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HYDRAULIC LABORATORY REPORT NO. 110

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FINAL REPORT
OF
FREEMAN TRAVELING SCHOLARSHIP
IN HYDRAULICS FOR 1940-1941
TO
AMERICAN SOCIETY OF CIVIL ENGINEERS

By

HAYWOOD G. DEWEY, JR., JUN. AM. SOC.
C. E. ASSISTANT ENGINEER, BUREAU OF RECLAMATION

- - -

Denver, Colorado,
April 1942

FOREWORD

The Freeman Fund was first established in 1924 by the late John R. Freeman, Past President and Honorary Member, American Society of Civil Engineers, who presented the Society securities, the income from which has been used in the aid and encouragement of young engineers, especially in research work; for underwriting the cost of publications on hydraulic science; or assisting in the translation or publication in English of works in foreign languages pertaining to hydraulics.

One of the principal benefits from this fund has been the Freeman Traveling Scholarship in Hydraulics which has been awarded nearly every year since 1924. The recipient is entitled to travel in Europe to study hydraulic laboratory practice. Because of the present war, the author confined his travel to the United States, the thirteen previous Scholars having traveled in European countries.

The author is deeply grateful to the Society for being selected as the Freeman Scholar for 1940-1941, and to those who gave recommendations and encouragement. The cooperation of those who gave so generously of their time during the author's visits to hydraulic laboratories and projects in the field is greatly appreciated. Particular thanks are due the Bureau of Reclamation, Denver, Colorado, for granting the author leave of absence during the scholarship, and for preparing this report for publication.

The information contained in this report does not necessarily represent the opinions of the American Society of Civil Engineers or of any other organizations referred to. In general, the opinions expressed are those of the author except where reference is made to another's work.

CONTENTS

SECTION PAGE

CHAPTER I - INTRODUCTION AND SUMMARY

Introduction

1. Field of study 1
2. Scope of report. 2

Summary

3. Problems confronting hydraulic engineers 3
4. Solution of problems by research 3
5. General conclusions. 5

CHAPTER II - THEORY AND APPLICATION OF HYDRAULIC MODELS

General

6. Hydraulic models and fundamental research. 7
7. History of model development 7
8. Purpose of models. 8
9. Types of models. 8

Models of Hydraulic Structures

10. Adapting the model 9
11. Selecting a scale ratio. 11
12. Preliminary design considerations. 12
13. Materials used and methods of construction 13
14. Testing procedure. 21
15. Measurements 22
16. Report of model tests. 24

River and Harbor Models

17. Adapting the model 24
18. Selecting a scale ratio 31
19. Preliminary design considerations. 32
20. Materials used and methods of construction 33
21. Testing procedure 34
22. Report of model tests. 36

Turbine Models

23. General. 36
24. Use of model 37
25. Comparison with other types of model tests 39

SECTION PAGE

Turbine Models - Continued

26. Construction of model. 39
27. Testing procedure 40

Pump Models

28. General. 43
29. Use of model 44
30. Cavitation tests 45
31. Model construction and laboratories. 47

- - - - -

32. Bibliography 47

CHAPTER III - HYDRAULIC MODEL TESTS

Introduction

33. General. 49

Friant Dam

34. The prototype. 50
35. Crest calibration - 1:25 scale model 50
36. Stilling pool - 1:24 scale model 50
37. River outlets - 1:24 scale model 54
38. Study of general performance - 1:60 scale model. . 58
39. River outlet stilling pool - 1:34.375 scale model. 61
40. Madera and Friant-Kern Canal headworks 63
41. Aeration of coaster gates - 1:17 scale model . . . 65
42. Needle valves - 1:18.33 scale model. 69

Check Drop 4

43. The prototype. 72
44. The model. 73
45. Tests on model 73
46. The recommended design 74
47. Inspection of structure in field 74

Outlet Entrances - Madden Dam and Grand Coulee Dam

48. Problem involved 76
49. Test methods 76
50. Bellmouth design 78

SECTION PAGE

John Martin Dam

51. The prototype 79
52. Purpose of model study 80
53. The model 80
54. Model measurements 82
55. Test procedure 82
56. Summary of results 83

Mississippi River Flood Control Model

57. The model 84
58. Similitude 85
59. Method of testing 86
60. Results of tests 87

Head of Passes Model Study

61. The problem 87
62. The model 89
63. Verification tests 90

Galveston Bay

64. The prototype 91
65. Need for model tests 92
66. The model 92
67. Scope of tests 92
68. Clear-water base test 93
69. Example of clear-water test 93
70. Shoaling tests 94
71. Recommendations 95

Hydraulic Machinery

72. References 95

Laboratories

73. Model testing 96

CHAPTER IV - FUNDAMENTAL RESEARCH

Introduction

74. Empiricism versus rationalism 97
75. Principal research 99

SECTION PAGE

Fluid Turbulence

76. Nature of turbulence 99
77. Statistical theory 101
78. Turbulence and energy 104
79. Diffusion 105
80. Energy dissipation 106
81. Summary 108

Turbulence Concepts Applied to Sedimentation

82. Transportation of suspended material 109
83. Calculation of suspended sediment 111
84. Relation of suspended to bed material 118
85. Validity of assumptions 120

Density Currents

86. Relation to suspended sediment 124
87. Silting of reservoirs 125
88. Analysis 127
89. Field study 128

Meandering of Streams

90. Type of problem 130
91. Characteristics of meandering streams 131
92. Laboratory investigations 131

Academic Research

93. Open channels 133
94. Stream double refraction 136

Laboratories

95. Research 138

CHAPTER V - FIELD INVESTIGATIONS

General

96. Purpose 140
97. Scope and method of investigations 140

Spillways

98. Types 142
99. Overfall 142

SECTION PAGE

Spillways - Continued

100. Chutes 161
101. Tunnels. 163

Outlet Works

102. Types 165
103. Control gates upstream 166
104. Regulating valves downstream 166

Stilling Pools

105. General 168
106. Types used in relation to tailwater. 168
107. General design rules 173

Irrigation Structures

108. Types 175
109. Main dam 176
110. Diversion dam. 176
111. Trapezoidal and rectangular stilling pools at drop structures 177
112. Flow measurements 178
113. Wasteway 178

Model-Prototype Correlation

114. Summary 179

- - - - -

APPENDIX I

115. Itinerary. 1
116. Progress reports 5

APPENDIX II

117. Expenses 44

LIST OF FIGURES

FIGURE	TITLE		PAGE
1.	Friant Dam	(facing)	51
2.	Sloping apron		52
3.	Hydraulic jump on sloping apron		53
4.	General plans and sections - river outlets	(facing)	54
5.	Transition and trough		55
6.	Jet spreading on apron		57
7.	Sand on apron		60
8.	River outlet stilling pool		64
9.	Model of final design	(facing)	64
10.	Madera Canal headworks		64
11.	Friant-Kern Canal headworks		65
12.	Details of coaster gate	(facing)	67
13.	Needle valve model		70
14.	Model of existing structure at Check Drop 4	(facing)	73
15.	Check Drop 4 before revision - model and prototype	(facing)	74
16.	Model of recommended design		75
17.	Check Drop 4 after revision - model and prototype	(facing)	75
18.	Sluice entrance and trashrack model tests-Grand Coulee Dam	(facing)	80
19.	Spillway of John Martin Dam		80
20.	Model of spillway		81
21.	Mississippi River flood control model		85
22.	Passes of the Mississippi River - vicinity map		88
23.	Model of Head of Passes		89
24.	Passes of the Mississippi River		91
25.	Suspended sediment in Mississippi River		115
26.	Velocity distribution		115
27.	Beginning of underflow		126
28.	Underflow in model reservoir	(facing)	127
29.	Silt drawn-off by outlets	(facing)	127
30.	Incomplete mixing below confluence		130
31.	Meandering streams	(facing)	132
32.	Spillways	(facing)	161
33.	Spillways	(facing)	162
34.	Spillways	(facing)	163
35.	Outlet works - Grand Coulee Dam	(facing)	166
36.	Outlet works	(facing)	166
37.	Claytor Dam		172
38.	Baffle pitted by cavitation		173
39.	Relations between variables in stilling basin design	(facing)	174
40.	Irrigation structures	(facing)	176
41.	Irrigation structures	(facing)	177

LIST OF FIGURES
(Continued)

FIGURE		PAGE
42. Irrigation structures	(facing)	178
43. Irrigation structures	(facing)	178
44. Route of travels	(facing)	Appendix 1

Figures 1-14; 15A; 16; 17A; 18; 35A; and 39 courtesy of the Bureau of Reclamation, Denver, Colorado. Figures 19-22 courtesy of the U. S. Waterways Experiment Station, Vicksburg, Mississippi. Figures 25 and 26 courtesy of Professor E. W. Lane, Associate Director, Iowa Institute of Hydraulic Research, University of Iowa, Iowa City, Iowa. Figures 27-29 courtesy of cooperative laboratory of the Soil Conservation Service and California Institute of Technology, Pasadena, California. Figure 38 courtesy of the Appalachian Electric Power Company at Claytor Dam, New River, Virginia.

LIST OF TABLES

TABLE	TITLE	PAGE
I	Materials used and methods of construction - hydraulic structure models	15
II	Hydraulic structures and projects	143
III	Itinerary for Freeman Scholarship, 1940-41.	Appendix 1
IV	Expenses	45

CHAPTER I - INTRODUCTION AND SUMMARY

INTRODUCTION

1. Field of study. The Freeman Scholarship discussed in this report was for the nine months' period of October 1940 to July 1941. Subject to approval by the Committee on Freeman Fund, an itinerary was planned which would permit the field of study selected to be evolved. There are apparently two types of programs which may be followed, the first having more precedent than the second. In the first type, one may choose to remain in one place, preferably a university, for extensive study of hydraulic subjects and to do research, and occasionally traveling to nearby places for inspection of other laboratories or hydraulic structures. The second program suggests travel to several laboratories and field projects to broaden one's vision and knowledge of hydraulic developments occurring in practice. The latter program was selected and was based on the author's experience in the hydraulic laboratory of the Bureau of Reclamation, Denver, Colorado. It was realized that considerable benefit could be gained from studying the current research, technique, and completed projects of other laboratories. It would be equally as beneficial, moreover, to study hydraulic structures in the field in an attempt to correlate the performance of a model with its prototype, to obtain a better idea of the magnitude of the prototype phenomena which tends to be distorted from thinking in model terms, and to study those features of hydraulic structures displaying an improper or inadequate design.

To complete such a program, hydraulic laboratories were visited mostly from early fall to early spring in the Eastern States, followed by inspections of hydraulic structures, some in the East, but generally in the Western States. Thirty laboratories were inspected and the study of hydraulic structures was made in the Tennessee Valley

Authority, and on the major irrigation projects of the Bureau of Reclamation and on some flood control projects of the Corps of Engineers. The itinerary as finally completed is given in Appendix I; an expense account is given in Appendix II.

2. Scope of report. In the following pages, some of the problems that face the hydraulic engineer will be discussed, and it will be shown how these problems are being solved by research conducted in private industry, in Government agencies, and in universities. It is not the purpose of this report to record in detail the work being done in hydraulic research at each place visited (this may be obtained from copies of progress reports in Appendix I) nor will a description of laboratory equipment be given, except in special cases[1]. Rather, it is desired that a review be given of some of the more important work to show how problems of the hydraulic engineer are being solved. Finally, a discussion will be presented of the hydraulic structures observed in the field to show the various designs employed, the advancement made in their design due to the efforts of hydraulic research, and the correlations made between models and prototypes.

Throughout this report it has been expedient to borrow freely from the works of others, acknowledgment being given in each case. This has been necessary, particularly in Chapter IV, Fundamental Research, because of insufficient knowledge on the author's part of certain phases of the subjects involved, and because some of the research discussed was studied in early phases and completed and published after the scholarship had been finished. The use of illustrations has been limited due to war economies and because some of the illustrations are related to National Defense regions.

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[1]A description of leading hydraulic laboratories is given in Journal of the Boston Society of Civil Engineers, Hydraulic Section, Vol. 25, No. 1, Section 2, January 1938, "Representative Hydraulic Laboratories in the United States and Canada," by L. J. Hooper.

3. Problems confronting hydraulic engineers. Referring only to phenomena concerning the flow of water, disregarding structural and mechanical problems, it may be said that the problems of hydraulic engineers are in three main groups related to (1) hydraulic structures and appurtenant works, (2) hydraulic machinery, and (3) rivers.

In the first group, the principal problems are concerned with energy dissipation of high-velocity flow, cavitation and pitting of boundary surfaces, air entrainment in open channel flow, venting of subatmospheric regions in tunnels and outlet works, and solution of unique problems peculiar to each structure. The problems of hydraulic machinery entail many mechanical and structural ones as well as hydraulic, each being more or less dependent on the other. With turbine design, cavitation of the runner and throat ring are problems of great importance, as well as elimination of vibration, finding reliable methods of measuring discharges in acceptance tests for installations with short intakes, increasing the efficiency, and obtaining efficient designs of draft tubes. Similar problems exist in pump design, particularly with reference to cavitation, efficiency, and acceptance tests on large pumps. The chief problems concerning rivers are sediment transportation, channel capacity, development and maintenance of navigable channels, and meandering. Included in this group is the silting of reservoirs.

It is recognized that this grouping is arbitrary and that all of the problems in each group are not included, but, in general, those mentioned are the ones given the most emphasis today. A discussion of some of these problems and examples of how they are being solved will be presented in succeeding chapters.

4. Solution of problems by research. There are two distinct kinds of research available for solving problems associated with hydraulics. The first type includes laboratory experiments on models,

wherein empiricism is predominant; the second type, on the other hand, may be called the rational type, involving mostly fundamental research. It is generally admitted that a hydraulic model is one of the best tools available to solve problems quickly, efficiently, and economically. This particular type of research yields results applicable only to problems similar in nature and seldom reveals any basic principles, yet it does provide general information of considerable scope and value to the hydraulic engineer. For example, in studying hydraulic structures, models of them are tested to obtain a satisfactory stilling basin, outlet works, shape of piers and crests, rating curves, and to observe flow conditions in general. After several studies of nearly similar structures, it is possible to develop general rules useful for hydraulic design purposes, these rules being empirical. In spite of these rules and the knowledge gained from studying the same phenomenon many times, it is customary to make model tests of similar structures during their design. It has been asked, why is it necessary to repeat what apparently are duplicate experiments; why not rely on information already gained? Even if duplication is apparent, further study reveals enough differences in design to justify experiments. In addition, it is comforting to know that each structure will perform as anticipated and that the performance and economics of the design is based on model results rather than being based on guess work.

In contrast to this type of research is fundamental research which aims at discovering basic principles of flow phenomena, and presenting the results in a form applicable over a wide range of conditions. In this method, a systematic, rational analysis is made to include all of the variables in the problem, leaving only certain numerical constants to be determined by experiments. Thus, in pipe flow, friction factors have been expressed as a function of Reynolds

number, as has the velocity distribution. Such functions may be extrapolated a considerable distance beyond the limit of the experiments. To do this with empirical relations will produce serious error, since empirical data may only be interpolated.

Because of fundamental research, the profession is constantly improving its knowledge of flow phenomena and gradually reducing the amount of empiricism. The progress of this type of research is slow, however, and the results obtained in some instances are of small practical value, or are extremely difficult to apply to practical problems. Consequently, the use of empirical methods must be maintained until such time as fundamental research has closed the gap between theory and practice.

5. General conclusions. Both from model and fundamental research, the knowledge of flow phenomena has steadily advanced the designs of hydraulic structures and machinery are vastly improved over designs of even a decade ago, and, finally, hydraulic research itself has expanded rapidly in the past decade. The chief reason for most of this development and improvement is traced primarily to the large construction program undertaken by the Government, starting in 1932. Because of the development of many large flood control, power, and irrigation projects, it has been necessary to design hydraulic structures unprecedented in size. The new hydraulic problems originating from these designs have been solved mostly by research which had to expand and improve to meet the demand. Were it not for this impetus, the very existence of some of the problems and their solutions would probably be unknown today, and certainly the challenge to those in fundamental research would be lacking.

More specifically, hydraulic research has developed efficient stilling pools for dissipating the energy of high velocity flow at the toe of overfall dams and at the end of spillways, regulating valves free from cavitation, more economical and safer structures, more efficient

hydraulic machinery, the explanation of pitting of boundary surfaces by cavitation, efficient improvement plans and flood control measures on rivers, the laws of transportation of sediment by flowing water, an insight into the mechanism of fluid turbulence, just to mention a few.

As stated above, hydraulic research is divided into model and fundamental research, the former generally empirical, the latter more rational. To these might be added academic research which is made primarily to demonstrate the laws of fluid mechanics, or to derive certain fundamental concepts already understood. It is believed that more cooperation and tolerance is needed between the practical and fundamental research groups, the former being found in Government or private laboratories dealing primarily with model studies, while the latter group is found in the universities dealing with problems both academic and fundamental. There is a tendency, it seems, for one group to scoff at the other, while a more tolerant point of view should find the problems of one group of equal interest and importance to another group, and the findings of new principles and procedures made more useful for practical application.

It was observed that many well-equipped laboratories are devoting much work to student development with little or no research in progress. With all the many problems to be solved, it is questionable that such a condition should exist. Obviously, student training is of importance, but it is equally important to find out what the practical problems are and devote some efforts in that direction. On the other hand, those laboratories working on practical problems are not doing sufficient fundamental research, probably because their principal work is more pressing. Fortunately, however, the progress of research as a whole is steady and is generally alert to the important problems of today.

CHAPTER II - THEORY AND APPLICATION OF HYDRAULIC MODELS

GENERAL

6. Hydraulic models and fundamental research. Generally speaking, hydraulic models are thought of in terms of their application to the solution of problems of hydraulic structures, hydraulic machinery, and rivers. Fundamental research, although requiring the use of certain types of models, is thought of more in the light of solving unknown or unexplained fluid flow phenomena, or of checking assumptions made in and the results of a rational or mathematical solution of such phenomena. It is expedient, therefore, in this report to separate the research by hydraulic models from that by the rational or fundamental method. Accordingly, this chapter and Chapter III will be devoted to observations made of research by model studies, while Chapter IV is devoted to fundamental research problems.

7. History of model development. Hydraulic models as we know them today owe their origin to the principles of similitude which were developed first by Newton nearly 260 years ago. Prior to that time, Galileo in 1638 studied the similarity of machines from a statical point of view. It was not until about 50 years ago, however, that the principles of similitude were actually applied to hydraulic models in Germany. In America, models were used in 1907 in connection with the Panama Canal, in 1926 with Wilson Dam, and in 1928 to study the Mississippi River. Thanks to the late John R. Freeman, Past President and Honorary Member, American Society of Civil Engineers, for his generosity in establishing traveling scholarships in hydraulics, the hydraulic laboratories in this country have grown rapidly and have learned to adapt the hydraulic model to solution of difficult engineering problems. In the last decade, the large program of hydraulic

construction undertaken by the Government has further developed the hydraulic laboratories, both as to the use of hydraulic models and to the use of fundamental research to solve phenomena never before contemplated or given serious consideration.

8. Purpose of models. To those not familiar with the science of model studies it may be of interest to explain the purpose of hydraulic models, as used in engineering today. Primarily, models are used to study the flow of water in hydraulic structures, machinery, rivers, and other engineering works. By principles of similitude, the model is able to reproduce with considerable accuracy flow phenomena and flow characteristics that would be displayed by its prototype, or field structure. Thus, the engineer can see in the laboratory just how his structure would perform in the field. Furthermore, he can see faulty design and correct it in the model, which is obviously more economical than trying to revise a completed structure. Models are also used to establish design criteria by relating variables through dimensional analysis. Finally, coefficients and similar data are readily obtained which have immediate application to the problem being studied and which may be applied with precaution to problems of a similar nature.

9. Types of models. Models as witnessed in hydraulic laboratories may be grouped according to the type of structure being studied. These are hydraulic structures, rivers, and harbors, hydraulic machinery, and ship models, the latter not being included in this report. Other models revert to fundamental research if we consider those that study waves, silting of reservoirs, and sedimentation, some of these being discussed in Chapter IV. For purposes of discussion, each type of model will be discussed separately to illustrate current laboratory practice in adapting the model to a problem, designing the model, testing procedure, and presenting the data in report form; following this, examples of several types of model tests will be given in Chapter III.

10. Adapting the model. As in nearly all professions any instrument or procedure should be used or managed by properly trained personnel. So it is with the hydraulic model, only those familiar with its applications and limitations should be trusted with its use in solving problems.

A first requirement in adapting a model to any problem is an understanding of the principles of similitude. It is not the purpose here to discuss these principles in full, the reader being referred to any text on fluid mechanics and to "Hydraulic Laboratory Practice," Appendix 15, published by the American Society of Mechanical Engineers, and edited by John R. Freeman. The important facts to consider in the similitude considerations is the geometric, kinematic and dynamic similarity between the model and its prototype. Geometric similarity requires a similarity in shape only, that is, the linear dimensions of the model shall be obtained from those of the prototype by applying the scale ratio, no consideration being given to motion, force or mass. In hydraulic structure models, the horizontal and vertical dimensions are almost always reduced the same, thus giving an undistorted model. River models, on the other hand, are usually distorted, the horizontal dimensions being reduced more than the vertical dimensions. Kinematic similarity requires that the time, velocity, acceleration and direction of a particle in the model be similar and proportional to the corresponding particle in the prototype, which automatically requires geometric similarity. Dynamic similarity requires first that there be both geometric and kinematic similarity, and in addition that all corresponding forces acting on corresponding particles in the model and prototype systems be in the same ratio. To summarize, we find that rigorous similitude between the model and its prototype requires; first, that there be a similarity of shape, geometrical similarity; second, there must be a similarity of shapes, paths, and times, kinematic similarity;

and, third, there must exist a similarity of shapes, paths, times and masses or forces, dynamic similarity.

In considering the forces acting on homologous fluid particles, it is found that these are: (1) gravitational forces; (2) viscous forces; (3) capillary forces; and (4) elastic forces. Thus, to have dynamic similarity between the model and prototype systems these forces must be in a constant ratio, model to prototype. In practice, however, it is rarely possible to maintain rigorous similarity, especially if two or more of these forces are predominant in the problem to be studied. For example, it can be shown that if water is used in model and prototype and that if both gravitational forces and viscous forces (fluid friction) are predominant, then in the first case similitude requires, according to Froude's Law, that the ratio of velocities, prototype to model, be equal to the square root of the scale ratio (prototype to model); in the second case it will be found from the Reynolds Law that the velocity ratio is equal to the reciprocal of the scale ratio. By using a different fluid in the model, it is theoretically possible to provide correct similitude in this case where gravity and viscous forces are both predominant; however, in practice it is generally impractical to do this. Nevertheless, it has been found that rigorous similitude is not necessarily required, primarily because of the fundamental fact that models are tools for determining qualitative instead of quantitative results.

In general, most model tests involve only gravitational forces, the others (viscous, capillary, and elastic) are assumed to be absent or inconsequential. Accordingly, dynamical similitude requirements are satisfied by the Froude Law, which may be stated that the ratio of prototype to model velocities is equal to the square root of the scale ratio, as mentioned above. This is derived from Newton's second law of motion

which may be considered as a general law of dynamic similarity when it is stated thus: The ratio of forces, model to prototype, equals the mass ratio times the acceleration ratio, model to prototype, or the ratio of inertia to gravity forces is the same in model and prototype.

11. Selecting a scale ratio. The factors that govern the scale ratio are; viscous effects, water supply and space available. In all cases, a scale ratio is so chosen that the flow in the model, as in the prototype, will be turbulent throughout the range of operation, thus excluding viscous effects due to laminar flow. From experience it is possible to select a scale ratio and know that no laminar flow will occur. However, it is customary to check the Reynolds numbers of the model flows to see that they are of sufficient magnitude. If no operating conditions will produce a value of velocity times hydraulic radius less than 0.02 then turbulent flow will always occur. A scale ratio must also be chosen so that it will not give a model discharge in excess of the available water supply or require a model larger than can conveniently be handled.

Consideration is generally given to the roughness of the model when selecting a scale ratio. For example, it can be shown that if a prototype spillway has a Manning's n of 0.014 and the model an n of 0.010, then the correct scale ratio should be 1:7.53. This is established from the fact that the scale ratio must equal the ratio of Manning's n, prototype to model, to the sixth power. It is only occasionally that a scale ratio of 1:7.53 is possible, hence for smaller ratios (1:10, 1:60, etc.) the model surfaces are too rough, and the losses are greater, while the velocities in the model are less than required by the Froude Law. Assuming the smoothest surface that can be obtained in the model corresponds to an n of 0.009, then the theoretical limit of the scale ratio is about 1:15. In practice, however, scale ratios as small as

1:100 have been used, more for general performance models than for detailed studies of individual hydraulic features. The error introduced by having model surfaces too rough is usually small, but it is necessary to analyze model data taking into account this lack of similitude. Nevertheless, it is essential to make the model surfaces as smooth as possible, and if it is desired to correct for the model velocity, the slope in the model may be increased. Recent investigations and observations of air entrainment in high velocity flow reveal a bulking and a consequent reduction in velocity below the value based on Manning's formula. Accordingly, with no air entrainment in a model because of insufficient velocity, the model velocities are more nearly correct considering the fact that they are slightly reduced by model surfaces which are too rough.

Frequently a structure to be studied is so large that even with a practical scale ratio the model is too small to study anything but the general flow conditions for various operations. When this occurs, it is customary to study individual features by sectional models to a much larger scale (less reduction). This permits a more careful study of the flow conditions and greatly reduces the magnitude of errors in taking measurements, since these errors are magnified when converted to prototype dimensions.

12. Preliminary design considerations. After the scale ratio has been selected, the design of the model proper is started. There are certain fundamental considerations necessary to include in the design. The model must be of rigid construction, easily altered, accurately constructed, and economical. Careless and inadequate construction is a waste of time and money and certainly reflects on the results obtained. It is usually known during the model design just what portions of the structure are to be studied and probably revised. Accordingly, these parts are so constructed that they are easily

altered or entirely replaced. Materials such as wood or sheet metal are better adapted than concrete or metal castings, although the latter are often used for parts of the model not subject to much change. Accuracy of construction is essential, otherwise the results obtained are questionable. Careful checks are frequently required not only during construction but after completion of construction to insure correct alignment, differences in elevation, and workmanship. Economical models are desirable but not to the extent that the other essentials of design are sacrificed. Frequently cheaper materials are used to construct the model for preliminary tests, and later, when the final data are taken, the model may be rebuilt of the same materials or better materials if desired. Thus, wood covered with sheet-metal may be sufficient at first, followed by concrete or even metal castings.

13. Materials used and methods of construction. The methods and details of model design are usually expressions of the individual. In general, it has been observed that most laboratories employ the same technique. The elements of a hydraulic structure model include a box, either concrete, steel or wood (lined with sheet metal) to contain the model, a pipe supplying water to the model at the upstream end of the flume, a baffle to still this supply, a tailgate at the downstream end of the model to regulate the depth of flow. Where sectional models are used, a flume with a glass side is frequently used to aid in observing the flow conditions. Examples of model construction can be seen from several of the illustrations in Chapter III.

The parts of the model proper are, of course, dependent on the type of prototype structure to be studied. Accordingly, it is necessary to build models of dams of the overfall type, or dams of the non-overfall type with emergency spillways, and their appurtenant works, which may include outlet sluices, needle or tube valves, and in some cases a powerhouse. Lock models must include the culvert systems, both emptying and filling, and the lock chamber. The proper design and selection of material for these models is important in order to satisfy

the fundamental considerations of model design. Accordingly, a review of current practice may be of value. Table I gives, for various types of hydraulic structure models, the materials commonly used for various parts, methods of construction, and pertinent remarks. It will be noted that wood, sheet metal, pyralin and lucite, and cement mortar are the materials most commonly used in model construction. The properties of each material, in regard to its use in models, sometimes governs its selection. For example, wood is rigid, easily worked and altered, and economical, but will warp out of shape, so its use is limited where submergence occurs unless the wood is treated or covered with sheet metal to exclude moisture. Sheet metal is also rigid, easily worked and altered, economical, and will not warp from moisture, so this material is frequently used for most parts of a model. Plastic materials, such as pyralin and lucite, are used particularly when observation of flow is desired. Thus sluices, penstocks, draft tubes, transitions, and elbows are frequently made from plastics. Pyralin is gradually being replaced by lucite because the former will shrink and discolor, while the latter will not. Because of the cost and added work required in molding, these materials are not used as extensively as sheet metal and wood. Cement mortar, together with sheet metal ribs or templets, is also widely used. This type of construction is not readily altered so its use is often limited to parts which have been previously studied and revised sufficiently to require little or no change in dimensions.

Table I

MATERIALS USED AND METHODS OF CONSTRUCTION
Hydraulic Structure Models

Parts of prototype structure	Materials used in model	Methods of construction	Remarks
The dam proper	Sheet metal (iron, brass, copper)	Sheet metal or wood ribs cut to profile and covered with sheet metal. Ribs fastened to heavy gage base plate with angles or soldered.	This method good because of ease in making changes, and inserting sluices and piezometers. Brass or copper sheets may be used to eliminate corrosion, but more expensive.
	Wood (redwood, cypress, pine, plywood)	Ribs or truss cut to profile and covered with plywood. If small model, solid piece may be used. Crest usually formed from solid piece. Ribs may also be covered with sheet metal.	Good for initial tests if frequent changes are anticipated. Warping must be kept to a minimum by use of linseed oil, varnish, or paint treatment. Redwood is recommended.
	Cement mortar (1 cement to 3 sand); concrete	Metal ribs cut to profile and filled between by concrete. Surface made smooth by scraping or use of abrasives.	Difficult to alter or to add sluices, piezometers and to attach other parts. Concrete used in large models.
Piers	Sheet metal (iron, brass, copper)	Piers made hollow by shaping and soldering metal around pieces of sheet metal cut to shape of horizontal section of pier.	Pier easy to attach to dam. Piezometers readily adapted.
	Wood (redwood, cypress, plywood, pine)	Shaped from solid piece or made hollow if pier large. Metal templets cut to guide shaping.	Piers sometimes difficult to attach to dam. Piezometers not easily attached.
	Cement mortar, concrete, plaster	Piers usually cast.	Surfaces must be made smooth by abrasion. Not easy to alter or to attach to dam.

Table I (Continued)

Parts of prototype structure	Materials used in model	Methods of construction	Remarks
Piers (cont.)	Pyralin or lucite	Piers formed by heating and molding sheets of material to shape, then cementing together with acetone or cement recommended by manufacturer. Heating and molding not always necessary for simple shapes.	Permits observation of flow along pier. Piezometers easy to attach.
	Lead, iron, brass, aluminum or bronze castings	Piers cast and smoothed by machining.	Not economical unless large number required. Alterations difficult.
Regulating gates	Sheet metal (iron, copper, brass)	Heavy gage sheet metal bent to shape for radial gates. Vertical ribs cut to shape and covered with sheet metal for drum gates. Vertical lift gates made in similar manner.	Piezometers easily installed.
	Wood (redwood, cypress, pine)	Best adapted for vertical lift gates by using solid piece.	Wood must be treated to prevent warping.
Outlet conduits, sluices, penstocks, culverts.	Standard pipe or tubing (iron, brass, copper)	Proper inside diameter selected or nearest size reamed. Pipe bent to correct profile, Bellmouths formed by inserting a die in end of pipe.	Convenient because ready to use. Flow cannot be observed.
	Pyralin or lucite	Lengths made to proper size and profile in convenient sections from sheets. Connected together by flanges of same material	Used extensively because flow can be observed. Simple construction. Piezometers easily installed by using pyralin or lucite nipples.

Table I (Continued)

Parts of prototype structure	Materials used in model	Methods of construction	Remarks
Outlet conduits, sluices, pen-stocks, culverts (cont.)	Sheet metal (iron, brass, copper)	Pipe formed from sheets and connected by metal flanges.	Simple construction, economical. Flow cannot be observed.
Outlet valves (needle, tube, Pelton, Howell-Bunger)	Castings (bronze, brass)	Castings used almost exclusively with accurate machining.	Construction by castings used almost exclusively with accurate machining. This permits close control of dimensions and water passages. Operating mechanism readily incorporated although not necessarily similar to prototype.
	Sheet metal (iron brass, copper)	Hollow construction of parts by ribs covered with sheet metal.	This method is used only where it is desired to reproduce jet action. In testing valves for pressures and general performance, castings are recommended.
Open channels (spillways, canals, chutes)	Wood (redwood, plywood, pine)	Channels formed using boards for walls and bottom with braces spaced uniformly along channel. Vertical curves formed by cutting vertical ribs to be covered with plywood, or using solid piece of wood if curve very short. Channels also formed by covering long longitudinal ribs with plywood and attaching walls to plywood bottom.	To avoid excess leakage and warping, sheet metal lining is advisable in most cases. Joints may be sealed with white lead but warping soon develops to produce leakage.

Table I (Continued)

Parts of prototype structure	Materials used in model	Methods of construction	Remarks
Open channels (spillways, canals, chutes) (cont.)	Sheet metal (iron, brass, copper)	Ribs cut to proper section and lining shaped in from rib to rib; solder joints customary.	Used more for channels having a section other than square, rectangular, or trapezoidal.
Transitions (warps, changes of section)	Sheet metal (iron, brass, copper)	For changes in section in closed conduits. Flanges cut for each end to assist in shaping and joining to normal conduit sections, and development made of transition on sheet metal or on full size drawing transferred to sheet metal by punch. Solder joints throughout.	Excellent method for changes from circular to horseshoe, rectangular, etc. This method not adaptable for warps.
	Pyralin, lucite	For changes in section in closed conduits. Flanges of material fastened to each end to assist in shaping and connecting to normal conduit sections. Sheets molded to proper shape to form transition. Joints fastened by cements--acetone for pyralin, ethylene dichloride for lucite.	Used when it is desired to observe flow conditions. In using these materials an oven is required to heat sheets of material before being pressed in models. Pyralin requires 120°C., lucite 105°C. to 125°C. When working to inside dimensions, core is cut to correct size, female is increased accordingly.
	Wood (redwood, cypress, pine)	Male sheet metal templets cut to give sections. Transition cut from solid block using templets as a guide.	Used primarily in open channel, but frequently used in closed conduits, by using two blocks one above the other.

Table I (Continued)

Parts of prototype structure	Materials used in model	Methods of construction	Remarks
Transitions (warps, changes of section) (cont.)	Cement mortar, concrete, plaster	Metal ribs cut to give changes in section with material filled in between and finished to ribs. For changes in section in open channels such as rectangular to trapezoidal, place metal guide strips along top and bottom trace of warp, add wire screen and coat with material. Screed material following guide strips and side walls at each end of warp transition.	The first method is used for both open channels and closed conduits; the second usually for open channel changes in section. Plaster not recommended for outdoor construction.
Topography	Sand, sawdust concrete, cement mortar, concrete blocks, falsework with templets.	Sand (1) Female templets cut to cross-sections are filled between with sand finished to templets; (2) male templets are made of cross-sections and supported on rails to correct elevations. Sand molded to bottom trace of templets; (3) pantograph with point gage to indicate points along cross-sections, sand molded to settings	If scour is to be studied, templets in (1) may be removable and reset to restore topography. To prevent sand from moving, dust dry cement over wet sand (finished surface), or add cement mortar about one-half inch thick. Sand will settle over period of time, so tamp while placing and keep well saturated.

Table I (Continued)

Parts of prototype structure	Materials used in model	Methods of construction	Remarks
		indicated; point gage on reference bar across model to indicate correct elevation to bed; (4) pegs cut to proper length and attached to floor of model, sand shaped to top of pegs.	
Topography (cont.)		Sawdust Concrete (5) Used for finish coat to give rough surface. Applied on top of sand, gravel, or cinders placed between templets as in (1) and (2).	Mix used is 1½ cement, 3 sawdust, and ½ to 1 plaster. This mix should be placed quickly, usually by hand. Mix should not be too wet.
		Cement Mortar (6) Used for finish coat where rough surface not required. Placed as in (5).	Mix used is 3 cement to 1 plaster with enough water added for workable consistency.
		Concrete Blocks (7) Sometimes used when it is desired to remove and replace sections of topography in accordance with different designs studied in models. Blocks are cut in convenient sizes to facilitate handling.	Any average mix is used with aggregate very small. Cement mortar may also be used as in (6). Paper is placed over bottom topography before new topography is poured. This allows sections to be removed and later replaced.
		Falsework (8) It is frequently necessary to reproduce topography above	This type of topography is not generally used as other types above usually extend above high water. Method

Table I (Continued)

Parts of prototype structure	Materials used in model	Methods of construction	Remarks
Topography (cont.)		the maximum water surface. This can be done by building a wooden framework and covering with wire mesh. Plaster or cement mortar is then added and colored if desired; (9) sheet metal trays fastened between wooden ribs are sometimes used. Wet sand is placed in trays and covered with dry cement to form crust; (10) topography of reservoirs above and below water has been reproduced by building wooden frameworks and covering with sheet metal.	(9) is not suited for steep topography. Method (10) requires considerable work in building a framework to give proper relief and sheet metal must be cut in peculiar shapes to fit. Any of these methods will save considerable weight.
Baffles, sills, dentates	Wood	Teeth made in solid pieces and fastened to board which is fitted correctly into position.	Piezometers not easily installed. Metal tooth with piezometers may be substituted in row for certain wood teeth.
	Sheet metal (iron, brass, copper)	Hollow construction used with teeth fastened to metal plate or directly to model.	Piezometers easily installed.
	Pyralin, lucite	Hollow construction. Pieces cut to shape and cemented together.	Rarely used, and then for test of only one tooth in section. Flow viewed through glass panel in side of test flume.

14. Testing procedure. Upon completion of the model, and after a careful check on dimensions, leaks, discharge-operation data, and

verification of model flow, a preliminary test is made to observe flow behavior with attention being paid to all unfavorable conditions observed. It is customary to be familiar beforehand with that part of the design which is in doubt. This is made possible from liaison with the engineers responsible for the design, and such liaison is usually maintained throughout the testing of the model. Once sufficient data and observations have been made to explain the unfavorable conditions existing, plans are made for revising the design to improve its performance. These revisions are based on structural and economic limitations, experience rising from studies of similar problems, and not infrequently on imagination and common sense. Continued revisions require that each modification be studied and data collected which can be used as a basis of comparison. Hence, it is necessary to take measurements of velocities, pressures, discharges, bed and water surface profiles, forces, vibrations, and photographs.

It was observed in the leading laboratories that experience of the personnel and their judgment was combined with the data taken to analyze the steps to follow. Thus, worthless tests were eliminated and, above all, too much data were not taken, but enough to conclusively show the relative merits of each design tested. When a satisfactory solution had been obtained, a carefully planned series of tests were made to record all data necessary to confirm reasons for its selection, together with data to establish operation procedure for use in the field to assure a maximum efficiency of field performance. Approval is then obtained from the designing engineers and the testing may be considered finished.

15. Measurements. For model tests it is usually necessary to obtain some or all of the following: Discharge, velocity, water surface profiles, pressures on boundary surfaces, profiles of sand beds, vibrations, air-demand, forces, current directions, and photographs.

The discharge in a laboratory is obtained from calibrated weirs, venturi meters, and orifices, the flow being measured either before it enters the model or after it has passed through the model. Velocities are obtained by pitot tubes or miniature current meters, and sometimes from surface or subsurface floats. Water surface profiles are recorded by standard point gages attached to reference bars or beams which may be moved along level supports placed on each side of the model. Pressures on boundary surfaces are obtained from copper tube piezometers placed flush with the surface and connected to water or mercury manometers. Electric pressure cells in series with a piezometer and connected to an oscillograph have been used to obtain the instantaneous pressure fluctuations, which cannot be found from manometers because of the inertia of the water columns. Profiles of sand beds before and after scouring are procured by point gages; the general relief may be recorded by placing string contours over the bed and photographing it from above. Vibrations may be obtained from reeds, or from accelerometers and pressure cells attached to the model and connected to an oscillograph. Air-demand is measured by orifices or by anemometers, while forces such as required to raise gates or to control tows in locks may be obtained from calibrated springs or other calibrated instruments designed for the particular problems involved.

To obtain the direction of currents in a model, it has been convenient to use sawdust or confetti which may be photographed to produce streak pictures. Currents below the surface have been defined by strings and by injecting dye from a needle-like piezometer tube. It is customary during any model test to record all revisions, tests, and flow phenomena by photographs. To do this, it is expedient to have a standard reflex and view camera as well as a moving picture camera which can take at least 64 frames per second for slow motion pictures.

16. Report of model tests. A report of the model study of a hydraulic structure is important and is generally considered an essential part of the testing. The types of the reports read during visits to laboratories were many, some too bulky and padded, others too brief. It is generally agreed that most reports should be concise and clearly written. To obtain this, it is perhaps necessary that the inclusion of too much data such as pages of scour profiles or calibrations be omitted or replaced by photographs, and that only enough data be included to conclusively show why changes were made and why the final solution was selected. Wherever possible, general rules might be presented which would be applicable to problems of a similar nature.

RIVER AND HARBOR MODELS

17. Adapting the model. In Section 10 under hydraulic structure models, brief mention was made of the principles of similitude involved in model studies. Under geometric similarity it was stated that hydraulic structure models are usually not distorted, the horizontal and vertical scale ratio being the same. With river and harbor models, however, it is customary to use distorted models with the exceptions to be stated. The reasons for distortion are based on the cost of the model and laboratory facilities, the necessity of having turbulent flow and measurable quantities, and tractive force. Since most prototypes in the country are large rivers and since the problem area includes long reaches, an undistorted model would be so large that its cost would be prohibitive and the water requirements would be excessive. Since the width-depth ratio of rivers is large and the slopes flat, an undistorted model might produce in certain reaches laminar flow or a depth and corresponding velocities so small as to be nearly unmeasurable. In this regard, the model velocities would also be too small to move any bed material. As a result of these factors, the model must be distorted by using a length ratio greater than the depth ratio, giving geometric similarity in plan only.

The effects of geometric distortion may be summarized as follows: (1) Change in shape of cross-section of stream (and of structures) with a resulting variable scale of hydraulic radius from section to section; (2) change of hydraulic capacity of the model stream due to distorted depth and slope, necessitating a change in model roughness from section to section and at any section for changes in depth; (3) change of the magnitude of velocity and velocity distribution, current direction, eddy formation, and increase of the transverse slope of water surface; and (4) influence on secondary currents which, together with the other factors, influence the bed configuration obtained in the model.

Accordingly, geometric distortion seriously affects the obtaining of strict dynamic similarity, but when these influences are taken into consideration in analyzing the results obtained, it is evident that distorted models are practical tools for solving river problems.

In undistorted models it was shown that the model was operated according to Froude's Law, namely, the velocity (and time) scales are equal to the square root of the linear scale, with the discharge being equal to five-halves power of the linear scale. In distorted models, somewhat similar relationships are used which are later modified from verification tests on the model. In other words, strict dynamic similarity is not obtainable. If consideration be given to two types of river models this difference may be seen. First, consider models of rivers involving no movable bed, but dealing primarily with channel capacity studies, and second, those models of the movable bed type.

Of the first type of model, the study may treat uniform channels or nonuniform channels, the primary interest being the study of flood hydrographs and profiles, thereby requiring of the model that it must be able to convert--to the vertical scale of the model--pressure head to velocity head, or vice versa, at points of changing

cross-sectional area. To cause this condition in the model, it can be shown that the velocity scale should be equal to the square root of the depth scale, the time scale be equal to the volume scale divided by the discharge scale, and the discharge scale be equal to the length scale times the three-halves power of the depth scale. Because of the geometric distortion and its effects as summarized above, it is necessary to consider a roughness scale in order to bring the model into agreement with the prototype. Thus a verification test is made in which model roughness is varied uniformly, or varied from reach to reach, and the discharge scale is varied simultaneously in order to make the model reproduce the correct profiles and flood hydrographs. Thus, in distorted models it is necessary to increase the model roughness, while in undistorted models the problem is to make the model smoother. When this has been accomplished, it is possible from the discharge scales and roughness scale just established to recompute the velocity scale and time scale, using an average hydraulic radius scale obtained from several sections in case of non-uniform channel, or a constant hydraulic radius scale in uniform channels.

In these models there is no special need for an exact reproduction of velocity distributions, paths of flow of corresponding water particles, etc., but the conversion of kinetic energy to potential and vice versa is required, thus the model energy gradient is at the same elevation above the water surface in the model--to the vertical scale--as in the prototype. With modified discharge and velocity scales this condition is also not exactly obtained, but this error is slight in comparison with the overall accuracy of the model.

In the case of movable-bed models, considerable adjustment is required to obtain a set of scale ratios for velocity, time and discharge. In fixed-bed models the accurate reproduction of prototype velocity distribution was not essential, but in movable-bed models, problems are concerned with the movement of materials of the riverbed,

which is directly related to velocity, its magnitude and distribution. Accordingly, problems are concerned primarily with bed movement occurring under a hydrograph of average flow, together with the effects on this movement by changes made by man, such as cutoffs, dikes, dredging, jetties, etc. With a length and depth scale established to provide a practical model, theoretical scale ratios for velocity, time, and discharge are computed. Because of distortion, as explained previously, the model velocities are usually of small magnitude, especially during low stages, so that bed load movement does not readily develop. Consequently, resort is made to using bed materials having a specific gravity near the value of water and to changing the slope, discharge, and the time scales. In practice, bed materials may consist of processed coal (s. g. 1.30) carefully prepared as to grain size and specific gravity, gilsonite (1.03) an almost pure bitumen, various resins (1.09 to 1.13), hawdite (1.85), and sand of various sizes (2.65). The material selected depends primarily on the velocity magnitude expected in the model.

On first consideration it would seem reasonable that, if a proper bed material were used, and the scale ratios for velocity, time, and discharge based on Froude's Law be applied, the model would reproduce reasonably well the prototype bed movement. To determine if this were true, it would be necessary to mold the model bed to a survey of field conditions, then run the model to reproduce an average hydrograph, using the theoretical time and discharge scales for a certain time interval, established by the time elapsed between the date of the prototype survey set in the model and a more recent one. The latter would be compared with the model bed configuration at the end of the proper time to see if the model verified this survey. Unfortunately, this verification would be lacking because the model bed configuration would not agree with the recent survey. Thus the effect of geometric distortion on the model is enough to invalidate the use of theoretical scale ratios. In this assumed case, hydraulic similitude was maintained

to the sacrifice of bed verification which is the primary concern. The other extreme would be to obtain verification by diverging too far from reasonable hydraulic similitude. Evidently then, recourse must be made to a procedure lying within these two extremes, but with hydraulic similitude being maintained as rigidly as possible to obtain the most reliable verification.

Since it is impossible to use theoretical scale ratios in this type of model, a cut-and-try process must be used with the middle course just explained to verify the model. Based on logic, this verification is carried one step further by saying that if the model can be made to reproduce events that happened in the prototype in the past, then it can be relied upon to predict events which will occur in the prototype in the future. The wisdom of this has been successfully demonstrated many times at the U. S. Waterways Experiment Station, but certain factors must be considered in making the verification before the reliability of the model for predicting future events is warranted. These factors may be summarized as follows: (1) The event of the prototype upon which the model verification is based must be directly related to the problem being studied in the model. Hence, a prototype flood would not be a suitable event if the model is attempting to verify bed movement produced over a long period of time in the prototype; (2) the prototype event upon which the model verification is based must represent action of long duration, that is, a navigation problem requires bed configuration changes developed over a long period of time, not just for one week; (3) the verification event must be a representative one, not one resulting from abnormal flow conditions which extend over a relatively short period of time. These factors, then, apply to the verification event itself. One additional factor must be considered which applies to the reliability of the model to predict future events--assuming the three factors just given have been adhered to. This factor requires that the proposed plans to be studied in the model after verification

do not depart too widely from the conditions under which the model was verified. Consequently, if the verification event included a system of dikes and its effects, while the proposed plans to be studied after verifications called for a series of cutoffs, then the reliability of the model is greatly reduced.

In the hypothetical verification based on theoretical scale ratios, it was shown that verification was impossible. Resort must then be made to a cut-and-try process. In determining in this manner a set of operating conditions which will reproduce a carefully selected prototype event, these factors must be adjusted and manipulated: Discharge scale, slope of water surface and bed, type of bed materials, magnitude of time scale, roughness of fixed boundaries, and permeability of structures (dikes, jetties, etc.). This is an arduous task which takes much time, skill, and patience. Of these items, the time scale is of considerable importance as it affects the movement of the bed material. It is generally agreed at the Experiment Station and elsewhere, that there is no suitable rational expression for the time scale, that is, the ratio of the volume scale to the discharge scale is not valid. The empirical time scale established for the verification must be used, and this is further complicated, as will be seen, because two time scales are used, one for running the hydrograph and one based on the time required to reproduce the bed configuration occurring in a definite prototype interval.

To arrive at a time ratio one is assumed for the model, say 10 hours in the model equals 1 year in the prototype for the period chosen for verification. Using this assumed ratio, the complete hydrograph is run for the verification period, then the model bed is compared with the known bed of the prototype at the end of the verification period. It will probably be found that this procedure

overemphasizes the effect of the higher stages because the movement of the bed material in the model during medium and flood stages moves more bed material than do the lower stages. This unfavorable condition is modified by use of one or both of the following methods: (1) Vary the time scale with the stage, allowing more time for low stages, less for high stages, or both; (2) decrease the slope scale (this reduces the discharge scale also) of the higher stages. If these methods work to give proper verification, it is conceivable that the time required to produce verification in the model is different than the length of time represented by the number of hydrograph cycles used. For example, if 10 hours on the model represents 1 year on the prototype, and 5 cycles of hydrographs are run to verify, then 5 years on the prototype have been represented, but the verification period may only be 3 years for the prototype surveys used for verification.

The effects of distorting models have been shown in the foregoing discussion using as examples river models involving channel capacity and bed movement. These are perhaps the most common studies involving distorted models; some river problems, however, must be studied by undistorted models. These include the study of transitions involving supercritical velocities, standing waves, backwater from dams and bridge piers or other obstructions. In general, similitude according to roughness must be followed in these cases with the ratio of roughness (prototype to model) equal to the sixth power of the scale ratio. If the problem is concerned with the form, pattern, and travel of waves, occurring not so much as in rivers but as in harbors relative to backwater location for reducing wave height, then an undistorted model is used since it is impossible to distort surface waves. Wave models are governed by Froude's Law, with roughness adjustment sometimes required, but it has been found that surface roughness has only a small influence on wave reproduction. Models of tidal areas, wherein current directions

and velocities are involved, require undistorted models for accurate work, because of the importance of proper simulation of velocity distribution. It has been found, however, that if the prototype area is reasonably wide and shallow, or if the study is more concerned with average velocities than with exact velocity distributions, resort may be made to distorted models with relatively small geometric distortion. Where beach movements are combined with tidal areas, wave action is involved, so distortion of the model is theoretically incorrect, but experience has shown that slight geometric distortion is allowable, depending on the slope of the beach and the amount of bending of the waves as they approach the area under study. Thus, the distortion should not cause the model beach to be on a slope much greater than that of the prototype beach, otherwise the waves might break at the wrong position on the beach. Slight distortion would not influence the direction of approach of the model waves provided the prototype waves approach the beach without much change of direction; however, if the waves bend appreciably approaching the beach, then distortion would change the wave pattern, causing the waves to travel in more of a straight line without sufficient bending.

18. Selecting a scale ratio. In the preceding section some views were presented concerning geometric distortion; mention is now made of the allowable distortion found in practice today. In any event, the distortion used is based on the size of the prototype, laboratory facilities, costs, and requirements of turbulent flow and sufficient tractive force.

Accordingly, each problem is given preliminary consideration based on these factors and on the natural characteristics of the prototypes themselves.

For river models involving channel capacity studies, the geometric distortion used is considerably greater than for movable bed models. This is due to the fact that the divergence from Froude's

Law in the former is less important than in the latter, as long as the conversion of kinetic to potential energy and reverse, to the vertical scale, is in close agreement. Thus, for channel studies the length scales may vary from 1:50 to 1:2000, with depth scales from 1:50 to 1:200, the distortion varying from one to ten. For models of the movable bed type, the maximum distortion is usually six, with four or less more desirable. The amount of distortion in any event depends on the shape of the prototype stream. If wide and shallow, greater distortion is allowable (less depth reduction); if narrow and deep, smaller distortion is used. For models involving wave action, which must be undistorted, the scale ratio depends upon the size of the prototype waves and the measuring instruments of the model. For models of tidal areas dealing with current directions and velocities, a small distortion is required as in movable-bed models, but no distortion is desirable wherever possible.

19. Preliminary design considerations. The general views expressed in Section 12 concerning hydraulic structure models apply also to river and harbor models. Whereas the former type may include many different kinds of structures requiring the use of varying technique, river and harbor models follow a more uniform technique. Changes to occur in the model proper will consist mainly in channel shape and alignment, bed configuration, and location of structures. Consequently, it is necessary to provide for easy alterations. Consideration must be given also to the type of bed material to be used, as previously discussed, and in the case of tidal models or wave models, a study is required of the prototype phenomena to place wave machines and other pertinent equipment in their proper location so as to reproduce the prototype phenomena correctly in the model.

Because river and harbor models are much more extensive than hydraulic structure models, it is customary to plan a horizontal and vertical control to assist in construction. Since sheet metal templets

are generally used to form the channels, they must be properly located with reference to each other. The horizontal control may be accomplished by use of a grid laid out on the floor of the model; by using a traverse approximating the centerline of the stream; or by plotting the location of templets on a grid to a small scale and then projecting this drawing on to a screen and tracing this projection, which can be made full model size by proper location of the screen from the projector. This tracing is then placed on the floor of model. The vertical control in all cases can be referred to any conveniently located benchmark.

20. Materials used and methods of construction. River and harbor models, because of their size, are generally constructed directly on the ground and covered by a shed. A trench of sufficient width is dug following the alignment of the channel or area involved. A retaining wall of brick is then placed around the perimeter of the trench, the wall extending six inches or more above the ground surface. Female sheet metal templets with wooden stakes attached are placed in the trench, the stakes being driven into the ground. These templets are so cut that the movable bed section is omitted, later to be molded by male templets. A compacted fill is then placed between the female templets and is covered with a layer of concrete to within an inch or so below the top of the templets. At the same time, supports for side rails and point gages are concreted in place. Then the movable bed material is placed on top of the concrete layer in the bottom of the model. This is molded into shape with male templets located and supported on the previously installed side rails or reference rails. Other methods of forming topography may be found in Table I, similar methods being used both in hydraulic structures and river models although the above method is customary for river models. Similar to the hydraulic structure models, the elements of a river model include the pump and supply system, a forebay with baffles for stilling the

water entering the model, a flume, box, or a prepared trench in the ground for containing the model, a transverse trough to trap sediment, and a tailgate for regulating water surface elevations.

Harbor models are constructed in a manner similar to river models, with the exception of the movable bed section which is usually not required in harbor models. Accordingly, the concrete top coat is molded to proper configuration throughout the model. When, however, shoaling is involved this is provided for by adding a suspension of the bed material selected.

When it is not possible to construct river and harbor models on the ground, a suitable shallow brick-wall flume can be formed directly on a laboratory floor. For small models, a metal-lined wooden box or flume is sometimes used.

21. Testing procedure. For river and harbor models, the testing procedure is divided into two parts: verification and test of proposed improvements. It was noted in hydraulic structure models that verification is not of primary importance because the Froude Law is maintained within close limits, so a check on coefficients, velocities, and water surface profiles is usually sufficient. In practice, it is rarely necessary to consider verification which is usually impossible anyhow because the prototype is yet to be built. In river models, on the other hand, it has been shown that distortion requires a check on the accuracy of the model for reproducing past events and for predicting future events. The verification of river models is accomplished by adjusting the time, velocity, discharge, and roughness scales until water surface profiles and bed load movement in the model are comparable with those in the prototype for a selected time interval. This requires prototype surveys for two different dates and of sufficient time interval to furnish average conditions of the prototype. If, during this prototype interval between surveys, any dredging or change of structures occurs, such

occurrences must be incorporated in the model tests at the correct time. In addition, bed load material must be added at the upper end of the model to compensate for the material washed out at the lower end.

Upon verification of the model and the obtaining of adjusted scale ratios, the model bed is replaced to represent present conditions and the proposed regulating works are installed. A test is then made consisting of a number of runs, each of which reproduces once an average hydrograph obtained from the yearly hydrographs used during verification. The number of runs required should be sufficient to produce stability of the movable bed so that the test will show the ultimate effects of the regulating works being studied. The model values of discharge and tailgate regulation necessary to produce correct stages for the average hydrograph used, are obtained from the model conditions established during the verification tests. The prototype stage curves for various prototype gages are used to select stages desired in the model at corresponding gages. This procedure eliminates any errors in slope or discharge which might be caused by basing model operation on gage heights influenced by the installation of proposed works.

Bed material is introduced into the upper end of the model during each run of each test, and surveys of the model bed are made at frequent intervals to show the progressive and final results of the various tests made on the proposed regulating works. These surveys are analyzed and predictions as to probable effects of each plan made. In this regard, it is realized that a quantitative interpretation of the results of the model tests is impossible because of the many factors introduced by distortion of the model.

In harbor models, the procedure followed is quite similar to the procedure for river models, except for bed load movement. Since harbor models deal primarily with tides and currents, by proper design of tide

controls in the model and by proper adjustment of surface roughness, the model is made to reproduce the tide and currents surveyed in the prototype area. With verification established, the present condition of the prototype is set up in the model and data collected of its performance with no improvement works installed. Using these data as a source of comparison, improvement works are then installed and tested. Finally, a study is made of the various plans tested to determine the proper plan to follow in the prototype.

22. Report of model tests. The statements made in Section 16 generally apply to any technical report on model tests. Reports on model studies of rivers and harbors usually, of necessity, include more data in order to show the quality of the verification and the relative events of the many proposed plans studied. A general outline for these reports usually includes introductory statements, followed by the main text divided into several parts covering the problem and purpose of the model study, the model, adjustment and verification of the model, tests of proposed improvement plans, and conclusions. Photographs and maps of bed configuration accompany the text. The inclusion of what may at first seem to be excessive data is justified in reports of model studies when the designing engineer and the laboratory which conducted the tests are located some distance apart. Where more direct collaboration is possible, it is believed that only data sufficient to show reasons for conclusions and some intermediate steps are required.

TURBINE MODELS

23. General. Model tests of hydraulic machinery are devoted almost entirely to turbines and pumps, but occasionally tests are made on large regulating gates and more frequently on regulating valves. The increase in the development of power sites by the Government has brought about great increase in the number of turbines and regulating valves. Irrigation and water supply expansion has produced large

installations of pumps. As a result, the manufacturers have had to meet specifications and furnish machinery not only in large quantities but also in sizes not heretofore developed. To meet the specifications established, it has been necessary to continue and to improve their research by model studies. Accordingly, consideration is not only given to the performance and guaranteed efficiency, but tests are made to establish operating criteria necessary for best efficiency and minimum effects of cavitation.

24. Use of model. It has been known for many years that tests of small turbines can be used to predict the performance of larger turbines if the two are homologous. Reduction of such test data to unit head makes it convenient to study the characteristics of any wheel, and at the same time to establish its performance under different operating conditions. When field tests were first made of turbines, however, and their performance compared with studies made of their models or homologous wheels, it was found that the agreement between efficiencies, in particular, was not as close as desired. Accordingly, it was customary for the manufacturers to obtain an "experience" curve, whereby the model predictions of efficiencies were modified sufficiently to assure their satisfying the efficiencies guaranteed for the prototype. In 1925, the Moody step-up formula was developed as an aid in predicting prototype efficiency from the model, which took into account the reduction in losses with increase in size, and thus provided for an increase in prototype efficiency. The results of this relation must be modified, nevertheless, somewhat by "experience" curves. Although this formula applies only to the peak of efficiency of a model, it is customary to obtain efficiencies at off-peak points by adding the increment of efficiency obtained at the peak to the model efficiencies at off-peak points. The Moody formula has been found to give good agreement in complete homologous models from field tests for small and medium diameters, but for larger units the step-up is actually less than that given by the Moody formula.

In turbine testing, it is not necessary to think in terms of a scale ratio or of the Froude Law, which are so frequently thought of in terms of other types of models already discussed. Scale ratios are not selected in the manner previously indicated but are made to follow, that is, it is customary to use approximately an eleven-inch or sixteen-inch runner in turbine models regardless of the size of the prototype, so all other dimensions are reduced according to the diameter ratio. The size of model runners has been selected so as not to require too large a water supply, dynamometer, and appurtenant parts of a turbine. In regard to Froude's Law, it is not used in the form usually considered, that is, the velocity ratio equals the square root of the scale ratio. Rather, all test data of the model are reduced to unit values, that is, the discharge, speed, and power are reduced to those for a one-foot head by dividing by the square root of the test head, the discharge and speed; and dividing by the three-halves power of the test head, the power. In addition, the data may also be reduced to terms of a unit wheel, so there results a characteristic curve for a wheel one inch in diameter operating under a head of one foot. Then for any homologous wheel, it is possible to establish its performance for any head, for the corresponding gate openings and speeds. This procedure has been amplified by the development of the Moody step-up formula using modifications of predicted efficiencies as a factor of safety. Accordingly, the model data is expressed in terms of unit head for the particular size of runner used in the model, then the Moody formula is used with the results tempered by experience and judgment.

Since the efficiency of the model is so vital because of its use in establishing the prototype efficiency, the model should reproduce all water passages of the prototype, that is, geometric similarity is required; the model roughness must be kept to a minimum, and all parts must be carefully machined and installed.

25. Comparison with other types of model tests. The tests performed on models for turbines are more or less standard, that is, characteristics are determined and cavitation studies made, the primary purpose being to develop a high operating efficiency and operation criteria to reduce cavitation. In contrast to hydraulic structure, river, and harbor models, hydraulic machinery models are quantitatively more reliable; furthermore, the efficacy of various designs is not based on merely observation as is done sometimes with the other types of models.

The laboratory and instrumentation required for hydraulic machinery model tests is more elaborate than the equipment needed for model tests of hydraulic structures and river models. Precision and accuracy are more important, since a great deal depends on a variation of efficiencies by even less than one percent. Accordingly, these laboratories usually work for an accuracy of one percent or less. The essential features of such a laboratory include a closed circulating system in which the head on the model may be varied and, at the same time, produce different absolute pressures on the runner blades necessary for producing cavitation in the model. Besides the usual mercury gages for recording pressures and discharge of venturi meters, a dynamometer is required to measure power output and to regulate speed in model turbines.

26. Construction of model. As previously mentioned, turbine models are usually made with wheels between eleven and sixteen inches in diameter. This establishes the scale ratio, which is of little importance. Furthermore, the laboratory is designed to test models within a certain size limit, so it is merely a question of making the model design conform to the laboratory equipment which receives it for testing. As far as practicable, all water passages are accurately reproduced in the model with friction of the model kept to an absolute minimum. Bronze castings, carefully machined, are used for the runner while the scrollcase is made from welded plate steel, the former being shaped to

templates, the latter die-pressed, as is convenient or practical to do so. Draft tubes are either made from transparent material or from plate steel with an observation window provided at the bend of the tube to permit stroboscopic analysis of cavitation and observation of flow. For excellent pictures obtained in this manner, the reader is referred to Transactions A.S.M.E., October 1940, "Cavitation of Hydraulic Turbine Runners," by R. E. B. Sharp.

27. Testing procedure. The procedure in making complete tests of the model is, first, to determine the efficiency characteristics, and then to study the cavitation phenomenon. The efficiency test requires measurement of the head, speed, power output, and discharge. A value of phi, which is the ratio of the peripheral speed of the runner to the theoretical spouting velocity of the water, units in feet per second, is computed from the head and speed readings; the power and discharge are changed to unit values; the efficiency is computed; and these values plotted against phi. When Kaplan or adjustable blade turbines are being studied, the blade angle is also included by making efficiency tests for given blade angle at corresponding gate openings, and for various combinations of head and speed to cover the desired range of phi values corresponding to prototype speed and head conditions. With these data for the given blade angles, the efficiency, power, and discharge characteristics can be obtained for various values of phi by cross-plotting. In these tests, high values of sigma, the cavitation parameter, are tested to eliminate any effects of cavitation.

A study of cavitation in the model establishes a proper turbine setting for the prototype and its operating limits in order to keep cavitation at a minimum. Making use of the fact that cavitation reduces power and increases discharge with a resulting drop in efficiency, the above tests are repeated for each blade angle for adjustable blade propeller turbines or for various gate openings for Francis type runners, by selecting three gate openings for the above tests which bracket the best range of operation for fixed blade runners, or which

bracket the best combination of blade angle and gate for Kaplan turbines, at each of four or more values of phi covering the desired operating range. The same measurements are taken in these tests as for the efficiency tests. At each gate opening, the phi value in question is held constant by varying the headwater and tailwater to change the ratio between draft head and total head. This varies the Thoma cavitation parameter, sigma, which is given as

$$\sigma = \frac{H_B - H_V - H_D}{H_T}$$

where H_B = barometer pressure, feet of water

H_V = vapor pressure, feet of water

H_D = draft head, from center line of runner to tailwater

H_T = total head acting on turbine.

It is seen in this relation that as the draft head H_D increases, while the total head is constant, the value of sigma decreases. By plotting against sigma the unit discharge, unit power, and efficiency for the three selected gate positions, it will be noticed that for higher values of sigma the efficiency, power and discharge are constant, but where sigma is reduced below a certain value the power and efficiency drop off, while the discharge increases. The value of sigma at which this change occurs, is called the critical sigma for a given set of operating conditions, and is the point at which cavitation starts.

From the data so obtained and plotted against sigma for various values of phi and blade angle or gate opening chosen, the values of unit discharge at peak efficiency and the corresponding value of critical sigma are plotted, and lines of constant phi are obtained. Using these curves, lines of constant phi, it is possible to determine the proper elevation of the turbine above the tailwater and also to determine the safe load which can be carried at heads other than the rated head. After it has been decided what the prototype

turbine diameter, speed, and maximum output at rated head will be, the unit discharge corresponding to the maximum output at rated head is determined and used in the above phi curves to establish the critical sigma for the value of phi at the rated head. Substituting this value of critical sigma into the Thoma relation for sigma, the allowable draft head may be computed. As a factor of safety, it is customary to set the turbine runner a certain amount below this value, say one to three feet.

In practice, it is customary to use two values of sigma, one for the turbine, just described, and one for the plant, which is the effective sigma computed for the installed unit. This latter value, as can be seen from the Thoma relation, varies with the headwater and tailwater, the barometer pressure, and vapor pressure. If the plant sigma is ever less than the critical sigma determined from the model, cavitation would develop. Hence, after the turbine setting has been determined, the plant sigma values are computed for the various operating conditions expected; and from the curves of constant phi, established from the model tests, the allowable unit discharge is obtained from which is computed the allowable power output. If these data are plotted to give constant power lines for headwater and tailwater elevation, the plan operations can be guided to operate the units correctly.

It is customary in some laboratories to use the break in the unit discharge-sigma curve to obtain critical sigma. If, on the other hand, the efficiency and unit power curves break so as to give a higher sigma than the unit discharge curve, the higher value is used, together with the unit discharge for that value. When such a condition as this occurs for a particular runner, it is believed that local cavitation is occurring which is not severe enough to affect the unit discharge until a lower value of sigma is reached. This failure of the sigma break to coincide is indicative of improper design at some point,

which induces local cavitation to take place ahead of the general cavitation which affects the unit discharge break.

General cavitation is the result of the lowering of pressure in a turbine until the pressure at some point, or over a certain area of the turbine blades, is equal to the vapor pressure of the water. When this occurs, general cavitation is unavoidable regardless of how well the turbine is designed. But from the model test, it is possible to set the runner at a correct elevation to prevent general cavitation, or make its occurrence remote. Local cavitation, on the other hand, is exceedingly difficult to eliminate altogether. Experience has shown that improper design will permit local cavitation even though the pressures in the turbine are above the critical. For example, local cavitation has been caused by too abrupt curvature of the blade surfaces, reversals in curvature in the direction of flow, roughness of small obstructions, excessive runner clearances, poorly designed wicket gates, gates overhanging the throat ring at or near full opening, improper shape of stay vanes or vanes at improper angle to the flow, and poor design of curb ring, head cover, and scrollcase. With all these possible errors, together with the fact that it is not always economical to use all of the margin between plant and turbine sigma required, it is necessary to preweld portions of the runner with 18-8 stainless steel, particularly for units operating under heads in excess of 50 feet.

It is interesting to observe that each manufacturer usually has his own testing laboratory and sealously guards his design data and results of model tests. Competition among turbine and pump manufacturers requires much secrecy.

PUMP MODELS

28. General. As with turbine models, current engineering practice demands that the performance of pumps should be studied by testing a model pump, geometrically similar. The use of the model data to predict

prototype performance is, as in turbines, referred to unit speed, unit discharge, but specifies speed, instead of phi. The flow conditions in geometrically similar pumps are, according to Froude's Law, similar in similarly located points, thus maintaining dynamic similarity. The Reynolds number is, therefore, not the same in the prototype and model. Here again, it is considered that the influence of viscosity and the relative roughness between model and prototype are not of particular significance, but relative roughness does influence the setup of efficiencies as taken into account by the Moody relation, which is also used in the step-up of model pump efficiencies.

In turbine models, it was noted that tests in the laboratory are made in such a way that the range of phi in the model is the same as in the prototype; in pump tests the specific speed is used instead, it being defined as the speed (rpm) of a pump of such a size that it will deliver one cfs (or one gpm) under a head of one foot. In the model tests the model head may or may not be the same as the prototype head; other factors, such as discharge and speed, being adjusted to give the correct values of specific speed, which means the model pump is run much faster and with less discharge than the prototype pump. For any series of tests, the head and capacity at which the maximum efficiency is obtained are used for computing the specific speed of the model pump.

29. Use of model. In large pump installations such as those for the Colorado River Aqueduct and the Grand Coulee Dam, there are no similar installations which may be used as a guide in the design, which is usually possible on smaller installations. Accordingly, the designing engineer wants to know the following: (1) Should the pump be centrifugal (double or single suction) or of the propeller type; (2) what is the proper rotation speed (specific speed); (3) intake pressure to avoid cavitation; (4) performance of pumps if power is suddenly shut off; (5) control equipment necessary;

(6) efficiency to expect; (7) changes necessary in proposed design to improve efficiencies. When resort is made to model tests, a reasonable assurance is given to the engineer that his questions have been properly answered. The pump efficiency is of vital importance to the owner regarding the cost of operation over a period of years, as well as the manufacturer supplying the pump regarding any bonus based on efficiency.

30. Cavitation tests. The pump setting and operation must be carefully selected to eliminate, as much as possible, the effects of cavitation not only on the machine itself, but also on its efficiency over its normal operating range.

In a manner similar to the cavitation tests previously described for turbines, cavitation tests are made on pumps by operating them at a constant capacity and speed while the inlet head is decreased gradually until cavitation begins. This is noticed by abrupt drops in efficiency, head, and discharge. From such studies, the necessary inlet pressures are derived which automatically determine the excavation necessary for best operation, or checks may be made on settings already established. In these and the characteristic tests, it is necessary to measure speed, torque, inlet and discharge pressures, and rate of flow.

The cavitation characteristics of a pump are also correlated with the Thoma parameter sigma. Extending cavitation test results to other similar pumps may be accomplished quite readily if both model and prototype operate under the same pump head. Thus, H_s, the suction head, is the same for both thereby obtaining the same conditions with respect to cavitation. In this step-up, with both model and prototype operating under the same head, all machine and fluid velocities are equal to each other at similarly located points in the two pumps, while all water passages are changed in the model according to one fixed ratio. In another case, a generalization of the first and the more common type, occurs where, in addition to changing

the water passages according to a certain ratio, all velocities at similarly located points are changed according to another fixed ratio. Thus similarity, both geometric and dynamic, exists. It is only under this condition that simple and reliable conclusions can be drawn for the performance of geometrically similar pumps or of the same pump under different operating conditions.

In order to include cavitation characteristics under this more rigorous similarity, it is logical to expect that cavitation will commence in the fluid in the pump when at some point the pressure has reached the vapor pressure of the fluid. If this is true, the difference in total head between the section pipe of a pump (corrected for elevation) and the point of vapor pressure, must equal the net suction head, H_s, (the total suction head above the vapor pressure of the fluid including the velocity head in the suction pipe). According to the similarity of flow conditions, all pressure or head difference inside of geometrically similar pumps will change proportionally to the square of the velocities as long as the flow conditions remain similar. Since with cavitation, the net suction head, H_s, actually exists as a head difference inside of the pump, then in geometrically similar pumps, H_s is proportional to the square of the velocity (any suitably chosen fluid velocity in the pump). Likewise, the total head, H, is proportional to the square of the fluid velocitites in similar machines. Thus, H_s and H are proportional to each other as long as the flow conditions are similar; but for geometrically similar pumps the specific speeds are the same, if flow conditions are similar. Hence, if the Thoma parameter, sigma (ratio of H_s to H), and the speed of the pump are held constant, similarity is maintained with respect to cavitation, so test results can be extended from one pump to a similar pump.

The final case of extension of cavitation test results deals with similar pumps but with different specific speeds. This is a matter of finding some theoretical approach for calculating changes of sigma as

a function of the specific speed. So far, a rigorous solution has not been possible, but resort may be had to an approximation in which a "suction" specific speed is used, $S = \frac{N\sqrt{Q_{3/4}}}{H_s}$; and for extension of cavitation test results, S is held constant for pumps with similar inlet passages but of different specific speeds. This is analogous to holding sigma and specific speed constant.

31. Model construction and laboratories. Model pumps are usually made by the manufacturer according to standard practice in use today. Compared to the relatively simple wood and sheet metal construction of other types of hydraulic models, pump and turbine models must be made of castings, carefully machined, thus making these models more expensive.

Laboratory facilities for testing pumps are indeed extensive and require careful planning and utmost accuracy. Probably one of the best equipped pump testing laboratories is found at the California Institute of Technology. For a description, the reader is referred to an article by R. T. Knapp, "The Hydraulic Machinery Laboratory at the California Institute of Technology," Transaction A.S.M.E., November 1936.

32. Bibliography. The following is a list of references applicable to Chapter II:

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CHAPTER III - HYDRAULIC MODEL TESTS

INTRODUCTION

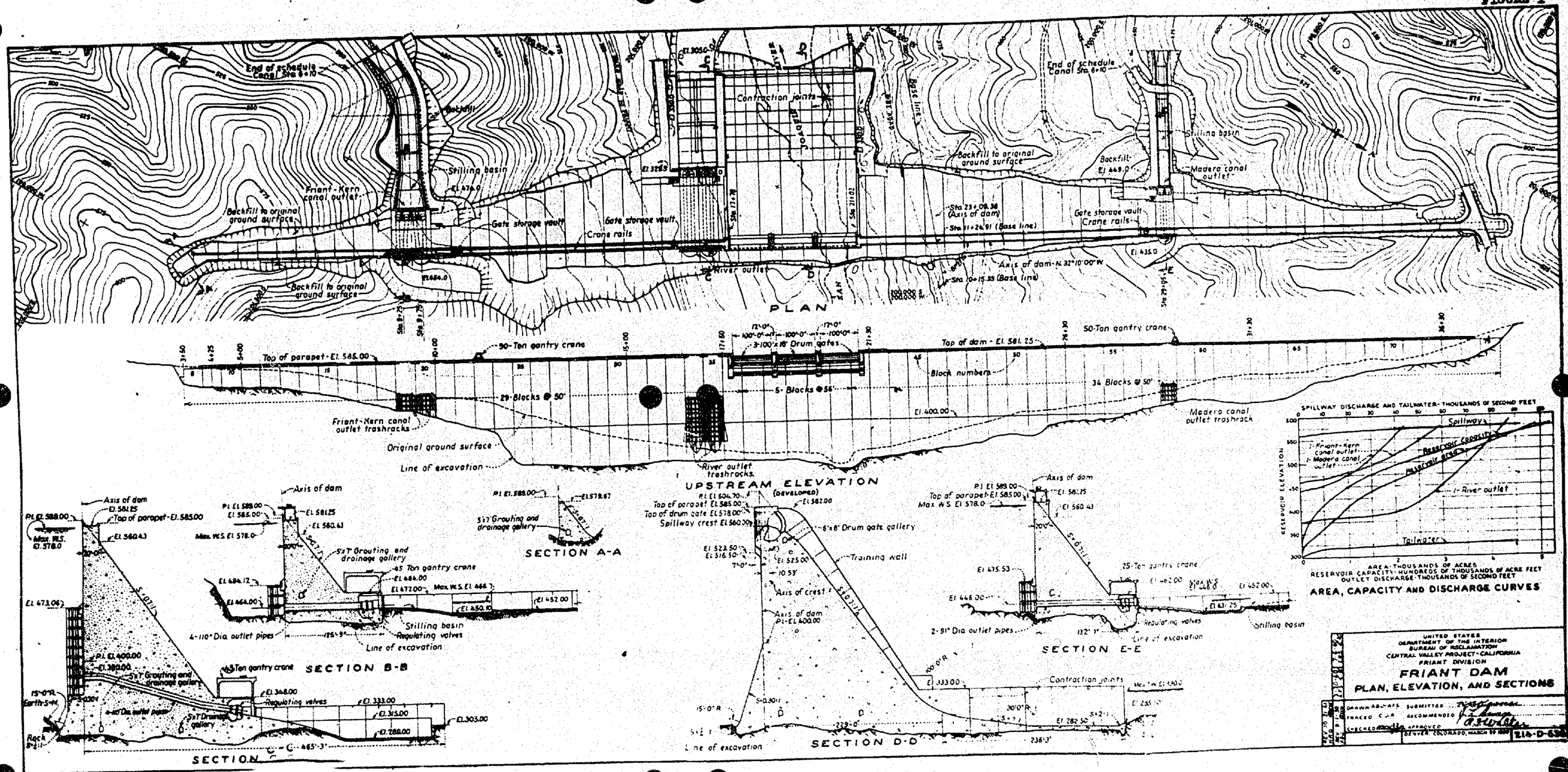
33. General. In the foregoing chapter, emphasis was placed on the basic considerations in adapting a model to a problem, the model design and construction, and testing procedure. In this chapter, examples of actual model tests will be given, as observed in some of the hydraulic laboratories in the United States, to illustrate this type of research, as well as to show the nature of some of the problems confronting the hydraulic engineer today.

FRIANT DAM

34. The prototype. This model study is selected because it is representative of the studies made in the hydraulic laboratory of the Bureau of Reclamation, Denver, Colorado, in connection with the Bureau's program of constructing large dams in the West for irrigation projects. This study is also selected because of the many different models which were made and tested before all the hydraulic problems could be solved. It is necessary to be brief in reporting on this model study, so much of the detail will be necessarily omitted, but enough material will be given to show the laboratory practice involved and the results of the tests.

The Friant Dam, on the upper San Joaquin River near Fresno, California, is the fourth largest dam built by the Bureau and is the fourth largest in the United States. It is one of the major structures of the Central Valley Project, being 286 feet high and 3,300 feet long, of the straight gravity type (Figure 1). The water stored in the reservoir will supply the Friant-Kern and Madera Canals, which have their headworks at the dam. An overflow spillway controlled by three 100- by 18-foot drum gates is located in the center section of the dam, the maximum capacity of the spillway being 90,000 second-feet.

FIGURE 1


PLAN
End of schedule Canal Sta. 6+10
Backfill
Contraction joints
Stilling basin
Friant-Kern canal outlet
Backfill to original ground surface
Gate storage vault
Crane rails
River outlet
Sta. 23+09.38 (Axis of dam)
Sta. 11+24.91 (Base line)
Axis of dam-N. 32°10'00"W
Sta. 10+15.33 (Base line)
Madera canal outlet
UPSTREAM ELEVATION
Top of parapet-El. 585.00
50-Ton gantry crane
3-100'x18' Drum gates
Top of dam - El. 581.25
Block numbers
29-Blocks @ 50'
5-Blocks @ 56'
34 Blocks @ 50'
Friant-Kern canal outlet trashracks
Original ground surface
Line of excavation
River outlet trashracks
Madera canal outlet trashrack
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
Axis of dam
Top of parapet-El. 585.00
Max. W.S. El. 578.0
5'x7' Grouting and drainage gallery
45 Ton gantry crane
4-110" Dia. outlet pipes
Regulating valves
Stilling basin
Rack
Training wall
6'x8' Drum gate gallery
Top of drum gate El. 578.00
Spillway crest El. 560.00
Axis of crest
2-91" Dia. outlet pipes
25-Ton gantry crane
SPILLWAY DISCHARGE AND TAILWATER-THOUSANDS OF SECOND FEET
RESERVOIR ELEVATION
Spillway
Reservoir capacity
Reservoir area
Friant-Kern canal outlet
Madera canal outlet
River outlet
Tailwater
AREA-THOUSANDS OF ACRES
RESERVOIR CAPACITY-HUNDREDS OF THOUSANDS OF ACRE FEET
OUTLET DISCHARGE-THOUSANDS OF SECOND FEET
AREA, CAPACITY AND DISCHARGE CURVES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT-CALIFORNIA
FRIANT DIVISION
FRIANT DAM
PLAN, ELEVATION, AND SECTIONS
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO, MARCH 29, 1949
216-D-638

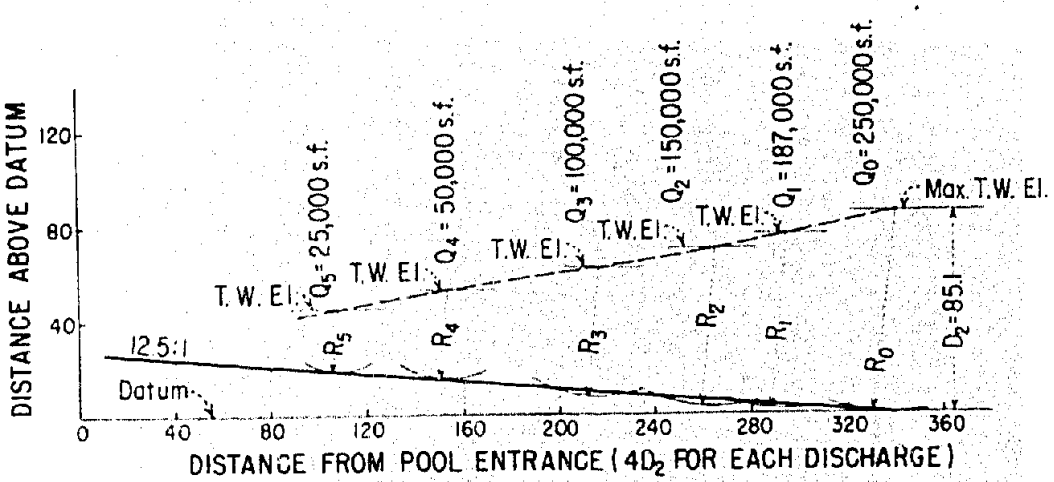
35. Crest calibration, 1:25-scale model. The first model was built to study the discharge capacity of the crest with the drum gates at various positions and to obtain pressures on the crest and drum gates, the latter to be used for estimating hinge-pin reactions. This model was built to a scale of 1:25 including only a 50-foot (one-half) section of one gate. The model was constructed of angle-iron frames covered with sheet metal, and placed in a metal-lined flume two feet wide and six feet deep. The gate was made of iron bents covered with sheet metal. A head-gage well was attached to the flume and piezometers were installed in the gate and crest to obtain the desired pressure distribution on a given profile and for the gate in several raised positions.

It was concluded from these tests that there would be no negative pressures on the crest profile for discharges up to and including the maximum of 90,000 second-feet; the spillway crest will have adequate capacity for handling the design discharges; and that the model calibration curves should prove helpful in determining the magnitude of floods passing the dam after its completion.

36. Stilling-pool, 1:24 scale model. The second model of the Friant Dam was of the sectional type built to a scale ratio of 1:24. The purpose of this model was to design in the laboratory a satisfactory hydraulic jump stilling pool at the toe of the dam. As mentioned in Chapter II, it is frequently necessary to use large sectional models instead of complete models because of the size of the prototype, the laboratory space available, and because more accurate results are possible on large models, since flow conditions are more readily observed as to effect of changes in design. This model represented a prototype width of 90 feet equally divided on each side of one 12-foot pier. Since the stilling pool was the only concern, the crest was built to correct shape omitting the gates. A large head-box was used as a reservoir with the crest section cut into one side near the top of the box. The spillway attached to the crest

was made from timber bents covered with sheet metal, and extended to the flow of another metal-lined box containing the stilling pool which was formed from sheet metal bents and covered with sheet metal. The flume or box containing the stilling pool had one side made with glass panel to permit observation of the floor.

Because the agreement between the tailwater and the height of water necessary to form an efficient hydraulic jump was lacking, resort was made in the study to the use of a sloping apron in the stilling pool floor. This expediency produced an approximate but close agreement between the tailwater curve and the jump-height curve. To obtain the correct slope of the stilling pool floor, values of d_2, or height required for a jump, are computed for several discharges, including the maximum, from the well-known momentum formula of the hydraulic jump. Then assuming the length of jump is $4d_2$, each value so obtained is laid off as abscissas from a reference point. Above each point is plotted the corresponding value of tailwater establishing the lines of tailwater elevation for each discharge selected. Then at each of these points as a center, an arc is swung having a radius of the corresponding d_2 computed. A line tangent to these arcs gives the slope of the apron as shown by Figure 2, which illustrates a similar procedure used for the apron at Shasta Dam.



NOTE

R_0, R_1, R_2, R_3, R_4 and R_5 are equal to the values of D_2 for the respective discharges, Q_0, Q_1, etc.

DESIGN OF SLOPING APRON, SHASTA DAM

FIGURE 2, SLOPING APRON

The point of intersection of this slope with the slope of the face of the dam must be at least 0.20 d_2 above the lowest point of the apron (See also Section 106, Chapter V).

Several tests were made at various discharges to observe the hydraulic jump performance without any sill at the end of the apron, and it was found that the computed slope of 7:1 was satisfactory. Further tests determined the proper length of horizontal apron required below the sloping apron, and the size sill to be used at the end of the apron to eliminate scour to the stream bottom. In these tests, progressive scour tests were made, that is, the profile of the channelbed was taken by a point gage at certain time intervals over a range of discharges. The required length of apron,

including the sloping part, was found to be 250 feet measured from the intersection of the face of the dam and the sloping apron, for a maximum discharge of 90,000 second-feet, as shown by Section D-D, Figure 1. A solid sill located at the end of the apron 10 feet wide and 3.25 feet high with an upstream slope of 3:1 was found to give excellent prevention from scour. Figure 3 shows the hydraulic jump at the maximum discharge of 90,000 second-feet.

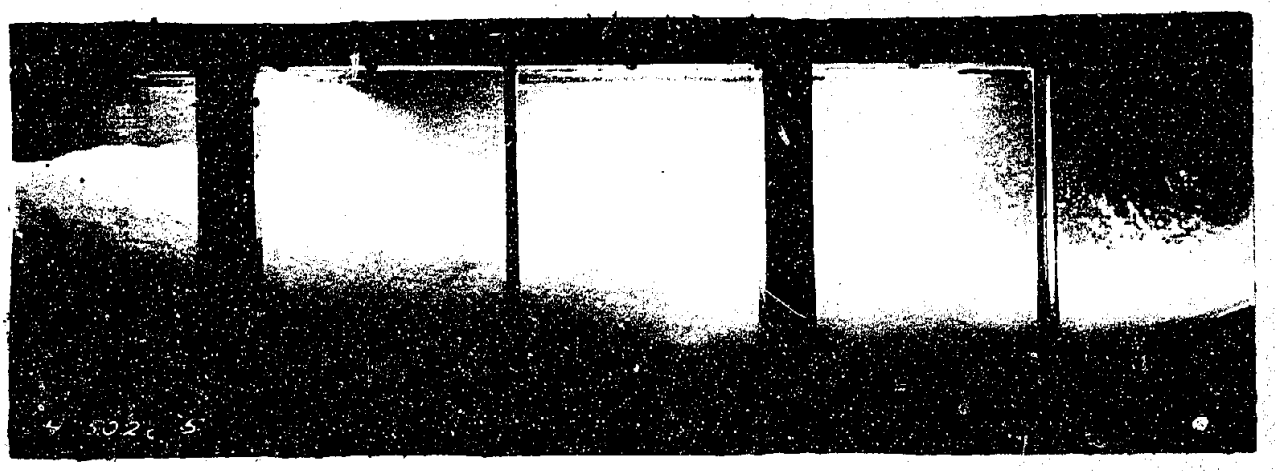


FIGURE 3, HYDRAULIC JUMP ON SLOPING APRON

A typical scour test consists of establishing a profile in the sandbed downstream from the apron, representing the prototype excavation. The model is then operated over a range of discharges; at the end of each discharge (usually run for 45 minutes each), a profile of the sand is taken with a point gage, and photographs taken of each discharge condition in the stilling pool. As types of sills, and lengths and slopes of aprons are varied, comparison between each design may be made from the sand profiles which indicate relative scour, and from visual observations of the stilling pool. Frequently, velocity measurements are required to give a better comparison of different designs.

37. River outlets, 1:24-scale model. The next model was of the sectional type, 1:24-scale, built to test a design of one of the proposed 102-inch diameter river outlets which were to pass through the dam and discharge into the main overfall spillway (Figure 4). These outlets, four in all, would operate to supply water to the San Joaquin River and to pass excess flow in addition to that passed by the spillway. The discharge in each outlet was to be controlled by tube valves at the upstream end, the maximum discharge being 4,100 second-feet through each outlet under a maximum head of 220 feet.

In the design of these outlets, the end of the outlets was placed only a short distance above the stilling pool; this required that the jets be spread laterally very quickly so as to spread across the entire pool to form a hydraulic jump. If the flow from the outlets is not spread, but is allowed to enter the pool as more solid jets, severe eddies may form, and these, together with excessive surface and bottom velocities, may cause undesirable flow conditions in the pool and immediately downstream.

From a study of somewhat similar outlets for the Marshall Ford Dam in Texas, it was found that the best way to spread a jet as required at Friant Dam was to place a transition section in the very

A typical scour test consists of establishing a profile in the sandbed downstream from the apron, representing the prototype excavation. The model is then operated over a range of discharges; at the end of each discharge (usually run for 45 minutes each), a profile of the sand is taken with a point gage, and photographs taken of each discharge condition in the stilling pool. As types of sills, and lengths and slopes of aprons are varied, comparison between each design may be made from the sand profiles which indicate relative scour, and from visual observations of the stilling pool. Frequently, velocity measurements are required to give a better comparison of different designs.

37. River outlets, 1:24-scale model. The next model was of the sectional type, 1:24-scale, built to test a design of one of the proposed 102-inch diameter river outlets which were to pass through the dam and discharge into the main overfall spillway (Figure 4). These outlets, four in all, would operate to supply water to the San Joaquin River and to pass excess flow in addition to that passed by the spillway. The discharge in each outlet was to be controlled by tube valves at the upstream end, the maximum discharge being 4,100 second-feet through each outlet under a maximum head of 220 feet.

In the design of these outlets, the end of the outlets was placed only a short distance above the stilling pool; this required that the jets be spread laterally very quickly so as to spread across the entire pool to form a hydraulic jump. If the flow from the outlets is not spread, but is allowed to enter the pool as more solid jets, severe eddies may form, and these, together with excessive surface and bottom velocities, may cause undesirable flow conditions in the pool and immediately downstream.

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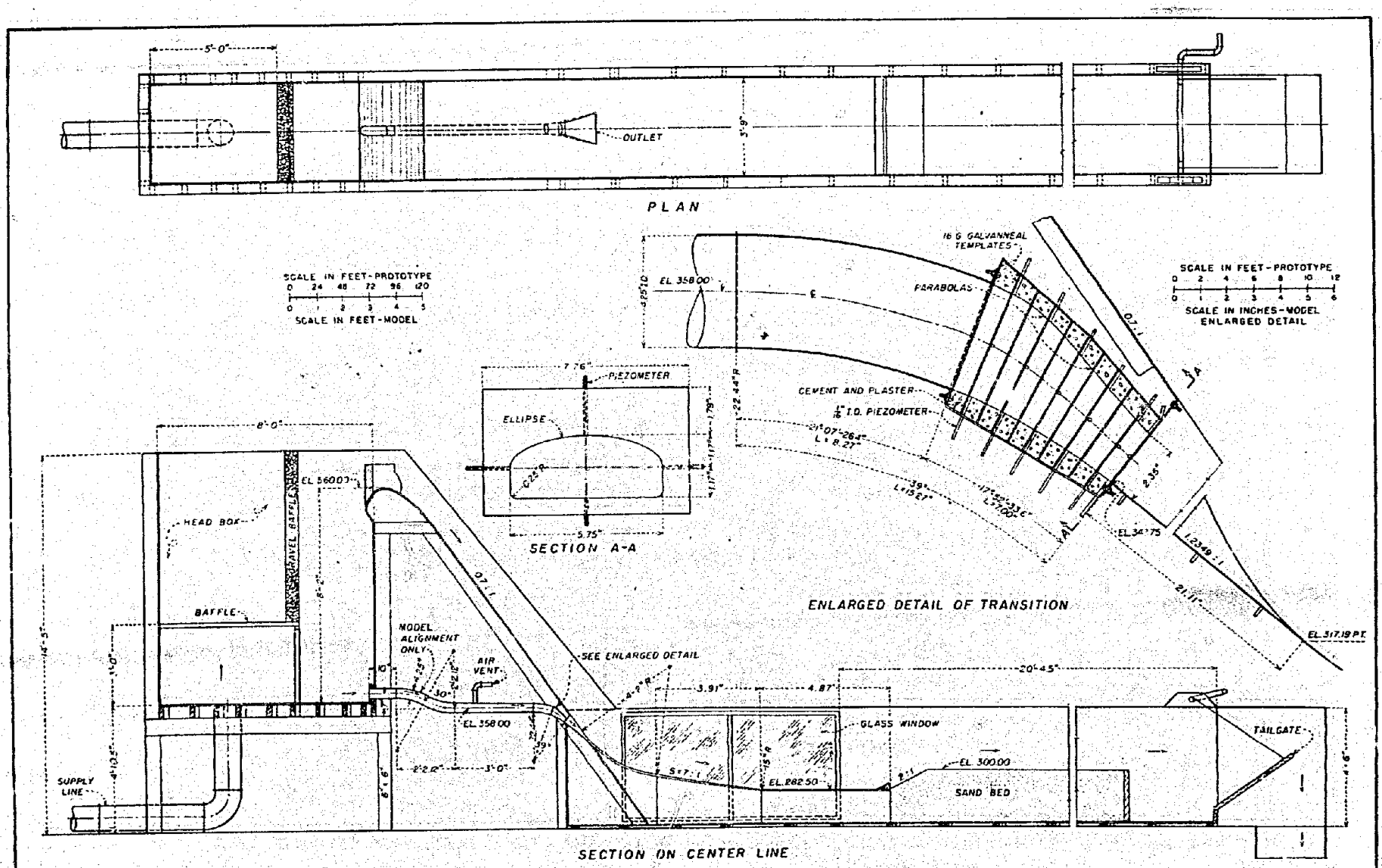


FRIANT DAM HYDRAULIC MODEL STUDIES - 1:24 SCALE
GENERAL PLAN AND SECTIONS - RIVER OUTLETS

FIGURE 4

HYD 110

downstream end of the outlets followed by an exit trough or "beaver tail" cut into the face of the dam. In addition, the transition must be so shaped that the flow starts to expand under pressure within the outlet, otherwise no expansion or spread would occur. Furthermore, the outlet end of the transition must have an area less than that of the normal conduit to maintain positive pressures within the conduit and transition. Accordingly, the outlet transition was reduced to give an area at the extreme end section of 85 percent of the area of the 102-inch outlet. The transition as finally designed is shown on Figures 4 and 5.

FIGURE 5, TRANSITION AND TROUGH

It will be noticed that its upstream section is a circle of 102-inch diameter and in a length of 14 feet the section changes to form an ellipse in the upper half and a rectangle in the lower half of the section. The crown is composed of two parabolas, and the invert is composed of the arc of a circle. The trough or "beaver tail" in the face of the dam diverges and its side walls are made tangent to the horizontal projection of parabolas forming the side walls of the transition.

A model of one of the four outlets was installed in the previously described model used for studying the hydraulic jump stilling-pool (Figure 4). The transition was formed by cutting out of sheet metal several section templets and fastening them on a base plate, as shown on Figure 4. Piezometers (1/16-inch inside diameter) were attached to each templet section and were made flush with the inner surface of the transition, which was formed by placing cement-plaster mortar between the templets and screeding to a smooth surface. Additional piezometers were placed in the floor of the exit trough.

The model studies of this transition were made to determine: (1) The pressures within the transition under different heads; (2) the spread of the issuing jet on the apron, and effect of spillway flow across the exit trough; (3) the velocity distribution in the flow at the extreme end section of the transition, at the end of the exit trough in the face of the dam, and on the horizontal portion of the apron; and (4) the calibration curves for the transition, treating it as an orifice.

From the results of the tests on the transition, it was found that: (1) The pressures within the transition, although not all positive, were satisfactory, assuring no bad effects from cavitation; (2) the spreading of the issuing jet was entirely satisfactory with only a small fin occurring along the side walls of the exit trough, and excellent spreading occurring on the sloping apron of the dam (Figure 6). This demonstrates conclusively the importance of starting

the jet to spread in the transition and under pressure; (3) regarding the effect of spillway flow passing over the exit trough, it was found that no serious disturbance occurred even though no aeration existed when the outlet was closed; nevertheless, aeration was recommended. It was concluded that in any design of this type, when the outlets are closed, the spillway flow is deflected from the exit trough and then springs away somewhat from the face of the dam. Accordingly, the higher the outlets are on the face of the dam, the more pronounced is the disturbance caused by the deflected water before it enters a stilling-pool. In this regard, for the outlets on the Grand Coulee Dam and Shasta Dam, it was necessary to place streamlined deflectors or "eyebrows" on the face of the dam immediately above the end of the outlet; (4) the velocity distribution

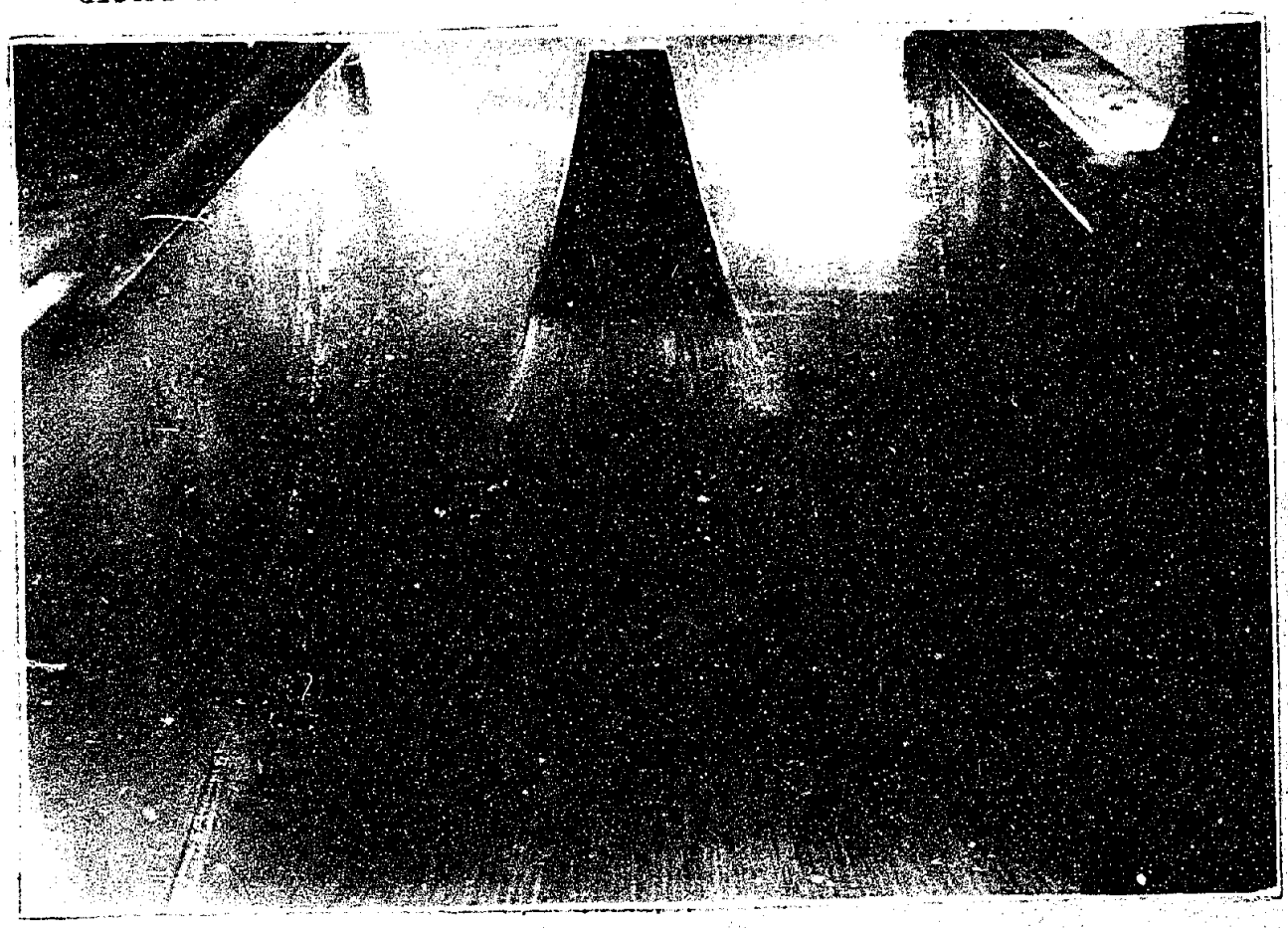


FIGURE 6, JET SPREADING ON APRON

at the downstream end of the transition, at the end of the exit trough, and on the apron is fairly uniform, but slightly higher on one side than the other. The prototype velocities would vary from 100 to 115 feet per second at the end of the transition; and (5) the coefficient C, in $Q = CA\sqrt{2gh}$, where H is the head above the centerline of the end of the transition, and A, the area at the exit, varied from 0.80 to 0.85 for a prototype head varying from 35 to 230 feet. This was for no upstream control, such as a valve or gate, in the outlet.

Although this study of the river outlets was successful on the sectional model, the type of design proposed had to be abandoned for two reasons: (1) It would be impossible to obtain close regulation of the discharge by tube valves upstream in the 102-inch conduit because of excessive negative pressures developed in the valve and even in the conduit below the valve; and (2) as later discovered on a 1:60-scale complete model of the dam, it was difficult, though not impossible, to make the issuing jets from the outlets form a uniform hydraulic jump in the stilling-pool at the toe of the dam.

38. Study of general performance, 1:60-scale model. The purpose of this model was to study the performance of the river outlets; to check the calibration for the crest and drum gate made on the 1:25-scale model, the scour performance of the hydraulic jump stilling-pool, and the separate river outlet stilling-pool which was finally used instead of the river outlets passing through the dam. The discussion pertaining to the change in design of river outlets follows in Section 39.

The model for studying the general performance consisted of all of the spillway, stilling-pool, and sufficient river topography to assure a proper scour study. It was constructed from sheet metal bents covered with sheet metal. The crest section, resting on a shelf cut into the downstream side of the head box, was made from sheet metal ribs soldered to a base plate which was stiffened by angles. Each drum gate, also made of sheet metal, was operated by a gear and rack so that the gate could be manipulated from one side of the model. The topography was

placed by using vertical pegs soldered to the metal lining in the downstream box. Sand was then added and shaped to just cover the top of these pegs. The tailwater was controlled by a movable tailgate and the tailwater elevations were read at a manometer tube on the side of the box connected by a pipe to the center of the model topography. The river outlets passing through the dam were made of brass tubing, the downstream transition being made from templets contained in a metal box; the shape desired in the transition was obtained by filling with cement mortar in between the templets. No piezometers were placed since the pressures had been studied in the 1:24-scale model of the outlets.

The first problem studied in this model was the performance of the issuing jets from the four river outlets into the stilling-pool. It was immediately seen that the jets, although spreading somewhat as expected, did not spread laterally enough to form a uniform jump in the pool. Instead, the jets, after they plunged into the tailwater, were unraveled and diffused by the tailwater and prevented from spreading by the hydrostatic pressure in the pool. It was observed that because of this improper spreading, severe eddy action developed in the pool and downstream in the river channel. These eddies carried sand onto the apron in the model, as shown by Figure 7; such a condition in the prototype would eventually cause erosion not only to the concrete, but also to the riverbed itself. Attempts to isolate each jet by training-walls in the pool was tried. This was not too successful and was objectionable because of the cost, and because of the impact effects of the spillway flow entering the pool on the training-walls. This design was therefore abandoned, partly for the reasons just mentioned, but for the most part, because of the inability to regulate the outlet flow by tube valves in the upstream end of the outlets due to excessive negative pressures which were found to exist in the tube valves and outlets below the valves in a similar design being studied for the

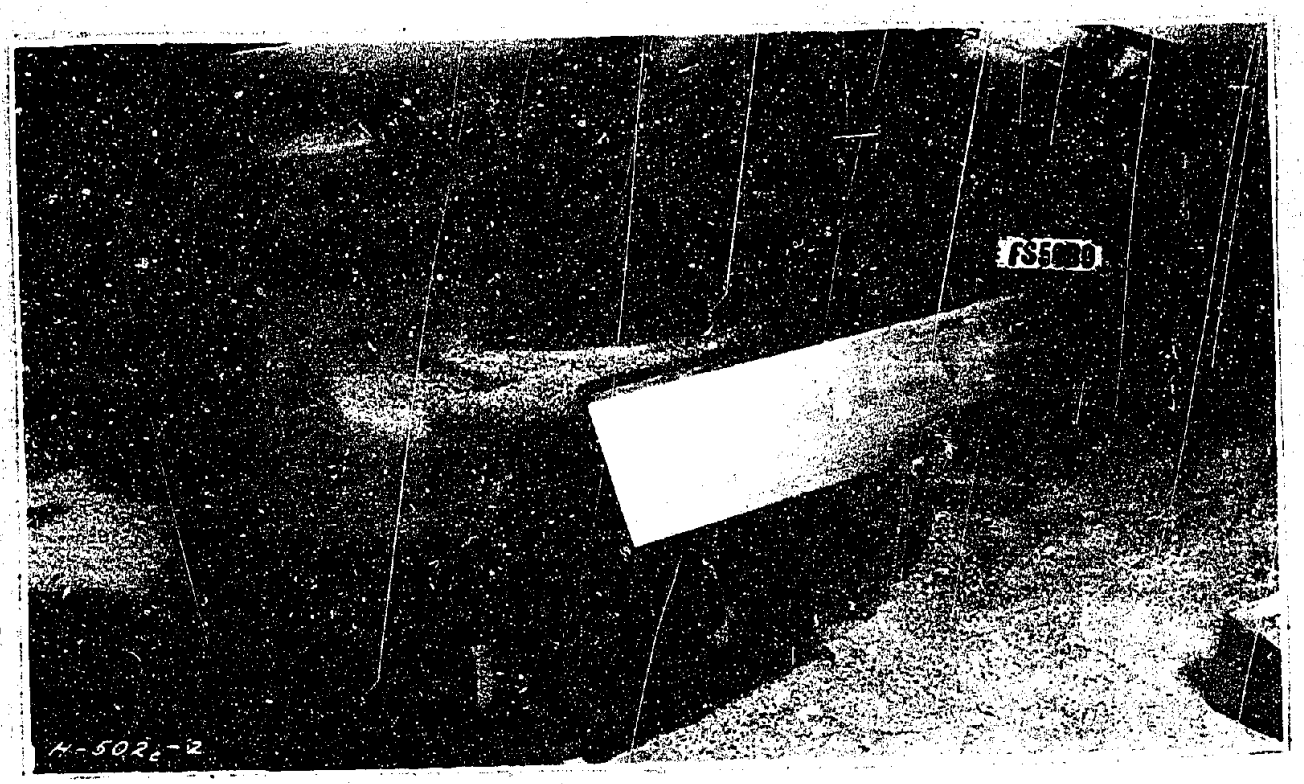


FIGURE 7, SAND ON APRON

Shasta Dam. Time did not permit a study to improve such a design for the Friant Dam, but a successful tube valve design has been subsequently obtained for the Shasta Dam outlets. Before abandoning the river outlets through the dam, horizontal outlets were tried through the dam. The flow left the outlets as solid jets at the face of the dam and followed a trajectory into the pool. This also produced heavy erosion to the riverbed and carried material onto the apron.

Tests made to check the hydraulic jump for the spillway stilling-pool previously designed on the 1:24-scale model, indicated the design to be entirely adequate as shown on Figure 9B. Water surface and sand profiles were recored with a point gage and compared with the 1:24-scale

model test. Excellent agreement was obtained not only for these tests, but for the check on the calibration study made on the 1:25 model. Tests were then discontinued on the 1:60-scale model until a larger-scale model had evolved a separate stilling-basin for the river outlets.

39. River outlet stilling-pool, 1:34.375-scale model. When it was realized that it would be impractical to use the river outlets through the dam, regulated by tube valves, it was decided to construct a separate stilling-pool to the left of the main spillway for the outlets. The regulating valves would be placed at the end of the outlets, enabling them to discharge directly into the atmosphere. Each valve was to be a 110- by 96-inch needle valve, capable of discharging about 4,500 cfs each under a maximum head of 220 feet. The original design proposed that the valves be placed 18 feet on centers and each pair separated by 32 feet to conform to the block construction of the dam. Starting at the invert of each valve at the exit flange, was a mildly sloping apron extending 165 feet downstream to a horizontal apron 10.75 feet below the invert of the valves and extending 86.25 feet downstream. At the end of the apron, a large sill 14 feet high was placed, to force a jump into the pool and maintain it there, since the maximum tailwater for the maximum discharge in the river channel was one foot below the top of the sill. This condition meant that the sill would cause sufficient depth of water for a jump to form in the pool, then the water would plunge over the sill to the tailwater in the exit channel leading to the river. A training-wall placed between each pair of valves permitted operation of individual valves. The model of this design was installed in the boxes used for the sectional model studies. The valves used had been made two or three years before for another study, but they were well suited for this problem. Since the scale ratio had to be based on the ratio of inlet diameter in the prototype and model, the scale ratio of 1:34.375 was established. Each valve was connected by rubber hoses to sheet metal pipes connecting to the head box. Wooden bellmouth entrances were slipped over these pipes

in the head box, and a heavy steel stand was used for supporting the valves. It had a tilting top enabling the valves to be pointed downward from the horizontal. The stilling-pool itself was made of wood to enable changes to be readily made. Heads on the model were read by piezometers connected to the head box, and discharges were measured by venturi meters in the water supply line.

The original design proved to be entirely inadequate. The jets from the valves repelled the tailwater in the pool and impinged directly against the sill at the end of the horizontal apron. At small discharges, however, a jump would form but the toe of the jump was continually being repelled by the jets from the valves. At larger discharges, with the jump repelled, the flow plunging over the large end sill was in itself another problem in energy dissipation. Accordingly, this design was inadequate and was tried only because an effort was made to save rock excavation, which did not provide a pool of sufficient depth.

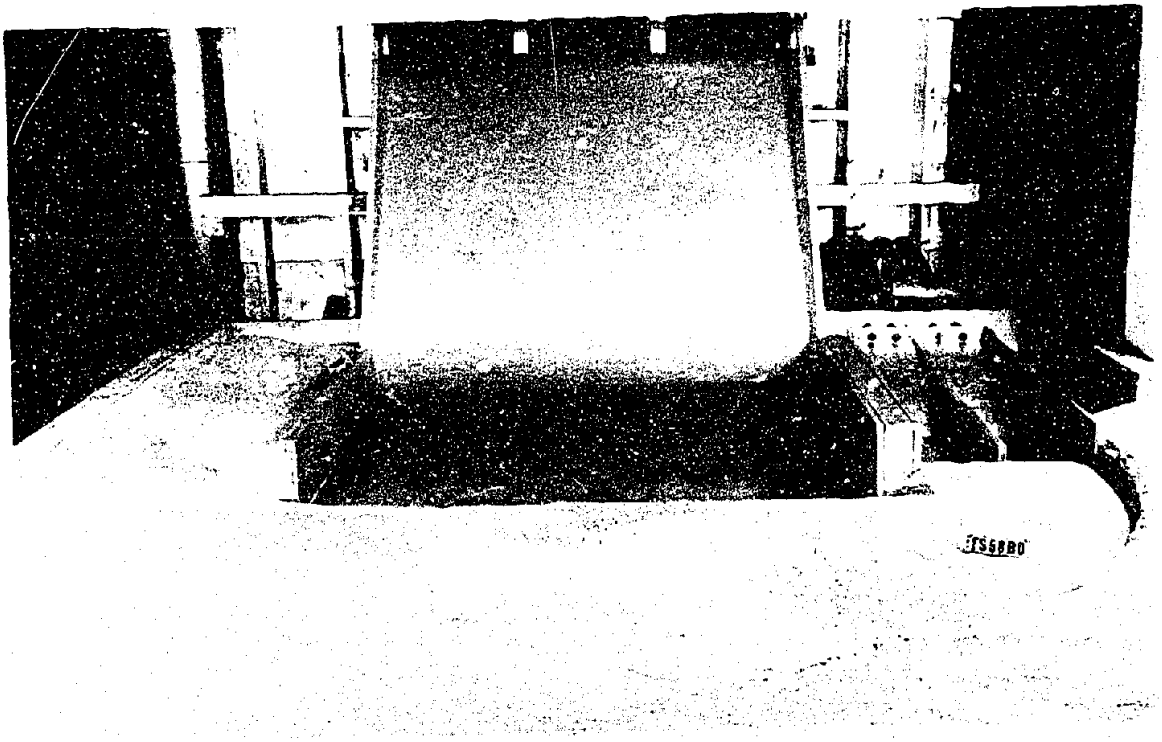
To make the pool deeper, but at the same time not excessively long, the four valves were tilted ten degrees downward. The depth of pool was revised to give the proper depth of tailwater for a hydraulic jump, and an apron on a ten degree slope was added from the end of the valves to a horizontal floor. This design was an improvement over the original design, but it was necessary to experiment with the sloping floor extending from the valves to the bottom of the pool. Tests were made using a parabolic apron of curvature flatter than the prajectory of the valve jets; this caused the jets to be spread uniformly across the apron as they entered the tailwater in the pool. This curvature of the apron was an important factor influencing the stilling-pool performance. If the apron were too steep in curvature, the jet would not spread sufficiently, and if too flat, would cause excessive length of structure and fins springing up between the jets just as they plunged into the tailwater. The final design pool is shown by Section C-C, Figure 1, and on Figure 8. The two needle valves

adjacent to the main stilling-pool were changed to tube valves of corresponding size.

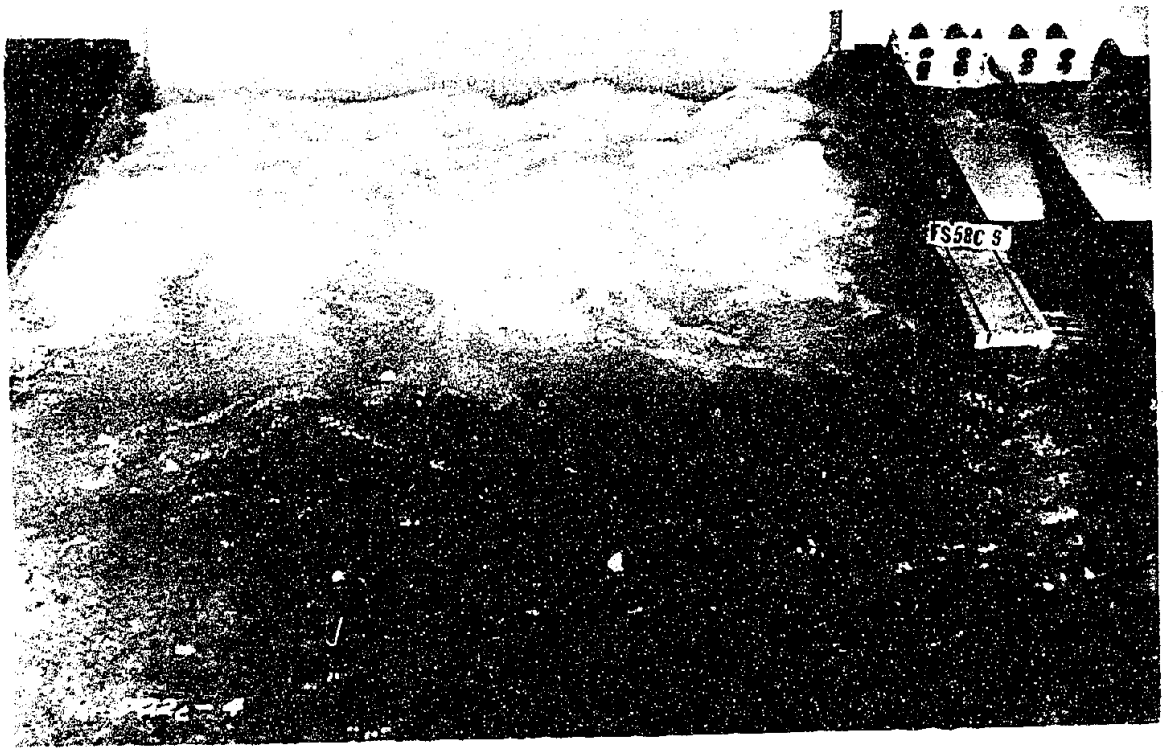
Because this study was confined to the river outlet stilling-pool only, a more detailed study had to be made of the pool performance on the complete 1:60 model. Here it was possible to plan the excavation necessary below the river outlet pool to the river channel proper. This may be seen on Figure 9A, which shows the 1:60 scale model of the final design.

40. Madera and Friant-Kern Canal headworks. The next models studied in connection with the Friant Dam were those pertaining to the stilling-pools provided on each side of the dam for the Madera and Friant Canal headworks (Sections E-E and B-B, Figure 1). The problems at these headworks were similar to those of the river outlet stilling-pool studies. The Madera Canal pool contained two 91- by 78-inch needle valves, each discharging 750 second-feet under a head of 132 feet onto a parabolic apron and into the pool. The original design of the stilling-pool was shortened 50 feet and raised 15 inches, its performance being excellent for all operating conditions. The model for this study was built to a scale of 1:28.4375 as shown on Figure 10.

FIGURE 9



A. SPILLWAY AND RIVER OUTLETS



B. HYDRAULIC JUMP

MODEL OF FINAL DESIGN

FIGURE 8, RIVER OUTLET STILLING POOL

FIGURE 10, MADERA CANAL HEADWORKS

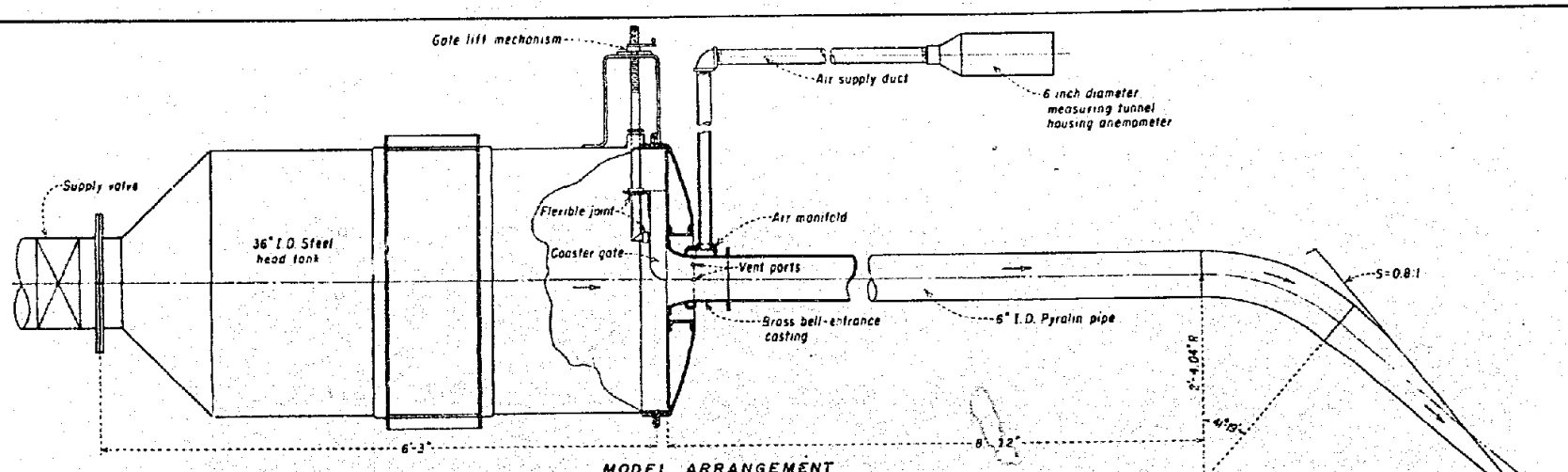
The Friant-Kern Canal headworks consists of two 110- by 102-inch tube valves and two 110- by 96-inch needle valves which toe-in a few degrees. This was necessary because the width of structure at the valves was greater than the canal width farther downstream. The total capacity of the valves will be 3,500 second-feet under a maximum head of 114 feet. A parabolic apron was also used at the stilling-pool and the arrangement was found to be satisfactory. The pool floor was not raised, but a sill 3.5 feet high was added 52.75 feet downstream from the end of the parabolic apron. Figure 11 shows the 1:28.4375 scale model of the final design.

FIGURE 11, FRIANT-KERN CANAL HEADWORKS

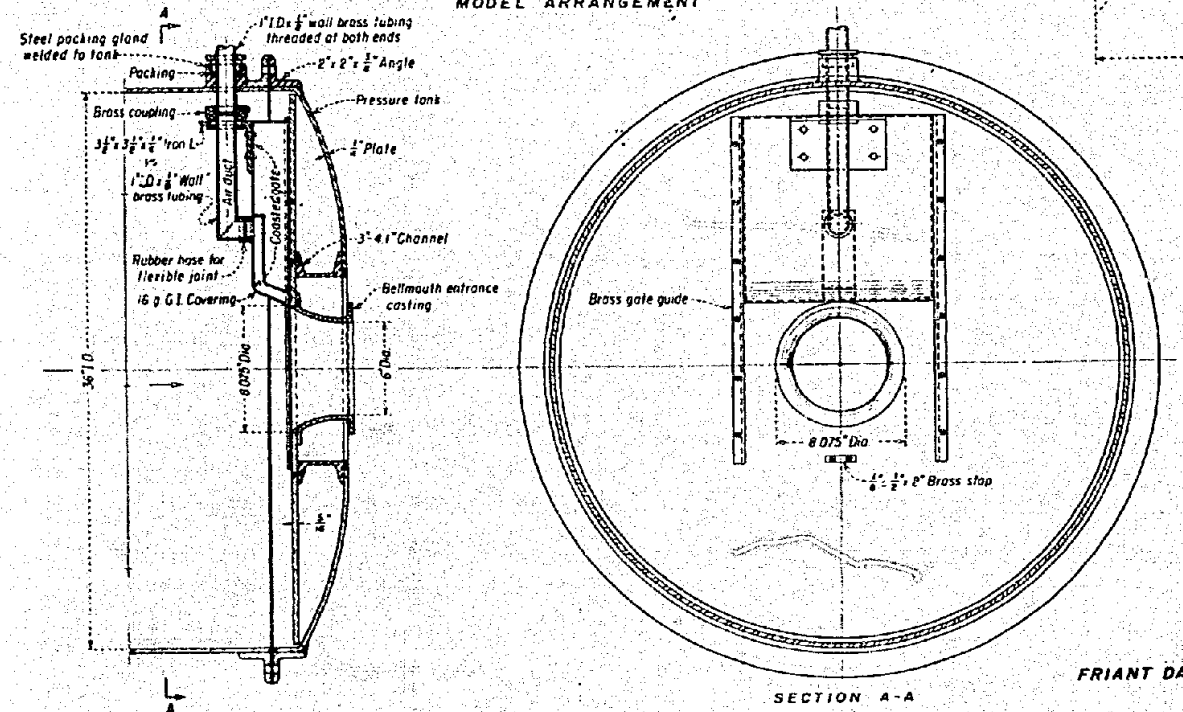
41. Aeration of coaster gates, 1:17-scale model. The tests previously described have, with the exception of the river outlet transition, been concerned mainly with open channel flows. It is

frequently necessary to make studies also of the more mechanical appurtenant works, such as coaster gates and needle valves.

In the design of outlets passing through a dam or otherwise, it is customary to include some means of emergency closure in the event a regulating gate or valve in the outlet becomes immovable when either open or closed. When gates are used, the emergency closure may be obtained by using two gates, the downstream one being used as a service gate, the upstream one as the emergency gate. For this type of design the service gate usually is either fully open or completely closed; it is not used for regulating flows. If a valve is used, such as a tube, butterfly, or needle valve, either within the conduit or at the downstream end, then emergency closure is usually obtained by lowering a coaster gate down the upstream face of the dam to close off the outlet entrance. This procedure is also useful to unwater an outlet furnished with both an emergency and service gate. In the Friant Dam, the design proposed but later abandoned, as explained in Section 38, called for a tube valve a few feet downstream from the entrance of the outlets to regulate the flow in the conduit. No emergency gate was provided within the outlet, so a large bulkhead or coaster gate was designed to be lowered down the back of the dam to close off the entire conduit. It was assumed that the most unfavorable condition would occur for emergency closure when the tube valve was fully open. Under this condition, a severe drop in pressure or drawdown would be expected to occur as the gate passed over the bellmouth entrance of the outlet. Since the closure could be made in a short time, the effects of pressure reduction on the bottom of the coaster gate were not of primary importance, but it would be necessary to investigate the possibility of vibration of the gate because of differential pressures, and if vibrations did occur, to determine how they should be eliminated. The idea of aeration seemed to be paramount and was tried and found successful.



MODEL ARRANGEMENT



SECTION A-A

DETAILS OF COASTER GATE

MODEL SCALE RATIO 1:17

FRIANT DAM - HYDRAULIC MODEL STUDIES

FIGURE 12

HYD 110

A 1:17-scale model was built to test the coaster gate. The model consisted of a steel head tank connected directly to a 12-inch pump, a bellmouth entrance of machined brass attached to the end of the tank, and sections of 6-inch pyralin pipe shaped to the outlet profile (Figure 12). The coaster gate, made of galvanized sheet metal, operated in slide grooves attached to the end of the pressure tank. Threaded brass tubing fastened loosely to the gate and connected to a duct through the gate provided a passage for admitting air to the bell entrance and a means for raising and lowering the gate. Flexible connections between the stem and gate allowed free movement in case the gate vibrated. Location of the vent system in the stem permitted initial studies to be made without altering the bell entrance, while the air-supply-duct size was varied by placing telescopic brass tubing in the gate stem. A pressure gage attached to the head tank was used to indicate the head on the outlet, and piezometers in the outlet conduit were used for measuring pressure along its inner surface. The outlet regulating valve farther downstream was omitted because of an uncertainty of its design; the reduced loss because of its omission was considered a factor of safety. Because of a lack of similitude between the Reynolds number of the model and prototype, the model was tested for heads in excess of those representing the prototype.

Initial tests revealed that the coaster gate of the model vibrated for all gate openings (or for various closures), due apparently to the pressure differential on the gate, which was caused by hydrostatic pressure on its upstream face and negative pressures in the conduit just downstream from the gate. To remedy this condition, air was admitted to the bell entrance. This reduced not only the vibration and the pressure differential, but at the same time made it easier to operate the gate by reducing the forces creating sliding friction. To ascertain the best place to aerate the gate, pressures were recorded in the conduit. By establishing the longitudinal limit of the negative pressure region within the conduit, it was possible to locate the vent

positions. Studies over a wide range of gate openings and heads showed that an average position should be one-half conduit diameter from the upstream end of the conduit; thus the vents would enter the bellmouth section of the outlet.

Upon completion of these initial tests, the model was changed to provide a bellmouth section which would facilitate testing. Accordingly, one was designed with a variable supply duct, a manifold, and variable vent openings. An anemometer placed in a measuring tunnel attached to the supply duct recorded the air flow through the model system. To obtain relative vibration measurements, a metal reed and a DeForest strain gage were fastened to the pyralin pipe outlet section immediately downstream from the bellmouth. Piezometers were again installed in the bellmouth walls for determining the degree of aeration.

From tests on the revised model, it was discovered that the airduct size affected not only the pressures within the bell entrance, but also the vibration characteristics. It was found in the first case that pressures increased rapidly (negative pressures became lower) as the duct size increased, but beyond a certain size duct, the change in pressure was less pronounced. In this manner, a model duct size of 1-3/4-inch pipe was established, and being independent of the head, estimates were made for prototype ducts for various size conditions. A critical duct size was also established in the model considering only vibration of the model, it being found that a maximum dampening effect was obtained for a duct of 1/2-inch in diameter, independent of the head. Again a curve was derived for various prototype duct sizes for various outlet sizes. Completing the airduct tests, it was found necessary to admit air to the bell entrance from more than one point, in fact, six were selected, the combined openings of the holes being slightly greater than the area of the supply duct to offset individual entrances losses. The lowest vents were placed 30 degrees off the invert of the outlet.

42. Needle valves, 1:18.33-scale model. The final model test to be discussed for the Friant Dam is one which reveals some difficulties frequently encountered with the performance of needle valves.

The concern felt by hydraulic engineers on the effects of cavitation in flowing water was mentioned in connection with the cavitation tests made on turbines and pumps. The main concern there was to prevent as much as possible, the pitting of the runner blades or impellers and to prevent cavitation from affecting the efficiency of the machines. In needle valves, similar pitting has also been a major problem, since incorrectly shaped needle valves have been eroded within the body of the valve and on the shoulder of the needle after only a short time of operation, the cavitation occurring generally for valve openings of from 5 to 30 percent. For a description of needle valves and regulating gates, see Dams and Control Works, Second Edition, United States Department of the Interior, Bureau of Reclamation, Washington, D. C., February 1938, pp 182-198.

Inspection of some needle valves recently put in operation by the Bureau of Reclamation discloses pitting to develop on the needle immediately downstream from its point of seat with the nozzle, and inside the body of the nozzle a short distance upstream from the downstream end of the nozzle. Older valves have had to be repaired by welding metal to the pitted areas. Because of the pitting, especially at the newer installations, tests were made on a model in an attempt to make the needle valves being designed for the Friant Dam outlets entirely free from pitting, and thereby develop a design suitable for other installations.

The Friant Dam needle valves will be 110- by 96-inch, that is, they will have an inlet diameter of 110 inches and an outlet diameter of the nozzle of 96 inches. Each will discharge 4,500 second-feet under a maximum head of 220 feet. A 1:18.33-scale model was made of one of the valves from bronze castings carefully machined to the true dimensions, as shown on Figure 13. Piezometers were installed on the body of the valve from the inlet to the outlet, and on the needle. The approach pipe to the

valve was connected to a pressure tank which, in turn, was connected

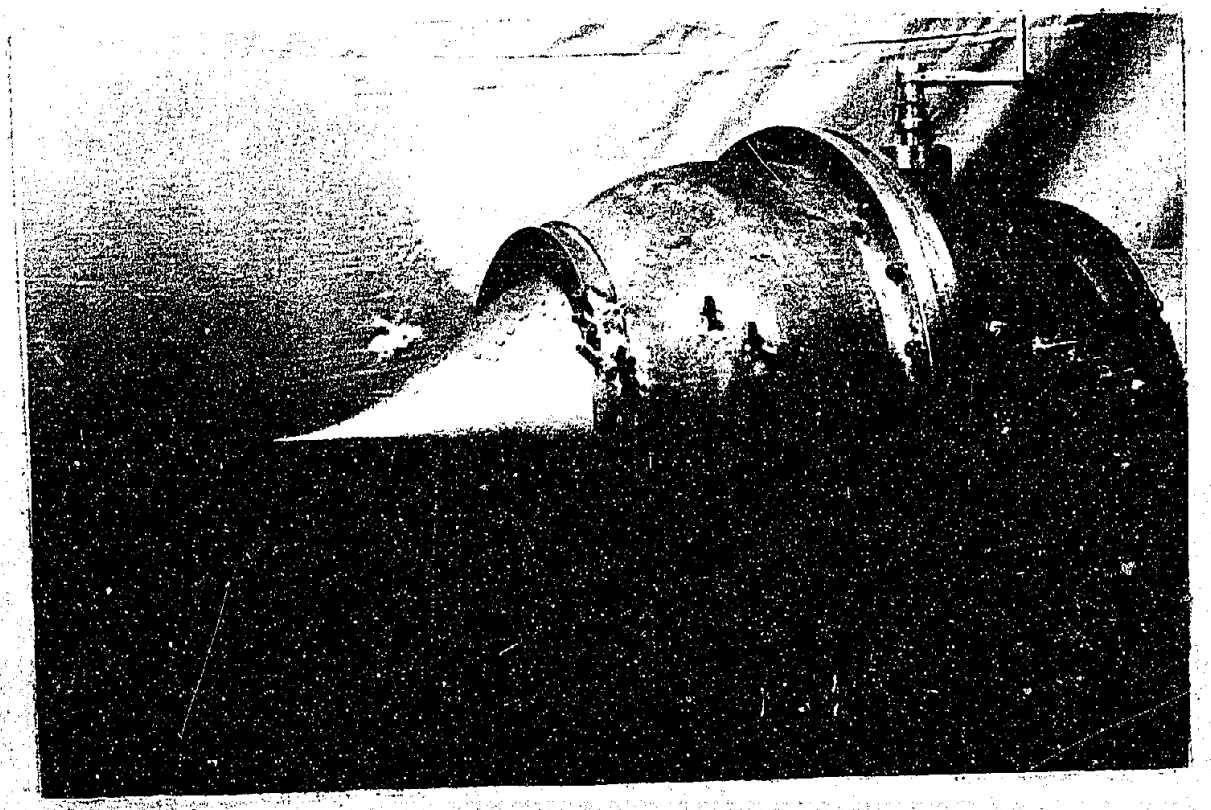


FIGURE 13, NEEDLE VALVE MODEL

directly to a 12-inch centrifugal pump. The head on the valve was measured one diameter upstream from the inlet flanges, and the discharge was measured over a calibrated weir in the return system of the laboratory water supply. Pressures were measured by manometers connected to the piezometers on the valve by rubber tubing. The valve was opened or closed by a crank mechanism which controlled the movement of the needle.

The first tests were concerned with the pressure gradient through the valve for various heads and valve openings. It was found immediately that the low-pressure area found on the model agreed almost exactly with the position of the pitted area observed on several prototype structures. These, as mentioned, occurred on the needle

just downstream from its shoulder and on the curved portion of the nozzle near its downstream end. Although the pressure conditions on the surface of the valve were unfavorable, the valve had a high coefficient of discharge. To improve the pressure gradient, the model was so revised that the orifice of the nozzle was changed from a round-edged to a sharp-edged orifice, and the divergence between the needle and the body of the valve at the nozzle measured at the open position, was changed to a convergence of 3 degrees. This change produced a positive pressure gradient throughout the valve, but the coefficient of discharge dropped 15 percent. From these results, the limitations were evident: The pressure gradient must be positive, but the discharge coefficient must remain at a high value.

The tests that followed were concerned, therefore, with the pressure gradients and the discharge coefficients over the operating range, as affected by changes in the length of needle travel in opening and closing, in the outlet (nozzle) diameter, and changes in the angle of the outlet section (divergence or convergence of the annular water passage between the nozzle and the needle). The procedure for each test after the valve was changed as desired, consisted of measuring the valve opening (this varied from 5 to 100 percent), establishing the discharge for the head desired at any opening selected (heads varied from 2 to 22 feet), and recording the discharge and pressures on the needle and nozzle. For full-valve opening, the profile of the jet was measured.

From these tests, it was discovered that by changing the orifice of the nozzle from a round-edged to a sharp-edged orifice, the control was maintained at the orifice resulting in a positive-pressure gradient, instead of a negative gradient with the control upstream from the orifice. It was determined, furthermore, that variation in the outlet diameter and the length of travel of the needle had no effect upon the pressure gradients, but that the control point at the orifice was the governing factor, and that a converging outlet section will place the control at the orifice for all valve openings. Further testing allowed conclusions

to be drawn regarding positive pressure gradients; that a small divergence of about one degree, parallel outlet section, or small convergence would be permissible. Relative to discharge coefficients, it was possible to state that the length of travel of the needle and size of outlet diameter affected the coefficient of discharge. For a valve with a given outlet diameter and needle travel, the discharge coefficient would depend upon the shape of the outlet section, which is governed by the shape of the needle and nozzle body.

CHECK DROP 4

43. The prototype. This model study is selected to illustrate a type of problem found on irrigation projects, and it represents a study made of an existing structure.

On the Yakima Irrigation Project, Washington, the Sunnyside Main Canal supplies water to 90,000 acres out of a total of 106,000 acres on the project. The capacity of this canal has been steadily increased from 650 second-feet in 1912 to its present capacity of 1,300 second-feet. Because of this increase in capacity, it was necessary to place in the canals at intervals of about two miles 23 check drops to maintain normal depths of flow and velocities. These structures were added between 1907 and 1916. As the capacity was increased, and after the drop structures had been in operation only a few seasons, severe scour developed below each drop. This scour widened and deepened the canal to such an extent that it began to encroach on valuable farm lands adjacent to the canal at the drop structures. With all efforts failing in the field to prevent this excessive erosion, the problem was finally submitted to the Hydraulic Laboratory of the Bureau of Reclamation in Denver.

These drop structures, all nearly similar, consist of a check basin placed between two vertical walls. In the check basin are concrete piers surmounted by steel brackets which permit the placing

of flashboards across the canal to regulate the depth of water between successive drops. Riprap is placed in the bottom of the canal, both upstream and downstream from the check basin to reduce scouring. The total drop in water surface at Check Drop 4, the one selected for study, is only 15 inches. The maximum drop in the series of 23 structures is 27 inches.

From field observations and from an analytical analysis made previous to testing the model, it was found that because of the small drop at each structure, standing waves formed instead of the hydraulic jump. Accordingly, only a small amount of energy dissipation occurred enabling relatively high velocity flow to proceed downstream, particularly along the water surface. This produced large eddies on each side of the canal and eroded the canal banks. As erosion progressed and widened the canal, the eddies also increased and continued cutting the banks. Velocities were of sufficient magnitude along the bottom to erode the canal bottom to a depth of 15 feet below normal.

44. The model. A 1:15-scale model of Check Drop 4 was built as shown on Figure 14. The drop structure was made of redwood, and the steel brackets of sheet metal. It was then installed in a large, metal-lined box which provided sufficient length of approach upstream and sufficient length and width downstream so that the scouring in the model would not be restrained. Gages were installed to measure the depth of flow, which was controlled upstream by flashboards at the drop structure and downstream by a tailgate. Because the model velocities were considerably less than the prototype velocities, it was necessary to use in the model the finest sand available to reproduce the prototype scour, which occurs in volcanic tuff.

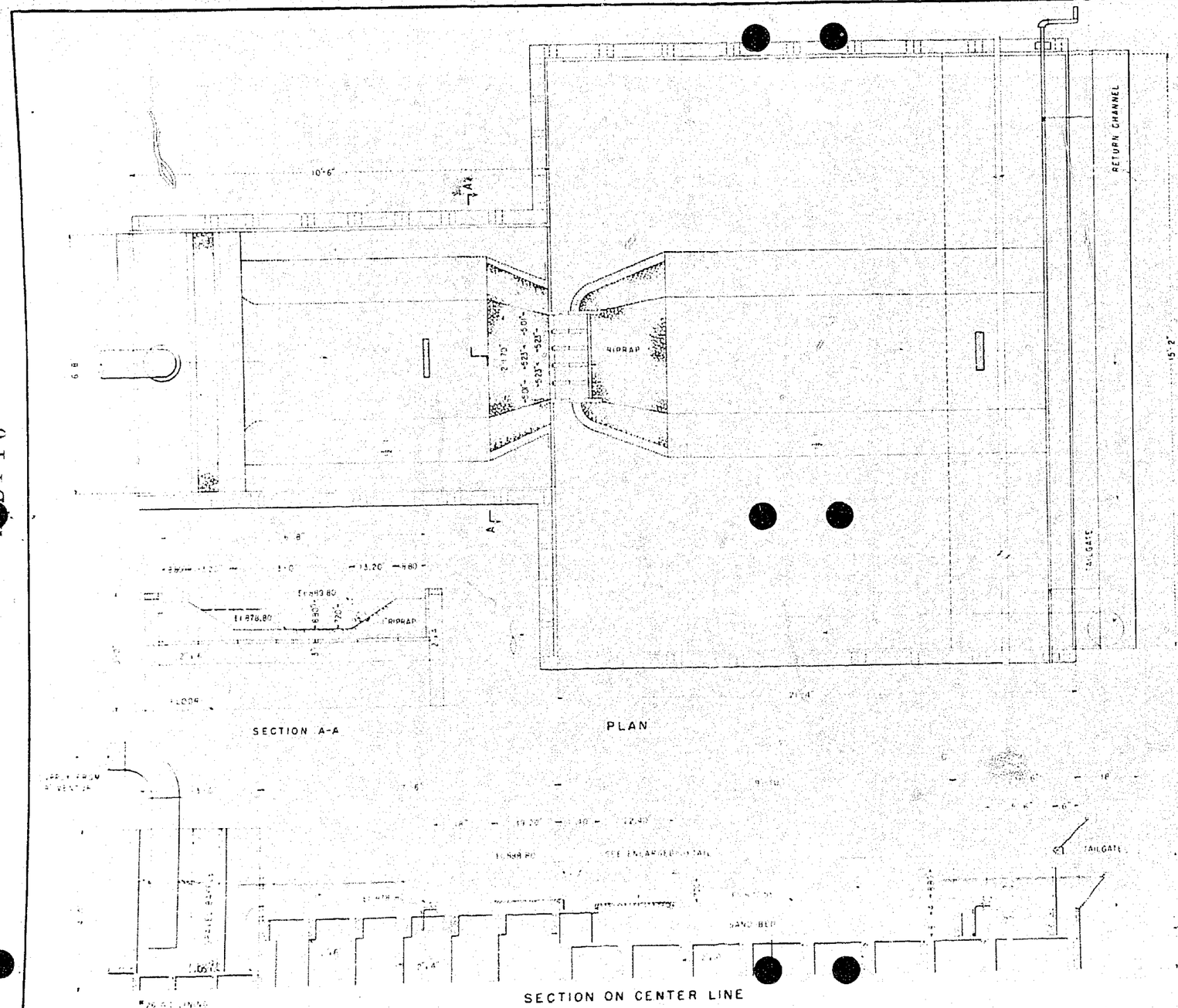
45. Tests on model. Tests were made, at first, to check or verify the model to see if it would reproduce the flow conditions and scour noted in the prototype. To do this, the model discharge was run for 26 hours, during which time the discharge, tailwater,

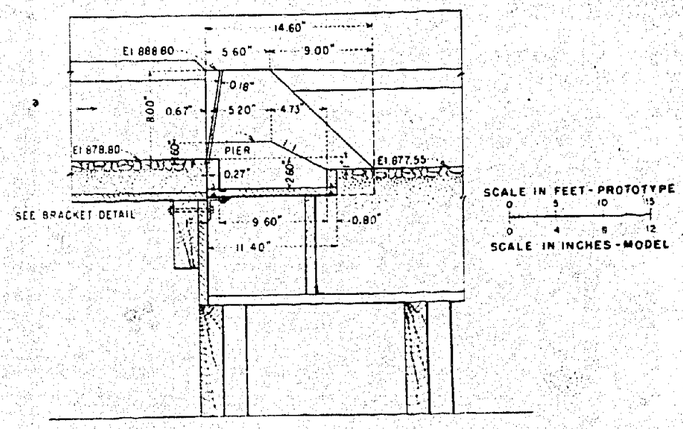
and number of flashboards were varied, duplicating an operation continuous during an irrigation season. The results of this test were most gratifying (Figure 15A); the standing wave flow and excessive scour were faithfully reproduced. An interesting comparison with the prototype may be seen by comparing Figures 15A and B. It was apparent that the following conditions existing at the prototype had to be eliminated: (1) Unsymmetrical flow distribution through the drop structure; (2) standing wave and excessive velocities below the drop; (3) eddies along the side of the canal immediately below the drop; and (4) scour to the canal.

46. The recommended design. To improve the existing structure was a difficult and trying task. Many designs were tested to increase the energy dissipation of the flow and thereby reduce the scouring action in the canal. The first revision extended the vertical side walls downstream to form a stilling-pool, and added curved training-walls at the entrance to the check basin. Sloping floors were then placed between the piers of the check basin, followed by a horizontal floor extending to the end of the vertical side walls. This and similar designs were unsuccessful. Finally, a deflector was placed across the stilling-pool immediately downstream from the brackets. This caused the entire flow to plunge under the tailwater and into the pool. By placing baffle piers on the pool floor and by setting the floor at a proper elevation, the desired results were accomplished. The high-velocity flow no longer escaped along the water surface, no eddies developed, and a 26-hour scour test revealed practically no scour at all to the model canal, as shown by Figure 16.

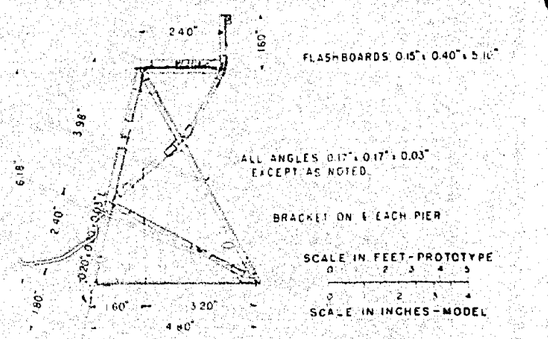
47. Inspection of structure in field. It is indeed fortunate when one is able to test a model and then to be able to observe the prototype in operation at nearly the maximum discharge. The author was fortunate, therefore, to be able to study Check Drop 4 while inspecting the Yakima Project in Washington. It can be said that the prototype is performing as predicted by the model tests.

FIGURE 14





ENLARGED DETAIL OF DROP



BRACKET DETAIL

FOR DETAILS OF EXISTING STRUCTURE SEE FIGURE 2

SCALE IN FEET - PROTOTYPE

SCALE IN INCHES - MODEL

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT - WASHINGTON

REDESIGN OF CHECK DROPS - SUNNYSIDE MAIN CANAL

HYDRAULIC MODEL STUDIES - 1:15 SCALE

MODEL OF EXISTING STRUCTURE AT CHECK DROP 4

DRAWN	H.E.D.J.R.	SUBMITTED	
TRACED	F.A.J. F.B.	RECOMMENDED	
CHECKED	H.J.D.J.R.	APPROVED	
		DENVER, COLORADO - DEC 17, 1958	33-D-1774

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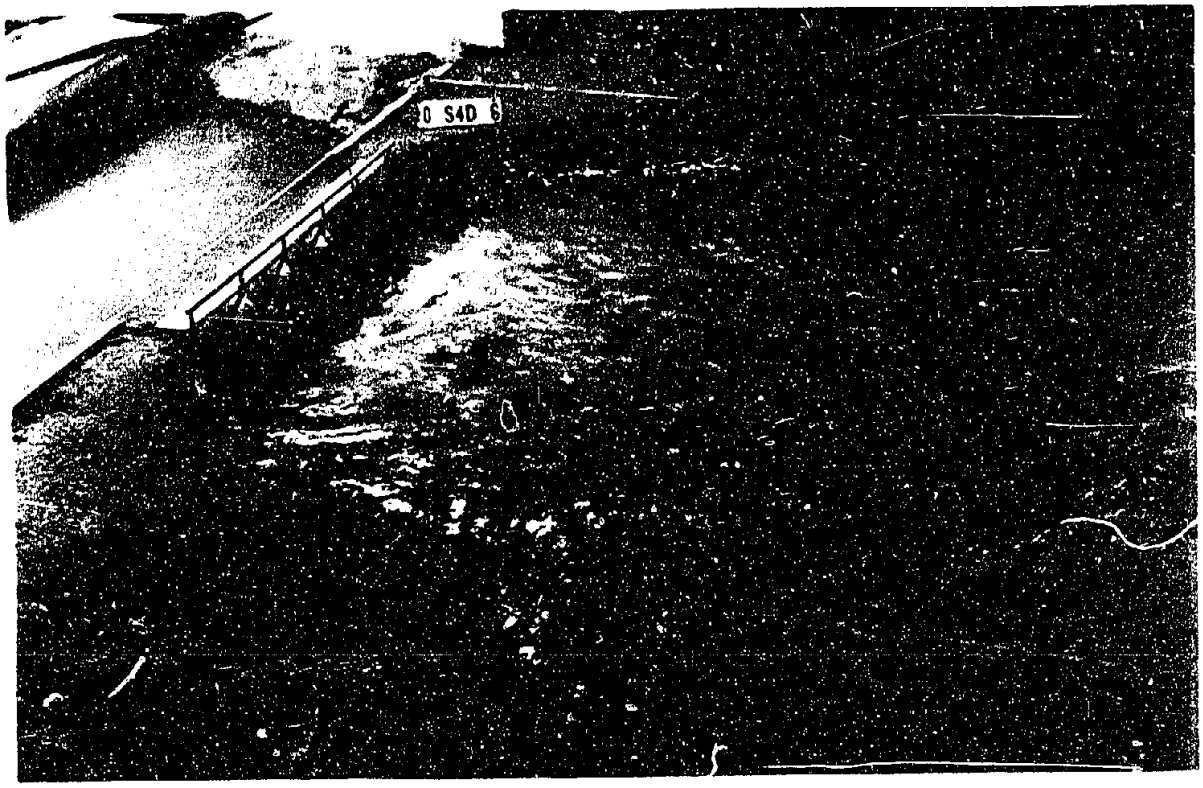
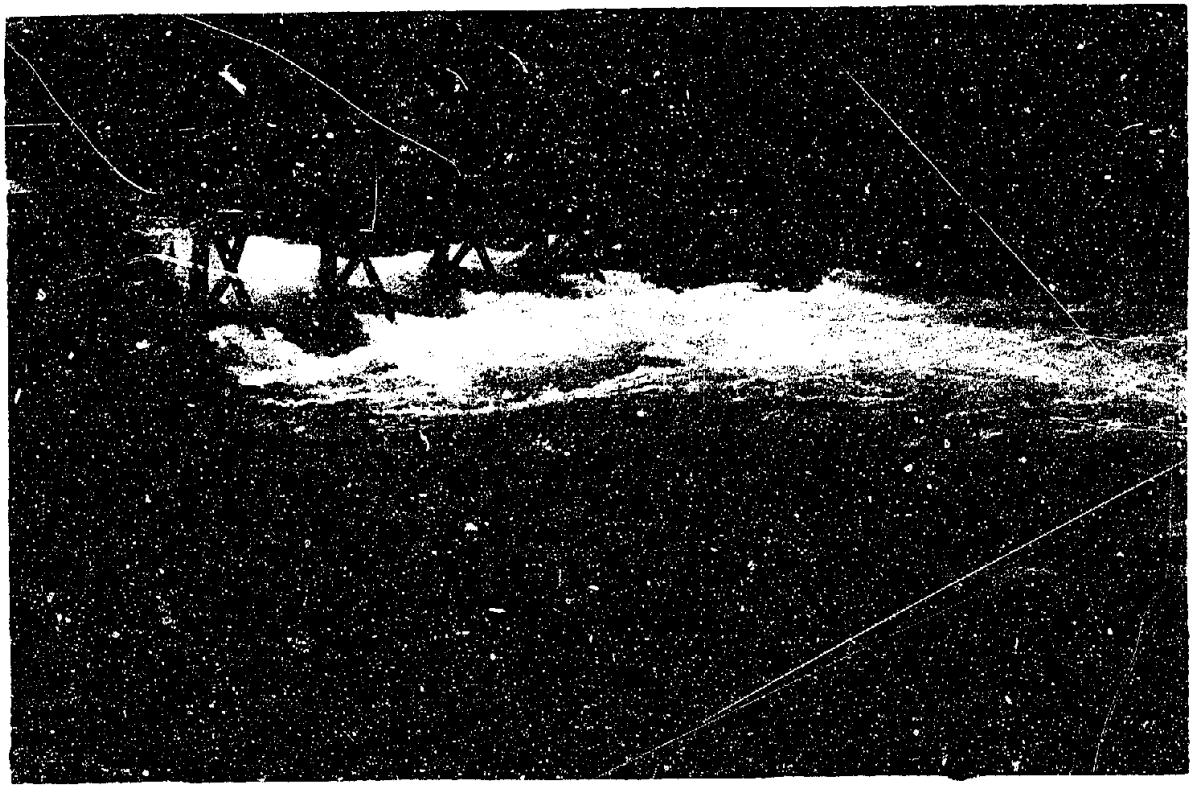


A. MODEL - DISCHARGE 1,300 SECOND-FEET



B. PROTOTYPE - DISCHARGE 1,100 SECOND-FEET

Figure 17 shows the revised structure in the model and in the field. The collection of trash at the prototype structure is evidence of roller action in the stilling-pool. This is indicative of energy dissipation throughout the entire depth, instead of only on the surface

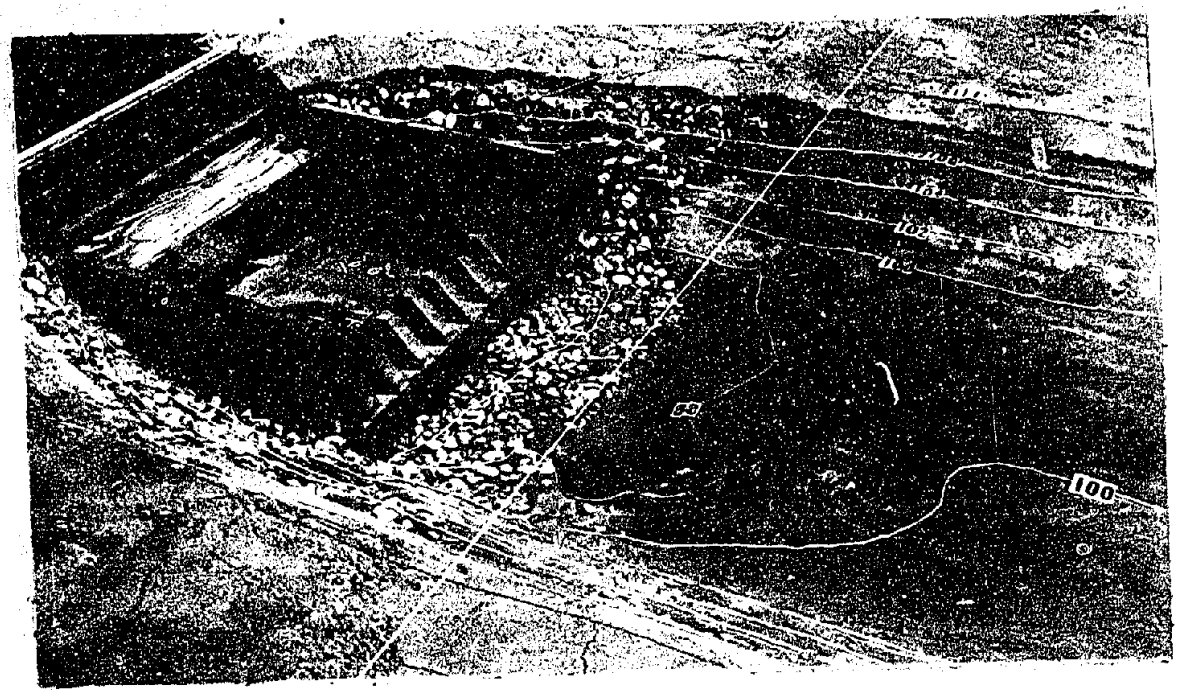


FIGURE 16, MODEL OF RECOMMENDED DESIGN

as noticed on Figure 15. A similar collection of debris was noticed during the laboratory experiments, but it was given little consideration since it interfered with photographing the model. In the field, however, it has been of considerable value in clearing the canal of large amounts of trash, conveniently removed at the structure. Check Drop 6, farther downstream, has also been revised in a similar manner, although the flow conditions are somewhat different than at Drop 4. No model study of Drop 6 was necessary, since the design of Drop 4 was easily modified to meet the conditions at any of the 23 drop structures. For additional discussion of this model-prototype comparison and others, the reader is referred to a symposium of this subject to be published in a 1942 issue of Proceedings, A.S.C.E.

OUTLET ENTRANCES, MADDEN DAM AND GRAND COULEE DAM

48. Problem involved. One of the more important design problems is that of proportioning the shape of entrances to rectangular or circular outlets for high dams. The shape developed is sometimes referred to as a bellmouth entrance. If incorrectly designed, the flow will separate from the boundary surfaces and cavitation will develop, causing pitting of the surfaces, vibration, and noise. One of the best examples of this occurred at the outlets of the Madden Dam. In their model test at Carnegie Institute of Technology to study the outlet entrances of the Madden Dam, Thomas and Schuleen reveal the damage done at the prototype by cavitation.[2] The entrances to the six rectangular conduits, 5 feet 8 inches wide by 10 feet high, were formed with a 4-foot radius at the crown, invert and sides of the inlet end, which which was not lined with steel. After operating at almost maximum head for a brief time, it was clear that because of severe crackling and popping noises and leakage into a gallery from a porous tile drain in a partition wall between conduits 1 and 2, that cavitation and its pitting effects were tremendously large. By lowering slide gates to about 90 percent open, the cavitation was minimized. After emptying the reservoir, an inspection was made of the inlets to the conduits. In the outlets damaged the most, the cavitation in the flow had heavily pitted the side walls, invert and crown to depths of 2 feet in many places, and had exposed reinforcing bars that had been placed 10 inches below the surface of the concrete.

49. Test methods. To correct these unfavorable conditions, the models at Carnegie Institute of Technology were tested in a cavitation apparatus which permitted the absolute pressure on the models to be reduced to the vapor pressure of the water in almost direct proportion to the model scale ratio. By this method cavitation would actually

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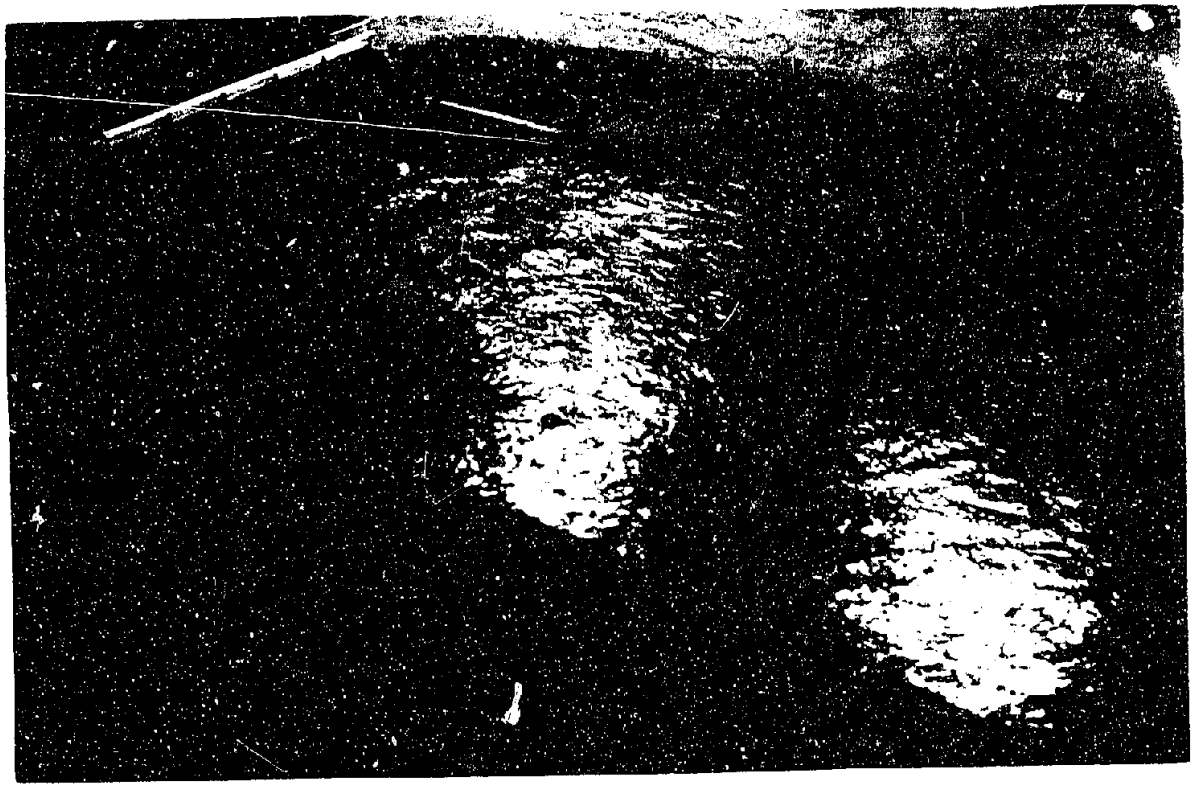
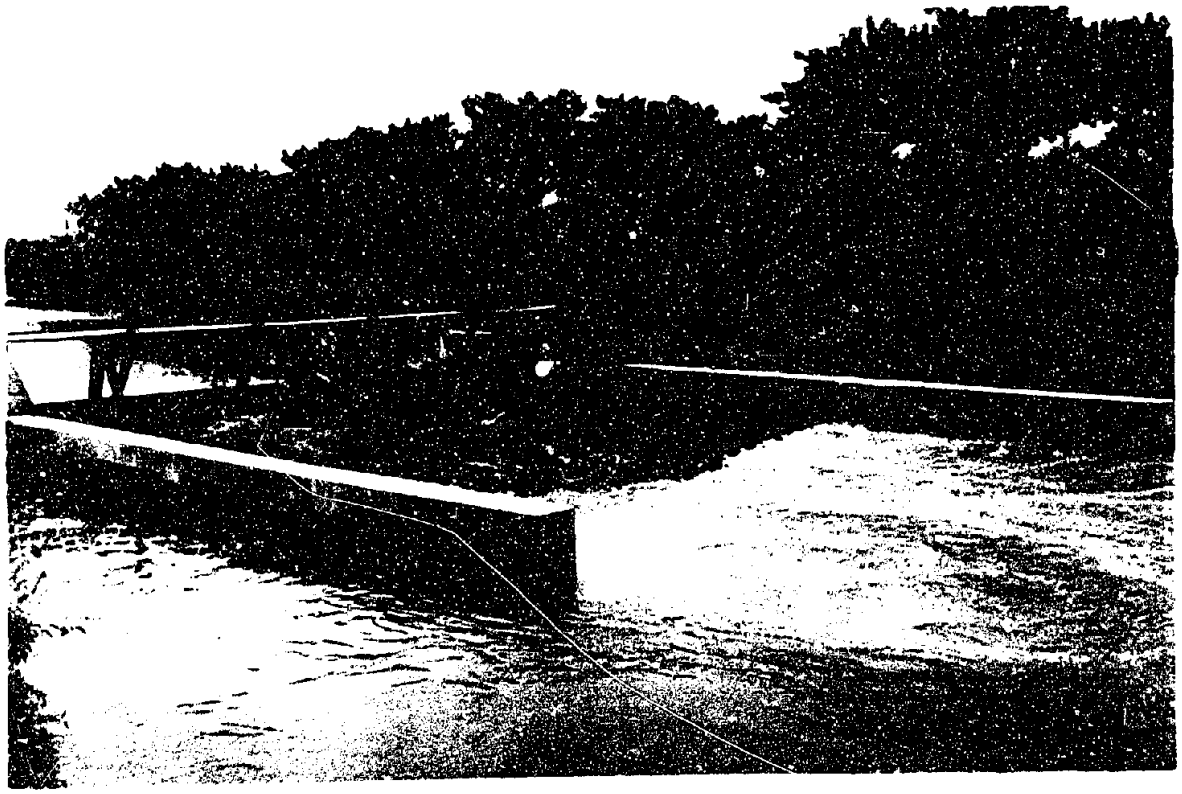
[2]Thomas, H. A. and Schuleen, Emil P., "Cavitation in Outlet Conduits of High Dams," Proc. A.S.C.E., November 1940.



A. MODEL - DISCHARGE 1,300 SECOND-FEET



B. PROTOTYPE - DISCHARGE 1,100 SECOND-FEET

develop in the model, and by recording pressures and changing shapes it was possible to redesign the inlet shape.

This example illustrates the seriousness of inadequate entrance design of outlets through dams. The studies made in the Denver Hydraulic Laboratory of the Bureau of Reclamation, on the entrances for the Grand Coulee Dam outlets, were in progress when the Madden Dam outlets were reported to be damaged from cavitation. Accordingly, tests were made to provide an adequate design for the bellmouth entrances of the outlets in the Grand Coulee Dam.[3] These outlets, 102 inches in diameter, occur in three tiers separated vertically by approximately 100 feet, 20 outlets in pairs in each tier. Steel lining is placed in the upper two tiers throughout the entire length of conduit, but only at the gate section in the lower tier.

Instead of testing a model outlet by the vacuum method used by Thomas and Schuleen, the model was tested at atmospheric pressure, so cavitation, if it were to occur in the prototype, could not actually be produced in the model, but piezometers were installed on the boundary surfaces to record pressures. It is possible to predict by this method that if, for any operating conditions, slight positive pressures (above atmospheric) existed, then only positive pressures would occur in the prototype; and if negative pressures (subatmospheric) occurred, then they would also occur in the prototype, and if these negative pressures, when expressed in terms of the prototype, approach the vapor pressure, then cavitation would probably occur at the prototype. This method is more convenient and less expensive and is generally used more than the vacuum method. Nevertheless, recognition is being given to the vacuum method and a comparison of the results of tests in a problem studied by both methods is awaited with interest.

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[3] "Hydraulic Model Studies for the Design of Sluice Entrances for Grand Coulee Dam," Report No. HM-1, Bureau of Reclamation, Denver, Colorado, September 15, 1939.

50. Bellmouth design. The objective set for the pressures on the surface of the inlets to the Grand Coulee sluices was that they should not fall below atmospheric. To accomplish this, it was decided, since the outlets were circular, to establish the exact shape of a jet issuing from a circular sharp-edged orifice. A 12-inch pump giving the desired range of heads and discharges was connected to a pressure tank 3 feet in diameter and 5 feet long. The water entered the tank through a series of concentric distributing cones, passed through a specially constructed checkered rack, designed to produce an even flow distribution, and thence out through the circular orifice. The orifice was made 3 inches in diameter in a brass plate and set in a floating steel plate which was designed to remain in a plane regardless of shape changes in the pressure tank. A specially designed instrument was attached to the downstream flange of the pressure tank to measure the profile of the issuing jet. The test apparatus is shown on Figure 18.

The first tests measured the jet from the 3-inch orifice for heads varying from 10 to 95 feet. A profile of the jet was taken in longitudinal planes inclined 0, 30, and 60 degrees from the vertical. For the heads tested the profile of the jet was quite uniform, so a curve was drawn through all points. The coefficient of contraction, the ratio of the area of the minimum section to the area of the orifice, was found to be 0.598. This figure agreed closely with the value of 0.611 used in orifice studies by Professor Charles Harris at the University of Washington. From the shape of the jet issuing from the circular orifice and by applying a factor of safety, that is, using a coefficient of contraction of 0.590 instead of 0.598, an equation in terms of the outlet diameter was found that would give a satisfactory bellmouth. This equation is $\frac{x^2}{(0.50D)^2} + \frac{y^2}{(0.15D)^2} = 1$, an ellipse, which, when rotated about the axis of the outlet, forms the desired bellmouth. From this equation, it may be seen that the length of the bellmouth is 0.50D and the contraction is 0.30D. By neglecting the back pressure in the model, which would be present in the prototype due to frictional loss downstream from the entrance, it was believed best to converge the conduit slightly

just downstream from the entrance. This was provided for in an elbow section which was necessary because the axis of the bellmouth entrance is tilted to be normal with the 0.15 slope of the upstream face of the dam, while the conduit axis is horizontal (Figure 18). The convergence in the elbow is not necessary at all outlet designs, but was an added factor of safety and readily applied because of the elbow required in the conduit. For horizontal outlets, the above equation is entirely adequate as far as pressures are concerned in the entrance. Since these model tests were made for a single outlet and with no trashrack, a check test was made to determine the effect of a trashrack and the operation of adjacent outlets. It was found that these factors had a negligible effect on the previously recorded positive pressure conditions.

In 1940 and 1941 prototype tests were made of the outlets at Grand Coulee Dam and it was found that the prototype pressures were in good agreement with the model results. Reference is again made to the model-prototype symposium to be published in a 1942 Proceedings of the Society.

JOHN MARTIN DAM

51. The prototype. The John Martin Dam (formerly Caddoa Dam) is located on the Arkansas River about two miles from Caddoa, Colorado.[4] It will be a concrete and earthfill structure 4,000 feet long, with a spillway 1,174 feet long, designed to pass 630,000 second-feet in the concrete overfall section (Figure 19). The height of the dam will be 105 feet from foundation to the crest, and the reservoir will have a maximum capacity of 655,000 acre-feet. The flow from the reservoir will be controlled by four 6- by 7.5-foot rectangular conduits and two 4- by 4-foot square conduits for low-water flow, operating under a

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[4]Model Study of the Spillway and Stilling Basin for the John Martin Dam, Arkansas River. Technical Memorandum No. 166-1, U. S. Waterways Experiment Station, Vicksburg, Mississippi, December 15, 1940.

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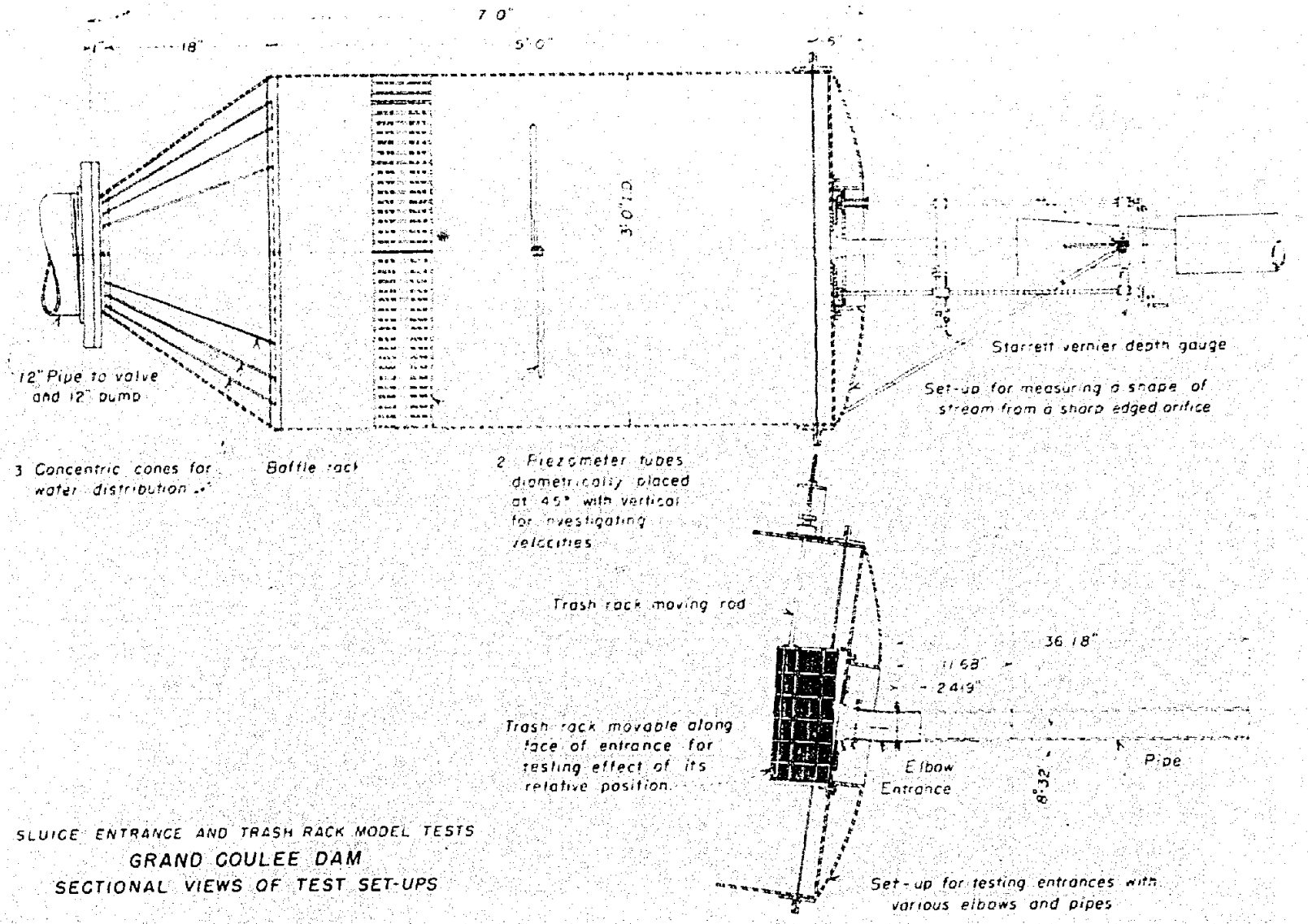


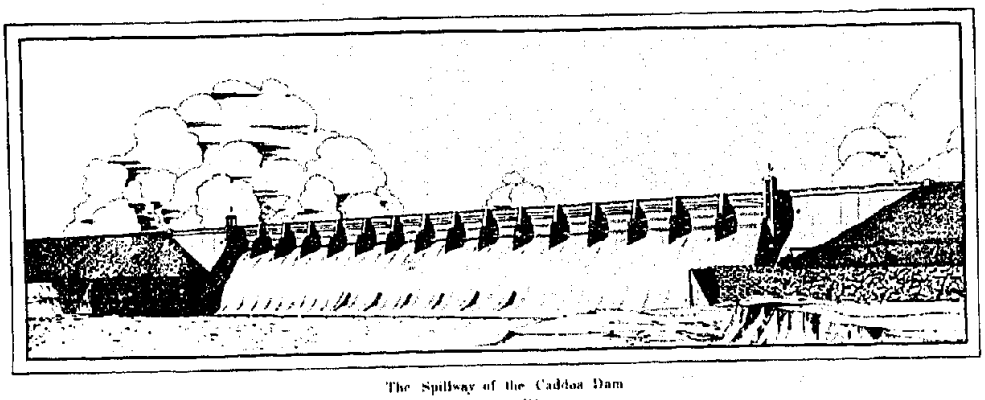
SLUICE ENTRANCE AND TRASH RACK MODEL TESTS

GRAND COULEE DAM

SECTIONAL VIEWS OF TEST SET-UPS

E.S.G. 1-26-38

maximum head of 102 feet above their entrance and discharging a combined flow of 16,700 second-feet. Each conduit will be controlled by a slide



The Spillway of the Caddoa Dam
Arkansas River

FIGURE 19, SPILLWAY OF JOHN MARTIN DAM

gate, and air vents will be located immediately downstream from the gate to relieve negative pressures in the conduit during partial or even full-gate openings. Maximum flood flows will be controlled at the crest by 16 radial gates, 64 feet long by 30 feet high. The purpose of the dam is to control floods and to provide water for irrigation.

52. Purpose of model study. To determine the most economical design and the safest design for the spillway, outlets, and stilling-basin, a model was made to study the capacities of the spillway and contents and the pressure on their surfaces; the performance of the spillway and stilling pool; and to make the necessary revisions to the original design submitted for testing.

53. The model. The model of the spillway and stilling-pool was built to a scale ratio of 1:36. The model reproduced 500 feet of approach channel; eight bays of the spillway section, together with six conduits; the stilling-basin; and 720 feet of exit channel as shown by Figure 20.

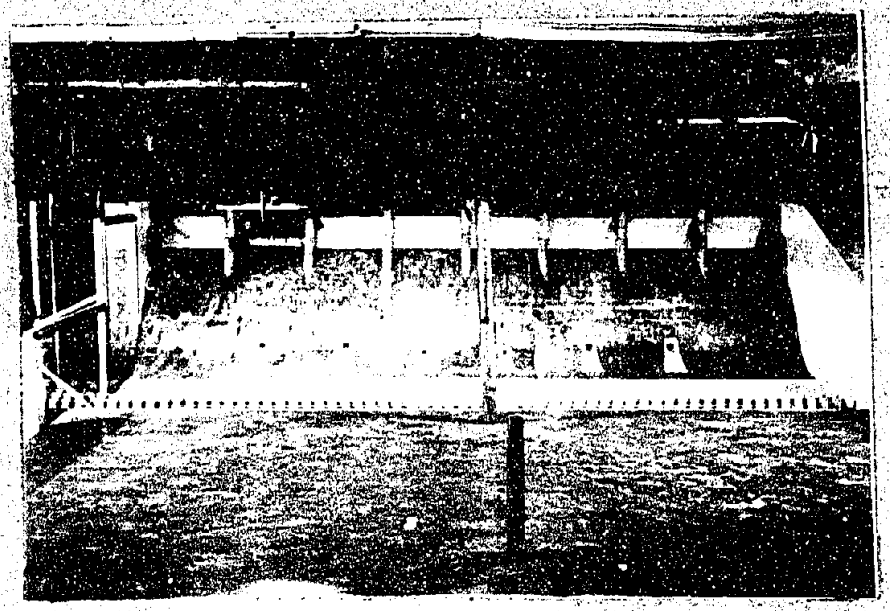


FIGURE 20, MODEL OF SPILLWAY

The reservoir was represented by a brick headbay large enough to permit quiet approach conditions. The spillway structure was made with a structural steel base and with sheet metal templets to which a cement mortar was molded, smooth surfaces being maintained. The piers were made of wood and waterproofed, then fastened to the base supporting the spillway. The radial gates at the crest of the dam were fabricated from sheet metal and brass, while each of the six conduits were formed by molding pyralin. A tunnel under the spillway section permitted observation of the flow in the conduits. The stilling-pool, consisting of the apron, baffle piers, end sill, and training walls were reproduced in wood, the baffle piers being attached to sheet metal strips to facilitate their moving to various positions. A glass panel was placed on the side of the pool to afford observation of the flow phenomenon on the apron, but the exit channel was contained in a watertight brick-and-concrete flume. This held the sand which was molded to the configuration of the exit channel by male templets supported on steel beams.

The water used for the tests was supplied by a battery of three pumps connected in parallel, the quantity being measured by venturi meters, then dumped into the forebay where it was stilled by baffles.

After passing through the model, the water returned to the pump sump by a return pipe.

54. Model measurements. A run or test consisted of establishing a desired discharge, head, gate opening, and tailwater for the various designs being examined. Standard types of hook and point gages were used for measuring water-surface elevations. Current directions were established by tracing confetti on the surface and dye beneath the surface. Pressures were measured with manometers connected by tubes to piezometer openings along the surfaces of the model. Velocities were recorded with a pitot tube, and scour to the exit channel was measured by a special point gage. The amount of scour was obtained by plotting the scour profile and comparing it to the plot of the sand bed in the exit channel before a test was run.

In designing the model, a scale ratio was selected such that the roughness on the model would closely simulate the prototype roughness. Accurate measurements could be made of the various quantities involved and accurate construction of structural elements could be made; flow throughout the model would be turbulent; and the model should include areas and surfaces pertinent to the problem.

55. Test procedure. To determine whether each element studied was adequate and to record the final results obtained, several tests were made. Briefly, these tests were concerned with the following:

a. The relation of the reservoir elevation to the discharge for various combinations of full and partial gate openings to establish rating curves for the Taintor gates on the crest.

b. The flow conditions over and through the spillway structure to indicate the general hydraulic performance relative to eddies, turbulence, and other undesirable conditions detrimental to the safe and efficient operation of the spillway.

c. The magnitude of pressures over and through the spillway structure in order to establish a hydraulic gradient through the outlet conduits and to estimate the forces acting on the spillway and stilling-pool surfaces.

d. The profile of water surfaces over the spillway and through the stilling-basin and exit channel to determine the height of training walls and to check the formation of the hydraulic jump phenomenon.

e. The magnitudes of velocities in the stilling-pool and exit channel to determine the flow distribution, effect of bottom velocities on scour of bed material, and to compare different designs tested in the stilling-pool.

f. The magnitude and extent of scour in the exit channel to compare various apron designs tested. Since this was a qualitative study, no quantitative estimate could be made relative to prototype scour.

56. Summary of results. Considering each feature of the prototype separately, the model studies disclosed the following:

a. Spillway. For all conditions of discharge and gate openings, the flow passed smoothly over the spillway crest. Slight negative pressures were recorded along the ogee section for all gate openings, and slight revisions to the crest had no effect either on the flow or pressures. The model rating curves agreed closely with the computed curves.

b. Spillway piers. All intermediate piers were satisfactory, but the end piers at the training-walls were changed to provide a streamlined pier nose.

c. Crest gates. All gates should be operated uniformly so that no unbalanced flow conditions would develop on the spillway and in the stilling-pool. The maximum flow for one gate operation should be distributed among at least six gates. For larger flows, the gates adjacent to the training walls should be open to eliminate upstream eddies in the stilling pool.

d. Conduits. The flow in the conduits was satisfactory for all operating conditions. Negative pressures were measured in the

downstream portion of the outlets, but were (except for low heads) relieved by lowering the crown at the exit six inches. This throttling reduced the discharge capacity 10 percent.

e. Stilling-pool. The original design submitted for testing was entirely adequate, but to improve structural conditions at the beginning of the pool, a sloping apron was added. The pool still was adequate, but its performance was contingent upon the operation of the spillway gates uniformly.

f. Training-walls and earthfill embankment. Increasing the length of the training wall at the stilling-pool and adding to the end an extension on a 50-foot radius reduced the scour to the river at the corners of the pool, the eddy condition along the earth embankment, the exit velocities attacking the earth embankment, and confined the turbulent flow below the pool so as to provide an easy transition of flow into the exit channel.

MISSISSIPPI RIVER FLOOD-CONTROL MODEL

57. The model. The floods of the Mississippi River are too well-known to be discussed here, but it is not generally known just what steps are being taken to eliminate these floods. The methods employed consist of making cutoffs, providing floodways, and building levees, all of which may or may not be adequate. Cutoffs shorten and straighten the main river channel, while floodways are auxiliary channels or areas provided for handling excess water to decrease the stage or to regulate the stage within certain limits in the main river channel.

Making these changes is one thing, but to predict their efficiency is another. Accordingly, the U. S. Waterways Experiment Station has constructed a fixed-bed type of model of the Lower Mississippi River to include the 600-mile reach from Helena, Arkansas, to Donaldsonville, Louisiana. In addition to the Mississippi River channel, there is also included limited lengths of the principal tributaries, the White, Arkansas, Ouachita, and Red River west of the Mississippi, and the

Yazoo River to the east; and their backwater area together with the entire Atchafalaya River Basin and a portion of the Gulf of Mexico. With scale ratios of 1:2,000 horizontal and 1:100 vertical, this model covers 2½ acres, representing a prototype area of nearly 10,500,000 acres. The maximum length of the model is 1,055 feet, its maximum width 168 feet. Figure 21 shows the lower end of the model.

FIGURE 21, MISSISSIPPI RIVER FLOOD CONTROL MODEL

The water for the model is supplied by pumps from a storage reservoir. After passing through the model, the water drains back into the reservoir. Devices for regulating the inflow and outflow of water are so located that a given flood can be directed through the entire model, or, as is frequently done, particular reaches of the main channel can be operated for study of flows at selected reaches.

58. Similitude. As discussed in Chapter II, distorted models are required in problems of this type because of the large area to

be reproduced; hence, the linear scales must be so selected that the model will be economical and all topographic features can be reproduced.

The discharge scale based on Froude's Law is 1:2,000,000 with a corresponding time ratio of 1:200, but due to the distortion, the theoretical scale ratios must be adjusted. For this flood-control model, a varying discharge scale was tried at first, but proved unsatisfactory due to the varying length of day with the varying discharge scale. Next, a constant discharge scale of 1:2,000,000 was tried, but this also proved inadequate because of the excessive roughness required to make it applicable. By trial and error, using several constant discharge scales, a scale ratio of 1:1,500,000 was finally found to satisfy the hydraulic and geometric conditions prevailing.

The roughness required on the model was varied from reach to reach by using wire screen and stucco to reproduce forests, underbrush, and swamps. This change of roughness was required because of the changes in hydraulic radius scale from section to section. The velocity scale, as obtained from adjustment tests, was 1:7.6, compared to Froude value of 1:10. An investigation was made after the model adjustment to determine the effect of not adhering strictly to the Froudian relationships. It was demonstrated that the maximum probable error in water surface elevations would be about one-half of a foot in the prototype, whereas the overall accuracy of this model is about one foot (prototype).

59. Method of testing. With gages distributed around the model to correspond to prototype gage locations, the model is made to reproduce the stages for various flows observed in nature. This is the verification test and it is during this test that the various discharge velocity, and time ratios are varied along with the roughness, until the model reproduces the known prototype stages. After this test was made, the model was laid to a certain survey of the river channel, then the proposed changes to the river channel of the prototype were installed in the model and tested. A comparison of data from the two tests will then reveal the efficacy of the proposed improvements. As later surveys

are made, the model is changed accordingly, verified, and tests made on new improvement works as they are proposed. In this way, it is apparent that the various plans proposed can readily be compared and the best ones selected. Superfloods are studied on the model by determining maximum peak flows of tributaries plus an assumed value of rise in the Mississippi proper; this might give a flood of about three million second-feet. The model, however, is certified only for the maximum recorded flow, so that superfloods exceed the verification range. It is realized that the model data may be in error, therefore, but since the superfloods do not exceed the highest peak verified in the model by enough to cause serious error, a valuable estimate can be obtained.

60. Results of tests. With the aid of the study made on this flood-control model, together with the vast program started since 1928 to reduce floods on the Mississippi by use of cutoffs, floodways, emergency reservoirs, dredging, and levee systems, the stages of recent floods have been greatly reduced. By the time this program is completed to include flood-control measures on the tributary rivers, the Mississippi River will be under control for the first time in history.

HEAD OF PASSES MODEL STUDY

61. The problem area. That part of the Mississippi River called "The Passes," occurs at the lower extremity of its delta region, about ninety-four miles south of New Orleans. The point at which the main river channel divides into several channels is called "Head of Passes." The three main passes or channels occurring at this point are Pass a l'Outre, South Pass, and Southwest Pass (Figure 22).

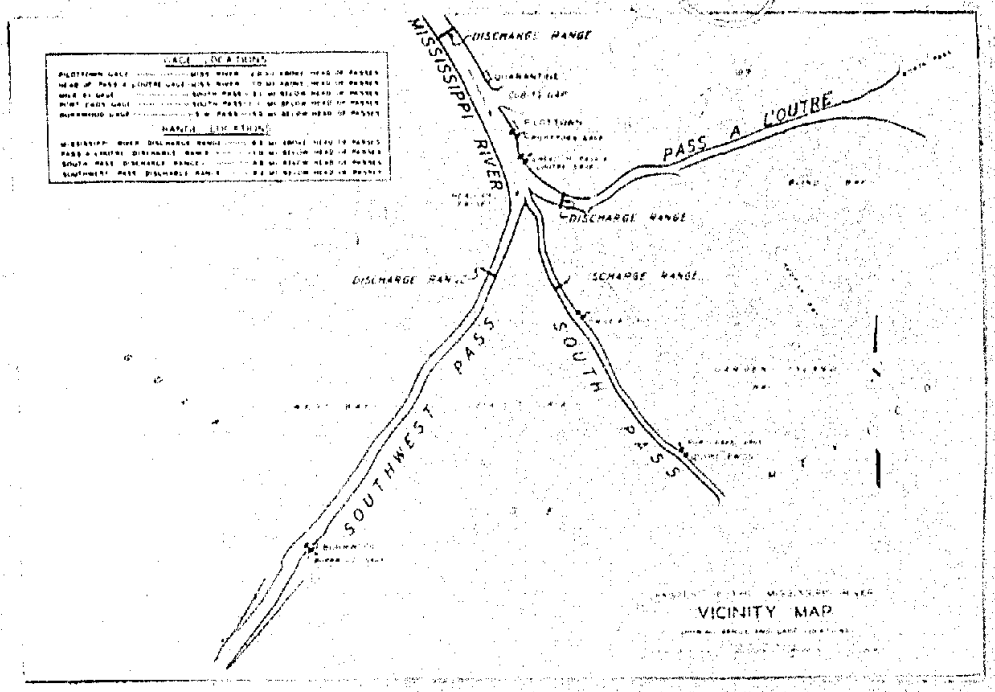


FIGURE 22, PASSES OF THE MISSISSIPPI RIVER, VICINITY MAP

Another pass, Cubit's Gap, occurs three miles upstream from the Head of Passes. South Pass, which is 13-1/2 miles long, and Southwest Pass, which is 20 miles long, are the only navigable channels, and as such must be kept open to navigation since considerable sea-going commerce uses these passes to reach New Orleans and Baton Rouge.

The task of keeping the channels open has kept the Corps of Engineers busy for many years. The land surrounding the passes is a salt marsh, typical delta country, which continually subsides. During floods the low banks are overtopped and sufficient material is deposited to correct the subsidence somewhat, but to maintain and strengthen the banks, resort has been made to permeable jetties and spur dikes. By so confining the flow and by dredging, a navigable channel has been maintained in South and Southwest Passes, but only as long as a proper distribution of flow is maintained in each pass, which, in turn, is dependent on the improvement works installed

to maintain navigable channels. Thus, a difficult problem is evident, particularly so when it is realized that the river branches at one main point, and is on a very flat slope.

Since the passes of the Mississippi River are so vital to sea commerce, a model study was instigated at the U. S. Waterways Experiment Station to study the effect on the discharge in the passes of a dredging operation to deepen the channels; to study the reasons for shoaling at the Head of Passes and the best dredging methods to be used to eliminate it; and to study the effect of gapping a submerged sill, which lies across Pass a l'Outre, to permit silt to fill a large scour pocket just below the sill along the right bank.

62. The model. The model was built to a horizontal scale of 1:500 and a vertical scale of 1:50. It includes all of South and Southwest Passes to the Gulf of Mexico, and a mile or two of Cubit's Gap and Pass a l'Outre (Figure 23).

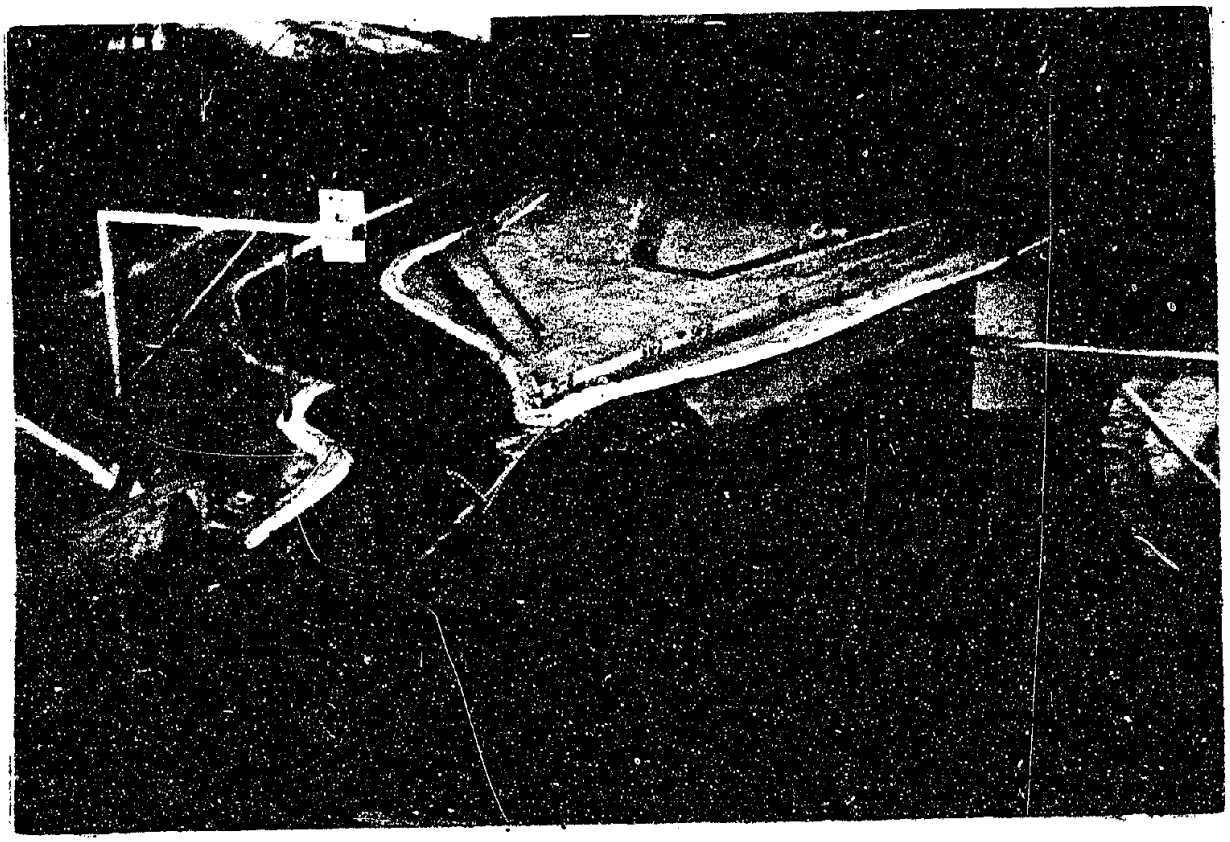
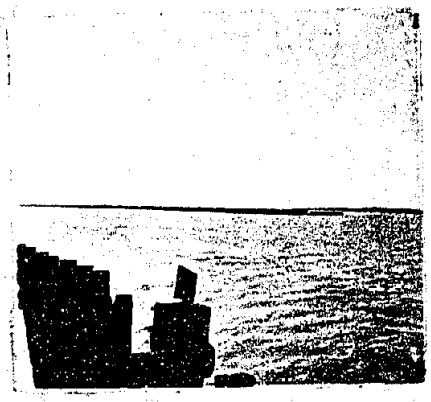


FIGURE 23, MODEL OF HEAD OF PASSES

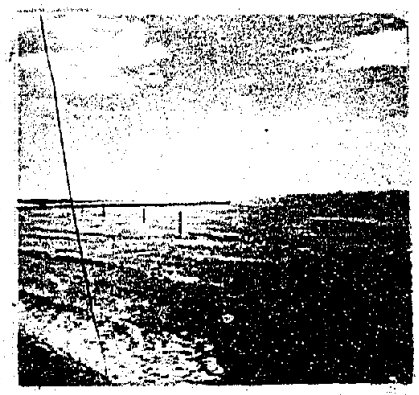
It was constructed according to the standard practice of river models explained in Chapter II. The movable bed was composed of pulverized coal. Where dredge cuts are planned, the concrete beneath the coal has been poured in blocks to enable its removal as desired.

63. Verification tests. The model was verified according to a 1937-1939 survey, with a hydrograph corresponding to that period. Because of the extent of the model and the difficulties in adjusting the scale ratios pertaining to the flow, the region at the Head of Passes was verified first, and later on each pass was treated separately for verification.

This verification was made with a movable bed prior to studying the shoaling action at the Head of Passes. In a previous test using a fixed bed in the model, a study was made of the effect on discharge distribution in the passes due to a dredge cut at the foot of Southwest Pass, and to determine whether the progressive decrease in the percentile discharge in Southwest Pass, as observed in the prototype between 1937 and 1940, was due to regulating works installed during that period. By obtaining a discharge distribution on the model for one set of conditions, and comparing it to a condition, including the regulating works and dredge cut, it was possible to show that the discharge distribution would be affected. The testing had not progressed far enough during the author's study of the model to give any solution of the other problems mentioned. After reviewing the model study as far as it had progressed, an inspection was made of the problem area in the field in company with the engineer in charge of the model study. This aided considerably in understanding the difficulties involved in the problem and its magnitude. Figure 24 shows some scenes of The Passes.



A. HEAD OF PASSES. SOUTH PASS LEFT, SOUTHWEST PASS RIGHT



B. FOOT OF SOUTHWEST PASS LOOKING TOWARD GULF OF MEXICO

FIGURE 24, PASSES OF THE MISSISSIPPI RIVER

GALVESTON BAY

64. The prototype. Galveston Bay, located in southeastern Texas on the Gulf of Mexico, is approximately sixty miles west of Port Arthur, Texas, and fifty miles south of Houston, Texas.[5] The entire bay area, which covers approximately four hundred and seventy-five square miles, is relatively shallow, varying from 7 to 9 feet in depth, with the exception of Bolivar Roads, and the ship channels which are maintained by dredging. The bottom of the bay is composed of silt which is kept in a more or less constant state of agitation by the action of waves and tidal currents. As a

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[5] "Model Study of Plans for Elimination of Shoaling in Galveston Channel and Connecting Waterways, Galveston, Texas," Tech. Memo, No. 127-1,

result, heavy shoaling occurs in Galveston Channel, and in several others in the bay. It is believed that the principal sources of shoaling are the various spoil banks over the area which have been formed by hydraulic dredging.

65. Need for model tests. Because of the constant need for dredging and danger to shipping entering the bay, several plans were proposed to eliminate these unfavorable conditions. Since the efficiency of the plans could not be readily determined due to lack of precedence to follow and because no rational analysis was possible, resort was made to hydraulic model tests.

66. The model. After careful study of the prototype phenomena, a model was designed to give reliable and useful information. The model represented approximately two hundred and sixty-five square miles of the prototype with a horizontal scale of 1:800 and a vertical scale of 1:80. It was of the fixed-bed type, all topography being laid in the model with sheet metal templets and molded in concrete. Appurtenant works of the model consisted of water-surface gages, automatic tide-control and tide-recording apparatus, tide clock, and wave machine. Gilsonite, having a specific gravity of 1:03 to 1:04, was used to approximate the silt of the prototype for use in the shoaling tests.

67. Scope of tests. Careful study of pertinent field data gave information as to how the model tests should be made to best reproduce the prototype phenomena. Tides, currents, wind, and silt samples were examined and properly coordinated to permit the model operation. The model tests were divided into three distinct phases as follows: (1) The adjustment and verification; (2) the clear-water tests; and (3) the shoaling tests.

The adjustment and verification phase consisted of producing tides by the tidal machinery in the model and observing the tides and the currents over the problem area. Comparison with field data, disclosed the necessity for increasing the roughness of the model surfaces,

which, when properly accomplished, gave good agreement with the prototype. At the same time, it was found that the currents agreed very well with the field measurements.

The clear-water test phase was initiated only after the model had been properly adjusted to reproduce accurately certain known phenomena at the prototype area. The phase consisted of testing various plans to determine their effects on the tides and currents over the problem area, and thereby to determine their effects on shoaling in the various channels.

The shoaling tests consisted of silt-source investigations to determine on the model the source of silt in the prototype. Ten possible locations were tried, including all of the spoil area, as well as intermediate positions between. Four of the source locations were then studied to provide silt-path base tests, the locations used having been found to be the principal contributors to shoaling in the ship channels. Following these tests, two plans for improvement were tested by introducing shoal material into the model and then studying the effect of these plans on the deposition of this material.

68. Clear-water base test. The several plans tested in the model, with one exception, consisted of placing single dikes at critical locations over the model area for improving one or more of the main ship channels. Before proceeding with any of these tests, a base test was made to establish a basis of comparison. The conditions of the prototype at the start of the test were selected as the basic conditions, and a base test made of them, consisting of a comprehensive study of the tides and currents in the model, with only existing improvement works in place. The base test was actually the results of the adjustment and verification test, since the model was constructed initially to represent the existing conditions.

69. Example of clear-water test. Plan 1 was based on the probability that a dike extending from Pelican Island to Galveston Island,

closing the western end of Galveston Channel with a 200-foot opening for passage of traffic, would reduce the rate of shoaling in Galveston Channel and possibly benefit connecting waterways.

The results of tests on Plan 1, indicated that shoaling in the western section of Galveston Channel would probably be considerably decreased, while that in the eastern 4,000 feet would not be decreased to the same degree. The conditions in Texas City Channel were such as to indicate an increase in tidal prism, though not necessarily an increase in shoaling. There was no indication of change in the action in the jetty channel, Houston Channel, or Bolivar Roads.

70. Shoaling tests. Plan 1 was also selected for a shoaling test to determine, as far as possible, the effects of the closing dike on the shoaling action in various channels. The information from this test was qualitative only and merely supplemented the data obtained from the clear-water study of the tide and currents. The silt path tests enabled a distinction to be made between Plan 1 and any other plan. They were performed by placing shoal material (gilsonite) in strategic locations, operating the model according to predetermined tidal cycles, and observing the deposition and movement of the gilsonite. A base test was first made, however, in a similar manner in order to compare the performance of proposed improvements. The base test for shoaling revealed that the spoils banks produced by hydraulic dredges in maintaining navigable channels were important contributions to the shoaling in the ship channels, and indicated that dredging methods, regardless of improvement works, could be revised.

Tests of Plan 1 were judged by "individual-shoal indices" which were the ratios obtained by dividing the quantity of material deposited in the test of the improvement plan by the quantity of material deposited in the base test. Computations were then made to weigh the various figures obtained from the silt-path or shoaling tests of Plan 1, as compared with the base test, and to determine for each channel in the problem area an overall valuation expressing the merit of the proposed

improvement plan by a "final channel index." From a study of these data for Plan 1, the "final channel index" indicated a considerable reduction in shoaling in the western section of Galveston Channel, an increase in shoaling in the channel between the jetties, and some reduction in other channels.

71. Recommendations. One of the most valuable contributions from this study was the elimination of worthless improvement plans. It was convincingly shown from the clear-water tests that Plan 1 was the best of five plans, while the shoaling tests revealed Plan 1 to be the best of two under consideration.

Analysis of all results definitely showed that the reduction of shoaling would be accomplished if the dredging methods are revised so that material once removed from ship channels will have no chance to be returned to the channels by tidal action. For this purpose, it was recommended that more use be made of hopper dredges, the material being dumped at sea, and the use of spoil areas in locations where tidal currents will not return the material, namely, in the bay area to the east of the Houston ship channel, If dredging changes alone are not sufficient, it was recommended, in part, that consideration be given of Plan 1, or minor adjustments thereto. In any event, the intra-coastal waterway opening should be kept to the minimum possible for navigation.

HYDRAULIC MACHINERY

72. References. Because hydraulic machinery problems pertain more to mechanical engineering and, as such, are not too familiar to the author, and since the testing of such models is fairly standard, as briefly described in Chapter II, examples of hydraulic machinery model tests are not reported upon. Since turbine and pump manufacturers do not generally permit a perusal of their reports, but discuss their problems only in technical literature, references are given below to papers illustrating the technique of this type of model testing and the design problems involved; some of the references at the end of Chapter II are also apropos:

"Development of the Automatic Adjustable-Blde-Type Propeller Turbine," by R. V. Terry, A.S.M.E. ? 1941.

"Complete Characteristics of Centrifugal Pumps and Their Use in the Prediction of Transient Behavior," by R. T. Knapp, Trans. A.S.M.E., November 1937.

"Experimental Determination of the Flow Characteristics in the Velocities of Centrifugal Pumps," by R. C. Binder and R. T. Knapp, Trans. A.S.M.E., November 1936.

LABORATORIES

73. Model testing. Hydraulic model testing is being conducted at many laboratories in the United States. The leading laboratories in this field in 1940-41 being at the U. S. Waterways Experiment Station, Vicksburg, Mississippi; Bureau of Reclamation, Denver, Colorado; and at the Tennessee Valley Authority, Norris, Tennessee. Other organizations doing this work, but somewhat less extensively, are: Bonneville Hydraulic Laboratory, Bonneville Dam, Washington; National Hydraulic Laboratory, Bureau of Standards, Washington, D. C.; U. S. Engineer Offices at the University of Iowa, Iowa City, Iowa; at Los Angeles, California, at Worcester Polytechnic Institute, Worcester, Massachusetts; and at Carnegie Institute of Technology, Pittsburgh, Pennsylvania; and the California Institute of Technology, Pasadena, California.

CHAPTER IV - FUNDAMENTAL RESEARCH

INTRODUCTION

74. Empiricism versus rationalism. In the previous discussion, the role of hydraulic models in hydraulic research has been demonstrated. One fact is evident from this type of research: The information gained is usually applicable only to the specific problems involved, although such information may be invaluable in solving similar problems. Thus, the model studies of Friant Dam solved problems associated with the hydraulic design of that structure; the model studies of the Mississippi River likewise answered specific questions, yet the knowledge gained would generally not be applicable to other structures or rivers unless the hydraulic conditions were nearly similar, a rare circumstance.

Consider now the role of fundamental research in hydraulics. In this case, experiments are made of various flow phenomena to determine the reasons for their behavior and to relate all the variables by dimensional analysis to establish laws governing these phenomena, laws which are applicable over a wide range of conditions and which may facilitate the transfer of results from one field to other related fields of research. Rational formula expressing these laws are dimensionally correct, while many empirical functions are dimensionally incorrect, their successful use being dependent on the judgment of engineers to use them only within the experimental range from which they were derived. Accordingly, empirical data may be interpolated but rational data may be extrapolated in most instances. For example, it has been shown in pipe flow that the friction coefficient and velocity distribution are a function of Reynolds number and that reasonable extrapolation is justified. Perhaps the extrapolation of empirical data is encouraged because almost all empirical functions plotted on logarithmic paper appear as a straight line for small ranges of data. Even though such pipe flow data is empirical, all the variables are included and the limited functions expressing pipe

resistance probably point the way to theoretical or rational expressions for the same phenomenon. The theoretical formula for fluid friction in a smooth pipe developed by von Karman is based on the assumption that the influence of viscosity is negligible except within a small range near the walls. Dr. von Karman believes it is probable that this function will agree with the facts more and more, the larger the Reynolds number corresponding to the actual construction.

In regard to empirical versus rational or analytical investigation, Rouse states that there are four, rather than two, methods of developing expressions for fluid motion, but one must distinguish first between experimental measurement and empirical formulation of working rules for design. Experimental measurement is the mechanical determination of one or more flow characteristics for a given state of motion. Pure empiricism, on the other hand, includes the effort to develop a practical generalization of the results of experimental measurement for a number of different conditions; it may provide simple working formulas for design within the range of available data, but, although the data may be accurate, there is no guaranty whatever for the physical truth of the empirical statement, for only by chance are natural laws discovered in this way. Were this still the only recourse of the hydraulician, the subject of pipe resistance would probably be in the same jumbled state as it was before Blasius published his analysis of Schoder's experimental data.

On a considerably higher plan, Rouse explains, is the second method, that of Blasius, which is partly analytical in character, since it is based upon physically sound dimensional analyses; although experimental measurements are still necessary to determine the type of function and the numerical constants relating the several dimensionless parameters, the investigator is well beyond the chance of pure empiricism when he interprets his results. Still further advanced is the method combining dimensional analysis with a reasonable and closely

approximate physical analysis, thus determining the probable form of a function and leaving only the numerical constants to experimental investigation. The fourth method, he concludes, that of complete rational analysis, leaves nothing to be found experimentally; it is the ultimate goal of every field of science, but, as yet, only in isolated cases has it been fully a success.

Although it is obvious that the rational or analytical methods are superior, engineers in practice usually must rely on methods which will yield results quickly, accurately, and economically. To attempt to solve some of the problems discussed in Chapter III by fundamental analyses would be impractical and would yield, if at all, results of a questionable nature. Accordingly, empirical methods are predominant but fundamental research is gradually advancing our knowledge of fluid motions and will eventually reduce the amount of empiricism required by the hydraulic engineering profession to solve its problems.

75. Principal research. Perhaps the predominant research project in the United States today is that related to fluid turbulence. What is turbulence and how can a knowledge of it be applied to practical problems? This and other pertinent questions may be partly answered by reviewing the work already accomplished. The next few pages, therefore, will be devoted to show what is meant by turbulence and how practical problems are being solved by application of some of its concepts. In addition, other types of fundamental research will be illustrated, which are not as important, perhaps, as the study and application of turbulence to practical problems, but of considerable significance.

FLUID TURBULENCE

76. Nature of turbulence. Starting with Osborne Reynolds' experiments, later followed by the work of Blasius, Stanton and Pannell, Lees, and Nikuradse, the relation between Reynolds number

and the friction coefficient for fluid flow in smooth and rough pipes was fairly well developed for laminar and turbulent flow. At the same time, theories were presented relative to the boundary layer by Prandtl, the Prandtl-von Karman theory of the mixing length, the significance of the laminar boundary layer with relation to relative roughness, and von Karman's universal relationship between the velocity distribution and resistance to flow.[6]

During the past century most of these studies have been concerned with understanding the nature of resistance in turbulent flow; more recently, research has attempted a solution from an analytical basis. The first successful approach to the analysis of turbulence came from the aeronautical engineering field in which many problems of fluid motion had to be met, including turbulence. Since turbulence is general in its nature, a study of it by the methods of fluid mechanics is as applicable to hydraulic as aeronautical engineering.

Those somewhat familiar with the modern studies of turbulence, but engaged in practical engineering work, might ask what is turbulence in flowing water and of what value is it to know the application of the theoretical findings. Rouse answers that the engineer, first of all, thinks of turbulence in water as consisting of merely large eddies or severely agitated flow. Although such flow is turbulent, the true meaning of turbulence is related first to Reynolds' classic experiment in a long glass tube in which dye was introduced into the flowing water to observe the change from laminar to turbulent flow. In this small tube or in hydraulic structures fully developed turbulence is the same. Basically, turbulence exists in a fluid when at any point the direction and magnitude of the velocity vary irregularly with time. Such variation is rapid and is usually produced by the whirling about in the fluid of

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[6]A resume of these developments may be found in "Fluid Mechanics for Hydraulic Engineers," by Hunter Rouse, Engineering Societies Monograph, McGraw-Hill, 1938; or "Modern Conceptions of the Mechanics of Fluid Turbulence," by Hunter Rouse, Trans. A.S.C.E. 1937, p. 463.

eddies of varying size. It is by these eddies that the transfer of momentum, mass, and heat occurs from one point to another in the fluid, causing the energy dissipation associated with turbulent flow. Accordingly, at any instant there may be superimposed upon the translating movement of a fluid past a certain point either positive or negative velocity components in each of the three coordinate directions. As a result, there is a variable movement of small fluid masses in a direction normal to the general direction of flow. An illustration of this somewhat complex motion can be had from observing dense clouds of smoke rising from a chimney. The flow or motion is changing from instant to instant, but a typical sequence of rolling billows of smoke is present, the general movement of which is by no means the individual helter-skelter of minute particles of soot. It is not to be construed, however, that each fluid particle describes its own irregular path, regardless of the motion of neighboring particles; rather, it is to be understood that the behavior of every particle is dependent upon that of others in its immediate vicinity.

77. Statistical theory. Since the turbulence phenomenon is one of random motion with no regularity, it is convenient to study it from a statistical analysis. Following the usual pattern of fundamental research, theories have been first developed which are then followed by experiments to check these theories and/or to determine certain constants included in the fundamental analysis. Prandtl, Taylor, and von Karman have developed the basic turbulence theories, although the theories are not as yet complete and valid. At present, therefore, experimenters in this country are checking the theories and developing techniques for laboratory work on turbulence. At the same time, attempts are being made to present the various findings to engineers and to show applicability to present-day practice.

In applying the statistical analysis to turbulent flow,[7] it is convenient to represent the varying velocity vector at any point in

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[7] "Relation of the Statistical Theory of Turbulence to Hydraulics," by A. A. Kalinske, Trans. A.S.C.E. 1940, p. 1547.

the fluid by U, V, and W along the axes x, y, and z. The velocity in the direction of mean flow along the x-axis at any instant is represented as $(U \pm u)$, in which U is the mean velocity in that direction and u is the fluctuation in that direction. Along the other axes, y and z, the velocity is represented by $(V \pm v)$ and $W \pm w)$, respectively. Since u, v, and w are quantities that vary with time, it is useful to represent them by certain statistical averages. First, the statistical distribution of these quantities is determined by use of a block diagram indicating the frequency of occurrence of various values of v, for example. This function in reality is the ordinary normal error law.

To determine the simultaneous values of u and v, or u and w in water, a study is made of the motion of particles suspended in flowing water. When illuminated, such particles cause streaks on motion-picture film, and, from the length and direction of the streaks, two components of the velocity can be determined. Particles made of carbon tetrachloride and benzene, having a specific gravity of water, have been used. These particles are illuminated by a narrow plane of light, so that only those particles in the light are visible. Kalinske, at the University of Iowa, obtained the transverse velocity v for short time intervals by injecting a thin color stream of dye into a flow by a fine needle and taking motion pictures of the color stream close to the needle. The velocity v for these short intervals of time was obtained by measuring the transverse travel Y for short distances, x, downstream. The values of x are short enough so that the direction of the color stream is really a straight line. The value of v is then $\frac{Y}{t}$, in which t is $\frac{X}{U}$.

A large number of values of v enable the calculation of $\sqrt{\overline{v^2}}$. Kalinske found that good measurements of simultaneous values of u and v can be made photographing with a constant-speed motion-picture camera the travel of immiscible droplets in an open channel for a distance of about one inch from the point of injection. A magnifying type of close-focusing lens is used on the camera so that a picture is obtained of an

area of about two square inches. Following the movement of the particles from frame to frame on the film, permits the determination of u and v, and, if sufficient data are obtained, the quantities $\sqrt{\overline{v^2}}$, $\sqrt{\overline{u^2}}$, and $\overline{uv}$ can be calculated.

John S. McKnown at the University of Minnesota, to determine the velocity fluctuations in turbulent flow, used a narrow channel with a grid painted on the bottom and one sidewall. By placing a mirror above the bottom and adjacent to the sidewall, moving pictures could trace the movement of a specially prepared droplet in the three directions, x, y, and z. Although the photographic method is accurate for studying velocity fluctuations, it is agreed by those concerned that it is exceedingly tedious to obtain and analyze the data. An instrument that would give accurate recordings of the instantaneous velocity would be welcomed.

Having determined the distribution law or normal error law indicating the random nature of the velocity fluctuations, the next step is to establish the degree of turbulence by some average value of the quantities u, v, and w. The root-mean-square or standard deviation, $\sqrt{\overline{v^2}}$, is the most significant average. It indicates the spread of the distribution curve and its square $\overline{v^2}$ is proportional to the energy of turbulence for that velocity component. Another factor useful for treating turbulence is a correlation coefficient. Since two different turbulent flows may have the same intensity, but the average size of the eddies different, Taylor[8] has suggested that a correlation coefficient, $R_x = -\frac{\overline{v_1 v_2}}{\overline{v^2}}$ be used. In this analysis the instantaneous velocity components, either u or v, are measured simultaneously at two different points, along either the x-axis or the y-axis, over an appreciable period of time. If the two points are near enough that they are within the average-size eddy, v_1 and v_2 will tend to be nearly equal and of the same size, and R_x will be nearly unity. If the two points are an average eddy diameter or more apart, R_x will approach zero.

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[8] "Statistical Theory of Turbulence," by G. I. Taylor, Proc. Royal Society of London, Vol. 151A, 1935, p. 421.

This has been verified by experiments. Kalinske showed from measurements of the transverse velocity v that $R_x = \frac{\overline{v_1 v_2}}{\overline{v^2}}$ would approach unity for small values of x, and would approach zero for larger values of x. Taylor then defined a quantity characteristic of the turbulence thus:

$$\ell' = \int_0^x R_x dx$$

in which x is the distance when R_x approaches zero.

78. Turbulence and energy. In addition to the knowledge of the transverse velocity fluctuations and the relative intensity of turbulence, a concept of the energy of turbulence is valuable. If the turbulence is uniform in the direction of flow, as in pipe flow, the energy present due to turbulence may be neglected in any energy equation. But, if the energy of turbulence varies from point to point, as occurs in conduit transitions, downstream from valves, bends, etc., a slight error will be introduced in a study of energy changes. Turbulence produces energy dissipation through the medium of viscosity, for without viscosity, either in viscous or turbulent flow, energy dissipation would not occur. Without viscosity, there would be practically no loss of energy, so that any turbulent energy produced would persist indefinitely. Accordingly, the high internal shear stress produced by the turbulent eddies causes the energy dissipation. In pipe flow, potential energy is being dissipated into heat. The rate at which potential energy is transferred into turbulent energy is not necessarily equal to the rate of energy dissipation into heat at any point since the turbulent energy can be diffused to other points by the action of eddies and then dissipated into heat.

For isotropic turbulence, Taylor shows that the mean rate of dissipation of energy per unit volume of fluid is $K + 7.5\mu\overline{\left(\frac{\partial v}{\partial v}\right)^2}$; that is, the rate is proportional to the viscosity μ times the mean square of the various instantaneous turbulent velocity gradients, such as $\overline{\left(\frac{\partial v}{\partial x}\right)^2}$. To determine the value of $\overline{\left(\frac{\partial v}{\partial x}\right)^2}$, for example, use is made of the above-mentioned correlation coefficient. The coefficient

R_x is so defined that $R_x = 1 - \frac{\overline{(v_1 - v_2)^2}}{2\,\overline{v^2}}$ where v_1 is the velocity at one point and v_2 at another point in a fluid at the same instant, at a distance x apart. If the values $\overline{(v_1 - v_2)^2}$ are obtained for small values of x, then it is approximately correct to write:

$$\frac{\overline{(v_1 - v_2)^2}}{x^2} = \overline{\left(\frac{\partial v}{\partial x}\right)^2}$$; substituting in the expression for R_x yields $\overline{\left(\frac{\partial v}{\partial x}\right)^2} = 2\,\overline{v^2} \lim_{x \to 0} \left[\frac{1 - R_x}{x^2}\right]$; therefore,

from Taylor's concept $K = \frac{15\,uv}{\lambda^2}$, in which $\lambda = \lim_{x \to 0} \left(\frac{x}{\sqrt{1 - R_x}}\right)$.

Here λ is a length term and is a characteristic of the turbulence as regards energy dissipation.

79. Diffusion. An important part of turbulence investigation is the diffusing or exchange power causing the exchange of heat, momentum, and matter from one part to another. Such a mixing process is sometimes called "eddy diffusion." Kalinske studies this phenomenon by injecting a constant stream of color through a fine tube, and also by injecting colored droplets of a mixture of carbon tetrachloride and benzol, having the same specific gravity as water, and photographing the transverse spread of the color, or droplets, with a motion-picture camera. The important quantity to determine from these studies is the mean-square transverse travel of the color or droplets at various distances downstream from the point of injection. If the transverse travel at any distance x downstream is Y and $\overline{Y^2}$ its mean square, Taylor[9] has related in theory x and $\overline{Y^2}$. Kalinske's experiments in a small channel show a linear relationship at larger values of x (5 to 8 inches).

A mathematical relationship between x and $\overline{Y^2}$ is not as convenient as a diffusion coefficient for characterizing the diffusing power of turbulence. Applying the ideas of molecular diffusion theory, such a coefficient would be defined as $D = \frac{U}{2}\left(\frac{d\overline{Y^2}}{dx}\right)_{max}$. The dimensions

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[9] "Diffusion by Continuous Movements," by G. I. Taylor, Proc. London

of D are $\frac{L^2}{T}$, a velocity times a length, the velocity being the root-mean-square transverse velocity, $\sqrt{\overline{v^2}}$, and the length is associated with mixing processes being proportional to the size of the turbulent eddies and to Prandtl's "mixing length."

80. Energy dissipation. There are several problems in hydraulic engineering in which the knowledge of turbulence is important. Considering the general problem of energy dissipation, there are various problems requiring a knowledge of the manner in which energy is dissipated. Some hydraulic problems require a maximum dissipation while others may require a minimum dissipation of energy. In a conduit of expanding cross-section, that is, a transition section, the additional losses of energy associated with the transition are traced to the creation of turbulence energy beyond that usually present in the flow.

In order to study the energy transformations completely, the energy of turbulence must be found in addition to the ordinary kinetic energy computed from the average velocity distribution. The total mean kinetic energy for a pipe may be expressed as:

$$E_m = \pi\rho \int_0^r y\, U^3\, dy$$

in which U is the mean velocity, r the radius, y any distance from the centerline of the pipe and ρ is the density. The total energy of turbulence is expressed as:

$$E_t = \pi\rho \int_0^r yU\, (\overline{u^2} + \overline{v^2} + \overline{w^2})\, dy$$

in which $\overline{u^2}$, $\overline{v^2}$, and $\overline{w^2}$ are the mean squares of the velocity components in the x, y, and z directions, respectively. Other values are as above.

In a 3-inch pipe, Kalinske found the total mean kinetic energy E_m = 0.255 foot-pound for a discharge of 0.08 second-foot. The value of E_t was 0.008 foot-pound, assuming the transverse velocity component $\overline{w^2}$ was equal to $\overline{v^2}$, which is true for flow in a circular conduit except at the wall. For these values, E_t was 3 percent of E_m. In a 3- by 5-inch expansion in the pipeline at the 3.75-inch diameter of the expansion, the value of E_m was 0.157 foot-pound while E_t was 0.014 foot-pound, or 9 percent of E_m. Hence, in the transition, the turbulent energy E_t had had a higher value.

Turbulent energy, as this example shows, can be created rapidly, but the dissipation of that energy does not occur so quickly. Since a sudden change in velocity causes extreme turbulence, the energy so produced is dissipated gradually downstream due to viscosity, without which there would be no dissipation of energy into heat.

The theory of turbulence mechanism can be applied to the dissipation of energy at hydraulic structures, such as below overfall dams, spillways, or to decrease the velocity of flow in open channels. For fully developed turbulent flow, the shear stress is equal to $\rho\varepsilon \frac{dU}{dy}$, in which ε is proportional to the diffusion coefficient, or to the quantity $\sqrt{\overline{v^2}} \cdot \ell$, in which $\sqrt{\overline{v^2}}$ is the intensity of the transverse turbulent velocity and ℓ is a factor determined by the size of the eddies. Accordingly, the rate of potential energy transformation can be increased, assuming the discharge constant, by increasing ε, which can be accomplished by introducing large-scale roughness to produce large, intense eddies. For a certain intensity of turbulence, the smaller the eddies the higher the rate of energy dissipation, as may be seen from the above-mentioned relation for rate of energy dissipation, $K = \frac{15 \mu \overline{v^2}}{\lambda^2}$. Since λ is a length, which is related to the correlation coefficient R_x, and is characteristic of the energy dissipation in turbulence, then λ will be smaller for small eddies so that K will be greater. It is concluded, then, that once the potential or mean kinetic energy of flow is in the form of turbulent energy, the dissipation of this energy can be quickened by the breaking down of large eddies into small ones. For flow in a pipe or channel in converting potential energy into turbulent energy, the mean velocity can be reduced by the creation of intense large-scale eddies. To destroy the kinetic energy of high-velocity flow below a dam, it is desirable to produce intense, small-scale eddies. If model studies are made of problems pertaining to energy dissipation, Kalinske concludes that, in order for such studies to be fundamentally sound, dynamic similarity must be maintained as far as intensity and scale of turbulence are concerned. Turbulence should be thought of in terms of definite parameters

instead of just as a qualitative descriptive term relating to a general flow condition.

81. Summary. Conclusions from Kalinske's experiments and his discussion just given show a proper concept of turbulence by introducing such terms as intensity, scale, and energy of turbulence. Although not new, these terms are introduced for use by the practical hydraulic engineer. A statistical analysis of velocity fluctuations, both parallel and normal to the direction of flow, revealed that the velocity fluctuations in true turbulence are statistically distributed according to the normal error law. Checks of Taylor's theory of turbulent diffusion were made by experimental means, using motion pictures to obtain the required data. The recognition of energy of turbulence as part of total kinetic energy in turbulent flow is required to determine a full picture of the phenomena involved. Practical application of these theories is yet to be made with complete understanding, but, in the field of sediment transportation, the pioneer application will be discussed below.

The work of Rouse, Kalinske, Bakhmeteff, and others in the field of turbulence as applied to hydraulic engineering is commendable, and it is to be hoped soon that applications of the theories to practice will convince skeptical engineers of the importance of at least thinking in terms of the concepts of turbulence. Unfortunately, this will take considerable time and effort and must require tolerance by these research men of the point of view of practical engineers. For example, Rouse[10] suggests that,

> "... many so-called 'energy dissipators' could be made more effective and less dangerous in their downstream influence if designers realized not only that the formation of large-scale eddies effectively reduces the mean velocity of flow but also that the presence of such eddies is sometimes as harmful as

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[10] Trans. A.S.C.E. 1940, discussion, p. 1568.

that of a high mean velocity; as pointed out by Professor Kalinske, the production of intense small-scale eddies (approaching the state of isotropic turbulence) is an essential means of hastening energy dissipation."

With the exception of structures designed many years ago, it is the author's conviction that designers today fully realize the facts pointed out by Rouse and Kalinske, especially since the hydraulic model so quickly and efficiently reveals any excessive eddy formation or high mean velocities. Those of more experience with model studies of hydraulic structures and their design problems are quick to recognize faulty design, sometimes before model tests begin, and the theory of turbulence mechanisms is not essential to this analysis; rather, the methods of obtaining sufficient energy dissipation by use of large or small eddies is the problem. Certain limitations of design frequently prevent the ideal hydraulic solution, so, to some, a structure may seem to perform in an inefficient manner.

TURBULENCE CONCEPTS APPLIED TO SEDIMENTATION

82. Transportation of suspended material. A problem of great interest in hydraulic research seems to be the study of bed load and suspended sediment in rivers. The importance of gaining more and more knowledge on this subject is readily realized when it is considered that all of our large power development, irrigation, and flood control projects are vitally affected by the amount of sediment carried in streams and the amount deposited in reservoirs. Although it is comparatively easy to estimate the amount of sediment carried in a stream, the problem lies in simplifying the analysis so that laws of transportation can be developed relating certain properties of the individual stream. Bed load and suspended load act quite differently, so that separate laws must be found for each. On the face of it, it is hard to imagine that any law or laws could be found which would predict the sediment transportation of a stream, considering the mass of water and silt so intermixed and changing. Fortunately, research

engineers have made rapid progress in this problem, and, by applying the knowledge gained from turbulence studies, certain laws have been evolved and checked both in the laboratory and field.

It is not the purpose here to go into detail regarding all the developments made so far, but rather to show how new developments have been proposed by research relative to suspended sediment transportation. The development and clarification of the theory of suspended load are due mainly to Leighly[11], O'Brien[12], von Karman[13], and Rouse[14]. It has been known for considerable time that, if the average concentration of suspended sediment at any point in a wide stream was known, the average concentration at any other point could be determined. This has been demonstrated from actual measurements in streams and channels by Christiansen[15] and Richardsen[16]; for artificial turbulence, it has been checked by Rouse[14]. To develop this for equilibrium conditions, the general statistical equation may be written:

$$\uparrow \overline{vN} = \overline{cN} \downarrow$$

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[11] "Toward a Theory of the Morphologic Significance of Turbulence in the Flow of Water in Streams," by J. B. Leighly, University of California Pub. in Geog., Vol. 6, No. 1, pp. 1-28, Berkeley, 1932.

[12] "Review of the Theory of Turbulent Flow and its Relation to Sediment Transportation," by M. P. O'Brien, Trans. American Geophysical Union, pp. 487-491, 1933.

[13] "Some Aspects of the Turbulence Problem," by Th. von Karman, Proc. Fourth International Congress of Applied Mechanics, Cambridge, England, 1934.

[14] "Experiments on the Mechanics of Sediment Suspension," by Hunter Rouse, Proc. Fifth International Congress of Applied Mechanics, p. 55, 1938.

[15] "Distribution of Silt in Open Channels," by J. E. Christiansen, Trans. American Geophysical Union, Part II, 1935, p. 478.

[16] "Suspension of Solids in a Turbulent Stream," by Richardsen, Proc. Royal Soc. London, Vol. 162, p. 583, 1937.

in which v = instantaneous vertical velocity component, N = instantaneous sediment concentration per unit volume, c = average velocity of fall of single sediment particles in water; and $\overline{N}$ = mean sediment concentration at any point above the stream bed. The left side of the equation indicates the net transport of material upward by the turbulence velocity components while the right side is the quantity that settles due to gravity. They are equal for equilibrium conditions. At any instant N can be expressed as $(\overline{N} \pm \Delta N)$, so that the value of vN is $(v\overline{N} \pm v\Delta N)$. When v is upward, ΔN will be positive; when v is downward, ΔN will be negative since the sediment concentration increases from top to bottom in the stream. The mean value of vN is then $+ \overline{v\Delta N}$, since the mean value of $v\overline{N}$ is zero, because the arithmetic mean value of v is zero. The magnitude of ΔN depends on the distance which the suspended material moves up or down, or on the scale of the turbulence or size of eddies. If we introduce a length factor, ℓ, characterizing the diffusing scale of the turbulence, then $\Delta N = \ell \cdot \frac{dN}{dy}$, and the equilibrium equation becomes: $\overline{v\ell \frac{dN}{dy}} = c\overline{N}$, since the mean value of $\overline{vN}$ was taken to be $+ \overline{v\Delta N}$. The term $v\ell$, which may be called ε, is proportional to the diffusion coefficient defined above in Section 79. It has been assumed, and some preliminary studies indicate it to be so[17], that this mixing or diffusion coefficient is identical with the momentum exchange coefficient ε_m in the formula for shear stress caused by turbulent exchange of momentum, $\tau = \rho\varepsilon_m \frac{dU}{dy}$. Integration of $\overline{v\ell \frac{dN}{dy}} = c\overline{N}$ gives a relation between the unknown sediment concentration $\overline{N_a}$ at a point "a" distance above the bottom of the stream:

$$\log_e \frac{N}{N_a} = -c \int_a^y \frac{dy}{\varepsilon} \text{ - - - - - - - - } (1)$$

In order to integrate the right side of this equation, the variation of ε with y must be established.

83. Calculation of suspended sediment. Lane and Kalinske demonstrate a method of establishing this variation and offer valuable

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[17] "Turbulence in Open Channel Flow," by A. A. Kalinske and J. M. Robertson, Engineering News-Record, April 10, 1941, p. 53.

equations for practical use[18]. The turbulence theory shows that at any point the unit shear, $\tau = \rho\varepsilon \frac{dV}{dy}$ where V is the mean velocity at any depth y, and ρ is the water density ($\frac{w}{g}$). Now, if the variation of τ and V with depth is obtainable, then ε can be determined for various depths y. Confining an analysis to wide streams, it is known that $\tau = \tau_o \left[1 - \frac{y}{D}\right]$, where τ_o is the bottom shear and equal to wDS, where D is the depth and S is the friction slope.

The vertical distribution of velocity varies with the stream cross-section. For wide streams, it is closely approximated by the Prandtl-von Karman logarithmic curve for velocity distribution in turbulent flow, thus:

$$V/V_m = 1 + \left(\sqrt{gDS}/KV_m\right) \left[1 + \log_e \frac{y}{D}\right] \quad - - - - - (2)$$

where V_m is the mean velocity in a section and K is von Karman's universal constant equal to about 0.40. Using the above velocity distribution relation (2) and the relationship for unit shear for wide streams, an expression for ε is obtained as follows:

From (2): $V = V_m + \frac{\sqrt{gDS}}{K} \left(1 + \log_e \frac{y}{D}\right)$

or $\frac{dV}{dy} = \frac{\sqrt{gDS}}{Ky}$

Since $\tau = \tau_o \left[1 - \frac{y}{D}\right] = \rho\varepsilon \frac{dV}{dy}$, where $\tau_o = wDS$

and $\rho = w/g$, then $gDS \left[1 - \frac{y}{D}\right] = \varepsilon \frac{dV}{dy}$

hence, $gDS \left[1 - \frac{y}{D}\right] = \varepsilon \frac{\sqrt{gDS}}{K \cdot y}$

or $\varepsilon = K.y \sqrt{gDS} \left(1 - \frac{y}{D}\right)$, and if

$$z = \frac{y}{D}$$

then $\varepsilon = K \cdot D \sqrt{gDS} \; (1 - z)\, z \; - - - - (3)$

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[18] "Engineering Calculations of Suspended Sediment," by E. W. Lane and A. A. Kalinske, Trans. American Geophysical Union, Part III, August 1941, p. 603.

This equation shows that the diffusion coefficient, ε, is zero at the stream top and bottom and is a maximum at mid-depth. Kalinske and Robertson[17], at the University of Iowa, showed this to be true from measurements in a channel 2.5 feet wide at different slopes, mean velocities, and depths of flow, and for a channel 0.94 foot wide with a larger slope and two different bottom roughnesses.

If the expression for ε in (3) is substituted in the sediment distribution relation in (1), the integration can now be performed and the relative sediment concentration (N/Na) can be determined. From a practical point of view, the results of the integration are not convenient for use. It is simpler to assume that ε is constant throughout the stream depth. It has just been mentioned, however, that ε is not constant; nevertheless, data on sediment distribution in wide rivers, as will be shown below, indicate that such an approximation is valid for most practical purposes. Hence, from the relation of ε obtained in (3), the average value is calculated:

$$\varepsilon_{avg.} = \int_0^D \frac{\varepsilon}{D}\, dy = \frac{D}{15} \sqrt{gDS} \quad \text{- - - - - (4)}$$

Substituting this assumed constant value into the sediment distribution, relation (1) yields:

$$\frac{N}{N_a} = e^{-15t\,(z\,-\,a)} \quad \text{- - - - - - - - - - - - (5)}$$

where $t = \frac{c}{\sqrt{\tau_o/\rho}}$ and for wide streams $t = \frac{c}{\sqrt{gDS}}$.

This relation plots as a straight line on semilogarithmic paper, so that the concentration at any point in the vertical can be found by first plotting a known concentration, N_a, and then drawing a straight line through this point at a slope of -15t (-6.5 t for logarithmic scale to the base 10). This is of considerable practical importance since, from a determination of measurement of sediment concentration at a single point in the vertical, if the sediment composition is known, it is then possible to determine the concentration at all other points in the vertical section by a simple calculation or graphical construction.

From measurements of sediment concentration at various depths in the center of the Mississippi River at Muscatine, Iowa, during the flood of September 1938, Lane and Kalinske[18] plotted a curve, relative depth z versus sediment concentration, for various sizes of material (below 0.005 mm to 0.85 mm) as measured. Following the procedure outlined above, a straight line was drawn through the points with a slope of $-15t$, where $t = c/\sqrt{gDS}$, for which c was the average fall velocity for the size range considered, $D = 33.9$ ft., and $S = 0.000071$, as measured. From these field data and others, it was clear that the approximate expression for sediment distribution, $\frac{N}{N_a} = e^{-15t(z-a)}$, is sufficiently accurate for practical use. In several cases, the line was an excellent average for all the points plotted, as shown by Figure 25.

The velocity distribution was also measured and plotted to show how it checked with the logarithmic velocity distribution curve:

$$\frac{V}{V_m} = 1 + \left(\frac{\sqrt{gDS}}{KV_m}\right)\left(1 + \log_e \frac{y}{D}\right)$$

If, in this equation, we substitute for V_m the Manning formula and let von Karman's universal constant $K = 0.40$, we get:

$$\frac{V}{V_m} = 1 + \frac{1.70\sqrt{g}\,n}{D^{1/6}}\left(1 + \log_e \frac{y}{D}\right)$$

Letting $n = 0.030$, this equation was found to agree very well with the actual velocity distribution measurements as shown on Figure 26. It is pointed out by the authors, however, that the data and analyses are for wide rivers and that the turbulence developed is due to drag on unconsolidated sandbed. Accordingly, this approach cannot apply to streams or open channels where turbulence is developed by roughness or by flow-disturbing items such as rocks, shrubs, trees, etc.

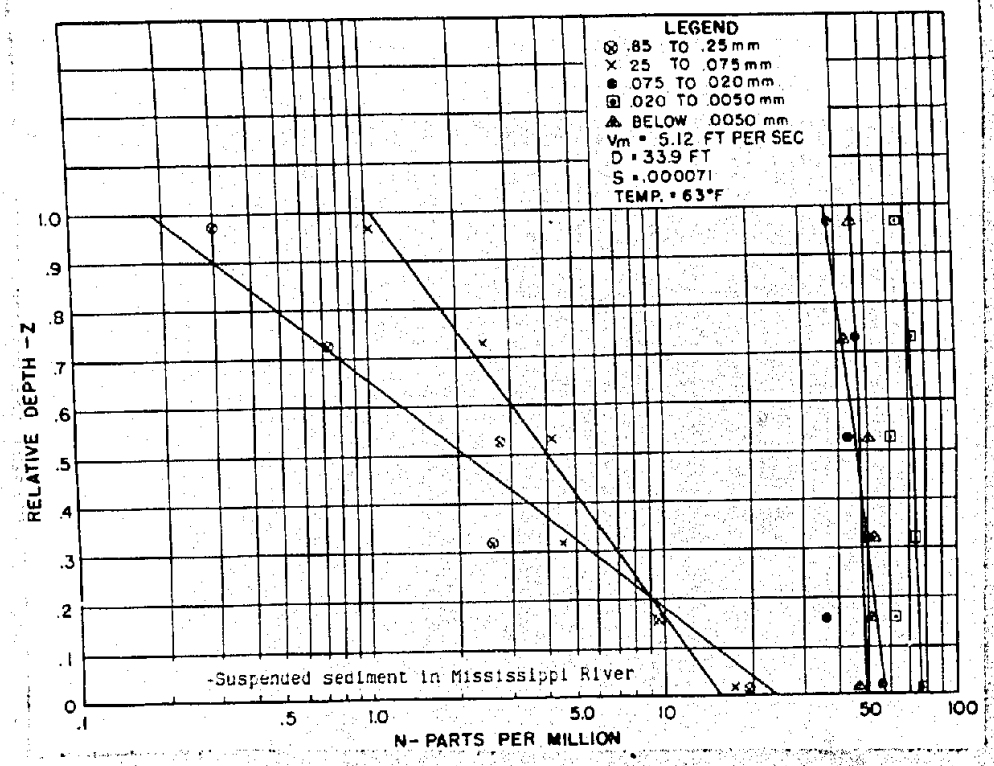

FIGURE 25. SUSPENDED SEDIMENT IN MISSISSIPPI RIVER

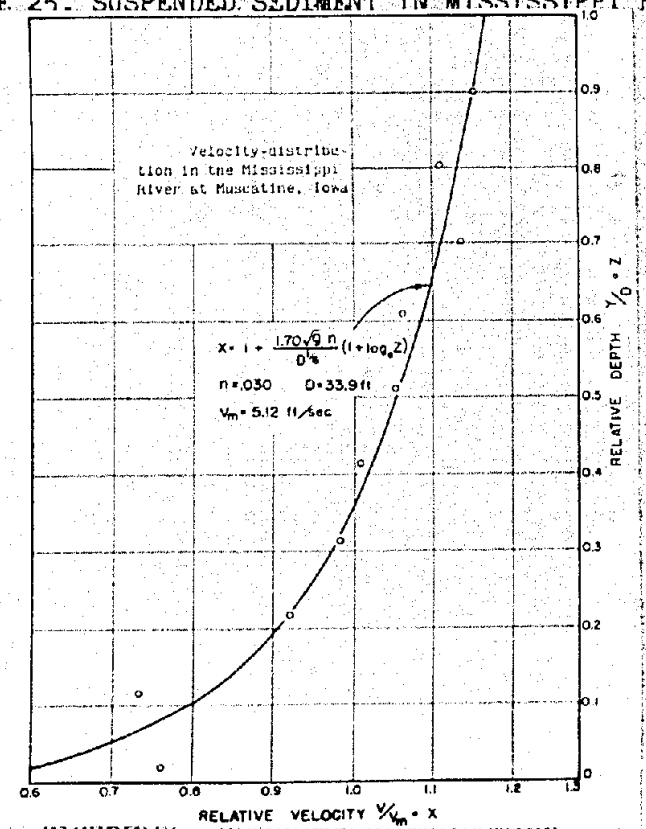

FIGURE 26. VELOCITY DISTRIBUTION

The final result to be desired is the total sediment load carried by a stream per unit of time. If the distribution of sediment in any vertical section is known, then the total load, M, carried per unit of time per unit of stream width is determined by multiplying the concentration per unit volume, N, at each point by the velocity at that point and integrating from top to bottom of the stream. This is accomplished by the authors by combining the sediment-distribution equation (5) with that for the velocity distribution (2):

$$\frac{N}{N_a} = e^{-15t(z-a)}$$

$$\frac{V}{V_m} = 1 + \left(\frac{\sqrt{gDS}}{KV_m}\right) \left[1 + \log_e \frac{y}{D}\right] = x$$

Since the total load $M = D \cdot V \cdot N$, then

$$M = D.V_m \cdot \frac{V}{V_m} \cdot N$$

Therefore, $M = V_m \cdot D \cdot N_a \int_0^1 xe^{-15t\,(z-a)} dz$

or $M = q\, N_a\, e^{15\,ta} \int_0^1 xe^{-15\,tz} dz$

where q = water discharge in second-feet per unit width. Finally,

$$M = q\, N_a\, e^{15ta} \cdot P$$

where $P = \int_0^1 xe^{-15\,tz} dz = \int_0^1 \left(\frac{1 + 1.70\sqrt{g}\,n}{D^{1/6}}\right)(1 + \log_e z)e^{-15\,tz} dz,$

recalling that the use of Manning's formula and K = 0.40 reduces $\frac{\sqrt{gDS}}{K \cdot V_m}$ to $\frac{1.70\sqrt{g}\,n}{D^{1/6}}$. P is therefore a function of the parameter t and the relative roughness $\frac{n}{D^{1/6}}$. To facilitate the use of these expressions, P was plotted against t for several values of relative roughness.

For the aforementioned measurements on the Mississippi River, the value of M, total, was computed by multiplying the concentration at each point by the measured velocity. The actual total of all sizes transported was found to be 1.85 pounds per second per unit width. The value of M was also determined from the expression $M = V_m \cdot D \cdot N_o \cdot P$, where N_o was the concentration of any size range at

the zero level as determined from the curve previously plotted for sediment concentration versus relative depth (Figure 25), and P was obtained from the P versus t curve. The value of M so calculated was 1.89 pounds per second per unit width as compared to 1.85 before, thus indicating the general accuracy of this method of total suspended-load determination as given.

Another item of practical significance is the evaluation of the mean sediment concentration in any vertical section. This mean value would be (M/q), which is:

$$N_{avg.} = N_a \; e^{15\,ta} \; P$$

When a = o, then $\frac{N_{avg.}}{N_o} = P$, so that P is actually the ratio of the average sediment concentration in the vertical to the concentration at the zero level. For very fine sediment, such a ratio approaches unity.

In summarizing the development above, as given by Lane and Kalinske, it was shown that, relative to wide rivers, the total suspended sediment transported can, with fair approximation, be determined by measuring or determining the concentration at some one level, preferably in the lower half-depth, and finding the total rate of sediment carried per unit time per unit width from the expression $q\,N_a . e^{15\,ta}\,P = M$. The parameter $t = c/\sqrt{gDS}$, in which c is the average fall velocity of the size range of sediment considered, g is the acceleration due to gravity, D is the depth of stream, and S is the friction slope. The value of P is obtained from a plotting indicating values of this term for various values of t and the relative roughness $(n/D^{1/6})$.

It is proposed then that theoretically, at least, field measurements be made of the concentration of suspended material at a single level to permit calculation of the total suspended load in the unit width of stream in which the measurement was made. Measurements should also be made at other sections across the stream since the mean velocity in the vertical section and the sediment concentration vary transversely in any river; in addition, as many measurements should be taken as can be economically analyzed and as the accuracy of the problem warrants.

84. Relation of suspended to bed material. In another analysis, Lane and Kalinske demonstrated a proposed method of determining the suspended material in a stream under equilibrium conditions from consideration of the composition of the bottom material and the depth and slope of the stream[19]. The analysis is approached from the considerations that the average rate at which particles of any size are picked up and placed in suspension should be proportional to the relative amount of those particles present in the bottom, to the magnitude of the vertical velocity present (due to turbulence eddies) capable of picking them up, and to the relative amount of time during which velocities capable of picking up particles of that size exist. Expressing these facts mathematically finally reduces to an equation

$$\frac{N_s}{\Delta F(c)} \propto P$$

where N_s is the concentration per unit volume or in parts per million of sediment in suspension at the bottom of an average size characterized by the settling velocity c; $\Delta F(c)$ is the relative amount of particles having an average settling velocity c; and P is a function t_c, where t_c equals

$$\frac{c}{\sqrt{\frac{\tau_o}{\rho}}}$$, and is expressed in a series,

$$P = \left(\frac{1}{2} t_c\right) - \left(\frac{t_c}{2}\right) + \left(\frac{t_c^3}{4}\right) \cdots\cdots \frac{t_c^{2n-3}}{2(n-1)}!$$

A plot of P against t_c reveals that as t_c approaches zero, P approaches infinity, and that P approaches zero rapidly when t_c is greater than unity. This, it is assumed, indicates that sand sizes giving a value of t_c of one or greater would not be found in suspension in any great quantity. This is an important fact, if true, and has been demonstrated entirely by a mathematical analysis of the problem of how sediment is placed in suspension by turbulence. The approach of P to infinity as t_c gets small, indicates that for small sizes of silt no exact relation can be established between the bottom composition and the material of that size in suspension.

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19 "The Relation of Suspended to Bed Material in Rivers," by E. W. Lane and A. A. Kalinske, Trans. American Geophysical Union, 1939, p. 637.

Using field data from several sources, an effort was made to determine the functional relation of the above proportionality. From field data, it was possible to plot $\frac{N_s}{\Delta F(c)}$ against t_c. Since t_c is a function of P, t_c is thus functionally related to $\frac{N_s}{\Delta F(c)}$. The function so obtained from the limited data was in the form:

$$\frac{N_s}{\Delta F(c)} = bP^n$$

The upper limit of one for t_c for material in suspension seems to hold for these data. It was also concluded that for practical use no accurate prediction of the value of N_s for values of t_c less than about 0.020 can be expected. Sediment sizes giving values of t_c less than 0.020 are likely not to be present in the bed of the stream in measurable quantities.

In general, therefore, it seems probable that prediction of suspended-material concentration from a knowledge of the hydraulic characteristics and bottom composition of a river is quite possible within certain limits. The theory on which this analysis is based is fundamentally correct; the approximations which are made simplify the mathematical expressions to such an extent that their use is warranted even though the accuracy under certain conditions may not be great.

This method of determining sediment concentration from bottom measurements can be used in conjunction with the method just previously discussed, wherein only a single sediment measurement is required. From a practical point of view, however, since it was pointed out that the finer material cannot be accurately estimated from the bed-material composition, the usefulness of the two methods in conjunction is limited to the coarser sediments. As an example of using the sediment concentration at the bottom, it was shown in Section 83 that the rate of transportation, $M = V_m \cdot D \cdot N_o \cdot P$, where N_o was the sediment concentration at the zero level, or bottom, equaled 1.89 pounds per second per unit width; while using an integration in the vertical, 1.85 pounds per second per unit width was obtained.

Any rational analysis of suspended-sediment distribution in a stream and of the rate of transportation as presented here is useful also in comparing the relative accuracy of different sampling methods. Even though the expressions given for sediment distribution have definite limitations, the values obtained approach the actual sediment distribution close enough to be used as a basis for comparing the errors obtained in total suspended-sediment transportation as calculated from samples taken in accordance with various methods now in use.

85. Validity of assumptions. The foregoing discussion has been taken directly from the work of Lane and Kalinske as cited. In their derivation of relations pertaining to sediment concentration, assumptions have been made to simplify the analysis; namely, the transfer coefficient for momentum equals the sediment-transfer coefficient, and that the shear is distributed linearly. The most profound assumption made for the sake of convenience was that the mixing coefficient ε is constant throughout the vertical. As the authors explained, however, this was done to simplify the mathematical treatment. As far as the evidence goes, these various assumptions have not caused the theory to be radically in error since experimental and field data convincingly demonstrate the practical use of the functional relationships developed.

There is some difference of opinion, however, among those associated with the research of sediment transportation. In a worthy contribution to the subject, Vanoni[20] has recently arrived at conclusions contrary to some heretofore assumed to be valid. His experiments were made at the Cooperative Laboratory of the Soil Conservation Service and the California Institute of Technology. The experiments were conducted in a flume 33-1/4 inches wide by 12 inches deep by 60 feet long. By using a closed-circuit system, the sediment was circulated directly with the water. Velocities were measured with a 3/16-inch diameter pitot static tube of the Prandtl type. The sediment distribution in the flow

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[20] "Some Experiments on the Transportation of Suspended Load," by Vito A. Vanoni, Trans. American Geophysical Union, Part III, 1941, pp. 608-621.

was determined from samples siphoned from the flow through a 5/16-inch brass pipette, shaped much like a pitot tube. The inlet of the sampler was directed upstream and the rate of siphoning adjusted so that the entrance velocity in the pipette was the same as the velocity of flow at the sampling point, which was previously determined by pitot-tube measurements. The velocity into the sampler was determined from the time required to fill a liter bottle and from the area of the sampler tip. The average sediment load of the flow was determined from samples siphoned from a collecting tank in the circulating system.

A semilogarithmic plot of the velocity profiles at the centerline of the flume with and without sediment loads showed the experimental points to agree very well with the universal logarithmic velocity-defect law (same as (2) in Section 83):

$$\frac{V_{max.} - V}{\sqrt{\frac{\tau_o}{\rho}}} = 5.75 \log_e \left(\frac{y}{D}\right) = \frac{2.3}{K} \log_{10} \left(\frac{y}{D}\right)$$

except near the bottom, where the measured velocities exceed those calculated from this relation. Since this departure was small and occurred over a small portion of the flow, the assumption of logarithmic velocity distribution is essentially accurate when applied to open channels, although derived primarily for pipe flow. Since the slopes of these curves are inversely proportional to K, von Karman's universal constant, the lines should all be parallel if K is a constant (taken to be 0.40 for clear water). These data show, however, that, since the slope of the lines was not the same for all tests, K is not constant for sediment-laden water; in fact, it decreased with the sediment concentration.

In a manner similar to that already described under Lane and Kalinske's work, Vanoni uses the following equation for the relative sediment concentration, as first presented by Rouse[6]:

$$\frac{N}{Na} \left[\left(\frac{d - y}{y}\right) \left(\frac{a}{d - a}\right)\right]^z = H^z$$

in which $z = c/K \sqrt{\tau_o/\rho}$, the other terms being defined previously in Section 83. By plotting the relative concentration $\frac{N}{Na}$ against the

distribution function, H, for a = 0.05 d, the slope of the plotted experimental lines, z_1, would be equal to z, if the theory is correct. Vanoni found, however, that z and z_1 were not always equal, for, in general, z_1 was less than z, indicating that the actual distribution was more uniform than that predicted by theory.

A particularly interesting effect of the suspended sediment in the flow was to reduce the resistance to flow and thereby increase the velocity. For the same discharge (water), the average velocity was found to be larger for sediment-laden flow than for clear flow. Since it was found that the universal constant K in the logarithmic velocity-distribution law was decreased by sediment in the flow, this means that, for a given shear stress, a larger velocity gradient must occur. It follows from this that, in the fundamental relation of shear, $\tau = \rho \varepsilon_m \frac{dV}{dy}$, if the shear is constant while the velocity gradient increases, then the transfer coefficient, ε_m, for momentum must decrease. Vanoni explains that sediment causes this decrease because the stream itself must do work against the settling effect of gravity on the sediment. Since this work or energy must be derived from the vertical components of the turbulent velocity fluctuations, a dampening effect will result to reduce the turbulence somewhat and thereby the mixing coefficient ε_m, since it varies with the turbulence fluctuations. Manning's n was found to decrease 10 percent in some cases for the velocities used, which were about three and five-tenths feet per second.

Since the exponent z_1 obtained from experimental data did not agree with the theoretical value z, except for coarser sediments, this argument was continued to explain why. The author concludes that the intensity of the turbulence fluctuations and the settling velocity w are involved. Accordingly, if the former is greater, sediment is lifted, but, as the sediment grows coarser and w increases, the weaker turbulence can no longer lift sediment so that a reduction in the sediment transfer coefficient, ε_s, occurs. Furthermore, the assumption that ε_s and ε_m are equal is not borne out. This difference is explained by a purely random turbulence which can support sediment,

which, in turn, would increase ε_s above ε_m and by the relative magnitudes of the turbulence fluctuations and settling velocity, as just mentioned, which tends to increase ε_m.

Vanoni plotted curves to show the comparison between ε_s and ε_m for several sizes of sediments as obtained from his experiments. It was revealed that for the coarser sediments the two agreed very closely, while for the finer material ε_s was greater than ε_m.

From these revealing experiments, the following conclusions may be drawn:

a. The assumption that the transfer coefficients for momentum and sediment are equal does not agree with the experimental measurements. This assumption, it will be recalled, is made to simplify the theory of suspended load distribution.

b. The logarithmic velocity-distribution law is valid for open channels, since it is applied to derive the relative concentration of sediment at various levels which agreed closely with the measurements.

c. For fine materials, the coefficient for sediment mixing, ε_s is larger than the coefficient for momentum, ε_m, the opposite being true for coarser sediments, but in between the values are very nearly equal.

d. An addition of sediment to clear water increases the velocity of flow, depending on the size and amount of sediment. This effect, in turn, reduces the resistance coefficient.

e. The addition of sediment to clear water causes ε_m to decrease and the velocity gradient and velocity to increase.

f. The universal constant for momentum transfer, K, is reduced in sediment-laden water.

g. Conclusions d, e, and f are related to the effect of sediment damping the turbulence.

In the light of these experiments and conclusions, Kalinske[21] points out "that the exchange of a vector quantity such as momentum is not identical with the exchange of quantities such as heat and mass under all conditions of turbulent flow. Experiments seem to indicate

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[21] See discussion p. 620, Trans. American Geophysical Union, Part III, 1943

that the exchange of momentum and heat in the uniform turbulence created at boundaries is similar, while the exchange in turbulence formed in the wake of immersed bodies is definitely different. It appears that the exchange of scalar quantities such as heat, water vapor, or sediment ought to be similar." Experiments by Kalinske[17] using immiscible droplets in water having the same specific gravity as water showed that for all practical purposes the exchange of momentum and sediment is equal. Nevertheless, Kalinske does not believe any too definite statements can yet be made in this regard. The principal reason is the difficulty in calculating the momentum exchange coefficient since there is a certain amount of inaccuracy in obtaining the velocity gradient from the velocity distribution curves, especially where the curve is almost vertical, and in estimating the unit shear at any point which must be assumed as to value and distribution. The assumption of linear variation of shear is probably valid, but, in open channels, this may not be strictly true because of side and surface effects as well as effects of secondary currents. Accordingly, the logarithmic velocity-distribution law is not correct near the water surface, nor, as Vanoni showed, near the bottom.

DENSITY CURRENTS

86. Relation to suspended sediment. Associated with suspended sediment transportation in streams is the phenomenon of density currents in reservoirs. These have been defined as "the movement, without loss of identity of mixing at the boundary surfaces, of a stream of fluid under, through, or over a body of fluid, the density of which differs from that of the current, the density difference being a function of the differences in temperature, salt content, and/or silt content of the two bodies of fluid." This problem of sediment deposition in reservoirs is, of course, a serious one due to eventual reduction in storage capacity. Accordingly, any bit of information on this subject should be welcome to the engineering profession. In the past, concern has often been expressed over silting of reservoirs, but little effort was made to attempt a solution. Most certainly, it is a difficult problem to prevent this deposition; for example, in Lake Mead

at Boulder Dam where the Colorado River brings tons of silt to the upstream end each day, how could this be prevented? More logically, what can be done to remove the silt in an economical and effective manner?

87. Silting of reservoirs. A possible answer to the latter question has recently been suggested by Knapp and Bell[22] at the California Institute of Technology. In their analysis of density currents, it is recalled that there is a difference between sediment-laden flow in a river and sediment-laden flow when in a reservoir, for in the former case the water carries the sediment, while in the latter case the water is transported by the sediment. This peculiar circumstance supposedly explains some of the characteristics of density currents. It has usually been assumed that density currents were extremely delicate, occurred only rarely, and that they could easily be dissipated and lose their identity; however, more recent studies reveal that density currents are quite prevalent and that they are not easy to destroy. The lack of knowledge and misconceptions is explained since most underflows occur in large bodies of water and are not visible.

When a silt-laden stream is diluted upon entrance into a reservoir, the muddy water either mixes with the clearer reservoir water or it becomes a density current; usually the latter develops. Once underflow has started, mixing can occur only at the "boundary surface" between the muddy and overlying water. It is believed by Knapp and Bell that this mixing is a slow process so that it will rarely destroy the underflow since the density difference must first be destroyed, but the time of transit is too short for this to occur. Accordingly, there may be only two methods of stopping a density current: (1) By settling out the suspended sediment, thus removing the density difference and "driving power"; and (2) by eliminating the effective slope of the underwater channels, making the density difference ineffective.

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[22] Trans. American Geophysical Union, Part II, 1941, pp. 257-261.

Since density currents cannot be observed directly, still there are some evidences to be seen where underflows occur. From some observations of a model reservoir with glass sides to permit visual study and from field investigations, it has been observed that, as a muddy flow entered the clear water, it at first was retarded with some local mixing occurring. Then the density current started to underflow and carried with it downstream some clear water because of the friction at the interface. An upstream current at the surface resulted in order to replace the clear water moved downstream. When this clear-water upstream current meets the incoming muddy water, both flows turn downward toward the bottom, making a rather definite line of demarcation at the surface. Floating debris is also left stranded at this line by the two currents plunging toward the bottom. These two evidences of underflow, sharp line of demarcation between muddy and clear water and a localized collection of debris, are clearly visible at the upper end of Lake Mead. A sketch illustrating this phenomenon is shown by Figure 27.

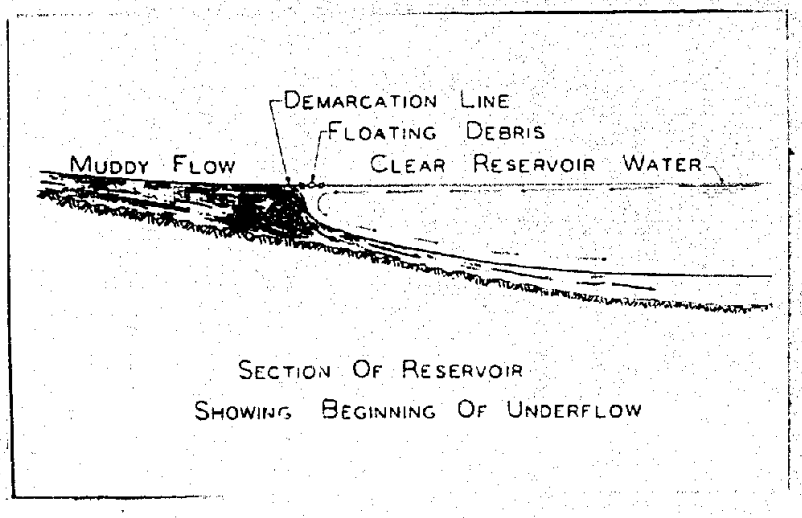
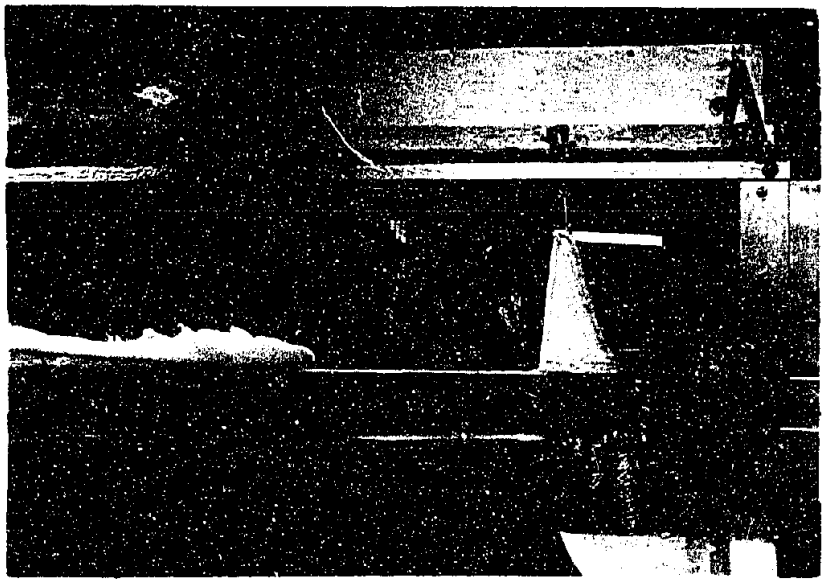


FIGURE 27, BEGINNING OF UNDERFLOW

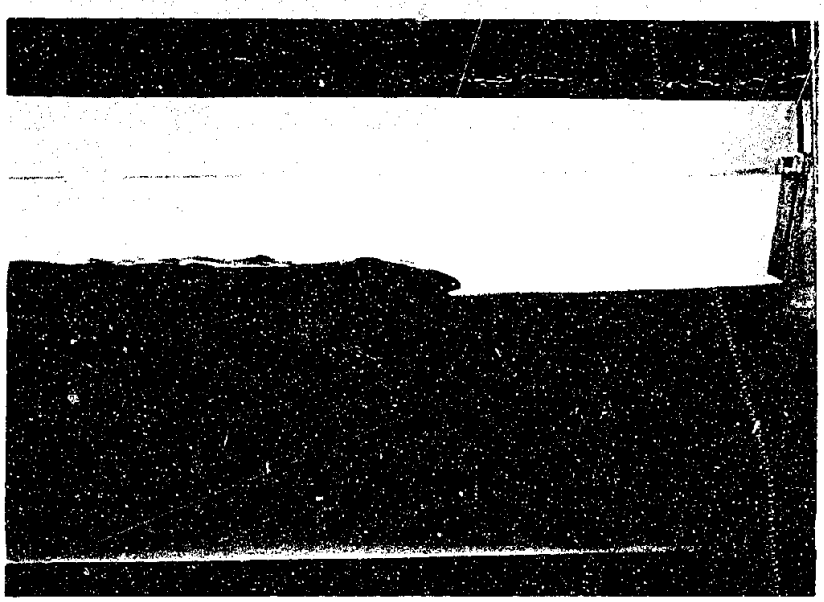
Knapp and Bell have further demonstrated in a laboratory reservoir a possible method of removing silt-laden density currents by a judicious use of outlets placed through a dam. The apparatus consists of a narrow glass-sided flume blocked at one end by a model dam with small outlets extending through it near the base. The bottom of the model reservoir is represented by a sloping floor. By temporarily blocking off a small portion of the upstream end of the reservoir until the density of this water has been changed by adding crushed ice, sugar, salt, or a silt suspension, and by adding dye for observation, it is readily possible to observe the underflow or overflow of separate density currents downstream to the dam (Figure 28), see them flow up the back of the dam, and then return upstream. Only a negligible amount of mixing occurs between the density flow and the clear water. When a density current or a layer of them (composed of lighter underflows superimposed) is stabilized, the outlets to the dam are opened at various elevations to demonstrate the removal of one or more layers of underflow (Figure 29A). If excessive outflow is permitted, it is possible to obtain a mixture of flow from the superimposed layers or of only an upper layer (Figure 29B). Accordingly, it is believed that care must be taken to withdraw the underflows else clear water above will be included with the silt-laden stratum immediately below, or only clear water will be removed.

88. Analysis. At the National Hydraulic Laboratory of the Bureau of Standards in Washington, D. C., other experiments are being made of density currents. Three experimental channels, two of them geometrically similar, of rectangular cross-section have been constructed. Clear tap water is introduced into the experimental channel so as to flow over a pool of aqueous salt solution, the density of which varies from 1.02 to 1.20. The velocity of the clear water is increased until waves which form at the boundary surface begin to break, enabling mixing to occur between the two liquids of different density. This velocity is recorded as the critical velocity. By methods of dimensional analysis, a criterion for this critical velocity was developed in the form:

$$K = \frac{g \cdot V}{U^3} \cdot \frac{\Delta\rho}{\rho}$$

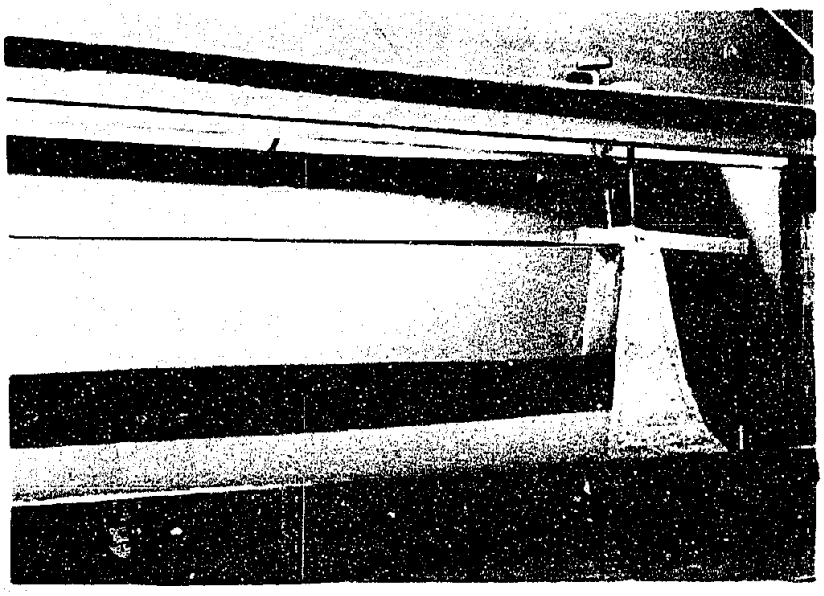


A. DENSITY FLOW APPROACHING DAM

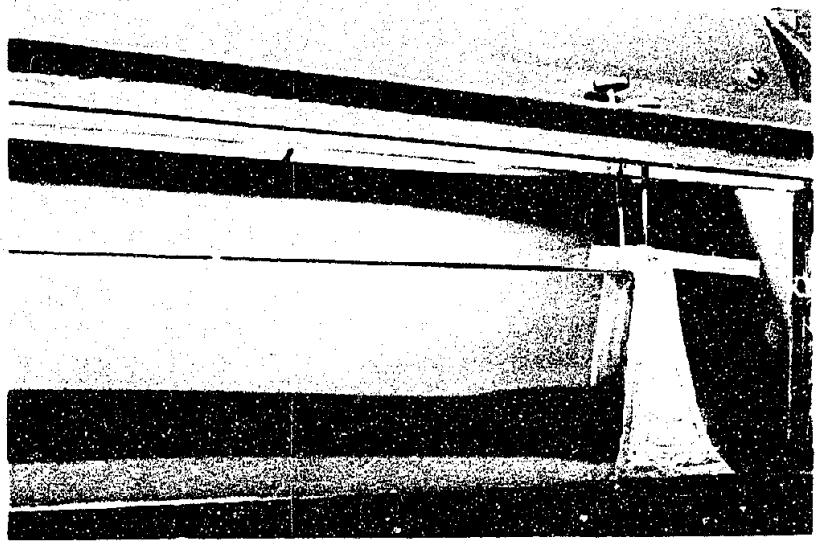


B. LIGHTER FLOW APPROACHING DAM

UNDERFLOW IN MODEL RESERVOIR



A. FLOW FROM EACH LAYER



B. FLOW FROM UPPER LAYER ONLY

SILT DRAWN-OFF BY OUTLETS

where K = a dimensionless coefficient,

ν = kinematic viscosity of the moving liquid,

ρ = density of the moving liquid,

$\Delta\rho$ = difference in density of the two liquids,

U = mean velocity of the moving liquid when mixing first occurs, and

g = acceleration due to gravity.

The values of K determined for different densities of the heavier liquid and with different channels were found to be in good agreement. An average value of $K^{1}/3$ appears to be about 0.155. Scale effects are as yet undetermined either from theory or experiment. It has also been difficult to explain why in geometrically similar channels the rate and development of mixing were rapid in the smaller channel while in the larger channel they were slower, especially for larger values of U. Later experiments (1942) in a larger testing channel have shown that the start of mixing is affected by the nature of disturbances at the entrance to the channel, a greater length of channel is desirable, and the visual method of observing the beginning of mixing is not suitable for the experimental channel. Attempts are to be made in 1942 to correct these undesirable features of the apparatus and the method of experimentation.

89. Field study. To further develop the knowledge of density currents, a committee of the National Research Council has been formed. Since the phenomenon of density currents concerns not only engineers but also geologists, chemists, and physicists, all these professions are represented on the committee. Their program of study includes: (1) Passage of silt-laden water through reservoirs; (2) the failure of water of a tributary to mix with the water of the main stream when the water of the tributary is of a different quality and/or silt content from that of the main stream; (3) the passage of fresh water over salt water, or salt water under fresh water; (4) possible cause of the formation of the submarine canyons that have been discovered recently on the outer edges of the continental shelves by density currents of silt-charged water; and (5) passage of currents of warm air over pockets of cold air.

At the present time the main efforts are being concentrated on the flow of silt-laden currents of water into reservoirs. Through the cooperation of various Government agencies, measurements have been made on Lake Mead and Elephant Butte Reservoir. Such measurements include temperature-depth, velocity, salinity, turbidity, conductivity, and elevation of the top of the silty layer. It is interesting to note that the location of the silty layer is found not only by sampling the water, but also by its temperature since the temperature of the silty layer is higher than that of the clear water immediately above. This fact may also be useful in determining the velocity of a silt-laden density current through a reservoir. In the Elephant Butte Reservoir two thermometers have been installed, one at the outlet of the dam and the other some known distance upstream. By noting the time elapsed between a temperature rise upstream and a similar rise downstream, the mean velocity of the silt layer passing downstream may possibly be determined. The Lake Mead tests have been submitted (1942) to the committee for later publication by the National Research Council.

In regard to the phenomenon of a silt layer in the aforementioned glass-sided model reservoir flowing up the back of the dam and returning upstream, the author recalls during an inspection of the Bartlett Dam on the Verde River in Arizona, that while standing on the parapet of the dam, the water in the reservoir, particularly near the dam, was seen at first to be a vivid blue. Within a few moments, however, a muddy color was easily discerned adjacent to the dam, which gradually extended upstream across much of the reservoir. This phenomenon was evidently a density flow overturning and returning upstream. Proof of a density current was enhanced by the occurrence of abnormal run-off in the watershed and a subsequent load of silt being carried to the reservoir by the Verde River.

In regard to the failure of water of a tributary stream to mix with the flow of the main stream, the author noticed this occurrence below Arrowrock Dam in Idaho. A tributary, Morse Creek, carrying considerable silt flowed into the Boise River which was much clearer

because its flow was released at the dam. Figure 30 shows the lack of mixing between the two streams one-half mile below the confluence. The Boise River is on a steep slope at this point so that the velocity was appreciable with numerous rapids, but still the muddy water was not completely mixed with the main stream until two to three miles downstream from the confluence.

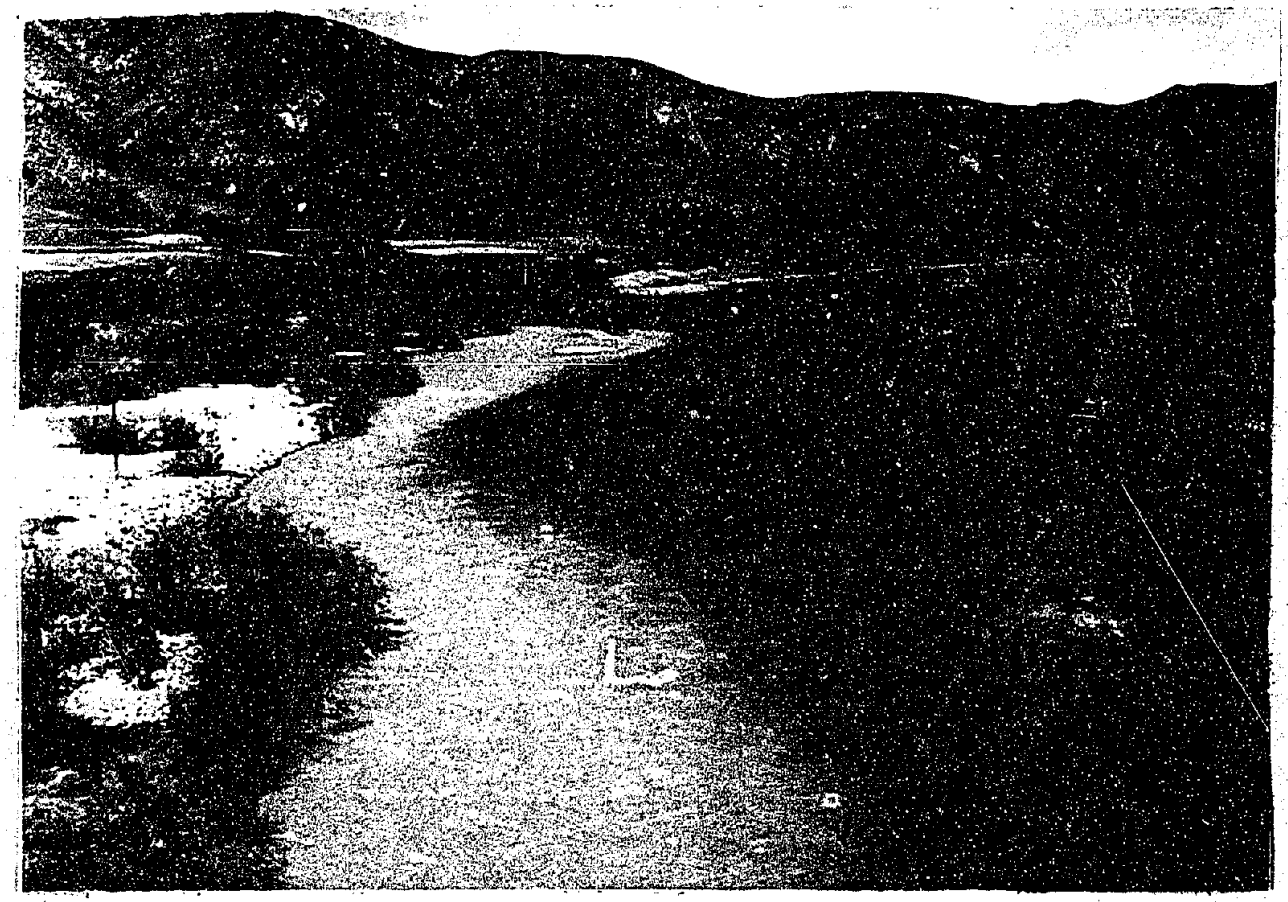


FIGURE 30, INCOMPLETE MIXING BELOW CONFLUENCE

MEANDERING OF STREAMS

90. Type of problem. In Chapter III a description was given of the Mississippi River flood-control model and of the Head of Passes model to illustrate the use of models for solving some particular problem of rivers, in these cases pertaining to the Mississippi River. A research problem associated with any meandering stream,

such as the Mississippi River, is now presented to illustrate a special research problem, fundamental in purpose, in which it is hoped that conclusions can be drawn which will be applicable to meandering streams in general, no attempts being made to derive any complex or far-reaching mathematical laws.

91. Characteristics of meandering streams. Based mostly on observations, some facts have been established relative to meandering streams, but more definite information should be gained from laboratory research to explain the mechanics of some of the phenomena involved. It has been observed that true meandering occurs only in rivers having erodible beds and banks, so that the channel is free to shift its location and adjust its shape as part of a migratory movement of the channel as a whole down a valley; that meanders migrate downstream; that a meandering stream maintains a nearly constant length or overall slope; and that the formation and maintenance of meanders are a function of valley slope, discharge, and character and amount of bed load transported[23]. The effect of cutoffs, bank revetments, and dikes on bed load and meandering, on the other hand, is not clearly understood, and it is particularly desirable to know what the limit is in straightening a meandering stream before the depth and other factors are affected.

92. Laboratory investigations. In view of the lack of more complete knowledge of meandering streams, the United States Waterways Experiment Station has undertaken a study of the problem with the purpose of determining: (1) The fundamental laws controlling the meandering of streams; (2) to check the constant length and overall slope theory; (3) the effect of the amount of bed load on meanders, its direction of movement, and the source of material forming bars; (4) the effect of different flow stages on bank caving and bar formation; and (5) the effect of cutoffs, revetments, and dikes on meandering streams. When this comprehensive study is completed, valuable

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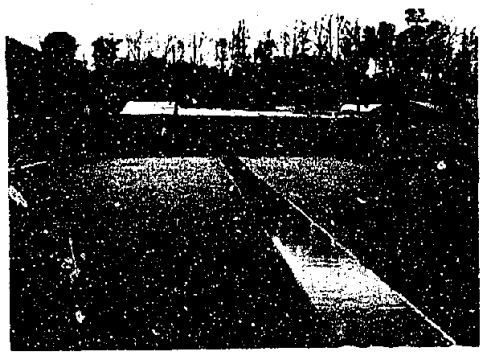
[23] "Basic Aspects of Stream-Meanders," by G. H. Mathes, Trans. American Geophysical Union, 1941, Part III, pp. 632-636.

information should be obtained which would permit a better understanding of the movement of bed load in meandering streams and of the movement of the stream itself, that is, of the characteristics of the river.

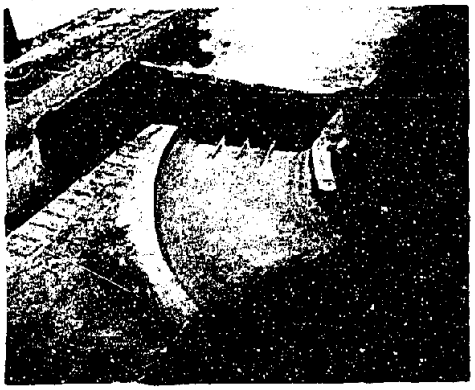
To perform these experiments, a brick flume 125 feet long, 50 feet wide, and 18 inches deep is partly filled with a layer of crushed coal to a depth of about eight inches. A straight channel with a bottom width of 2.20 feet, a depth of 0.20 foot, and slide slopes of 2 to 1 is then molded into the coal bed on a given slope extending the full length of the flume, as shown on Figure 31A. This stream channel has no direct scale relationship with any existing stream. The flow is introduced into the upstream end of the channel in the proper direction to encourage development of meanders quicker than would occur if a straight entrance were used (Figure 31B). The slope of the water surface is set initially parallel to the original stream bed, and, as a test progresses, a tailgate at the downstream end of the flume is adjusted to compensate for any change in water-surface slope due to meander development.

For the first 8 hours of a test, a constant discharge of 0.149 cubic foot per second is maintained, with bed-load material of pulverized coal added at a rate of 0.30 cubic foot per hour (Figure 31C). With an initial meander pattern quickly developed at this discharge (Figure 31D), a varying hydrograph is then run through repeating cycles of 3 hours and 10 minutes until the meanders have stabilized, which may take as long as 50 hours. For the varying hydrograph, a low flow of 0.02 cubic foot per second is run for 1 hour and 20 minutes, no bed load added; then, an intermediate flow of 0.10 cubic foot per second is run for 30 minutes with bed load added at a rate of 0.10 cubic foot per hour; finally, a high flow of 0.18 cubic foot per second, with bed load of 0.30 cubic foot per hour, is run for 30 minutes. A 5-minute period is allowed for adjusting the discharge between each stage, with the cycle being completed by returning to the low flow. These quantities are subject to change, because a definite test program cannot be developed until the bed slope, initial channel size and shape, bed material, and discharge have been adjusted to produce meander

FIGURE 31

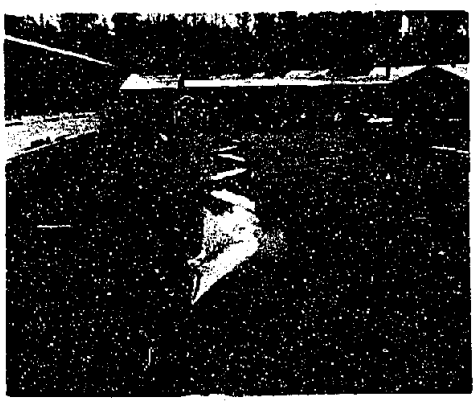


A. CHANNEL IN COAL BED



B. ENTRANCE

C. BED LOAD FEEDER



D. MEANDERS DEVELOPING

MEANDERING STREAMS

characteristics in the model stream similar to those observed in nature. These quantities as described and the method of testing are as of February 1941.

During each test the development of meanders is recorded by mapping and by photographs from a 20-foot tower. Water-surface elevations are measured periodically at a sufficient number of ranges to determine the longitudinal slope in all the reaches and the transverse slope of the bends. To obtain these measurements, and to mold the stream channel, a rolling bridge is used which is supported by rails on each side of the brick flume; the 20-foot photographic tower is also supported on this bridge.

In connection with this investigation, an inspection was made in company with engineers associated with the study of meandering of small streams to observe meandering characteristics in nature. It was noticed in several instances that the laboratory stream displayed features quite similar to those observed in the field. For example, it could be seen that the bed load at the downstream end of a point bar was crowded toward the concave bank almost normal to the direction of flow. This was easily discerned in the model stream and was seen in nature from the axis of the sand ripples in a stream bed and from the curved alinement of gravel streaks across point bars.

From laboratory tests already completed (1941), there are indications that, for any given valley slope, discharge, and bed load, a stabilized condition will occur in the slope, size of meanders, and channel proportions, but migration of meanders continues downstream. It has also been shown that meander width is directly proportioned to the amount of bed load.

ACADEMIC RESEARCH

93. Open channels. A further type of research that is frequently observed in university laboratories pertains to exploring some aspect of flow in pipes or open channels, for example, more with the purpose of establishing relationships between certain flow factors than for exploring some basic problems of flow or solving vital practical

problems. In this sense this research is considered to be academic and somewhat limited in its application to practical problems of primary importance. Nevertheless, this type of work is important and frequently produces new developments and fundamental concepts.

An example of such studies is one observed at the Massachusetts Institute of Technology where extensive tests have been made to determine the variation of the friction factor f with Reynolds number for uniform flow in open channels. In 1938, a series of experiments was started to develop a function between these two variables with constants to be applied for various shapes of channels at various roughnesses. The program of experimentation consisted of a series of related master theses treating various phases of the problem.

The first study by J. A. Downs in 1938, used a wooden channel tested at two widths, 24 and 12 inches, on a fixed slope. The bottom and sides of the channel were roughened with a sand coating. After a few tests, it was found that a relationship did exist in the form $f = aR^x$, but exact nature of the function was not determined.

In 1939 Kerkering and Jacoby continued the work of Downs, using the same channel but testing at a different slope and using three widths, $10\frac{1}{4}$, 6 and 3 inches, They found that a different relationship existed between f and R for each channel width. Conclusions were drawn that this was due either to the fact that an unknown physical quantity was not included in their analysis, or that the hydraulic radius was not the proper linear dimension to use in calculating the friction factor and Reynolds number. The authors tried to find a different linear dimension to use instead of the hydraulic radius, but after trying many different combinations, they concluded that the hydraulic radius was better than anything they had tried.

Also in 1939, Smyser and Harvey tested a channel 12 inches wide by 10 inches deep for various slopes. They attempted to find the effect

of varying the slope and hydraulic radius on the relationship between f and R, but found that neither had any appreciable effect.

In 1940, two more thesis investigations were completed. One study was made by Seedlock and Little using a channel whose width and slope were varied. They reached the following conclusions: (1) Although there exists a functional relationship between f and R (within the range of R investigated of 3,000 to 50,000), this function does not appear to be a straight line on logarithmic paper; (2) that there is probably some width of channel beyond which the function between f and R is unaffected, and this width is greater than 12 inches; (3) the width of channel has a pronounced effect on the relation between f and R when the hydraulic radius is used as the linear dimension in computing f and R; (4) the width of channel does not have a marked effect on the function between Chezy's C and R, and between Manning's n and R; (5) there is some other linear dimension besides the hydraulic radius which would better describe hydraulic size in the determination of f and R; (6) a weighted value of the hydraulic value $\frac{A^2}{P}$, and $\frac{A}{P_{total}}$, is not better suited to describing hydraulic size than the hydraulic radius; and (7) in general, over the ranges studied, there is a tendency for f to increase for a given value of R with increase in either width or slope.

The second study in 1940 was made by Swift and Morrison using the channel for the 1939 tests. In this part of the research, the effect of changing the roughness condition of the channel was investigated. Two roughness conditions were investigated: Rough bottom and smooth sides, and rough bottom and sides. The size of sand used was that passing a No. 8 sieve (2.362 mm) and retained on a No. 14 sieve (1.168 mm). The hydraulic radius was varied from 3 cm to 9 cm and the slope from 0.0001 to 0.0019. Reynolds number was varied from 3,000 to 65,000 with accompanying variation in the friction factor from 0.021 to 0.048. These two experimenters arrived at these conclusions: (1) f is definitely a function of R; (2) the slope does not materially affect

the relationship between f and R for slopes greater than 0.003; (3) the hydraulic radius does not greatly affect the relationship between f and R for R greater than 30,000; (4) the variation of this relationship with hydraulic radius for Reynolds numbers less than 30,000 is especially marked when both the bottom and sides of the channel are rough; and (5) for the channel with only the bottom rough, the effect of the roughness decreases as the depth increases.

This extensive series of tests was continued during 1941, testing a triangular-shaped channel with varnished walls and continuing with a channel of rectangular cross-section with roughened sides and bottom. By 1942, it is planned that the tests will all be completed, whereupon a careful analysis of the entire work will be made to determine just what the functional relationship is between f and R for variation in channel width, slope, and roughness. Possibly another linear dimension than the hydraulic radius will be evolved for computing the friction factor and Reynolds number for open channel flow with particular reference to smaller Reynolds numbers, for which the friction factor varies considerably more for small changes in R.

94. Stream double refraction. Another study observed at the Massachusetts Institute of Technology was of particular interest because of its possibilities in hydraulic research. This study pertains to the examination of flowlines by introducing a dispersion of bentonite into the flow and studying the streamlines by use of polarized light. When polarized light is transmitted through a flow containing a double-refracting colloidal suspension, the lines of flow show up in varicolored patterns wherever there is a variation in velocity or direction of the moving particles. The method is analogous to the determination of strain in transparent models in photoelasticity.

The phenomenon of double refraction occurs when plane-polarized light is passed through a double-refracting crystal of uniform thickness, and then through an analyzer (plane polarizer), whereupon the multicolored pattern is produced, since some of the colors are reinforced while others are weakened by interference. When plane-polarized light passes through a double-refracting or birefringent crystal, the light is split up into two rays, each of which is polarized again, but

having planes of vibration which are normal to each other. This also occurs when plane-polarized light strikes the particles of bentonite in the water, but polarization into two normal planes of vibration occurs in this case by reflection from the surfaces of the particles. When a ray of light is incident upon a transparent surface at an angle called the polarizing angle, part of the light is refracted through the surface and part of it is reflected from the surface. It has been found that there is a partial resolution in these two rays of the vibrations into components, respectively in and at right angles to the plane of incidence. The bentonite crystals are birefringent so that polarization into two planes and interference producing the color effect are due to a lining up of the small particles which are plate-like in shape. Accordingly, it has been postulated that the birefringence is due to polarization by reflection from the small plate-like particles. It is believed more reasonable to assume, however, that the birefringence is due to single transmission through a medium presenting more resistance to vibration in one plane than it does in another, so that, when the components of vibration are resolved into one plane by an analyzer, one of the wave components lags behind the other, causing interference and a variation in color.

The apparatus used to observe the color bands produced in the flow containing bentonite consisted of a small glass channel. Two sets of polarizers were used; one set consisted of a pair of 1-foot square polaroid sheets mounted in glass plates to plane-polarize the light and to serve as a plane-polarizing analyzer; the other set was a pair of circularly shaped combination one-quarter-wave and plane-polarizing plates used for studying the flowlines with circularly polarized light, that is, light in which the two rays are out of phase by one-quarter wave length. The sets of polarizers were placed against the glass channels, one plate on either side, in the line of a light beam from a spotlight or Edgerton light. The observer faced the channel on the opposite side from the light, looking directly into the light source. Various flow conditions were established in the channel

in order to determine the significance of the visual study and to explore potential uses of this method of analysis. The transition from laminar to turbulent flow was studied, flow over a weir, etc. In each case, the bands of color varied as the velocity and direction of flow changed. Kodachrome pictures were taken of the various color patterns for use in future analysis. It is entirely possible that, after more study has been made of this method, a reliable qualitative analysis can be used for the flowline study of various hydraulic phenomena and even a quantitative analysis may be developed, but, as yet, the full significance of the variation of the color bands with velocity and direction of flow is not understood.

It is well to remember that bentonite corrodes metal and that a dispersion to be used depends on the size of channel, since too much will cause the flow to jell. A centrifuge is required to obtain the desired size of particles for testing. A well-prepared 1-percent dispersion of bentonite in water, obtained by fractionating bentonite in a supercentrifuge and selecting fractions with particles below 50 mu, will be particularly clear to the eye, have a viscosity and a surface tension near that of water, and produce pronounced birefringence for low rates of flow.

LABORATORIES

95. Research. Since only the work of a few laboratories can be presented in this chapter, mention is made here of the laboratories which are most prominent in fundamental research together with some of their problems being studied in 1940-1942, excluding the work on model studies and other tests that these laboratories might be making. At the University of Iowa, the main effort is being concentrated on fluid turbulence and its application to suspended-sediment transportation, followed by the conversion of kinetic to potential energy in diverging conduits, the simultaneous flow of air and water in closed conduits, and the entrainment of air in pipes by flowing water (Waldo

Smith Hydraulic Fellowship). At the University of Minnesota work is being done on the sedimentation at the confluence of rivers, high velocity flow in open channels (for Committee on Hydraulic Research of the Society), and a study of fluid turbulence as related to sediment transportation. The University of Illinois has an interesting study concerning the study of turbulent flow through annular tubes. It is expected soon that the University of Columbia and the City College of New York, through the efforts of Dr. Bahkmeteff and Dr. Allen, will publish some new and broader concepts of fluid turbulence mechanism. The National Hydraulic Laboratory of the Bureau of Standards is studying the phenomenon of density currents as described in this chapter, the theory of flood waves, and the laws of similarity as applied to model tests. At the Massachusetts Institute of Technology the work includes the study of the variation of the friction factor with Reynolds number as described above, plus the development of the stream-refraction method of studying flowlines. The California Institute of Technology is concentrating on the mechanics of suspended load transportation and density currents in reservoir in cooperation with the Soil Conservation Service. At the University of California studies are being made of flow through porous media and the mixing of parallel streams. The United States Waterways Experiment Station has work in progress on meandering streams, as discussed above, and on the force of waves on breakwaters.

For further information of fundamental research being conducted in the United States, reference is made to several progress reports in Appendix I, and to the Hydraulic Laboratory Bulletin, Series A, National Bureau of Standards, for 1940, 1941, and 1942.

CHAPTER V - FIELD INVESTIGATIONS

GENERAL

96. Purpose. As mentioned in Chapter I, the field of study for this scholarship included not only investigation of current hydraulic research, but also a study of hydraulic structures in the field. The basis for such a program was derived from the author's work in the hydraulic structures laboratory of the Bureau of Reclamation. It is one thing to be familiar with hydraulic research and the problems associated with hydraulic structures as studied by models in a laboratory, but it is an exception when an opportunity is available to inspect the prototype structures in the field to observe the several hydraulic designs employed and to study their hydraulic performance, particularly when one's normal work does not include such an assignment or responsibility, and when the structures involved are distributed over seventeen Western States. Besides the benefit gained from inspecting a prototype structure, it is possible to acquire a more balanced sense of size and proportion which may readily be distorted somewhat from working with small scale models. Finally, by conversing with the field personnel it is possible to obtain different points of view and suggestions pertaining to hydraulic design and operation of a hydraulic structure.

97. Scope and method of investigations. Table II lists the hydraulic structures on power, flood control, and irrigation projects inspected from October 1940 to July 1941. Because of war restrictions, it was possible in several instances to make only a rather hurried inspection of a structure; nevertheless, when shown about by a competent guide, it was usually possible to obtain the information desired pertaining to the hydraulics of the structure. In the Eastern States, the personnel of the large power projects were reluctant to give much information, regardless of personal identification, particularly about model studies which had been made recently to correct some of the prototype spillways; this handicapped efforts to determine

correlation between model and prototype. In the Western States, the personnel of the various projects were most helpful, making it possible to obtain the desired information. This difference in attitude was due to the fact that war restrictions had not yet been fully applied to several of the isolated structures which were inspected.

While traveling over an irrigation project, for example, the author was accompanied over the area by an engineer from the field office. Usually, the inspection started at the main storage reservoir to study the dam and appurtenant works. Proceeding downstream along the main canal, observations were made of drop structures, wasteways, turnouts, diversion dams, and similar structures in the principal laterals. Since the irrigation season was in progress, it was a good opportunity to observe the behavior of hydraulic jump stilling pools, flow through transitions, outlet works, and several flow measuring devices. In a few cases, the spillways of the main storage dams were operating, particularly along the Salt River in Arizona, and this gave a chance to compare the model and prototype qualitatively, even though the discharges observed were a small percent of the design discharge.

In most of these inspections, the primary interest was centered about flow in spillways and outlet works of all sizes, the energy dissipation of flowing water, and the several methods employed to obtain this dissipation. Accordingly, the following discussion is given to show the type of hydraulic design employed in recent years, some of which have been evolved or improved considerably as a result of model tests, which will be evident from the numerous references to model studies given in Table II in the column headed "Features Investigated and Remarks."

SPILLWAYS

98. Types. The several types of spillways observed included the following: (1) Overfall type, controlled and uncontrolled, forming a part of the dam itself; (2) open channel chutes separate from the dam, controlled and uncontrolled, straight or curved (in plan), or the side-channel type where the direction of the flow is changed and is carried in either a chute or tunnel; (3) glory hole type, controlled, where the flow drops down a vertical shaft to a horizontal tunnel outlet; and (4) inclined tunnels, controlled or uncontrolled, through abutments connecting to a horizontal tunnel, which may have been part of a diversion tunnel during construction. The type used in any case is determined from a study of economic and physical conditions, and the discharge capacity is determined from a study of the particular watersheds involved, factors which are not in the province of this discussion.

99. Overfall. The overfall type of spillway, which is a part of the dam itself, consists of an ogee crest with piers to support regulating gates, which are either drum gates, forming a part of the crest, radial gates, or Stoney gates. Below, and tangent to the ogee section, is the inclined spillway confined between vertical training walls, the spillway finally entering a stilling pool. This stilling pool is used to dissipate the energy of the water and to reduce its velocity, thereby preventing scour to the river channel at the toe of the dam; it has either a horizontal floor, a sloping floor, or is curved upward to form a bucket. In the first two cases, a hydraulic jump is used to dissipate the energy, while a roller is used with the bucket; in some cases, no attempt is made to provide any apron or dissipation, it being assumed that the rock of the river channel will withstand the erosive forces--an assumption seldom made in more recent designs. The rules generally followed in selecting a particular type of stilling pool are given in the discussion under Section 106.

Table II HYDRAULIC STRUCTURES AND PROJECTS

Location	Agency	Purpose	Description	Features Investigated and Remarks
Mississippi River Minneapolis - St. Paul, Minn.	Corps of Engineers U.S.A.	Provide navigation	Twin locks 56 ft. by 400 ft., lift 37.9 ft.; overflow dam 574 ft. long; powerhouse 34 ft. head--18,000 hp. Completed in 1917.	General inspection.
Mississippi River Hastings, Minn.	do.	do.	Lock 110 ft. by 500 ft., lift 12.2 ft.; dam consists of 695 ft. of tainter gate section, 100 ft. of Boule' Dam, and about 3,277 ft. of nonoverflow earth fill dike. Completed in 1930.	Action of filling and emptying system of locks, and stilling pools at tainter gate section.
Mississippi River Red Wing, Minn.	do.	do.	Lock 110 ft. by 600 ft., lift 8.0 ft.; dam consists of 365 ft. of roller gate (80 ft. by 14-1/2 ft.) section and 3,972 ft. of non-overflow earth fill dike. Completed in 1938.	Roller gates and action of stilling pools below gates.
Wisconsin River, Prairie du Sac, Wisconsin.	Wisconsin Power and Light Co.	Power development	Hollow concrete dam 1,000 ft. long by 40 ft. high with flow controlled by 41 tainter gates 20 ft. by 14 ft. Navigation lock 35 ft. by 170 ft. with 34-ft. lift. Powerhouse 8 units of 4,000 hp each. Completed about 1914.	Lowering of tailwater due to scouring below dam and its effect on stilling pool and tailrace. Model studies made at Univ. of Wisconsin several years after completion to eliminate scour.
Wisconsin River, Wisconsin Dells, Wisconsin	do.	do.		General inspection.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Safe Harbor Dam	Susquehanna River Safe Harbor, Pennsylvania	Pennsylvania Water & Power Co.	Power development	Overall length 4,869 ft. West spillway 1,356 ft. long with 24 Stoney gates (35 ft. by 50 ft.); east spillway 454 ft. long with Stoney gates and 4 double leaf gates; maximum spillway discharge 1,100,000 cfs bucket-type stilling pool; nonoverflow section between spillways. Powerhouse 12 units (ultimate), 55 ft. head, 510,000 hp. Completed in 1931.	General inspection. Model studies made of spillway at Worcester Polytechnic Institute, before and after completion of structure.
Holtwood Dam	Susquehanna River Holtwood, Penn.	do.	do.	Overall length 2,350 ft., maximum height 60 ft. uncontrolled weir spillway capacity about 800,000 cfs. Powerhouse capacity 135,000 hp. Completed about 1911.	General inspection of structure and cavitation laboratory. Model studies of spillway made at Worcester Polytechnic Institute.
Conowingo Dam	Susquehanna River Conowingo, Md.	Philadelphia Electric Co.	do.	Overall length 4,648 ft. Spillway section (ogee-type) 2,385 ft. long with 50 Stoney gates (22.5 ft. by 41 ft.) and 3 smaller regulating gates; bucket-type spillway maximum discharge including turbines 880,000 cfs. Powerhouse 11 units (ultimate), 89-ft. head, 594,000 hp. Completed in 1928.	General inspection. Model studies of spillway made at Worcester Polytechnic Institute before and after completion of structure.
Ware River Intake Works	Ware River, Mass.	Boston Metropolitan District Water Supply Comm.	Water supply	Diversion dam 174 feet long. Diversion accomplished by 9 automatic siphon spillways discharging into central sump, which discharges into vertical shaft lined with helical vanes to maintain flow around circumference. Total drop in shaft to aqueduct is 225 feet. Completed in 1931.	General inspection. Model study of shaft made at Worcester Polytechnic Institute.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
aytor Dam	New River, Pulaski County, Virginia	Appalachian Electric Power Co.	Power development	Overall length 1,150 ft. Spillway section about 520 ft. long and 123 ft. high to top of 9 Stoney gates (28.5 ft. by 50 ft.). Stilling pool consists of short horizontal apron with end sill. Maximum discharge 250,000 cfs. Powerhouse 4 units (ultimate), 115-ft. head, 104,000 hp. Completed about 1939.	Inspection of cavitation to end sill on apron at toe of dam as result of flash flood of 200,000 cfs. Model study made at Cornell University before construction, and at U.S. Waterways Exp. Sta. after flood to study cavitation of sill.
orris Dam	Clinch River, Norris, Tenn.	Tennessee Valley Authority	Multipurpose (power, flood control, navigation)	Straight gravity concrete structure 1,570 ft. long by 265 ft. high. Overflow spillway near right abutment controlled by three 14 ft by 100 ft. drum gates, capacity 54,000 cfs. Hydraulic jump stilling pool. Outlet works composed of 8 slide gates 5 ft. 8 in. by 10 ft. discharging through dam into stilling pool, capacity 39,500 cfs. Powerhouse--two units, capacity 132,000 hp. Maximum head 207 ft. Completed in 1936.	General inspection. Model studies made by Bureau of Reclamation, Denver, Colorado.
iwassee Dam	Hiwassee River near Murphy, N. C.	do.	do.	Straight gravity concrete dam 1,292 ft. long by 308 ft. high. Overflow spillway in center controlled by seven 23- by 32-ft. radial gates; hydraulic jump stilling pool. Four 102-in. outlets through dam discharge into pool. Combined spillway and outlet discharge = 150,000 cfs. Powerhouse two units 60,000 kw each. Completed in 1940.	General inspection. Model studies made by TVA.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Chickamauga Dam	Tennessee River, Chattanooga, Tennessee	Tenn. Valley Authority	Multipurpose (power, flood control navigation)	Overall length 5,794 ft., maximum height 129 ft. Spillway section concrete gravity type with 18 bays; gates 40 ft. by 40.44 ft. in two sections, maximum capacity 600,000 cfs. Powerhouse with 4 units, 36,000 hp each, 36-ft. head. Navigation lock 60 ft. by 360 ft. Maximum lift 58 ft. Completed in 1939.	General inspection. Model studies made by TVA.
Hales Bar Dam	Tennessee River, near Chattanooga, Tennessee	do.	do.	Concrete overflow weir uncontrolled 1,200 ft. long by 65 ft. high. Navigation lock right abutment 60 ft. by 312 ft. with 40-ft. lift. Powerhouse 14 units. Completed in 1913.	General inspection.
Guntersville Dam	Tennessee River, near Guntersville, Alabama	do.	do.	Overall length 3,979 ft. Maximum height 94 ft. Concrete gravity spillway section 856 ft. long--18 bays with gates 40 ft. wide by 20 ft. high in two sections. Crest 20 ft. above rock, piers extend 60 ft. above crest. Maximum capacity 625,000 cfs. Powerhouse contains 4 units of 136,000 hp total output. Navigation lock 60 ft. by 360 ft. with maximum lift of 45.44 ft. Completed in 1940.	General inspection. Model studies made by TVA.
Wheeler Dam	Tennessee River, near Rogersville, Alabama.	do.	do.	Overall length 6,502 ft. Spillway section 2,700 ft. long by 50 ft. high with 60 openings controlled by 40 ft. by 15 ft. radial gates; stilling pool divided into 12 sections by training walls, maximum capacity 687,000 cfs. Navigation lock 60 ft. by 360 ft. maximum lift 53 ft. Powerhouse with 8 units of 45,000 hp each at 48-ft. head. Completed in 1937.	General inspection. Model studies made by Bureau of Reclamation, Denver, Colorado.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Wilson Dam	Tennessee River, near Florence, Alabama	Tenn. Valley Authority	Multi-purpose (power, flood control, navigation	Overall length, 3,098 ft. Gravity spillway section 2,668 ft. long with 58 38-ft. by 18-ft. gates, discharge per bay 11,800 cfs. Powerhouse 18 units ultimate developing 500,000 hp. Nav. lock 60 ft. by 350 ft. Completed in 1918.	General inspection.
Pickwick Landing Dam	Tennessee River, near Savannah, Tennessee	do.	do.	Overall length 7,715 ft. long, maximum height 113 ft. Concrete gravity spillway section 1,141 ft. long by 47 ft. high, 22 bays with gates 40 ft. by 40 ft. in two sections, maximum capacity 460,000 cfs. Concrete apron stilling pool 73 ft. long. Navigation lock 110 ft. by 600 ft. maximum lift of 63 ft. Powerhouse with 6 units of 48,000 hp each, 43-ft. head. Completed in 1938.	General inspection. Model studies made by TVA.
Roosevelt Dam	Salt River, north east of Phoenix, Arizona.	Salt River Valley Water Users' Assn.	Irrigation and power development	Masonry arch-gravity type, 723 ft. long by 280 ft. high; radial gate spillways, unlined, at each abutment, total length 402 ft. discharge down canyon walls, capacity 150,000 cfs; two 38-in. needle valves and two 54-in. butterfly valves for releasing irrigation demands. Powerhouse capacity 24,000 hp under maximum head of 222 ft. Completed in 1911, additions and revisions in 1923 and 1937.	Spillways of all Salt River structures were operating, some for first time in 21 years. Inspection made of flow in spillways, at outlet valves, and in tailrace. Total discharge passing structure was about 16,000 cfs during inspection. Model studies made by Bureau of Reclamation in 1935.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Horse Mesa Dam	do.	do.	do.	Concrete arch-gravity type, 600 ft. long by 300 ft. high; radial gate spillways at each abutment, total length 264 ft. discharge down canyon walls, capacity 100,000 cfs spillway tunnel right abutment, capacity 47,000 cfs. Powerhouse capacity 43,000 hp. Under maximum head of 234 ft. Completed in 1927, spillway revisions made in 1937.	Made observations of flow in spillway tunnel amounting to 8,600 cfs, other spillways not operating. Model studies made by Bureau of Reclamation in 1936.
Mormon Flat Dam	Salt River, northeast of Phoenix, Arizona.	Salt River Valley Water Users' Assoc.	Irrigation and power development	Concrete arch-gravity type, 380 ft. long by 224 ft. high; spillway left abutment superelevated curved channel controlled by two 50 ft. by 50 ft. Stoney gates, capacity 150,000 cfs. Power generated 10,000 hp under 132-ft. head. Completed in 1925, spillway revised 1936-38.	Spillway passing 8,800 cfs under right gate open 5 ft. Studied flow conditions. Model studies made by Bureau of Reclamation in 1936.
Stewart Mountain Dam	do.	do.	do.	Concrete arch-gravity type, 1,260 ft. long by 207 ft. high. Spillway left abutment, superelevated channel controlled by 9 radial gates, capacity 150,000 cfs. Irrigation releases made by two 42-in. needle valves and one 84-in. butterfly valve. Powerhouse capacity 17,500 hp under 116-ft. head. Structure completed in 1930, spillway revised in 1937.	Spillway discharging 4,500 cfs--one gate open; needle valves 100% open passing 650 cfs each; butterfly valve 100% open discharging 2,100 cfs. Studied jet action of valves and conditions in river produced by these flows. Model studies made by Bureau of Reclamation in 1936.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Granite Reef Dam	do.	do.	Diversion dam	Uncontrolled overflow weir 1,000 ft. long by 29 ft. high; canal headworks and sluice each abutment. Completed in 1908.	Discharge 19,500 cfs over weir. Noted action of flow over weir and in river below, and diversion into canals. Followed main canals and laterals noting flow conditions at weirs, turnouts, checks, and drops.
Bartlett Dam	Verde River, northeast of Phoenix, Arizona	do.	Irrigation	Concrete multiple-arch type, 950 ft. long by 273 ft. high (highest of this type in the world); spillway right abutment, super-elevated channel controlled by three 50-ft. by 50-ft. Stoney gates, capacity 225,000 cfs. Irrigation releases made by three slide gates, 6 ft. by 7 ft. 6 in., in base of an arch barrel, and by two 66-in. needle valves. Completed in 1939.	Discharge in spillway 12,000 cfs. Needle valves open 30% passing total flow of 1,350 cfs. Studied spillway action and valve jets. Noticed silt-laden density current in reservoir reverse itself. Found incorrect use being made of rating curve for needle valves—explained proper use. Model studies made by Bureau of Reclamation in 1936.
Boulder Dam	Colorado River, near Boulder City, Nevada.	Bureau of Recl.	Irrigation, flood control,	Concrete arch-gravity type, 726 ft. high by 1,282 ft. long (highest dam in the world); side channel spillways each abutment, capacity	Inspection of all features and equipment. Model studies made by

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Boulder Dam (Cont.)		Boulder Canyon Project	power development	400,000 cfs. Outlet works consist of six 84-in. needle valves on each side of the canyon, total capacity 46,000 cfs.; and six 72-in. needle valves at tunnel plugs each side of canyon, total capacity 44,000 cfs. Ultimate powerhouse capacity 1,835,000 hp. Completed in 1936.	Bureau of Reclamation 1933-35.
Parker Dam	Colorado River, near Earp, Calif.	Bureau of Reclamation. Parker	Irrigation, flood control, water	Curved concrete arch 320 ft. high by 800 ft. long, only 85 ft. rise above stream bed, sometimes being called the world's "deepest" dam. Overflow spillway along crest consists of five panels each controlled by 50 ft. by 50 ft. Stoney gates. Dam completed in 1938. Powerhouse to be completed in 1942, containing 4 units, total capacity 100,000 hp.	Three of five gates passing 9,000 cfs. Made careful study of pitting of concrete due to cavitation on pier walls just downstream from gate slots. Model study made by author Jan. 1942 to correct this condition and to evolve new design of gate slot for 50 ft. by 50 ft. Stoney gates. Inspected powerhouse under construction, and pumping plant and storage reservoir of Colorado River Aqueduct. Model studies made by Bureau of Reclamation in 1938.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Imperial Dam and Desilting Works	Colorado River, near Yuma, Ariz.	Bureau of Reclamation Boulder Canyon Project-All-American Canal System	Diversion dam for All-American Canal System	Slab-and-buttress type with total length of 3,430 feet including nonoverflow sections, headworks, gate structures, sluiceway, and overflow spillway. The maximum height is 45 ft. raising the normal water surface 23 ft. At right abutment is headworks to All-American Canal consisting of four 75- by 22-ft. roller gates, capacity 15,155 cfs. Adjacent to the headworks is a sluiceway consisting of twelve 16- by 7-ft. radial gates, maximum capacity 42,500 cfs. The overflow spillway, 1,197.5 ft. long by 31 ft. high, is uncontrolled and passes a maximum flow of 150,000 cfs. At the left abutment is located the headworks and desilting basin of the Gila Canal. Below the All-American Canal headworks the flow passes through an extensive desilting basin, consisting of three basins 500 ft. by 800 ft. in plan and 12.5 ft. deep, with an influent channel through the center of each and effluent channels between the basins leading to the All-American Canal. The silt settling in each basin is removed, by rotating scrapers which bring the silt to central collecting pipes discharging into the river.	A comprehensive inspection was made of this structure and of the old Laguna Dam just downstream, as well as the entire All-American Canal System from the vicinity of Yuma, Ariz. to as far west as El Centro, Calif. in the Imperial Valley, and north past the Salton Sea to Mecca following the Coachella Canal, a branch of the All-American Canal. Flow conditions at many canal structures were carefully observed, several of which had been tested by models, including the Imperial Dam, by the Bureau of Reclamation since 1936.
Prado, Brea,	Los Angeles and	Corps of	Flood Control	In general, all are earthfill dams of considerable length with [illegible] regulated	General inspection to become familiar with

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Prado, Brea, Fullerton, Hansen, Supulveda, Santa Fe, and Whittier Narrows Dams (Cont.)	vicinity	Corps of Engineers	Flood Control	by gates. Some are still under construction (1942). No detailed information available.	flood control problems and collection of debris in these and other basins to protect densely populated areas from floods. Prototype measurements of outlet works made at Prado and Hansen Dams. Model tests made by U.S.Engineer Office, Los Angeles, Calif.
San Gabriel Dam No. 1	San Gabriel River near Los Angeles,	Los Angeles County Flood Control District	Flood Control	Rockfill type, 1,470 ft. long by 376 ft. high. Spillway beyond right abutment, 456 ft. wide at crest (uncontrolled) converging downstream; capacity 290,000 cfs. Outlet works consist principally of two 129- by 117-in. needle valves and one 51- by 39-in. needle valve. Structure completed in 1938. See Procs. A.S.C.E., Sept. 1941, p. 1199, "Design and Construction of San Gabriel Dam No. 1," by P. Baumann.	Study made of outlet works and field tests for determining losses and friction factors of outlet conduits. One 129- by 117-in. needle valve discharging 720 cfs at 8-1/2% open under 230-ft. head during inspection. Model studies made by Los Angeles Co. Flood Control District. Morris Dam just downstream observed from distance being unable to gain permission for closer study.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Friant Dam	San Joaquin River near Fresno, California.	Bureau of Reclamation Central Valley Project	Irrigation	Straight gravity-section concrete dam 300 ft. high by 3,430 ft. long. Overflow spillway in center of dam controlled by three 100- by 18-ft. drum gates, capacity 90,000 cfs. River outlet works adjacent to spillway consist of two 110- by 102-in. tube valves and two 110- by 105-in. needle valves and two 18-in. needle valves, capacity 17,000 cfs discharging into separate stilling pool. Friant-Kern Canal outlet consists of same valves as river outlet, except no 18-in. valves, discharging at higher elevation into stilling pool, capacity 3,500 cfs. Madera Canal outlet consists of two 91- by 86-in. needle valves discharging 1,500 cfs into stilling pool at higher elevation. Structure to be completed in 1942.	Dam about 40% completed during inspection. Model studies made by Bureau of Reclamation
Contra Costa Canal	Vicinity of Antioch, Calif.	Bureau of Reclamation Central Valley Project	Irrigation, water supply.	Canal takes out of San Joaquin River and by means of four pumping plants supplies water at higher elevation for distance of 46 miles, capacity 350 cfs.	Followed canal up to limit of construction observing appurtenant structures and construction of new section.
Stoney Gorge Dam	Stoney Creek, near Orland, California.	Bureau of Reclamation Orland Project	Irrigation	Ambursen type 125 ft. high by 868 ft. long. Overflow spillway in center of dam controlled by three 30- by 30-ft. caterpillar gates of the overflow type, capacity 30,000 cfs. Outlet works of two 42-in. needle valves, capacity 1,050 cfs. Completed in 1928.	General inspection of dam and surrounding irrigation district noting flow conditions at many types of irrigation structures.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Shasta Dam	Sacramento River, near Redding, California.	Bureau of Reclamation Project	Irrigation, flood control, power development, navigation.	Curved gravity-type concrete dam 560 ft. high by 3,500 ft. long, the second largest concrete dam in the world. Overflow spillway (highest of this type) at center of dam controlled by three 110- by 28-ft. drum gates capacity 185,000 cfs. Outlet works through dam discharge down spillway from three tiers 100 ft. apart, four in lower tier, eight in middle tier, and six in upper tier, each outlet 102 in. in diameter, total capacity 65,000 cfs. Powerhouse 522,000 hp under maximum head 475 ft. Structure to be completed in 1943.	General inspection of construction which was about 30% completed. Viewed work from top of 460-ft. head tower. Model tests made by Bureau of Reclamation.
Klamath Project	Vicinity of Klamath Falls, Oregon.	Bureau of Reclamation Klamath Project	Irrigation	Irrigation project of 150,000 acres situated in an ancient lake bed. Water derived from lakes in vicinity by damming small rivers. Some land under water used as wild bird reserve. Drainage from irrigated areas collects in low areas and is pumped through hogback to dry lands now being developed.	Studied hydraulic performance of many irrigation structures.
Warm Springs Dam	Malheur River, near Juntura, Oregon.	Bureau of Reclamation Vale Project	Irrigation	Concrete arch-type 549 ft. long by 104 ft. high; overflow crest with flashboard control. Outlet works consist of two 3.25-ft. by 6-ft. outlets controlled by slide gates, capacity 2,000 cfs. Completed in 1919.	General inspection.
Agency Valley Dam	North Fork Malheur River, near Juntura, Oregon	do.	do.	Earthfill-rock-face dam 103 ft. high by 1,900 ft. long. Spillway right abutment controlled by three 18- by 17-ft. radial	General inspection. Observed vibration of radial gate as small

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Agency Valley Dam (Cont.)			Irrigation	gates, capacity 12,000 cfs. Spillway is concrete-lined chute trapezoidal in section with stilling pool at end. Outlet works consist of two 36-in. needle valves discharging into pool, capacity 600 cfs. Completed in 1935.	flow spilled over top of gates. Model studies made by Bureau of Reclamation.
Owyhee Dam	Owyhee River, near Nyssa, Oregon.	Bureau of Reclamation Owyhee Project	do.	Concrete arch-gravity type 417 ft. high by 600 ft. long. Morning glory type spillway near right abutment consists of vertical shaft 309 ft. long and 22.6 ft. in diameter connecting by elbow to horizontal tunnel. Crest is 52.33 ft. in diameter at lip, controlled by a 60- by 12-ft. ring gate, capacity 40,000 cfs. Outlet works consist principally of three 48-in. needle valves, total capacity 2,800 cfs. Release of water to project is made through a tunnel upstream from dam. Completed in 1932.	Inspection of dam and irrigation structures on project such as drops, wasteways, siphons, distribution works, etc. Model studies made by Bureau of Reclamation.
Arrowrock Dam	Boise River, near Boise, Idaho.	Bureau of Reclamation Boise Project	do.	Concrete-curved, gravity-type 354 ft. high by 1,100 ft. long. Spillway at right abutment is side-channel type controlled by six 62- by 6-ft. drum gates, capacity 40,000 cfs. There are 25 outlets passing through the dam in three tiers. Lower tier consists of 5 outlets, 60 in. in diameter by 164 ft. long, controlled by 60- by 60-in. slide gates 28.5 ft. from upstream face of dam. The middle tier consists of 10 outlets, three 72-in. for future power, and seven 52-in., each 129 ft.	Observed spillway passing 5,400 cfs. and one upper outlet discharging 1,100 cfs. Spillway discharges down wall at canyon producing spray sufficient to damage road on opposite side of canyon.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Arrowrock Dam (Cont.)				long, controlled by 58-in. balanced valves. The top tier consists of 10 outlets 70 ft. long similar to the 52-in. outlets in the middle tier. Total capacity of outlets is 26,700 cfs. The outlets are horizontal and discharge into the river directly. Structure completed in 1915. Spillway and face of dam repaired 1936-37.	
Black Canyon Dam	Payette River, near Emmett, Idaho.	Bureau of Reclamation Boise Project	Irrigation, power development	Concrete gravity dam, slightly curved, 1,100 ft. long by 160 ft. high. Overfall spillway section near left abutment consisting of three 64- by 14.5-ft. drum gates, maximum capacity 40,000 cfs. Cushion pool at toe of dam. Canal headworks left abutment, capacity 1,000 cfs. Powerhouse and pumping plant supplying canal at right abutment, 266 cfs under 28-ft. head. Completed in 1925.	Spillway flow of 9,000 cfs over center gate and 1,000 cfs over left gate.
Unity Dam	Burnt River, southwest of Baker, Oregon	Burnt River Irrig. Dist. and Bureau of Recl.	Irrigation	Earthfill dam 83 ft. high by 700 ft. long. Spillway right abutment drops 90 ft. to stilling pool, controlled by two 24- by 16-ft. radial gates, capacity 10,000 cfs. Water release for irrigation made by 2.75- by 2.75-ft. slide gates through tunnel, capacity 680 cfs.	General inspection of spillway and outlet works. Model studies made by Bureau of Reclamation.
Bonneville Dam	Columbia River, Bonneville, Oregon	Corps of Engineers U.S.A.	Power development	Spillway section concrete gravity structure 1,250 ft. long by 170 ft. high. Contains twelve 50'x50' and six 60'x50' Stoney gates, maximum capacity 1,600,000 cfs. Powerhouse capacity 10 units each 67,000 hp	Study of stilling pool at spillway section for discharge of 200,000 cfs. General inspection of other features.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Bonneville Dam (Cont.)				at 55-ft. head; 10,500 cfs each unit. Navigation lock 76 ft. by 500 ft. with 66-ft. lift. Fish ladders 40 ft. wide, each pool 16 ft. long with 12-in. drop, on Washington shore, Bradford Island and at lock.	Visit to hydraulics laboratory on reservation to observe model studies. Model tests of Bonneville made by Corps of Engineers.
Tieton Dam	Tieton River, west of Yakima, Washington	Bureau of Reclamation Yakima Project	Irrigation	Earth-rockfill dam 321 ft. high by 905 ft. long. Side-channel spillway left abutment controlled by six 85- by 8-ft. drum gates, maximum capacity 50,000 cfs. Outlet works consist of two 60-in. and one 24-in. balanced needle valves, capacity 2,400 cfs. Completed in 1925.	Inspection of scour to rock below end of spillway and performance of needle valves.
Cle Elum Dam	Cle Elum River, near Cle Elum, Washington.	do.	do.	Earth-rockfill dam 135 ft. high by 750 ft. long. Spillway at right abutment open-channel chute controlled by five 37- by 17-ft. radial gates, capacity 40,000 cfs, hydraulic jump stilling pool. Outlet works consist of vertical cylinder gate in vertical shaft, capacity 5,000 second-feet. Completed in 1933.	General inspection. Model studies made by Bureau of Reclamation.
Kachess Dam	Kachess River, near Easton, Washington.	Bur. of Recl. Yakima Project	Irrigation	Earthfill dam 63 ft. high by 1,400 ft. long. Spillway right abutment concrete chute controlled by one 50- by 8-ft. radial gate, capacity 4,000 cfs. Hydraulic jump stilling pool. Completed in 1912.	General inspection.

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Easton Dam	Yakima River, near Easton, Washington	Bur. of Reclamation Yakima Project	Irrigation	Straight-gravity, overflow, concrete-diversion dam 248 ft. long by 66 ft. high. Flow controlled by drum gate in crest, 64- by 14.5-ft. high, capacity 13,000 cfs. Diversion is made into Kittitas Canal at right abutment into intake structure controlled by two 12- by 11-ft. radial gates, capacity 1,320 cfs. Fish ladder at left abutment has 20 bays 10 ft. long by 6 ft. wide and risers of 2 ft. 1 in. Each bay is separated by weirs 3 ft. 6 in. high. Completed in 1929.	Study of fish ladder.
Roza Diversion Dam	Yakima River, near Yakima, Washington	do.	do.	Concrete-gravity dam 241-ft. long by 35 ft. high surmounted by two 110 ft. by 14 ft. roller gates, discharge capacity 50,000 cfs. Stilling pool--short apron with baffles. Headworks to Yakima Ridge Canal right abutment consist of six openings containing 20 ft. by 13 ft. 1 in. rotating fish screens, flow into canal regulated by 28 ft. by 15 ft. radial gate, capacity 2,200 cfs. Fish ladder left abutment consists of 28 pools 6 ft. wide by 8 ft. long on 8:1 slope passing 86 cfs. Portal to tunnel through dam in right pier abutment permits fish to enter and reach ladder on other side.	Careful examination of fish ladder and flow into headworks. Model studies made by Bureau of Reclamation.
Yakima Project	South Central Washington	Bur. of Reclamation	Irrigation	Irrigation project of 300,000 acres with a multitude of small canal and lateral structures, some of which were tested by models	Spent considerable time studying check drops in Sunnyside

Table II (Continued)

Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Yakima Project (Cont.)				in laboratory by author.	main canal and several other types of structures such as wasteways, siphons, turnouts, and larger dams described above.
Grand Coulee Dam	Columbia River	Bur. of Reclamation Columbia Basin	Irrigation, power, flood control	Straight-gravity-type concrete dam, 4,500 ft. long by 550 ft. high (largest in volume, 12,000,000 yds. of concrete, and second highest dam in world). Overflow spillway section in center of dam 1,650 ft. long controlled by eleven 135 ft. by 28 ft. drum gates at the crest, capacity 1,000,000 cfs. Bucket-type stilling pool at toe of dam. Outlet works consist of 60 outlets, 110 in. in diameter, in three tiers 100 ft. apart (vertically), 20 outlets in each tier discharging down face of spillway, capacity 250,000 cfs. Flow in outlets controlled by ring-follower gates. Powerhouse contains 9 turbines (ultimate) of 150,000 hp each--maximum head 360 ft., one powerhouse each abutment, total capacity 2,700,000 hp. Completed in 1942.	Dam was 98% complete at time of visit. Made complete inspection of entire project, including powerhouse, outlets, drum gates, and bucket action with outlets discharging 145,000 cfs. See correlation between model and prototype in paper "Model-Prototype Comparisons of Hydraulic Structures," by J. E. Warnock and H. G. Dewey, Jr., to be published in a symposium in 1942 Proceedings, A.S.C.E.
Gibson Dam	North Fork of Sun River, west of Fairfield, Mont.	Bur. of Reclamation	Irrigation	Concrete arch dam 195 ft. high by 960 ft. long. Morning-glory spillway controlled by six 34- by 12-ft. radial gates, concave	General inspection. Model studies made by Bureau of Reclamation

Table II (Continued)

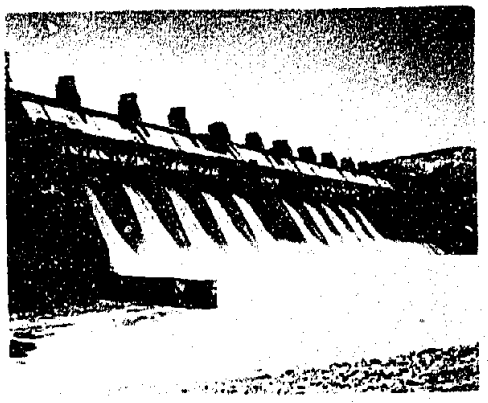
Name	Location	Agency	Purpose	Description	Features Investigated and Remarks
Gibson Dam (Cont.)		Sun River Project		face toward reservoir; shaft converges from 100-ft. to 29.5 ft. diameter, total drop 204 ft., capacity 50,000 cfs. Outlet works consist of two 66-in. needle valves, capacity 2,750 cfs. Completed in 1930.	Inspected irrigation project of 110,000 acres to observe chute drops in operation which had been revised by model tests at Bureau of Reclamation.
Shoshone Dam	Shoshone River, west of Cody, Wyoming	Bur. of Reclamation Shoshone Project	Irrigation, power	Rubble concrete arch 328 ft. high by 200 ft. long. Uncontrolled side-channel spillway 300 ft. long at left abutment leading to 20-ft. diameter unlined tunnel, through abutment, capacity 30,000 cfs. Outlet works consist of 10-ft. tunnel leading to cylinder gate controlling flow into scrollcase which permits flow to rise into rectangular chamber with horizontal perforated concrete baffles. Flow then enters tunnel 50 ft. above gate. Capacity of outlet works 1,200 cfs under 150-ft. head. Dam completed in 1910; outlet works in 1940.	General inspection of dam and outlet works. Traveled over irrigation project to observe recently completed irrigation structures--no water in system. Outlet works tested by models at Bureau of Reclamation.
Bull Lake Dam	Bull Lake Creek, west of Riverton, Wyoming	Bur. of Reclamation Riverton Project	Irrigation	Earth fill, rock-face dam 75 ft. high by 3,400 ft. long. Straight concrete chute spillway, right abutment, controlled by three 29 ft. by 11 ft. radial gates, capacity 10,000 cfs hydraulic jump stilling pool. Outlet works consist of tunnel through dam, flow controlled by 5- by 5-ft. slide gates, discharge into separate stilling pool, capacity 4,000 cfs. Completed in 1938.	General inspection. Model studies made by Bureau of Reclamation.

The use of the overfall type of spillway forming a part of the dam is not limited to any particular size of structure, but is used at large and small dams. For example, Figure 32A shows the spillway section of the Grand Coulee Dam, which will discharge for the first time about June 1942; the discharge shown in the photograph is from the outlets passing through the dam, the quantity being 145,000 second-feet. This spillway is controlled by eleven 135- by 28-foot drum gates in the crest; the stilling pool is a bucket, the tip of the bucket being 360 feet below the crest (see Figure 35A). Some other pertinent dimensions of this and other dams referred to will be found in Table II. Figure 32B shows the spillway of the Hiwassee Dam, in which the crest is controlled by seven 23- by 32-foot radial gates; the stilling-pool is of the hydraulic jump type with a sill formed at the end of a horizontal floor, the drop from the crest to pool floor being 254.5 feet. For much smaller dams, the controlled overflow spillway may be designed as observed at the Pickwick Landing Dam, and several others on the Tennessee River, Figure 32C. Here the gates are of the Stoney type but in two sections, the flow passing either under or between the sections. The stilling-pool is a long horizontal apron with baffles attached, the tailwater conditions being unfavorable for the use of a true hydraulic jump, so a drowned jump or submerged roller is formed; the drop from crest to pool apron is 43 feet. Figure 32D shows an uncontrolled spillway which is actually, in this case, the entire dam. The structure is the Granite Reef diversion dam on the Salt River, which was passing 19,500 second-feet during the inspection. There is no stilling-pool at the toe of the dam, which is only 29 feet high, but some paving is in place to prevent erosion. In the 34 years since this dam was completed it has become good practice to use a bucket or hydraulic jump pool at the toe of small dams, particularly if a rock foundation is not present.

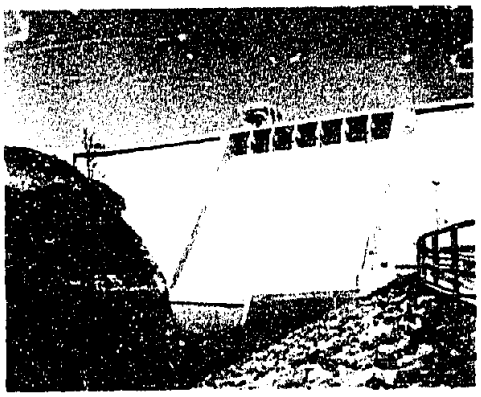
100. Chutes. The open-channel chute spillway is probably the most common type, and of necessity must be used with, but is not limited to, earth- or rock-fill dams. In general, this type of spillway is located near an abutment of the dam, excavation being made to form an

approach channel to the spillway. When controlled, the spillway flow is regulated by radial gates, Stoney gates, and occasionally by drum gates, set at the crest which is usually straight. The inclined concrete-lined channel carrying the water to the river channel below the dam is either straight or curved, with either sloping or vertical side walls. At the end of the chute, a hydraulic jump stilling-pool, and occasionally a bucket, is used when the material at this point is unable to withstand erosion, or when the high velocity flow entering the river from the spillway would cause eddies and surface disturbances serious enough to erode the toe of the dam and river channel. The width of the spillway may be uniform from the crest to the stilling-pool, or it may diverge, or it may be shaped in the form of an hourglass converging at first below the crest and then diverging to the stilling-pool, the width in any case being dictated by the design discharge, cost, and necessity of providing a proper depth at the entrance to the stilling-pool to assure the formation of a hydraulic jump. In contrast to the straight type of chute spillway, is the side channel type, wherein the crest is made parallel to the direction of flow in the chute. The chute below the crest section is usually straight and the flow is confined between sloping walls or in a circular tunnel. Where tunnels are employed, the flow is permitted to enter the river channel directly, no stilling pool being used, since the discharge from the tunnel usually occurs quite far downstream from the dam.

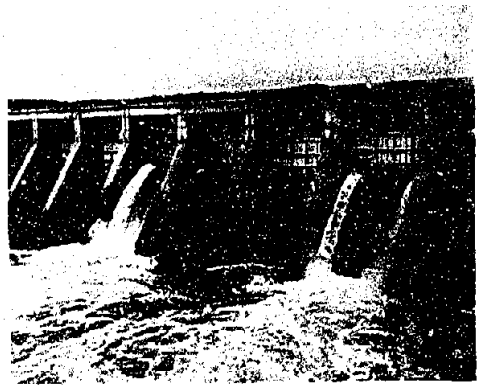
An example of the usual type of straight chute spillway is seen on Figure 32E. This is the spillway at the Unity Dam, Burnt River, Oregon, a drop of 90 feet occurring between the reservoir and floor of the stilling pool. Another example of the same type is the spillway at the Bull Lake Dam, Figure 32F; here the drop from the reservoir to the pool floor is 80 feet. In both examples, the crest is straight and contains radial gates for flow regulation. Figure 33A illustrates the curved superelevated type of open-channel spillway, regulated by two 50- by 50-foot Stoney gates at the spillway crest, the discharge being 4,400 second-feet at the time of the inspection. This is at the



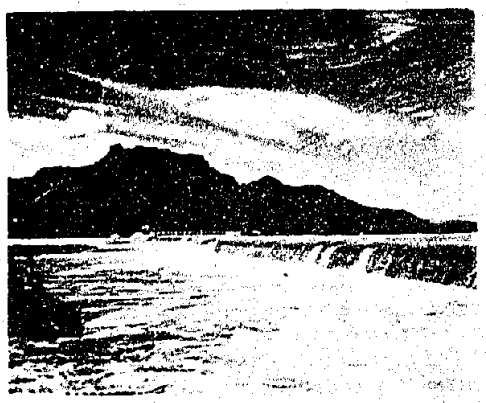
A. GRAND COULEE DAM



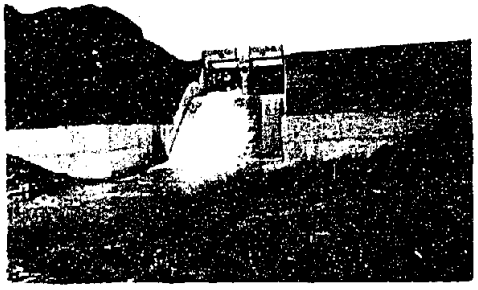
B. HIWASSEE DAM



C. PICKWICK LANDING DAM



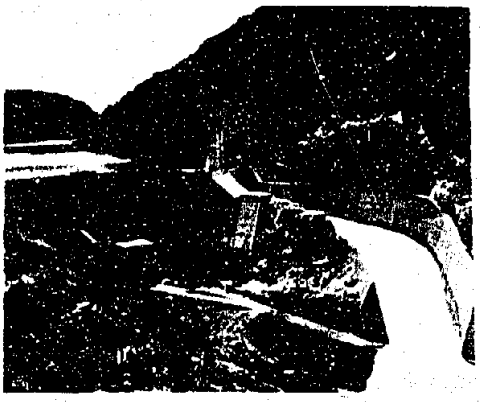
D. GRANITE REEF DIVERSION DAM



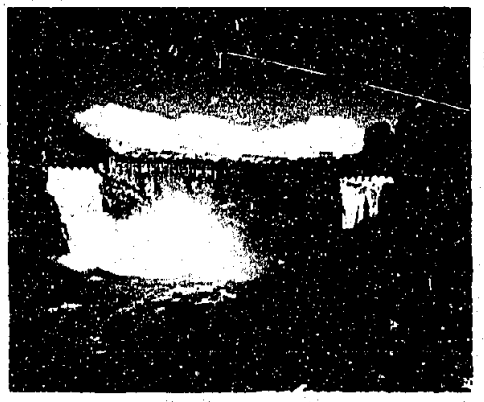
E. UNITY DAM

F. BULL LAKE DAM

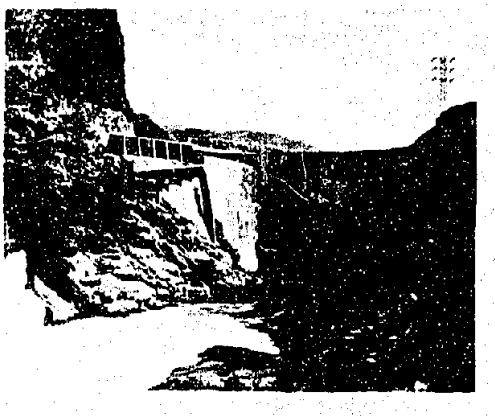
SPILLWAYS



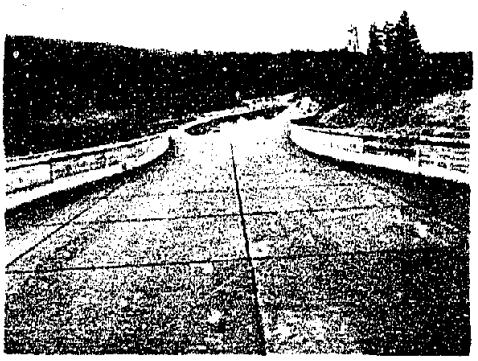
A. MORMON FLAT DAM



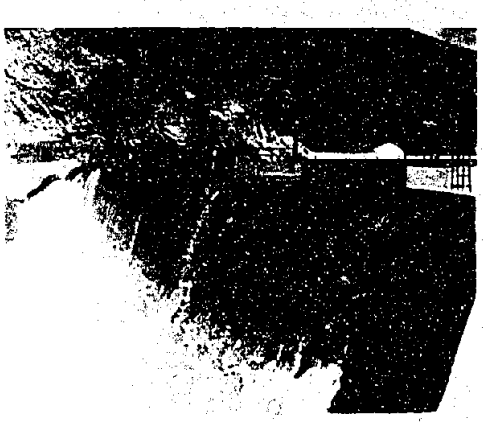
B. ROOSEVELT DAM



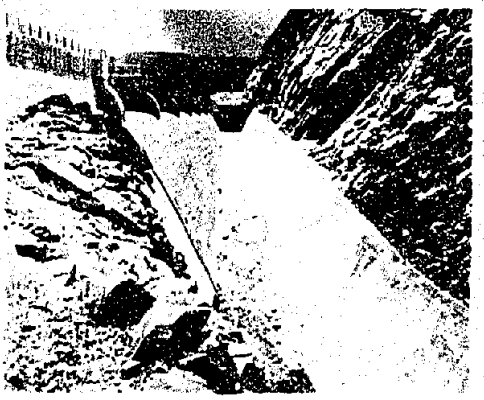
C. HORSE MESA DAM



D. CLE ELUM DAM



E. ARROWROCK DAM



F. TIETON DAM

SPILLWAYS

Mormon Flat Dam, Salt River, Arizona, where the rock canyon does not require the use of a stilling-pool. In several cases, it is convenient to use only a regulated crest with little or no spillway proper, but only the canyon wall to carry the water to the river. Figure 33B shows this type at the Roosevelt Dam, Salt River, Arizona, with 9,000 second-feet passing, while Figure 33C shows a short spillway at the Horse Mesa Dam, Salt River, Arizona, which projects the flow away from the canyon walls. In each case, the spray developing from these spillways may seriously affect the powerhouse in the canyon below each dam. The so-called hourglass type of spillway channel is shown by Figure 33D, the spillway at Cle Elum Dam, Cle Elum River, Washington. This is a view looking downstream from the crest, which is controlled by five 37- by 17-foot radial gates. The channel between side walls diverges from 201 feet at the crest to 100 feet in about 430 feet downstream, and then increases to 200 feet at the stilling-basin, which is 143 feet below the maximum reservoir water surface.

The side-channel type of spillway is shown by Figure 33E, taken at the Arrowrock Dam, Boise River, Idaho. A trapezoidal channel carries the water to a canyon where it spills down the face of the canyon into a stream below, the discharge shown being 5,400 second-feet. Figure 33F shows another of the same type at the Tieton Dam, Tieton River, Washington. In both cases, drum gates have been used at the crest for regulating the discharge. The best examples of side-channel spillways are those at Boulder Dam, one on the Arizona and one on the Nevada side, Figure 34A showing the Arizona spillway which is capable of discharging 200,000 second-feet. The flow passes into a 50-foot diameter concrete-lined tunnel on a 1:1 slope, which changes to the horizontal some 600 feet below the portal, the tunnel being 2,200 feet long.

101. Tunnels. The gloryhole or morning-glory type of spillway consists of a vertical shaft with a bellmouth entrance converging to a smaller diameter in the direction of flow. At the bottom of the shaft, a bend is made to a horizontal tunnel which carries the flow some distance below the dam, where it discharges freely into the

river channel. Figures 34B and C show the glory hole spillway at the Owyhee Dam, Owyhee River, Oregon. This spillway shaft changes through a 156-foot transition from 52.33 feet to 22.6 feet in diameter. The flow is controlled by a 60- by 12-foot spillway ring-gate operating in an annular pressure chamber formed in the crest structure; the ring-gate being a floating type crest, similar in operation to a drum gate. The drop from the crest to the invert of the horizontal tunnel is 309 feet, the total length being 1,100 feet from crest to outlet. Another gloryhole spillway may be found at the Gibson Dam, North Fork of Sun River, Montana. This shaft is surmounted by six 34- by 12-foot radial gates with the concave face toward the reservoir as shown on Figure 34D. The diameter of the shaft changes from 100 feet at the crest to 29.5 feet some distance below, the total drop being 204 feet from crest to horizontal tunnel, the total length being 452 feet. In both cases, the flow emerges from the tunnel into the river uncontrolled, since the jets from the tunnels are far enough downstream from the dam.

Instead of using a side channel or gloryhole spillway and tunnel combination, it is convenient to use an inclined tunnel spillway with the crest submerged and flow regulated by a Stoney gate. From the gate section, the tunnel enters a transition to a circular tunnel which bends to the horizontal at a lower elevation and discharges the water some distance below the dam. Such is the design at the Horse Mesa Dam already referred to, the tunnel spillway being used for flows up to 47,000 second-feet to eliminate the use of the small spillways at each abutment, one being shown on Figure 33C, because of spray affecting the powerhouse operation. The tunnel spillway at Horse Mesa Dam is shown discharging 8,600 second-feet in Figure 34E, the dam being to the right just upstream. This tunnel is regulated by a 40- by 44.5-foot fixed-wheel gate which rests on the crest at the tunnel invert beneath the water. The tunnel changes from 40 to 30 feet in diameter just below the crest, it then drops nearly 200 feet on an incline finally changing to the horizontal near the outlet, the total length being 43[illegible] feet.

102. Types. At all dams on irrigation projects and at most flood control projects, provisions are made to release water when the reservoir lies below the crest of the auxiliary spillway. This is usually done by providing outlets or sluices which pass through the dam at one or more elevations, the discharge being regulated at the upstream or downstream end of the outlets. Where power is developed, the turbine discharge is used as a supplemental flow.

In general, an outlet passing through a dam is either circular or rectangular in section, completely horizontal or inclined, or a combination of the two. The discharge is regulated by gates or valves either at the upstream or downstream end of the outlet. When controlled upstream, each outlet is provided with an emergency and service gate, the service gate being immediately downstream from the emergency gate. These gates may be of the ring follower, paradox, or ring seal type for high heads, or slide gates for much lower heads, say below 75 feet.[24] Complete regulation cannot be had with these gates, since they cannot be used at partial openings as the gate leaf would obstruct the flow and would cause undue vibration, coupled with excessive drawdown forces. The portion of the outlet in the vicinity of the gates is usually lined with steel, while farther downstream concrete may be sufficient, except perhaps for high-head outlets. The entrance to an outlet of this type is protected by trashracks, the entrance being in the shape of a bell-mouth, as previously mentioned in Section 50, Chapter III. At the exit, it is customary to converge the outlet about 15 percent of the normal area to maintain a positive hydraulic gradient. At this part of the outlet, a bend may be formed to cause the water to discharge down the face of the dam, or the flow may be allowed to discharge horizontally so as to fall free to the river below, provided spray from the jets would not harm any appurtenant works.

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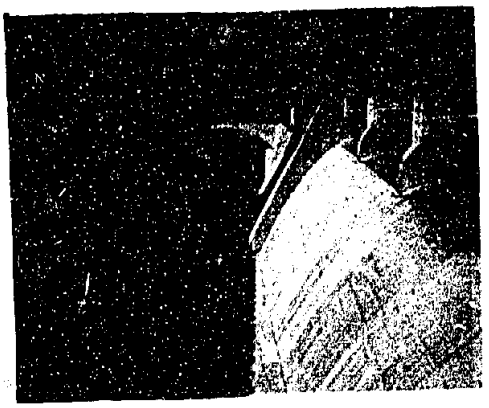
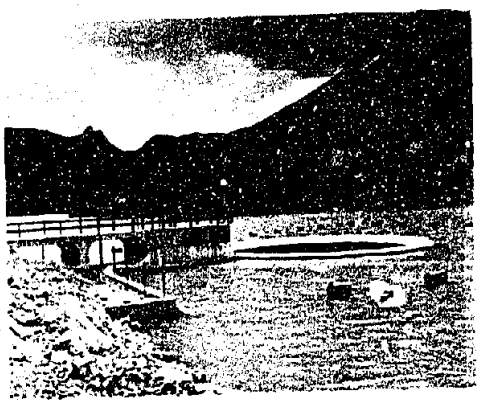
[24] Dams and Control Works, Second Edition, United States Department of Interior, Bureau of Reclamation, Washington, D. C., February 1938, pp. 177-204.

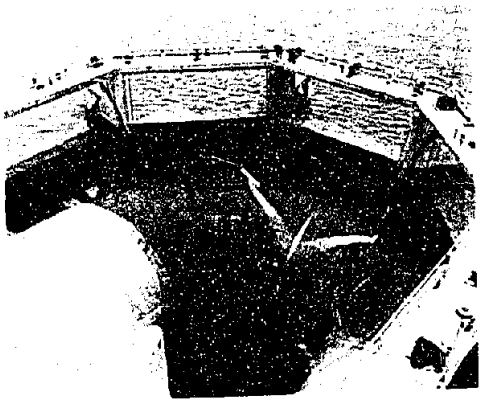


A. BOULDER DAM



B. OWYHEE DAM

C. OWYHEE DAM



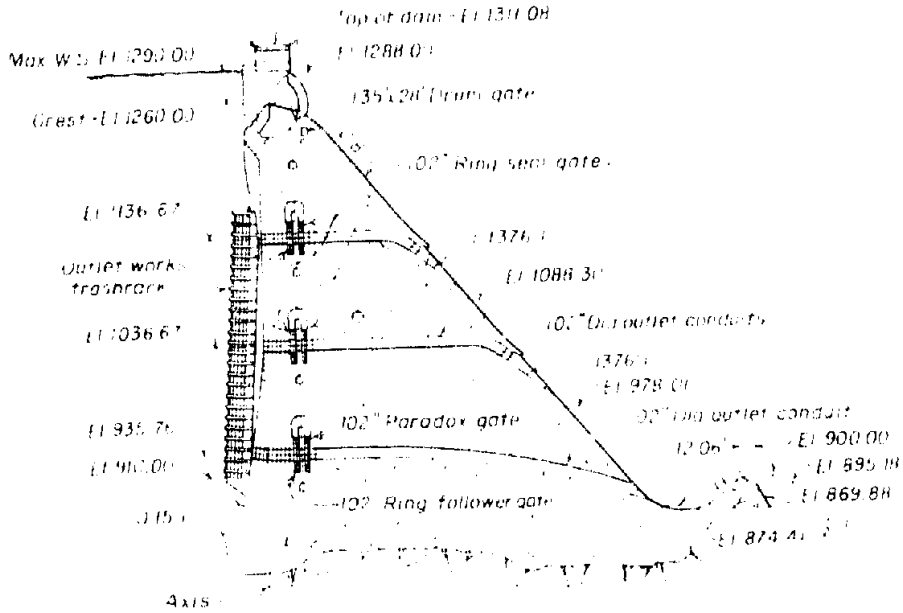
D. GIBSON DAM

E. HORSE MESA DAM

SPILLWAYS

103. Control gates upstream. An example of the former may be seen on Figure 35A showing the outlets at the Grand Coulee Dam, while Figure 35B shows a discharge of 2,400 second-feet at one of the upper tier outlets. (See also Figure 32A). Where the formation of spray from free falling jets is not harmful, the outlets may be completely horizontally arranged as shown on Figure 36A at the Arrowrock Dam, Boise River, Idaho. The Grand Coulee outlets are controlled by gates in the upstream end of the conduits (Figure 35A); the outlets at Arrowrock Dam are regulated by balanced needle valves at the very upstream end of the outlets, instead of by gates.

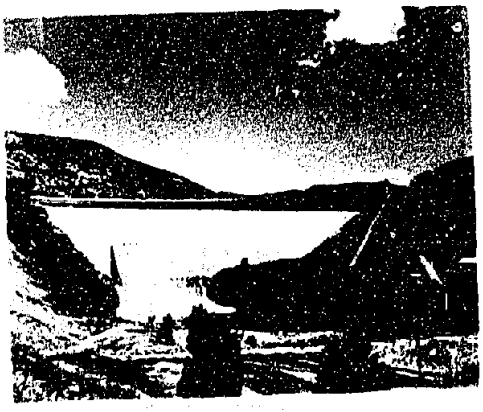
104. Regulating valves downstream. Where outlets are controlled at the downstream end, use is made of needle valves[24], and occasionally of butterfly valves and Howell-Bunger valves. Needle-valve outlets are more generally used with steel conduits passing through a dam near its base, the conduits being encased in the dam or passing through a large tunnel enclosing the outlet conduit, permitting access thereto. The use of valves at the outlet ends is not limited by the head, and permits the jet to discharge freely into a stilling-basin or river channel directly, if scour in the vicinity of the toe of a dam is not of importance. The problem of venting is not involved in this arrangement, whereas in outlets with gates upstream air vents must be provided for partial gate openings during opening and closing operations. In some instances, needle valves have been placed some distance upstream from the end of an outlet, but this practice seems to produce several problems in addition to supplying air; one in particular is the so-called gulping action caused by the flow alternately sealing and breaking away from the crown of the outlet. The really big advantage of needle valves is the ability to regulate the discharge to small quantities, less than 100 second-feet, for example, while at the same time large valves can discharge as much as 5,000 second-feet under high heads. Probably two of the largest needle valves (Pelton) in the United States are found at San Gabriel Dam No. 1, San Gabriel River, California. Figure 36B shows one of these valves, 129- by 117-inch, discharging 720 second-feet under a head of 230 feet with the valve open 8-1/2 percent,



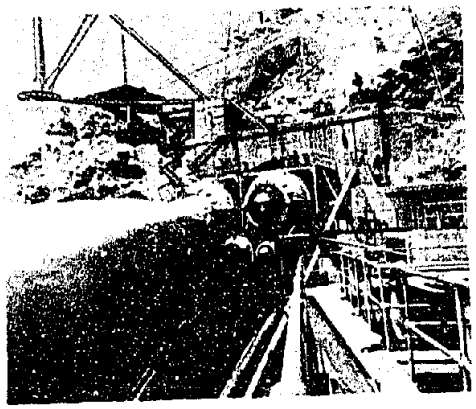
A. SPILLWAY SECTION

B. UPPER OUTLET - DISCHARGE 2,400 SECOND-FEET

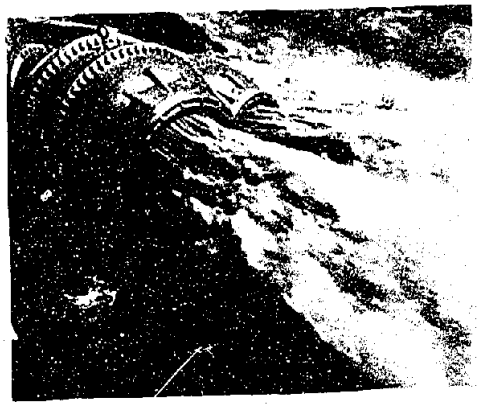
OUTLET WORKS - GRAND COULEE DAM



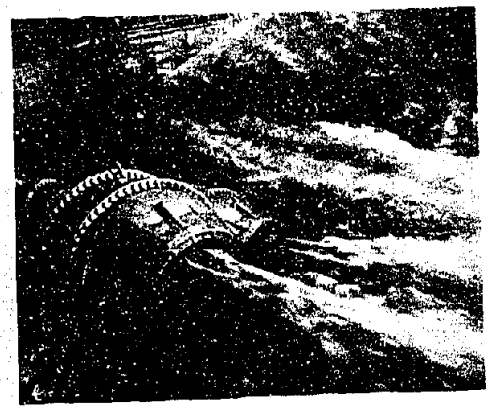
A. ARROWROCK DAM



B. SAN GABRIEL DAM NO. 1



C. STEWART MOUNTAIN DAM



D. STEWART MOUNTAIN DAM

E. OWYHEE DAM

F. GENE WASH DAM

OUTLET WORKS

the discharge falling freely into the river channel below the dam. These outlets are discussed in Proceedings, A.S.C.E., September 1941, p. 1199, by P. Baumann, in a paper entitled, "Design and Construction of San Gabriel Dam No. 1." It is interesting to note how horizontal the jet is when discharging under such a high head; note also the disintegration of the jet immediately below the vena contracta. Figure 36C shows two 42-inch needle valves at Stewart Mountain Dam, Salt River, Arizona, discharging 650 second-feet each at 100 percent opening under a head of approximately one hundred and fifty feet. These valves discharge into the tailwater which backs up to the toe of the dam, there being sufficient rock and water cushion to prevent erosion. Figure 36D shows an 84-inch butterfly valve adjacent to the needle valves discharging 2,100 second-feet at full-gate opening. These valves are not used where close regulation is required because of their inadequacy at openings less than 100 percent; at wide open position, they discharge more than a corresponding size needle valve. An example of needle-valve outlets releasing water at a higher elevation is shown by Figure 36E taken at Owyhee Dam, Owyhee River, Oregon. Here again the jet falls freely to the river channel. Where such a condition is not possible, either because the main river channel is too far from the outlet or because the channel is easily eroded, regardless of the energy dissipation of the jet caused by air-water friction, a separate stilling pool may be used to form a hydraulic jump in which the high velocity jet is considerably dissipated. At the Friant Dam, San Joaquin River, California, which is now nearly completed, the needle valve outlet works for two irrigation canals will use a hydraulic jump pool as described in Section 40, Chapter III. Likewise the river outlet works at Friant Dam will also employ the same principle, as discussed in Section 39, Chapter III; see Figures 1, 10, and 11. Howell-Bunger valves are particularly useful where spray is of no consideration and where a more or less solid jet is not desirable. Figure 36F shows a 42-inch Howell-Bunger valve at the Gene Wash Dam on the Colorado River Aqueduct, near Parker Dam, California.

STILLING POOLS

105. General. In the preceding discussion of spillways and outlet works, reference has been made to the use of a hydraulic jump stilling-pool, and a bucket for the dissipation of energy of high-velocity flows. It was stated that stilling pools at the end of certain spillways are essential if the river channel into which the spillway discharges is composed of material easily eroded by high velocity flow, particularly when such a spillway ends abruptly at or very near the toe of a dam. The question of what type of stilling-basin to use has been answered from the results of many model experiments and from the application of well-known principles concerning the phenomenon of the hydraulic jump. It is desired to explain now very briefly some of the general rules currently being followed to aid in the design of stilling-basins. Although most of these rules are derived from experience and sound judgment, some are almost completely empirical, yet they have been shown to be reliable and must suffice until a more rational solution has been obtained.

The reason why energy dissipation and velocity reduction is obtained by a hydraulic jump has already been explained by several authors, notably Bakhmeteff and Kalinske, so no discussion of this will be given here. The method of applying this knowledge to practical problems is more pertinent. In general, a properly designed stilling-basin will protect channels from erosion by reducing the velocity of the water, by insuring that the high-velocity flow does not come in contact with the bottom of the channel, or that it is diverted to regions of the bottom which are far enough downstream not to endanger the structure.

106. Types used in relation to tailwater. When using a hydraulic jump in a stilling-basin, it is essential that the tailwater elevation of the river below the dam be in agreement with the jump-height relation obtained from the conjugate depth relation for the jump. Although essential, such a condition rarely exists, since the tailwater, is a function of some control in the stream channel. As a result, when one plots the tailwater elevation required to form a jump on

an apron for several discharges, and then compares this with the tailwater curve, it is seen that the two curves do not coincide, but usually cross each other, or the jump-height curve lies entirely above or below the tailwater curve. Consequently, it is found that the tailwater may be excessive or deficient throughout the desired discharge range, or it is excessive for low discharges and deficient for high discharges or vice versa.

For the condition where the jump-height and tailwater curves are in good agreement, a horizontal apron stilling pool will be adequate, in which the pool floor is set at a depth below the tailwater, for the maximum discharge, equal to the depth d_2 required for a jump to form; it usually being confined between vertical training walls on each side of the basin.

Where it is found that the tailwater is always excessive, a jump would be submerged or drowned at all discharges. Were a stilling pool to be used with the floor placed at the elevation of the river channel, the jump would be drowned and practically no energy dissipation or velocity reduction would occur. Since it would be usually uneconomical to raise the basin floor to an elevation which would not cause drowning of the jump, it is convenient to use a bucket at the end of the spillway. When this is done, the jet entering the bucket is deflected upward to form a large elliptical roller, and a ground roller immediately downstream which conveys material upstream to the tip of the bucket. Such a design was used at Grand Coulee Dam as shown on Figure 35A. As yet there have been but few experiments made on buckets, so general design rules regarding their shape and position are not conclusive. In a paper in Civil Engineering, March 1940, "Model Study of Green Mountain Dam Spillway," by J. H. Douma, a design criterion for buckets was tentatively established by a relation between the energy of the falling jet at the tailwater and the volume of the bucket, which was taken as the area bounded by the spillway and bucket and tailwater per unit width. The curve was obtained from model experiments on four different structures. These same experiments, which were made by the Bureau of Reclamation, also indicated that the

lip of the bucket should be about one-sixth of the depth from the lowest point of the bucket to the maximum tailwater, the bucket invert being placed at about the elevation of the riverbed at the toe of the spillway. The slope of the spillway, tangent to the bucket, should not exceed 1:1, and the radius of the bucket is derived by furnishing the proper bucket volume for the maximum discharge. Training-walls on each side of the bucket should extend above maximum tailwater and to the end of the bucket to prevent the erosion of side eddies and their escaping from the bucket. If the wall height is excessive, it may be decreased enough to confine normal flows and thereby be overtopped for the maximum floods. Scour due to this would probably be a minimum for the short time a maximum flood would be sustained.

When it is found that the tailwater curve lies completely below the jump-height curve, the jet will repel the tailwater and proceed downstream unabated, so it may be possible to do the following: (1) Use a bucket which will project the falling jet upward and far enough downstream from the end of the spillway or dam so that scour resulting from the jet plunging into the tailwater will not have any effect on the stability of the structure; (2) raise the tailwater by a control immediately downstream using a level apron; (3) slope the pool floor to bring the two curves into closer agreement; and (4) place a system of baffles on the apron to force the jump to develop, thereby saving excavation.

The first possibility is only advisable when the rock of the river channel is massive and can resist erosion. Unconsolidated material or stratified rock would probably scour over a period of years, eventually causing a deep hole near the structure; furthermore, for small flows a bucket designed for the maximum discharge would not sufficiently elevate the jet of lower velocity to prevent scour immediately downstream from the lip of the bucket.

If the second possibility is tried, the control, such as a weir, will provide sufficient tailwater for the maximum discharge, but excessive tailwater may occur for the lower discharges. At high dams this weir would necessarily have to be of considerable height, which

might be prohibitive, and were the weir crest to be above the maximum tailwater, there would be a plunging of the flow from the stilling pool to the tailwater which might again require another pool below the control weir.

Use of the third possibility is frequently found to be adequate when the pool floor is sloped downstream. By doing this, the jump can adjust itself for various discharges to a position on the sloping apron which provides the correct tailwater. Accordingly, a deficiency in tailwater at any discharge is corrected by the jump moving down the apron, while excessive tailwater is corrected by the jump moving upstream, which is usually the case at lower discharges, so even if the jump is partly drowned the length of apron will be sufficient to correct for a less efficient jump. The method of determining the position and slope of the apron has already been treated in Section 36, Chapter III. In connecting the upstream end of the sloping apron to the slope of the face of the dam, a radius equal to one-third the maximum drop in water surface is frequently used. Too short a radius reduces the effectiveness of the jump, while a long radius increases the cost. For ordinary auxiliary spillways, not a part of the dam itself, the velocity entering the pool is usually much less than that occurring at high dams and the slope of the spillway channel is usually flatter than 1:1, so no radius is used.

The use of baffles or dentates on an apron, the fourth possibility, is not recommended at high overfall dams and at other spillways where the velocity at the pool entrance exceeds 75 feet per second regardless of whether a hydraulic jump forms naturally or not. Recent installations of this type have suffered from pitting due to cavitation, notably the baffle piers at the spillway of Bonneville Dam and at the Claytor Dam, New River, Virginia, and these two structures are not particularly high dams. Figure 37 shows the stilling-basin at the toe of Claytor Dam, New River, Virginia, the maximum drop in water surface being about one hundred and twenty-five feet; Figure 38 is a view of one side of a dentate in this pool, showing the pitting due to cavitation in the flow developed during a flood of 200,000 second-feet of relatively short duration.

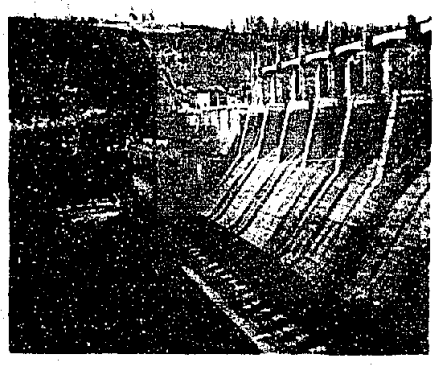


FIGURE 37, CLAYTOR DAM

The U. S. Waterways Experiment Station is now studying (1942) this particular stilling-pool by models to ascertain the phenomenon involved, while Carnegie Institute of Technology is testing the Bonneville spillway apron by a model to discover the reasons for cavitation and pitting of baffles on that stilling-pool apron. Instead of baffles at high dams, therefore, use is made of a solid end sill triangular in section, the top face sloping upwards and the height of the sill being about one-tenth the height of the jump at maximum discharge (Figure 1). This sill deflects the bottom velocities away from the river channel immediately downstream from the end of the pool, assuming a hydraulic jump is correctly formed on the apron. The use of baffles to force a jump to form or to dissipate the energy of the overfalling sheet of high-velocity water when insufficient tailwater occurs, is not recommended for the reasons given, in addition to their being damaged by falling logs or other debris.

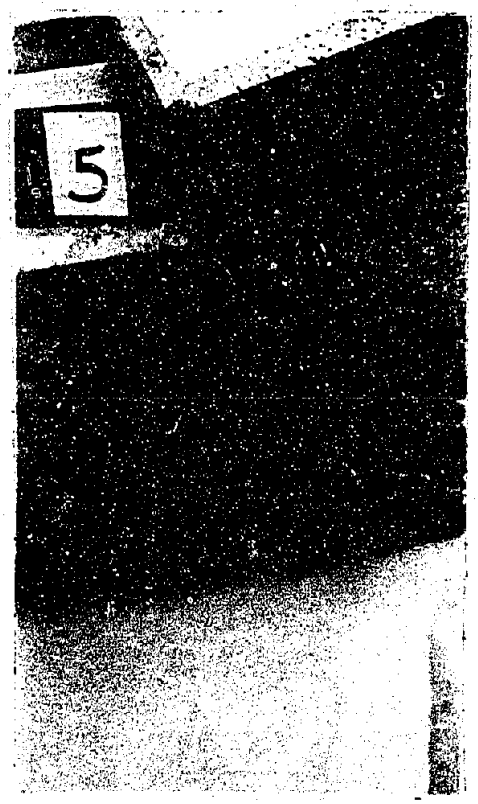


FIGURE 38, BAFFLE PITTED BY CAVITATION

In the stilling-pools where baffles and dentates are allowable, steel angles are frequently placed on all sharp corners to prevent erosion and damage by abrasion. Chamfering the corner is not particularly good practice, since this reduces the effectiveness of the baffles and will decrease the efficiency of the jump.

107. General design rules. General design rules derived from a series of model experiments at the Bureau of Reclamation illustrate one method of designing stilling-basins for use with auxiliary spillways not forming a part of the dam itself.[25] For this type of pool the tail-water curve and jump-height curve should be in fairly good agreement over the range of discharge anticipated, and the basin must be rectangular in section. These rules are mostly empirical and are not

[25]"Hydraulic Model Studies of the Wickiup Outlet Works Stilling Basin-Deschutes Project, Oregon", Memorandum to Chief Designing Engineer, by J. H. Douma, Denver, Colorado, June 30, 1939.

based on enough data to make them infallible, yet it has been found that they are valuable for preliminary designs, which should usually be checked by model tests. Referring to Figure 39, these statements are applicable:

a. The width, w, of the basin is determined to result in the most economical structure.

b. The discharge, q, per foot of width at the pool entrance is equal to the design maximum flood discharge divided by the width at the pool entrance.

c. The theoretical velocity, v_1, at the pool entrance is computed from the available energy head and properly evaluated losses. (For spillway chutes use King's formula for flow in steep chutes). [Or take $v_1 = \sqrt{2gH}$, where H is the maximum drop from reservoir pool floor. The latter neglects losses due to friction which it has been found increases d_2, the height of the jump, about 5 percent, so a factor of safety is obtained in case the tailwater is not developed as anticipated.]

d. The theoretical depth, d_1, at the pool entrance is equal to q/v_1.

e. The theoretical jump depth, d_2, is computed by the momentum formula.

f. The experimental jump depth, d_2', is equal to 85 percent of d_2.

g. The required stilling basin floor elevation is equal to the maximum discharge tailwater elevation minus d_2'.

h. The required stilling basin length L, is equal to $3d_2$.

i. The height of chute blocks, h_1, is equal to d_1 or $1/9\ d_2$, whichever is largest.

j. The height of basin blocks, h_2, is equal to $1/4\ d_2$ for values of d_2 from 0 to 8 feet, follows a straight line variation of $1/4\ d_2$ from 8 to 24 feet and is equal to $1/8\ d_2$ for values of d_2 above 24 feet.

k. The height of solid end sill, h_3, is equal to $1/8\ d_2$.

l. The distance, a, from the ends of the stilling-basin to the vertical upstream faces of the basin blocks is equal to 1/3 L.

m. The maximum width of blocks and spaces between them are equal to h_1, and the minimum width is limited to about 18 inches.

n. The top dimensions of the floor blocks and end sill, parallel to the basin centerline, are equal to 1/4 h_2 and 1/4 h_3, respectively, with a minimum value of about eight inches.

o. Chute and basin blocks should be staggered with no blocks against the side walls and one more basin block than chute blocks.

p. The back slope of the basin clocks and end sill may be such as to be the most economical, usually 1:1, and for economical reasons the end sill may be rectangular in cross-section when less than 3 feet.

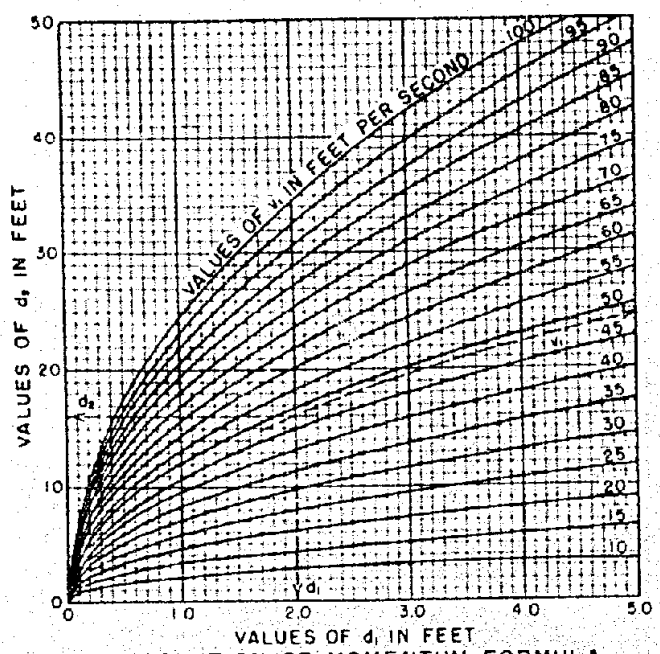
q. The slope of the transition bottom at the end of the basin may vary from horizontal to 6:1 when of earth, rock excavation, or riprap and up to 3:1 when concrete.

r. The chute slope entering the basin may vary from horizontal to 1:1.

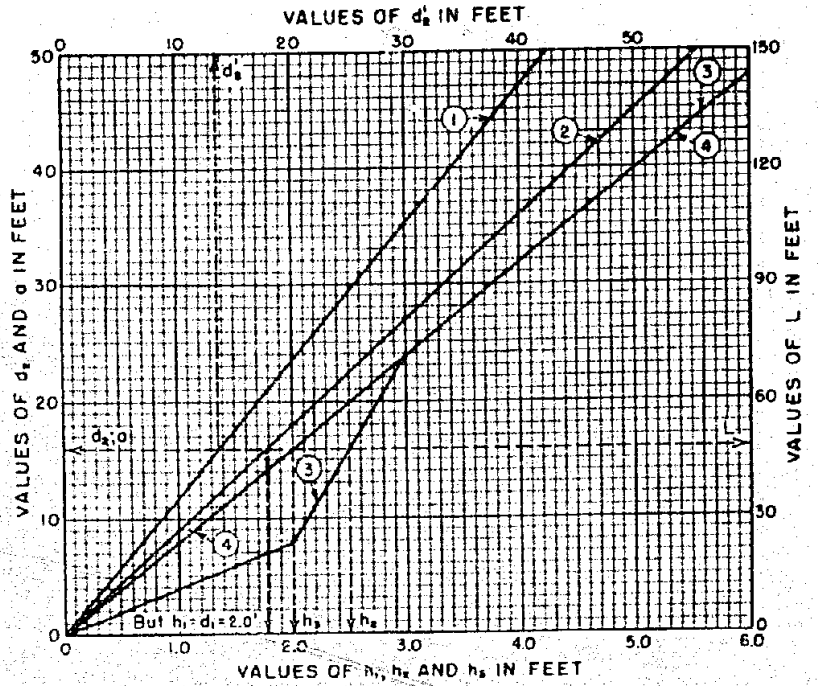
It should be emphasized that stilling-basins of rectangular cross-section perform much more efficiently than those trapezoidal in cross-cross-section. This has been demonstrated many times in the laboratory and is especially convincing when studie in the field. With the former type, the velocity and discharge are spread uniformly across the pool, while in the latter type the high-velocity jet does not spread completely so that along each training-wall there is flow moving upstream causing a large eddy and general unbalance. Even though the momentum formula be adjusted to take into account the pressure added by triangular prisms on each side of the basin, the jet cannot spraad across the increased width of pool caused by sloping the side training-walls. Several examples of this will be shown below in reference to irrigation structures studied in the field.

IRRIGATION STRUCTURES

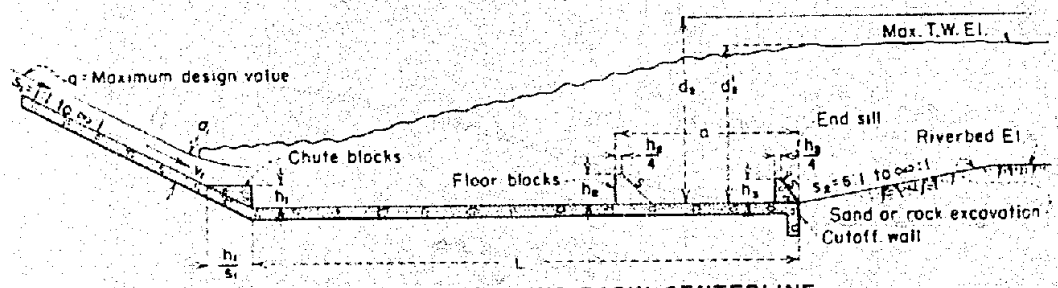
108. Types. On most irrigation projects the water system is divided as follows: (1) Main storage dam and outlet works; (2) headworks to main canal either at main dam or at a diversion dam farther



A. SOLUTION OF MOMENTUM FORMULA



B. STILLING BASIN DIMENSIONS



C. ELEVATION ALONG STILLING BASIN CENTERLINE

MOMENTUM FORMULA

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{d_1^2}{4} + \frac{2 v_1^2 d_1}{g}}$$

1. For values of d_2'
2. For values of h_1 when larger than $1.0\, d_1$
3. For values of h_2
4. For values of h_3

EXAMPLE

q = 96.0 sec-ft per ft width	d_2 = 16.0 ft.	h_1 = 2.0 ft.	$h_2/4$ = 6 in.	a = 16 ft
v_1 = 48.0 ft per sec	d_2' = 13.6 ft.	h_2 = 2.5 ft.	$h_3/4$ = 6 in.	S_1, S_1 and S_2 are all variable.
d_1 = 2.00 ft	Floor el = max T.W. el. − 13.6 ft.	h_3 = 2.0 ft	L = 48 ft.	

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

RELATIONS BETWEEN VARIABLES IN STILLING BASIN DESIGN FOR RECTANGULAR SPILLWAY CHANNELS BASED ON HYDRAULIC MODEL EXPERIMENTS

DRAWN J.H.D.	SUBMITTED Jacob E. Warnock		
TRACED O.E.W.-C.J.L.	RECOMMENDED		
CHECKED J.H.D.	APPROVED		
	DENVER, COLORADO MARCH 28, 1939	X-D-[illegible]	

FIGURE 37

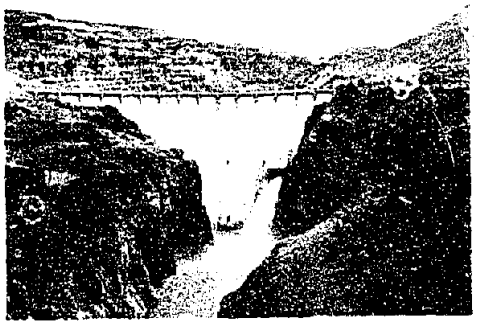
downstream; (3) division works where flow of main canal is divided to supply two or more canals leading to the irrigated lands; and (4) laterals, supplied by main and secondary canals, which bring water to individual parcels of land. In each of the supply channels, sudden changes in grade require drop structures which are actually small spillways similar in many respects to some of the larger spillways discussed above. The problem of energy dissipation is just as important and in many cases is made more difficult because of the ease with which unlined canals will erode with water moving at velocities less than 8 feet per second. In addition to drop structures, there are turnouts which consist of slide gates at the canal side of a concrete or timber flume recessed in the canal bank to supply individual lands with water; weirs, submerged orifices, and Parshall flumes to measure quantities delivered; siphons to cross topographical features; and wasteways, usually automatic, located just upstream from a gate or system of gates in the main channel, these being used to isolate a portion of a canal which might fail and to release the excess water through the wasteways.

109. Main dam. As an example of a main storage dam, Figure 40A shows the Owyhee Dam, Owyhee River, Oregon (see Table II). This dam creates a reservoir of 1,120,000 acre-feet to supply the Owyhee Project of the Bureau of Reclamation situated in eastern Oregon. The glory-hole spillway at this dam has been discussed above and is shown on Figures 34B and C. The Owyhee River is supplied by the needle-valve outlet works (Figure 36E), but the project is furnished water from a tunnel outlet located upstream from the dam and about eighty-five feet below the reservoir water surface. The end of this tunnel is shown on Figure 40B as it emerges from a ridge four miles from the intake. A few feet downstream, a bifurcation works occurs as shown in Figure 40C, the main canal swinging to the right. Control here is obtained by two radial gates; note the hydraulic jump forming in the flume of the main canal.

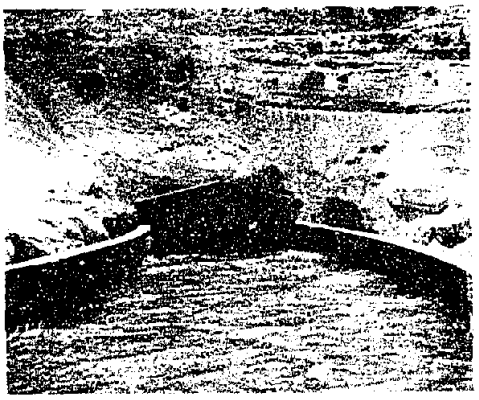
110. Diversion dam. The Roza Diversion Dam is a good example of a diversion structure located in stream fed by storage reservoirs in its upper watershed. Figure 40D is a view from the left bank of the

Yakima River, Washington, showing the roller-gate section and the headworks to the Yakima Ridge Canal in the right background. The large cylindrical object at the headworks is a spare fish screen 20 feet long by 13 feet 1 inch in diameter, six of them being located in the entrance to the headworks to prevent small migrating fish (salmon) from entering the canal. To prevent clogging of the screens by debris, each screen is rotated by motors at a circumferential speed of two feet per minute, the maximum allowable being five feet per minute. As listed in Table II, the maximum diversion into the canal at the headworks will be 2,200 second-feet. The Granite Reef Diversion Dam previously referred to is a different type of diversion structure in that strict regulation of the river elevation upstream is not required for diversion purposes as is the case at the Roza Diversion Dam. The ability to divert water is regulated by the amount released into the river above the diversion dam from the storage reservoirs located on the stream. As a result, a simple uncontrolled weir is adequate as shown on Figure 32D.

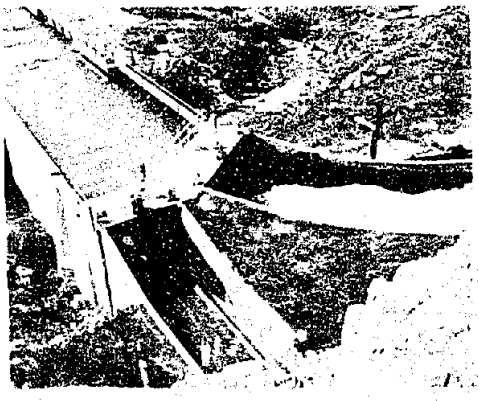
111. Trapezoidal and rectangular stilling pools at drop structures. As examples of drop structures in laterals or main canals, two types will be shown to demonstrate the ineffectiveness of stilling pools trapezoidal in cross-section. Figure 41A is a view of a trapezoidal stilling pool of a drop turnout on the Klamath Project, Oregon. The flow in the pool is clockwise, that is, the flow on the right is moving downstream along the side of the pool and upstream along the opposite side. As a result, the energy dissipation and velocity reduction is a minimum, which is borne out by the constant efforts required to maintain the canal banks immediately downstream. Figure 41B illustrates the same phenomenon with the flow in the pool moving counterclockwise at a drop on the Yakima Project, Washington. Figure 41C illustrates the same lack of dissipation at a structure on the Owyhee Project, Oregon, even though the flow at the structure is moving downstream uniformly, but notice the waves as a result of high velocity flow escaping the stilling pool. In comparison, a rectangular stilling pool of the type shown on Figure 41D is much more effective. A



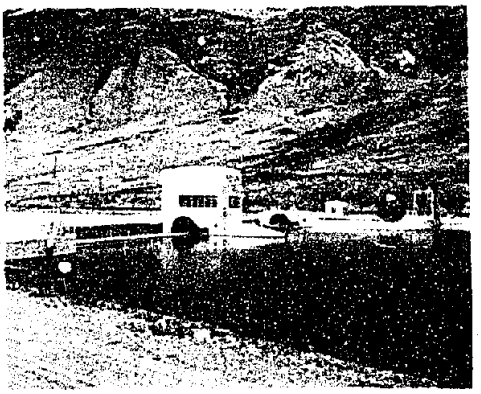
A. OWYHEE DAM



B. TUNNEL NO. 1

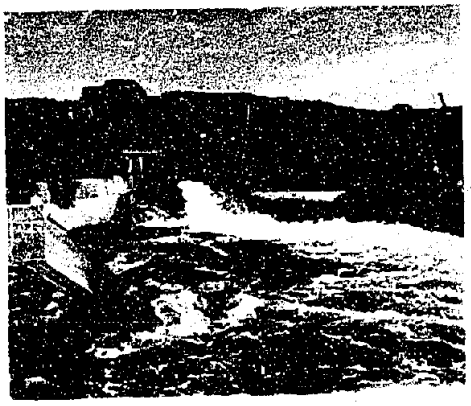


C. BIFURCATION WORKS

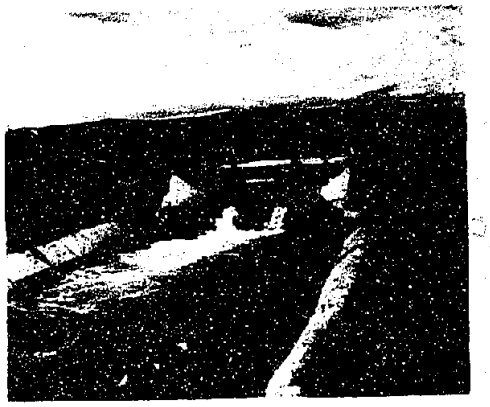


D. ROZA DIVERSION DAM

IRRIGATION STRUCTURES



A. TRAPEZOIDAL POOL AT TURNOUT



B. TRAPEZOIDAL POOL AT DROP

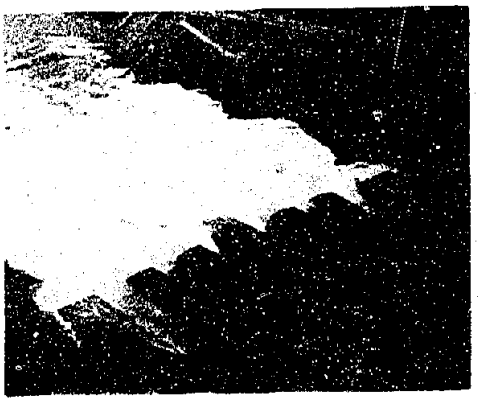
C. TRAPEZOIDAL POOL AT DROP

D. RECTANGULAR POOL AT DROP

E. RECTANGULAR POOL AT DROP

IRRIGATION STRUCTURES

A. REVISED POOL



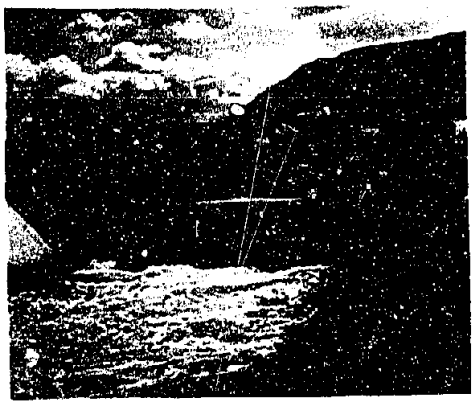
B. REVISED TRAPEZOIDAL POOL

C. REVISED TRAPEZOIDAL POOL

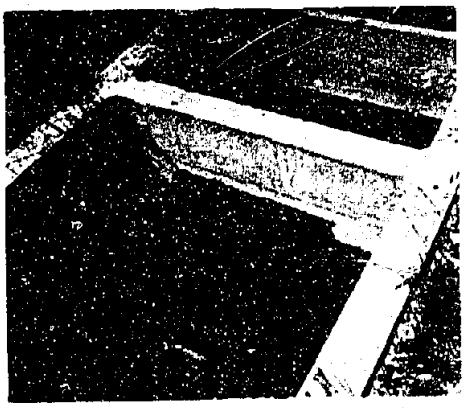
D. H-DROP

E. H-DROP

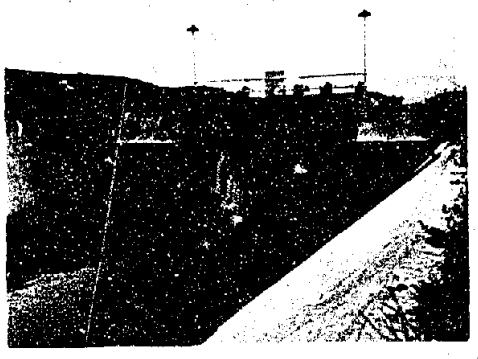
IRRIGATION STRUCTURES



A. CIPOLLETTI WEIR



B. SUBMERGED ORIFICE



C. SIPHON WASTEWAY

D. WASTEWAY (SAME AS C)

IRRIGATION STRUCTURES

recently completed drop structure similar to this may be seen on the Shoshone Project, Wyoming, as given on Figure 41E. At the end of the rectangular pool, a warped transition occurs followed by hand-placed riprap. In this manner, the stilling pool is made as wide as the jet entering the drop, and the transition is made to the width of the lateral or canal downstream from the jump, instead of in the stilling pool as occurs at trapezoidal drops. It is common experience to find the riprap in place at the end of an irrigation season downstream from rectangular drop structures, but below trapezoidal drops the riprap is usually washed into the canal and the banks and bottom of the canal are heavily scoured.

On the Sun River Project, Montana, several trapezoidal drop structures failed as a result of the stilling pool being unable to retain the jump within the pool. As explained above, the reason for this and the slight amount of dissipation occurring is a result of unequal flow distribution throughout a trapezoidal section. These drops that failed were rebuilt to the rectangular type as shown by Figure 42A, while those that did not fail completely were revised by placing dentates at the pool entrance to spread the flow across the pool width as shown on Figures 42B and C. These structures were studied by models before this solution was obtained.

Where the drop is particularly small, H-drops are frequently used similar to the one shown on Figure 42D, on the Owyhee Project, Oregon. The drop at this structure is two feet. Figure 42E shows a similar structure with a drop of about seven feet, on the Yakima Project, Washington.

112. Flow measurements. For measuring the flow of water in canals and laterals the Cipolletti weir is used almost exclusively. Figure 43A shows a typical installation in the Kingman lateral, Owyhee Project. A submerged orifice frequently seen at delivery points to farms is shown on Figure 43B.

113. Wasteway. Wasteways usually are automatic, being provided with siphons as shown by Figure 43C. This wasteway is on the Rosa Canal, Yakima Project, Washington. A view downstream from the siphon

outlets is shown on Figure 43D. If the capacity of the siphons is exceeded, two radial gates can be raised to supplement the flow. Model tests were made of this wasteway and the siphons, which were designed with several improvements to increase the time of priming under a minimum priming head.

MODEL-PROTOTYPE CORRELATION

114. Summary. In attempting to correlate model and prototype performance during this field investigation, no opportunity was available for quantitative comparisons and only one opportunity was had to compare qualitatively, this with reference to Check Drop 4 of the Sunnyside Main Canal as discussed in Chapter III, and included in the aforementioned paper entitled, "Model-Prototype Comparisons of Hydraulic Structures," by Jacob E. Warnock, and H. G. Dewey, Jr. to be published in a 1942 issue of Proceedings of the Society in a symposium on this subject. In this paper a discussion is made of the various phases of model-prototype confirmations: Reasons for desiring comparisons, the types made, difficulties encountered, and the measurements and instruments required. In addition to the correlation of Check Drop 4, an account is given of the pressure and discharge tests on the 102-inch river outlets of the Grand Coulee Dam showing a comparison with model data. A bibliography is also included to present all available information on the subject of correlation. With the exception of the latter example, most of the information in this paper was based on findings hitherto known or later obtained from discussions with engineers during visits to hydraulic laboratories and projects in the field. It will be evident from reading the above symposium just why correlations have been so few and hard to obtain. Accordingly, the author is not particularly disappointed that more correlations were not obtained during the period of the scholarship.

APPENDIX I

APPENDIX I

1. Itinerary. With few exceptions the following itinerary is similar to the one originally submitted to the Committee on Freeman Fund for approval just prior to starting the scholarship period. Figure 44 shows, in general, the route followed:

Table III ITINERARY FOR FREEMAN SCHOLARSHIP, 1940-41.

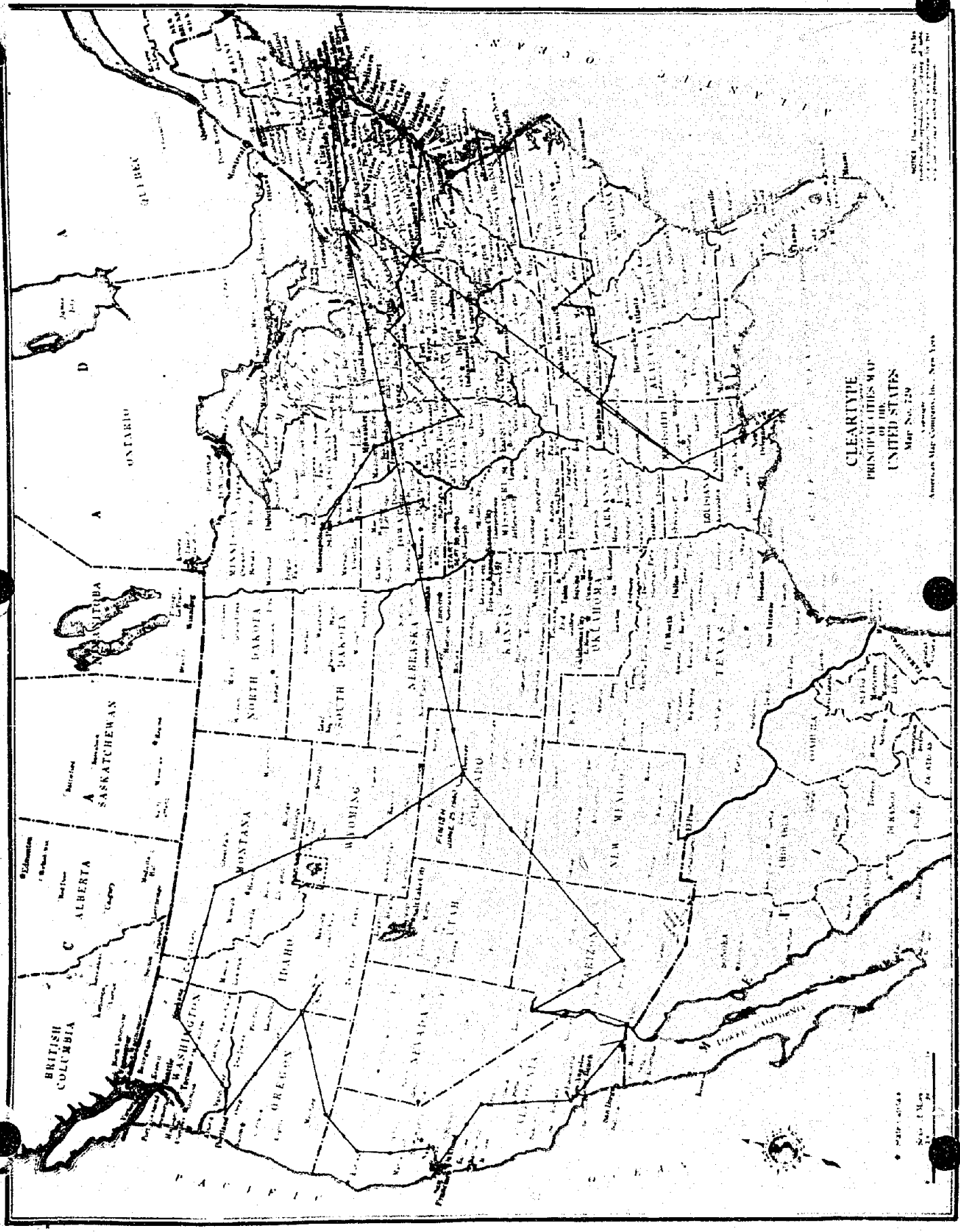
Organization	Place	Dates	Remarks
University of Iowa	Iowa City, Iowa	Sept. 30—Oct. 12	Hydraulic Laboratory
University of Minnesota	Minneapolis, Minn.	Oct. 13-25	" "
University of Wisconsin	Madison, Wisc.	Oct. 26-31	" "
University of Illinois	Urbana, Illinois	Nov. 3-8	" "
University of Michigan	Ann Arbor, Mich.	Nov. 9-12	" "
Case School of Applied Science	Cleveland, Ohio	Nov. 13-16	" "
Carnegie Institute of Technology	Pittsburgh, Pa.	Nov. 17-20	" "
S. Morgan Smith Company	York, Pa.	Nov. 21-22	" " and shops
Pennsylvania Water & Power Company	Safe Harbor & Holtwood, Pa.	Nov. 22-23	Safe Harbor and Holtwood Dams, cavitation laboratory (turbine testing)
Philadelphia Electric Co.	Conowingo, Md.	Nov. 23	Conowingo Dam
University of Pennsylvania	Philadelphia, Pa.	Nov. 24-26	Hydraulic Laboratory
I. P. Morris Division, Baldwin Southwark Corp.			

Organization	Place	Dates	Remarks
Columbia Univ.	New York City	Nov. 27--Dec. 11	Hydraulic Laboratories,
New York Univ.	"	"	Meetings, Etc.
Brooklyn Polytechnic Inst.	"	"	"
Board of Water Supply	"	"	"
A.S.M.E. Annual Meeting	"	"	"
A.S.C.E. Office	"	"	"
University of Connecticut	Storrs, Conn.	Dec. 12	Hydraulic Laboratory
U. S. Engineer Office	Providence, R. I.	Dec. 13	Model-prototype information; visit with Mr. Clarke Freeman
Mass. Inst. of Technology	Cambridge, Mass.	Dec. 14-19	Hydraulic Laboratory
- - -	Rochester, N. Y.	Dec. 20--Jan. 5	Christmas with family
Cornell Univ.	Ithaca, N. Y.	Jan. 6-7	Hydraulic Laboratory
Worcester Poly. Institute	Worcester, Mass.	Jan. 8-13	" "
A. S. C. E.	New York City	Jan. 14-20	Annual Meeting
National Hydraulic Laboratory	Washington, D. C.	Jan. 22-25	Hydraulic Laboratories
Beach Erosion Board	" "	"	" "
D. W. Taylor Model Basin, U. S. N.	" "	"	" "
Newport News Shipbuilding & Dry Dock Co.	Newport News, Va.	Jan. 27-29	" " and shops
Appalachian Electric Power Company	Near Pulaski, Va.	Jan. 30	Claytor Dam
Soil Conservation Service	Spartanburg, S. C.	Jan. 31	Hydraulic Laboratory

Organization	Place	Dates	Remarks
Soil Conservation Service	Greenville, S. C.	Feb. 1	Hydraulic Laboratory
Tennessee Valley Authority	Norris, Tenn.	Feb. 2-9	Hydraulic Laboratory and Norris Dam
Tennessee Valley Authority	From near Murphy, N. C. to Savannah, Tenn., following Tennessee River	Feb. 10-12	Hiwassee, Chickamauga, Hales Bar, Guntersville, Wheeler, Wilson, and Pickwick Landing Dams
U. S. Waterways Experiment Sta.	Vicksburg, Miss.	Feb. 13--Mar. 18	Hydraulic Laboratory and field trips
- - -	Rochester, N. Y.	Mar. 21--April 3	With family
Bureau of Reclamation	Denver, Colorado	April 7-12	Hydraulic Laboratory
Salt River Valley Water Users' Association	Phoenix, Arizona	April 14-21	Inspection of dams on Salt River and irrigation project
Bureau of Reclamation	Boulder City, Nev.	April 23-24	Boulder Dam
Bureau of Reclamation	Parker Dam, Ariz.-Calif.	April 25-26	Parker Dam
Bureau of Reclamation	Yuma, Arizona	April 27--May 1	Boulder Canyon Project, All American Canal System
U. S. Engineer Office and California Inst. of Technology	Los Angeles, and Pasadena, Calif.	May 2-14	Hydraulic Laboratories and flood control project
Bureau of Reclamation	Friant, Calif.	May 15	Friant Dam
Univ. of Calif.	Berkeley, Calif.	May 16-20	Hydraulic Laboratory
Bureau of Reclamation	Antioch, Calif.	May 21	Contra Costa Canal
Bureau of Reclamation	Orland, Calif.	May 22	Orland Irrigation Project

Organization	Place	Dates	Remarks
Bureau of Reclamation	Redding, Calif.	May 23	Shasta Dam
Bureau of Reclamation	Klamath Falls, Oregon	May 24-25	Klamath Irrigation Project
Bureau of Reclamation	Ontario, Oregon	May 26-29	Owyhee Irrigation Project
U. S. Engineer Office	Portland, Oregon	May 31—June 3	Bonneville Dam and Laboratory
Bureau of Reclamation	Yakima, Wash.	June 4-9	Yakima Irrigation Project
Bureau of Reclamation	Grand Coulee, Washington	June 10-11	Grand Coulee Dam
Bureau of Reclamation	Fairfield, Mont.	June 13-15	Sun River Irrigation Project
Bureau of Reclamation	Cody, Wyoming	June 16-17	Shoshone Irrigation Project
Bureau of Reclamation	Riverton, Wyoming	June 17	Bull Lake Dam
- - -	Denver, Colorado	June 18	End of travels
Bureau of Reclamation	Denver, Colorado	June 25	Returned to work in hydraulic laboratory

FIGURE 44



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2. Progress reports. To complete the record of this scholarship, copies are given below of monthly progress reports submitted to Mr. George T. Seabury, Secretary, as stipulated by the Committee on Freeman Fund as part of the conditions of award.

PROGRESS REPORT FOR OCTOBER 1940

Introduction

The Freeman Scholarship was awarded in Denver, Colorado, on July 24, 1940. Because of personal affairs and because it was desired to start traveling after classes had begun at the various universities, the scholarship was established for the period of October 1940 to July 1941. An itinerary for this period was submitted September 6, 1940, and approval was made on September 17, 1940.

The work of this scholarship will be divided into two parts: a study of the current hydraulic research at representative universities and other organizations; and a collection of all available data on the correlation of model and prototype together with current hydraulic design practice.

Summary of Work Completed October 1940

During October, visits were made to the University of Iowa, Iowa City, Iowa, from September 30 to October 12; the University of Minnesota, Minneapolis, Minnesota, from October 13 to October 25; and the University of Wisconsin, Madison, Wisconsin, from October 26 to October 31. A brief follows of the work covered at these universities, together with a list of field structures observed.

University of Iowa, September 30 to October 12

The Iowa Institute of Hydraulic Research in cooperation with various Government agencies in concentrating on the study of fluid turbulence, the sedimentation of streams and a correlation of these two in an attempt to develop an analytical solution for the many problems met in determining suspended sediment concentration. Besides this problem, work is being done on fishways, simultaneous flow of air

and water in closed conduits, spreading of jets on flat floors and in open channel transitions, and many others.

The U. S. Engineers at the laboratory are finishing the work at hand preparatory to closing their office. Reports of model tests on locks are in preparation, and of particular interest is a report to be published early next year on the correlation of model and prototype lock tests.

University of Minnesota, October 13-25

The St. Anthony Falls Hydraulics Laboratory is perhaps the best in the United States, but the problems being studied are necessarily few at this time because the laboratory has been in operation for a relatively short time. Nevertheless, much work has been done and is being continued on bed load movement at the confluence of rivers and in river contraction works. Another problem of considerable importance and one only recently recognized is that of high-velocity flow in open channels. The results of this study will have an important practical value as regards the design of spillways and stilling pools. In addition to these studies, work is being done on the stability of sand dams, fluid turbulence as related to sediment transportation, and open channel flow using fluids other than water.

The Soil Conservation Service is testing many types of soil erosion structures, and the U. S. Engineers have been testing a river model of the St. Anthony Falls Navigation Project. This model is of particular interest because it includes the stretch of the river adjacent to the hydraulics laboratory.

A test was made on the cylinder gates of the large outside volumetric tanks in connection with some work being done on cylinder gates by the Bureau of Reclamation in Denver.

Inspection trips were made to Lock and Dam Nos. 1, 2, and 3, situated on the Mississippi River between St. Paul and Red Wing, Minnesota. An inspection was also made of the Minneapolis-St. Paul Sewage Treatment Plant, the largest of its kind in the world.

University of Wisconsin, October 26-31

The primary study at the University of Wisconsin is being made in cooperation with the National Truck Tank Association. Data have been obtained for designing oil-truck tanks' hydraulic systems enabling faster unloading of gasoline and fuel oil. To accomplish this, a careful study has been made of the losses in pipes, meters, valves and fittings used in oil-truck tanks for fuel oils, gasoline, cleaning fluid and water.

Problems completed include a study of weir coefficients for flow of water and oil, and a study of water hammer in small pipes. The latter experiments will be continued to include a study of water hammer in small compound pipes. The flow of various liquids in open channels will be studied to determine the effect of viscosity and surface tension.

After an interesting conversation with the venerable Daniel W. Mead, an inspection trip was made to two of his hydro plants, Prairie du Sac and Kilbourn on the Wisconsin River. Model studies have frequently been made of these structures when maintenance problems developed. These were studied and some data were obtained on prototype correlation.

PROGRESS REPORT FOR NOVEMBER 1940

Introduction

The first progress report of the Freeman Scholarship, which extends from October 1940 to July 1941, was submitted November 4, 1940. That report reviewed some of the hydraulic research observed during October 1940 when visits were made to universities and field structures in the Midwest.

This report for November 1940 outlines some of the hydraulic research observed at other universities in the same area, together with the work observed in a section of the East.

University of Michigan

Because of insufficient hydraulic laboratory facilities, the work at the University of Michigan has been devoted almost entirely to hydrological studies. In this regard, a direct method of flood routing has been developed recently and a paper on this subject has been accepted for publication in Proceedings of the American Society of Civil Engineers. In a synopsis of this paper the authors state: "A method of flood routing has been developed which depends only upon dependable stream flow records during a typical flood at various places on the main stream or on the tributaries whose flow is to be routed downstream. ... A hydrograph of inflow from the unmeasured area is determined. This flow and that at each of the upstream stations is then routed downstream. These routed floods show the extent to which each of the upper tributaries contribute to the flood peak at each downstream point. ... By this procedure, the benefits that may be derived from any proposed system of storage reservoirs or other plan of flood control can be definitely determined and the benefits can be weighed against the cost." It is the opinion of the authors that their method is easier to follow and that it requires less computations than some of the methods of flood routing in current use.

Another problem concerns the analysis of a flood hydrograph in an attempt to break it down into its component parts, that is, it is desired to know the contribution of surface runoff and of ground water. To do this, a study is being made of a small stream in North Carolina in cooperation with a Government agency. Because of the high porosity of the soil in the area studied, the surface runoff is that caught in the stream bed proper, so the observed difference in the shape of the summer and winter hydrographs, the former peaked, the latter more rounded, must be explained by the ground water contribution. An extensive analysis is being made, therefore, to isolate the contributions of runoffs involved.

The testing of ship models in the naval tank has been improved by placing a manifold sprinkler on the towing car. The jets from the manifold are directed downward into the water as the car traverses the length of the tank. The jets are then turned off and the model is towed the length of the tank. In this manner Reynolds number is increased by the added turbulence of the jets enabling a reduction to be made of the scale effects of model ship testing.

Case School of Applied Science

The Warner Hydraulic Laboratory is exceptionally well equipped for such a relatively small technical school. Equipment is available for studying the flow of water in pipes for heads up to 350 feet, and for studying flow in open channels. Although very little research is being conducted at the present time, the excellent work done at Case on the Muskingum Watershed Project is generally well know. In cooperation with the U. S. Engineers, model studies were made of eleven dams and their appurtenant structures during a period of eleven months, August 1934 to June 1935. Because of the accurate records kept of these tests and their costs, the profession was able to benefit greatly from this extensive test program.

Two hydraulic research projects are being planned: one will treat the movement of bed-load material by oscillatory waves, the other will study the effect of the shape of a bellmouth entrance on the discharge capacity of rectangular conduits of high dams. The first study has particular significance because it is believed that wave action in water fifth feet deep has certain dynamic effects on intakes to water supply systems, and thus certainly on bed-load movement.

Carnegie Institute of Technology

In cooperation with the U. S. Engineers and the Aluminum Company of America, the hydraulic laboratory has been devoted chiefly to model experiments of flood control structures for the Upper Ohio River Basin and of structures related to power development of streams. Although most of this work was of a routine nature, one phase of it has been

extensively developed. This relates to the study of cavitation in outlet conduits for high dams. During the experiments on the structures for the Upper Ohio River basin, attention was called to the severe effects of cavitation at the outlets of the Madden Dam in Panama, and since the outlets for the dams being tested in the laboratory were similar to those of the Madden Dam, apparatus was developed which would enable cavitation tests to be made of the model outlets. For an excellent description of these tests see "Cavitation in Outlet Conduits of High Dams" by H. A. Thomas and E. P. Schuleen, Proceedings A.S.C.E., November 1940.

At present, cavitation studies are being made of baffle piers which are sometimes placed at the toe of dams. Usually baffle piers are not placed in high velocity flow because of the fear that cavitation erosion will occur. This study, therefore, will attempt to either design a baffle pier free from cavitation or one which will be protected from cavitation effects. If this is successful, large savings can be made in the design of stilling pools by reducing their length and depth through the use of baffle piers.

S. Morgan Smith Company

The current work in the hydraulic laboratory is chiefly devoted to performance tests of adjustable blade axial-flow pumps. A 10-inch pump model is used for this study.

Although model tests were not being made on turbines at this time, the laboratory has made performance and cavitation tests for many large water power developments. One in particular was made of the Kaplan turbines for the Bonneville Dam. These turbines are the highest powered Kaplan units ever built, each rated at 60,000 hp under an effective head of 50 feet, with a generator capacity of 48,000 kva. It was intimated to the writer that although the Moody formula is used for stepping up model efficiency, the final result is necessarily altered for such large units.

An interesting inspection was made of the shops in which some of the Bonneville units were being fabricated. Their size is readily appreciated when one sees a propeller-type runner 23 feet 4 inches in

diameter, a turbine shaft 39-1/2 inches in diameter, and a speed ring whose largest outside diameter is 36 feet 6 inches having a water inlet height of 9 feet 6 inches.

Pennsylvania Water and Power Company
Philadelphia Electric Company

Inspection was made of the Safe Harbor and Holtwood powerplants of the Pennsylvania Water and Power Company, and of the Conowingo plant of the Philadelphia Electric Company on the Susquehanna River. The turbine testing laboratory at Holtwood has not been in operation this year.

University of Pennsylvania

The laboratory of the University of Pennsylvania is well known for the tests that have been made there on venturi meters by Professor Pardoe. Special studies are now being made on the effect of installation on venturi meter calibrations, and on the effect of the ratio of venturi throat diameter to the main line diameter.

In the study to determine the effect of installation, the venturi meter is placed at varying distances from reducers, elbows, valves, etc. It is also placed in bends which are in either horizontal or vertical planes. From this study it is possible to determine the best position of the venturi meter for producing a flat coefficient curve over most of the range of operation. One phase of the effect of installation is particularly interesting. When coefficients were plotted against Reynolds number, the dispersion was unsatisfactory, particularly in the lower values. It was finally discovered that the effect of ambient temperatures on the coefficients was appreciable. Accordingly, these tests were made with insulated meters resulting in practically no dispersion for lower values of Reynolds number.

By grouping together calibrations made on many sizes of meters, it has been possible to develop a relation which will take into account the effect of the ratio of throat to main line diameter. Thus, coefficients may be obtained for very large meters and for smaller meters whose ratio of throat to main line diameter varies from

0.3 to 0.75 by referring them to a coefficient curve for meters having a ratio of 0.5 and having the same throat diameter.

I. P. Morris Division, Baldwin Southwark Corporation

An inspection was made of the turbine testing laboratory and of the shops which were fabricating the Francis turbines for Boulder Dam.

A notable contribution to the study of cavitation in turbines has been made by the laboratory of the I. P. Morris Division. The progress of cavitation for decreasing values of sigma has been photographed in the model by means of a stroboscope and camera. The elbow portion of a draft tube is provided with an opening covered with Plexiglass conforming to the curved portion of the draft tube. Below this is a flat glass opening, the space between being filled with water to eliminate distortion of vision. The camera and an Edgerton stroboscope are located below this opening. The stroboscope is wired to a commutator geared to the turbine shaft, so that flashes are obtained at each revolution, thus giving a view of each separate blade rather than a composite effect. A full description of this apparatus and photographs of the model runner developing cavitation may be obtained from the paper "Cavitation of Hydraulic-Turbine Runners" by R. E. B. Sharp, Transactions A.S.M.E., October 1940.

New York City

During the period November 27-30 arrangements were made with New York University, Columbia University, Polytechnic Institute of Brooklyn, and other institutions for the writer to study their work in hydraulic research during the first week of December. Visits were made to the men of the Freeman Fund Committee and to the office of the American Society of Civil Engineers.

PROGRESS REPORT FOR DECEMBER 1940

Introduction

This is the third progress report submitted on the Freeman Scholarship of October 1940 to July 1941. The progress reports for October 1940 and November 1940 described some of the hydraulic research observed in the Midwest and East. This report for December 1940 continues the description of the work observed in the East to include New York City and New England.

Summary of Work Completed December 1940

In December 1940, observations were made of hydraulic research at the following places: New York City, December 1-11 (New York University, Columbia University, and the Polytechnic Institute of Brooklyn); University of Connecticut, December 12; and Massachusetts Institute of Technology, December 14-19.

Upon arrival in New York City on November 27, arrangements were made by November 30 with the institutions indicated, for the writer to observe their work in hydraulic research. This was done during the period December 1-11 as indicated. During this same period, visits were made to the Board of Water Supply and to the hydraulic sessions of the A.S.M.E. annual meeting.

On December 13, a stop was made at Providence, R. I., to meet Mr. Clarke Freeman.

Immediately after the stay at M.I.T. from December 14-19, a visit was made home to Rochester, N. Y., extending from December 20 to January 5.

New York University

The hydraulic laboratory of New York University has only been in operation four years, so a program of research has not been fully developed. It can be said, however, that the work in progress and being planned is of a practical nature, which cannot be said for some of the current work at other more established laboratories.

Experiments are being made on the free outfall from circular conduits. Such experiments have direct application to the sanitary engineering field relating to leaping weirs at outfall sewers. In this experiment, measurements are taken of the nappe at the outfall of 4-inch, 8-inch, and 12-inch horizontal transite pipes flowing partly full to determine the relation between the brink depth and critical depth, for a given discharge. Curves are then plotted enabling the discharge to be determined for a certain brink depth and size of pipe. The profile of the nappe at the outfall is so plotted that an analysis and comparison can be made for various diameters of pipe.

Another problem of a practical nature will be to investigate the laws governing the flow of water over side-channel weirs. A series of similar weirs will be studied by varying the width of channel and length and height of weir used. Extapolation of the results will therefore be possible and an attempt made to develop a general equation for the flow over the side-channel weirs.

Other experiments will include the investigation of Manning's "n" in varied flow, and the effect of spillway curvature (in the vertical plane) at the entrance to stilling pools on the hydraulic jump.

Columbia University

The philosophy governing the hydraulic research at Columbia University dictates that "too much stress cannot be placed upon the importance of striving for a basic knowledge of the physical aspects of flow." Accordingly, studies now being made on the hydraulics of the broad-crested weir, instead of dealing primarily with determining values of discharge coefficients, treat the physical characteristics of the flow upon such values. In these studies, observations are made of the flow forms over the weir and at its downstream end with and without aeration of the nappe; particular attention also being paid to the ratio of the depth of flow at the end of the weir to the critical depth. Submergence is also investigated to find out when it begins, what the water surface is like for different conditions of

submergence, and what the coefficients are and the effect of submergence upon them.

In addition to this study, an investigation has been made of the boundary layer in broad-crested weirs. A weir was chosen for the experimental work "to investigate the boundary layer as it occurs in a hydraulic structure under practical conditions of flow." Velocity measurements were taken along the weir as close to the bottom as possible, the upper limit of the boundary layer being taken at the point where the velocity becomes nearly uniform. By using these data, laws were established for this particular study in a manner similar to those laws of boundary layer thickness and velocity distribution developed by Karman.

Results of these studies on broad-crested weirs will be published sometime in 1941.

Polytechnic Institute of Brooklyn

An interesting problem is being studied in the hydraulic laboratory of the Polytechnic Institute of Brooklyn. It treats the flow of water in inclined pipes. Measurements are taken to determine the friction losses and velocity distribution in a 2-inch, 3-inch, and 4-inch brass pipe which may be rotated to any desired angle in a vertical plane. The data are then analyzed to see what effect the gravitational field will have on the flow. It is the opinion of many that the gravitational field will have no effect on the flow, so the results of this study should be of great interest. This experiment has been made possible by the J. Waldo Smith Fellowship in hydraulics of the American Society of Civil Engineers.

University of Connecticut

A new hydraulic laboratory has recently been completed at the University of Connecticut. Located in a wing of the new engineering building, it occupies one-half of the civil engineering laboratory which is 60 feet by 100 feet in plan. The water system in the hydraulic laboratory is of the recirculating type with two pumps,

one rated at 350 gpm and the other rated at 1,800 gpm, supplying water to a constant level tank located near the pump pit. The water flows from the constant level tank through a 4-inch and 10-inch pipe, both of which extend to the end of the laboratory. Weighing and volumetric tanks will be used for calibrating the measuring devices in the laboratory.

The laboratory offers many opportunities for students to test and calibrate the new equipment and to develop research problems. Additional facilities are planned whereby commercial testing and advanced research problems can be readily undertaken.

Massachusetts Institute of Technology

An extensive test program is being carried out on the variation of the friction factor "f" with Reynolds number for uniform flow in open channels. Three years' work has been completed and two years' additional testing is planned before the results will be fully analyzed prior to publication.

In studying the variation of "f" with R for uniform flow in open channels for R up to 65,000, tests are made in the laboratory in channels having variable width, slope, and roughness. From this procedure, data may be obtained to determine the relation between "f" and R and how it changes with the width, slope, and roughness of channel. During these experiments, attempts have been made to determine some other factor than the hydraulic radius for use in the expression for Reynolds number. So far no suitable substitute has been found. In completing this study, it is planned to use channels with cross-sections other than rectangular.

Several other interesting experiments and their results were studied, but because they are of a confidential nature, no mention will be made of them at this time.

PROGRESS REPORT FOR JANUARY 1941

Introduction

This is the fourth progress report submitted on the Freeman Scholarship of October 1940 to July 1941. The first three reports covering October, November, and December described some of the hydraulic research observed in the Midwest and East. This report for January 1941 completes the description of observations made in the East and describes some of the research studied in the South.

Summary of Work Completed January 1941

A visit home to Rochester, New York, for the Christmas Holidays extended from December 20 to January 5. Travel was resumed on January 6 with visits being made to the following places: Cornell University, Ithaca, New York, January 6-7; Worcester Polytechnic Institute, Alden Hydraulic Laboratory, Holden, Massachusetts, January 8-13; New York City for the Annual Meeting of the Society, January 14-20; National Hydraulic Laboratory, National Bureau of Standards, Washington, D. C.; Beach Erosion Board, War Department; and David W. Taylor Model Basin, Navy Department, Washington, D. C., January 22-25; Newport News Shipbuilding and Drydock Co., Newport News, Virginia, January 27-29; Claytor Dam on the New River, Pulaski County, Virginia, January 30; Spartanburg Outdoor Hydraulic Laboratory, Soil Conservation Service, Spartanburg, South Carolina, January 31.

Cornell University

Current research in hydraulics at Cornell University deals primarily with a study of the shape factor on open channel flow. No data were available on this work for study. Model studies have been made for various private organizations and for the U. S. Engineers in connection with flood control in New York State.

Worcester Polytechnic Institute
Alden Hydraulic Laboratory

Work in hydraulics at the Alden Hydraulic Laboratory is primarily designed for undergraduate instruction. As a result, only a limited amount of advanced research is done by graduate students from, perhaps, the academic point of view. Fortunately, however, as the writer prefers to call it, "practical" hydraulic research has been ably carried out by Messrs. Allen, Hooper, and Hubbard of the laboratory staff. This type of work is divided into model studies of hydraulic structures for commercial organizations, and into investigations relating to flow of water in pipe lines, the latter involving the Allen salt velocity method of measuring discharge, and the errors in pitot tubes.

In regard to commercial model studies, it is interesting to note that tests have been made at this laboratory for organizations located at a considerable distance from Holden, even though other hydraulic laboratories are located nearer to the work involved. For example, model tests have been made for the Pennsylvania Water and Power Co., the Philadelphia Electric Co., and the New York Board of Water Supply. Nearer by, tests have been made for the Boston Metropolitan District Water Supply, and at the present time for the Providence office of the U. S. Engineers.

In investigations on the flow of water in pipe lines, tests are continually being made to improve the salt-velocity method. This involves reducing the length of test section required and the length between pop-valves and the first electrode. Improvements will also be made in the electrodes themselves. Recent studies of salt-velocity measurements at low velocities in pipes reveal that, for flow in a long straight pipe and for the velocities lower than those normally found in practice, there exists a critical mixing velocity below which good mixing of the injected brine does not occur ("Salt-Velocity Measurements at Low Velocities in Pipes," by L. J. Hooper, presented at the spring meeting, Worcester, Mass., May 1940, A.S.M.E.). From

a curve of error in salt-velocity results versus mean pipe velocity, there is a break in the error curve showing a departure from the purely experimental type of error. The velocity at this break is the critical mixing velocity. It was also found in this study that the accuracy of the method when applied to a vertical pipe is not affected by gravity as long as proper mising is obtained.

The other study relating to the flow of water in pipe lines treats the errors of pitot tubes ("Investigations of Errors of Pitot Tubes," by C. W. Hubbard, Trans., A.S.M.E., August 1939). It is generally known that some pitot tubes do not have the same coefficient for all conditions of flow, so it was the purpose of this investigation to determine the factors involved. Tests were made first on an 84-foot rotating boom in still water and later in pipes of 12, 40, and 78 inches in diameter. After investigating angularity, pulsation, and the dynamic effect of the flow, it was decided that none of these is sufficiently great to cause an appreciable error. However, it was found that even though the support rod of the pitot tube does not affect the reading of the impact orifice, it does affect the reading of the wall piezometers if they are used for measuring the pressure-head, and the tip piezometers used with pitot-static tubes. It was also realized that if the pressure is not constant across a pipe, the wall piezometer does not measure the pressure which exists at the tip of the pitot tube. It was concluded, therefore, that the impact tip registered the true dynamic head whether the flow was smooth or turbulent, and that the pressure piezometers were in error. Accordingly, since the pressure piezometers (tip piezometers) are an integral part of a pitot-static tube, this instrument was considered more reliable than a simple pitot tube with separate wall piezometers and it is capable of being calibrated and used under different conditions of flow if necessary.

More tests will be made to observe the effects of turbulence on pitot tube measurements, but more specifically to study the relation between angularity of flow and turbulence.

Before leaving Holden, an inspection was made of the Ware River Intake Works of the Boston Metropolitan District Water Supply.

National Hydraulic Laboratory, National Bureau of Standards

The National Hydraulic Laboratory was founded for the purpose of obtaining fundamental data from hydraulic research which would be useful to the engineering profession as a whole. It was the dream of the late John R. Freeman to have such a laboratory and he worked hard to bring it into being. It is debatable, however, whether the laboratory measured up to his specifications, and it is to be regretted today that a limited budget greatly curtails the work being done. It is quite evident, moreover, that each agency of the Government and private organizations are doing their own research peculiar to their type of work, even though some of it is fundamental and could perhaps be done in the National Hydraulic Laboratory. Nevertheless, many interesting and important problems are being studied at the laboratory and the results so far published have been of value to the profession.

The work done at this laboratory may be divided into two groups: A study of existing theories with the purpose of advancing them by careful analysis and some experimentation, and a study of special problems yielding more directly to an immediate practical use.

An example of the first type of work is the current study being made of the motion of flood waves and other waves of translation in open channels. A series of papers is planned, the first of which has recently been published, "Mathematical Theory of Irrotational Translation Waves," by Keulegan and Patterson, Journal of Research of the National Bureau of Standards, RP1272, January 1940. This paper treats waves for which the forces of fluid friction are negligible with respect to the inertia and gravitational forces. Other papers in this series are in various stages of completion and will deal with the effect of turbulence and channel slope and configuration on the motion of translation waves; the theory of quasipermanent regime and the

methods of prediction of flood waves; and finally, the recent advance in the problem of the deformation of an intumescence.

Another example of the first type of study is a most interesting one and one which enlists the combined efforts of many engineers and Government agencies. This is an investigation of density currents, these being defined as "the movement, without loss of identity by mixing at the bounding surfaces, of a stream of fluid under, through, or over a body of fluid, the density of which differs from that of the current, the density difference being a function of the differences in temperature, salt content, and/or silt content of the two bodies of fluid." A bottom current is a "density current that flows under the adjacent body of fluid."

The following phenomena involving density currents will be studied:

1. Passage of silt-laden water through reservoirs.
2. The failure of the water of a tributary to mix with the water of the main stream when the water of the tributary is of different quality and/or silt content from that of the main stream.
3. The gliding of fresh water over salt water, or salt water under fresh water.
4. Possible cause of the formation of the submarine canyons that have been discovered recently on the outer edges of the continental shelves by density currents of silt-charged water.
5. Passage of currents of warm air over pockets of cold air.

To attack these problems, a committee of the National Research Council with Mr. Herbert N. Eaton, director of the National Hydraulic Laboratory, as chairman, was formed in 1937. This committee is known as the "Interdivisional Committee on Density Currents." Since the phenomena of density currents concern not only engineers, but also geologists, chemists, and physicists, all these professions are represented on the committee.

At the present time, the main efforts are being concentrated on the flow of silt-laden currents of water into reservoirs. Through the cooperation of the Government agencies involved, measurements are being taken on Lake Mead and Elephant Butte Reservoir. These measurements include temperature-depth, velocity, salinity, turbidity, conductivity, and elevation of the top of the silty layer. It is interesting to note that the location of the silty layer is found not only by sampling the water, but also by its temperature, since the temperature of the silty layer is higher than that of the clear water immediately above. This fact is also useful in determining the velocity of a silt-laden density current through a reservoir. In Elephant Butte Reservoir, two thermometers have been placed, one at the outlet of the dam and the other some known distance upstream. By noting the time elapsed between the temperature rise upstream and a similar rise downstream, the mean velocity of the silt layer passing downstream may be determined.

The National Hydraulic Laboratory enters into the problem by analyzing density currents experimentally. Three experimental closed channels, two of them geometrically similar, having rectangular cross-sections and of different sizes have been constructed. In these channels, clear tap water was made to flow over a pool of aqueous salt solution, the density of which varied from 1.02 to 1.20. The velocity of the flow of water was increased until the waves formed at the boundary surface began to break so that mixing occurred between the two liquids. This velocity was recorded as the critical velocity.

By the method of dimensional analysis, a criterion for the critical velocity was developed in the form:

$$K = \frac{g \cdot \nu}{U^3} \cdot \frac{\Delta \rho}{\rho}$$

where K = a dimensionless coefficient.

ν = kinematic viscosity of the moving liquid.

ρ = density of the moving liquid.

$\Delta\rho$ = difference in density of the two liquids.

g = acceleration due to gravity.

U = mean velocity of the moving liquid when mixing first occurs.

The values of K determined for different densities of the heavier liquid and with different channels were found to be in good agreement. An average value for $K^{1/3}$ appears to be about 0.155. Scale effects are as yet undetermined either from theory or experiment. It was also difficult to explain why in geometrically similar channels the rate and development of mixing was rapid in the smaller channel, while in the larger channel it was slower, especially for larger values of U.

In the second type of work done at the laboratory, that is, a study of special problems yielding more directly to an immediate use, the more recent work involves the investigation of artificial stream-control structures ("Investigation of Artificial Stream Control Structures," National Bureau of Standards, May 1939). These are structures used by the U. S. Geological Survey on nonnavigable streams in conjunction with recording water-level gages to obtain a continuous record of the discharge of the stream. As usually constructed, they are low concrete structures extending across the stream from bank to bank, sometimes having a crest that slopes from each bank to a low point or a deep notch at or near the center of the channel, and sometimes with a crest that extends horizontally across the channel.

Since there are many different designs of these structures, the tests at the laboratory were made to eliminate those with undesirable features, and to present data which would aid in the selection of one or more standardized stream control structures. To do this, experiments were made on models and full-size structures to determine the rating curve for each with free and submerged overfall, and the effect on the free overfall rating for various depths of approach (filling of the stream channel upstream).

A similar study is now in progress for the U. S. Forestry Department, except in this study a weir adaptable to conditions desired by the Forestry Department will be developed, so that a smooth rating

curve will be obtained for any conditions of flow or installation.

Other problems now being studied in the laboratory include an investigation of the laws of similitude as affecting model studies, the transportation of sand-water mixtures in pipes, and aging tests on pipes.

Beach Erosion Board, War Department

To those familiar with the study of waves and their erosive action on beaches, it is fully realized that since the existing theories are conflicting, much work must be done with waves in the laboratory and in nature to check these theories, and at the same time to obtain a clearer understanding of the various phenomena involved. Once this has been done, the problem of designing protective works will at least be simplified.

The Beach Erosion Board has been charged by law with the duty of making investigations in the field of shore protection and publishing facts of engineering value ("Recent Experimentation on Wave Action," by Major A. C. Lieber, Jr., Beach Erosion Board, April 1940). In order to do this, it is first necessary to study the basic problems of wave action in a laboratory. In this way the elements of the problem may be controlled and studied separately, and undesirable or unimportant factors may be eliminated when necessary.

The wave tank used by the Beach Erosion Board is 85 feet by 14 feet in plan, with a vertical plunger-type wave machine at one end, and a sloping wave absorber at the opposite end. The experimental work now being conducted is primarily for checking the existing theory for use in evaluating wave actions in nature. To improve the experimental work, an oscillograph is being developed for simultaneously determining all wave characteristics, including profile and direction. In addition, a long narrow steel channel is being built with glass sidewalls placed at the far end for observing wave action on the "beach" installed for testing. An improved wave machine will be placed at the other end of the channel, being so designed that the waves generated will be as near as possible to the correct oscillatory motion at the start.

David W. Taylor Model Basin, Navy Department

This recently completed model basin is 1,200 feet long. There are two basins provided, one for slow speed and one for high speed testing. During the time of inspection, work was rapidly nearing completion on the large towing car for the slow speed tests. This car will tow ship models up to twenty feet in length at a speed of about fifteen knots per hour. Because of national defense measures, no detailed information was obtained on types of ships to be tested, nor were any specific questions asked about methods of testing or other details.

In the administration building there are two large experimental setups for conducting cavitation tests on ship propellers, with stroboscopes installed for observing the cavitation phenomenon during testing. A materials testing laboratory and a shop for constructing ship models are also provided.

Newport News Shipbuilding and Drydock Company

The most interesting part of this visit was an inspection of the shipyards. The national defense program has turned the shops and yards into an extremely active place. Many new ships were seen under construction including the new aircraft carrier "Hornet" and several freighters for the Maritime Commission. Several older ships were in drydock being reserviced, presumably for the British.

The work in the hydraulics laboratory has recently been devoted more to model tests of ships than to hydraulic machinery. Accordingly, most of the time was spent in learning something about the technique of testing ship models. This type of model study is quite involved and requires considerable study and experience before a good understanding is obtained of the many steps required from testing the model to the final trial runs of the prototype ship.

Relative to hydraulic machinery, tests were just being started on models of relief valves or energy dissipators used for by-passing the flow in hydro plants in the case of sudden gate closure. The intake to the valves is made in the scroll case and the discharge from the valve is led into the draft tube.

Claytor Dam, New River, Pulaski County, Virginia

While in New York, during the Annual Meeting of the Society, it was learned that the Claytor Dam had passed a flood of 200,000 cfs last August, and that after the flood, severe cavitation erosion was noticed on the baffles on the apron at the toe of the dam. The writer was particularly interested in this erosion because of his experience in the laboratory with spillways similar to the Claytor Dam. Accordingly, through the kindness of Col. F. W. Scheidenhelm, M. Am. Soc. C. E., a pass was obtained for an inspection of the dam and appurtenant structures.

The Claytor Dam is on the New River, Pulaski County, Virginia. It is of the gravity type with a maximum height of 123 feet from rock line to top of spillway gates. The overall length is approximately one thousand one hundred and fifth feet. The spillway section at the center of the structure is of the ogee type about five hundred and twenty feet long with nine spillway gates 50 feet wide by 28 feet 6 inches high. The powerhouse on the right bank is equipped with four Francis-type turbines with a combined capacity of 104,000 hp or 83,332 kva.

The apron proper extends horizontally 54 feet downstream from the tow of the ogee section. At the end of the apron, a large baffle has been placed to reduce erosion of the river bed. The baffle consists of rectangular teeth with a curved upstream face, butting against a stepped sill. The sill is provided with venturi-like openings, the openings and teeth alternating.

The flood which occurred on August 14, 1940, was the first one passed by the spillway since its completion about two years ago. This flood was recorded at just above 200,000 cfs, the spillway being designed for 250,000 cfs. The maximum recorded flood up to that time had been 170,000 cfs (estimated) in July 1916.

The erosion to the teeth on the apron was evidently due to cavitation, since the full force of the jet impinged on the teeth and flowed between them with no tailwater above, and since the flow did not persist long enough to cause abrasion of the amount observed. The maximum

velocity at the apron was approximately sixty to seventy feet per second.

Inspection of the teeth revealed a pitted area on the longitudinal faces immediately downstream from the "leading edges" of the teeth. This is where cavitation erosion is normally found. Within these pitted areas, holes had been "dug" as deep as 4 inches. Many deep cracks were also observed in this area giving the appearance of tension cracks, as if a force had been applied upstream along the top of the teeth. Cracks were also evident along the extreme downstream face of the perforated sill as if a vertical force had been applied causing diagonal cracks to develop at the upper corners of the openings in the sill. Sufficient reinforcement had been placed in the baffle system based on existing knowledge of the stresses involved.

Spartanburg Outdoor Hydraulic Laboratory, Soil Conservation Service

The hydraulic studies being made at this laboratory are quite unique and present problems to the experimenter which are usually not encountered in normal type of hydraulic laboratory. This is due to the type of structures used in the conservation of soil and water and the resulting hydraulic problems to be solved in their design. Instead of dealing with usual flow of water in lined structures or pipes, for example, a large amount of work involves the design of channels with vegetal linings. These may include terrace systems, meadow strips, and other broad, shallow channels. Accordingly, the Spartanburg Outdoor Hydraulic Laboratory has been developed to obtain these data, and to conduct tests on the capacity of notches and tests on drop inlet spillways, drop boxes, culverts, transitions and chutes ("Spartanburg Outdoor Hydraulic Laboratory," by H. L. Cook, Civil Engineering, October 1938).

In testing channels with vegetal linings, two problems are considered: The carrying capacity of such channels, and the resistance to erosion offered by the vegetation used. In these tests various sizes of trapezoidal channels are laid on different slopes and planted with grasses common to South Carolina and vicinity. Each channel is

tested over a wide range of discharges and the values of Manning's and Kutter's roughness coefficients determined. The problem of erosion is likewise studied during these tests by determining the maximum allowable velocity before erosion develops in the underlying soil, which in these experiments is Cecil clay. One group of these tests indicates that a trapezoidal channel on a 30-percent slope lined with solid Bermuda sod, and with a bottom width of one foot and side slopes of 1:1, has a probable safe velocity of 8 feet per second and a Kutter's "n" of 0.035.

Comparative tests are made between the types of grasses with the grass long, cut short, dormant, and at different periods of growth. It is interesting to note that the resistance or retarding effect of vegetation decreases with the depth of flow in a channel because of a "shingled" effect produced by the plants flattening against the channel bed in the direction of flow. If the stems of the vegetation are woody, this effect is not as pronounced, so the retardance effect does not decrease as rapidly with increase in depth of flow.

The design of structures to prevent erosion in terraces, gullies and below drop structures is governed by the ease and economy of construction. This is not an easy problem to solve, as anyone knows who is familiar with the design of energy dissipators. U-drops and U-type flumes have been used extensively, but more work must be done to improve the stilling pools below these structures. This will be done at Spartanburg in connection with similar studies being made by the Soil Conservation Service at the University of Minnesota and at the California Institute of Technology.

A new laboratory is now being constructed at Stillwater, Oklahoma, similar to but larger than the Spartanburg laboratory. Studies will be made in Oklahoma similar to those in South Carolina, but testing grasses and other vegetation common to the West.

PROGRESS REPORT FOR FEBRUARY AND MARCH 1941

Introduction

This report, combining the work of February and March, is the fifth progress report submitted on the Freeman Scholarship of October 1940 to July 1941. It completes the observation made in the South, the previous reports having described some of the hydraulic research observed in the Mid-West, along the Atlantic Seaboard, and in part of the South.

Summary of Work Completed February and March 1941

Visits were made to the following organizations: Enoree River Sediment Load Station, Soil Conservation Service, Greenville, S. C., February 1; Tennessee Valley Authority, Knoxville and Norris, Tennessee, February 2-9; inspection of hydraulic structures of the TVA from Norris Dam to Pickwick Landing Dam, February 10-12; U. S. Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Mississippi, February 13 to March 18.

Upon completion of the work at Vicksburg, a visit was made home to Rochester, N. Y., from March 21 to April 5.

Enoree River Sediment Station, Soil Conservation Service

This experiment station was developed to obtain quantitative measurement of the total sediment load of a stream actually large enough to be representative of rivers in general. Investigations of sediment load, especially bed load, in laboratory flumes is not fully adequate nor as yet of great practical value, hence it is encouraging that now, from a study of an actual stream, a better understanding will be obtained of the suspended and bed load movement.

A project of this type will require considerable time and study before any definite results can be obtained. In an effort to include all aspects of the problem, investigations will be made on the Enoree River for determining: (1) The relation between the character

and amount of sediment load to the hydraulic and physical characteristics of the stream; (2) the relation between the properties of the watershed and the amount and composition of the total sediment load; (3) a practical and simple method of estimating the total sediment load of the stream; and (4) methods of controlling the movement of sediment in flowing water.

To accomplish this study, a carefully selected reach of approximately one hundred feet of the Enoree River has been paved with concrete and straightened by means of vertical retaining walls. This forms a control structure suitable for measuring the amount of bed load and suspended load. The bed load moving along the bottom of the lower end of the control structure passes across fourteen bottom openings separated by vertical vane walls. Each opening connects to a header below, which is connected to a pump on the right bank. This pump removes the bed load hydraulically as it flows over the openings in the bottom of the control structure and places it in a settling tank from which it is eventually removed for weighing and analyzing. The suspended load is taken by samplers simultaneously with the pumped bed load at a point immediately downstream from the bottom openings. This material is also carefully analyzed.

In addition to the control structure and its many appurtenances, a concrete flume is used for studying the transportation of various sand mixtures for any hydraulic conditions to supplement the observations made on the river. A well-equipped laboratory is also provided for making analyses of all sediment samples.

Continuous records have been taken only since January 1939, but results so far indicate that definite relations exist between sediment load and discharge for the coarser materials but not for the finer materials.

Tennessee Valley Authority

The hydraulic laboratory of the Tennessee Valley Authority in Norris, Tennessee, is devoted to the solution of hydraulic problems

pertaining to the design of structures used in navigation and power development of the Tennessee River and its tributaries. This laboratory and similar ones, such as the U. S. Waterways Experiment Station and the Bureau of Reclamation Laboratory in Denver, is by necessity limited to practical experiments as compared to the academic research usually found at universities.

All the more modern structures in the Tennessee Valley have been tested by models to check their design for safety of performance and economy. The more recent model studies include those made for the Cherokee Dam on the Holston River and the Fort Loudon Dam on the Tennessee River.

Dams of the type employed in navigable rivers like the Tennessee and Mississippi Rivers present problems not usually found at dams used primarily for flood control and power development. One problem, in particular, concerns the energy dissipation of the water immediately below the spillway. At dams of the first type, the hydraulic jump is not obtainable because of excessive tailwater created by navigation requirements; in the second type, however, excellent dissipation is usually obtained by the hydraulic jump. As a result, the laboratory tests of the type found at Norris are made more difficult and considerable resourcefulness is required to develop a satisfactory solution.

The problem of aerating nappes, on the other hand, is usually prevalent at both types of structures mentioned. The Norris laboratory had this problem to solve in connection with the double-leaf slide gates used on the Chickamauga, Guntersville and Pickwick Landing Dams. Since water may flow between the gate leaves or over the lower leaf, a region of subatmospheric pressure will form under the nappe between adjacent piers and thus increase the total pressure acting on the gate. This condition is instantly relieved by aeration of the nappe, but the problem is how much air will be required. To solve this, tests were made of the air demand for a large weir nappe in the Norris laboratory. Then by dimensional analysis, a curve was obtained giving the required amount of air based on the allowable reduction of

pressure beneath the nappe and the head on the gate. Once the amount of air required is known, the size of vent may be determined. Of particular interest is the fact that field tests have checked the laboratory curve quite closely.

In connection with the design of sluice outlets for high dams, it is necessary to investigate whether cavitation will occur in the outlets due either to their shape or due to operating conditions or both. At present there are two methods of testing models of outlets for cavitation. One method reduces the atmospheric pressure in the model according to the scale ratio, if possible, thereby causing cavitation in the model. The other method tests the model outlet under normal atmospheric pressure and bases its prediction of cavitation in the prototype on the conversion of model pressures to prototype pressures. The latter method, which is used in the Norris Laboratory, is much simpler and is more generally used elsewhere.

After a study had been made of the work at the Norris Laboratory, an inspection was made of the Tennessee Valley from Norris Dam to Pickwick Landing Dam. The following dams were included: Norris, Hiwassee, Chickamauga, Hales Bar, Guntersville, Wheeler, Wilson and Pickwick Landing. Particular attention was paid to the Kaplan turbines, spillways and navigation locks. Additional turbines are being installed at the last three structures listed in connection with increased power demand for national defense.

U. S. Waterways Experiment Station

During the visit to the Experiment Station, observations were made of the work in the laboratory, and field trips were taken to inspect some of the work on the Mississippi River. The paper attached hereto containing a description of the more interesting work observed is submitted for possible publication in "Civil Engineering." (This paper is on file at the American Society of Civil Engineers, New York City, and in the Personnel Section, Bureau of Reclamation, Denver, Colorado.)

PROGRESS REPORT FOR APRIL 1941

Introduction

This report for April is the sixth progress report submitted on the Freeman Scholarship of October 1940 to July 1941. The previous reports have briefly described some of the hydraulic research observed in the Midwest, East, and South. This report and succeeding ones will treat the inspections made of numerous irrigation projects of the Bureau of Reclamation and flood control developments of the U. S. Engineer Department in the Western States. The May report will also include observations made at California Institute of Technology and the University of California.

Summary of Work Completed April 1941

A visit home at Rochester, N. Y., starting March 21 was terminated April 3. Travel was then resumed to make observations at the following places: Bureau of Reclamation, Denver, Colorado, April 7-12; Salt River Valley Water Users' Association, Phoenix, Arizona, April 14-21; Bureau of Reclamation at Boulder Dam, April 23-24; at Parker Dam, April 25-26; and at Yuma, Arizona, April 27-May 1.

Bureau of Reclamation at Denver, Colorado

No detailed study was made of the work in progress in the hydraulic laboratory. This will be done and will be included in the final report after the writer has returned to his position in the laboratory upon completion of the scholarship June 30. During the visit in Denver, the itinerary covering projects of the Bureau of Reclamation was further developed and revised where necessary.

Salt River Valley Water Users' Association

The Salt River Project is one of the first large irrigation projects developed by the U. S. Reclamation Service (now Bureau of Reclamation). It consists of 240,000 acres of highly developed farm lands around Phoenix, situated in a broad, flat valley crossed by the

Salt River. In the mountainous country northeast of Phoenix, five dams have been built on the Salt River and one of the Verde River, a tributary. The structures on the Salt River starting with the farthest one upstream (about ninety miles from Phoenix) are: Roosevelt Dam, built 1905-11; Horse Mesa Dam, built 1924-27; Mormon Flat Dam, built 1923-25; Stewart Mountain Dam, built 1928-30; and Granite Reef Dam, built 1906-08. All of these structures are over 200 feet high and are concrete gravity-arch type, except Roosevelt Dam which is of masonry, and the diversion dam which is merely a low, long concrete weir. Bartlett Dam on the Verde River is a concrete multiple-arch dam 273 feet high, the highest of its kind in the world. The total storage capacity of this system is 1,954,000 acre-feet. Power is developed at all structures except at Bartlett and Granite Reef.

Although this project has been operated by the Water Users' Association since 1917, the Bureau of Reclamation has done nearly all the major design and construction work for the Association. It is in this connection that the writer helped conduct model tests of the spillways of the newer structures. Accordingly, it was most interesting to inspect the prototypes and particularly so because all spillways were discharging, some for the first time. This condition of excess water is quite a contrast to last year when only 165,000 acre-feet were in storage in the entire system, but now nearly two million acre-feet are available.

As is usually the case, only a small fraction of the maximum design flow was passing through the spillways, yet a good opportunity was had to observe the action of the flow at the regulating gates, in the spillways, and in the river immediately below. Observations were also made of the needle valves and butterfly valves which were discharging full capacity at some of the structures. The types of spillways seen included super-elevated ones curving in plan from the gate structures at the abutment of the dam toward the river channel so as to allow the water to plunge into the river directly, and concrete-lined tunnels from the reservoir through the canyon wall discharging at a lower elevation into the river below the dam.

The Boulder Canyon Project

The purpose of this project is defined in the Boulder Canyon Project Act of 1928 as follows: "controlling the floods, improving navigation and regulating the flow of the Colorado River, providing for storage and for the delivery of the stored waters thereof for reclamation of public lands and other beneficial uses exclusively within the United States, and for the generation of electrical energy as a means of making the project herein authorized a self-supporting and financially solvent undertaking."

Included in the project are parts of Nevada, Arizona, and California. Starting at Boulder Dam, where 30,500,000 acre-feet will ultimately be stored, the next structure built on the Colorado River was Parker Dam about 115 miles below Boulder Dam and 20 miles north of Parker, Arizona. Here 717,000 acre-feet of water will be stored with additional power for the project soon to be generated. Continuing downstream about 90 miles to a point 18 miles northeast of Yuma, Arizona, the Imperial Dam was built to provide a headworks for the All-American Canal and the Gila Canal and their desilting works on the California and Arizona side of the Colorado River, respectively. The All-American Canal flows westward through Southern California, furnishing water to the Yuma Project in Arizona, and to the Imperial Valley which heretofore had obtained its water from the Imperial Canal which flows for the most part through Mexico. The Gila Canal flows south on the Arizona side of the Colorado River to furnish water east and south of Yuma. The All-American Canal System will ultimately bring under irrigation 1,000,000 acres and the Gila Canal 585,000 acres.

An inspection was made of this entire project during the period April 14-May 1. The following structures were studied: Boulder Dam, Parker Dam and Pumping Plant for the Colorado River Aqueduct, Imperial Dam and Desilting Works, Laguna Dam (old diversion dam below the Imperial Dam), All-American Canal and Coachella Branch, and Yuma Canal. Many appurtenant structures in the canal systems were studied, including: wash inlets, wash overchutes, wash siphons, automatic wasteways, power

drops, check drops, headworks, desilting basins, and others. Most of these structures were tested by models in the hydraulic laboratory in Denver, the writer conducting model studies of the four power drops in the All-American Canal between Yuma, Arizona and Calexico, California.

Besides finding it exceedingly interesting to travel over these large irrigation projects from storage reservoirs to small laterals, and to see structures which had been tested by models in the laboratory, it was a matter of great satisfaction to see at first hand how water will transform desert lands to productive farm lands. Such a sight should be seen by those who do not know what it means to depend on irrigation.

PROGRESS REPORT FOR MAY 1941

Six progress reports have been submitted to date on the Freeman Scholarship of October 1940 to July 1941. These reports have described briefly some of the hydraulic research and hydraulic structures observed in the Midwest, East, South, and Southwest. This report for May, the seventh report, presents some of the work along the Pacific Coast, with particular reference to irrigation projects of the Bureau of Reclamation.

Summary of Work Completed May 1941

After an inspection of the area around Yuma, Arizona, from April 27 to May 1 (see progress report for April), visits were made to the following places: U. S. Engineer Department, Corps of Engineers, Los Angeles, California; and California Institute of Technology, Pasadena, California, May 2-14; Bureau of Reclamation, Friant Dam, California, May 15; University of California, Berkeley, California, May 16-20; Bureau of Reclamation, Contra Costa Canal, Antioch, California, May 21; Bureau of Reclamation irrigation project at Orland, California, May 22; Bureau of Reclamation, Shasta Dam, California, May 23; Bureau of Reclamation irrigation projects at Klamath Falls, Oregon, May 24, and at Ontario, Oregon, May 26-29; Portland, Oregon, and Bonneville Dam and hydraulic laboratory, May 31-June 3.

U. S. Engineer Department, Los Angeles, California

To obtain some understanding of the problems involved in controlling floods in the Los Angeles area, considerable time was spent in the U. S. Engineer office and in the field inspecting the work completed and in progress.

The flood control problem is derived from the fact that the Los Angeles metropolitan area is traversed by streams originating in mountain ranges close by and which nearly encircle this densely populated area. The runoff is carried into the Pacific Ocean chiefly by the channels of the Los Angeles River, San Gabriel River, and the Santa Anna River. These rivers, and in particular their many tributaries, have such steep gradients that super-critical velocities are developed. As a result, floods carrying tremendous amounts of debris reach the ocean in less than eight hours and inundate valuable citrus lands and residential and business areas.

To reduce the drainage and danger from floods, the U. S. Engineer Department is building flood control basins on the main streams and tributaries; debris basins on the tributaries; and improving channel capacities by placing concrete lining, straightening, and by streamlining bridge piers and other obstructions. As can readily be realized, such a program involves a great amount of engineering skill and research.

Seven major flood control basins are proposed, three of which have been completed. These basins are formed by long earth-fill dams provided with outlet works and emergency spillway. By proper regulation during floods, each basin will not only reduce the peak, but will also continually release water into channels below in amounts which will not tax their capacity to safely carry off the water released. The smaller debris basins will trap material carried by the tributary streams to prevent clogging of the channels below. In some cases it is planned to provide the trapping of debris in the large flood control basins.

In connection with this program, an outdoor hydraulic laboratory has been developed for testing the design of flood spillways, outlet works, and channel improvements. A most convincing demonstration of the effect of streamlining piers to increase channel capacity may be seen in the 1:50 scale model of a reach of the Los Angeles River flowing through the northern part of the city.

California Institute of Technology

The hydraulic laboratory at this institution is divided into the hydraulic machinery and the hydraulic structures laboratory. The former is probably best known for the tests made on the Colorado River Aqueduct pumps and on the pumps for the Grand Coulee Dam. It is generally believed that this laboratory is the best of its kind in the United States. Precision instruments are used for measuring all test elements to within one-tenth of one percent. Heads as high as 1,000 feet may be obtained.

The hydraulic structures laboratory is operated in cooperation with the Soil Conservation Service. The more recent research work involves the development of a standard design for erosion control drop structures, the transportation of suspended sediment by water, and the investigation of turbulence mixing as a factor in the transportation of sediment in open channel flow. Once again we find research being conducted on the transportation of sediment in streams. The need for obtaining fundamental knowledge of this phenomenon is being recognized more and more, especially now that so many dams have been built on our major rivers.

Associated more or less with the study of sediment transportation is the problem of density currents in reservoirs. The laboratory has developed a simple but clever apparatus for demonstrating the movement of density currents. A narrow glass-sided flume is blocked at one end with a model dam. In the reservoir behind this dam, a sloping false floor has been placed to represent the bottom of a reservoir. By temporarily blocking off a small portion of the far end of the

reservoir until the density of this water has been changed by adding crushed ice, sugar, salt, or a silt suspension, and by adding dye, it is readily possible to observe the underflow or overflow of a density current downstream to the dam, see it flow up the face of the dam and then return upstream. It is interesting to note that very little, if any, mixing occurs between the density current and the clear water above. It is also demonstrated how outlets properly placed and operated in the lower part of the dam may remove this density flow, or actually a sublayer of suspended silt in the case of a reservoir in nature. This removal of a silt layer has good possibilities of use in the field.

Bureau of Reclamation Irrigation Projects

No attempt will be made in this report to describe all the features observed in the inspection made of the various irrigation projects. It is planned that a comprehensive description of these will be made in the final report in order to show the many factors involved, particularly the hydraulic structures, their design features, performance in operation, and comparison to model studies where possible.

In general, an inspection of an irrigation project is made with an engineer familar with the project, first to the storage reservoirs where a study can be made of the dams and their appurtenant works; then downstream following the main canal, subcanals, laterals, and smaller ditches. In the course of this part of the inspection, many small hydraulic structures are observed and photographed. These include diversion dams, turnouts, wasteways, vertical and inclined drop structures, check drops, and such measuring devices as weir boxes and submerged orifices. The performance of all these structures is noted and additional information on them is obtained from the engineer. In this way it is possible to evaluate the present design practices and to formulate possible revisions.

Such experience will be invaluable to the writer not only in understanding what the problems are in the field, but also in better understanding the problems treated in the hydraulic structures laboratory in Denver.

University of California

The academic year had been completed by the time a visit was made to the laboratory of this university. Accordingly, it was impossible to obtain any detailed information. It is planned that contact will be made at a later date during preparation of the final report.

Bonneville Dam and Hydraulic Laboratory

An attempt was made to contact Mr. J. C. Stevens, M. Am. Soc. C. E., in Portland, but it was unsuccessful. A trip was then made to inspect the Bonneville Dam and hydraulic laboratory. The powerhouse at the dam is now being enlarged to its full capacity of ten units. Whereas it was argued only a few years ago that too much power was being developed, it is now generally agreed that there is not enough power to supply the demands made by national defense. The turbines being installed were previously seen during a visit to the S. Morgan Smith Co. last November, but because of precautions taken by the authorities, it was impossible to observe the installations of these units.

The hydraulic laboratory is doing work for the Portland Engineer District. Its main problems are concerned with the testing of spillways, outlet works, tunnels, siphons, and navigation locks. The procedure and technique employed is quite similar to that of the U. S. Waterways Experiment Station in Vicksburg, Mississippi.

PROGRESS REPORT FOR JUNE 1941

This is the eighth and final progress report to be submitted on the Freeman Scholarship for October 1940 to July 1941. Reports were submitted monthly during this nine-month period, with the exception of the fifth report which combined the work of February and March. The work in hydraulic research has been reported for all sections of the country, in addition to some of the hydraulic structures, irrigation projects, and flood control developments inspected, principally in the Eastern States. This final report completes the description of the observations made of irrigation projects.

Summary of Work Completed June 1941

Leaving Bonneville Dam on June 3, visits were made to the following projects of the Bureau of Reclamation: Yakima Project, Yakima, Washington, June 4-9; Columbia Basin Project, Grand Coulee Dam, Washington, June 10-12; Sun River Project, Fairfield, Montana, June 13-15; Shoshone Project, Cody, Wyoming, June 16; Riverton Project, Riverton, Wyoming, June 17. Travel was completed on reaching Denver June 18.

The outstanding feature of this month's work was the observations made of two structures which the writer had previously tested by models in the laboratory in Denver. One of these structures is a check drop in the Sunnyside Main Canal, Yakima Project. It consists of a concrete basin 32 feet wide and 14 feet long divided into five panels by low piers surmounted by steel brackets. Flashboards placed between the brackets raise the water surface in the canal for diversion into laterals. At this structure, the difference in water surface above and below the drop is only fifteen inches, yet excessive scour has developed in the canal immediately downstream. Attempts to control the scour and to correct the unfavorable flow conditions producing it were unsuccessful in the field. Finally, a model of the structure was tested in the hydraulic laboratory of the Bureau of Reclamation (Figure 1) until a satisfactory redesign had been evolved. It was revealed from these tests that because the drop in water surface was only 15 inches, sufficient energy was not available to produce a hydraulic jump regardless of changes made in the model. Instead, standing waves persisted in the canal which caused side eddies to scour the canal banks. (Figures 1 and 2). By placing a deflector across the check basin, the flow was forced to plunge into and under the tailwater. Extension of the side walls of the basin to form a rectangular stilling pool, and the addition of curved training walls in the approach to the drop completed the revisions (Figure 3). The revised structure has been in operation in the field for two seasons (Figure 4). It can be said without reservation that the revised prototype structure is performing as predicted by the model. (See Section 43, Chapter III.)

The other structure inspected was the Roza Diversion Dam in the Yakima River, near Yakima, Washington. This structure consists of two 14- by 110-foot roller gates placed above a concrete weir. A fish ladder is located at the left abutment and the headworks to the Roza Canal is located at the right abutment. Unfortunately, there was not sufficient flow in the river nor into the canal to permit any correlation between model and prototype. Nevertheless, it was enlightening to see the field structure after testing its model in the laboratory.

Inspection of the other projects included trips to the main storage dam, and along the main canals and laterals. Attention continued to be paid to the small hydraulic structures. On the Sun River Project, some of the inclined chute drops in the main canal were revised according to recommendations from the Denver laboratory. Although the writer was not associated with these model studies, the performance of the models was recalled and it could be observed that the revised prototype structures were performing as anticipated. Revision of these inclined drops, having a fall of over 10 feet, were difficult because the stilling basin was trapezoidal in section. It has been shown conclusively that sufficient energy dissipation is impossible in such a basin because of a return flow along the walls drowning a part of the hydraulic jump. A rectangular basin, on the other hand, permits a uniform flow distribution with excellent dissipation occurring. By using large concrete teeth, however, it was possible to improve greatly the trapezoidal basins of these Sun River inclined drops.

APPENDIX II

APPENDIX II

1. Expenses. Because the program of this scholarship was different from many of the others previously awarded, an account of expenses is given to show their distribution over a period of nine months in which the author traveled by automobile in nearly forty states for a total distance of almost 25,000 miles. During this period, the Society paid $2,000 to the author, $400 being given at the start of the scholarship in October 1940, and $200 a month for eight months thereafter.

To keep expenses at a minimum, it was convenient to stay at the Y.M.C.A. and in rooming houses; only occasionally was it necessary to stop at hotels. When visits were made at home or with friends, meals and lodging were, of course, gratis, so the cost of these and other items in the table below are slightly lower than if travel had been continuous for nine months. The cost of using and maintaining an automobile was not excessive, and travel in this manner was absolutely essential in order to reach isolated places.

The following table shows the expenses directly associated with the scholarship for the nine months' period of October 1940 to July 1941:

Table IV EXPENSES

Item	Cost	Percent of Total	Average Cost per Month	Remarks
Meals	$ 261.00	17.1	$ 29.00	
Lodging	228.87	15.0	25.45	
Personal appearance	106.51	6.9	11.82	Laundry, clothing, etc.
Automobile	463.84	30.4	51.50	Gas, oil, tires, repairs, and storage
Photography	157.09	10.3	17.44	Camera, light meter, and 800 photographs, size 2-1/8" x 2-1/2"
Correspondence	84.84	5.6	9.42	Postage, stationery, and telegrams
Miscellaneous	224.56	14.7	25.00	Entertainment, reading matter, sundries, etc.
Totals	$1,526.71	100.0	$169.63	

Contributing to family support, who resided with the author's father during the nine months' period, required an additional expense of $716.18. Included in this amount were life insurance premiums and similar commitments. The total expense, therefore, amounted to $2,242.89.