testing on rock and rock joints. In Part II of this review these laws are applied to the interpretations of full-scale features. The following topics are investigated; the conjugate shear angle of shear joints and faults, the scale effect on frictional strength, the lack of correlation between stress drops measured in laboratory-scale faulting experiments and those back-calculated from major earthquakes, the strength corrosion caused by moisture, and finally the possible effect of fault dilation and water pressure changes at shallow depth in the crust.

Barton, N. Bakhtar, K.	Instrumentation and Analysis of a Deep Shaft in Quartzite	24th U.S. Symposium on Rock Mechanics	1983	Terra Tek Engineering instrumented and monitored a concrete shaft lining and the surrounding quartzite at levels 2412 ft, 4063 ft, and 5191 ft during sinking of the Silver Shaft at Helca Mining Company's Lucky Friday Mine in Wallace, Idaho. Rock displacements were monitored with three 50 ft long multiple position borehole extensometers at each level. Stress build-up in the concrete lining was monitored with four pressure cells at each level, and the corresponding strains were monitored with embedded strain gages. Data from each level are compared, and a detailed analysis of rock and concrete behavior at the 5191 ft level is given. The results of the rock displacement and concrete lining stress monitoring have through light on several important aspects of deep shaft liner design, and on the behavior of jointed ground at great depth.
Barton, N.	Rock Mass Classification and Tunnel Reinforcement Selection Using the Q-System	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia, pp. 59-88	1988	This paper provides an overview of the Q-system and documents the scope of case records used in its development. A description of the rock mass classification method is given using the following six parameters: core recovery (RQD), number of joint sets, roughness and alteration of the least favorable discontinuities, water inflow, and stress-strength relationships. Examples of field mapping are given as an illustration of the practical application of the method in the tunneling environment, where the rock may already be partly covered by a temporary layer of shotcrete. The method is briefly compared with other classification methods, and the advantages of the method are emphasized.

Barton, N. Choubey, V.	The Shear Strength of Rock Joints in Theory and Practice	Rock Mechanics 10, 1-54 by Springer-Verlag	1977	The paper describes an empirical law of friction for rock joints which can be used both for extrapolating and predicting shear strength data. The equation is based on three index parameters; the joint roughness coefficient JDR, the joint wall compressive strength JCS, and the residual friction angle ϕ_r. All these index values can be measured in the laboratory. They can also be measured in the field. Index tests and subsequent shear box tests on more than 100 joint samples have demonstrated that ϕ_r can be estimated to within $\pm 1°$ for any one of the eight rock types investigated. The mean value of the peak shear strength angle (arctan τ/σ_n) for the same 100 joints was estimated to within ½°. The exceptionally close prediction of peak strength is made possible by performing self-weight (low stress) sliding tests on blocks with thoroughgoing joints. The total friction angle (arctan τ/σ_n) at which sliding occurs provides an estimate of the joint roughness coefficient JRC. The latter is constant over a range of effective normal stress of at least four orders of magnitude. <u>However, it is found that both JRC and JCS reduce with increasing joint length.</u> Increasing the length of joint therefore reduces not only the peak shear strength, but also the peak dilation angle and the peak shear stiffness. These important scale effects can be predicted at a fraction of the cost of performing large scale in situ direct shear tests.

Barton, N.	Recent experiences with the Q-system of tunnel support design	Proceedings of the Symposium on Exploration for Rock Engineering / Johannesburg	1976	The Q-system of rock mass classification and support design is based on a numerical assessment of the rock mass quality using six different parameters. The six parameters consist of the RQD, the number of joint sets, the roughness of the most unfavorable joint or discontinuity, the degree of alteration or filling of the most unfavorable joint or discontinuity, the degree of water inflow, and the stress condition. Another classification system, the Geomechanics Classification (Bieniawski, 1974, 1974) is also based on six parameters. Qualitative differences between the two methods are discussed. The 200 case records that were analyzed when developing the Q-system, include more than 30 cases of permanently unsupported openings. An analysis of the rock characteristics involved has shown that certain characteristics are essential if an excavation unsupported span for a given Q-value is exceeded, the safe life of the excavation may be shortened. A preliminary attempt is made to correlate stand-up time, rock mass quality Q, and span width. The Q-system has been applied on several projects in Scandinavia and abroad since its development in 1973/1974. An example of a recent application is given in detail. The preliminary estimates of permanent support for a 19 metres span underground power house were obtained from an analysis of corelogs. In a subsequent site visit the Q-system was applied *in-situ*. The final estimates of permanent support were found to compare well with the preliminary estimates. Core logs, seismic profiles and surface mapping were used as a basis for preliminary design of permanent support for the 9 metres span tailrace tunnel, again using the Q-system. This tunnel is presently under construction so comparison of predicted and actual support is not yet possible.

Barton, N. Lein, R. Lunde, J.	Estimation of Support Requirements for Underground Excavations	16th Symposium on Rock Mechanics	1975	An analysis of some 200 case records has revealed a useful correlation between the amount and type of permanent support and the rock mass quality Q, with respect to excavation stability. The rock mass quality Q is a function of six parameters, each of which has a rating of importance, which can be estimated from surface mapping and can be updated during subsequent excavation. The six parameters are as follows: the RQD index, the number of joint sets, the roughness of the weakest joints, the degree of alteration or filling along the weakest joints, and two further parameters which account for the rock load and water inflow. In combination these parameters represent the rock block size, the interblock shear strength, and the active stress. Analysis of the rock mass quality and corresponding support practice has shown that suitable permanent support can be estimated for the whole spectrum of rock qualities. Support measures include various combinations of shotcrete, bolting, and cast concrete arches together with the appropriate bolt spacings and lengths, and the requisites thickness of shotcrete or concrete.
Barton, N. Bandis, S.	Effects of Block Size on the Shear Behavior of Jointed Rock	23rd Symp on Rock Mechanics proc. Aug. 25-27	1982	The descriptive term "rock mass" encompasses individual block dimensions ranging from centimeters to many tens of meters. Strength and deformability vary both qualitatively and quantitatively as a result of this size range. A key issue is therefore the appropriate size of the test sample. A large body of test data was reviewed to determine the influence of block size on the displacement required to mobilize peak strength. It is shown that the shear strength and shear joint roughness, and due to reduced asperity strength. Both are a function of the delayed mobilization of roughness with increasing block size. A method of scaling shear strength and shear displacement from laboratory to in situ block sizes is suggested. It is based on the assumption that size effects disappeared when the natural block size is exceeded. This simplification appears to be justified over a significant range of block sizes, but is invalidated when shearing along individual joints is replaced by rotational or kink-band deformation, as seen in more heavily jointed rock masses. Recent laboratory tests on model block assemblies illustrate some important effects of block size on deformability and Poisson's ratio.

Berme, K.J. Sanks, R.L.	Model Study of Brown Canyon Debris Barrier	Proceedings of American Society of Civil Engineers	1946	The principal hydraulic tests of a 1:50 scale model of a debris barrier and the contiguous channel, with some corroborative prototype data, are included in this paper. Tests of the barrier include those for determining the spillway capacity, the performance of the overflow, the development of a stepped overflow crest, the operation of the barrier with and without detrital material impounded upstream from the barrier. Tests in the channel cover the problems of scour downstream and deposition upstream from the barrier. Because of meager field data, especially regarding bed load and rates of transportation, the initial tests of deposition upstream from the barrier were based on bed load and rates of transportation as developed in the laboratory. These results were reasonably verified in subsequent tests, based on data obtained from the first major storm after completion of the prototype structure. A discussion of these data was presented before the Waterways Division at the Los Angeles (Calif.) Meeting of the Society in July, 1943.
Bieniawski, Z.T.	The Rock Mass Rating (RMR) System (Geomechanics Classification) in Engineering Practice	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia, 1988, pp. 17-34	1988	This paper presents a state-of-the-art review of the Geomechanics Classification of rock masses, also known as the RMR System. Developed by the author in 1972-1973, and modified and improved as more case histories became available, the latest version of the system was published in 1979. This paper discusses the principles and philosophy of the RMR System and describes its varied applications. Although originally directed to tunneling in rock, subsequent applications and extensions featured the use of the RMR System in hard rock mining, coal mining, rock slope stability, foundations in rock, rock boreability, and others. These applications necessitated some adjustments to the classification ratings, and these aspects are presented in the paper. The strengths and limitations of the Geomechanics Classification are pointed out. It is emphasized that rock classifications are useful aids in engineering design, but that they form only a part of the design process.
Blaisdell, F.W., Anderson, C.L. Hebaus, G.G.	Ultimate dimensions of local scour.	Journal of the Hydraulics Division, ASCE, Vol 7, HY3	1981	The authors presents a hyperbolic logarithmic mathematical model for determining the ultimate depth of scour.

Blaisdell, F.W., Anderson, C.L.	Scour at cantilevered pipe outlets Plunge pool energy dissipater design criteria	U.S. Department of Agriculture Agricultural Research Service	1989	Cantilevered pipe spillway outlets are used at most farm-pond and many upstream flood-control principal spillways. Allowing pipe spillways to scour their own energy dissipation pools may be acceptable for small farm ponds. However, pre-excavated and riprap-lined plunge pools or more elaborate stilling basins may be required for larger, upstream flood-control reservoirs that flow at full capacity for prolonged periods. An extensive literature review on the general principles of local scour at pipe outlets revealed little practical information on scour and prevention of scour at culvert outlets. There was no specific information on the design of plunge pool energy dissipaters for pipe spillways. Presented is a research plan to provide the needed information, a description of the special experimental apparatus and instrumentation constructed for this study, and an explanation of the experimental methods used. Background information is: a description of the scour process, a method of computing the ultimate scour hole dimensions from short-time tests of the scour progress, and the geometry of the jet after emerging from the pipe. The test program completed and a summary of the test data define the scope of the tests. Equations and procedures for the design of cantilevered pipe spillway plunge pool energy dissipaters resulted from a detailed analysis of the experimental data. The design procedure and equations are summarized and examples are given.
Blaisdell, F.W.	Analysis of scour observations at cantilevered pipe outlets	U.S. Department of Agriculture Agricultural Research Service	1983	The publication uses laboratory-developed procedures to analyze field data collected by the Soil Conservation Service (SCS) on scour hole at cantilevered pipe outlets and compares the field and laboratory findings. Data were compiled on discharge, pipe diameter and slope, soil classification and size, length of time the spillway flowed full, and dimensions of 105 scour holes surveyed in 17 States. Results of the analysis will aid SCS in predicting the sizes of scour holes and plunge pool energy dissipaters at farm pond and flood-prevention cantilevered spillway exits.

Blaisdell, F.W., Anderson, C.L.	Pipe plunge pool energy dissipater	Journal of Hydraulic Engineering, Vol 117, N 3	1991	An extensive program of tests to develop criteria for the design of pipe spillway plunge pool energy dissipaters is planned and carried out. The pipe spillway discharge covers a 10-fold range that includes the minimum full pipe flow and a maximum flow exceeding the practical range, a range of pipe invert heights below and above the tailwater level from -2 to +8 pipe diameters, and a 16-fold range of noncohesive bed material sizes. The time development and the ultimate size of scour holes is determined. The scour hole geometry and other test data are expressed in mathematical equations that can be used to predict the time development and ultimate sizes of scour holes in natural noncohesive soils. These equations can also be used to design riprap pipe spillway plunge pool energy dissipaters. A procedure for the design of pipe plunge pool energy dissipaters is presented.
Blaisdell, F.W. Anderson,C.L.	A comprehensive generalized study of scour at cantilevered pipe outlets	Journal of Hydraulic Research, Vol. 27, 1989, No. 1	1989	The test program and its scope is described. Analyses of the experimental data produced equations describing the scour hole geometry that can be used for the design of cantilevered pipe spillway plunge pool energy dissipaters. Background for these analyses is presented in Part I [3]. The representativeness of these criteria and equations was evaluated and are considered to be satisfactory.
Bremen, R., Hager, W.H.	T-jump in abruptly expanding channel	Journal of Hydraulic Research, Vol. 31, No. 1	1993	Hydraulic jumps in an abruptly expanding, rectangular and horizontal channel are considered. Particular attention is focused on so-called T-jumps, of which the toe is located upstream from the expansion section. A literature review reveals that T-jumps have not been systematically analyzed until now, although considerable effort was made in understanding the phenomenon. Experiments were conducted in two channels. One channel served mainly to obtain results for the present study, while the second was three times larger and was used to check the results for scale effects. The present study aimed at analyzing the effects of expansion ration, inflow Froude number and toe position on the sequent depths ratio, the energy dissipation, the flow depth in the expansion corners, the length of jump and the lengths of the side vortices. The asymmetry of jump was also investigated and it was found that all results may basically be expressed by a new parameter phi according to equation (6) in this paper. the present study clearly indicates that an efficient T-jump without additional baffle and terminal elements are thus unacceptable for thorough energy dissipation.

Bush, D.D. Barton, N.	Pore pressure effects on the minimum principal stress direction in shallow tar sands	26th US Symposium of Rock Mechanics / Rapid City, SD	1985	Shallow tar sands for the most part are located in unconsolidated media. Data interpretation must consider these units as basically a soil where pore pressure will greatly affect the minor principal stress direction. If the tar sands are considered analogous to a confined aquifer, the role of pore (tar) pressure to total stress is more clearly defined. The magnitude and direction of the minimum principal stress will be dependent on the tar pore pressure. Test data show that under fully tar saturated conditions at shallow depth, less than 442 m (1450 ft) the minimum principal stress is vertical, while in undersaturated unconsolidated formations at similar depth, the minimum principal stress is horizontal. Under fully tar-saturated conditions, intragranular pressure would decrease due to dilatancy and the media would become almost cohesionless. Under these conditions the formation stresses would be nearly isotropic with the minimum principal stress equal to the overburden (vertical). At greater depth or where the formation is not fully saturated, a smaller fraction of the overburden load will be carried by the tar due to closer sand grain packing. This increased effective normal stress will reduce Poisson's ratio, resulting in a horizontal minimum principal stress. A vertical minimum principal stress direction will be indicated when the stress gradient would approximately equal 1 [stress (psi)/depth (ft)].
Cameron, C.P., Cato, K.D., McAneny, C.C., May, J.H.	Geotechnical aspects of rock erosion in emergency spillway channels - Analysis of field data and laboratory data.	Tecnical Report, Volume 2, U.S. Army Corps of Engineers, Washington.	1988	Cameron et. al. used a volumetric ranking and horizontal erosion ranking to categorize erosion problems. The volumetric ranking is the ratio between the actual volume of erosion and the volume of erosion which could cause failure of a spillway x 100, whereas the horizontal erosion ranking is the ratio between the horizontal erosion (length of eroded area) and the distance between the end of the eroded channel and the spillway wall x 100. These parameters were used to show how imminent the erosion threat is, and can be used for prioritization. In order to index erodibility they proposed an Erosion Probability Index (EPI), which is based on the concept that the key geotechnical factors controlling erosion during spillway flow are contained in the rock mass rippability and the lithostratigraphic continuity. EPI is the sum of the Geotechnical Erosion Probability Index (EPI_g) and the Hydraulic Erodibility Index (EPI_h). EPI_g is calculated by using the rippability rating chart which takes account of rippability and continuity, whereas EPI_h is determined from hydraulic studies in the laboratory.

Chanson, H.	Aeration of a free jet above a spillway.	Journal of Hydraulic Research, Vol. 29, No. 5	1991	A study of air entrainment above a spillway aerator is presented and discussed with a dimensional analysis. The author concluded that similitude of air entrainment processes for spillway aerator is not possible between model and prototype. New information on the aeration region are presented and an analytical solution of the upper nappe entrainment is developed.
Chryssanthakis, P. Barton, N.	Joint Roughness (JRC_n) characterization of a rock joint and joint replica at 1 m scale	Rock Joints, Barton & Stephansson (eds) 1990 Balkema, Rotterdam,	1990	Characterization of a 1 m long joint sample in syenite has been performed using the JRC-JCS concept and large-scale tilt testing. A replica of the joint has also been made using a rubber molding technique. Tile testing of a surface that has very steep asperities (steps) is investigated first using the natural joint, then using a modified surface with contact points artificially removed. Effects of asperity wear are also evaluated. Successive tilt tests of the model material replicas give tilt angle values with reducing tendency due to continuous breakage of the small asperities.
Coleman, H.W.	Prediction of Scour Depth from Free-Falling Jets	ASCE Hydraulics Division Conference, Jackson, MS	1982.	Abstract: A simple method for predicting limiting scour depth in plunge-pool energy dissipators is proposed. The scour depth as determined by the Veronese formula is compared with scour hole depths observed for several model studies. It was found that the scour depth from the formula, $d_s = 90H^{0.225} q^{0.54}$, is a reasonable estimate for the resulting model scour, when d_s is measured in the direction of the tangent to the jet entering the tailwater. The effect of size of gravel on limiting scour depth appeared to be minimal for the range of sizes included in these tests.

Deere, D.U. Deere, D.W.	The Rock Quality Designation (RQD) Index in Practice	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia pp. 91-101	1988	The Rock Quality Designation (RQD) index was introduced 20 years ago at a time when rock quality information was usually available only from geologist' descriptions and the percent of core recovery. The RQD is a modified core recovery percentage in which unrecovered core, fragments and small pieces of rock, and altered rock are not counted so as to downgrade the quality designation of rock containing these features. Although originally developed for predicting tunneling conditions and support requirements, its application was extended to correlation with *in situ* rock mechanical properties and, in the 1970s, to forming a basic element of several classification systems. Its greatest value, however, remains as an exploratory tool where it serves as a red flag to identify low-RQD zones which deserve greater scrutiny and which may require additional borings or other exploratory work. Case history experience shows that the RQD red flag and subsequent investigations often have resulted in the deepening of foundation levels and the reorientation or complete relocation of proposed engineering structures, including dam foundations, tunnel portals, underground caverns, and power facilities.

Denton, R.A.	Energy curves for multi-layer flow through obstructions	Journal of Hydraulic Research, Vol. 27, 1989, No. 5	1989	One method for obtaining solutions for single-and multi-layer flow over bottom humps and through sidewall contractions is to consider the variation in mechanical energy with layer thickness at key cross-sections. However, previous models of gradually-varying multi-layer flow have been restricted to flows which remain subcritical with respect to all but the slowest internal wave speed. This paper shows how the energy extrema method can be used over the full range of flow conditions; from subcritical with respect to all internal wave speeds to supercritical with respect to the free surface wave. The complete energy curves for three-layer flow are found to have up to seven different energy extrema at a given cross-section. The energy extrema are classified according to whether the flow is critical with respect to the first- or second-mode internal wave or the free-surface wave, whether the energy is a local maximum or minimum, and which layers are active. Froude numbers based on the actual wave speeds are used to determine whether the flow at a given cross-section is subcritical, critical or supercritical with respect to each of the three possible wave modes. All seven energy extrema represent possible energy conditions at a hydraulic control. Which energy extrema are applicable for a particular three-layer flow depends on the obstruction geometry and the upstream and downstream boundary conditions.

Doddiah D., Abertson, M.L., Thomas, R.A.	Scour From Jets	Proceedings, Minnesota International Hydraulics Convention (Joint Meeting of International Association for Hydraulic Research and Hydraulics Division, ASCE), Minneapolis, MN, p. 161	1953	Abstract: Scour from jets of water which might be found under natural conditions was given rather detailed treatment by Schoklitsch in 1035 [5]. In this work, Schoklitsch describes a variety of conditions under which scour might occur and gives data for design of certain structures involving scour. Since that time, however, Rouse [4] and Krumbein [3] have demonstrated that (1) Scour continues directly with a geometric progression of time. (2) Fall velocity of sediment particles is that property which is best related to the behavior of the particles subjected to the influence of moving water. Because of the results of this more recent research, Doddiah [2] in 1949 made a study of scour resulting from circular jets issuing vertically downward onto a bed of alluvial material covered by a pool of water having various depths. More recently Thomas [6] has studied the scour resulting from a two-dimensional jet or sheet of water issuing from a free overfall and impinging on an alluvial bed also covered by a pool of water having various depths. This paper reports the studies made by Doddiah and Thomas and compares the study of Thomas with the equation of Schoklitsch.
Ervine, D.A. Falvey, H.T.	Behaviour of turbulent water jets in the atmosphere and in plunge pools	Pro. Instn Clv. Engrs, Part 2, pp. 295-314	1987	This Paper describes measurements made on a free water jet discharging almost horizontally; from these, the Authors conjecture the fundamental nature of free turbulent jets plunging through the atmosphere and diffusing in the plunge pool. Consideration of jet diffusion in the plunge pool includes a comparison between submerged jet and impinging jet diffusion, the role of inner core decay for impinging jets in a pool, the effect of entrained air on the reduction of impact pressures, and an analysis of the pressure fluctuations caused by turbulence in the plunge pool shear layers.

Fiorotto, V. Rinaldo, A.	Turbulent pressure fluctuations under hydraulic jumps	Journal of Hydraulic Research, Vol. 30, 1992, No. 4	1992	New experimental evidence on the statistical structure of turbulent pressure fluctuations at the bottom of hydraulic jumps is brought in this paper in view of its relevance on stability of the linings in stilling basins. Maximum values and the structures of temporal and spatial correlation of the anisotropic field of fluctuating pressures are described. Pressures are measured in the zones of the jump where the uplift load produced on the slabs is maximum for a 5 ÷ 10 range of Froude numbers. The results define a novel design criterion for hydraulic engineering practice. It is concluded, upon comparison with analogous results from extensive although seemingly incomplete literature, that the systematic experimentation described herein completes the information needed to characterize the statistical structure of pressure fields past hydraulic jumps.
Fiorotto, V. Rinaldo, A.	Fluctuating Uplift and Lining Design in Spillway Stilling Basins	Journal of Hydraulic Engineering, Vol. 118, No. 4, April, 1992.	1992	The paper deals with the characterization and the control of transient uplift generated by turbulent pressure fluctuations in spillway stilling basins. Theoretical and experimental analyses are presented, defining the maximum instantaneous uplift acting on the linings, which lead to a design criterion for suitable floor slabs. The thickness of concrete linings (not accounting for the possible contribution of anchors) required for stability is defined in this paper as a function of (1) The incoming kinetic head and/or Froude number; (2) a dimensionless function of the longitudinal and transversal dimensions of the lining slabs, the areal extent of extreme turbulent pressure fluctuations, and anisotropic integral scales of the instantaneous spatial correlation of pressure fluctuations; (3) the ratio of specific weights of water and immersed concrete; and (4) a dimensionless pressure coefficient qualifying the maximum difference of positive and negative pressure fluctuations about the (stationary) mean value. It is concluded that safe design of protection works is significantly affected by the structure of instantaneous spatial distribution of pressures at the bottom of hydraulic jumps. Consideration of this factor leads to recommendations for the design of slabs in stilling basins.

Hager, W.H., Wanoschek, R.	Hydraulic Jump in triangular channel	Journal of Hydraulic Research, Vol 25, No. 5	1988	The hydraulic jump in the triangular channel is analyzed using an elementary approach. The results regarding the sequent depth ratio are confirmed by model observations. Furthermore, the length characteristics, the surface profiles and typical velocity distributions are discussed. Finally, these results are compared with the hydraulic jump in the rectangular channel. The authors found that the energy loss on the jump in triangular channels is approximately 30% greater than that in rectangular channels, and that the triangular jump is more sensitive to tailwater depths. (The higher rate of energy loss implies higher pressure fluctuations, which for purposes of the BuRec research project implies higher erosive power.)
Hager, W.H. Li, D.	Sill-controlled energy dissipater	Journal of Hydraulic Research, Vol. 30, 1992, No. 2	1992	The effect of a continuous, transverse sill on the hydraulic jump in a rectangular channel is analyzed. Based on the results pertaining to the classical jump, the sill-controlled jump may be shown to correspond to a perturbed classical jump, particularly as regards to overall jump pattern. A novel normalization procedure is introduced by referring to the roller length of classical jump. Further, the two- and three-dimensional flow patterns are discussed. The design procedure involves both the hydraulics and the erosion sensitivity of tailwater bed.
Hager, W.H. Bremen, R.	Classical hydraulic jump: sequent depths	Journal of Hydraulic Research, Vol. 27, 1989, No. 5	1989	The effect of wall friction on the sequent depths ratio in classical hydraulic jumps is analyzed. Based on experimental data, an approach is presented by which it is seen that the sequent depths are not only influenced by the inflow Froude number, but also by the inflow Reynolds number, and the inflow aspect ratio. The prediction is compared to own and other experimental data, and a fail agreement between the two is found. Finally, an expression is given for the limit condition for which scale effects are still absent. All the results are presented in terms of jump inflow quantities and thus ensure a simple application to design problems.
Hager, W.H.	B-jump in sloping Channel	Journal of Hydraulic Research, Vol. 27, 1989, No. 1	1989	The overall phenomenon of a hydraulic jump in prismatic, rectangular channels, whose bottom slope is considerable in the upstream and horizontal in the downstream reach is investigated. Based on detailed observations, expressions are derived for the sequent depths, the jump efficiency, the length characteristics and the horizontal bottom force component.

Hallmark, E.	Influence of particle size gradation on scour at base of free overfall	Colorado State University - Thesis	1955	Hallmark studied the influence of particle size gradation at the base of a free overfall. Basically his research focused on armor plate design for avoiding excessive scour. His most important conclusions are: a) a 50 % decrease in the standard deviation of the size distribution resulted in a 50% increase in depth of scour when $qT/H^2 = 3 * 10^5$; b) only a relatively small amount of armor plating material is necessary for a relatively large decrease in the rate of scour; c) the rate of scour decreases with a decrease in the size of the armor plate material while the armor plate material remains larger than the largest particle size of the bed material; d) the rate of scour decreases with an increase in the amount of armor plate placed in the scour hole; and e) graded armor plate material decreases the rate of scour more effectively than uniform material.
Hanson, G.J.	Development of a Jet Index to Characterize Erosion Resistance of Soils in Earthen Spillways	Transactions of the ASAE (Vol. 34, No. 5, pp. 2015-2020, 1991)	1991	A soil parameter is developed based on concepts from local scour studies of non-cohesive and cohesive soils and the results of a site-specific submerged jet testing device. Submerged jet test results on four soils with a range of physical conditions are analyzed. The submerged jet had a nozzle diameter of 13 mm, and was set on a jet height of 0.22 m prior to testing the soil. The soils were tested over a range of jet velocities at the nozzle of 166 cm/s to 731 cm/s. The results indicate that erosion, expressed as the depth of scour divided by time, in the site-specific submerged jet testing device may be related to the jet velocity, a time function, and a soil parameter (jet index, J_i). The jet index is intended to provide a common method of expressing erosion resistance, to assist those who work with different soils and soil conditions for measurement with design, and possibly to develop performance and prediction relationships for earthen spillways. Keywords. Spillways, Soil erodibility, Jet index.

Harza	Plunge Pool Performance Study: Project Data/Literature Search. Theodore Roosevelt Dam. Lower Colorado Region. Salt River Project, Arizona		1993	The US Bureau of Reclamation (Reclamation) proposes to raise the Theodore Roosevelt Dam by approximately 77 feet in order to provide increased flood protection for downstream areas, including Phoenix, Arizona. As a part of their design, the existing plunge pool was re-evaluated and redesigned with the aid of a physical hydraulic model. Based on the model study, along with geologic and other design considerations, a pilot plunge pool was designed and included in the contract documents of on-going construction. As a separate and independent task, Harza was asked by Reclamation to provide a procedure for analyzing expected plunge pool performance whereby the long-term performance of the plunge pool at Theodore Roosevelt dam in particular, and any other plunge pool in general, could be predicted. The plunge pool performance prediction was to be based on a literature search of model experience and prototype case histories where available, with consideration of geology as well as hydraulics.
Kawagoshi, N. Hager, W.H.	B-jump in sloping channel, II	Journal of Hydraulic Research, Vol. 28, No. 4	1990	This second paper on B-jumps deals with flows in a 30° upstream sloping, and a horizontal downstream channel portion. The main flow features such as the sequent depths, the lengths of both the roller and the jump are investigated. Further, two-dimensional velocity fields for typical flow patterns are discussed and compared with those for other hydraulic jumps.

King, D.L.	Hydraulic Model Studies for Morrow Point Dam	USBR Engineering Monograph No. 37	1967	Abstract: The preliminary design of an overflow spillway at the crest of the dam and a slide-gate-controlled outlet works near the dam base was abandoned because of undesirable flow conditions in the stilling basin resulting from high velocity efflux from the outlet works. An alternate design, consisting of four fixed-wheel-controlled outlets near the dam crest and a small outlet works near the base, was recommended following several modifications. A 1:24 scale model was used in developing the design of the spillway and to determine the hydraulic operating characteristics of the recommended free fall orifice-type spillway, slide-gate controlled outlet works, and tailrace channels of the underground powerplant. Comprehensive data were obtained concerning pressure distribution in the spillway conduits and bellmouth entrances, on the stilling basin floor, and on the stilling basin weir. Minor modifications were made to the topography in the tailrace area to improve flow conditions during large spillway discharges. The stability of riprap protection was determined, and tests were made concerning movement of material in the stilling pool. Operating characteristics of all features of the recommended design were observed.

Kirsten, H.A.D.	Case Histories of Groundmass Characterization for Excavatability	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia pp. 102-120	1988	A classification system that has been developed for the characterization of the excavatability of soils and rocks is reviewed briefly. The system comprises an adaptation of the Norwegian Q classification developed for the characterization of tunnel support. It enables a through-going classification of geotechnical materials from soft soils to hard rocks to be made. Characterization with regard to single line ripping and bucket hoeing is also provided. The system has been applied for a number of years to a variety of problems and has generally enabled reasonably accurate predictions to be made of mechanical excavatability. Several case histories are described in the paper. These include the diggability of a 30 km long trench for a 2.3 m diameter pipeline, the rippability and dozability of construction materials from shallow borrow pits in geological deposits in limited depth, the excavatability of the terraces for two 50 ML bulk water storage reservoirs, and the excavatability by ripping and blasting of a railway cutting of some 850,000 m^3 in volume. It is demonstrated that the variable conditions in each of the examples could be reasonably accurately characterized for excavatability by means of the classification system; the latter is based on the rockmass parameters which affect ploughsharing or equivalent comminution processes.
Kirsten, H.A.D., Moore, J.S., Annandale, G.W.	Empirical Classification for hydraulic erodibility of natural and engineered earth	(In preparation)	1993	The paper describes the Erodibility Index method which can be used to determine the relative ability of earth and engineered earth material to resist erosion. The index takes account of mass strength, block size, interparticle strength, relative shape and dip of the material. The index values can be obtained from a set of tables which utilizes either field or laboratory data to set values. The indexing method was originally derived from Barton's Q system, and was previously successfully applied to determine the rippability of earth material.
Kohl, R.E.	Rock tunneling with high speed water jets utilizing cavitation damage.	Rock tunneling with high speed water jets utilizing cavitation damage.	1968	The erosion strength is defined as the energy absorbing capacity of the material up to fracture under the influence of erosive forces. With the strength known, the erosion intensity (defined as the power absorbed by the unit area of the eroded portion of the material) is given by: $I_e = (i/t)\ S_e$, where Ie is the erosion intensity (power/area), i is the depth of erosion (length), t is the exposure time (seconds) and S_e is the erosion strength (force/area).

Kuti, E.O.	Erosion of cohesive bed downstream a spillway	Howard University - Thesis	1973	Kuti wrote about erosion of cohesive bed downstream a spillway. In this study, he analyzed the relationship between parameters that are inherent to cohesive soils (such as plasticity, liquid limit, etc) to scour. Plasticity is a valid soil index to compare the erodibility of two types of soil, provided that all conditions are equal. Since electrochemical processes take place in clays, contain of chemicals in water can determine erosion. As a result, percentage of clay and plasticity index are good indicators of erosion if the chemical contain of water is known. In addition, he showed that void ratio can be a functional variable of rate of scouring in inactive clays (those that has no ability to adsorb water). Furthermore, inplace void ratio has direct influence on when scouring reaches an equilibrium condition.
Long, D. Steffler, P.M. Rajaratnam, N.	LDA study of flow structure in submerged hydraulic jump	Journal of Hydraulic Research, Vol. 28, 1990, No. 4.	1990	This paper presents the results of a Laser Doppler Anemometry (LDA) study of submerged hydraulic jumps in a horizontal rectangular channel of constant width with the submergence factor S varying approximately from 0.20 to 1.70 and inlet Froude number F_1 approximately equal to 3.0, 5.5 and 8.0. Measurements include surface profiles, mean velocity components of u and v, turbulence shear stress - $u'v'$ *and turbulence intensities.* Major flow characteristics of submerged hydraulic jumps are discussed and analyzed. The flow in the fully developed region is found to have some degree of similarity. It is also found that a submerged jump is three dimensional in nature.
Long, D., Steffler, P.M., Rajaratnam, N.	A numerical study of submerged hydraulic jumps.	Journal of Hydraulic Research, Vol. 29, No. 3	1991	A standard two-dimensional k-epsilon model is used to predict the mean flow and turbulence characteristics of submerged hydraulic jumps. An offset control volume method is developed to facilitate computation of the variable free surface. The numerical predictions are compared with experimental measurements under three conditions with supercritical Froude numbers ranging from 3.2 to 8.2 and submergence factors ranging from 0.24 to 0.85. Finally the numerical performance is evaluated and discussed in detail. It is concluded that the model is adequate for predicting the surface profile, mean velocity field and to some extent, the turbulence structure of submerged hydraulic jumps.
Lowe III, J. Chao, P.C. Luecker, A. R.	Tarbela service spillway plunge pool development	Water Power & Dam Construction	1979	The two spillways at Tarbela dam have been in operation since 1975. This article summarizes observations on one of the plunge pools during four flood seasons and briefly describes the works being constructed to control further development of the pool.

Mason, P.J.	Erosion of plunge pools downstream of dams due to the action of free-trajectory jets	Proc. Instn of Civil Engineers	1984	Mason proposes that the erosion resistance, S_e (MPa), of the bedrock may be presented by: $S_e = f[\sigma_c (RQD)] = I_r = (T/D)$, where σ_c = the uniaxial compressive strength , RQD is the rock quality designation, I_r = the threshold absorption of deflection power per unit area of the bedrock beyond which the rock will degrade, T (seconds) is the giver period for the scour process and D (m) is the depth of scour.
Mason, P.J.	Effects of Air Entrainment on Plunge Pool Scour	Journal of Hydraulic Engineering, Vol. 115, No. 3, ASCE	1989	Plunge pool scour depths are normally assumed to be dependent on unit flow *q* and head drop *H*. It is hypothesized by the writer that this process may be linked to plunge pool air entrainment. Tests are carried out using a hydraulic model on which *q*, *H* and the air/water ratio ß in the plunge pool could each be varied separately. Scour depths are found to depend only on *q* and ß with the apparent effects of *H* recorded in the past being possibly due to associated variations of ß with *H*. Equations are developed explaining the results in terms of forces on the particles of bed material. These are shown to apply to a wider body of model test data and also to prototype data. Particular areas of relevant future research are also identified. It is proposed that any future plunge pool scour studies should recognize air entrainment as a significant variable.
Mason, P.J. Arumugam, K.	Free Jet Scour Below Dams and Flip Buckets	Journal of Hydraulic Engineering, Vol. 111, No. 2, ASCE	1985	Formulas proposed to date for calculating ultimate scour depth under jets, such as issue from free dam overfalls and flip buckets, are examined. The accuracies of the formulas are evaluated by using each to process sets of scour data from prototypes and models of such prototypes. It is established that scour depth is as adequately calculable using only unit flow *q* and head drop *H* as using more complex considerations, but that those formulas most applicable for model purposes are not those best for prototypes. It is demonstrated that where bed particle size *d* is also considers, the use of the mean particle size d_m is more appropriate than the d_{90} size. Chute friction loss allowances and jet impact angle are also examined in terms of improved accuracy; the relevance of Froude law scaling for this type of scour is verified. Lastly, a new formula for predicting ultimate scour depth, which includes an allowance for tailwater depth *h* is presented and shown to give improved accuracy for both models and prototypes.

Mason, P.J.	Erosion of Plunge Pools downstream of dams due to the action of free-trajectory jets	Pro. Instn Clv. Engrs, Part 1, pp. 523-537	1984	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 analyzing 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 models and prototypes.
Mendoza, C.	Headwall influence on scour at pipe outlets	Colorado State University - Thesis	1980	The object of this study was the investigation of the headwall influence on the scour phenomenon downstream of culvert outlets. Scour cavities produced by different discharges of varying duration through circular culverts onto a horizontal sand bed, were observed an mapped. The data were obtained for a uniform sand with mean size, d_{50} = 1.86mm and a standard deviation s = 1.33. The culvert's diameter was 4 inches (102 mm). These data were used to develop expressions describing depth, length, and volume of the scour hole as function of time and of the Discharge Intensity ($Q\ g^{-0.5}\ D^{-2.5}$) at the culvert outlet. Results are presented in graphical form. A criterion is given for designing the depth below the bed level of the upstream headwall at culvert outlets. Detailed design examples are presented which demonstrate the use of the headwall criterion and predict the scour hole geometry.

Mohamed, M.S., McCorquodale, J.A.	Short-term local scour.	Journal of Hydraulic Research, Vol 30, No. 5	1992	Local scour downstream of an apron with a swept-out hydraulic jump is shown to develop very rapidly, that is in lees than 1% of the time to reach the ultimate scour depth. The short-term scour although not as deep as the long-term scour, occurs much closer to the apron. The limiting short-term scour depth is found to be related to flow regime of type of hydraulic jump that dominates the flow in the scour hole. The deepest short-term scour was associated with the plunging jump (B-jump) and the adverse jump regimes. A modification of the B-jump equation of Hager and Bretz is proposed to estimate the limiting depth of short-term local scour. They identified seven jump types which occur during the development of short term scour. (George Annandale's conclusion is that this study demonstrates why nick point erosion often initiates the head cutting process.)

Moore, J.S.	Critique of the Rock Material Field Classification Procedure	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia, pp. 52-58	1988	The Soil Conservation Service uses the Rock Material Field Classification Procedure (TR-71) to classify rock and assess rock performance for engineering purposes. The classification process consists of identifying the rock units at the site of investigation and describing the rock in terms of its rock material, rock mass, and hydrogeologic properties. The performance assessment is the selection of performance objectives according to the engineering use of the rock. Objectives include erosion resistance, excavation characteristics, construction quality, water transmission, and rock mass stability. The rock is then classified according to criteria in the appropriate performance assessment tables. The procedure uses established methods and tests developed in some widely accepted rock mass classifications, particularly the Unified Rock Classification System (URCS). The URCS is included in its entirety in Appendix 1 of TR-71 as a convenient reference, since it is a major part of three of the five performance assessment tables. The prominent attributes of the procedure include: (1) rock can be classified according to its intended use, (2) terms are nontechnical and unambiguous, (3) classification elements are based on observable and measurable field conditions, (4) significant properties can be quickly tested in the field, (4) the need for additional evaluation can be recognized, and (6) a rock mass can be assigned to groups with similar behavior. The main caveats of the procedure include: (1) some of the criteria in the performance assessment tables are empirical, (2) the procedure is new and not widely accepted, and (3) the classification elements are not weighted.
Moore, J.S.	Standard Field Procedures for Evaluating the Geological Parameters controlling Hydraulic Erosion	(final draft of manuscript prepared and ready for submittal)	1993	Hydraulic erodibility of natural or engineered earth materials, including both soil and rock, may be evaluated in terms of an empirical correlation between stream power and an erodibility classification of the materials as developed by Kirsten, Moore and Annandale. The classification system generates an empirically determined index that enables characterization of virtually any type of earth material. The geologic parameters that constitute the index include earth mass strength, particle or block size, discontinuity/inter-particle bond strength, and relative orientation and shape of material units. This paper presents field procedures and terminology use in the determination of these parameters.

Neelamani, S. Sundar, V. Vendhan, C.P.	Wave induced dynamic pressures on a vertical cylinder in the diffraction regime	Journal of Hydraulic Research, Vol. 27, 1989, No. 5	1989	The dynamic pressures around a large diameter vertical cylinder resting on the flume bed and piercing the free surface are measured under regular wave conditions and compared with the results of MacCamy and Fuch's linear diffraction theory. The linear diffraction theory has been modified to account for the nonlinear effects. The results mostly pertain to deep water conditions with drag effects being negligible and are presented in nondimensional form. The agreement between the theory and experiments are in general found to be good.
Nicholson, G. A.	A Case History Review from a Perspective of Design by Rock Mass Classification Systems	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia pp. 121-129	1988	The design of the Park River Auxiliary Tunnel is reviewed. The tunnel is a 2800 m (9100 ft) long, 6.7 m (22 ft) inside diameter inverted siphon-shaped tunnel constructed to bypass flood water beneath the city of Hartford, Connecticut, and transmit it directly to the Connecticut River. The tunnel was designed by and constructed for the New England Division of the U.S. Army Corps of Engineers. The approach selected for design consisted of a rational evaluation of recommended supports and rock load heights as determined from two rock mass classification design system. Rock load heights predicted by the two classification systems were cross checked with a discrete element numerical model. The three-tiered approach of checks and balances incorporated into the design of the Park River Auxiliary Tunnel resulted in significant cost savings in final support. A comparison is also made between support requirements recommended by two additional classification systems.
Ohtsu, I., Yasuda, Y., Yamanaka, Y.	Drag on vertical sill of forced jump	Journal of Hydraulic Research, Vol. 29, No. 1	1991	As a basis for designing a forced-hydraulic-jump-type stilling basin, the pressure magnitude on the upstream and downstream faces of a continuous-vertical sill has been investigated experimentally, and the nature of the pressure distribution on the faces of the sill has been discussed. Furthermore, an experimental formula for the drag force acting on the sill has been proposed considering the characteristics of the flow condition over the sill. The authors offer three equations for calculating the drag force, viz. for Type I and II jumps and for the Spray condition.

Ohtsu, I., Yasuda, Y.	Transition from supercritical to subcritical flow at an abrupt drop.	Journal of Hydraulic Research, Vol. 29, No. 3	1991	This paper presents a systematic investigation on the transition from supercritical to subcritical flow at an abrupt drop. Over a wide range of experimental conditions various types of flow have been classified and the concept of low and high drops defined. When the flow on the step is supercritical, the direction of the main flow passing over the step can be decided from the momentum equation. The reason why various flow conditions are formed has been explained. The hydraulic conditions required to form various types of flow and their length characteristics have been clarified. This paper provides insight into the hydraulic conditions at the end of sills, e.g. spillway sills and can be used to calculate the rate at which energy is dissipated (and therefore the erosive power at such locations.)
Opie, T.R.	Scour at culvert outlets	Colorado State University - Thesis	1967	The procedures used in, and results of, experiments to determine the size and geometry of scour holes in flat, loose rock beds at culvert outlets are given. A review of four recent approaches to the problem is also included. From a dimensional analysis, the depth of scour at such an outlet is related to the discharge and bed characteristics. The depth of scour has then been related to the length, width and volume of scour. The relations are severely restricted in their application to the range of outlet conditions. Practical examples of data use are given. Results are presented in graphic form.
Palmerton, J.B. May, J.H. Banks, D.C.	Geotechnical Aspects of Headcutting in Emergency Spillway Channels	Bulletin of the Association of Engineering Geologis Vol. XXVIII, No. 1, 1991	1991	Severe headcutting at moderate flows within spillway channels at public and private reservoirs have caused the Corps of Engineers to reexamine the processes causing channel erosion. The process of headcutting could result in undermining and failure of flood control structures. Flume experiments which simulated the flow over overfalls showed that a partial vacuum pressure may develop beneath the nappe of the overfalling water plume. This partial vacuum will cause the plume to be pushed (by the atmospheric pressure) closer to the face of the overfall and a reverse scour roller to undermine the materials of the overfall. The propensity for partial vacuum formation to cause rapid headcutting is more predominant during low to moderate channel discharges.

Pemberton, E.L., Lara, J.M.	Computing degradation and local scour	Bureau of Reclamation	1984	Several methods are presented in this technical guide, which can be applied to estimate degradation of a stream channel, occurring because of changes in flow regimen or reduced sediment load below a dam or diversion structure. Estimation of degradation include two types; those limited by channel armoring and degradation limited by a stable slope. Detailed procedures are introduced to use in estimating maximum scour depth of channels at bridges or siphon crossings.
Powledge, G.R. Ralston, D.C. Miller, P. Chen, Y.H. Clopper, P.E. Temple, D. M.	Mechanics of Overflow Erosion on Embankments. I: Research Activities	Journal of Hydraulic Engineering, Vol. 115, No. 8, August 1989 ASCE	1989	Part 1 of a two-part report presents model and prototype research studies which have been conducted in the United States and Great Britain to evaluate how embankments for dams, levees, roadways, etc. perform when subjected to overtopping flows from probable maximum flood (PMF) or near-PMF events. With improvements in the collection of flood records and the development of estimated storm events a significant increase in predicted PMF has been realized in some areas of the United States and other countries. These studies provide the engineer with an understanding of the mechanics of overflow erosion through subjecting scale model and prototype embankments to overtopping flow events. The effectiveness of various protection systems in preventing overflow erosion is evaluated. These systems range from grass-lined embankments and spillways to more sophisticated protection using geotextiles, gabions, riprap, cellular concrete blocks and soil cement. Such research efforts provide for the development of cost-effective measures that allow embankments to pass these extreme flood events without breaching.
Rajaratnam, N., Pochylko, D.S., Macdougall, R.K.	Further studies on the erosion of sand beds by plane water jets	University of Alberta, Edmonton, Alberta, Canada	1981	This report presents the results of two further studies on the erosion of sand beds by plane turbulent water jets. The first study considers the erosion of sand beds by obliquely impinging plane turbulent submerged water jets. The second study considers the erosion of sand beds by plane turbulent water wall jets with the tailwater depth approximately equal to the jet thickness.

Reinius, E.	Rock Erosion	Water Power and Dam Construction	1986	Reinius performed model tests to measure water pressures around a simulated rock block. Pressures caused by water propagation into cracks of the rock, act on the sides and bottom of a rock block where pressure from flowing water, acts mainly on the surface of the top fragments. The resistance to uplift force depends on the shape, weight of rock blocks and shear forces between adjoining blocks. Because of turbulence of the flow, the forces affect the joints and induce instability of the block. Rocks with planes or joints dipping downstream as well as rocks of poor quality with closely spaced, open joints, are more prone to erosion than rocks with upstream dipping planes.
Robinson, K.M.	Gully Erosion Research	ASAE Meeting Presentation Columbus, Ohio	1990	This paper examines gully movement in earth emergency spillways. Both fixed-bed and moveable-bed model results are presented. Boundary stress and pressure measurements, as well as gully migration rates, are presented and discussed.
Robinson, K.M.	Gully Erosion in Earth Spillways	Applied Engineering In Agriculture (Vol. 6, No. 3, pp.279-284, 1990) American Society of Agricultural Engineers	1990	The formation and movement of gullies in earth emergency spillways often produce the major damage and pose the greatest threat of a dam breach during a flood flow. This study examined the rate of gully overfall movement through a compacted cohesive soil placed in the center of a spillway channel. The gully movement rates and modes of failure resulting from a constant flow rate were observed as the gully moved up the escape slope and through the level of crest section.
Ru, S.X. Liu, Y.H.	Pressure fluctuations beneath an air-core vortex and their effect on cavitation	Journal of Hydraulic Research, Vol. 30, 1992, No. 2	1992	The results of model experiments are described on the effect of an air-core vortex on pressure fluctuations within the tunnel of a hydroelectric river diversion installation. Data from sets of pressure transducers within the tunnel have been analyzed and it is shown that the presence of an air-core vortex leads to significant increases in (1) the dominant frequency of the pressure fluctuations and (2) the strength of the pressure fluctuations within the tunnel. The implications of the results for the cavitation characteristics of the system are discussed.

Ruff, J.F., Abt, S.R., Mendoza C., Shaikh A., Kloberdanz A.	Scour at culvert outlets in mixed bed materials	Federal Highway Administration	1982	The study of localized scour at culvert outlets has been on-going to control and manage erosion along highway embankments. Herein is presented an investigation of scour at culvert outlets which refines and extends the state-of-the-art of predicting the dimensions of scour holes. Over 100 experiments ranging from 20 to 1000 minutes in duration were conducted in cohesive and non-cohesive bed materials. Culverts having 4-in (10.2 cm), 10-in (25.4 cm), 14-in (35.6 cm) and 18-in (45.7 cm) diameters were tested with discharges from .11 cfs (.003 cms) to 28.13 cfs (.82 cms). Tailwater elevations were maintained at zero, 0.25D and 0.45D ± 0.05D above the culvert invert where D is the diameter of the culvert. The results yielded a series of empirical relationships expressing the depth, width, length and volume of scour as a function of the culvert diameter and discharge. Parameters including the shear number, equivalent depth, pipe shape, soil gradation and extent of scour were investigated. General observations concerning scour, hole formation, growth and stabilization were reported.
Scimemi, E.	Discussion of Paper 'Model Study of Brown Canyon Debris Barrier' by Bermel and Sanks	Trans. ASCE, vol. 112, p. 1016	1947	Synopsis: The principal hydraulic tests of a 1:50 scale model of a debris barrier and the contiguous channel, with some corroborative prototype data, are included in this paper. Tests of the barrier include those for determining the spillway capacity, the performance of the overflow, the development of a stepped overflow crest, and the operation of the barrier with and without detrital material impounded upstream from the barrier. Tests in the channel cover the problems of scour downstream and deposition upstream from the barrier. Because of meager field data, especially regarding bed load and rates of transportation, the initial tests of deposition upstream from the barrier were based on bed load and rates of transportation as developed in the laboratory. These results were reasonably verified in subsequent tests, based on data obtained from the first major storm after completion of the prototype structure.

Sen, Z. Eissa, E.A.	Rock Quality Charts for Log-normally Distributed Block Sizes	Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol. 29, No. 1, pp 1-12, 1992 Pergamon Press Ltd.	1992	Rock blocks due to different joint types are classified qualitatively into three categories as bars, plates and prisms. In engineering evaluations, their quantitative descriptions based on the field observations leading to reliable design values are of prime importance. Simple conceptual models of rock fragments coupled with scanline measurements provided objective relations between the rock quality designation (RQD), volumetric, areal or linear (along scanline) joint counts (J_V) and block volumes (V). The main purpose of this paper is to derive relevant relations for logarithmically-distributed intact lengths. Due to the complexity of these relations, the results are presented in the form of various charts which it is hoped provide useful tools for any rock engineer. The charts presented are for standard deviations equal to unity; however, charts for any desired value of standard deviation can be prepared from the relevant equation. The implementation of the methodology is performed for actual field data. A significant conclusion is that the negative exponential distribution provides a single volumetric RQD-value (which is different from the directional RQDs) and the log-normal distribution gives almost the same result within practical limits, both for directional and volumetric RQD values.
Shafai-Bajestan, Albertson, M.	Riprap criteria below pipe outlet	Journal of Hydraulic Engineering, Vol 119, N 2.	1993	In this study, a general relationship for incipient motion of the sediment below a jet is first developed. Then, for the particular case of a pipe outlet, an experimental program has been conducted to test the resulting equation and to supply the necessary coefficients. For visual observations of sediments, the half jet technique was used, which made it possible to conduct a large number of tests in a relatively short period of time. The methods of sizing riprap, for a no-scour plunge-pool energy dissipator, based on incipient motion and incipient failure are presented. Other topics investigated and discussed in this paper include effects of gradation and thickness of riprap as well as criteria for a gravel filter. A unified relationship for various riprap gradations is proposed by using D30 for the incipient motion and D90 for the incipient failure as the characteristic particle size.

Shaikh, A.	Scour in uniform and graded gravel at outlet culverts	Colorado State University - Thesis	1980	The purpose of this study was to investigate and develop a means of estimating the extent of scour downstream of culvert outlets. Scour holes produced by different discharges of varying duration through circular culverts onto a horizontal blanket of non-cohesive soil were observed and contoured. Data were obtained for two types of bed material: 1) uniform gravel with d_{50} (median size) of 7.62 mm and a standard deviation (σ) of 1.32; and 2) graded d_{50} gravel with d_{50} of 7.34 mm and σ of 4.78. The model pipe diameter was 10 inches. These data were used to develop generalized expressions describing maximum depth, length, width, and volume of scour as a function of flow duration and discharge intensity ($Q\ g^{-0.5}\ D^{-2.5}$) at the culvert outlet. Empirical equations and charts are presented to estimate localized scour downstream from culvert outlets. Utilizing these estimations, designers can select appropriate alternative means of controlling erosion downstream of culvert outlets.
Shi, Gen-Hua R.E. Goodman	The Key Blocks of Unrolled Joint Traces in Developed Maps of Tunnel Walls	Internations Journal for Numerical and Analytical Methods in Geomechanics, Vol 12, 131-158	1989	This paper presents a general method of key block analysis for cylindrical surfaces with numerous real or statically produced joint tracts. General tunnel curves, analytically represented, are unrolled to yield a `developed view' on which the joint traces are continuous curves. Then, using extended block theory, the maximum key block regions are delimited from the curved polygons of the unrolled joint tract map. The methods discussed here apply to a cylindrical tunnel of any shape in section and in any orientation, including inclined tunnels and shafts. The trace maps for which the method applies can be generated statically, as described herein, or surveyed from real traces on exposed tunnel walls. A brief introduction to the basic theory about tunnel key blocks is provide here so that this paper can be understood without reference to other papers on block theory.

Skinner, E.H.	A Ground Support Prediction Concept: The Rock Structure Rating (RSR) Model	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia, pp. 35-51	1988	The Rock Structure Rating (RSR) for prediction of ground support requirements was developed under contract research by the Bureau of Mines. The RSR value index is on a numeric scale of 0 to 100 and is the sum of weighted numerical values determined by considering three basic parameters. Parameter A combines the generic rock type with an index value for rock strength along with the general appraisal of the geologic structure. Parameter B relates the joint pattern with respect to the direction of drive. Parameter C considers the overall rock quality determined by the sum of Parameters A and B along with the degree of joint weathering and alteration, and the anticipated geologic and construction factors. The RSR correlation with support further utilizes the "rib ratio" were determined for steel ribs, shotcrete, and rock bolts. The RSR method of support analysis provides ground support requirements prior to excavation and effective control of ground support practices for project management and operating personnel.
Spurr, K.J.W.	Energy Approach to estimating scour downstream of a large dam	Water Power & Dam Construction	1985	A rational approach is presented for estimating the equilibrium scour downstream of large dams taking into account the mean surplus jet energy in relation to the geology and estimated spill durations. The features most relevant to the hydraulic bedrock interaction are identified and a system for mapping plunge pools geologically is proposed.
Spurr, K.J.W.	Energy approach to estimating scour downstream of a large dam.	Water Power and Dam Construction	1985	The author suggests that the phase difference between the turbulent hydraulic pressure fluctuations, which act at the bedrock surface and those in the bedrock itself develop differential pressures within the fractured rock mass.
Tan, Soon-Keat	Rainfall and soil detachment	Journal of Hydraulic Research, Vol. 27, 1989, No. 5	1989	Literature reviews on (1) process of water drop impact on dry or wet surfaces and (2) splash soil loss due to water drop impact are presented. It appears that water drop impact generates large instantaneous pressure of a magnitude comparable to that of water hammer pressure. However, this pressure only lasted for a very short time and decays within milliseconds. Published studies on soil loss due to water splashing on soil surface shows that splash soil loss is largest when the soil is covered by a thin film of water. This feature is made use of to develop a soil detachment model based on eroding pressure. The various parameters needed in the model are derived from published data. The developed model appears to produce encouraging results.

Thomas, R.	Scour in a gravel bed at the base of a free overfall	Colorado State University - Thesis	1953	Thomas studied a freely falling jet of water and its resulting scour on a uniform gravel bed material. For this study a free overfall was constructed with a sudden drop in elevation from one horizontal bed to a lower horizontal bed. The tailwater depth was varied for different runs of the experiment. The geometric mean was held constant, but the standard deviation for the size was varied. The following conclusions were reached: a) the depth of scour continues to increase with a geometric progression of time; b) an increase in discharge causes a greater increase of depth of scour; and c) A critical depth is reached at which either an increase or decrease in tailwater causes a decrease in the scour depth.
Vittal, N. Al-Garni, Ayed M.	Modified type III stilling basin - new method of design	Journal of Hydraulic Research, Vol. 30, 1992, No. 4	1992	Type III stilling basin, according to U.S.B.R. classifications, is conventionally designed like other basins for a single discharge which is usually the design discharge and its performance at other discharges is tested in a hydraulic model. This paper presents a new method of design for a modified type III basin over a major range of discharges passing the spillway structure. The modified basin has two rows of friction blocks in contrast to one row in the U.S.B.R. basin, to improve velocity distribution of post-jump subcritical flow in the basin. The design is accomplished by developing a dimensionless semi-theoretical relationship for forced jump height curves (FRJHCS) for the spillway-stilling basin combination and matching the dimensionless tailwater rating curve (TWRC) of the river site with one of the FRJHCS, translating the TWRC vertically if necessary. So as to avoid preparation of a large family of FRJHCS for the purpose of matching, an alternative method of matching has been developed based on the assumption that both the FRJHCS and TWRCS may be represented by simpler equations of power law form.

Williamson,, D.A. Kuhn, C.R.	The Unified Rock Classification System	Rock Classification Systems for Engineering Purposes, ASTM STP 984, Louis Kirkaldie, Ed., American Society for Testing and Materials, Philadelphia, pp. 7-16.	1988	The Unified Rock Classification System (URCS) makes possible the initial assessment of geotechnical rock conditions in the field by verifiable and reproducible means. The URCS is flexible in regard to the scale and scope of the project. In addition, the URCS provides for further refinement by as much laboratory testing as is required to meet the cost and risk considerations of the project. The URCS defines by simple or refined procedures four fundamental properties: Weathering, Strength, Discontinuity, and Density of rock materials or rock masses for analysis and documentation. To be effective, a rock classification for civil engineering or engineering geology purposes must meet the criteria of Objectivity, Reliability, Validity, Sensitivity, Comparability, and Utility. The URCS was designed to meet these criteria. The system has been used for over 27 years in a wide variety of geotechnical investigations for the design and construction of civil engineering projects. Results have demonstrated that the URCS does meet these criteria.
Woodward, R.C.	Geological factors in spillway terminal structure design	Proc. 4th Australian-New Zealand Conference on Geomechanics, Perth	1984	Woodward studied 14 spillways of dams in New South Wales, Australia. He suggested that spillways on rock masses with RQD values of lwss than 50 % and containing erodible seams should be provided with complete energy dissipaters. If the RQD is greater than 50% but without erodible seams, a flip bucket was recommended by him. If the rock downstream of the spillway is durable and lowly stressed with RQD greater than 70 % without erodible seams, an unlined or a partially lined terminal structure would be safe enough to withstand the flows. He concluded that unlined spillways which are cut into rock are usually associated with rock mass fracture frequencies of less than 4 fractures per metre. He recommended that if the fracture frequency is more than 4 per metre and the rock is open-jointed with permeability greater than 5 Lugeons, short concrete-lined chutes downstream of such control structures are required.

Young, R.	Surface Erosion of Compacted Cohesive Soil: A Dimensional Analysis.	R-91-05 US Dept. of Interior US Bureau of Reclamation.	1991	Early in the Bureau of Reclamation's development of an EFT (erosion flume test), it became apparent that new data reduction and analysis techniques were needed to apply the test data successfully. Erosion flume test data from other investigators and their data analysis techniques were reviewed, and no consistent methodology was evident. Further review of the erosion phenomenon led to a dimensional analysis of the physical/mechanical variables in the soil-water system. The dimensional analysis resulted in a dimensionless two-constant hyperbolic relationship between mean erosion depth (d), hydraulic radius (R), mean applied shear stress (τ), time (t), and dynamic viscosity (μ). This relationship will be used for data reduction and analysis when the EFT is fully developed and placed in operation.