UNIVERSITY OF MINNESOTA

ST. ANTHONY FALLS HYDRAULIC LABORATORY

Project Report No. 258

U.S.–ROMANIAN WORKSHOP ON WATER RESOURCES ENGINEERING

VOLUME I

Prepared for

NATIONAL SCIENCE FOUNDATION

Washington, D.C.

Grant No. NSF/INT–8506630

May 1987

Minneapolis, Minnesota

U.S./ROMANIAN WORKSHOP
ON WATER RESOURCES ENGINEERING

Bucharest, Romania

July 21 - 27, 1986

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ABSTRACT

This report summarizes the formal presentations made at the U.S./Romanian Workshop on Water Resources Engineering held in Bucharest, Romania from July 21 to July 27, 1986. There are two volumes. Volume I is a summary of presentations made by the American delegation. Volume II is a collection of reprints authored by the American delegates. For brevity, only one copy of Volume II is complete, and that has been sent to Romania. All other copies of Volume II contain a listing of the reprints submitted.

BACKGROUND

In response to a request from the Romanian Embassy, a visit was made to the Hydraulic Engineering Institute of Romania (ICH) in July 1984. The purpose of the visit, which was jointly sponsored by NSF and the Romanian government, was to investigate the possibilities of cooperative research in the broad area of water resources engineering. During this stay, the author of this report visited the hydraulic laboratories in ICH where he had a chance to acquaint himself with the main aspects of the problems currently under study. A visit to the Civil Engineering Institute (ICB) provided insight into the technical education program in Romania. Visits to the various laboratories in each institute and discussions with technical staff were supplemented by field trips. Technical tours were made to the dams and hydropower stations at Paltiner, on the rivers Doftana and Vidraru, on the Arges River, as well as the Sacele and Maneciu Dams.

On the last day, a lecture was given on cavitation research at the St. Anthony Falls Hydraulic Laboratory. An outline of the talk is given in the Appendex. Throughout the visit, extensive discussions were held with Professor Hancu, Director of ICH and to a lesser extent with Professor Iamandi, Rector of the ICB.

Hydraulic engineeirng is a very important discipline within Romania. Presently, there is extensive development of hydropower, navigation, and irrigation schemes. The ICH is active in a broad range of applied research in hydraulics and geomechanics. Some of the ongoing research involves the following types of work:

Dams
River embankments
Irrigation
Waterways and harbors
Coastal protection works
Pumping station
Offshore platforms
Wave energy, etc.

Some of the projects are very extensive; a chain of dams on the Danube, the Bucharest-Danube Canal, and the Danube-Black Sea Canal. The laboratory space at ICH is extensive, covering some 10,500 m^2 for hydraulic research, with additional space for geotechnical and chemical laboratories. There is an additional 10 hectacres of space available for open air models. The applied research program at ICH is complemented by an apparently active fundamental research program at ICB. At both institutes, extensive upgrading was evident. New instrumentation was being calibrated, e.g. a 3-color Laser Doppler velocimeter at ICB, and new computer equipment was being installed.

As already mentioned, the development of water resources is a cornerstone in the Romanian economic plan. Three hundred small hydropower projects are under construction, with another 2,000 sites yet to be developed. The involvement of hydraulic engineering research specialists in this development is extensive. In addition, the ICH and ICB are actively involved in water resources research related to projects in several other countries. A large number of foreign students are enrolled at ICB.

On the basis of discussions held during the one week visit, it became evident that there are substantial opportunities for cooperative research. A protocol was written and signed by this author and Professor S. Hancu, which identified the need for a workshop to more fully explore potential areas of cooperative research.* A proposal was submitted to hold a workshop in Romania in July 1986. The purpose of the workshop was to identify areas of research of mutual interest and to determine methods for implementing cooperative research in the identified areas.

THE WORKSHOP

As a result of the 1984 visit, a workshop on water resources engineering was organized and held in Romania in July 1986. The number of participants was 20, consisting of 9 Americans and 11 Romanians. The format of the workshop consisted of overview lectures on each topic, followed by a discussion of the state of the art. Three areas were covered:

1. Cavitation in Hydraulic Structures

   o Cavitation scaling

   o Measurement techniques

   o Mitigation of cavitation via aeration and investigation of any adverse effects of cavitation.

2. Modelling of Sedimentation

   o Theory of modelling, including use of air models

   o Extrapolation to prototype experience

3. Computational Modelling

   o Application of numerical modelling to a broad range of hydraulic engineering problems

A summary of the conference is being prepared which would summarize the state of the art in these areas and would provide suggestions for future cooperation. The summary will be published in both languages.

In order to select participants, a committee consisting of Professor John Kennedy, University of Iowa, Professor James Liggett, Cornell, and Roger E. A. Arndt, University of Minnesota, was formed. Twenty seven prospective candidates were contacted to determine interest in this project. Table I is a listing of the American delegation and the title of their presentations. Table II lists the Romanian delegation. Table III lists the program. This is followed by summaries of the talks and an Appendix containing the protocol resulting from the deliberations of this workshop.

*A copy is appended.

TABLE I

Presentations of U.S. Participants

Dr. John F. Kennedy University of Iowa	"The Milestones and Millstones of River-Engineering Research
Dr. A. Jacob Odgaard The University of Iowa	"Streambank Projection by Submerged Vanes"
Dr. Gary Parker University of Minnesota	"River Armoring"
Dr. James Liggett Cornell University	"Computational Hydraulics - The Boundary Element Method"
Dr. N. D. Katopodes University of Michigan	"Analysis and Simulation of Shallow-Water Flow"
Dr. Sam Martin Georgia Institute of Technology	"Characteristics of Hydraulic Machinery for Hydraulic Transient Analysis"
Dr. Allan Acosta California Institute of Technology	"Cavitation Inception"
Dr. Roger E. A. Arndt University of Minnesota	"Cavitation Research in Hydraulic Engineering"
Dr. Henry Falvey Bureau of Reclamation	"Cavitation in Hydraulic Structures"

TABLE II

Romanian Specialists for U.S./Romanian Workshop on Hydraulics

1.	Professor Simion Hancu	Hydraulic Engineering Research Institute
2.	Dr. eng. Jeler Valer	Hydraulic Engineering Research Institute
3.	Dr. eng. Didi Duma	Hydraulic Engineering Research Institute
4.	Dr. eng. Dan Batuca	Hydraulic Engineering Research Institute
5.	Dr. eng. Ioan Jelev	Hydraulic Engineering Research Institute
6.	Dr. eng. Arcadie Spataru	Hydraulic Engineering Research Institute
7.	Prof. Constantin Iamandi	Civil Engineering Research Institute
8.	Prof. Andrei Popovici	Polytechnical Institute of Timisoara
9.	Prof. Ptre Roman	Polytechnical Institute of Bucharest
10.	Eng. Teodor Stoicescu	Hydroenergetic Studies and Design Institute
11.	Eng. Florin Ionescu	Research and Design Institute for Water Resources Engineering

TABLE III

Program

<u>Monday, July 21</u>

9:00 - 11:00	Presentation of ICH activity and visit of laboratories.
11:00 - 13:30	Opening of the Workshop debates. Working session on subject no. 3. Presentation of reports by the American specialists and discussions.
13:30 - 15:30	Lunch break.
16:00 - 18:00	Working session on subject no. 3 (continuation). Presentation of reports by the Romanian specialists and disucssions.

<u>Tuesday, July 22</u>

9:00 - 11:00	Working session on subject no. 2. Presentation of reports by the American specialists and discussions.
11:00 - 11:30	Refreshment break.
11:30 - 13:30	Working session on subject no. 2 (continuation). Presentation of reports by the American specialists and discussions.
13:30 - 15:30	Lunch break.
16:00 - 18:00	Working session on subject no. 2 (continuation). Presentation of reports by the Romanian specialists and disucssions.

<u>Wednesday, July 23</u>

9:00 - 11:00	Working session on subject no. 1. Presentation of reports by the American specialists and discussions.

11:00 - 11:30	Refreshment break.
11:30 - 13:30	Working session on subject no. 1 (continuation). Presentation of reports by the American specialists and discussions.
13:30 - 15:30	Lunch break.
16:00 - 18:00	Working session on subject no. 1 (continuation). Presentation of reports by the Romanian specialists and discussions. Closure of Workshop debates.

<u>Thursday,July 24</u>

7:00	Breakfast - Hotel Bucuresti
8:00	Departure by coach to Constantza
13:00	Arrival at Mamaia, accomodation and lunch at Hotel International
15:00	Visit to the Danube - Black Sea Canal and to seashore resorts.
20:00	Dinner

<u>Friday, July 25</u>

7:00	Breakfast at Hotel International (Mamaia)
8:00	Departure by coach to Tulcea
12:00	Visit of Tulcea
13:00	Lunch
14:00	Departure by boat on the Danube (Sulina arm)
18:30	Arrival at Crisan, Accomodation and dinner at Hotel Lebada

For Prof. Kennedy

8:00 - 12:30	Visit of Constantza
12:30 - 13:30	Lunch
13:43 - 16:55	Departure by train to Bucharest, arrival and accomodation to Hotel Bucuresti
July 26, 8:15	Departure from Otopeni Airport

Saturday, July 26

7:00	Breakfast at Hotel Lebada
8:00	Departure by boat for a visit to Danube Delta (to the Rosu Lake)
14:00	Fish dinner (Caraorman village)
19:00	Accomodation at Hotel Lebada and dinner

Sunday, July 27

7:00	Breakfast at Hotel Lebada
8:00	Departure by boat to Tulcea
13:00	Lunch
14:00	Departure by coach to Bucharest
20:00	Accomodation at Hotel Bucuresti

For Prof. G. Parker

21:35	Departure by train (North Railway Station)

Monday, July 28

Departure from the Otopeni Airport; Prof. Acosta and wife, Prof. Arndt, Prof. Falvey, Prof. Ligget, Prof. Martin, Prof. Odgaard

Tuesday, July 29

Departure from Otopeni Airport; Prof. Katopodes

**Summaries of
Presentations given
by American Delegation**

Lecture Presented at U.S.-Romanian Workshop on Hydraulics
20-27 July 1986
Bucharest

THE MILESTONES AND MILLSTONES OF RIVER-ENGINEERING RESEARCH

by

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Abstract

The sediment and water discharges of the world's principal rivers are discussed, and the reasons for the seemingly disproportionate sediment yields of some watersheds are considered. The principal developments in river mechanics since the publication of the DuBoys formula, in 1895, are recounted, and the major impediments to rapid progress in the field of river engineering are enumerated and discussed.

Introductory Remarks

Sancho Panza, you will recall, was the squire and constant companion of Don Quxiote, the dreamer of impossible dreams in Miguel de Cervantes' international literary classic. Sancho Panza was as rustic and down to earth as Don Quixote was elegant (or so he supposed himself) and quixotic (how else to describe him?). There is one famous incident in which Don Quixote dons a barber's wash basin, declares that it is a golden helmet, and is dubbed the Knight of the Woeful Countenance. Meanwhile, Sancho Panza, always the matter-of-fact philosopher, observes that, by creating men with hair that never stops growing, God has given barbers a wonderful economic gift: constant employment.

It is apparent, at least to me, that God has been equally kind to river engineers. Rivers don't keep growing like hair, but the problems they pose surely do. There are other similarities as well. For example, just as a bald man must spend nearly as much for haircuts as a shaggy one, ephemeral rivers in arid regions pose as many and as difficult problems as perenial ones in humid areas. Similarly, river flow, like hair, is most troublesome when there is either too much or too little of it. Finally, it is a near certainty that there will never be a hair restorer or hair-growth retardant; nor will there ever be fully rigorous, reliable mathematical formulations of river flow.

Both barbers and river engineers will go on hacking forever, much to their monetary delight.

In this age when modern technology is displacing or replacing so many traditional occupations, it is reassuring to know that we river engineers, who pursue one of the world's oldest professions, will continue to have such bright prospects for employment. It is also of interest to consider why. In the process of doing this I will include some of my thoughts on our profession's greatest accomplishments over the past century or so.

Sediment: Where From, Where To, and How Much?

Most scientific disciplines are built around a central underlying theorem or physical phenomenon. Examples are the fundamental law of calculus, Newton's laws of gravity and motion, the central limit theorem in probability, and so on. The fundamental law of sedimentary geology, it seems to me, is that the highlands are moving into the oceans. It is the things that happen along the way that provide us a profession.

In view of the enormous sizes of both the mountains and areas of earth, one would expect that a lot of sediment is in motion at any time, and this is indeed the case. Table 1 (Jansen et al., 1979) summarizes the water and sediment discharges of the major rivers of the world; and Table 2 (Milliman and Meade 1983) lists the 21 rivers of the world carrying the largest sediment discharges. Figure 1 (Milliman and Meade 1983) depicts the average annual sediment yield to the ocean from various watersheds. In examining these data, several things should be borne in mind. First, the sediment data include only the suspended load, because bed-load discharge is seldom measured. Second, only about one-half of the sediment delivered to the main stems of most large river systems reaches the oceans. The balance, comprising principally bed-load material, is deposited in the channels or over-bank areas in the plains between the mountains and the river mouths. This is the process which produced much of the worlds most agriculturally productive and densely populated land. Finally, even "good" sediment-discharge data are subject to considerable uncertainty.

River	Station	Catchment area 10^6 km²	Discharge: Water $m^3 s^{-1}$	Water mm yr.$^{-1}$	Sediment 10^6 ton yr.$^{-1}$	Sediment 10^{-3} mm yr.$^{-1}$	Sediment as ppm of discharge (mg l^{-1})
Amazon	mouth	7.0	100 000	450	900	90	290
Mississippi	mouth	3.9	18 000	150	300	55	530
Congo	mouth	3.7	44 000	370	70	15	50
La Plata/Parana	mouth	3.0	19 000	200	90	20	150
Ob	mouth	3.0	12 000	130	16	4	40
Nile	delta	2.9	3 000	30	80	15	630
Yenissei	mouth	2.6	17 000	210	11	3	20
Lena	mouth	2.4	16 000	210	12	4	25
Amur	mouth	2.1	11 000	160	52	15	150
Yangtse Kiang	mouth	1.8	22 000	390	500	200	1 400
Wolga	mouth	1.5	8 400	180	25	10	100
Missouri	mouth	1.4	2 000	50	200	100	3 200
Zambesi	mouth	1.3	16 000	390	100	50	200
St Lawrence	mouth	1.3	14 000	340	3	2	7
Niger	mouth	1.1	5 700	160	40	25	220
Murray-Darling	mouth	1.1	400	10	30	20	2 500
Ganges	delta	1.0	14 000	440	1 500	1 000	3 600
Indus	mouth	0.96	6 400	210	400	300	2 000
Orinoco	mouth	0.95	25 000	830	90	65	110
Orange River	mouth	0.83	2 900	110	150	130	1 600
Danube	mouth	0.82	6 400	250	67	60	330
Mekong	mouth	0.80	15 000	590	80	70	170
Hwang Ho	mouth	0.77	4 000	160	1 900	1 750	15 000
Brahmaputra	Bahadurabad	0.64	19 000	940	730	800	1 200
Dnjepr	mouth	0.46	1 600	110	1.2	2	25
Irrawaddi	mouth	0.41	13 000	1 000	300	500	750
Rhine	delta	0.36	2 200	190	0.72	1	10
Magdalena (Colombia)	Calamar	0.28	7 000	790	220	550	1 000
Vistula (Poland)	mouth	0.19	1 000	160	1.5	5	50
Kura (USSR)	mouth	0.18	580	100	37	150	2 000
Chao Phya (Thailand)	mouth	0.16	960	190	11	50	350
Oder (Germany/Poland)	mouth	0.11	530	150	0.13	1	10
Rhone (France)	mouth	0.096	1 700	560	10	75	200
Po (Italy)	mouth	0.070	1 500	670	15	150	300
Tiber (Italy)	mouth	0.016	230	450	6	270	850
Ishikari (Japan)	mouth	0.013	420	1 000	1.8	100	140
Tone (Japan)	Matsudo	0.012	480	1 250	3	180	200
Waipapa (New-Zealand)	Kanakanaia	0.0016	46	900	11	5 000	7 500

Table 1. Water and Sediment Discharges of 38 Major Rivers (Jansen et al., 1979).

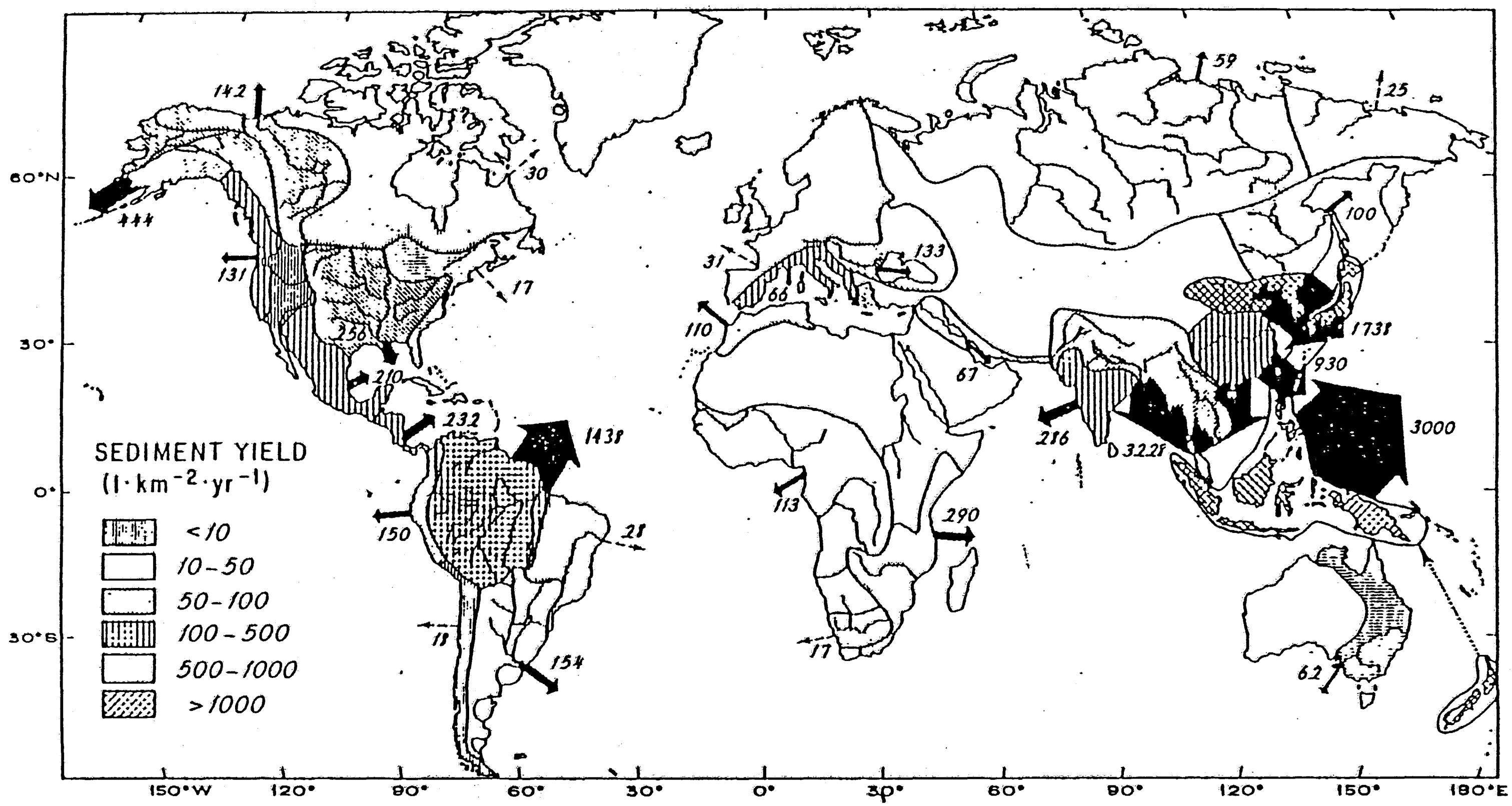

Figure 1. Global View of Erosion Rates and Sediment Discharge to Oceans (Milliman and Meade 1983).

River	Average Sediment Discharge (10^6t/yr)	Adequacy of Data Base
1. Ganges/Brahmaputra	1670	Inadequate
2. Yellow (Huangho)	1080	Good
3. Amazon	900	Inadequate
4. Yangtze	478	Good
5. Irrawaddy	285	Inadequate(?)
6. Magdalena	220	Inadequate
7. Mississippi	210	Good
8. Orinoco	210	Sufficient
9. Hungho (Red)	160	Inadequate
10. Mekong	160	Sufficient
11. Indus	100	Sufficient
12. MacKenzie	100	Poor to fair
13. Godavari	96	Inadequate
14. La Plata	92	Inadequate to Sufficient
15. Haiho	81	Good
16. Purari	80	Inadequate
17. Zhu Jiang (Pearl)	69	Sufficient to good
18. Copper	70	Sufficient
19. Danube	67	Good
20. Choshui	66	Sufficient
21. Yukon	60	Sufficient

Table 2. The 21 Largest Sediment Discharges to Oceans (Milliman and Meade 1983).

Table 2 highlights the fact that, in terms of sediment discharge, the three or four largest rivers of the world, are very big indeed compared to the others. The third largest, the Amazon River, carries nearly twice as much sediment as the fourth largest, the Yangtze, which in turn carries nearly twice as much as the fifth, the Irrawaddy. The same is also true of water discharge, as can be seen in Table 1. The average annual discharge of the Amazon River is more than twice as great as that of the river with the second largest flow, the Congo, which in turn is nearly a factor of two greater than numbers 3 and 4 in this hierarchy, the Orinoco and the Yangtze. (It appears that the production of water and sediment are much like the production of many manufactured goods; dominated by three or four "super companies" in each industry.) The dominance of the world's sediment budget by these four or five "super rivers" is apparent in figure 1. Even more striking than the skewed frequency distribution of the sediment discharges, noted above and apparent in figure 1, is the distribution of sediment concentrations, included in table 1. The Hwang Ho (Yellow River), which has a very modest water discharge, is in a class by itself in terms of sediment concentration. Its average concentration is a factor of two greater than the second river in this category, the Waipapa River of New Zealand, which is merely a stream in terms of water discharge. The annual average concentration of the Hwang Ho is seen to be greater by a factor of nearly four than that of the Ganges/Brahmeptura system, which has the largest sediment discharge of any river basin in the world. Figure 1 also highlights the strong correlation between the sediment yield and level of economic development of a region. A surprisingly large amount of the sediment reaching the world's oceans comes from areas with low per capita annual income. Agricultural development, an essential prerequisite for economic development, assuredly is hindered by excessive erosion.

Table 3 (Milliman and Meade, 1983) summarizes, by geographical region, Earth's sediment delivery to its oceans. Asia and, somewhat surprisingly, the large islands of the Pacific Ocean account for nearly three-fourths of the total soil loss to the oceans. Note in table 2 that eight of the largest sediment discharges are from Asian rivers. The sediment runoff from the large Pacific islands, on the other hand, appears not to be concentrated in a few large rivers. Only one of the 21 rivers included in table 2, the Choshui River (Taiwan), is in this region. It is particularly surprising that it should make the list of the "top 21" when it is recalled that the total length of this river is less than about 100 km, and that it flows only about 4 months of the year. By contrast, the very large rivers of Africa (except the Nile and Congo) and of Siberia are seen in table 1 and figure 1 to produce very modest sediment yields.

Area	Drainage area (10^6 km^2)		Sediment Yield (t km^{-2} yr^{-1})		Sediment Discharge (10^6t yr^{-1})	
	Holeman	This paper	Holeman	This paper	Holeman	This paper
N. & C. America	20.48	17.50	87	84	1780	1462
S. America	19.20	17.90	57	97	1090	1788
Europe	9.2	4.61	32	50	290	230
Eurasian Arctic	...	11.17	...	8	...	84
Asia	26.6	16.88	543	380	14,480	6349
Africa	19.7	15.34	25	35	490	530
Australia	5.1	2.20	41	28	210	62
Large Pacific Islands	...	3.00	...	1000	...	3000
Totals	100	88.60	183	150	18,300	13,505

Table 3. Sediment Budgets Proposed by Holeman and by Milliman and Meade (Milliman and Meade 1983).

The foregoing data and discussion suggest that either or both of two conditions lead to large sediment yields from a watershed. First, the geologic and hydrologic conditions of the area must set the stage for rapid erosion. The watersheds of most of the rivers of the large Pacific islands, for example, are steep, highly erodible, and subjected to intense rainfall during a three or four month period of every year. The Yellow River (Hwang Ho) of China drains the famous loess hills of Western China and Mongolia, which is some of the most erodible land on earth.

The second condition is intense utilization of the land by a dense population. It is probably no coincidence, for example, that China alone provides over one-fourth of the total annual sediment delivery to the world's oceans, and also has nearly one-fourth of the world's total population. This correlation between sediment production and population is also true of the Indian subcontinent. The conjunction of dense population and high sediment yield likely has evolved historically because areas which are flooded regularly by heavily sediment-laden rivers are very fertile, and therefore have historically attracted large numbers of equally fertile people. The large rivers of Siberia and Africa, where drainages are only sparsely populated and produce very little sediment, lend support to this conjecture.

This by no means accidental congruity of muddy rivers and the world's population centers certainly is not bad news for those of us who make our livings as river engineers. Most river-management and flood-control problems, I have observed, are primarily sediment problems. Consider, for example, how after a large flood the excess water soon runs off, but the sediment it deposited remains and generally accounts for a major portion of the flood damage. The river-engineer services are not needed in areas where there is little or no population, or in areas that are not along the thoroughfares traveled by the sediment as it makes it way from the mountains to the oceans. Accordingly, God was indeed beneficient to river engineers when He created man as a prolific species of creature, most of whom till river valleys.

Ten River-Engineering Achievements that Shook the World!

A genre of writing that seems never to go out of style comprises magazine articles, books, scholarly papers, etc., bearing titles like: "Ten Decisions that Changed History", or "Ten Lives that Changed the World", or "Ten Stars who Revolutionized Movies", and other such scholarly titles. For no particular reason that I can discern, except that most of our ancestors had ten fingers and therefore first developed a decimal counting system, the number involved in these titles is always ten, and the events or lives treated are always of epic proportion. It occurred to me recently that our profession has not heretofore had benefit of such an article, and so I undertook to prepare a list of what I consider to be the key milestones in the development of the understanding and mathematical formulation of river flows. The criteria I utilized in developing the list were as follows: First, tradition dictated that it must include ten--no more and no fewer--entries. Second, in order to qualify for the list, the development must have been a significant departure from the then-accepted way of thinking in the subject, and must have been followed by other publications which built on it. Finally, the idea, or others it led to, must have found application in river-engineering practice. We are, after all, supposed to be engineers. Accordingly, developments which do not eventually find use by the practitioners in our subject are not contributions to river engineering, notwithstanding their merit as contributions to physics or fluid mechanics.

The ten "Earth-Shaking (sediment-shaking?) breakthroughs in River Engineering" I chose for my list are as follows.

1. DuBoys (1879) formula. DuBoys' model of bed-load motion envisioned each layer of moving bed sediment as sliding along with parade-like precision over the layer beneath it. (His view of bed-load motion suggests a strong military influence on his scientific training!) DuBoys' formula resulted from a rational analysis of a reasoned (although not realistic) physical model, and may be written

$$q_t \sim \tau_o (\tau_o - \tau_c) \qquad (1)$$

in which q_t = sediment discharge (actually, bed-load discharge) per unit width; τ_o = bed shear stress; and τ_c = critical shear stress for incipient motion. Despite the fact that it was based on an unrealistic physical model, DuBoys' formula attracted considerable attention, has been widely utilized, and has served as the basis for more sophisticated models. This is likely the result of it being algebraically simple, and because it expresses sediment discharge in terms of readily measured quantities, depth and slope.

DuBoys' formula appeared in 1879. Although it had been preceeded by analyses of sediment discharge by Dupuit, in 1865, and Lechales, in 1871, it seems reasonable to accept the DuBoys formula as signifying the dawn of rational, theoretically-based, river engineering. Accordingly, our subject is now roughly one century old.

2. Regime theory (1895). The first contribution to the systematic description of canals that were "trouble-free", in that they produced neither objectionable shoaling or scour, was made by Kennedy, whose formula reads

$$U = 0.84\ d^{0.64} \tag{2}$$

or, in almost nondimensional terms

$$F = 0.15\ d^{0.14}\ \text{(British units)}$$

in which U = "nonsilting, nonscouring velocity"; d = flow depth; and F = Froude number. Further development and refinement of the regime approach continued through the 1960's, and probably beyond. Despite the poor (or nonexistent) theoretical basis for the methdology, it has served practicing river engineers very well while they waited for the theoreticians among us to devise something more reliable and equally usable.

3. Gilbert's (1914) experiments. These classical experiments, conducted at the University of California from 1907 to 1909 under sponsorship of the U.S. Geological Survey served as models for many succeeding laboratory investigations. This contribution is significant not only for the data it produced and the many insights into the mechanics of alluvial streams which resulted from it, but also for setting standards for conduct of sediment-flume experiments that scarcely have been improved upon since.

A careful reading of Gilbert's paper reveals that these experiments were carried out towards the end of his remarkably diverse and productive career, at a time when he was largely incapacitated by illness. The experiments were conducted under the direction of one Edward Charles Murphy, who is also credited by Gilbert with having written much of the report. Once again, the man who did most of the work is largely forgotten.

4. Exner's (1925) equation. This simple expression of continuity of sediment motion,

$$\frac{\partial \eta}{\partial t} + \frac{\partial q_t}{\partial x} = 0 \tag{4}$$

in which η = bed elevation; t = time; and x = distance along the stream, is an essential equation in practically all analyses of bed-form formation, channel evolution, and sediment routing.

5. Shields' (1936) parameter and curve. The Shields' parameter, θ,

$$\theta = \frac{\tau_o}{\frac{\Delta\rho}{\rho}\, gD} \tag{5}$$

in which ρ = fluid density; ρ_s = sediment density; $\Delta\rho$ is $\rho_s - \rho$; g = gravitational constant; and D = particle diameter. This quantity has probably been derived in more diverse ways than any other nondimensional quantity in the whole of hydraulic engineering and fluid mechanics. However, none of them is more convincing than simple dimensional analysis. Shields' curve is still the most widely used criterion for prediction of incipient sediment motion, and the Shields' parameter appears, under various guises, in practically every bed-load transport formula.

After completion of his doctoral thesis on sediment transport, which was submitted to the University at Charlottenburg, Germany, Shields abandoned hydraulics, because he could not find employment in this field. He became a packaging engineer, invented numerous ingenious machines for production of packaging, and held many patents on packaging machinery. His contribution to hydraulics likely would have been lost if Hunter Rouse had not, quite by accident, encountered his work in Germany in the 1930's. Rouse had the thesis translated into English, while he was at Caltech. Rouse and Shields both were in attendance at the 1938 IUTAM Congress in Cambridge, Massachusetts. However, Rouse did not realize this until later, when he found Shields in the group photograph. Rouse included Shields' plot in several of his works, and is responsible for first, drawing a line through the cloud of points which Shields presented; and subsequently dropping the points altogether. Shields died in 1974.

6. The Rouse/Ippen/Vanoni/Karman suspension formula (1937). This, probably the most famous equation in the whole of river engineering, reads

$$\frac{c}{c_a} = \left[\frac{d-y}{y}\,\frac{a}{d-a}\right]^z ; \quad z = \frac{w}{\kappa\beta u_*}$$

c = sediment concentration at elevation y; c_a = sediment concentration at reference elevation y = a; w = particle fall velocity; κ = Karman constant; β = turbulent Schmidt number for sediment diffusion; and u_* = bed shear velocity.

The uncertain parentage of this equation is described by Vanoni (1975) and by Kennedy (1984). This relation, together with rational bases for prediction of velocity profiles, permitted calculation of the suspended-load discharge of rivers. It, or some variant of it, is included in practically every "complete" theory for sediment discharge.

7. Einstein's (1950) bed-load function. The famous U.S. Department of Agriculture Bulletin 1026, and Einstein's papers which preceeded it, introduced statistical concepts to our field. These publications were the first to take formal recognition of the fact that river-bed sediments are not uniform, and that the interactions among the different sizes of particles are of primary importance in calculating the sediment discharge of a river. The 1950

Bulletin also presented the first complete, rational formulation which permitted calculation, by size fraction, of both the bed-load discharge and suspended-load discharge. This classical monograph became one of the underpinnings of river engineering for the next three decades, and continues to be of great importance.

8. The Einstein-Barbarossa (1952) bar-resistance graph. It long had been recognized that the friction factor of alluvial-stream flows is far from constant, but varies widely with both water and sediment discharges. The Einstein-Barbarosa paper finally provided a quantitative predictor for river-channel roughness.

9. Application of stability theory to the analysis of bed forms (Anderson 1953) and meanders (Callender 1969). From the 1880's onward, and especially after about 1930, fluid mechanicians made tremendous strides in the analysis of a wide variety of fluids problems by the application of small-perturbation theory. Examples are the breakup of laminar liquid jets discharged into air; initiation of turbulence; generation of water waves by wind; and wave formation on free-surface laminar flow down inclined planes. Exner (1925) had recognized that sand dunes could be analyzed as a stability problem by utilizing his equation, (4), and a sediment-transport relation. It remained, however, for Anderson (1953) to be the first to apply stability theory to the analysis and prediction of bed forms (ripples, dunes, antidunes, etc.), although his analysis predicted only neutral stability. It was to be another ten years before an analysis was made which incorporated the phase shift between the local bed elevation and local sediment-transport rate, and predicted instability (dune growth). Similarly, Callender (1969) exploited this line of analysis in development of the first rational analytical model of river-channel stability in the formation of meanders. The quarter century since about 1960 has seen the publication of numerous, steadily more sophisticated and comprehensive papers in which stability analysis (small-perturbation theory) is applied to the study of river bed forms and channel pattern (braided, meandering, and straight). The predictions of the more recent papers of this type are remarkably accurate over stunningly wide ranges of variables, from those characteristic of laboratory flumes to values found in large natural rivers. The results of these analyses have yet to find wide application, however, because they generally are embedded in systems of simultaneous equations, eigenfunctions, eigenvalues, and extremum principles which are understandable only to specialists in fluid mechanics who have strong grounding in applied mathematics. Nevertheless, the results of these analyses are too useful and important to continue to be overlooked by practicioners.

10. HEC-6 (Thomas 1977). The response of real-world alluvial streams to imposed disequilibria (such as curtailment of sediment supply, narrowing, flow regulation, etc., etc.) is clearly so complex, and involves so much descriptive input detail, that the problems can only be treated by numerical simulation on modern high-speed computers. Several models of this type have been developed earlier, but Thomas' HEC-6 was an order-of-magnitude more sophisticated and comprehensive in its treatment of river response to man-made changes, and soon gained widespread acceptance. Thomas had the vision to incorporate the contributions of many earlier engineers into a nearly comprehensive computer code for sediment routing, and the intelligence to present the code in a form that, although perhaps is not really user-friendly, is at least not user-surly.

11. Brownlie's (1981) and Karim's (1981) earth-shakers of the 1980's. At the beginning of this section, I promised a list of ten milestone developments. However, that list took us only through the first century of our subject. Moreover, as every good salesman knows, customers are happier if they are given slightly more than promised. This consideration, and the belief that it is not too early to start compiling the second-century list of breakthroughs, prompted me to include an eleventh item in the list. Unhappily, this development is not so much exciting as it is sobering. It is the finding by Brownlie (1981) at Caltech, and by Karim (1981) at Iowa, that suitably formulated, "brute-force" regression analysis can produce predictions of river-flow sediment discharges and friction factors that are statistically more accurate than the sophisticated, "theoretical" analyses developed during the preceeding century. Perhaps judgment of the accuracy of river-flow relations on statistical bases is not valid. Indeed, one can make a supportable argument that the most accurate prediction of the sediment discharge of rivers is $q_s = 0$; one is then in error by only 100 percent, which isn't bad in this line of work.

In any event, after more than a century it appears that the profession has come full cycle in that regime theory, which traces its roots to the late 18th century, is in reality a primitive form of regression analysis. The early proponents of regime theory surely would have relished the large bodies of laboratory and field data and the modern electronic data-handling capabilities which we take for granted. It is also noteworthy that much of the world's river engineering, particularly that related to design and analysis of sediment-carrying irrigation canals, continues to be done on the basis of regime theory. Perhaps the practicioners of river engineering have known all along what we academically based academics are just learning: to analyze the river, first examine and quantify the interrelation among variables in rivers and canals that have achieved satisfactory equilibrium; i.e., that are "in regime". Sancho Panza knew this. He exclaimed: "Experience, the universal Mother of Sciences".

The foregoing list of ten "World-Shakers" includes, to be sure, several genuine milestones, but most of the major problems remain unsolved. As it relates to continued full employment of river engineers, this observation has a major element of good news in it. Consider, by contrast, the lot of specialists in elasticity. Research in their field is virtually dead--most of the basic problems have been solved to an engineering degree of satisfaction. Moreover, most of the constituent phenomena can be described quite accurately by equations. The result is that modern computers permit rapid, accurate solution of even very complicated problems involving plates and shells by problem-oriented computer languages which can be mastered in a few weeks of study.

If it is true that our profession has come a long way during the past century only to return, in a certain sense, to where it began, it is of interest to consider the underlying reasons.

The Six Millstones of River Engineering

The foregoing list of ten (plus one) milestones of river engineering concludes on a not very happy note. Therefore, it appears to be in order to examine the millstones that have made the progress in our field less than dramatic. I believe that the following are the key ones.

1. River hydraulics is a descriptive subject--not an inventive one. Like geology, on which it depends heavily, river hydraulics is concerned with description and analysis of natural phenomena and their utilization to serve man's needs, rather than with invention. No one invents a new type of river or a new bed form (just as no one invents new glaciers or new types of rocks). Accordingly, our field cannot boast, nor could it have expected to, dramatic developments and inventions like the transistor (which resulted from the quantum theory of electrodynamics); radio and television (which was preceeded by the study of electromagnetic waves); lasers (an outgrowth of the study of optics and the theory of light); xerography (an ingenious merging of optics, electrodynamics, and material properties); or the gas turbine and jet engine (a result of the study of gas flow). Perhaps this aspect of our profession is the price we pay for continuing to be fully engaged. No inventions come along to do for us what the high-speed computer did for specialists in elasticity, and the desk calculator did for slide-rule manufacturers. The negative aspect of this consideration is the fact that river engineers hardly ever become rich from their inventions, the notable exception being Shields (who had to leave hydraulics to do it).

2. Turbulence. River flows are turbulent flows, and even the simplest, single-phase turbulent flows cannot yet be analyzed in detail that is satisfactory for most engineering purposes. Ultimately, God's economic gift to all fluids engineers are those wonderful nonlinear terms on the left-hand side of the Navier-Stokes equations, which are responsible for turbulence. Never in the annals of technology have so many scientists and engineers derived so many research projects from such simple-appearing mathematical terms.

3. Free Surfaces. In fluid mechanics, free surfaces are defined as those whose position is not known a priori, but is part of the solution sought. For example, it is the presence of a free surface, and the additional phenomena it makes possible, that makes open-channel hydraulics a far more complex subject than analysis of closed-conduit flow. In the case of rivers, all boundaries are free surfaces, in the sense that their position and form are determined by the flow itself. Rivers are aptly said to be "authors of their own geometry", because they produce the geometry and the roughness of the channels in which they flow. An added complication arises from the fact that, unlike the situation with the upper, water-air free surface, the dynamical and kinematical boundary conditions for the water-sediment interface are not formulated exactly.

4. Randomness/Regularity of River Processes. Many aspects of river behavior, including the structure of the large-scale eddies of river-flow turbulence and the geometry of bed forms, are neither wholly regular nor completely random. Instead, their structure appears to consist of forms with a certain regularity on which random perturbations are imposed. Thus, the phenomena cannot be treated as either completely random or completely regular.

5. Hydrologic Inputs. One of the principal inputs to a river is, of course, the water flow. This, in turn, is determined by overland runoff and must be predicted using the tools of hydrology, which are by no means exact. In addition, hydrologic inputs are almost inherently random.

6. Man's intervention. In their natural states, most rivers have reached equilibrium conditions which enables them to transport, with the available

water flow, the sediment discharges they receive from their watersheds. However, this equilibrium generally takes the form of wide, constantly shifting channels. Man's avarice for the land that rivers formed and now want to occupy, his unquenchable thirst for rivers' water but no corresponding hunger for their sediment, and his constant efforts to derive a host of other benefits from rivers (from electric-power generation to disposal sites for his wastes) motivate him to make major, disruptive interventions in the riverine phase of the hydrologic cycle. These include construction of dams, river channelization, water diversion, and on and on. As a result, natural rivers have become an endangered species in most parts of the world, and our profession is being confronted now with the problems of predicting river responses to large-scale interventions. These problems are proving to be even more formidable than the description and prediction of the behavior of natural streams.

12. Concluding Remarks.

It is a thoughtless speaker or writer indeed who concludes his presentation on a negative note. I fear I may have done this in concluding that, after a century of arduous work, river engineers may have succeeded in doing little more than applying an invention from another field--the modern computer--to a 19th-century methodology: regime theory.

When my spirits are low, I often look for inspiration to my favorite philosopher, Sancho Panza whom I cited at the start of this article. Once, while consoling his ever heroic though seldom victorious leader, Don Quixote, Sancho Panza consoled him with the words: "Fortune may have yet a better success in reserve for you, and they who lose today may win tomorrow". I offer this consolation also to you. Be of good cheer! The great developments in our field are yet to be made, and while we await them we can derive a tremendous quantum of solace from the knowledge that our services, like those of barbers, will continue to be in demand.

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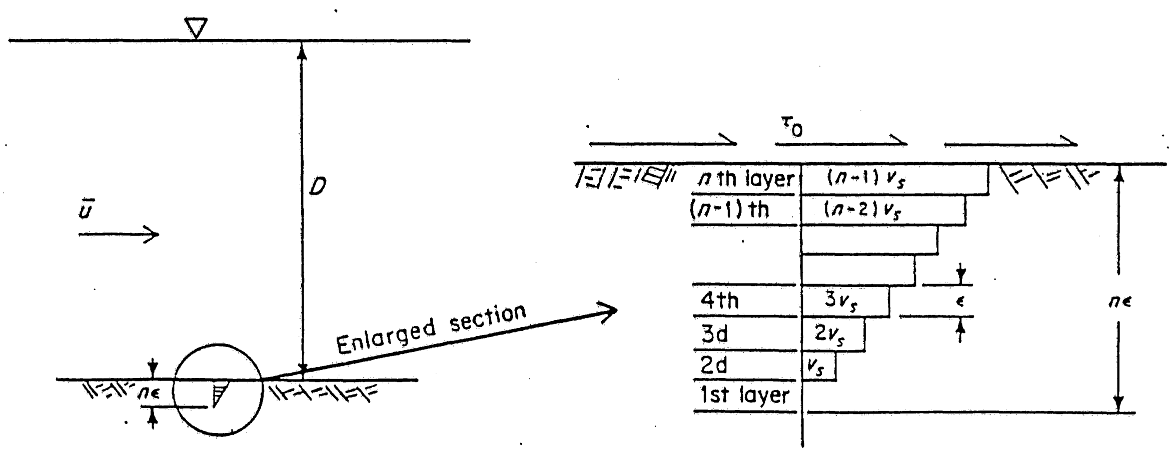
TRANSPARENCIES USED WITH LECTURE

RIVER MILESTONE NUMBER 1

The Duboys Formula (1879)

$$q_t = \left[\frac{\epsilon V_s}{2\tau_c^2}\right]\tau_o(\tau_o - \tau_c)$$



Sketch of DuBoys bed–load model.
(Graf 1971)

RIVER MILESTONE NUMBER 2

Regime Theory (1895; ➡)

"Channels which do not alter appreciably from year to year – though they may vary during the year – are said to be in regime..."
(Sir Claude Inglis, 1949)

"I think any channel you can maintain with the available budget is in regime!"
(Joe Cynic, Irrigation District Engineer, 1986)

Kennedy (1895): "Nonsilting, nonscouring velocity"

$$U = 0.84\ d^{0.64}$$

$$(F = 0.15\ d^{0.15} \cong 0.2)$$

Lacey (1929); Inglis (1949); ....

Blench (1961):

$$U^2 = F_B\, d$$

$$W = \frac{U^3}{F_S}$$

$$S = \frac{F^2}{3.63\ (1 + a_B C)}\ R_W^{1/4}$$

Simons (1957).

RIVER MILESTONE NUMBER 3

Gilbert's Experiments (1907–09; 1914)

DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY
GEORGE OTIS SMITH, DIRECTOR

PROFESSIONAL PAPER 86

THE

TRANSPORTATION OF DÉBRIS BY RUNNING WATER

BY

GROVE KARL GILBERT

BASED ON EXPERIMENTS MADE WITH THE ASSISTANCE OF

EDWARD CHARLES MURPHY

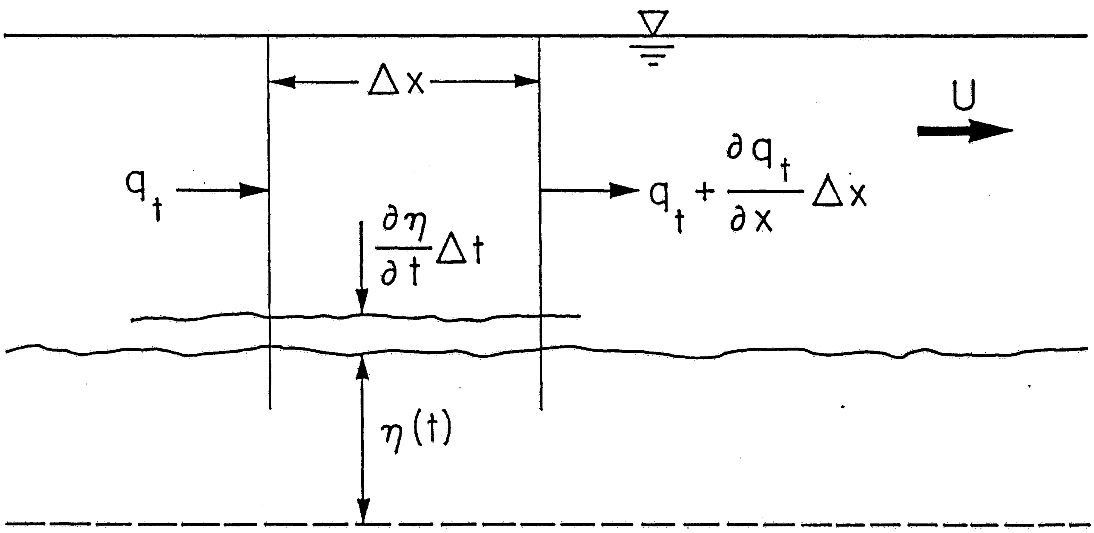
WASHINGTON
GOVERNMENT PRINTING OFFICE
1914

RIVER MILESTONE NUMBER 4

Exner's Equation (1925)

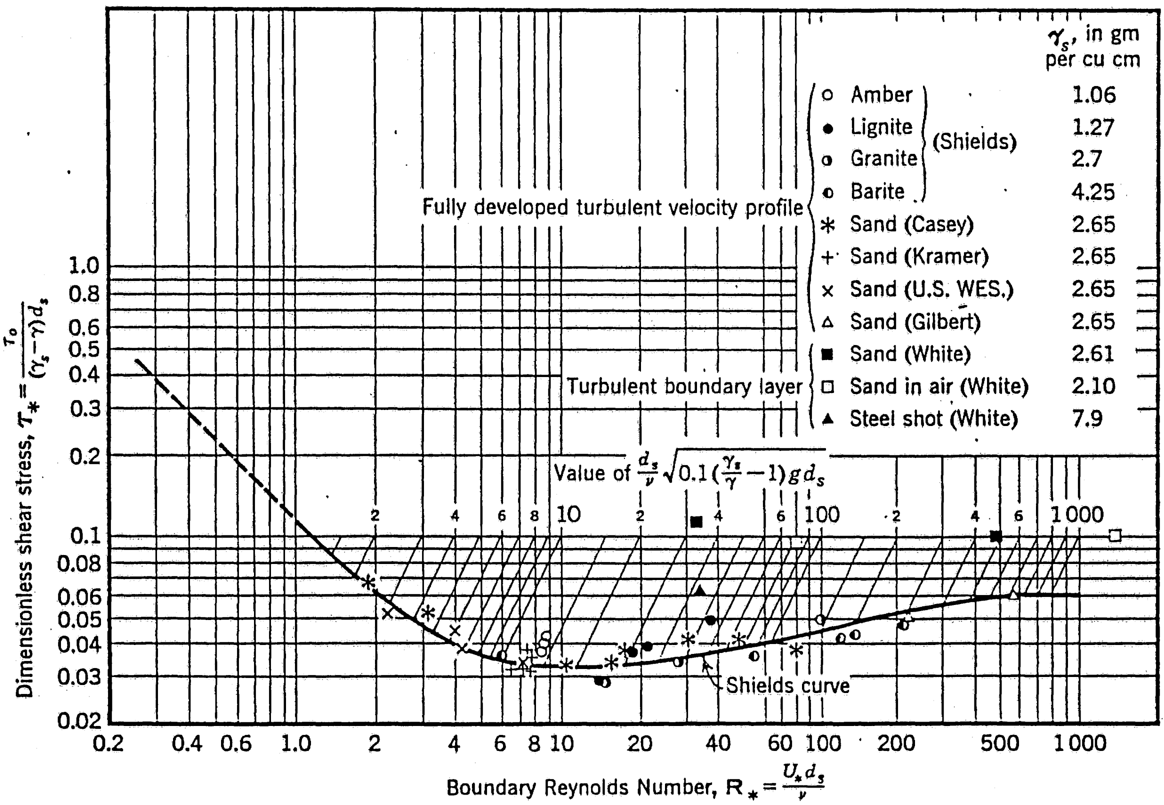
(Note: Special Relativity, 1905; General Relativity, 1915)

$$\frac{\partial \eta}{\partial t} + \frac{\partial q_t}{\partial x} = 0$$



RIVER MILESTONE NUMBER 5

Shields' Parameter and Curve (1936)

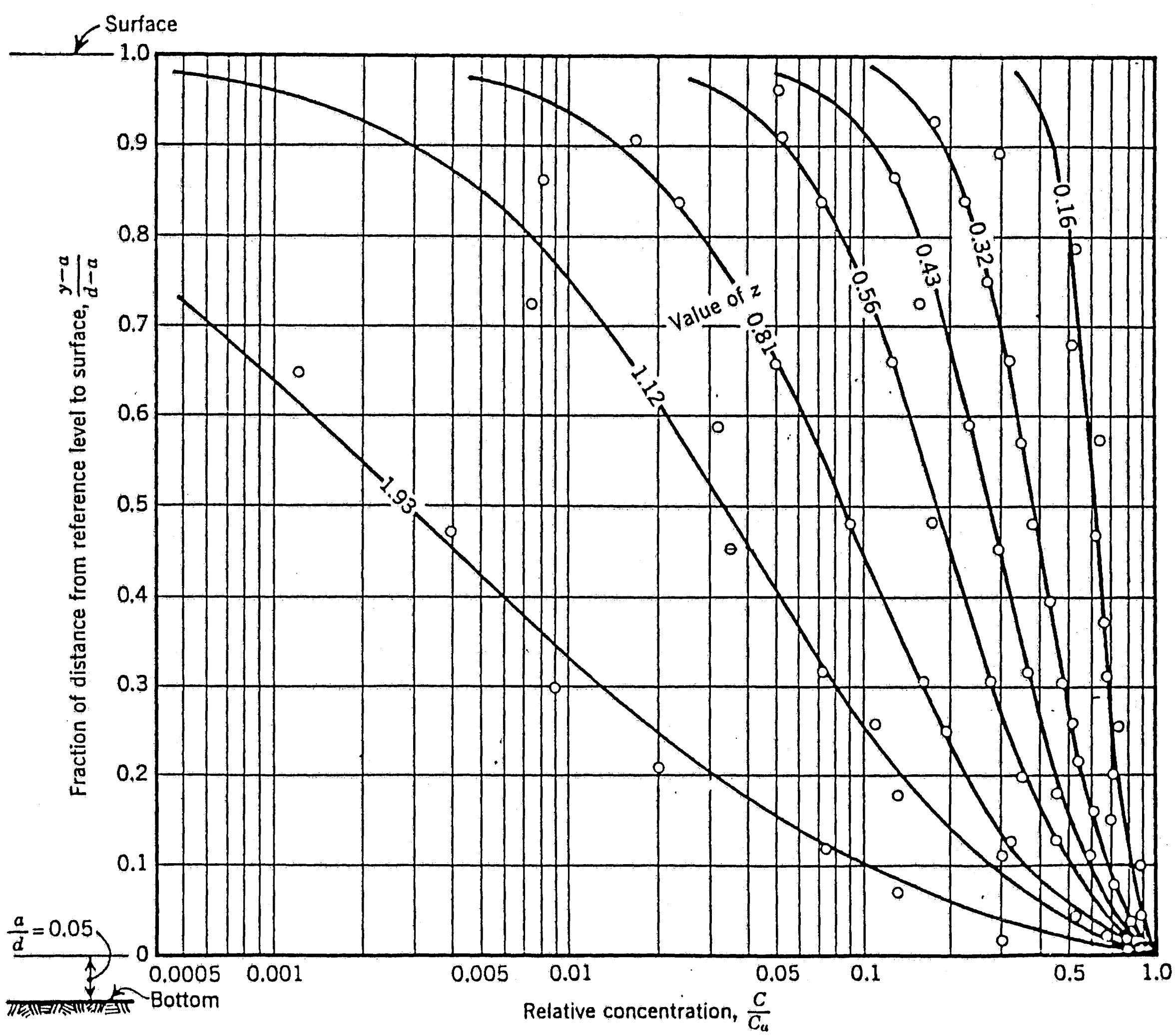


Shields' diagram with White's data added.
(Vanoni 1975)

RIVER MILESTONE NUMBER 6

The Rouse / Vanoni / Ippen Suspension Equation (1937)

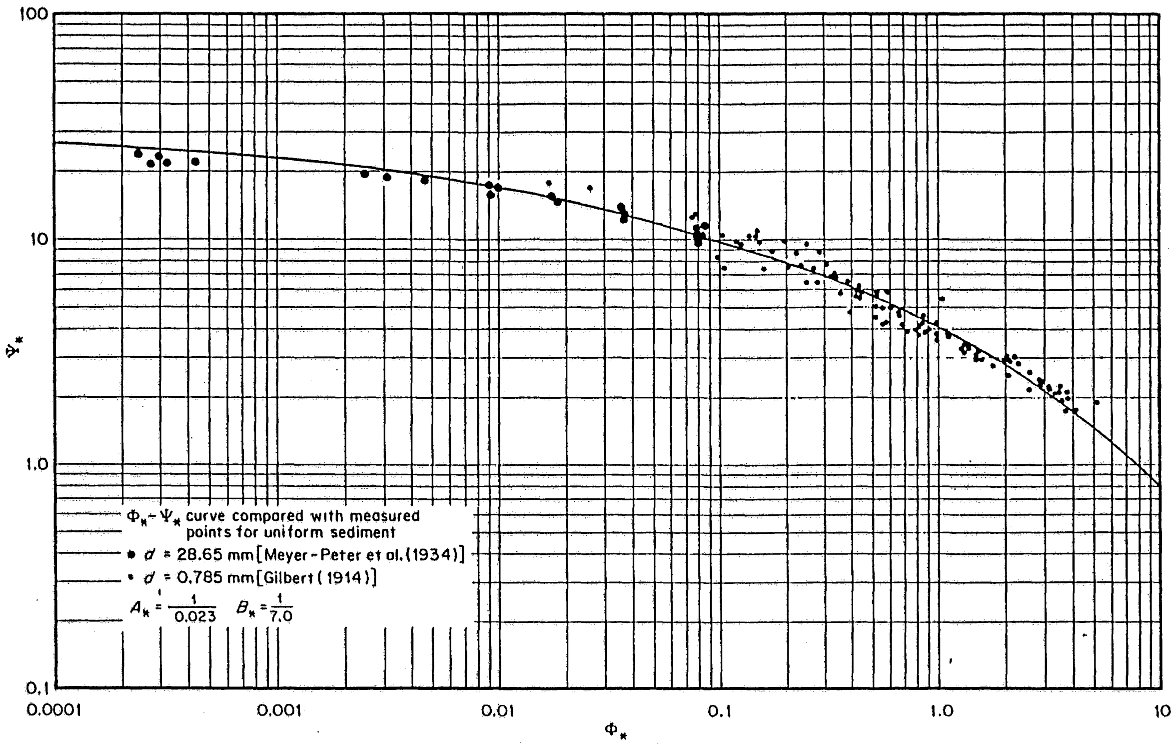
$$\frac{c}{c_a} = \left[\frac{d-y}{y}\,\frac{a}{d-a}\right]^{z} \; ; \quad z = \frac{w}{\kappa \beta u_{\star}}$$



Vertical distribution of suspended sediment. (Vanoni 1975)

RIVER MILESTONE NUMBER 7

Einstein's Bed–Load Function (1950)



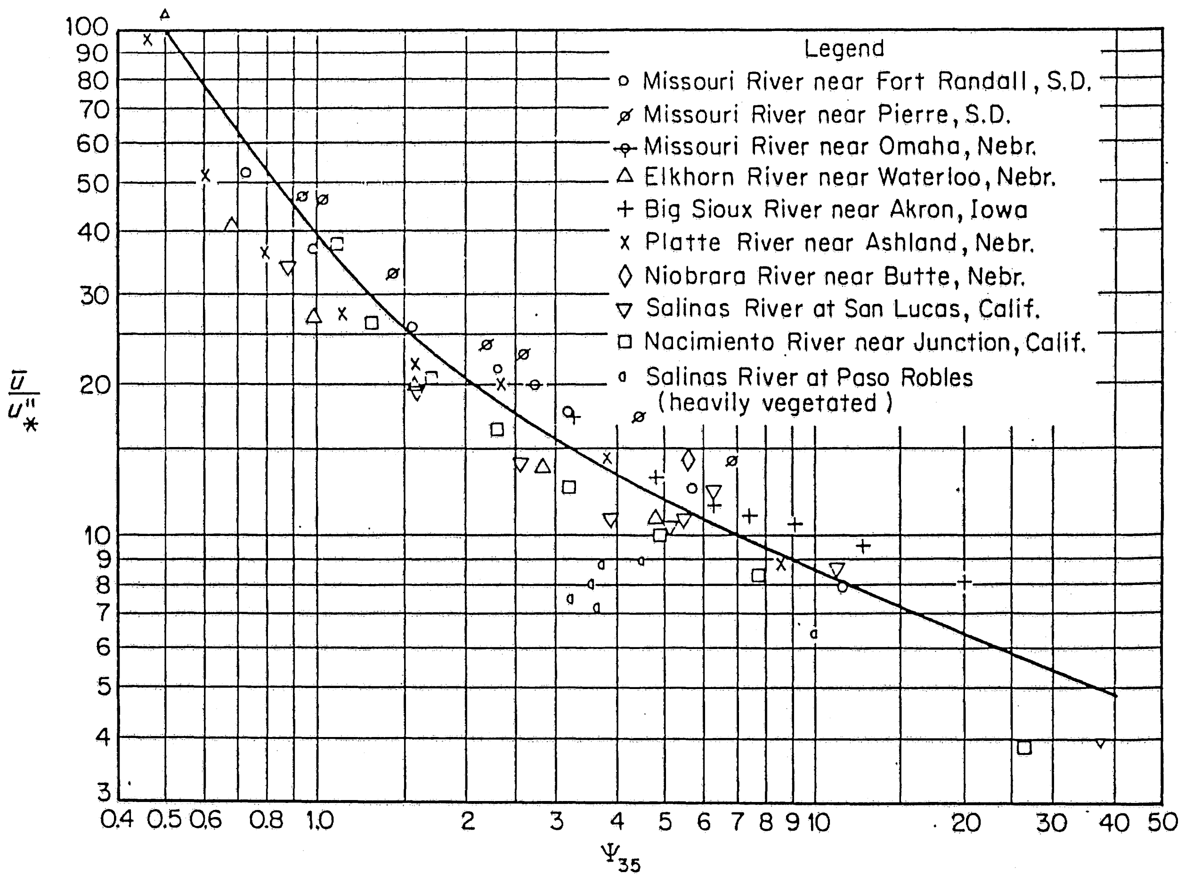
Einstein's bed–load function (Einstein 1950)

$$\Phi_\star \sim \frac{q_t}{g\rho_s}\sqrt{\frac{\rho}{\rho_s-\rho}\,\frac{1}{gD^3}}$$

$$\psi_\star \sim \frac{\rho_s-\rho}{\rho}\,\frac{D}{SR'} \sim (\text{Shields' parameter})^{-1}$$

RIVER MILESTONE NUMBER 8

The Einstein—Barbarossa Bar—Resistance Graph (1952)



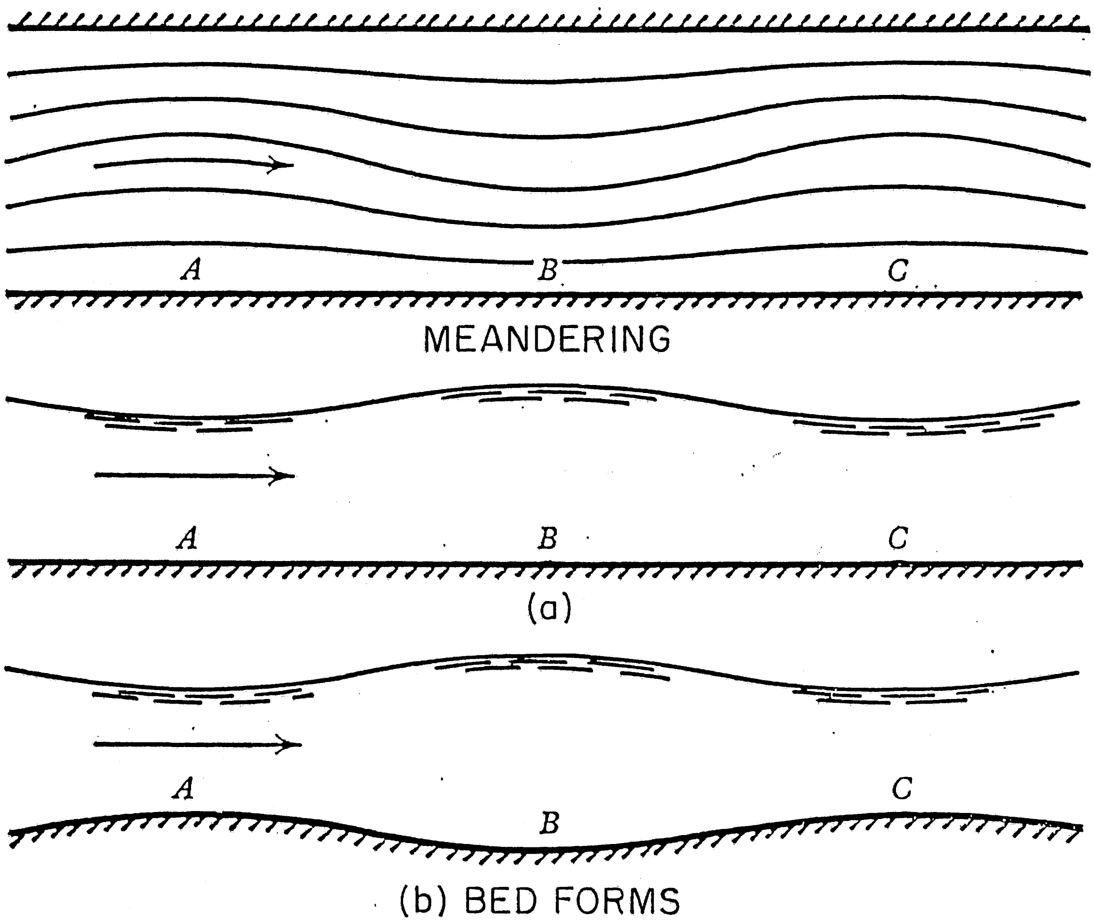
Einstein—Barbarossa bar—resistance graph.
(Graf 1971)

$$u''_{\star} = \sqrt{gSR''}$$

$$\psi_{35} = \frac{\rho_s - \rho}{\rho} \frac{D_{35}}{R'S}$$

RIVER MILESTONE NUMBER 9

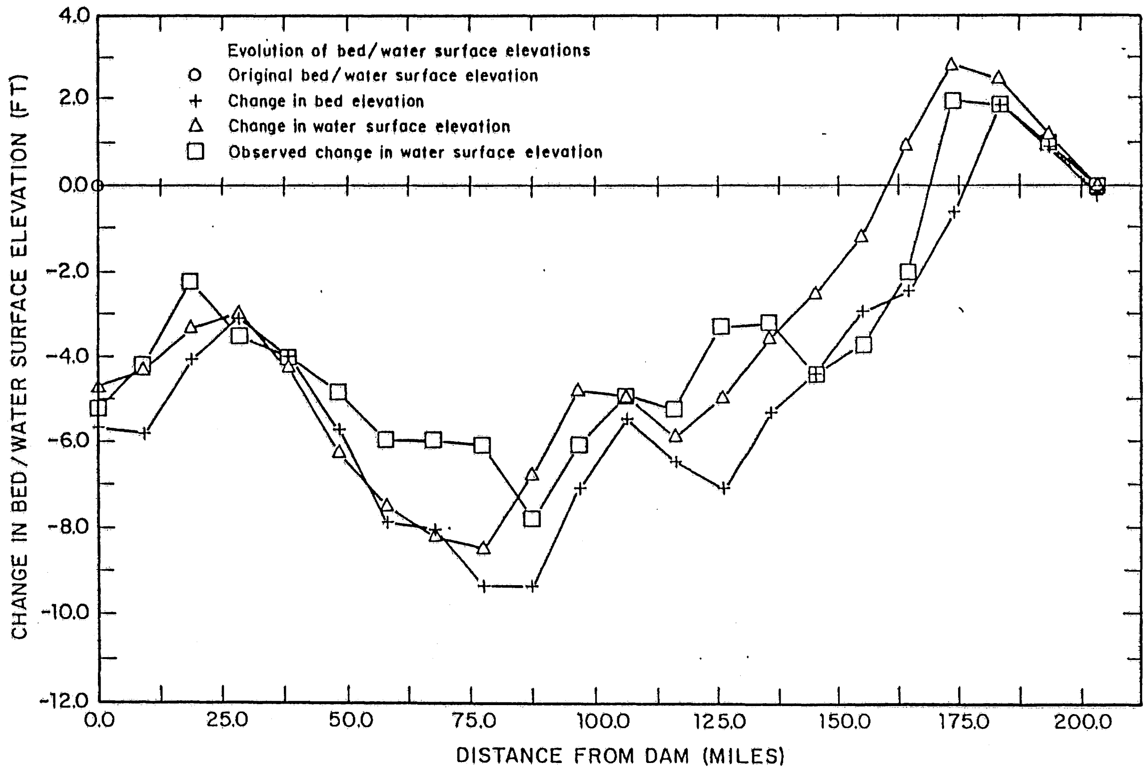
Application of Stability Theory to Bed Forms and Meanders (Anderson 1953; Callender 1969; →)



Stability model for meandering and bed–form growth. (Callender 1969)

RIVER MILESTONE NUMBER 10

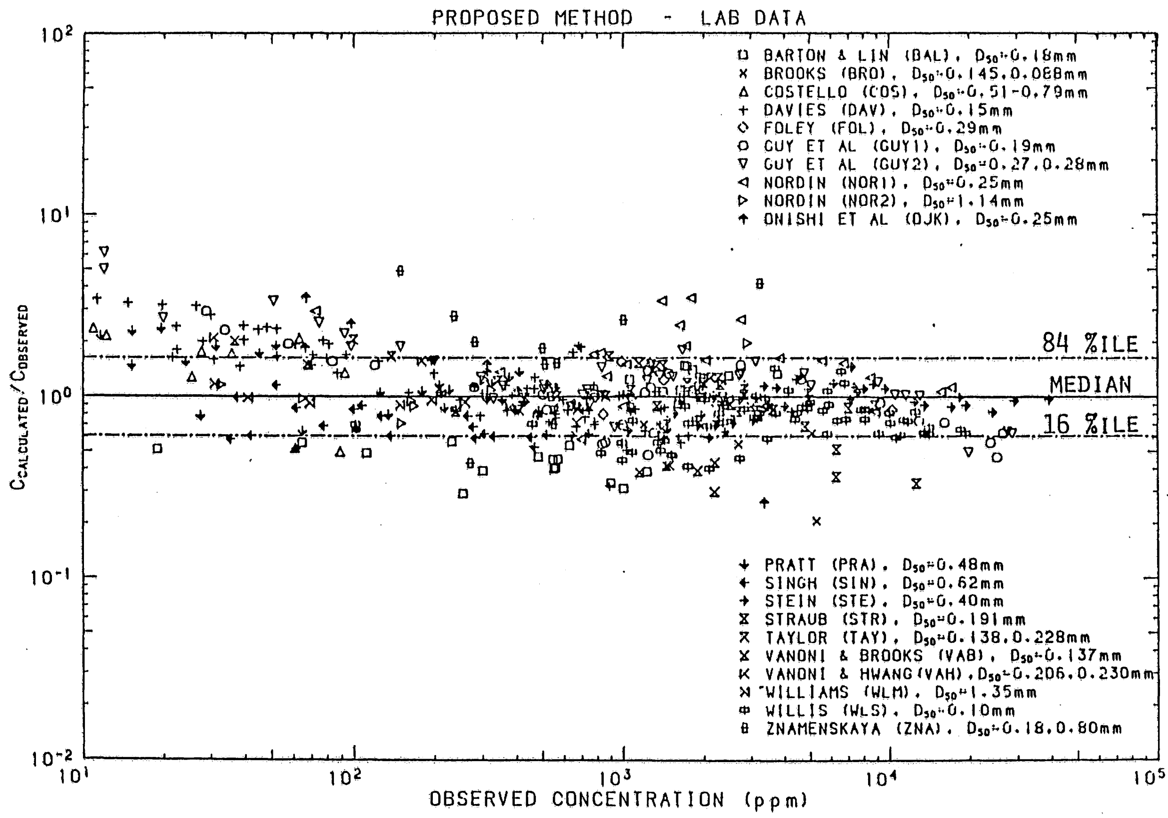
Thomas' HEC-6 (1977)



Computed and observed changes in Missouri River, Sioux City to Omaha 1960–1980 (IIHR 1985)

RIVER MILESTONE NUMBER 11

Brownlie's (1981) and Karim's (1981) Computer–Based Relations for Sediment Discharge and Friction Factors



Observed and calculated concentrations, lab data. (Brownlie 1981)

$$C = 7115\, C_f \,(F_g - F_{go})^{1.978}\, S^{0.6601}\, (R/D_g)^{-0.3301}$$

C_f = 1 and 1.268 for lab and field data, respectively

$$RS/D_{50} = 0.3724\, (q_\star S)^{0.6539}\, S^{0.09188}\, \sigma_g^{0.1050} \quad \text{(lower regime)}$$

$$RS/D = 0.2836\, (q_\star S)^{0.6248}\, S^{0.08750}\, \sigma_g^{0.08013} \quad \text{(upper regime)}$$

THE MILLSTONES OF RIVER ENGINEERS

1. It is a predictive, descriptive subject – not an inventive one.

   Study of electromagnetic waves ⟶ Radio,TV

   Study of light ⟶ Optics, lasers, Xerox

   Study of gas flow ⟶ Jet propulsion

   Study of rivers ⟶ Same old kinds of rivers

2. $u \frac{\partial u}{\partial x}$, etc.; and $\nu \nabla^2 u$, etc. ⟶ TURBULENCE!!
   (the basis for our full–employment law)

3. All river boundaries are "free surfaces"

4. River processes are neither random nor regular, but both (e.g., dune wavelength. Random or regular?)

5. Inputs predicted by hydrology.
   (Hydrology:Hydraulics = Astrology:Astronomy)

6. Man's intervention. "Wild" rivers are an endangered species. "Domesticated" rivers pose ever greater engineering problems.

CONCLUSION

After one century, subject has come full circle.
Now back to "regime theory", but now have computers and more data to base it on.

Statistically "best" equation for q_t??

$$q_t = 0$$

(in error by only 100%)

STREAMBANK PROTECTION BY SUBMERGED VANES

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SUMMARY

Described are efforts to develop a new technique for streambank protection. The technique consists of installing small, submerged vanes on the river bed to stabilize the flow, in particular the flow in river curves, where most of the erosion losses occur. In river curves, the interaction between the vertical gradient of the velocity and the curvature of the flow generates a so-called secondary or spiralling flow which, by moving high-velocity near-surface current outward (and low-velocity near-bed current inward) produces larger depths and velocities near the outer banks. The channel deepening diminishes the toe support of the bank, and the larger velocities attack it, setting the stage for bank erosion. The vanes are installed to counter the spiral flow. By generating a spiral flow in the opposite direction, the vanes eliminate the centrifugally induced spiral. As a result, the flow and sediment move through the channel curve as if the channel were straight. The analytical basis for the functioning of the vanes was given earlier by Odgaard and Kennedy (1982, 1983), and Odgaard and Lee (1984).

Based on the analysis and on results of laboratory tests (Odgaard and Lee 1984), a system was designed for a bend of East Nishnabotna River, Iowa. The layout is shown in Fig. 1. The system was installed during the summer of 1985, and was monitored during the above-average runoff of the spring of 1986. Overall, the system has so far performed satisfactorily. A report on the performance was given by Odgaard and Mosconi (1986). Their report also contains a modified procedure for vane emplacement, developed on the basis of the field experience.

The vanes in the East Nishnabotna installation consist of 3 ft planks held together in a vertical position by piles driven into the streambed (Fig. 1). Such vanes have a relatively low efficiency compared with vanes that are curved or shaped as an air foil. It is well-known from air-foil theory that,

for the same aspect ratio (vane height to vane length), a curved foil in uniform flow has a greater lift/drag ratio (and thus is more efficient) than a straight foil, and that a double-curved foil (airfoil) has an even greater lift/drag ratio (Fig. 2). Consequently, a vane design with a double-curved surface was subsequently adopted. Initially, the surfaces of the vane were vertical. Different shapes were tested for lift and drag and for ability to control the sediment transport in alluvial channels. The improvement over the straight vane was significant. A further increase in efficiency was obtained when the curved vanes were given a twist toward the tail end. The theory behind the twisted vane was given by Odgaard, Spoljaric and Mosconi (1986).

The lift an drag characteristics of the vanes were tested in a 30-ft long, 2-ft wide, rectangular, glass-walled tilting flume. The flume was provided with a lift-and-drag testing module consisting of a series of 2-ft by 4-ft sand coated sheet-metal panels that were inserted in the flume to form a false bottom about 3 inches above the flume bed; see Fig. 3. One of the panels in the module has an 18-inch by 14-inch cutout in which a frame supporting a sand coated plate is suspended such that it can move freely in any direction. The plate surface is coplanar with the surrounding panels, with a 1/16-inch clearance on all four sides. The lift F_L and drag F_D exerted by flow on the vane are transmitted through the suspended unit supporting the vane to two load cells, one for drag and one for lift. This module was used principally to determine the effect on the lift and drag coefficients of: (1) aspect ratio; (2) angle of attack; (3) water depth-vane height ratio; (4) relative bed roughness; (5) Reynolds number; (6) Froude number; and (7) vane profile.

The effectiveness of the twisted, double-curved vane in modifying the bed topography was tested in a 60-ft long, 6-ft wide, straight, rectangular sediment flume. The sediment was sand with a median diameter of 0.4 mm and a geometic standard deviation of 1.4. Both the water and the sediment it transported were recirculated through the flume and return circuits by centrifugal pumps. The effectiveness of the twisted vane was measured by comparing its performance with that of an equivalent straight vane and an equivalent double-curved vane without twist. Two arrays of vanes were installed on a level bed as shown in Fig. 4. The vanes were installed to generate two counter rotating helixes down the center portion of the channel. By deflecting the bed-shear stress toward the sides of the channel, the helixes produced a sideward transport of sediment and left a trench down the center of the channel. The analytical basis for this technique was described by Odgaard and Spoljaric (1986). The performance was measured by the depth that a vane system, emplaced with a given angle of attack, could generate in the trench without increasing the streamwise transport of sediment (that is, without changing the energy slope of the channel). The comparison in Fig. 5 between the average-bed topography produced by twisted vanes and that produced by the same number of curved and straight vanes clearly show that the twisted vanes are more efficient.

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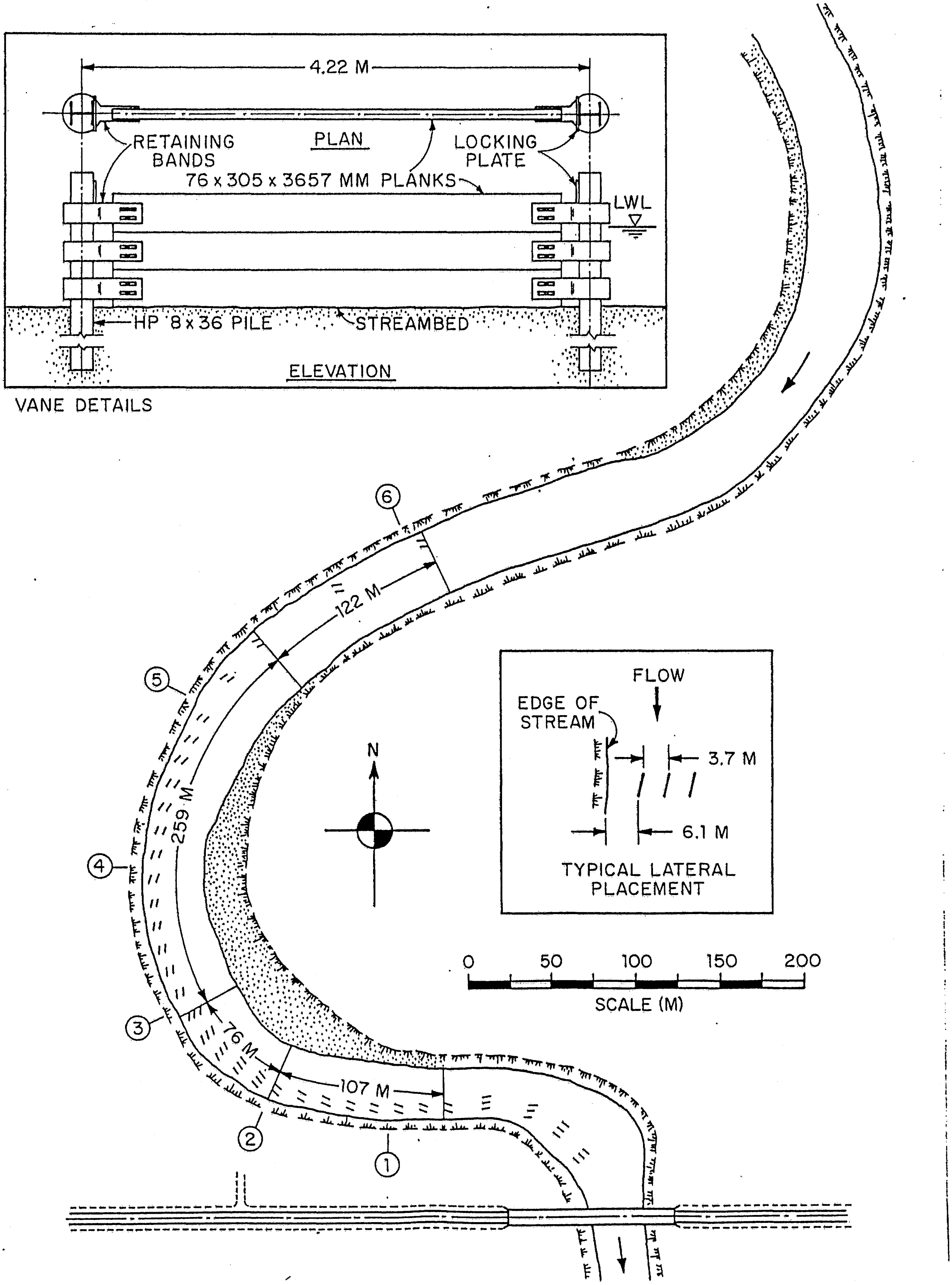

Fig. 1. Layout of vane system, East Nishnabotna River, Iowa.

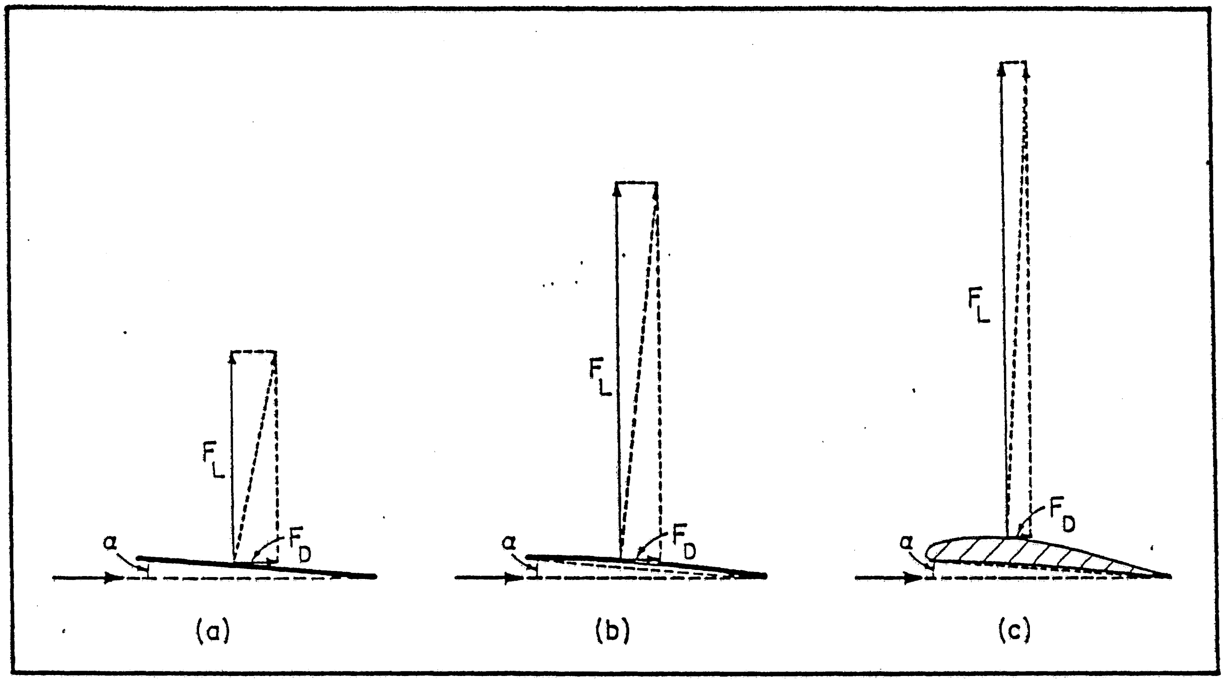

Fig. 2. - Lift F_L and drag F_D on: (a) flat plate; (b) curved plate with height of camber = one twenty-seventh chord; and (c) airfoil. Angle of inclination is α = 4 degrees, and aspect ratio = 6.

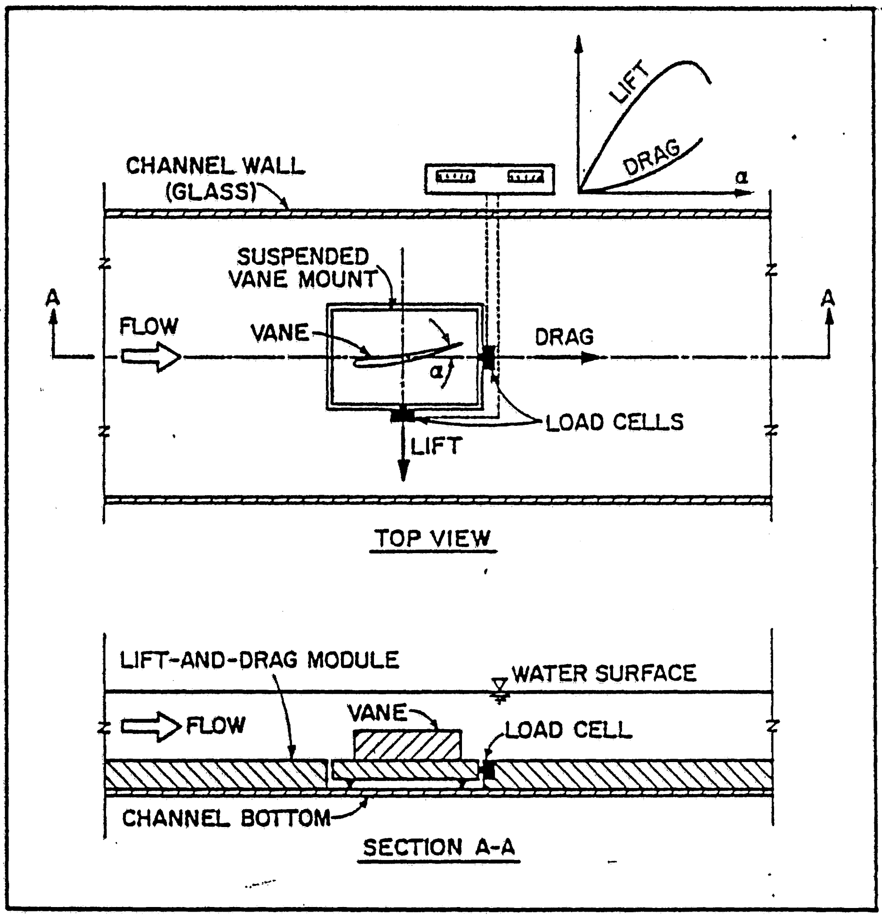

Fig.3. Schematic of lift-and-drag testing module.

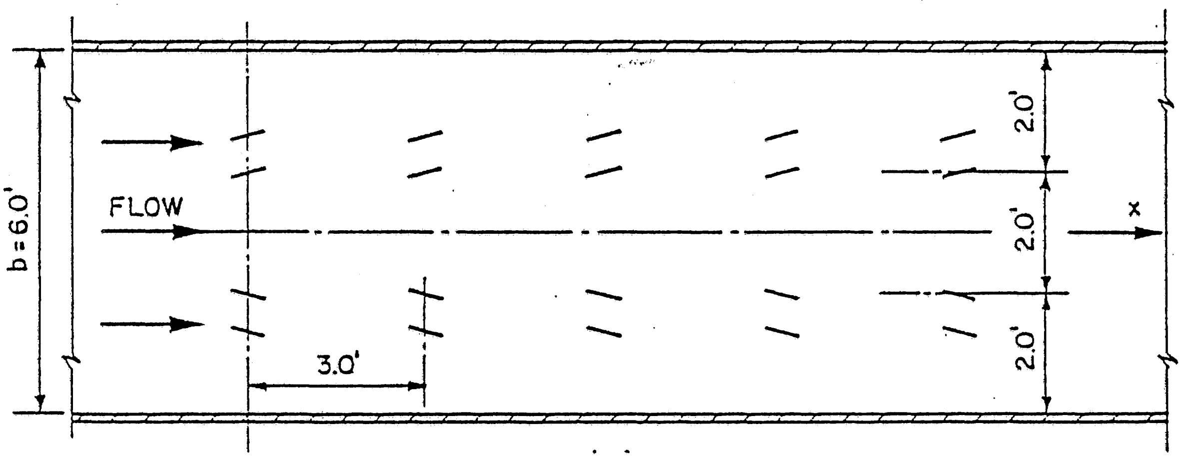

Fig. 4. Vane layout in sediment flume.

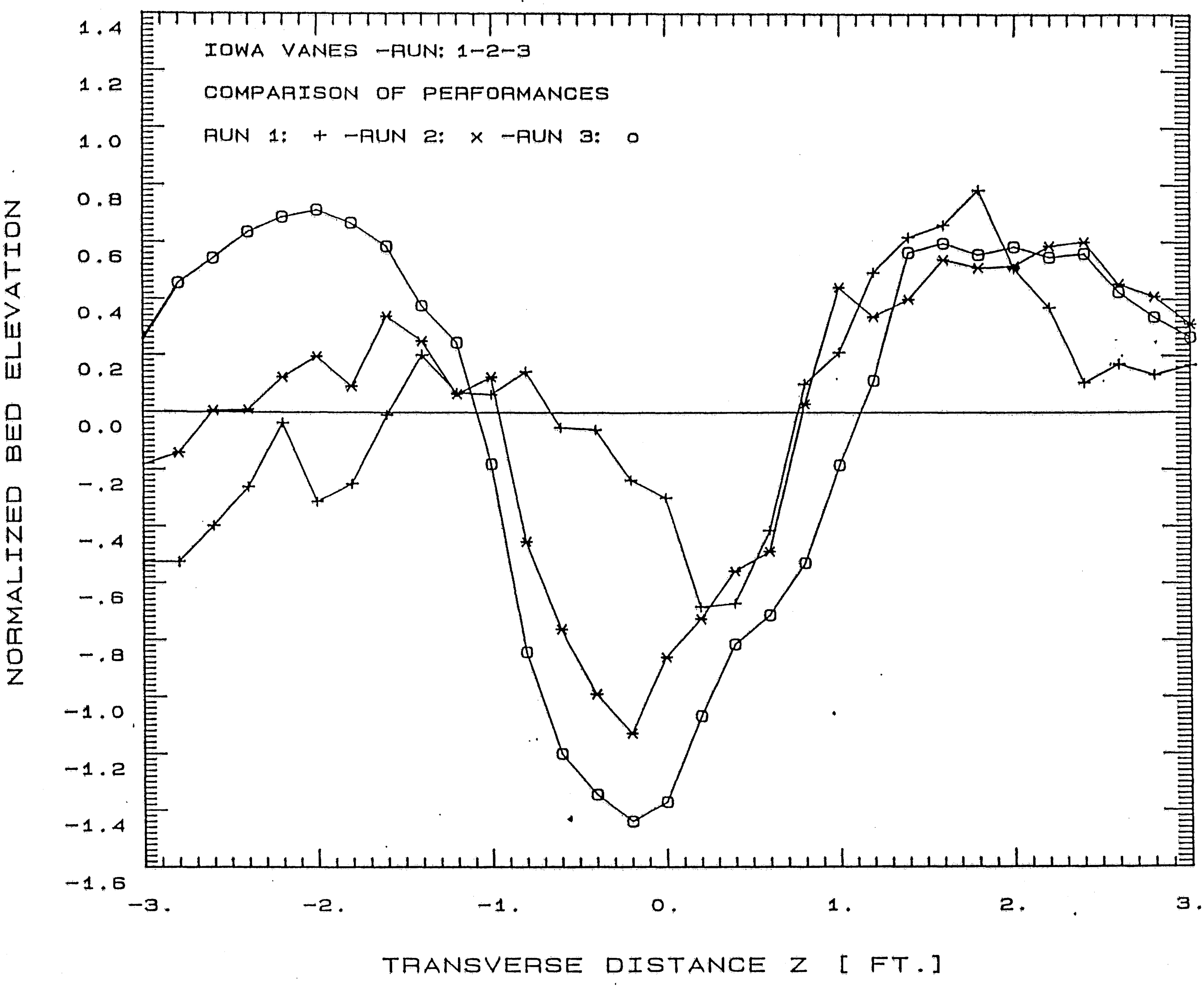

Fig. 5. Performance of flat (Run 1), curved (Run 2), and twisted (Run 3) vanes.

COMPUTATIONAL HYDRAULICS — THE BOUNDARY ELEMENT METHOD

James A. Liggett

1. INTRODUCTION

The boundary element method is another tool in the computation of fluid flow problems. It is especially useful in free surface problems. The uses in porous media and water waves have become fundamental applications. At this date the primary advantage of the boundary element method with respect to fluid mechanics problems is not its raw power in complicated problems, and neither is it the often mentioned efficiency, although that is certainly a point in its favor. The primary advantage is its ability to create good, usable software. The saving in engineering time over finite elements or finite differences is outstanding, even when those methods are equipped with grid generators and other preprocessing conveniences.

At the present the BEM carries some of the disadvantages of both analysis and numerical methods. The analysis disadvantage is that it is limited in the type of problem it can solve. That can be compared to the finite element method, which originally was thought to be limited to those problems that can be expressed in a variational statement. Like the finite element method, it is now obvious that the BEM is much more general than was originally thought, but unlike the finite element method there has not been nearly as much progress in making the BEM general.

The fundamentals of the boundary element method are discussed in the next section. These include the basic equations and methods of discretization for numerical calculation, along with the treatment of singularities, infinite fields and inhomogeneous domains. The porous media applications, one of the rapidly growing areas that employ the method, are explained in the fourth section. The various methods used in that section constitute many of the techniques used in boundary element calculations. The advantage of the reduction of the computational dimension of problems is most apparent in free surface calculations, which are treated in the section on porous media and the following section on water waves. Finally, some of the applications of viscous flow are treated in section six.

2. FUNDAMENTALS

2.1 Integral equations, elements, and nodes

A general explanation of the boundary element method usually begins with the example of Laplace's equation. That is the most logical beginning point for fluid mechanics applications because it is the simplest practical application and because the greatest use of the BEM is still in potential flow problems. The integral equation that is equivalent to Laplace's equation in two dimensions is

$$2\pi\ \Phi(p) = \int_{\partial D} [\Phi(Q)\frac{\partial}{\partial n}\ln r_{pQ} - \ln r_{pQ}\frac{\partial\Phi(Q)}{\partial n}]\ ds \qquad (1)$$

in which ∂D is the boundary of the domain D (figure 1a), n is the unit outward normal and r_{pQ} is the distance between point p (at which the Φ of the left side is evaluated) and the boundary point. Equation (1) states that if the variable Φ and its normal derivative, $\partial\Phi/\partial n$, are both known everywhere on the boundary, then the solution at any interior point consists of a boundary integration. One of these quantities, Φ or $\partial\Phi/\partial n$, or a relationship between them, is given as a part of the boundary condition in any well-posed problem. Moving point p to P, a point on the boundary of the domain D (figure 1b), equation (1) is used to find the "missing" boundary data, the Φ or $\partial\Phi/\partial n$ which is not given by the boundary conditions. In doing so the boundary of the region is discretised into elements, which may be straight lines or polynomials or whatever the analyst desires. The known boundary conditions are approximated in some manner (as constants on each element, linearly, quadratically, or otherwise) along the elements. For Laplace's equation the only approximation is in the boundary condition and the ultimate representation of the solution on the boundary; in the interior the governing equation is solved "exactly" (i.e., the boundary value problem using the approximate boundary conditions is exactly satisfied). The same statement can be made for any linear equation for which an explicit Green's function is known. Expansion of the method to linear and nonlinear problems for which the Green's function cannot be written explicitly requires some approximation of the governing differential equation. That approximation is, however, quite different than is made by finite element or finite different methods.

In many, perhaps the majority, of fluid mechanics problems the boundary solution is all that the analyst needs; the boundary solution consists of finding the fluid interaction with the boundary. If the interior solution is of interest, then equation (1), discretised in the same manner but with the source point at the position of the desired solution, supplies the answer. Another strength of the BEM is that the derivatives of the unknown can be found in the interior of the domain without numerical differentiation and its subsequent loss of accuracy. This property is especially important in potential flow problems where the velocity is often the relevant unknown. Differentiating equation (1) with respect to the coordinate directions gives

$$2\pi\frac{\partial}{\partial x}\ \Phi(p) = \int_{\partial D}\Bigg[\frac{\Phi(Q)\ \sin\mu}{r^2} + \frac{2\Phi(Q)\ \eta\ (\xi\ \cos\mu - \eta\ \sin\mu)}{r^4} \cdots \qquad (2)$$

$$+ \frac{\xi \cos\mu - \eta \sin\mu}{r^2} \frac{\partial}{\partial n} \Phi(Q) \Big] \, ds$$

$$2\pi \frac{\partial}{\partial y} \Phi(p) = \int_{\partial D} \Big[- \frac{\Phi(Q) \cos\mu}{r^2} + \frac{2\Phi(Q)\,\eta\,(\eta \cos\mu + \xi \sin\mu)}{r^4}$$

$$+ \frac{\eta \cos\mu + \xi \sin\mu}{r^2} \frac{\partial}{\partial n} \Phi(Q) \Big] \, ds \tag{3}$$

In obtaining equations (2) and (3) the differentiation was with respect to the position of the base point only; the variables which refer to the target point remain undifferentiated. The latter two equations can be discretised in the same manner as equation (1) and their solution is a direct boundary integration with (now) known values of $\{\Phi\}$ and $\{\partial\Phi/\partial n\}$ along the boundary.

2.2 Singularities

The flexibility of specifying boundary elements provides a system to handle accurately both boundary conditions at infinity in a finite computational domain and singularities. The singularities include the flow around sharp corners (e.g., cutoff walls in porous media calculations), sources and sinks, vortex flow, etc. Special elements can mimic the behavior of singularities. The ability to calculate infinite fields is another important property. Such problems often occur in environmental flows and this feature is especially useful in wave problems and calculations in porous media.

2.3 Zones

In dividing the solution region into zones the problem is treated in parts instead of a whole. The are several reasons for zoning in the boundary element method. The most important occurs in porous media problems where the hydraulic conductivity of the medium varies from place to place. It is easier to solve such problems if the physical properties are constants. In a porous medium it is sufficient to divide the solution region into zones in which the hydraulic conductivity is taken as constant. Usually the properties of the medium are not known in sufficient detail to justify a more detailed treatment.

In other fluid mechanics problems the analyst may wish to divide the problem into zones in order that the separate zones have a better aspect ratio than the problem as a whole. Or different equations may apply to different parts of the problem. Zoning also has the advantage, at least in one method, that the coefficient matrix of the simultaneous equations becomes block banded, and thus one can take advantage of the efficiency of banded solvers. The last attribute comes at the price of a larger system of equations, however. Finally, internal solutions are obtained on the first pass along zonal lines, which may be of limited advantage in some problems.

Consider a solution region such as is shown in figure 2. The internal conditions along the interzonal dividing line are obvious: the potential is the same at any node on that line whether viewed from zone 1 or from zone 2, and the flux across the dividing line calculated from zone 1 must be the same as the flux calculated from zone 2. In equation form

$$\Phi^1 = \Phi^2 \quad \text{and} \quad K_1 \frac{\partial \Phi^1}{\partial n} = K_2 \frac{\partial \Phi^2}{\partial n} \quad \text{on } \Gamma_D \tag{4}$$

in which the superscripts represent the zone numbers. The K_1 and K_2 would represent the hydraulic conductivities in porous media and could simply be unity in a potential flow problem. In the second of equations (4) the normal vector, n, is defined in the same direction for both adjacent zones.

3. POTENTIAL FLOW

The boundary element method is often illustrated with potential flow problems. Indeed, many people have the mistaken impression that it can only be applied to potential problems. The method was first used for potential problems and developed to its highest state in their solution.

One of the most valuable uses of the BEM has been in the solution of potential free surface problems. The problem formulations are easy (there are, of course, some difficult problems) because the problem is one of potential flow. The movement of the free surface constitutes the difficult part. That change in the configuration of the solution region requires, for nonlinear problems, the reformulation of the numerical grid. Such reformulation is much easier using boundary methods than using domain methods. Moreover, the inherent efficiency of a boundary method is often important due to the density of the grid and the small size required for the time step because of the stability and accuracy requirements. Much of this paper is devoted to free surface problems with applications to water waves and flow in porous media. There are a great many free surface problems that do not fall into the category of potential flow. These, too, can often be solved advantageously by the boundary element method.

4. POROUS MEDIA FLOW

A large part of porous media flow falls into the classification of potential flow and boundary methods began to be used to solve such problems for that reason. The technique proved so effective that it has been extended to many porous media problems that are not potential flows.

4.1 Equations of porous media flow

The calculation of flow through a porous matrix usually assumes the dimensions of the particular problem are large compared to the dimensions of the individual grains of the medium. The problem then ignores the microscopic processes and treats the macroscopic process by Darcy's law

$$v = -K\cdot\nabla\Phi \tag{5}$$

in which v is the macroscopic velocity vector, K is the tensor of hydraulic conductivity, and Φ is the head (elevation plus pressure). Applying conservation of mass, $\nabla\cdot v=0$, gives

$$(\nabla\cdot K)\cdot\nabla\Phi + K:\nabla\nabla\Phi = 0 \tag{6}$$

If the flow is isotropic, K can be represented by a scalar and (6) becomes

$$\nabla\cdot(K\nabla\Phi) = 0 \tag{7}$$

If, in addition, the hydraulic conductivity is independent of position, (7) becomes Laplace's equation and the flow is treated by the usual potential flow methods. In total, there are about five separate equations, some nonlinear, that arise in porous media flow that are solved by the BEM. All can be treated by the BEM, at least approximately.

4.2 Free surface flow

In many cases where we need to know local detail about the flow, the Dupuit-Forchheimer approximation is not valid. In the case that the aquifer is unconfined, the free surface must be treated as such. Two boundary conditions are applied to a free surface (see figure 2):

$$\Phi = \eta \tag{8}$$

$$\frac{\partial\eta}{\partial t} = -\left(1 + |\nabla_2\eta|^2\right)^{1/2}(\nabla\cdot n) \tag{9}$$

in which η is the elevation of the free surface above an arbitrary datum, and ∇_2 is the horizontal gradient. (Also, t in (9) is really time divided by the "effective porosity" of the aquifer.) At each time step the numerical grid must be redone in any numerical method. The advantage of the BEM is that the grid is only on the surface and hence there is no reformulation of the interior.

4.3 Axisymmetric flow

In dealing with well problems the geometry of the problem often is axially symmetric. Although such problems can be treated three-dimensionally, it is much more efficient to take advantage of the symmetry. The counterpart of (1) in three dimensions is

$$-\lambda\Phi = \int_{\Omega} \left(\Phi \frac{\partial}{\partial n} \frac{1}{r} - \frac{1}{r} \frac{\partial \Phi}{\partial n} \right) dA \qquad (10)$$

in which $\lambda=4\pi$ for interior points, $\lambda=2\pi$ on smooth boundaries and is the solid angle of the boundary at corners. The Green's function has been specified as $1/r$ for Laplace's equation. Taking advantage of the axial symmetry in the θ-direction, an axisymmetric Green's function can be written which is

$$g(r,z;r_i,z_i) = \int_0^{2\pi} \frac{1}{r} \, d\theta \qquad (11)$$

This function can be expressed in terms of elliptic integrals and they, in turn, can be efficiently evaluated using a rational approximation.

4.4 Leaky aquifers — the modified Helmholtz equation

A common equation for porous media flow is

$$\nabla^2\Phi - \lambda\Phi = 0 \qquad (12)$$

which has the Green's function in two dimensions

$$G = -K_0(\sqrt{s}\, r) \qquad (13)$$

where K_0 is a modified Bessel function of the second kind and of order zero.

4.5 Unsteady flow — the diffusion equation

An equation for unsteady flow in porous media (with the constants absorbed in the variables) is

$$\nabla^2\Phi = \frac{\partial\Phi}{\partial t} \qquad (14)$$

There are at least three separate methods of solution.

4.6 Nonlinear flow — the Dupuit-Forchheimer approximation

In almost all regional groundwater calculations the reduction of the problem to two horizontal dimensions is a excellent approximation. The groundwater aquifers are thin compared to their horizontal dimensions and the vertical flow can be neglected compared to the horizontal flow. This approximation, the Dupuit-Forchheimer approximation, is simply Laplace's equation in the square of the potential and can be solved by standard methods. In the case of a leaky aquifer the equation contains both linear and nonlinear terms. There are two general ways to solve the problem by the BEM.

4.7 Stream aquifer interaction

An application of the above techniques is the problem of the mutual response of a stream and an aquifer which are interconnected (figure 4 in which one-half of the problem is shown and symmetry is assumed for the other half). The aquifer may be "connected" or "unconnected". In the first instance the phreatic surface intersects the stream bed; in the second instance the stream recharges the aquifer through a section that is probably unsaturated. If the phreatic surface connects the stream bed, the flow may be from the stream into the groundwater or it may be out of the aquifer into the stream. In the latter case there appears a seepage surface on the sides of the stream in which the intersection of the free surface and the channel is an unknown point. All of these different conditions may appear in a single episode. The problem is a moving boundary problem, but one with the additional complications of multiple boundary conditions which change from one type to another over the section of boundary near the stream.

The outflow of the channel is governed by the permeability of a "blanket" that lines the channel. The blanket is less permeable than the underlying porous media. The boundary condition is

$$\frac{\partial \Phi}{\partial n} = K_b(p_s - p) \tag{15}$$

where p_s is the pressure in the stream, p is the pressure in the porous media adjacent to the stream, and K_b is a constant dependent on the permeability and thickness of the blanket.

As the free surface contacts or leaves the channel, the surface grid must be regenerated and the boundary conditions reset. Since the free surface positions and normal derivatives of the previous time step are important, the regeneration process must not forget those features. The grid regeneration, while retaining the features and accurate geometrical description, is the challenging part of the computer code. The actual code is somewhat self checking in that the conservation of mass is constantly monitored.

Mass conservation is difficult to achieve during the connection and disconnection processes but can be done with sufficiently small time steps and care in the solution.

4.8 Miscellaneous

There are many other problems in porous media which can be solved by the BEM. Multiphase flows consist of two or more fluids of different densities. The problem becomes a free surface case where the interface between the two fluids is free to move. Applications include sea water intrusion into coastal aquifers and flow in oil-water systems.

Perhaps the primary challenge of the method occurs in calculating problems in groundwater basins in which there are a variety of different conditions including confined and unconfined aquifers, wells, streams, leaky aquitards, recharge, anisotropy, and inhomogeneities. The computer program for such cases is necessarily large and complex. The BEM appears to hold a considerable advantage over domain methods for the reasons of efficiency, and the ease of data input.

5. WATER WAVES

One of the most elegant and advantageous uses of the BEM is in the calculation of waves on a free surface. The problem in its primitive form is governed by Laplace's equation since viscous damping is truly negligible for the vast majority of cases. The problem is challenging in that, since there is no damping, any numerical instability will destroy the solution. Waves have natural instabilities, especially in the three-dimensional form, which from the point of view of numerical calculation can be almost indistinguishable from numerical instabilities. There are several types of equations which govern the linearized version of the wave problem.

5.1 The nonlinear free surface problem.

In the usual manner the velocity vector is defined as the gradient of a potential

$$v = -\nabla\Phi \tag{16}$$

leading to Laplace's equation

$$\nabla^2\Phi = 0 \tag{17}$$

The free surface boundary conditions consist of the kinematic condition

$$\frac{\partial \eta}{\partial t} = \frac{\partial \Phi}{\partial z} - \frac{\partial \Phi}{\partial x}\frac{\partial \eta}{\partial x} - \frac{\partial \Phi}{\partial y}\frac{\partial \eta}{\partial y} \qquad \text{on } z=\eta \qquad (18)$$

where η is the vertical distance from an arbitrary datum (usually the equilibrium free surface) to the free surface, and the coordinate system is aligned with z vertical. The dynamic free surface condition is a form of Bernoulli's equation which states that the pressure is zero

$$\frac{\partial \Phi}{\partial t} = B - g\eta - \frac{1}{2}\left[\left(\frac{\partial \Phi}{\partial x}\right)^2 + \left(\frac{\partial \Phi}{\partial y}\right)^2 + \left(\frac{\partial \Phi}{\partial z}\right)^2 \right] \qquad \text{on } z=\eta \qquad (19)$$

in which B is the Bernoulli constant and the pressure has been set to zero.

As stated in this manner the problem is highly nonlinear. Both the kinematic and dynamic boundary conditions are nonlinear, but the primary difficulty is that these conditions must be applied to the unknown free surface.

5.2 Other wave equations.

The difficulties with the nonlinear free surface problem have caused the derivation of a number of approximate equations for special cases. Linearizing (18) and (19) and applying the boundary conditions to the equilibrium surface gives

$$\frac{\partial \eta}{\partial t} = \frac{\partial \Phi}{\partial z} \qquad \text{on } z=0 \qquad (20)$$

$$\frac{\partial \Phi}{\partial t} = -g\eta \qquad \text{on } z=0 \qquad (21)$$

which can be combined to yield

$$\frac{\partial^2 \Phi}{\partial t^2} = -g\frac{\partial \Phi}{\partial z} \qquad \text{on } z=0 \qquad (22)$$

If ω is the frequency of a wave component, then the harmonic time dependence can be removed

$$\Phi = \phi \exp(-i\omega t) \qquad (23)$$

where i is the unit complex number. The frequency, ω, must satisfy the dispersion equation,

$$\omega^2 = gk \tanh kh \qquad (24)$$

when (23) is used in (24). The wave number is $k=2\pi/L$ where L is the wave length, g is the acceleration of gravity, and h is the equilibrium water depth. The reduced potential, ϕ, satisfies the Laplace equation.

Two important two-dimensional approximations further simplify the problem. In problems in which there is a rigid obstacle (wall, breakwater, etc.) parallel to the y-axis, let

$$\phi(x,y,z) = \phi_v(x,z)\ \exp(ik_y y) \tag{25}$$

Using (25) in the three-dimensional Laplace equation produces a modified Helmholtz equation

$$\frac{\partial^2 \phi_v}{\partial x^2} + \frac{\partial^2 \phi_v}{\partial z^2} - k_y^2\ \phi = 0 \tag{26}$$

If the water depth is everywhere constant, then the incident waves approaching any region may be described as

$$\phi_i = -\frac{igA}{\omega}\ \frac{\cosh\ k(z+h)}{\cosh\ kh}\ \exp(ikx) \tag{27}$$

in which A is a constant related to the amplitude of the wave. If the incoming wave interacts with obstacles that are vertical and penetrate the fluid from the bottom through the free surface (the horizontal plane problem), then a reduced potential can be defined as ϕ_h in the equation

$$\phi = \phi_i - \frac{igA\ \cosh\ k(z+h)}{\omega\ \cosh\ kh}\ \phi_h \tag{28}$$

Using (28) in Laplace's equation produces the Helmholtz equation

$$\frac{\partial^2 \phi_h}{\partial x^2} + \frac{\partial^2 \phi_h}{\partial y^2} + k^2\ \phi = 0 \tag{29}$$

5.3 Nonlinear, two-dimensional waves

Defining β as the angle the free surface makes with the horizontal plane, the derivatives of Φ are

$$\frac{\partial \Phi}{\partial x} = -\frac{\partial \Phi}{\partial n}\sin\ \beta + \frac{\partial \Phi}{\partial s}\cos\ \beta \tag{30}$$

$$\frac{\partial \Phi}{\partial z} = \frac{\partial \Phi}{\partial n}\cos\ \beta + \frac{\partial \Phi}{\partial s}\sin\ \beta \tag{31}$$

where n is normal to the free surface and s is tangent to the free surface. Using (30) and (31) in the kinematic condition (18) gives

$$\frac{\partial \eta}{\partial t} = -\frac{1}{\cos\ \beta}\ \frac{\partial \Phi}{\partial n} \tag{32}$$

The same quantities substituted into the dynamic boundary condition yield

$$\left(\frac{\partial \Phi}{\partial t}\right)_x = B - \frac{1}{2}\left[\left(\frac{\partial \Phi}{\partial s}\right)^2 - \left(\frac{\partial \Phi}{\partial n}\right)^2 - 2\,\frac{\partial \Phi}{\partial s}\,\frac{\partial \Phi}{\partial n}\tan\beta\right] - g\eta \qquad (33)$$

where the subscript on the time derivative indicates that it is to be taken with x held constant but allowing z to vary with the free surface, or

$$\left(\frac{\partial \Phi}{\partial t}\right)_x = \frac{\partial \Phi}{\partial t} + \frac{\partial \Phi}{\partial z}\,\frac{\partial \eta}{\partial t} \qquad (34)$$

Equations (33) and (34) need to be discretised in a finite difference scheme in terms of the unknowns at the advanced time step, Φ^{k+1} and $(\partial\Phi/\partial n)^{k+1}$.

5.4 Stability and accuracy of the free surface problem

The stability of the method probably depends on the exact finite difference discretization, but the analysis of stability must come from a linearized scheme. Such an analysis is easily done and stability diagrams can be presented.

5.5 Far-field boundary conditions

The region of the numerical calculation must be finite even though the wave problem takes place in an infinite ocean. For hydrodynamic problems, perhaps more than other types of problems, the boundary conditions at the boundaries of the numerical region can contaminate the solution to an unacceptable degree. The radiation boundary condition expresses the condition that the boundaries admit incoming waves unchanged and permit outgoing waves to exit the computational region without reflection. In many engineering calculations the placement of the boundaries sufficiently far from the region of interest is accurate enough. That method will not, in general, work for water wave problems because the problem is completely undamped. No matter how far away the computational boundary is, any unwanted reflections will eventually travel back to the region of interest. The problem is further complicated by economics. Wave problems usually require on the order of a dozen nodal points (using linear elements) per wave length to insure sufficient accuracy (more if the waves are particularly steep). Placing the computational boundaries far away can be expensive.

The general problem of a nonlinear radiation boundary condition is unsolved. One approach is to linearize the problem outside of the computational region and use the linearized solution to extend the nonlinear solution. Another approach is to use damping (a "numerical beach") sufficiently far from the zone of interest.

5.6 Steady free surface flow

The steady state problem can be approached from the unsteady problem by the calculation through time under steady boundary conditions until there is no change in the solution. That calculation can be expensive, however, and there should be a more direct method. The direct method, unfortunately, is full of pitfalls. First, the nonlinear problem does not necessarily, and often not actually, have a unique solution; the solution depends on the history. Second, the steady state solution for a given set of boundary conditions does not necessarily exist, at least a solution which is smooth. Either of these conditions would probably be discovered in the course of the unsteady computation but could cause a more subtle difficulty in the direct solution of the steady problem. On the question of uniqueness, the solution would converge to the correct steady state form if the initial conditions are correct. As for the existence, the solution algorithm fails if the surface becomes discontinuous, that is, if the waves break at some point in the evolutionary development. Nevertheless, there are many instances where the direct steady state solution is valuable.

6. VISCOUS FLOW

The BEM has been used to compute flows outside of the boundary layer while coupled to the boundary layer flows. In this section, however, we consider flows at low Reynolds numbers. Defining the vorticity vector as $\omega = \nabla \times v$, the Navier-Stokes equations reduce to the equation which describe the diffusion of vorticity

$$\frac{D\omega}{Dt} = \omega \cdot \nabla v + \nu \nabla^2 \omega \tag{35}$$

in which the capital D represent the substantial derivative, v is the velocity vector, and ν is the kinematic viscosity. The first term on the right represents a gain in vorticity due to vortex stretching (only applicable to three-dimensional flow) and the last term is the vorticity production from viscous action.

The steady state equations of motion are

$$v \cdot \nabla v = -\nabla(p/\rho) + \nu \nabla^2 v \tag{36}$$

$$\nabla \cdot v = 0 \tag{37}$$

$$v \cdot \nabla \omega = \omega \cdot \nabla v + \nu \nabla^2 \omega \tag{38}$$

where the gravity term has been absorbed by the pressure. In the case that the Reynolds number is small, the inertial terms can be neglected. If, in addition we restrict the flow to two dimensions, the set of equations becomes (37) plus the following:

$$\nabla(p/\rho) = \nu\nabla^2 v \tag{39}$$

$$\nabla^2\omega = 0 \tag{40}$$

The common method of solution is to introduce a stream function

$$v_x = \frac{\partial\Psi}{\partial y} \qquad v_y = -\frac{\partial\Psi}{\partial x} \tag{41}$$

which automatically satisfies (37). Taking the curl of (39) produces

$$\nabla^4\Psi = 0 \tag{42}$$

and from the definition of the stream function

$$\nabla^2\Psi = -\omega \tag{43}$$

where ω is now written as a scalar since the vector ω can have only one component, the one out of the plane of the two-dimensional flow.

The free space Green's function for (42) in two dimensions is

$$G = r^2(\ln r - 1)/4 \tag{44}$$

Applying Green's theorem yields the boundary integral equation

$$\alpha\Psi = \int_\Gamma \{\Psi \frac{\partial}{\partial n}(\ln r) - \ln r \frac{\partial\Psi}{\partial n} - \omega \frac{\partial}{\partial n}\left[\frac{r^2}{4}(\ln r - 1)\right] + \frac{r^2}{4}(\ln r - 1)\frac{\partial\omega}{\partial n}\}\, ds \tag{45}$$

in which α is 2π in the interior or the boundary angle on the boundary (π on a smooth boundary). There are four boundary unknowns represented in (45): Ψ, $\partial\Psi/\partial n$, ω, and $\partial\omega/\partial n$. The boundary conditions of a well-posed problem should give two of the unknowns. The system is closed by considering the boundary integral equation equivalent to (40)

$$\alpha\omega = \int_\Gamma \left[\omega \frac{\partial}{\partial n}\ln r - \ln r \frac{\partial\omega}{\partial n}\right] ds \tag{46}$$

These equation will solve a variety of viscous flows in which the inertial terms can be neglected to a good approximation.

The BEM with its variety of elements and ability to handle singularities is particularly suited to this type of calculation.

With the biharmonic equation the accuracy may deteriorate when using finite elements or finite differences due to the necessity of approximating the fourth order derivatives. The boundary integral avoids that difficulty.

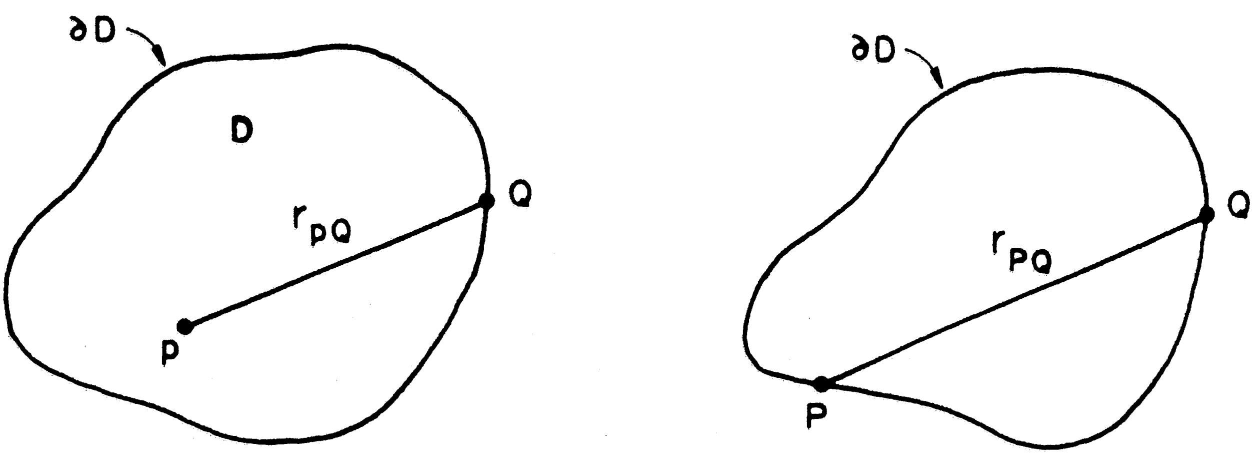

Figure 1. The solution region with (a) the base point in the interior and (b) the base point on the boundary.

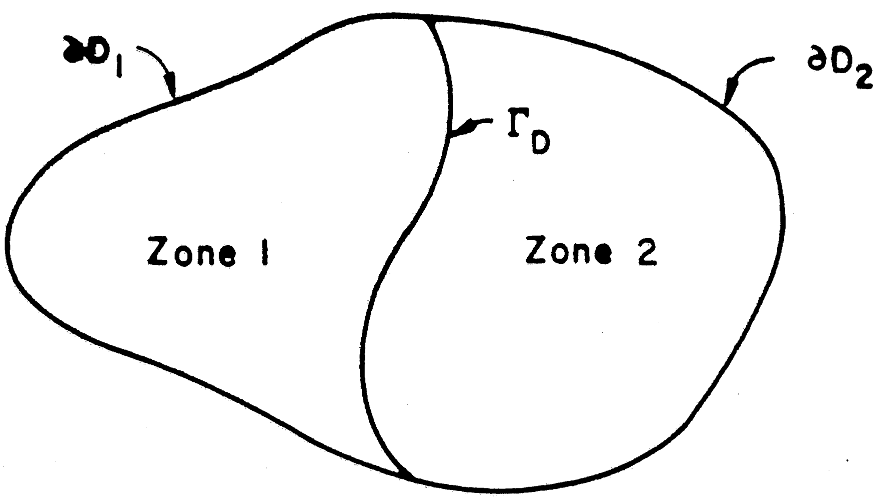

Figure 2. Division of the solution region into segments and elements.

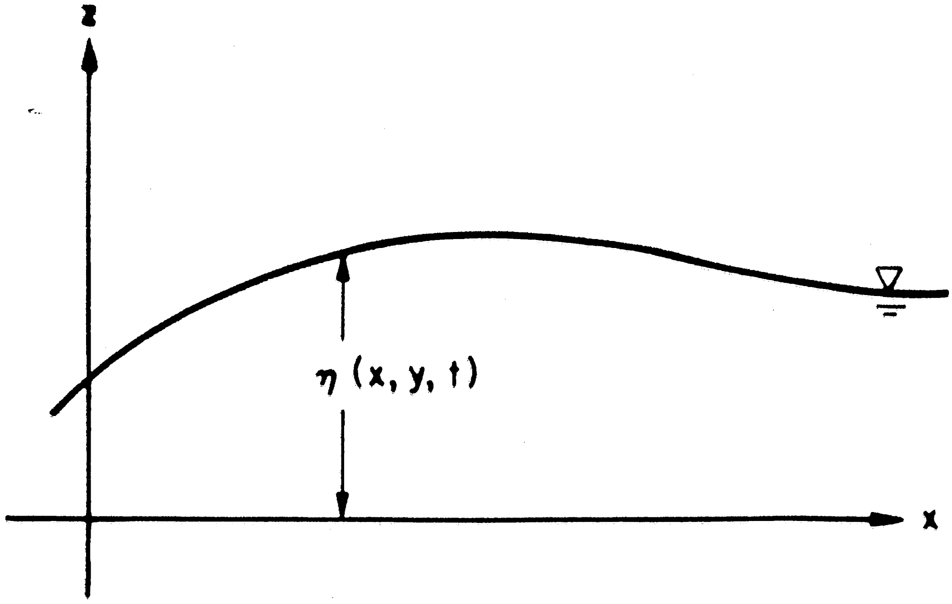

Figure 3. The coordinate system for free surface flow.

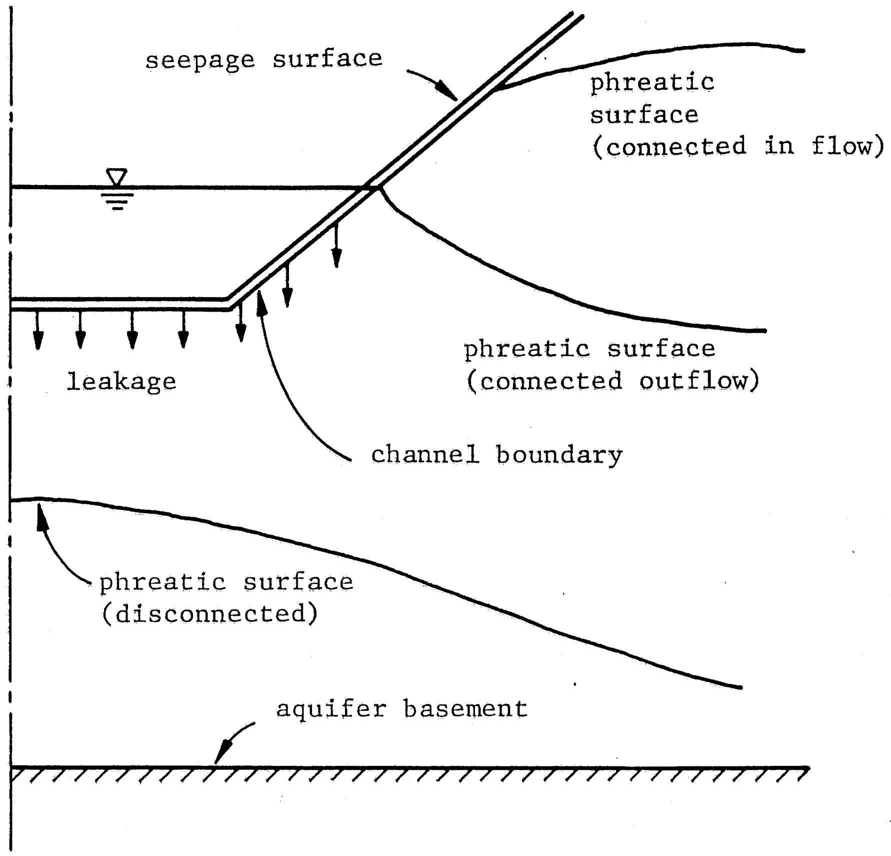

Figure 4. Definition sketch for stream-aquifer interaction showing both the connected and unconnected states.

ANALYSIS AND SIMULATION OF SHALLOW-WATER FLOW

by Nikolaos D. Katopodes[1]

INTRODUCTION

The Shallow–Water theory is derived from the turbulent Navier–Stokes equations for free–surface flow as a simplification suitable for practical applications characterized by large ratios of the horizontal to vertical scales of the problem. The approximation is justified by observations of the behavior of oscillatory gravity waves in shallow water, manifested by a diminishing curvature of the streamlines in the vertical direction. In the limit, the streamline curvature is assumed to vanish completely and the equation of momentum conservation in the vertical is replaced by a statement of pressure variation, which is found to be hydrostatic. The horizontal momentum and continuity equations can then be averaged over the vertical, which, in conjunction with the free–surface boundary conditions, leads to the shallow–water equations. The approximation has been found satisfactory in a wide range of applications and in fact it forms the basis for the so–called hydraulic analysis of free–surface flow.

On the other hand, the theory of shallow–water flow introduces some physical characteristics that are entirely different from those of the complete hydrodynamic theory of free–surface flow. Beyond the obvious approximation of the velocity and pressure variation in the vertical, shallow water flow implies that waves of intermediate and short wave length are filtered and that the celerity of propagation of long waves depends only on the local depth. Hence, the waves described by the shallow–water approximation are strictly non–dispersive. Another physical change occurs in the process of vertical averaging as the notion of compressibility is introduced to the governing equations. In the hydrodynamic theory, the flow is incompressible and the free surface motion is described by kinematic and dynamic conditions in addition to the basic conservation laws. In the shallow–water theory, the pressure is eliminated from the list of dependent variables and is replaced by the depth of flow. As a result of the averaging process, the depth appears in the momentum as well as the continuity equation, thus allowing for fluid to accumulate or be depleted from a given control volume depending on the net volume flux through the corresponding control boundaries. The velocity field is no longer solenoidal and in fact the governing equations resemble more those of compressible gas dynamics than incompressible free–surface flow. In mathematical terms, the shallow–water equations are quasi–linear partial differential equations of strictly the hyperbolic type in contrast to the turbulent Navier–Stokes equations, which are of parabolic type for small values of the Reynolds number and become gradually of mixed

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parabolic–hyperbolic type as the Reynolds number increases. This last statement implies that an additional assumption has been made in the derivation of the shallow–water equations, namely, that all energy dissipation takes place in the form of boundary resistance, i.e., that internal stresses are negligible. This assumption is justified in the majority of practical problems and results in an another physical difference between the hydraulic and hydrodynamic theories. Specifically, in the absence of bed friction, the shallow–water equations are non–dissipative, that is, their solution does not decay with time.

In mathematical terms, the shallow–water equations can be recast in the form of interior differential operators of reduced order of differentiation. These operators are valid along surfaces defined in the three–dimensional space outlined by the physical horizontal dimensions of the problem and time and are called characteristic surfaces. The solution of the governing equations is allowed to suffer discontinuities in its derivatives across these surfaces and therefore, any potential wave fronts travel along the characteristic surfaces. As a result, initial and boundary conditions cannot be specified arbitrarily but in fact certain compatibility conditions must be satisfied between the interior differential operators and the boundary data. This means that for a solution to be consistent with the physics of the problem, the characteristic surfaces should be constructed in detail and the appropriate differential equations valid along them need to be solved in the fashion dictated by the paths of the travelling waves and not on some pre–assumed mesh in space and time.

On the other hand, the nonlinear inertial terms of the equations are known to lead to discontinuities in the solution itself in the form of surges and shocks. In such cases the differential equations become invalid, although the physical situation corresponding to this behavior is quite possible. Shocks, however, are rather short waves that cannot be resolved in terms of the shallow–water theory, except in the form of a discontinuity in the solution. The discontinuity itself represents some form of failure of the theory, especially since it is well–known that appreciable energy is dissipated in a hydraulic bore due to intense turbulent mixing. It is obvious that in such cases the formulation not only should allow for possible discontinuities in the solution but also it should be based on momentum rather than energy conservation.

If the analytical aspects of shallow–water flow seem confusing, the numerical simulation of the corresponding practical applications are further complicated by the imperfection of the computational schemes employed. In reality, we are in search of a numerical method that can trace the characteristic surfaces in time and space, satisfy the compatibility conditions between the differential equations and boundary conditions, allow for the formation and tracking of shocks, conserve volume and momentum across such shocks, be free of all wave dispersion effects and exhibit non–dissipative behavior in the absence of bed friction. In addition, since it is known that the nonlinear terms occasionally lead to the formation of high frequency spurious waves, the solution technique should be selectively dissipative, i.e., capable of dissipating the very short parasitic waves without dissipating any of the longer wave components.

GOVERNING EQUATIONS

Conservation of mass and momentum in two-dimensional shallow-water flow leads to the following system of partial differential equations

$$\frac{\partial h}{\partial t}+\frac{\partial (uh)}{\partial x}+\frac{\partial (vh)}{\partial y}=0 \tag{1}$$

$$\frac{\partial (uh)}{\partial t}+\frac{\partial (u^2 h)}{\partial x}+\frac{\partial (uvh)}{\partial y}+g\frac{\partial (h^2/2)}{\partial x}=gh(S_{0_x}-S_{f_x}) \tag{2}$$

$$\frac{\partial (vh)}{\partial t}+\frac{\partial (v^2 h)}{\partial y}+\frac{\partial (uvh)}{\partial x}+g\frac{\partial (h^2/2)}{\partial y}=gh(S_{0_y}-S_{f_y}) \tag{3}$$

in which h=depth of flow, u, v=x and y components of fluid velocity, S_{0_x}, S_{0_y}=x and y components of bed slope, S_{f_x}, S_{f_y}=hydraulic resistance in the x and y directions, and g=gravitational acceleration.

If Manning's equation for the friction slope is adopted, the corresponding terms can be expressed as

$$S_{f_x}=\frac{n^2 u(u^2+v^2)^{1/2}}{h^{4/3}} \quad , \quad S_{f_y}=\frac{n^2 v(u^2+v^2)^{1/2}}{h^{4/3}}$$

in which n is Manning's resistance coefficient.

Eqs. 1–3 may be written in matrix form as follows

$$\frac{\partial U}{\partial t}+A\frac{\partial U}{\partial x}+B\frac{\partial U}{\partial y}+D=0 \tag{4}$$

in which

$$A=\begin{bmatrix} 0 & 1 & 0 \\ -\frac{p^2}{h^2}+gh & \frac{2p}{h} & 0 \\ -\frac{pq}{h^2} & \frac{q}{h} & \frac{p}{h} \end{bmatrix}$$

$$B=\begin{bmatrix} 0 & 0 & 1 \\ -\frac{pq}{h^2} & \frac{q}{h} & \frac{p}{h} \\ -\frac{q^2}{h^2}+gh & 0 & \frac{2q}{h} \end{bmatrix}$$

$$U=\begin{pmatrix} h \\ p \\ q \end{pmatrix}$$

$$D=\begin{pmatrix} 0 \\ gh\frac{\partial z_0}{\partial x}+gn^2\frac{p(p^2+q^2)^{1/2}}{h^{7/3}} \\ gh\frac{\partial z_0}{\partial y}+gn^2\frac{q(p^2+q^2)^{1/2}}{h^{7/3}} \end{pmatrix}$$

in which the volumetric flow rates per unit width $p=uh$ and $q=vh$, have been introduced.

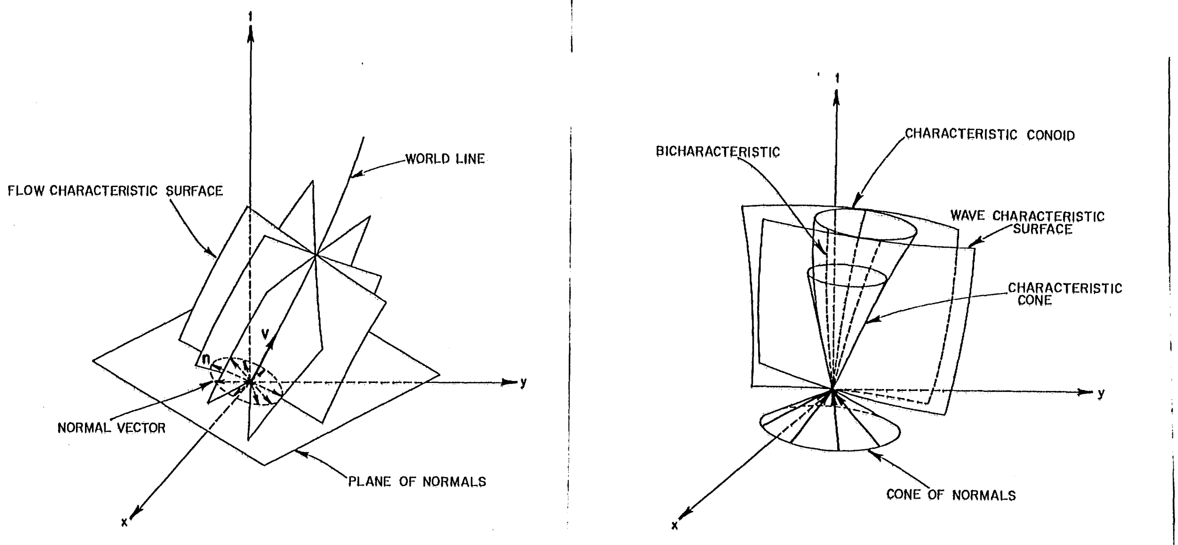

Fig. 1 Flow and Wave Characteristic Surfaces

Equation 4 can be written in the form of an interior differential operator that contains only two independent directions of differentiation. This is accomplished by rewriting the system of governing equations in non– conservative form, as follow s

$$a_{jki}\frac{\partial u_k}{\partial x_i} = b_j \tag{5}$$

in which

$$u_k = \begin{pmatrix} h \\ u \\ v \end{pmatrix}$$

$$V_i = \begin{pmatrix} 1 \\ u \\ v \end{pmatrix}$$

$$x_i = \begin{pmatrix} t \\ x \\ y \end{pmatrix}$$

$$a_{jki} = \begin{pmatrix} V_i & h\delta_{2i} & h\delta_{3i} \\ g\delta_{2i} & V_i & 0 \\ g\delta_{3i} & 0 & V_i \end{pmatrix}$$

$$b_j = \begin{pmatrix} 0 \\ g\left(S_{0_x} - S_{f_x}\right) \\ g\left(S_{0_y} - S_{f_y}\right) \end{pmatrix}$$

in which δ_{ij} is the Kronecker delta.

The system of Eqs. 5 pre-multiplied by an eigenvector w_j, results in a linear combination of the flow equations, whose coefficients vectors are co-planar. Thus, the direction normal to the plane of the coefficient vectors need not be included in the transformed equations, or equivalently, the derivatives of the dependent variables may suffer discontinuities in this normal direction. Two possible solutions for characteristic surfaces exist. The first family represents flow characteristic surfaces whose envelope is a space curve tangent to the world-velocity vector V_i, as shown in Fig. 1. The equation of the world line is given by

$$dx_i = V_i dt \tag{6}$$

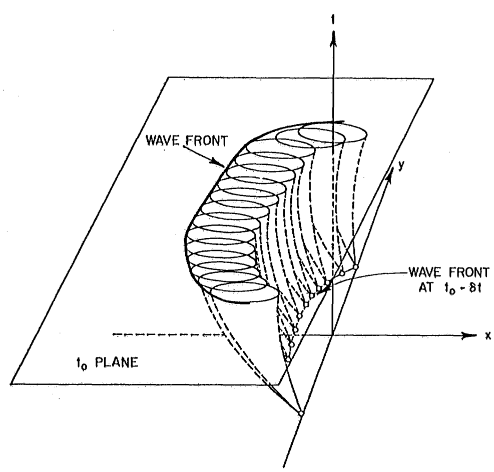

Fig. 2 Depression Wave Front

The corresponding compatibility equation is shown to be a form of the energy equation

$$-\frac{\partial h}{\partial t} + \frac{d_{WL}}{dt}\left[h + \frac{u^2+v^2}{2g}\right] = V_i b_i \tag{7}$$

The second family represents wave characteristic surfaces whose envelope is a conoid. The lines of tangency between the conoid and the characteristic surfaces are called bicharacteristics, with their equations given by

$$dx_i = (V_i + \nu_i c)\, dt \tag{8}$$

in which $c = (gh)^{1/2}$ and ν is the projection of the normal vector to the characteristic surfaces on the physical plane.

The compatibility equation along the bicharacteristics reads

$$\frac{d_{BC}[h]}{dt} + \frac{c}{g}\left(\nu_2 \frac{d_{BC}[u]}{dt} + \nu_3 \frac{d_{BC}[v]}{dt} + c m_j \frac{d_m}{dt}[V_j]\right) = \frac{c}{g}\nu_i b_i \tag{9}$$

It is obvious that this expression contains derivatives only along the bicharacteristic direction BC and the direction m, which lies on the characteristic surface and is normal to the bicharacteristic itself. Of the infinity of wave compatibility equations and the flow compatibility relation, only three wave equations are independent and thus a unique solution for the flow variables can be found at each point in two-dimensional space and time.

The existence of preferred directions for the propagation of wave information imposes strict conditions on the specification of initial and boundary data for shallow-water flow problems. Two of the flow variables need to be specified at subcritical inflow sections while one is sufficient at outflow regions. In supercritical flow all three variables must be supplied in the case of inflow while no conditions may be imposed at outflow sections. Perhaps most difficult to handle are propagating boundaries, which require a restructure of the computational grid at every time interval, as shown in Fig. 2. It is therefore quite clear that for a numerical solution of the shallow-water equations to be satisfactory, the characteristic directions must be preserved, boundary conditions must be properly enforced and in many cases the computational grid should be allowed to deform. If in addition, it is desirable to extend the solution to include discontinuities, irregular geometry and topography and at the same time we wish to maintain efficiency of computation, the choice of the numerical technique to be used becomes rather difficult. In recent years, advances in the finite element method have resulted in a numerical scheme that can perform quite satisfactorily under most of the conditions commonly encountered in practical problems. Similar methods based on finite difference schemes may be possible, the ease with which many tasks are carried out by finite elements, however, makes the following method very attractive.

DEFORMING FINITE ELEMENTS

The need to follow boundaries that behave dynamically during the course of the solution, makes it necessary to consider the implementation of deforming and moving finite elements. The development that has been used most extensively involves the reformulation of the governing equations in a more convenient set of space coordinates. Let (ξ, η, τ) be defined as a moving coordinate system with respect to the original frame of reference (x, y, t) such that the time derivative in Eq. 4 can be written as follows

$$\frac{\partial U}{\partial t} = \frac{\partial U}{\partial \tau} - \frac{\partial x}{\partial \tau}\frac{\partial U}{\partial x} - \frac{\partial y}{\partial \tau}\frac{\partial U}{\partial y}$$

Substitution in Eq. 4, leads to the following modified equation

$$\frac{\partial U}{\partial \tau} - \frac{\partial x}{\partial \tau}\frac{\partial U}{\partial x} - \frac{\partial y}{\partial \tau}\frac{\partial U}{\partial y} + A\frac{\partial U}{\partial x} + B\frac{\partial U}{\partial y} + D = 0 \tag{10}$$

In this set of equations $\frac{\partial x}{\partial \tau}$, $\frac{\partial y}{\partial \tau}$ are called the elemental velocities and are evaluated by interpolating the nodal velocities for each element. It is observed in Eq. 10 that the system reduces to the fixed grid formulation when the nodal velocities are set to zero. Hence the deforming grid scheme is suitable for both fixed grid and deforming element computations. While the system of equations in Eq. 10 adequately incorporates the effects of grid deformation on the dependent variable, it does not generate additional equations to specify the nodal motion. The latter, however, can be provided by the behavior of the moving boundary without any difficulty. In the case of a surge propagating on dry bed, for example, the normal component of the front velocity must be equal to the fluid velocity just behind the front, i.e.,

$$\int_{\Omega} (\mathbf{V} - \mathbf{W}).\mathbf{n}\, d\Omega = 0$$

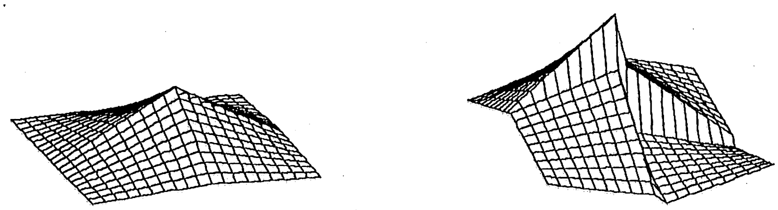

Fig. 3 Dissipative Galerkin Trial and Test Functions

in which **W** is the front speed and **n** is the outward normal to the boundary.

DISSIPATIVE GALERKIN FORMULATION

In the development of the dissipative finite element scheme, the choice of the weighting function, W, differs from the classical Galerkin method. Instead of selecting the weighting function as identical to the shape function, a discontinuous test function is used i.e.

$$N_\alpha{}' = N_\alpha + \epsilon_x A^T \frac{\partial N_\alpha}{\partial x} + \epsilon_y B^T \frac{\partial N_\alpha}{\partial y} \tag{11}$$

where ϵ_x and ϵ_y are the dissipation parameters.

For the special case that the coefficient matrices A and B are the identity matrix and $\epsilon_x = \epsilon_y = 1$, the axonometric representation of the bilinear shape functions in Eq. 11 and the discontinuous test functions are shown in Fig. 3.

For the approximation of the characteristic wave speed in one dimension to be accurate to the fifth order with respect to the mesh size, the dissipation parameter is required to assume a specific value. When this condition is generalized for the two–dimensional flow, the dissipation level can be estimated from

$$\epsilon_x = \epsilon_y = \frac{\Delta L}{\left[\sqrt{(p/h)^2 + (q/h)^2} + c\right]\sqrt{15}} \tag{12}$$

where $c = \sqrt{g\hat{h}}$, $\hat{h}$ = the fluid depth at the centroid of the element and ΔL = the length of a cord through the centroid of the element along the resultant flow direction. When the deforming element expression is cast in dissipative Galerkin form, the following equation results

$$\frac{\partial U}{\partial \tau} + A_* \frac{\partial U}{\partial x} + B_* \frac{\partial U}{\partial y} + D = 0 \tag{13}$$

where

$$A_* = A - I w_x$$
$$B_* = B - I w_y$$

and

$$w_x = \frac{\partial x}{\partial \tau} \quad , \quad w_y = \frac{\partial y}{\partial \tau}$$

and $I =$ identity matrix.

The weighted residual formulation corresponding to Eqs. 13 can be written as

$$f = \int_\Omega N'^T \left(\frac{\partial \hat{U}}{\partial \tau} + A_* \frac{\partial \hat{U}}{\partial x} + B_* \frac{\partial \hat{U}}{\partial y} + D \right) d\Omega \tag{14}$$

For the numerical integration in time, a second–order accurate finite difference scheme is applied between times $\tau = n\Delta\tau$ and $\tau = (n+1)\Delta\tau$, i.e.

$$\frac{\hat{U}^{n+1} - \hat{U}^n}{\Delta\tau} = \theta \left(\frac{\partial \hat{U}}{\partial \tau} \right)^{n+1} + (1-\theta) \left(\frac{\partial \hat{U}}{\partial \tau} \right)^n \tag{15}$$

θ is the weighting factor that is assigned a value of 0.5 in all computations in this study.

Eq. 15 can be rearranged as follows:

$$\left(\frac{\partial \hat{U}}{\partial \tau} \right)^{n+1} = \alpha \hat{U}^{n+1} - \beta \tag{16}$$

where

$$\alpha = \frac{1}{\theta \, \Delta\tau}$$

and

$$\beta = \alpha \hat{U}^n + \left(\frac{\partial \hat{U}}{\partial \tau} \right)^n$$

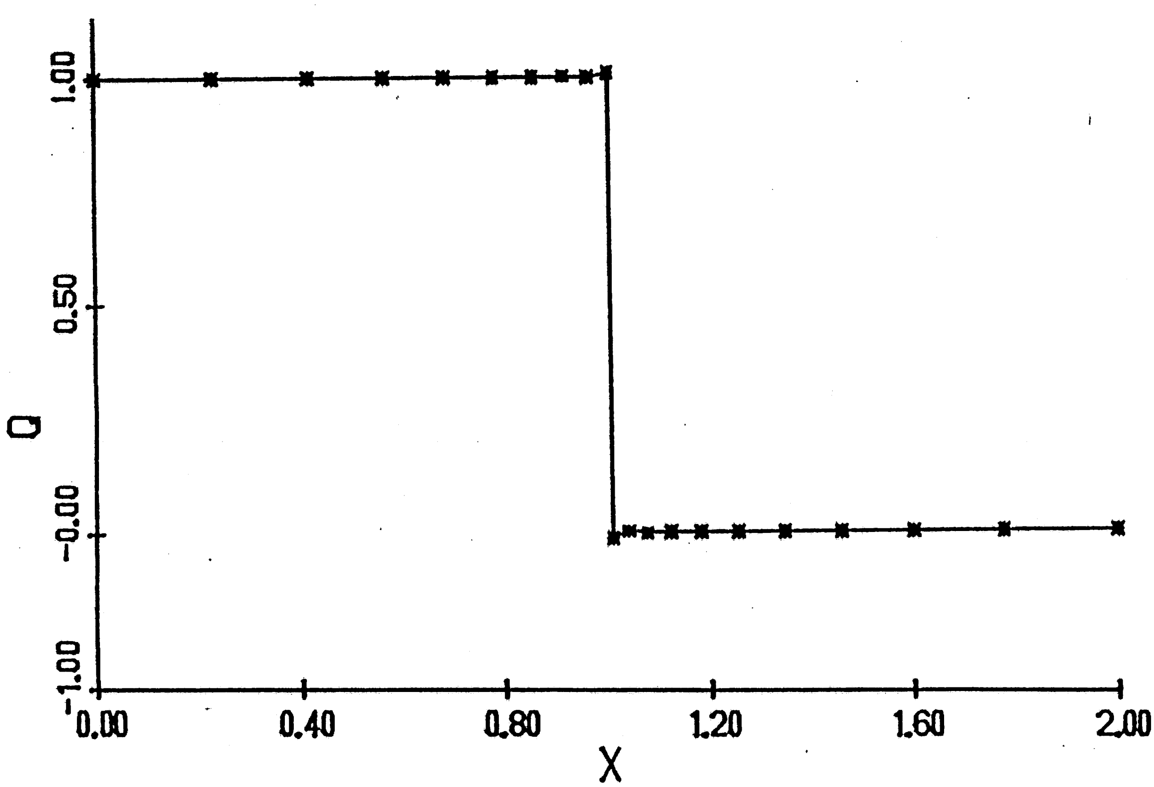

Fig. 4. Shock–Wave Simulation

Substituting Eq. 16 into 13 yields a set of nonlinear algebraic equations. The solution of these equations is obtained via the Newton–Raphson method. Given an initial estimate for the unknown nodal values, an improved estimate is obtained by

$$\hat{U}^{k+1} = \hat{U}^k - \Delta \hat{U}^k \tag{17}$$

where $\Delta \hat{U}^k$ is the solution of the linear system

$$\left\{ \frac{\partial f^k}{\partial \hat{U}^k} \right\} \Delta \hat{U}^k = -f^k \tag{18}$$

EFFECTS OF WAVE DISPERSION

In many instances, the theory of shallow water flow fails to describe the effects of pronounced free–surface curvature and leads to inaccurate results. In the case of sudden water release from a reservoir, for example, the shallow water theory predicts formation of a bore regardless of the relative depth on the two sides of the dividing barrier. Experimental evidence on the other hand, indicates that for moderate ratios of the initial depth an undular bore is generated with pronounced effects of wave dispersion. With very small modifications the method described in the previous section can be extended to account for wave dispersion, provided that the nonlinear terms in the governing equations are of the same order of magnitude with the dispersion terms.

For reasons of simplicity the following discussion is limited to flow conditions in a frictionless, horizontal channel of unit width. The equations of continuity and momentum can then be written as

$$\frac{\partial h}{\partial t} + \frac{\partial (uh)}{\partial x} + \frac{\partial (vh)}{\partial y} = 0 \tag{19}$$

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial h}{\partial x} - \frac{h_0^2}{3}\left(\frac{\partial^3 u}{\partial x^2 \partial t} - \frac{\partial^3 v}{\partial x \partial y \partial t}\right) = 0 \tag{20}$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial h}{\partial y} - \frac{h_0^2}{3}\left(\frac{\partial^3 u}{\partial x \partial y \partial t} - \frac{\partial^3 v}{\partial y^2 \partial t}\right) = 0 \tag{21}$$

These equations can be written in matrix form similar to that of Eq. 13 and the dissipative Galerkin method can be used once again, *mutatis mutandis*. The test function adopted is now a spline surface, which, following integration by parts, allows the third–order derivatives to be converted to lower–order terms. In fact, the test functions can become wave–dependent functions, which allows unified computation of dispersive and non–dispersive nonlinear waves.

DISCUSSION OF COMPUTATIONAL RESULTS

The simulation of shallow–water waves was shown to require a delicate balance of analysis and computation. The numerical method suggested possesses some attractive characteristics that are shown below in the form of some computational examples. Figure 4 shows the performance of the dissipative method in shock–wave simulation. The combination of deforming elements with the selective dissipation of the method result in excellent resolution of the shock. The discontinuity is contained within a single element and no effects of numerical dispersion or spurious oscillations are present. Figure 5 shows the selective dissipation ability of the scheme in the separation region near a channel expansion. The test corresponds to frictionless flow with vorticity–free initial conditions, so all signs of circulation are numerical in origin. It is clear that the dissipative Galerkin method out performs the standard Galerkin method with an added pseudo–viscosity term. The latter is needed by the Galerkin method for reasons of stability.

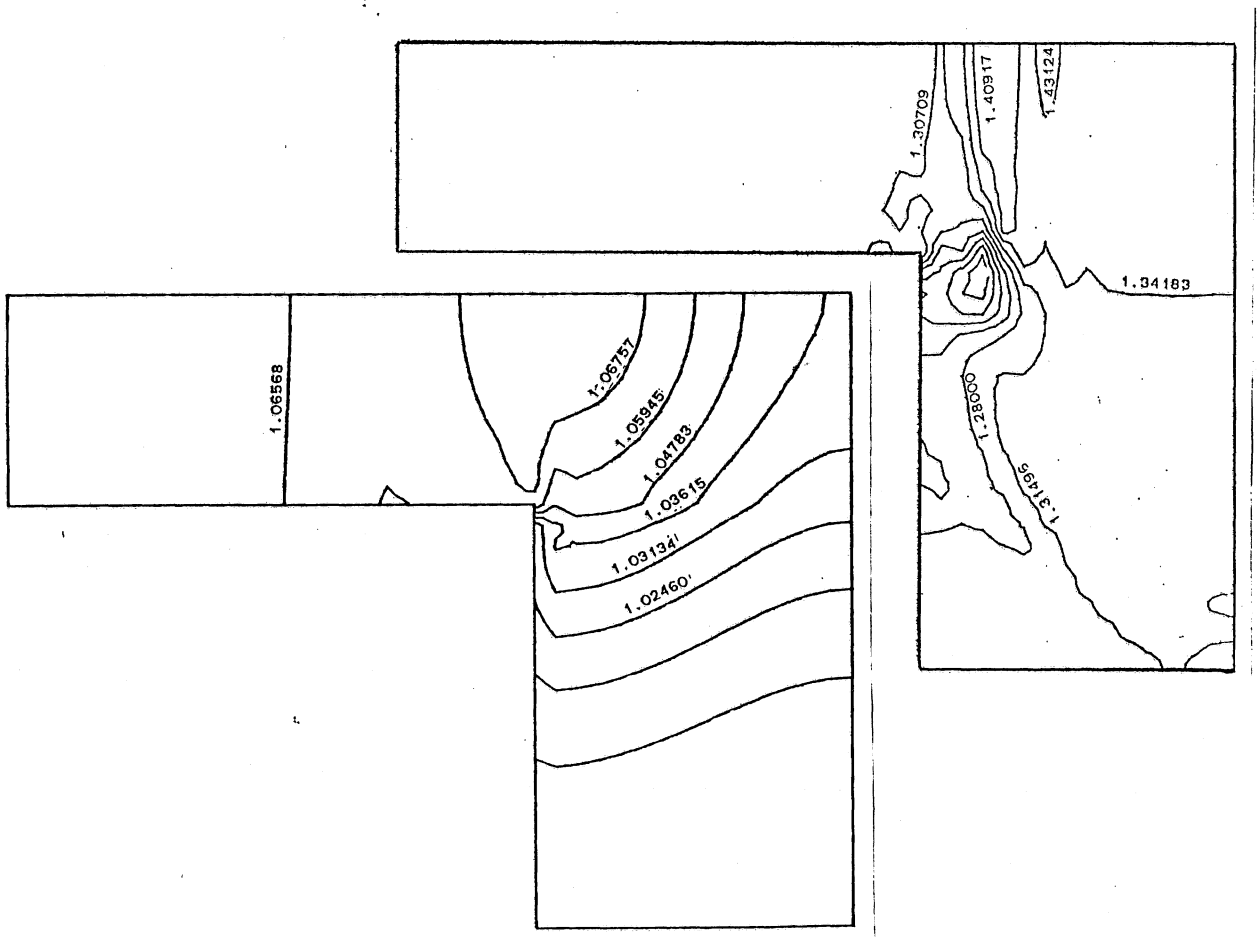

Fig. 5. Effects of Numerical Dissipation

Figure 6 shows a typical computation based on the dispersive form of the equations. The balance of non linearity and dispersion allows the solitary wave to travel without change in shape and makes it clear that in many cases the higher–order theory is necessary.

Finally, results of the dissipative Galerkin scheme from a practical application are shown in Fig. 7. The flow field corresponds to a section of the Detroit river and employs over a thousand elements.

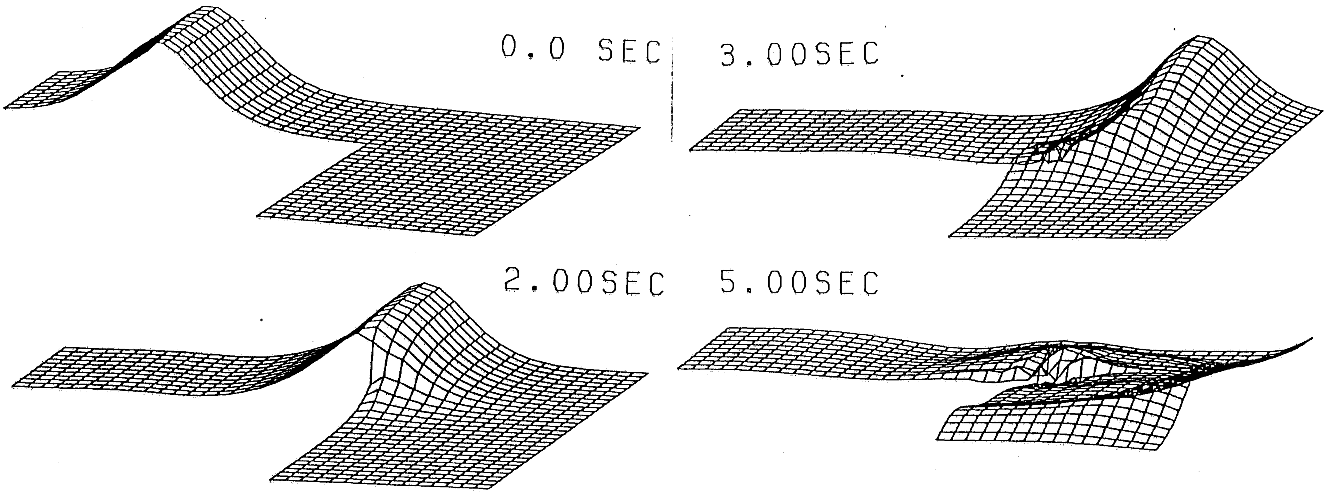

Fig. 6 Solitary Wave Simulation

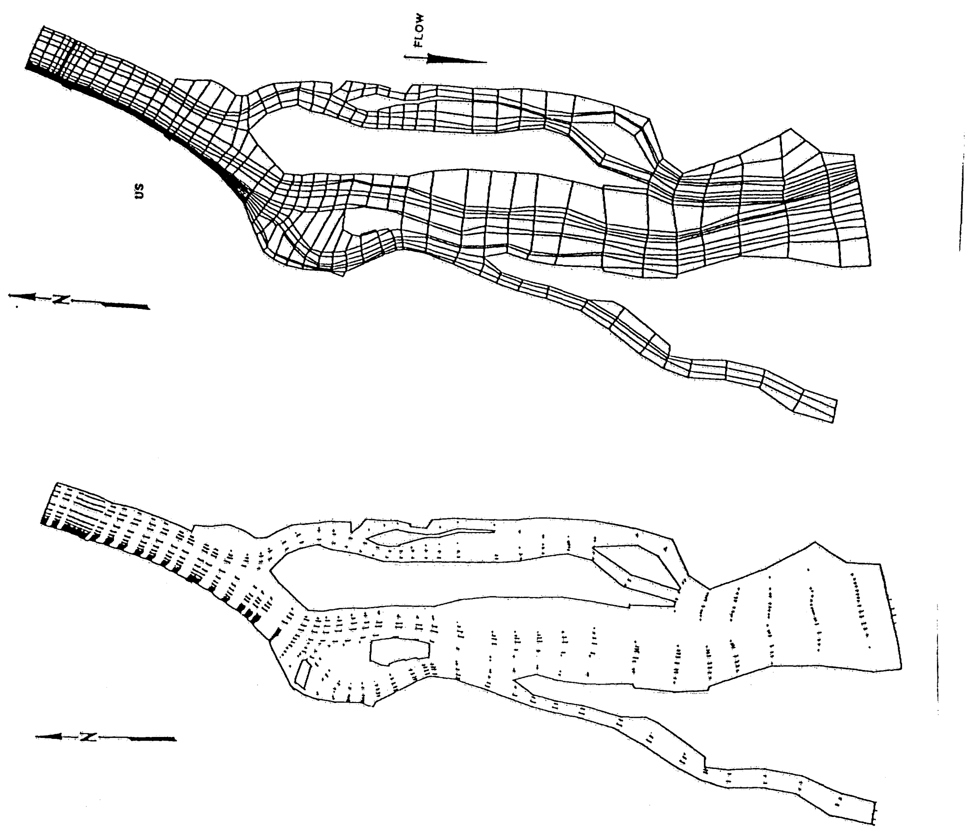

Fig. 7 Detroit River Model

In this particular application, a contaminant transport model has been coupled with the flow model, which necessitated a high density of computational points near the contaminant source. The performance of the method was found satisfactory even under severe variation and distortion of the computational grid.

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CHARACTERISTICS OF HYDRAULIC MACHINERY FOR HYDRAULIC TRANSIENT ANALYSIS

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SYNOPSIS

Calculation of transient performance of pump/turbine operation, while usually based on carefully collected model data, can be fraught with difficulty regarding numerical representation of the scaled-up model data. The difficulties are choice of data representation, numerical interpolation between guide vane openings, possibility of multiple values, and the crossing of curves. In this paper transformations based on model data taken by the author are utilized to open the curves to eliminate curve crossings and improve interpolation of data between guide vane positions. It is demonstrated that the occurrence of curves with multiple values can also be eliminated by proper stretching along the unit speed axis. The transformed and stretched curves, which are based upon correlations along the locked rotor and shutoff head axes, and the zero head lines in the pumping and reverse pumping zones, are deemed quite suitable for curvilinear interpolation, which is the recommended method of solution to the aforementioned difficulties.

INTRODUCTION

The computation of hydraulic transients in a hydroelectric system comprised of pump/turbines is sensitive to the manner in which the machine characteristics are used. For installations for which complete model data are available in all four quadrants in which the machine may operate, the analyst still must be able to interpolate properly between guide vane positions. Even for operation of a pump/turbine at a fixed guide vane position for which test data are available the problem of multiple values of unit discharge for a given unit speed can arise in the Q_{11} -- n_{11} plane. The crossing of lines in regions of normal pumping, energy dissipation, and reverse pumping can aggravate the problem of interpolation and selection of the proper unit values. By choice many analysts prefer to transform the unit plane into a head-flow plane. Such a transformation has the advantage of being more directly coupled to the dynamic equations of motion for the conduit hydraulics, as well as eliminating the problem of multiple values at a given guide vane position. The principal disadvantage is the further spreading of the curves, creating potentially greater numerical errors when interpolating between guide vane openings.

The interpretation of the hydraulic characteristics of a pump/turbine can be best understood by correlating test data along curves of specified conditions; namely, zero rotation (locked rotor), zero flow (shutoff), zero head, and zero torque (runaway). Although the flow patterns can be quite complicated for these situations, empirical correlations are possible. Furthermore, the characteristics of a pump/turbine near closed guide vanes can be interpreted better if the "zero" conditions outlined above are understood.

TEST PROGRAM

In order to have extensive data at numerous wicket gate positions, machine performance near closed gate and at zero head, and detailed test results on the axes of the unit plane, model tests were conducted by the author at the Technical University of Munich. A model of pressure diameter D_1 = 344 mm and specific speed n_{QT} = 44, using the units defined below, was employed for the testing. Test data for twelve guide vane positions are plotted in Figure 1 utilizing the standard definitions for unit quantities

$$n_{11} = \frac{ND_1}{\sqrt{H}} \tag{1}$$

and

$$Q_{11} = \frac{Q}{D_1^2\sqrt{H}} \tag{2}$$

in which N is the rotational speed in rpm, D_1 is the diameter in meters, Q is the flowrate in m^3/sec and H is the head in meters. The smooth curves depicted on Fig. 1 are cubic splines that pass through each of the some 80 test points for each gate position.

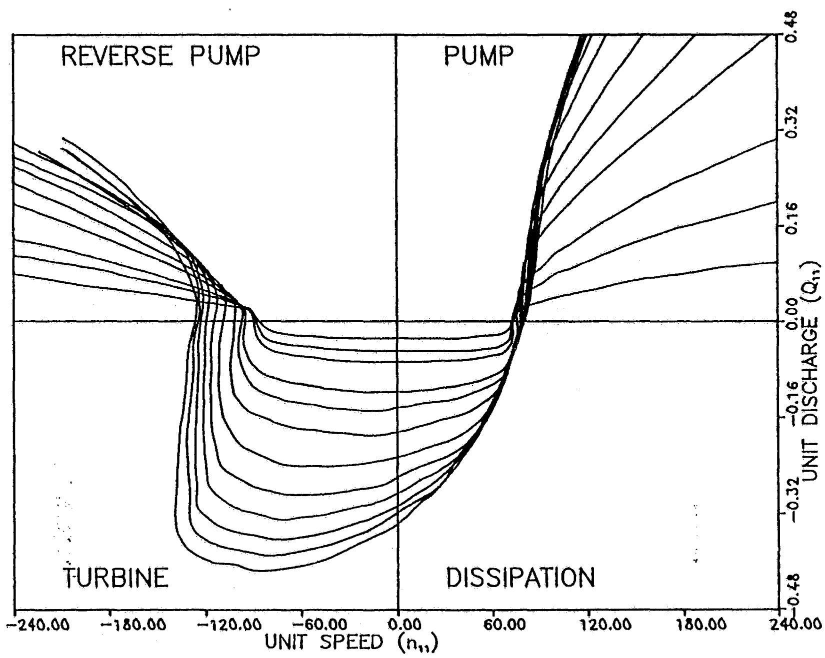

Fig 1 Smoothed machine data in unit plane

values of a_o, the major portion of the head loss is caused by the guide vane restriction, allowing for an evaluation of C_{dg} upon letting $H_r = 0$. Under the underlying assumption that both C_{dg} and C_{dr} are constants now C_{dr} can be evaluated at a large opening. For convenience the opening a_o, corresponding to optimum efficiency as a turbine, is chosen. If

$$\lambda = \frac{a_o}{a_{o\Lambda}} \tag{4}$$

then the final relationship is

$$v = \frac{Q_{11}}{Q_{11\Lambda}} = \frac{S_v \lambda}{\sqrt{1 + \lambda^2 \left[\left(\frac{S_v Q_{11\Lambda}}{Q_{11}}\right)^2 - 1\right]}} = \frac{S_v \lambda}{\sqrt{1 + B_v \lambda^2}} \tag{5}$$

where

$$S_v = \frac{a_{o\Lambda}}{Q_{11\Lambda}} \frac{dQ_{11}}{da_o} = \frac{a_{o\Lambda}\, n\, b}{Q_{11\Lambda}\, D_1^2} C_{dg} \tag{6}$$

CORRELATIONS

Locked Rotor

In order to determine the discharge characteristics of the pump/turbine under locked rotor conditions data were collected for very small guide vane openings and correlated with a_o. The data shown in Fig. 3 exhibit the linear correlation assumed in the theoretical development, allowing the determination of the discharge coefficient C_{dg} in Eq. (6) from the slope of the line. Over a range of $0<\lambda<2$, the nondimensional unit discharge exhibits the curvilinear relationships shown by the data in Fig. 3.

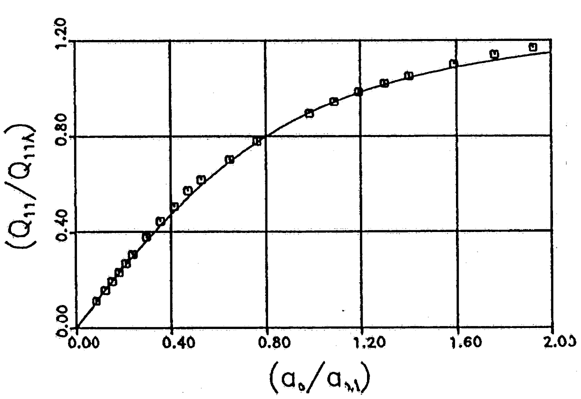

Fig 3 Dimensionless flow versus dimensionless guide vane opening for locked rotor (N=0)

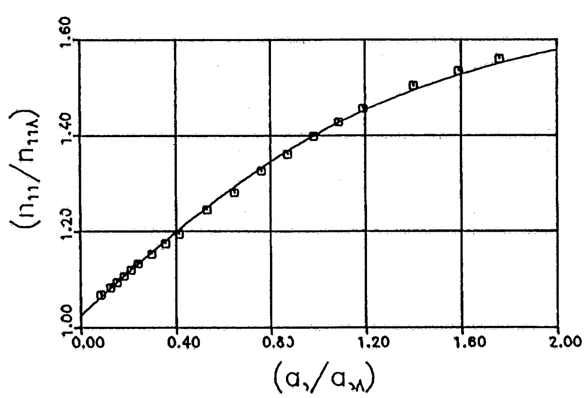

Fig 4 Dimensionless speed versus dimensionless guide vane opening for zero flow (Q=0) and reverse pumping (N<0)

Zero Flow

The zero flow axis for N<0 is very important with regard to extrapolation of data toward closed guide vane position, especially when data are not available below a certain opening. The values of n_{11} along this axis and the zero head asymptotes for reverse pumping are very useful in the proper formulation of algorithms for interpolation, extrapolation and transformation. Compared to the unit discharge versus guide vane relationship for small values of a_o under locked rotor conditions, the unit speed exhibits a similar curve along Q = 0, as shown by Fig. 4, on which the data are represented by the following equation

$$\frac{n_{11}}{n_{11\Lambda}} = \frac{n_{110}}{n_{11\Lambda}} + \frac{S_\alpha \lambda}{\sqrt{1 + B_\alpha \lambda^2}} \qquad (7)$$

in which n_{110} is the value of unit speed on the minus Q = 0 axis.

Zero Head

In order to calculate pump/turbine performance during reverse pumping, data corresponding to relatively low heads is sometimes needed. Furthermore, the transformation of normal pumping and reverse pumping data can be facilitated by knowing empirical relationships at not only zero flow, but also zero head. Figure 5 shows the resulting correlations for both modes of pumping under zero head conditions. The empirical correlations, albeit not founded on theory, are quite well represented by

$$\left| \frac{Q_{11}}{Q_{11\Lambda}} \; \frac{n_{11\Lambda}}{n_{11}} \right| = \frac{S_{v\alpha} \lambda}{\sqrt{1 + B_{v\alpha} \lambda^2}} \qquad (8)$$

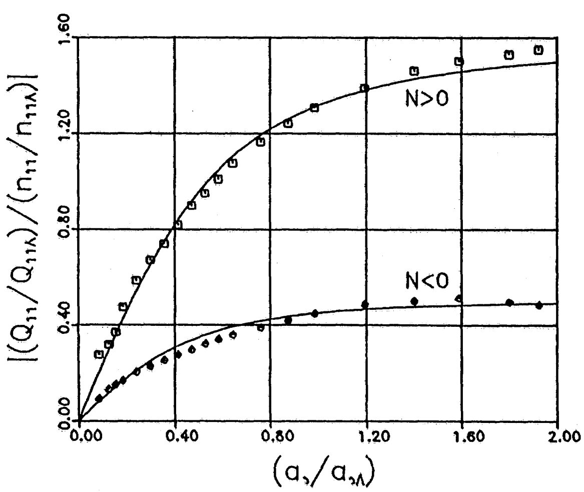

Fig. 5 Dimensionless ratio of unit flow to unit speed versus guide vane opening for zero head in quadrants I and II

TRANSFORMATIONS

The difficulties of multiple values, line crossings and numerical errors in interpolation addressed earlier can be reduced when using Fig. 1 by employing suitable transformations. If the curves of Fig. 1 can be opened and stretched by proper transformation, then linear curvilinear interpolation between guide vane openings can be employed with greater certainty in trouble regions. Furthermore, multiple values and line crossings can be eliminated.

Locked Rotor

Clearly, in Eq. (5), S_V is the flow coefficient of the guide vanes, showing a linear relationship between unit flow Q_{11} and vane opening a_o for smaller openings. If the model test data fit Eq. (5) well, then a transformed unit flow Q_{11} can be defined that is nearly linearly dependent upon the guide vane opening λ. In terms of a coefficient C_V, which depends upon λ,

$$\frac{Q'_{11}}{Q_{11\Lambda}} = v' = C_v\, v = \frac{\sqrt{1 + B_v\, \lambda^2}}{S_v}\, v \tag{9}$$

By comparing Eqs. (5) and (9) one sees that v' should vary linearly with λ, allowing for the nearly uniform spacing of curves of model test data at equal spacing of guide vane opening -- at least on the zero rotation axis. The collapsing of curves could be accomplished by defining

$$\frac{Q''_{11}}{Q_{11\Lambda}} = v'' = \frac{C_v\, v}{\lambda} = v \tag{10}$$

Zero Flow

The variation of the unit speed n_{11} along the normal pumping axis ($N > 0$) is very minimal, allowing for the assumption of constant speed for collapsing of curves, or a linear variation of the transformed speed versus guide vane position for nearly uniform spacing of curves. Analytically, the definitions are

$$\frac{n'_{11}}{n_{11\Lambda}} = \alpha' = C_\alpha\, \alpha = \lambda\, \alpha \tag{11}$$

for opening mode and

$$\frac{n''_{11}}{n_{11\Lambda}} = \alpha'' = \frac{C_\alpha\, \alpha}{\lambda} = \alpha \tag{12}$$

for collapsing mode.

For negative rotation ($n_{11} < 0$) along $Q_{11} = 0$, the spacing between curves is initially equal and then gradually increases with guide vane opening, allowing for an expression analogous to Eq. (5). In this case, however, there is an unique non-zero value of n_{11} as the guide vanes approach closed position. The correlation is

$$\alpha = \frac{n_{11}}{n_{11\Lambda}} = \frac{n_{11o}}{n_{11\Lambda}} + \frac{S_\alpha \lambda}{\sqrt{1 + B_\alpha \lambda^2}} \tag{13}$$

The respective transformed zero flow axes now become

$$\frac{n'_{11}}{n_{11\Lambda}} = a' = C_\alpha(\alpha-\alpha_o) = \frac{1+B_\alpha\lambda^2}{S_\alpha} \left(\frac{n_{11}}{n_{11\Lambda}} - \frac{n_{11o}}{n_{11\Lambda}}\right) \tag{14}$$

and

$$\frac{n''_{11}}{n_{11\Lambda}} = \alpha'' = \frac{C_\alpha(\alpha - \alpha_o)}{\lambda} \tag{15}$$

Zero Head

As shown by Fig. 1 the unit curves in the zones of normal and reverse pumping are quite sensitive to guide vane opening as the opening is reduced. The elimination of the folding down of these curves can be accomplished if appropriate correlations of the zero-head asymptotes can be formulated. As given in Eq. (8), the asymptotes for quadrants I and II are again similar to Eq. (5) in terms of

$$\left|\frac{v}{\alpha}\right| = \left|\frac{Q_{11}}{Q_{11\Lambda}} \frac{n_{11\Lambda}}{n_{11}}\right| = \frac{S_{v\alpha} \lambda}{\sqrt{1 + B_{v\alpha} \lambda^2}} \tag{16}$$

The correlation given by Eq. (16) will generate more nearly uniform spacing of curves with guide vane opening if transformed, but maintain the folding and crossing of curves. The collapsing of the data near the asymptotes can be accomplished by

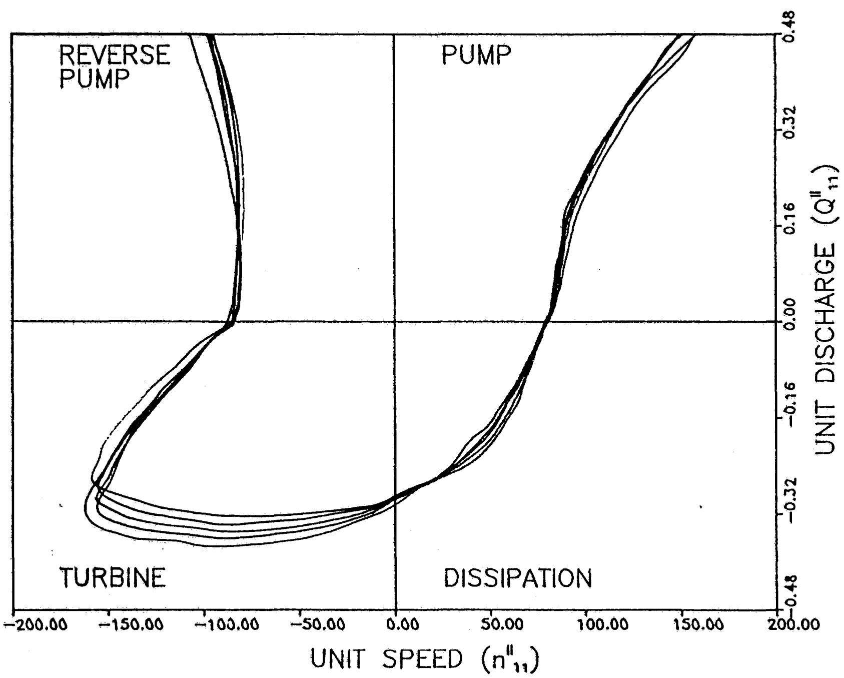

Fig 6 Transformed unit plane (collapsed mode)

$$\left|\frac{v''}{\alpha''}\right| = \frac{C_{v\alpha}}{\lambda}\left|\frac{v}{\alpha}\right| = \frac{\sqrt{1 + B_{v\alpha}\lambda^2}}{S_{v\alpha}\ \lambda}\left|\frac{v}{\alpha}\right| \qquad (17)$$

One possibility for curve opening is the division of Eq. (17) by λ, or

$$\left|\frac{v'}{\alpha'}\right| = \frac{C_{v\alpha}}{\lambda^2}\left|\frac{v}{\alpha}\right| \qquad (18)$$

Results

Using the empirical correlations for the axes and the two zero-head asymptotes some of the data of Fig. 1 has been transformed into Fig. 6 to illustrate how well the curves can be collapsed, and into Figure 7 for the purpose of opening the curves. For quadrant I Eqs. (12) and (17) or (11) and (18) have been applied, respectively. Equations (9) and (10) have been used for the entire negative flow region. For quadrants II and III Eqs. (14) and (15), which are valid on the $Q_{11}' = 0$ ($n_{11} < 0$) axis, have been modified by a coefficient times α_o. The unit speed axis for quadrant IV is transformed identically to that for quadrant I. The curves plotted on Fig. 6 indicate reasonable collapsing except in certain regions of quadrants III and IV.

Interpolation between guide vane positions is perhaps better accomplished using Fig. 7 than Fig. 6, but there is the problem of multiple values for $n_{11} < 0$. This difficulty can be eliminated by stretching the negative n_{11} axis by

$$n_{11s} = 2n_{11}' \left(1 - \frac{Q_{11}'}{Q_{11\Lambda}}\right) \qquad (19)$$

Figure 8 illustrates the stretched unit speed axis. If numerical interpolation in the unit-speed unit-flow plane is desired it is recommended that curvilinear coordinates with linear interpolation between guide vane positions is applied to Fig. 8, with the final values of n_{11} and Q_{11} determined upon transformation back to Fig. 1.

SUTER PLANE

Most of the data plotted in Fig. 1, which clearly shows multiple values in quadrants II and III and many line crossings, have been transformed into the so-called Suter plane (Fig. 9). The Suter representation is defined by

$$\frac{h}{v^2 + \alpha^2} = F_{v\alpha}(\theta) \qquad (20)$$

in which $\theta = \pi + \tan^{-1}(v/\alpha)$.

The advantage of the Suter representation over typical head-discharge plots is the absence of singularities that occur in head and discharge as n_{11} passes through zero. Neither $F_{v\alpha}$ nor θ are singular for zero values of either n_{11} or Q_{11}, yielding a continuous curve over the entire range of $0 < \theta < 2\pi$. Note that Fig. 9 does not contain negative head data in quadrants

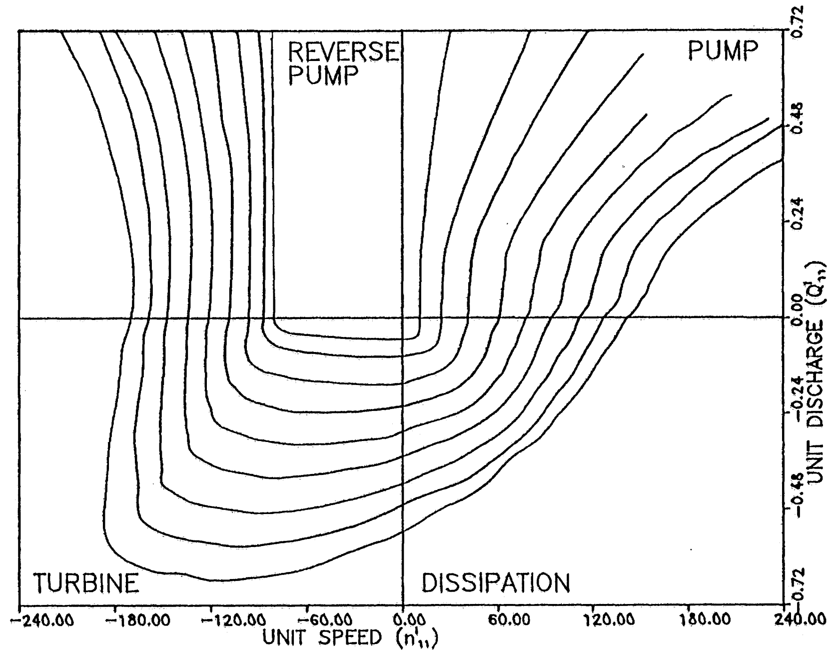

Fig 7 Transformed unit plane (opened mode)

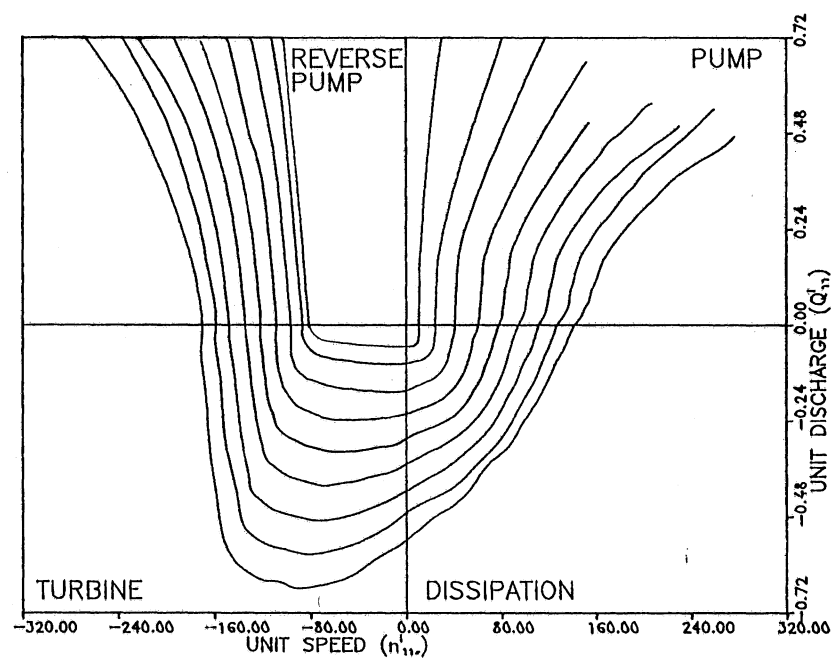

Fig 8 Transformed unit plane (stretched mode)

I and II -- as is typical for model tests of pump/turbines. A disadvantage of the Suter form of presentation is the lack of physical interpretation of either $F_{v\alpha}$ or θ. For computer representation this should not be a significant drawback, however. Indeed, the major disadvantage of Fig. 9 is the large variation in $F_{v\alpha}$ between guide vane openings at a given θ. The same difficulty occurs, of course, when using a head-discharge plot.

For computation of hydraulic transients using the Suter plane for representation of machine characteristics, the above transformations can be applied to Fig. 9 as follows. Modified Suter planes can be generated either in terms of v'-α' or v''-α'', depending upon whether one desires to open or collapse the curves, respectively. For opening the curves the modified Suter relationship for quadrant IV is

$$\frac{h}{v'^2 + \alpha'^2} = \frac{h}{C_v^2 v^2 + \lambda^2 \alpha^2} \tag{21}$$

Likewise, the collapsed version for quadrant IV is

$$\frac{h}{v''^2 + \alpha''^2} = \frac{h}{C_v^2 \left(\frac{v}{\lambda}\right)^2 + \alpha^2} = F''_{v\alpha}(\theta) \tag{22}$$

Transformation of the data of Fig. 9 is shown in Fig. 10 using the above relationships.

CONCLUSIONS

By employment of suitable transformations based upon careful empirical correlations along selected axes and asymptotes of the unit-discharge unit-speed plots, pump/turbine hydraulic characteristics can be represented either in planes for which the curves are (1) more proportionally spaced or (2) somewhat collapsed. The transformation is of especial value in the unit plane in that the problems of multiple values and line crossings can be eliminated. Interpolation in the unit plane using curvilinear coordinates can be improved because the spacing between curves becomes more proportional with guide vane opening. In the modified Suter planes interpolation errors will be reduced both for expanded and collapsed sets of curves.

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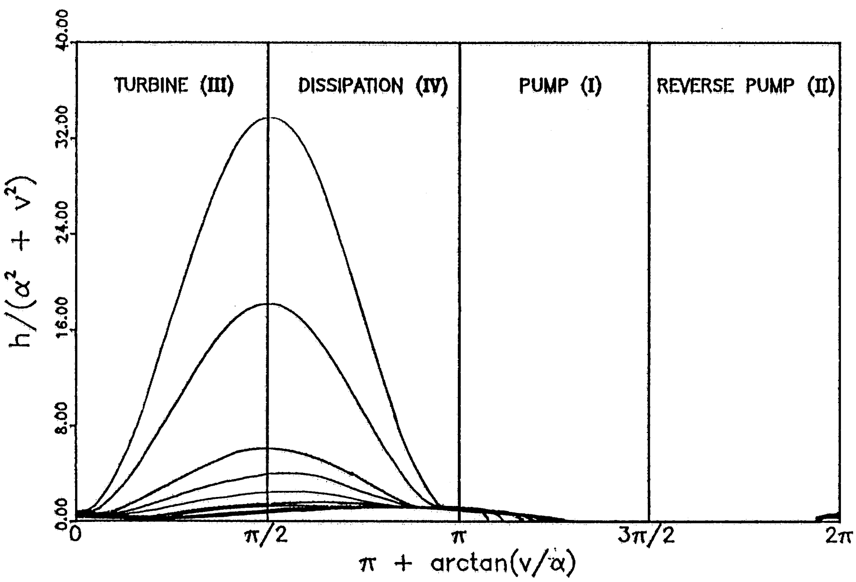

Fig 9 Standard suter plane

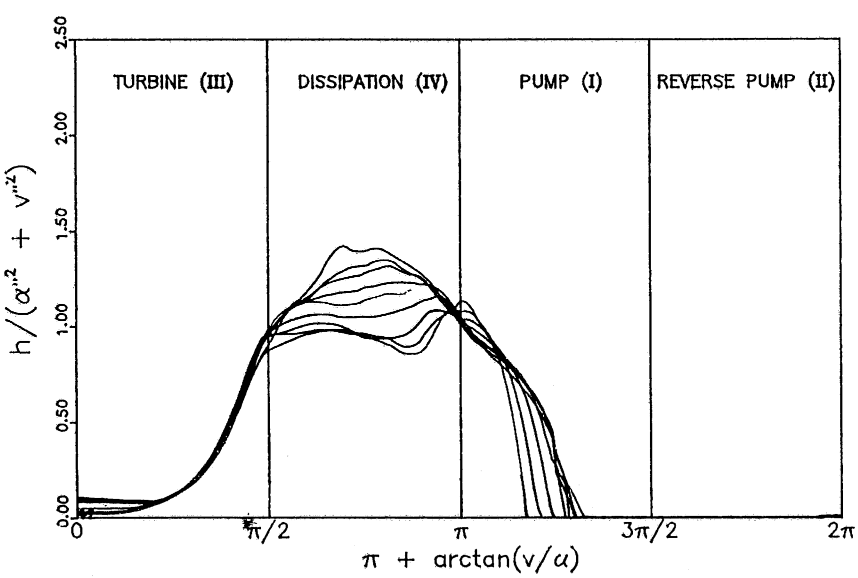

Fig 10 Transformed suter plane (collapsed mode)

CAVITATION INCEPTION

by

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JULY 21-28, 1986

BUCHAREST

1. **Introduction.** Cavitation is a pervasive phenomenon in the flow of liquids. International symposia are frequent on the subject and the number of publications each year devoted to cavitation shows no sign of diminishing. There are usually three phases of the effects of cavitation on hydraulic devices including structures; these are, briefly, cavitation inception, effects of performance change and cavitation damage. The onset or inception of cavitation is most associated with the minimum pressure in the flow which is sensible, of course, but in real fluids this minimum pressure is often not known, and in any event the process of liquid vaporization does not commence until the minimum pressure is less than the equilibrium vapor pressure. The normal cavitation index is the pressure coefficient $k = (p-pv)/q$ where p is a reference static pressure, pv is vapor pressure and q is a dynamic pressure. Such a definition is bound to be a poor predictor of inception because of the departure from perfect equilibrium in formation

of the vapor. Thus we have a "scale" effect is the estimation of inception if pv is equated to the minimum pressure, provided this is known.

In the talks to follow this scale effect on cavitation, inception is described with examples of several different kinds of flow occurring in practice. This is followed by a survey by Professor Arndt of cavitation damage theory with applications to hydraulic machinery and structures, and then by Dr. Falvey with spectacular examples of cavitation damage in large hydraulic structures, together with some new thoughts on the origins of this kind of damage and a description of means to alleviate the damage.

2. **Remarks on Inception:** The following comments cover the gist of the inception presentation. These are kept brief in the interest of time and because the general ideas are now fairly well disseminated. And it must be said that in hydraulic structures and hydraulic machines too, inception per se is not as much interest as it is in other fields. There are, however, still other examples of severe scale effect in the testing of hydraulic machines that may well be of general concern to the hydraulic community. These remarks are abstracted from the present work of the author, and colleagues such as Prof. Arndt who are active in the field, and many international co-workers. Fortunately, many of these results are available in recent symposia and systematic reviews. Several of these are appended for reference to the current literature dating from about the last ten years through the present. Abbreviated comments follow.

2.1 **The nature of the flow.** It is now more clearly recognized that the liquid flow itself can play a vital role in the behavior of developed cavitation as well as inception. In particular, it is essential to know if the boundary layer is laminar, turbulent, or if a separation is present. If a laminar separation is present in a model test it is essential to know if a prototypical application will have such. It is readily possible to cause a turbulent transition thereby forestalling laminar separation on the model body. However, this does not necessarily improve the modelling situation for cavitation as then a different and perhaps more subtle process occurs. This is, in short, the nature of the real liquid itself, the topic of the next section.

A rather different situation presents itself for flow characterized either by (a) skin friction primarily, or (b) a massive laminar or turbulent separation as occurs, for example, for the flow past a flat plate or bluff body, such as a cylinder. In (a), if the flow is turbulent, pressure fluctuations can lead to cavitation following a simple rule proposed by Arndt; a steady laminar flow of course does not lead to any pressure excursion and thus cavitation. In (b), the basic flow itself is exceedingly complex and is to this day a subject of intense study by the turbulence community. It has been known for years, however, that these cavitation indices are very large (from 1-3) and exhibit an apparent Reynolds number dependence that is quite strong. These kinds of flows are not academic curiosities because they are found in many applications. What is interesting is that cavitation is first see in the axial "secondary" vortex structures of these flows. This observation opens up a means possibly of understanding the Reynolds scaling of these flows and may contribute to the understanding of these complex flows.

2.2 The liquid itself. It has been known for some years, following pioneering work at the Saint Anthony Falls Hydraulic Laboratory that the presence or absence of microbubbles had an important effect on cavitation. One suspects that it was not generally known how important this property of the liquid actually is for the onset of cavitation as well as the development of more advanced stages. By the time of the early 1960's resorber water tunnels were becoming more common in cavitation testing and as a result significant "scale" effect particularly for inception was becoming evident, specially for certain test bodies. The climax of this work was no doubt the round-robin inception tests promoted by SSPA and published by C. A. Johansson in the Intl. Towing Tank Conference in 1969; there inception tests on one body shape revealed significant differences in cavitation index and the type of cavitation on geometrically similar bodies differed significantly. Part of the differences observed were due perhaps to the various turbulence levels in the several test facilities (nearly 10) but it seemed most clear that the differing amount of free-stream microbubbles was the determining factor. Since these seminal results appeared many workers have been able to show the dramatic effect of "nuclei" concentration on the onset and then development of cavitation among whom may be mentioned Prof. Andreas Keller at the University of Munich and J.H. van der Muelen at the Netherlands Ship Model Basin, in particular. Since then observation of and measurement of the cavitation "nuclei" have occupied many research groups around the world.

This aspect of cavitation, namely that of the liquid itself rather than the flow, has come to be called the "liquid quality" or "susceptability" after Oldenziel (Delft Hydraulic Laboratory) and Lecoffre (Neyrtec) and a special test Venturi tube to detect cavitation in a special way as developed by them is called a "cavitation susceptability meter" or CSM. It has not yet proven possible to connect directly the output of such a device with the actual concentration and size distribution of microparticulates or microbubbles although much effort is directed to this end. It may be safely stated that microparticulate and "nuclei" measurement remains an intense and active research field around the world. It is especially interesting to consider the environment of the natural waters for turbines, pumps, and propellers operate in lakes, rivers, and of course, the oceans. One such example is the recent work at my own institute in which a ruby holographic camera was used to record holograms in the ocean up to 33 meters depth. Analysis of these revealed microbubbles and many forms of microplankton. Whether these biota serve as cavitation nuclei is not yet known.

Cavitation erosion is naturally a major consideration in any hydraulic facility. It is an interesting observation that erosion measurements both in the laboratory and the field have not as yet taken into consideration these nuclei concentrations nor their source. I believe it is fair to say that in the near future such measurements in all fields concerned with cavitation including hydraulic engineering will become routine, once standard instrumentation becomes available.

3.0 **Concluding Remarks:** These brief remarks conclude this introductory section. Supportive materials will be found in the appendix to this and the presentations to follow.

4.0 **Appendix:** Recent publications in Cavitation (attached).

THEORY

The axes and asymptotes corresponding to the "zero" conditions are indicated on the unit-characteristics plane of Fig. 2. The locked rotor axis ($N=0$, $Q<0$) is the boundary between dissipation and turbine operation. The zero flow axis corresponds to shutoff conditions for normal ($N>0$) and reverse ($N<0$) pumping. In quadrants I and II each guide vane curve becomes asymptotic to the zero head line. Although none of these "zero" conditions are of any consequence in an operational sense except perhaps under very unusual circumstances or transient conditions, they are important regarding transformation to other planes and extrapolation toward closed guide vanes.

Although the locked rotor condition is almost never realized in testing, and only under transient situation in practice, the axis of zero rotation in four-quadrant representation is a useful one for the understanding of hydraulic performance. Under this purely dissipative condition there is a head loss across the unit due to two components -- the guide vanes and then the stationary runner. By assuming the total head loss $H = H_g + H_r$, where H_g and H_r are the individual head loss for the guide vanes and runner, respectively, the flow through the unit

$$Q = Q_{11} D_1^2 \sqrt{H} = C_{dg} \, n \, a_o \, b \sqrt{H_g} = C_{dr} D_1^2 \sqrt{H_r} \qquad (3)$$

where n is the number, b the height, and C_{dg} the discharge coefficient of the guide vanes, respectively, and C_{dr} is a discharge coefficient of the runner. Knowing the geometry and Q_{11} versus a_o from model tests, the task is to evaluate C_{dg} and C_{dr}. It is reasonable to assume that, at lower

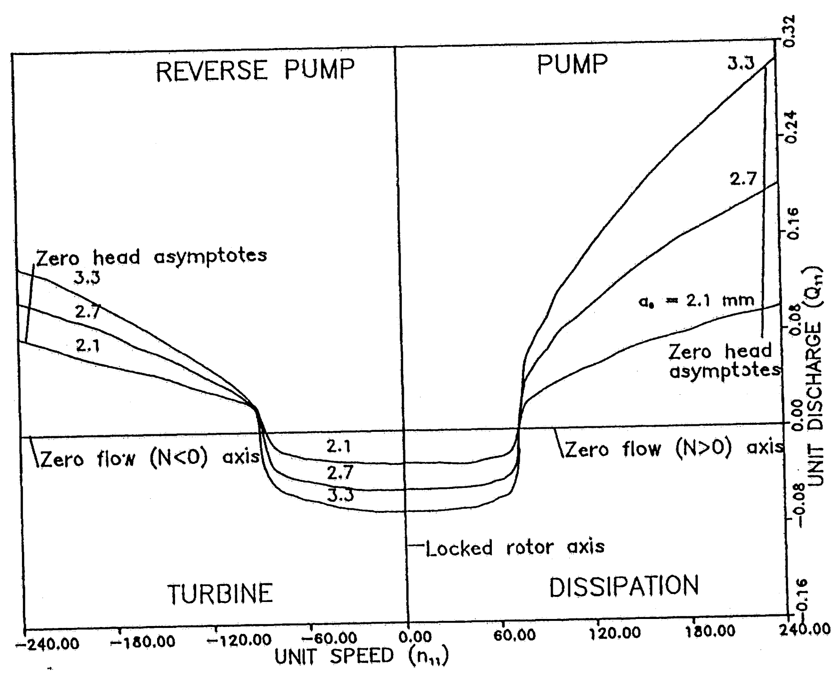

Fig 2 Plot of unit characteristics for small guide vane openings

REVIEW PAPER
ON
AERATORS TO PREVENT CAVITATION IN SPILLWAYS

by

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INTRODUCTION

In 1941, the spillways at Hoover Dam operated for the first time (1). After the spill it was found that a large cavity had developed in the invert of the circular spillway tunnel on the Arizona side of the dam, figure 1. In the studies which followed, several explanations were given for the cause of the damage, one of which was cavitation. As a result, a series of investigations were begun to investigate damage causing cavitation and means of mitigating the damage.

One of the ideas investigated was the use of a device which could entrain air near the flow boundary and thus "act as a cushion between the high-velocity water and the tunnel lining. Secondly, the same air would aid in relieving any subatmospheric pressures which may occur along the surface of the tunnel invert" (2). The device would rely upon hydromechanic forces to entrain the air and thus would not require air compressors or other mechanical devices to achieve the desired result. The generic name of these types of devices is an "aerator".

An aerator was not selected for Hoover because air could not be found to stay near the flow boundaries in laboratory investigations (2). The location of the aerators which were studied were just upstream of the vertical elbow, in the elbow, and near the crest at the inlet of the tunnel. Since surface irregularities were known to cause damage, the solution at Hoover was to finish the entire damaged area with a terrazzo type finish. Subsequent experience has shown that this solution was not adequate. As a result, aerators have been designed for Hoover and the first one was installed in the Nevada Tunnel in 1986. An aerator will be installed in the Arizona Tunnel in the next year or two.

The first successful application of aeration to alleviate cavitation damage in an operating structure was at Grand Coulee Dam (3). In this case, excessive damage was occurring at the intersection of the river outlet tubes with the downstream face of the spillway. Previous attempts to achieve smooth surfaces and to protect the surface with epoxy coatings had been unsuccessful. After installing the aerators, the outlets have operated successfully for thousands of hours with no visible signs of damage.

As a result of the successful application of aerators at Grand Coulee, aerators became the panacea for all cavitation produced damage both within the Bureau and world wide. Today one sees them scattered on structures much like salt strewn from a saltcellar. In many cases, the aerators provide the desired function of preventing damage. In other cases, the aerators have been improperly placed and consequently they have been quietly removed. Finally, there are many instances where an over abundance of aerators have been installed which serve no other function than to make the dam operators and owners feel more comfortable when the structure operates.

The application of aerators to the alleviation of cavitation caused damage requires some care in their design. To be efficient they must be sited in the appropriate location, have the proper dimensions, and be supplied with an adequate air supply. The purpose of this paper is to review the current theories which apply to the design of aerators, examine the design parameters, and to indicate areas where additional research are needed. The discussion is limited to aerators on chute and tunnel spillways although aerators are also useful in protecting other types of hydraulic structures. The intent of this review is to present the concepts of aerator design. The reader is encouraged to read the references for details.

THEORY

Some background information needs to be reviewed before the discussion of the aerator design can begin. For many years it has been assumed that the cavitation is caused by surface irregularities in the high velocity flows. Therefore, the cavitation characteristics of typical surface irregularities is considered first. This is followed by a discussion of other causes for cavitation. One of the principal mechanisms other than surface irregularities is longitudinal vortices in the flow. In many cases this vortex induced cavitation appears to be a much more severe inducer of damage than the surface irregularities. The discussion on cavitation is followed by a consideration of the damage mechanisms due to cavitation and efforts that have been proposed to quantify the onset of damage. Finally, the current theories for the air entrainment mechanism are reviewed in light of field and laboratory studies.

A. Cavitation Characteristics

Surface roughnesses fall into two main categories; isolated roughnesses and distributed roughnesses. Typical isolated roughnesses include calcite deposits, misalignments, localized voids in the flow surface, and changes in the flow alignment. A finished concrete surface would be a typical distributed roughness.

The cavitation characteristics of all types of irregularities are dependent upon local velocities, pressures, tubulence levels, and the presence of free air bubbles and cavitation nuclei. With this many variables, it is difficult to find a single parameter which can be used to correlate experimental observations and then apply the correlations to actual field conditions. In addition, experimenters have not standardized the location at which the significant measurements should be taken. Therefore, it is possible to find many different correlations which describe the same phenomenon. The correlations used in this paper appear to be the best which are presently available in the literature. However, it should become obvious that more comprehensive studies still need to be conducted.

The fundamental relationship to describe cavitation is known as the cavitation index. It is defined as

$$\sigma = \frac{p_o - p_v}{\rho V^2/2} \qquad (1)$$

where σ = cavitation index
p_o = reference pressure
p_v = vapor pressure
V = reference velocity
ρ = density of water

In this definition, the pressures must be expressed in consistent units. It is recommended that they be referred to absolute units to avoid confusion. In absolute units the local barometric pressure is added to the measured gage pressure and the vapor pressure of water is equal to 1.23 kPa at 10 °C.

1. Cavitation Inception at Isolated Irregularities

The cavitation characteristics of isolated into-the-flow irregularities was investigated in some detail by Arndt et al (4). In their studies the correlations were formed as functions of the Reynold's Number and the height of the irregularity relative to the boundary layer thickness, figure 2. The reference velocity is taken at the outer edge of the boundary layer and the reference pressure upstream of the irregularity.

Mefford and Falvey (5), using a similar approach, investigated the cavitation characteristics of circular holes in a boundary, figure 3. This configuration is similar to the holes left in concrete when the concrete is poorly consolidated during placement.

2. Cavitation Inception at Uniformly Distributed Roughnesses

If the roughness height is small enough, then the entire surface will begin to cavitate as the pressure is lowered or the velocity increased. Arndt et al (5) studied this process and discovered that the cavitation index is only a function of the local coefficient of friction. This relationship is given by

$$\sigma = 16\, C_f \tag{2}$$

where C_f = local coefficient of friction
= $f/4$
f = Darcy - Weisbach friction factor

Eroded concrete surfaces have a texture which looks very much like a uniformly distributed roughness. However, laboratory tests (6) of these surfaces yield coefficients which are many times greater than that given by equation 2. Upon closer inspection of the cavitation inception of these surface, it is seen that the cavitation actually begins on individual projections from the surface. Therefore, correlations of the form presented for isolated roughnesses are more appropriate.

From these observations it is conjectured that roughness elements which have dimensions on the order of the thickness of the viscous sublayer fall in the area of distributed roughnesses. Isolated irregularities are then defined as any asperity which is large relative to the thickness of the viscous sublayer. This means that for practical cases any surface irregularity which has dimensions larger than 0.5mm would be classified as an isolated irregularity. Therefore, irregularities which are greater than about 0.5mm in height need to be analyzed as individual roughness elements.

3. Cavitation Inception at Changes in Alignment

All of the studies of cavitation inception at changes in slope have been conducted with a very thin boundary layer, Falvey (7). Therefore, the correlations are not presented as functions of Reynolds Number and relative boundary layer thickness, figure 4. It is interesting to note that the cavitation inception at a 90° offset is as severe as at a 1 to 1 sloping chamfer.

Studies of the effect of the boundary layer on the cavitation inception characteristic at changes in slope need to be conducted.

4. Vortex Cavitation

As a fluid flows in a curvilinear path secondary currents are generated. For instance, Schlichting (8) shows that for boundary layer flow on a concave wall, Goertler vortices are formed. Most spillway crests have a concave surface over which the flow passes and must therefore generate two or more longitudinal vortices. As the flow passes down the chute or the spillway tunnel these vortices are stretched due to the increasing axial velocity and decreasing flow depths. This process intensifies the rotational velocity of the vortex. If the rotational velocity becomes large enough, it can be expected that the core of the vortex will begin to cavitate. The cavitating core then only needs to come into a region of higher pressure such as the elbow of a tunnel spillway or the flip bucket of a chute spillway in order for the core to collapse. Evidence of this type of collapse has been seen in the

Yellowtail, Glen Canyon, and Hoover Dam tunnel spillways and in the Lucky Peak flip buckets.

Unfortunately, no studies are available to quantify this type of cavitation. Neither the onset nor the location of the cavitation can be predicted.

5. Damaging Cavitation

As cavitation begins to form due to surface irregularities, it occurs as small bubbles in the flow. These bubbles are usually located far enough away from the boundary that no damage occurs. If the reference pressure decreases or the flow velocity increases, the cavitation becomes more intense. More and more bubbles form and many of these begin to implode near the flow boundary. Light damage will now begin to form. As the velocity is increased further, the individual bubbles begin to form into swarms of bubbles or coalesce into larger bubbles. In either case, their collapse leads to ever increasing amounts of damage. Further increases in velocity result in myriads of bubbles which apparently start to absorb some of the energy of the collapsing swarms. When this happens the intensity of the damage decreases. This progression is evident in the studies of Stinebring (9), figure 5.

For a portion of the damage process it can be seen that the damage is approximately inversely proportional to the cavitation index. Damage has also been shown to be roughly proportional to the 6th power of the velocity (9). This leads to the conjecture that a damage potential parameter might exist of the form

$$D = \left(\frac{1}{\sigma_d}\right)\left(\frac{\sigma_d}{\sigma} - 1\right)\left(\frac{V}{V_r}\right)^6 \qquad (3)$$

where D = Damage Potential
σ_d = Cavitation Index at which Damage Begins
σ = Cavitation Index of the Flow
V = Average Flow Velocity
V_r = Reference Flow Velocity Corresponding to the Initiation of Cavitation with Zero Gage Pressure
$= \sqrt{\frac{p_a - p_v}{\rho\sigma_d/2}}$
p_a = Atmospheric Pressure
ρ = Density of Water

Although this idea has not been systematically studied in the laboratory, field observations indicate that the parameter might give some guidance as to when damage can be expected. For instance, damage has not been observed in tunnel and chute spillways when the damage potential is less than 1000. For this computation it was assumed that the incipient cavitation index could be substituted for the cavitation index at which damage begins and the $1/\sigma_d$ term could be omitted.

Research is needed to define the conditions at which damage begins and reaches a maximum for various types of isolated irregularities.

B. Air Entrainment

1. Effects of Air Entrainment

Several mechanisms to explain the effects of air entrainment on the mitigation of the damage have been proposed. As mentioned in the introduction, it was once assumed that the air could act as a cushion between the high-velocity flow and the flow boundary. This concept was soon proven to be false by investigations with physical models. Later as more was learned about cavitation and cavitation bubbles, it was assumed that the free air somehow got into the interior of the cavitation bubble and thus slowed or cushioned its collapse. This theory is also somewhat erroneous because there just is not enough time for significant amounts of air to diffuse into the cavitation bubble as it grows. In addition, the air that does diffuse into the bubble comes from the dissolved air in the water, not the free air in the form of bubbles. Finally, in examining the effect of air content on the sonic velocity, it can be seen that minute quantities of air have a dramatic effect on the sonic velocity. Because of this, it could be reasoned that the air-water mixture is much more compliant than the water alone. This property tends to absorb the energy of the collapsing cavitation bubbles, preventing large pressure intensities from developing at the flow boundary. It appears that this latter explanation is probably the most valid.

Studies by Peterka (10) showed that minute amounts of air in the flow would decrease or completely eliminate damage to concrete in a venturi type cavitation erosion facility. It is primarily on the basis of these studies that extensive efforts are being made to entrain air into high-velocity flows of hydraulic structures.

2. Air Entraining Mechanisms

For many years it has been recognized that it was necessary for tubulence to be present at the air-water interface for air entrainment to begin. However, attempts to find correlations for the turbulent intensities necessary to entrain air have been unsuccessful. One of the more popular explanations of the air entraining mechanisms is that water droplets are ejected from the water surface. As they fall back into the water, they entrain bubbles of air, (11). This explanation appeared to be very satisfactory until aerators began to be designed. Then it was noticed that large quantities of air were being entrained on the underside of the nappe exiting from the ramp of the aerator. It was argued, using the water droplet analogy, that spray was the cause of the air entrainment. However, Pinto (12) showed with model studies that turbulence was the primary air entraining mechanism, not spray. Ervine and Falvey (13) have further argued that not only is turbulence a necessary condition, but the scale of the turbulence is also an essential factor.

Once the air is in the water it begins to detrain due to buoyancy. However, turbulence tends to keep the air diffused throughout the body of the water. The laws which govern the balance between the detrainment and the diffusion of the bubbles are not known. This is an area requiring urgent research since the spacing of the aerators is dependent upon the air content of the water near the boundary.

C. Damage Mechanism

It is clear that cavitation initiates the damage to the flow surface. Whether this initial cavitation damage is primarily caused by surface irregularities or by vortices within the flow is not clear at this point. Probably both mechanisms have been responsible for some of the spectacular damage that has been observed. In any case, once the concrete surface has been broken by the cavitation damage, other mechanisms come into play (1). The high velocity flow can now impinge upon the surface irregularities such as exposed pieces of aggregate, joints, etc. The kinetic energy of the flow will be converted to pressure head at these points. Once this happens, the pressures are transmitted through weaknesses in the concrete matrix leading to damage from fracturing. The pressure differentials which develop in the concrete cause large pieces of concrete to be broken from the surface. The process continues until the concrete liner is penetrated. Penetration of the concrete exposes the foundation material to the action of the high velocity jet. Subsequent damage depends upon the strength of the foundation material.

At Tarbella, the point of penetration occurred in fill material and erosion of the fill continued until complete failure of the structure occurred. In contrast to this, at Glen Canyon Dam the erosion stopped after a large enough cavity was eroded in the tunnel invert to dissipate the energy of the high velocity flow. Here the foundation material was sandstone. However, more erosion occurred in the left tunnel than in the right because of a fault located in the sandstone of the left abutment at the point of penetration.

Major damage occurs at a very fast rate having a time scale of hours to days. For instance, at Glen Canyon Dam, loud rumblings and major damage were observed after only four days of operation at discharges which were less than 10-percent of the design flow (14).

DESIGN

A. Location of Aerator

The location of the aerators is somewhat critical. The first aerator should be placed above the location where the surface could be damaged from cavitation. This point is best determined with a computer program which can calculate the cavitation index of the flow and the value of the damage potential for various types of surface irregularities (15). The use of these two values can be used to estimate a safe location for the aerator.

One criterion for the location of the aerator is that it must be upstream of any irregularity which would cause incipient cavitation. This would guarantee that no damage would occur. However, from a practical standpoint, this criterion is much too restrictive.

Field observations have indicated that damage has not been observed for flow cavitation indices greater than 0.2, figure 6, (7). This criterion, while a good guideline, does not take into account the type of irregularity nor the approximately sixth power effect of the velocity on the damage intensity.

A better criterion, when it is developed, is the damage potential. This parameter has the prospect of being able to include the effects of the type of the irregularity and the velocity of the flow. In its present form, it appears that if the value of the damage potential is less than 1 000, then damage of the flow surface will not occur.

The flow cavitation index and the damage potential guidelines are sufficient, with a little engineering judgement, to select a location for the most upstream aerator which will be free from cavitation damage.

The location of subsequent aerators is presently a point of much discussion. Some investigators recommend they be placed as close together as 30m, others recommend 100m. Both of these judgements are based upon estimates of the rate of detrainment and neglect the diffusion effects of turbulence in the flow. As a point of reference, with the 72m and 90m spacings on the Foz do Areia spillway, the flow was still bulked by 80-percent at a distance of 69.5m from the most downstream aerator (12). From this it appears that even the 100m criterion may be overly conservative.

Installation of an aerator in a vertical bend which is concave upward is not recommended. Centrifugal forces from the curvilinear flow make this location behave differently than locations on a uniform slope. Usually, an aerator can be made to function properly only for a limited range of discharges. At other flow rates, the aerator will become submerged and thus it is converted into a source of cavitation which may be worse than the irregularities which were present without the aerator.

B. Aerator Configurations

Aerators consist of various combinations of three elements, a deflector, a groove, and an offset of the downstream flow surface relative to the upstream alignment, figure 7, (16). The purpose of the deflector or ramp is to lift the flow away from the surface which allows the under side of the nappe to become aerated. Separating the flow from a surface is also of benefit because the flow surface under the free nappe does not need to be constructed with a great deal of care relative to the surface finish. The purpose of the groove is to allow the unrestricted flow of air to the underside of the nappe. This is especially important for deflectors which have a small dimension normal to the flow. Finally, the purpose of the downstream offset is to prevent the drowning out of the groove and the downstream face of the deflector for aerators which are placed on relatively flat slopes. When a free jet strikes a downstream surface, a portion of the jet flows upstream. The extent to which this upstream flow interferes with the operation of the aerator is a funtion of the angle of the slope and the angle the centerline of the jet makes with the downstream slope. Small slope angles and large angles of jet impact can cause problems. These negative effects can often be eliminated with offsets.

C. Aerator Dimensions

The significant dimensions to be determined for an aerator are the height of the the deflector, the angle the deflector makes with the upstream slope, the size of the groove, and the magnitude of the offset.

The height of the deflector, the angle the deflector makes with the upstream slope, and the magnitude of the offset determine how far the jet travels before it stikes the downstream slope. In general, it is desirable to make the distance between the aerator and the point of impact as large as possible since the quantity of air entrained is a function of the length of travel of the free jet.

In deciding how far to maintain a free jet, conditions at the impact point must be considered. In general, the jet should not impact in a vertical curve. This condition creates fins, spray, and a very rough water surface. For similar reasons, the angle between the free jet and the downstream chute invert should be small. If the ramp slope is too large, the free jet can be projected above the walls of the chute or high enough to touch the crown of the conduit in the case of tunnel spillways. Both of these conditions should also be avoided.

Mathematical relationships using trajectory equations can be used to estimate the distance to the impact point. However, two factors complicate this rather simplistic approach. The entrainment of air underneath the nappe creates pressure differential across the nappe which decreases the length of the trajectory. Secondly, if the ramp height is small relative to the depth of flow, the ramp will not affect the entire thickness of the nappe. This too results in decreased jet trajectories. Several methods are available to account for these effects.

One approach is that taken by DeFazio and Wei (17) who use a finite element program to calculate the free jet profile. This program takes into account both effects. Another approach is to develop a set of coefficients which take into account the relative height of the offset and the Froude Number of the flow. The coefficients are then applied to the theoretical trajectory path to determine the actual impact point. This approach was taken by Pan et al (18). Finally, Glazov (19) neglected the effect of the relative ramp height, but included the effects of the pressure differential and the offset through the direct use of equations which modify the theoretical jet trajectory equations. The unique part of Glazov's study is that he calculates the quantity of the air entainment through the use of turbulence considerations. The results of his analysis agree very well with prototype measurements from Foz do Aeria. This is the first case of a phenomenological description of the flow predicting prototype measurements without resorting to empirical observations.

The design of the groove must be coupled with the design of the air supply to the aerators. Therefore, these details are considered separately.

D. Air Supply to Aerators

There are several methods by which air is ducted from the atmosphere to the underside of the nappe, figure 8. These include

- piers in the flow
- slots in the sidewalls
- offset sidewalls
- ramps on the sidewalls
- ducts through the sidewall
- ducts under the ramp
- ducts under the offset.

Ramps, offset sidewalls, and piers in the flow are frequently used to supply air to aerators downstream of control gate structures. With slide gates, the offset sidewall or ramp height should be equal to one-twelfth the width of the gate but not less than 100mm (20). Similarly, with piers, the pier width should be one-sixth of the gate width. Normally these types of air ducts are not used on spillways because the required offsets are impractical from a design viewpoint.

Slots in the walls are also used in control gate structures. This solution lends itself to cases where installation in an existing structure is required. The downstream end of the slot may be offset and deflectors are frequently used to keep the water from entering the slot. If the cross sectional area of the slot is too small, water and spray will be pulled into the high-velocity air stream. The result will be insufficient air to protect the flow surface on the chute floor.

Ducts through the sidewall are used on wide chutes when the required slot size or sidewall offsets are excessive. In areas where freezing is a problem, ducts are routed through the embankment. This prevents the formation of ice plugs in the duct during times when water may be standing in the chute area.

A duct under the ramp is used on very wide chutes or in installations where a hydraulic jump may cover the ramp. In both of these cases, the system of ducts and vents ensures adequate aeration of the undernappe of the jet. The venting system should be sized so that the cross sectional area of the duct is larger than the total cross sectional area of all the vents. In addition, the velocities through the vents should be limited to a velocity of less than 100m/s.

A duct under an offset is used when the ramp height is too small to allow adequate venting. This scheme also simplifies construction. However, a drain for the duct must be provided to keep the duct free of water. Leakage and extremely low flows would tend to fill the duct of chutes having small slopes if drainage were not provided. During normal flows in the chute, air will enter not only the duct, but also through the supplementary drainage.

In circular conduits which flow part full, a duct or groove is used around the circumference of the tunnel. The ramp height is tapered from a maximum value at the invert to zero thickness at the depth which corresponds to the maximum flow rate in the structure. The tapered ramp prevents the formation of fins or "rooster tails" in the downstream conduit at high flow rates. Filling of the duct is not a problem with circular conduits on steep slopes because only a small portion of the groove can be filled with water.

The design of the air ducting system is an iterative procedure. The air flow rate is determined from the jet trajectory equations. Once the air flow rate is known, it is possible to calculate the pressure drop across a given size air duct system. Using the calculated pressure drop, it is necessary to once again determine the air flow rate using the jet trajectory equations including the pressure drop effect. The process continues until the variation in the pressure drop between successive steps is small.

SUMMARY

Aeration provides a very practical method for the protection of spillways from cavitition damage. The aeration can be provided relatively cheaply through the use of devices called "aerators". With these the aeration is provided hydrodynamically, thus eliminating the need for air compressors or other mechanical methods of putting the air in the flow.

Although much is known about the design of aerators, additional research is still needed in several areas. The three most critical areas needing study are;

1) the properties of cavitating longitudinal vortices in the flow,
2) the dispersion of air bubbles throughout the flow by turbulence, and
3) the prediction of damage to the flow surface.

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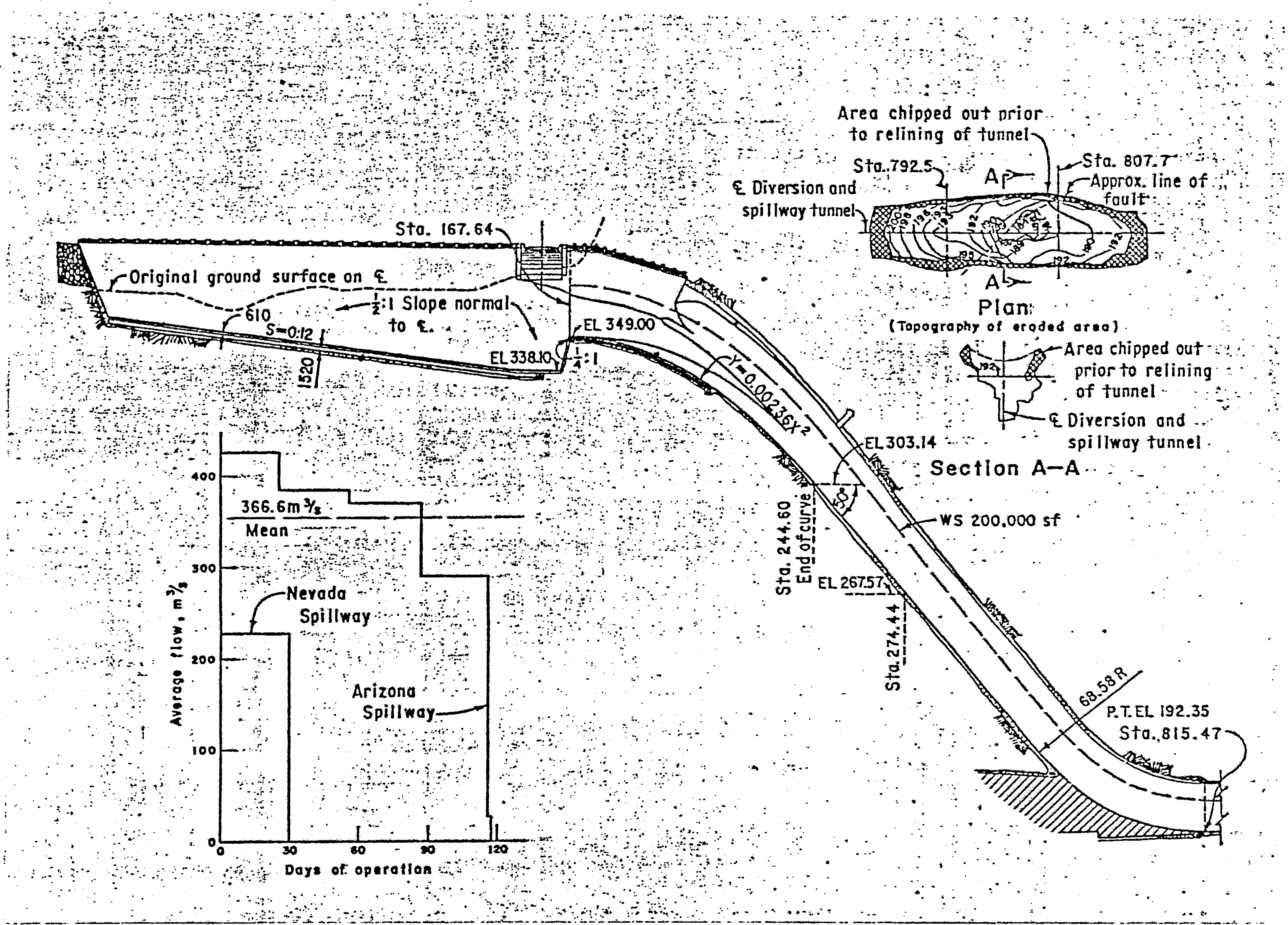
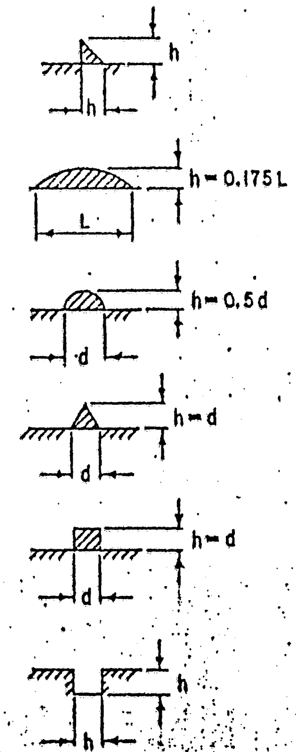
FIGURES

FIGURE 1. DAMAGE AT HOOVER DAM



Symbol	Irregularity	Flow dimensions	a	b	C
△	Triangles	2	0.361	0.196	0.152
○	Circular arcs	2	0.344	0.267	0.041
▲	Hemispheres	3	0.439	0.298	0.0108
◉	Cones	3	0.632	0.451	0.00328
▣	Cylinders	3	0.737	0.550	0.00117
⊔	Slots	2	0.041	0.510	0.000314



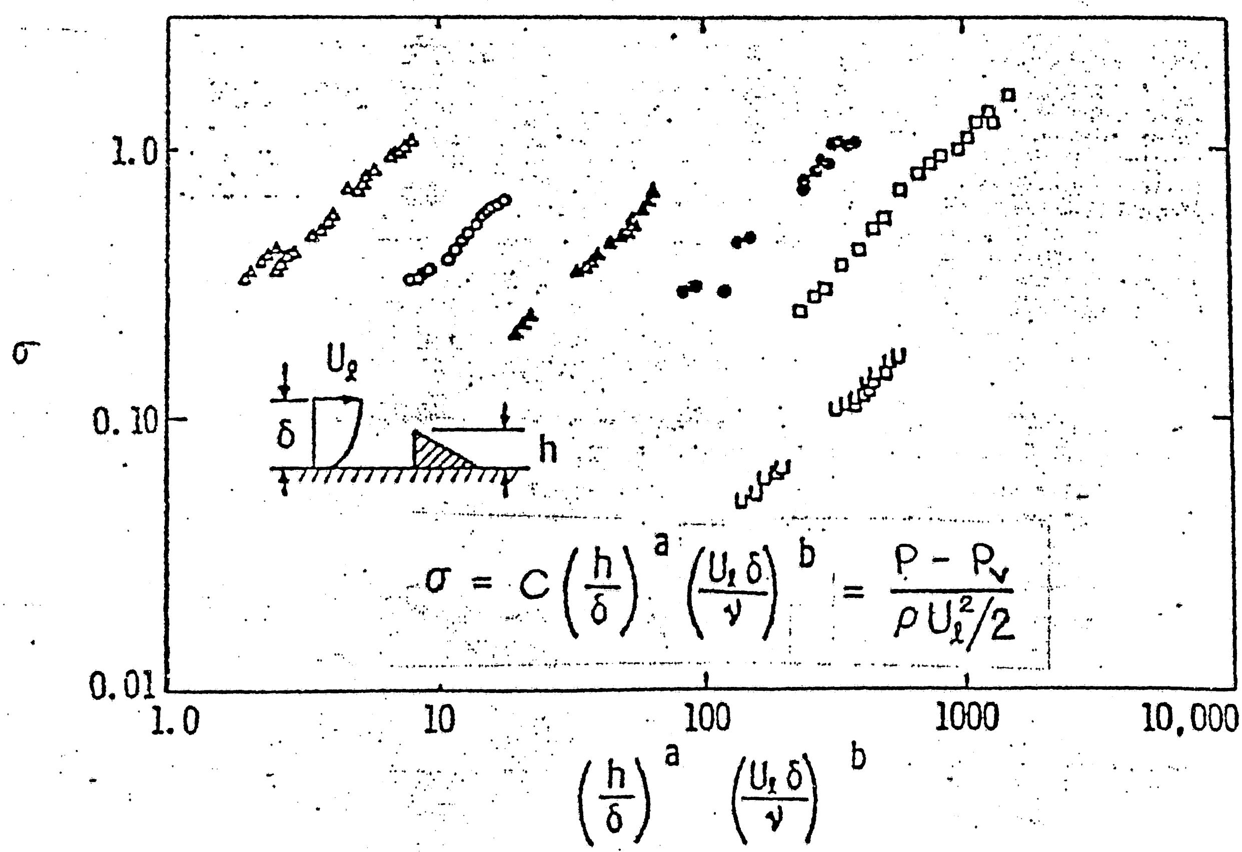


FIGURE 2. CAVITATION CHARACTERISTICS AT ISOLATED IRREGULARITIES

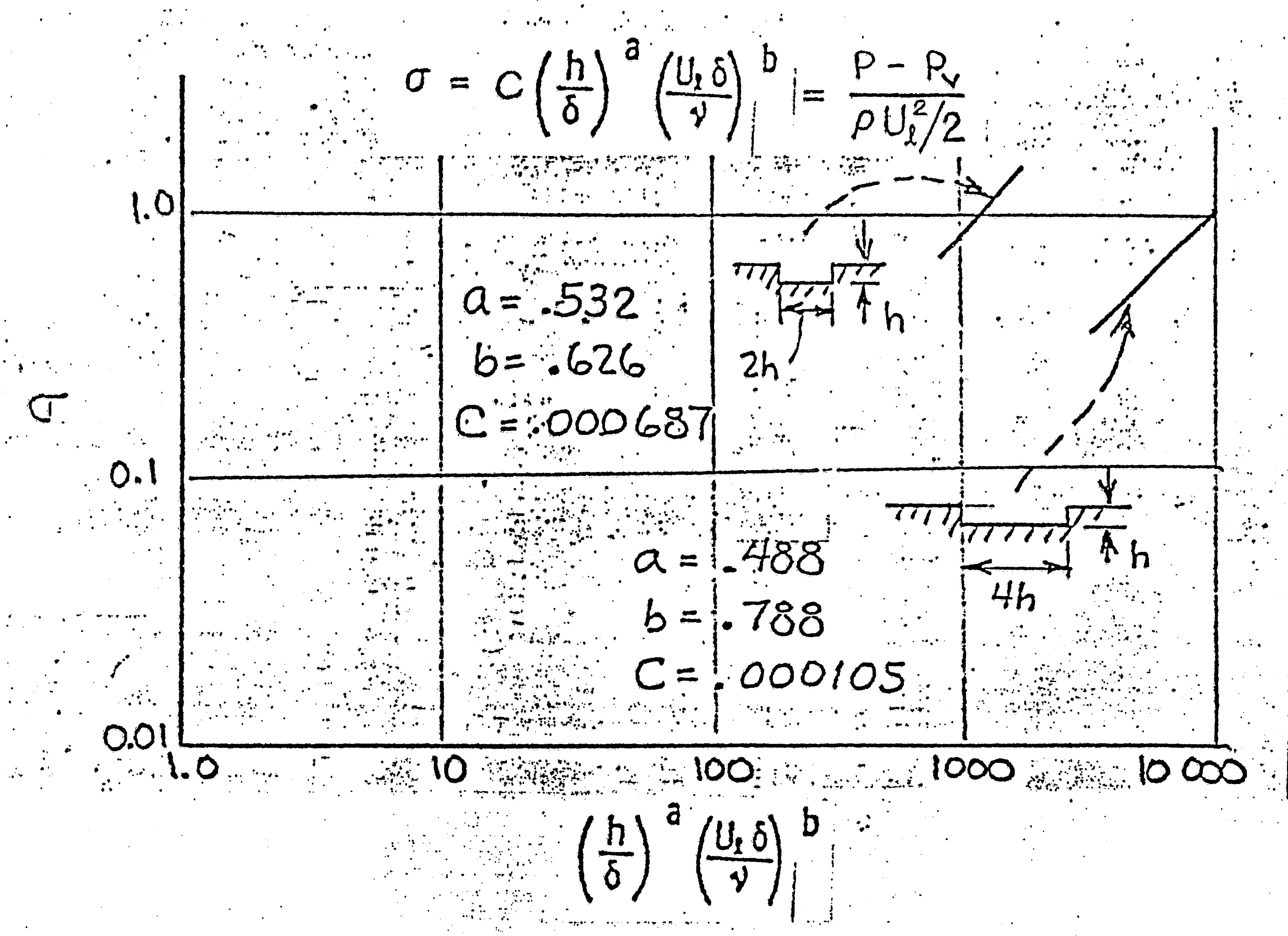


FIGURE 3. CAVITATION CHARACTERISTICS AT HOLES IN A BOUNDARY

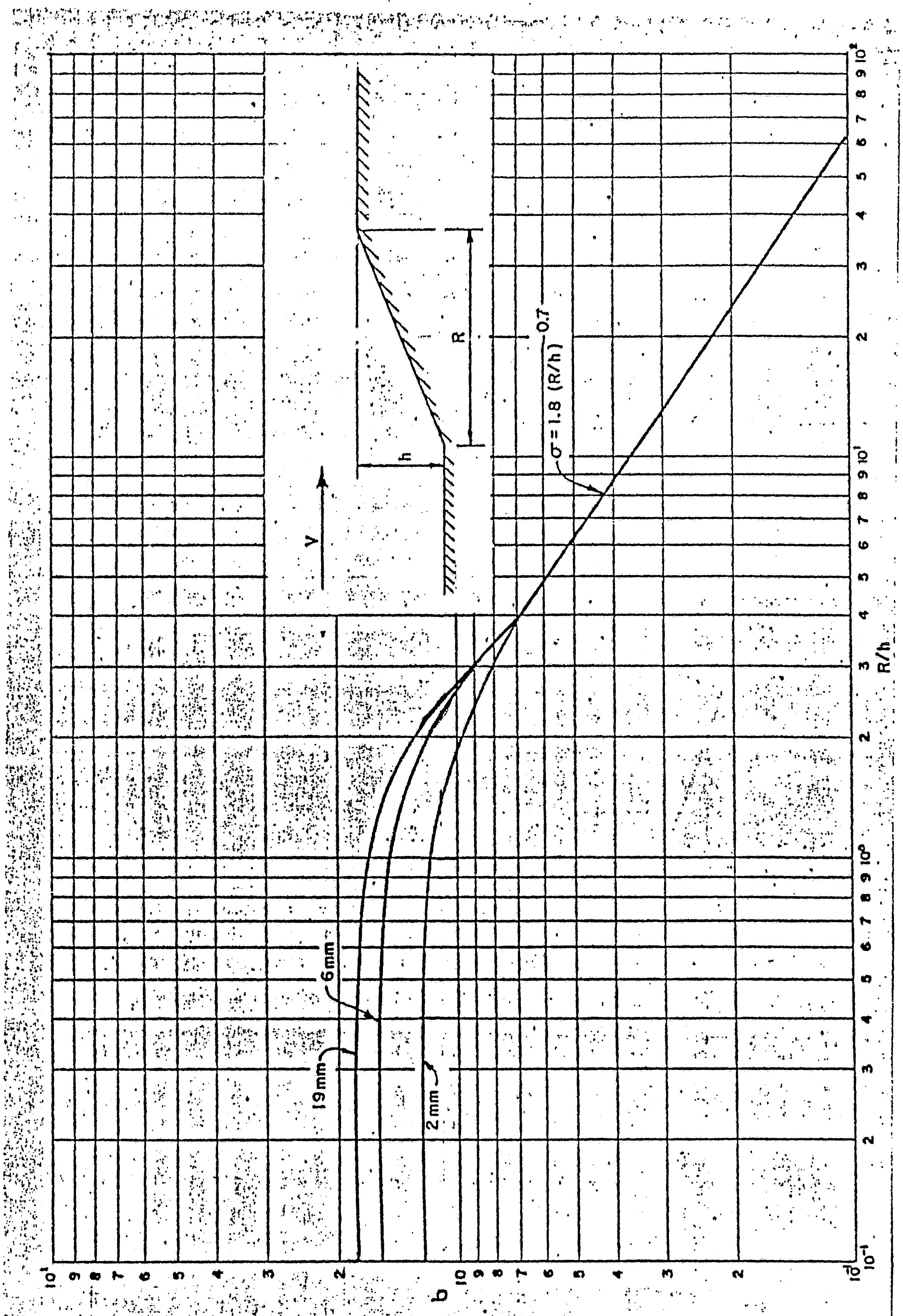


FIGURE 4. CAVITATION CHARACTERISTICS AT CHANGES IN ALIGNMENT

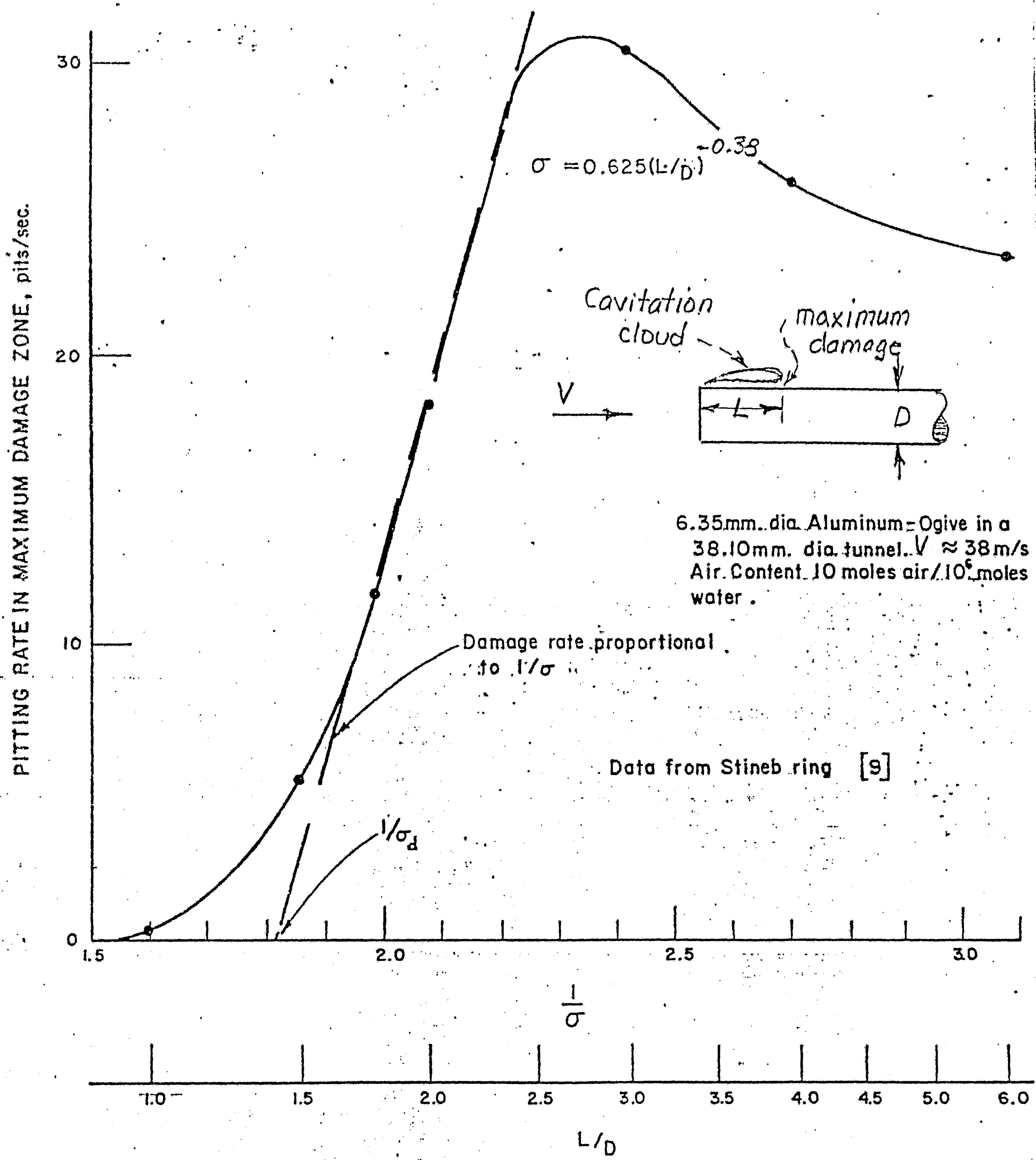


FIGURE 5. CAVITATION DAMAGE RATE AS A FUNCTION OF CAVITATION INDEX

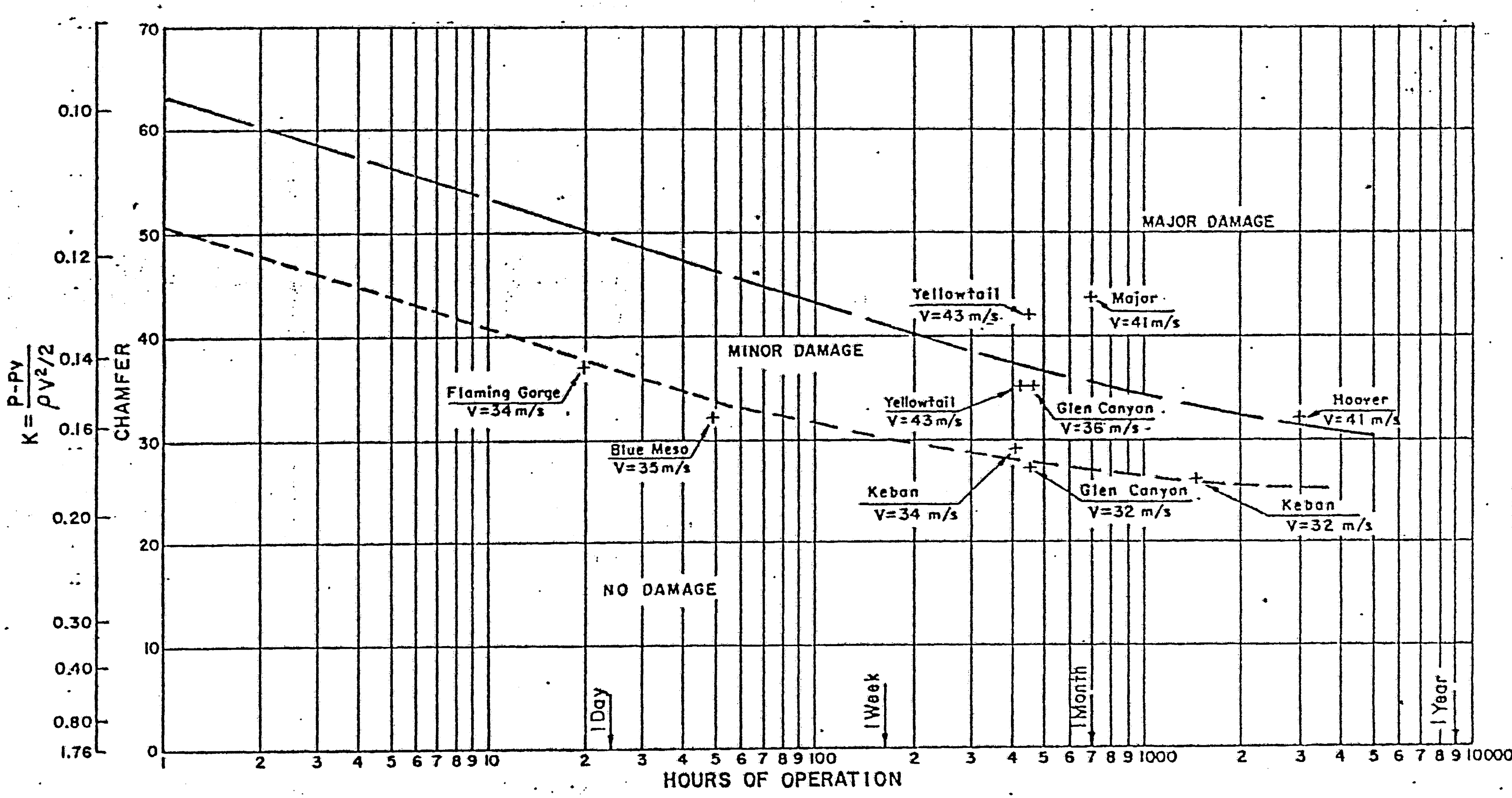


FIGURE 6. DAMAGE ON CHUTE AND TUNNEL SPILLWAYS

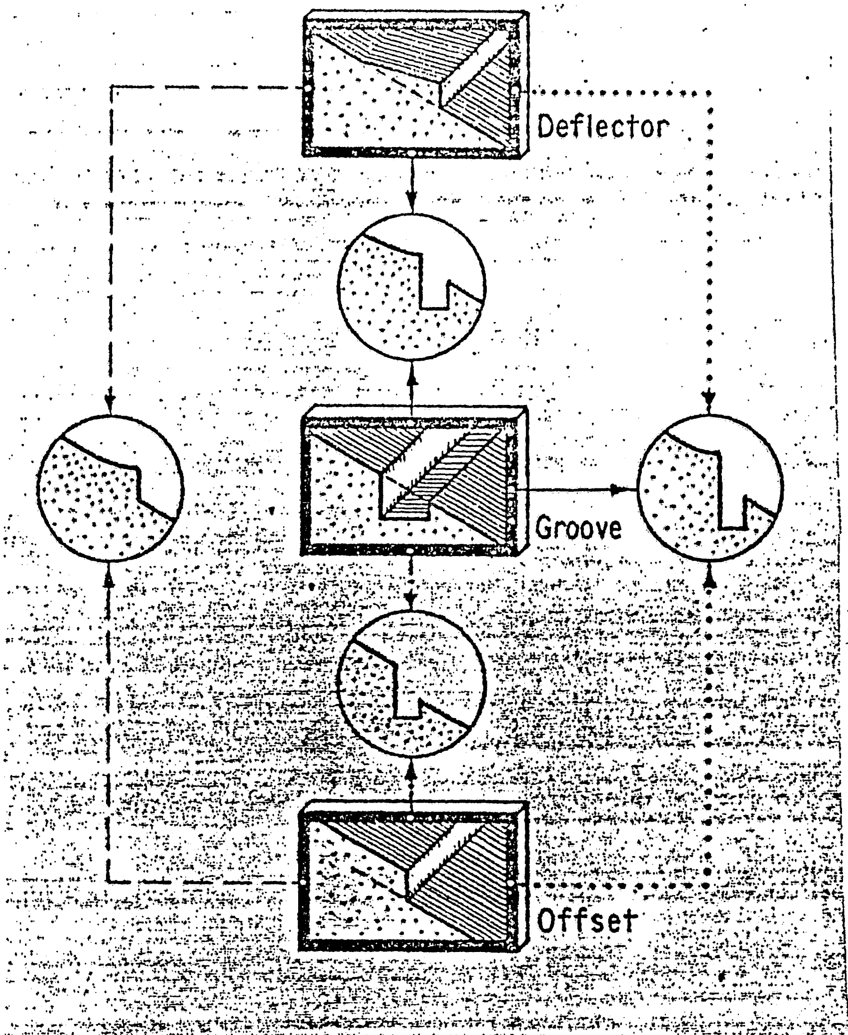


FIGURE 7. AERATOR CONFIGURATIONS

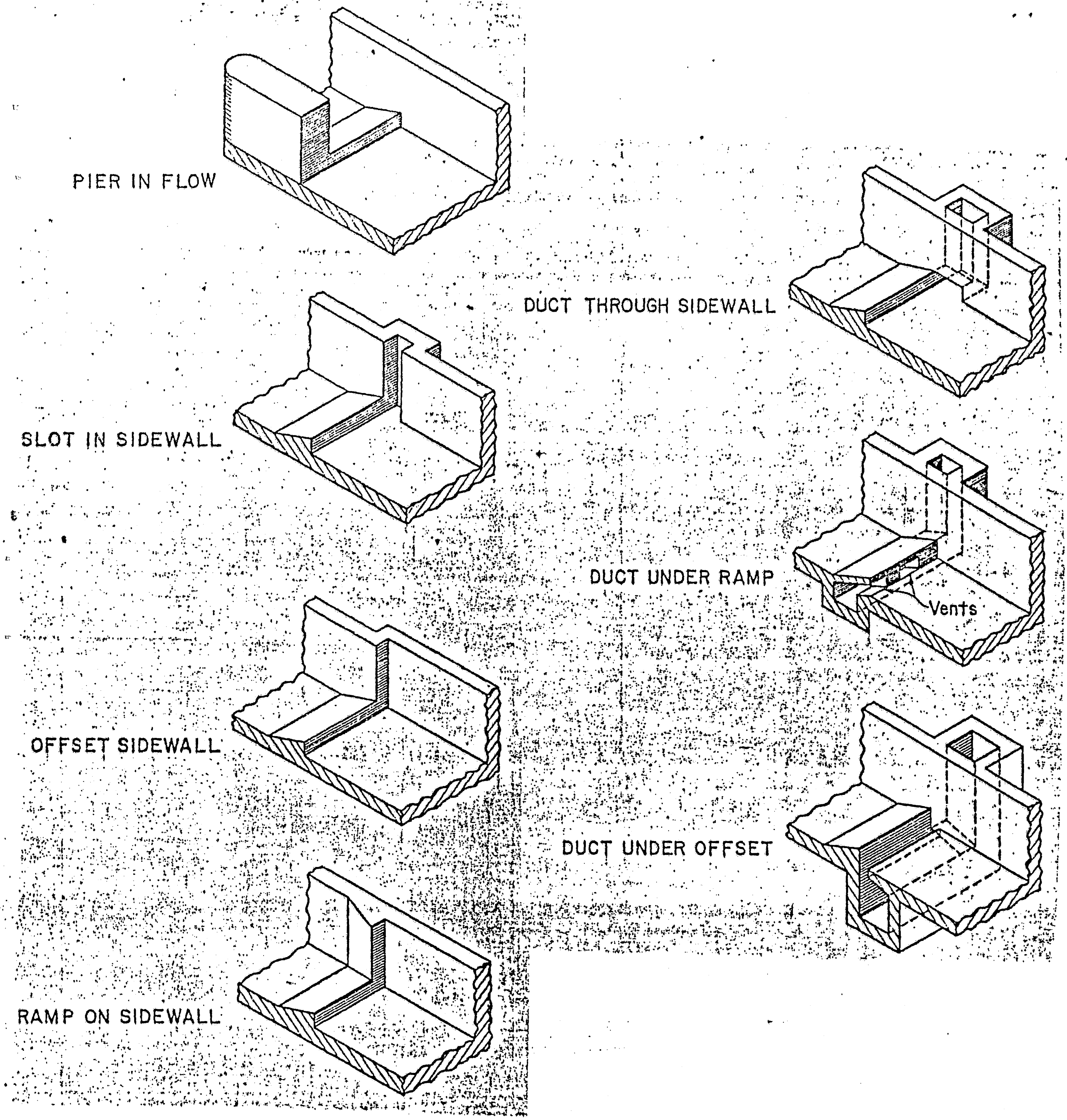


FIGURE 8. AIR SUPPLY CONFIGURATIONS

APPENDIX A

Protocol
of the meeting between
Prof. Roger E. A. Arndt, Director of the Hydraulic
Laboratory of the University of Minnesota
in Minneapolis and the representatives of the Hydraulic
Engineering Research Institute (ICH)
Bucharest, Romania

July 28, 1984

P r o t o c o l

of the meeting between
Prof. Roger E. Arndt, Director of the Hydraulic
Laboratory of the University of Minnesota
in Minneapolis and the representatives of the Hydraulic
Engineering Research Institute (ICH)
Bucharest, Romania

In the framework of the bilateral programme for scientific exchanges under the terms of the NSF/NCST programme, Prof. Roger E.A. Arndt, Director of the Hydraulic Laboratory of the University of Minnesota in Minneapolis made a visit to ICH between 22-28 July, 1984, with a view to investigating the possibilities of developing a cooperative research program in the area of hydraulic engineering research.

During his stay, prof. R.E.A. Arndt visited the hydraulic laboratories in ICH and got acquainted with the main aspects and problems that make the object of the ICH studies, and the Civil Engineering Institute (ICB) and its hydraulic laboratories where he held discussions with the teaching staff there concerning the aspects of technical education.

In this period prof. R.E.A. Arndt made a technical tour to the dams and hydro-power stations Paltinu, on the river Doftana and Vidraru, on the Argeş river, and to the dams Săcele and Mâneciu.

In the last day of his stay prof. Roger E.A. Arndt delivered a short lecture on the general activity of his laboratory and on problems of cavitation in the front of the research workers in ICH and held discussions on this basis.

As a result of the discussions held during the visit, between prof. R.E.A. Arndt and prof. S.Hâncu, director of ICH an initial outline of a conference was developed.

The aim of the conference will be to identify areas of mutual interest for collaborative exchange in the future. In order to initiate this venture in a mely manner the topics co idered will be limited to the following:

1. Cavitation in Hydraulic Structures:
   - Cavitation scaling
   - Measurement techniques
   - Mitigation of cavitation via aeration including any adverse effects of aeration

2. Modelling of Sedimentation:
   - Theory of modelling including use of air models
   - Extrapolation to prototype experience

3. Computational Modelling
   - Application of Numerical modelling to a broad range of hydraulic engineering problems

The format of the conference will consist of overview lectures on each topic followed by discussion of the state of the art in that field with emphasis on the identification of remaining problem areas. It is expected that one Romanian and one American specialist present a lecture in each of the cited areas. Abstracts of the lectures would be made available to all participants before the conference. A summary of the conference will be prepared which will include a summary of the state of the art in each of the cited areas and suggestions for future cooperation in these areas. The summary is to be afterwords published in both languages.

A target date of the conference would be July, 1985.

The present protocol is to be approved by the Division of International Programs, National Science Foundation on the American part and by the National Council for Science and Technology, on the Romanian part.

Roger E.A. Arndt	Simion Hâncu
Director of the Hydraulic Laboratory University of Minnesota Minneapolis	Director of the Hydraulic Engineering Research Institute

Bucharest, July 28, 1984

APPENDIX B

Protocol

P R O T O C O L

OF THE ROMANIAN AND AMERICAN SPECIALISTS

FOR THE

"ROMANIAN-AMERICAN WORKSHOP ON HYDRAULICS"

The workshop took place in Bucharest, Romania, July, 21-28, 1986 in the frame of the Cooperation Memorandum between Romanian National Council for Science and Technology and U.S. National Science Foundation (July, 1984).

Participants:

Romanian specialists:

1. Prof. S. Hâncu - Hydraulic Engr. Research Inst.
2. Dr. V. Jeler - Hydraulic Engr. Research Inst.
3. Dr. D. Duma - Hydraulic Engr. Research Inst.
4. Dr. D. Bătucă - Hydraulic Engr. Research Inst.
5. Dr. I. Jelev - Hydraulic Engr. Research Inst.
6. Dr. A. Spătaru - Hydraulic Engr. Research Inst.
7. Prof. C. Iamandi - Civil Engr. Inst.Bucharest
8. Prof. M. Popovici - Polytechnical Inst. Timişoara
9. Prof. I. David - Polytechnical Inst. Timişoara
10. Prof. P. Roman - Polytechnical Inst.Bucharest
11. Dr. T.Stoicescu - Hydropower Studies and Design Institute

American specialists:

1. Prof. A. Acosta - California Inst. of Technology
2. Prof. R. Arndt - Univ. of Minnesota
3. Prof. H. Falvey - Bureau of Reclamation Denver
4. Prof. N. Katopodes - Univ. of Michigan
5. Prof. J. Kennedy - Univ. of Iowa
6. Prof. J. Liggett - Cornell Univ.
7. Prof. S. Martin - Georgia Inst. of Technology
8. Prof. J. Odgaard - Univ. of Iowa
9. Prof. G. Parker - Univ. of Minnesota

Other Romanian participants:from the Hydraulic Engineering Research Institute (ICH) Bucharest, the Civil Engineering Institute (ICB) Bucharest, the Polytechnical Institutes of Bucharest (IPB), of Timişoara (IPT) and of Iassy (IPI), the Hydroenergetic Studies and Design Institute (ISPH) Bucharest and the Research and Design Institute for Water Resources Engineering (ICPGA) Bucharest.

The workshop took place in good conditions, permiting an useful exchange of experiences and also a better reciprocal knowledge of the solving level of the problems, which formed the topics, in both countries.

The Romanian and American specialists have decided as it follows:

1. To propose to the high Romanian and American authorities to establish and develop a scientific cooperation programme on hydraulics between the similar institutes of both countries on the following topics:

   i) Mathematical modelling of sediment transport, of river bed aggradation and deggradation and of density currents in reservoir.
   Elaboration of some mathematical models of sediment transport, as applied to the Danube River.
   The cooperation on this subject may be extended with the VITUKI Institute from Hungary.

   ii) Transient phenomena in closed conduits for special conditions of two-phases flows (water-air).

   iii) Numerical simulation of the unsteady flow during catastrophic floods (dam breaks, mud flows).

   iv) Cavitation phenomena. Topics of importance are: a) the mechanism of cavitation inception in hydraulic structures including the effects of surface roughness, turbulence and secondary, vortical flows, b) the mitigation of cavitation problems through the use of aeration as well as other techniques and c) physical modelling of turbines with the aim of identifying, in detail, cavitation erosion mechanisms and methods for eliminating cavitation erosion.

The forms of cooperation can be:

- exchanges of papers, reports and computational programmes;
- research programs based on common-established programmes;
- working visits of the specialists in reciprocal conditions.

After the approval of the cooperation topics the both sides will establish and propose the detailed work programme for each involved institution.

2. Lectures held during the workshop debates will be summarized and published by the end of this year.

The Polytechnical Institute of Timişoara received a technical film on cavitation from the California Institute of Technology.

The Romanian participants very much appreciated the technical level of the lectures held by the American specialists and their extreme usefulness and express their thanks to the American colleagues.

This present protocol is to be approved by the U.S. National Science Foundation and by the Romanian National Council for Science and Technology.

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Bucharest, July 28, 1986

UNIVERSITY OF MINNESOTA

ST. ANTHONY FALLS HYDRAULIC LABORATORY

Project Report No. 258

U.S.–ROMANIAN WORKSHOP ON WATER RESOURCES ENGINEERING

VOLUME II

Prepared for

NATIONAL SCIENCE FOUNDATION
Washington, D.C.
Grant No. NSF/INT–8506630

May 1987

Minneapolis, Minnesota

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Paper	Author
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