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HYDRAULIC LABORATORY REPORT NO. 174

- - -

HYD 174

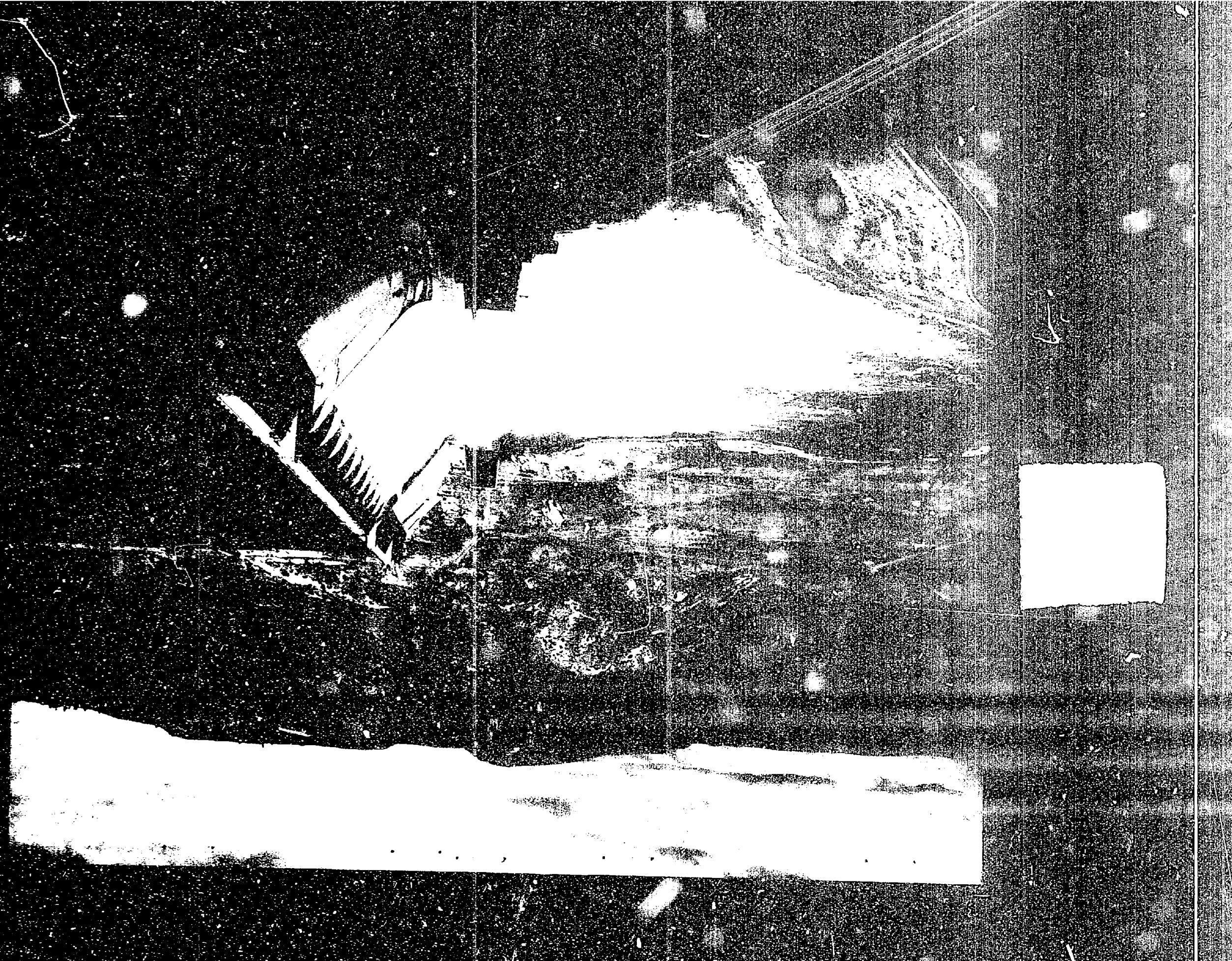
HYDRAULIC MODEL STUDIES PERTAINING TO THE REPAIR, MAINTENANCE, AND PROTECTION OF THE SPILLWAY BUCKET AND TO THE PROTECTION OF THE TAILRACE SLOPES AND DOWNSTREAM RIVER BANKS AT GRAND COULEE DAM COLUMBIA BASIN PROJECT, WASHINGTON

By

JAMES W. BALL

- - -

Denver, Colorado
June 15, 1945



PREFACE

The greater part of the program of hydraulic model studies concerning the development and design of the special floating caisson for inspecting and repairing the spillway bucket of Grand Coulee Dam, was made on a 1 to 60 scale model of the dam and approximately one mile of the downstream river channel, constructed at the project. These investigations pertained to operational phases of the hydraulic features of the dam, actions contributing to the damage incurred in the spillway bucket, measures to minimize or eliminate the source of the damage, and methods of protecting the tailrace slopes and river banks from destructive erosion by flood waters. Two 1 to 60 models of the caisson, constructed in the Denver shops, were used in many of these tests. Other pertinent tests were performed in the Denver hydraulic laboratory on a 1 to 20 scale working model of the floating caisson with its operating-barge and powered hoists, a 1 to 42 model of the caisson drydock and gate slab, a 1 to 60 model of the base of the right training wall and bucket section, a 1 to 24 model of the Grand Coulee turbine scroll case and draft tube, and a 1 to 30 model of an 84-inch needle valve and turbine draft tube. The investigations were begun in 1941 and the tests conducted between June 1943 and December 1944.

Plans for the 1 to 60 model dam were prepared by members of the concrete dams section in cooperation with members of the hydraulic laboratory staff. The model was constructed in the Vista Park area about a quarter of a mile upstream from the left abutment of Grand Coulee Dam by project personnel.

Plans and details for the 1 to 20 model of the caisson and operating barge were prepared by members of the canal section in cooperation with the hydraulic laboratory and the model was constructed by workmen in the Denver office shops.

Plans and details of the remaining models were prepared in the laboratory and the models constructed in the Denver shops.

Credit is due project personnel and members of the Denver office who took part in the planning, construction, and testing of the models.

Project workmen are to be commended on the excellent workmanship in the construction of the 1 to 60 model dam. Space and time do not permit individual mention of those who participated.

TABLE OF CONTENTS

Section | Page No.

I. INTRODUCTION

1. Hydraulic features of Grand Coulee Dam - - - - - - - - - - - 1
2. Desirability of inspecting the bucket section of Grand Coulee Dam - 8
3. The spillway bucket inspection and repair caisson - - - - 10
4. Model studies of the inspection and repair caisson - - - - 14

II. PURPOSE OF MODEL STUDIES

5. Scope of model tests - 17
6. Summary of results - 19

III. THE MODELS

7. Description of the models - - - - - - - - - - - - - - - - - 25
8. The 1 to 60 model of Grand Coulee Dam - - - - - - - - - - 26
9. The 1 to 60 scale model floating caisson for maneuvering tests - 51
10. The 1 to 60 scale model of the floating caisson used for pressure tests - 53
11. The 1 to 20 scale model of the floating caisson used for stability tests - 54
12. The 1 to 42 scale model of the caisson drydock - - - - - - 59
13. The 1 to 60 plaster model of damage at the base of right training wall - 64
14. The 1 to 24 and 1 to 30 models of outlets in right powerhouse - 66

IV. THE INVESTIGATIONS

15. Study of bypass outlets in right powerhouse - - - - - - - 66
16. Studies to place 1 to 60 model dam in operation - - - - - 68

TABLE OF CONTENTS (Continued)

Section

Page No.

17. Studies to determine resistance of caisson to currents of different velocity - - - - - - - - - - - - - - - 78

18. A study of conditions influencing the maneuvering of the floating caisson - 80

19. Studies to determine the number of lines required to maneuver the floating caisson - - - - - - - - - - - - - - 88

20. Tests to establish the optimum location for the shore ends of the maneuvering lines - - - - - - - - - - - - 90

21. Measurement of forces exerted on maneuvering lines to establish cable sizes - - - - - - - - - - - - - - - - - - 94

22. Determination of the optimum positions for hitching the maneuvering lines to the caisson - - - - - - - - - - - 95

23. Effect of drydock entrance channel on the maneuvering of the floating caisson - - - - - - - - - - - - - - - - - - 97

24. Operation of drum gates and river outlets with caisson in bucket - 101

25. Investigation on the 1 to 20 model concerning stability and operation of floating caisson - - - - - - - - - - - - - 106

26. Discussion of damage to spillway bucket reported by divers - 122

27. Optimum flow conditions for inspection of spillway bucket by divers - 130

28. Sand and gravel found in spillway bucket during 1943 fall inspection - 134

29. Study of flow conditions and erosion at ends of training walls - 137

30. Studies concerning the dredging of river fill to prevent or allay damage to the spillway bucket - - - - - - 145

31. Study of the effect of power outages on the tailwater elevation - 152

TABLE OF CONTENTS (Continued)

Section

Page No.

32. Observations concerning similarity of model and prototype - 155

33. Investigation concerning protection of tailrace slopes and river banks - 159

34. Study relating to the tipping of the concrete arch from the gate section of the drydock - - - - - - - - - - - 167

V. CONCLUSIONS AND RECOMMENDATIONS

35. Operation of 1 to 60 model dam - - - - - - - - - - - - - - - - 177

36. Bypass outlets in right powerhouse - - - - - - - - - - - - - 178

37. Resistance of caisson to currents - - - - - - - - - - - - - - 179

38. Maneuvering lines for caisson - - - - - - - - - - - - - - - - 180

39. Excavation adjacent to drydock entrance channel - - - - - - 182

40. Excess flow over spillway - - - - - - - - - - - - - - - - - - 183

41. Operation of drum gates and river outlets with caisson on bucket - 183

42. Stability and operation of floating caisson - - - - - - - - 184

43. Source of damage to spillway bucket - - - - - - - - - - - - 186

44. Flow distribution for optimum diving conditions - - - - - 187

45. Source of material in bucket - - - - - - - - - - - - - - - - 188

46. Flow and erosion at ends of training walls - - - - - - - - 188

47. Dredging of river fill to allay damage to bucket - - - - 189

48. Recession of tailwater for power outages - - - - - - - - - 191

49. Similarity of model and prototype - - - - - - - - - - - - - 191

50. Protection of tailrace slopes and downstream river banks - 192

TABLE OF CONTENTS (Continued)

Section		Page No.
51.	Tipping of arched slab from drydock gate section - - - -	194

APPENDICES

No.		
I.	Hydraulic Report No. 132	196
II.	Abstract of Test Records	
	Erosion at downstream ends of training wall, 1 to 120 scale model of Grand Coulee Dam - - - - - - - - - -	203
III.	Hydraulic Report No. 10	
	(With only most important drawings and photographs included) -	209
IV.	List of Pertinent Correspondence	228

LIST OF FIGURES

Figure	Title	Page No.
1.	Dam, power and pumping plants - Elevations and sections	2
2.	The 1 to 60 model of Grand Coulee Dam with the spillway and left powerhouse in operation	3
3.	The 1 to 60 scale model of Grand Coulee Dam with the lower, intermediate and upper tiers of river outlets in operation	6
4.	Flow lines in roller bucket stilling pool	7
5.	Floating caisson - General assembly - Plan and elevations	11
6.	Floating caisson - General assembly - Sections	12
7.	Results of spillway bucket inspection - March 1943	16
8.	Elevations and Sections - 1:60 scale-model dam	27
9.	Plan and elevations of right and left powerhouses 1:60 scale-model dam	28
10.	Sections of right and left powerhouses - 1:60 scale-model dam	29
11.	Model of Grand Coulee Dam (1:60 scale) with downstream river channel prepared for testing	30
12.	Model pump and supply system - 1:60 scale-model dam	31
13.	Hydraulic model - Structure details - Plan and Sections	32
14.	Revised water supply system. 1:60 scale-model dam	34
15.	Right powerhouse needle valve outlets - Assembly and details - 1 to 60 model dam	35
16.	Model outlet details - 1:60 scale-model dam	37

LIST OF FIGURES (Continued)

Figure	Title	Page No.
17.	Apparatus for forming bends in river outlet tubes of 1 to 60 model dam	38
18.	River outlet tubes supported for embedding and forms for right abutment of 1 to 60 scale-model dam	40
19.	Hinged form for downstream face of spillway section and completed left abutment of 1 to 60 scale-model dam	41
20.	Drum gate and crest details - 1:60 scale-model dam	43
21.	Crest and pier details - 1:60 scale-model dam	44
22.	Air entrainment device details - 1:60 scale-model dam	45
23.	1 to 60 hydraulic model - Topography, river bed and banks	47
24.	Stages in the construction of the porous-concrete topography of the 1 to 60 model of Grand Coulee Dam	48
25.	Completed porous-concrete section of the downstream river channel of the 1 to 60 model of Grand Coulee Dam	49
26.	Completed 1 to 60 plastic model of the floating caisson to be used in the repair and maintenance of the spillway bucket of Grand Coulee Dam	52
27.	Fabricated sections of 1 to 20 model of the floating caisson to be used in the repair and maintenance of the spillway bucket of Grand Coulee Dam	55
28.	Constructed sections of 1 to 20 model of spillway bucket, repair caisson and operating barge	56
29.	Completed 1 to 20 model of the floating caisson and operating barge to be used in the repair and maintenance of the spillway bucket of Grand Coulee Dam	57

LIST OF FIGURES (Continued)

Figure	Title	Page No.
30.	Manipulation tank for 1:20 model of the floating caisson	60
31.	Details of the 1:42 scale model of the caisson drydock	62
32.	The 1 to 42 scale model of the drydock for the floating caisson with the gate slab and adjacent topography	63
33.	The 1 to 60 model right training wall and a section of the bucket depicting damage to base of the wall and the lip of the bucket	65
34.	Needle valve by-pass outlets in the right powerhouse of the 1 to 60 scale model dam in operation. Flow in river representing 73,000 second-feet	69
35.	Calibration curve for 8-inch venturi meter in the model supply line of the 1:60 scale-model dam	70
36.	Calibration curve for pitot tubes in 14- and 24-inch supply pipes of 1:60 scale-model dam	72
37.	Comparison of water surface gradient in river below dam - Model-Prototype - 1 to 60 scale-model dam	74
38.	Discharge curve for flow nozzles in right powerhouse of 1:60 scale-model dam	76
39.	Resistance of caisson to currents of different velocity - 1 to 60 scale-model dam	81
40.	Floating caisson - Study of maneuvering operations	83
41.	Devices used on 1 to 60 scale-model dam to determine pull on maneuvering lines of caisson and to record drop of water surface for power outages	86
42.	Currents and velocities in downstream river channel of 1 to 60 scale-model dam	87

LIST OF FIGURES (Continued)

Figure	Title	Page No.
43.	Line forces to maneuver caisson from drydock to dam and return - Final arrangement - 1 to 60 scale-model dam	93
44.	Hydraulic model studies - Scale 1:60. Maintenance of spillway bucket - Drydock approach channel - Excavation of slopes	98
45.	Effect of topography on deposit of material in entrance channel to drydock	102
46.	The Metacentric Height of a Vessel	108
47.	Relation of variables for experimental location of center of gravity of model operating barge	111
48.	Curve tan θ vs. reactions for model operating barge	113
49.	Floating caisson - Sinking and raising operation diagram	117
50.	Results of spillway bucket inspection - November-December 1943	123
51.	Flow conditions of Grand Coulee Dam - June 1940	124
52.	Reproduction on 1 to 60 model dam of flow conditions existing during 1940 flood and resulting erosion	126
53.	Erosion at end of right training wall with and without concrete block in crevice between wall and rock topography	138
54.	Erosion at end of left training wall with and without bonding the rock and adding the concrete cap at end of wall	139
55.	Movement of river fill below bucket of 1 to 60 model dam for various depths and limits of excavation and flows representing 300,000 and 537,000 second-feet	148

LIST OF FIGURES (Continued)

Figure	Title	Page No.
56.	Movement of river fill below bucket of 1 to 60 model dam for various downstream limits of excavation and flows of 537,000 and 787,000 second-feet	150
57.	Hydraulic model experiments 1 to 60 scale recession of tailwater caused by power outage	154
58.	Downstream river channel on 1 to 60 model dam before test and wave action on right tailrace slope	161
59.	Erosion by wave action of tailrace slopes and downstream river banks	162
60.	Erosion by wave action of 6 to 1 and 4 to 1 sloped areas of tailrace slopes blanketed with graded riprap representing 6- to 26-inch rock	164
61.	Contours of erosion below ends of spillway of 1 to 60 model dam after floods of various magnitude	165
62.	Center of gravity of arched gate slab of drydock by balancing	169
63.	Graphical location of center of gravity of arched gate of drydock	169
64.	Force to tip drydock gate slab and distance gate slab to drydock base after tipping	171
65.	Arched gate slab of 1 to 42 drydock model ready for tipping and in position on entrance channel floor after being tipped	173
66.	Depth of water in drydock to tip gate slab - effect of submergence on sliding	176

Tables

No.	Title	
I.	Characteristics of maneuvering lines	95

TABLES (Continued)

No.	Title	Page No.
II.	Flow characteristics for emergency operation of spillway or river outlets with caisson in bucket -	104
III.	Summary of flow distribution, velocities and flow conditions for studies pertaining to diving operations -	131

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
June 15, 1945

Laboratory Report No. 174
Hydraulic Laboratory
Compiled by: James W. Ball

Reviewed by: Jacob E. Warnock

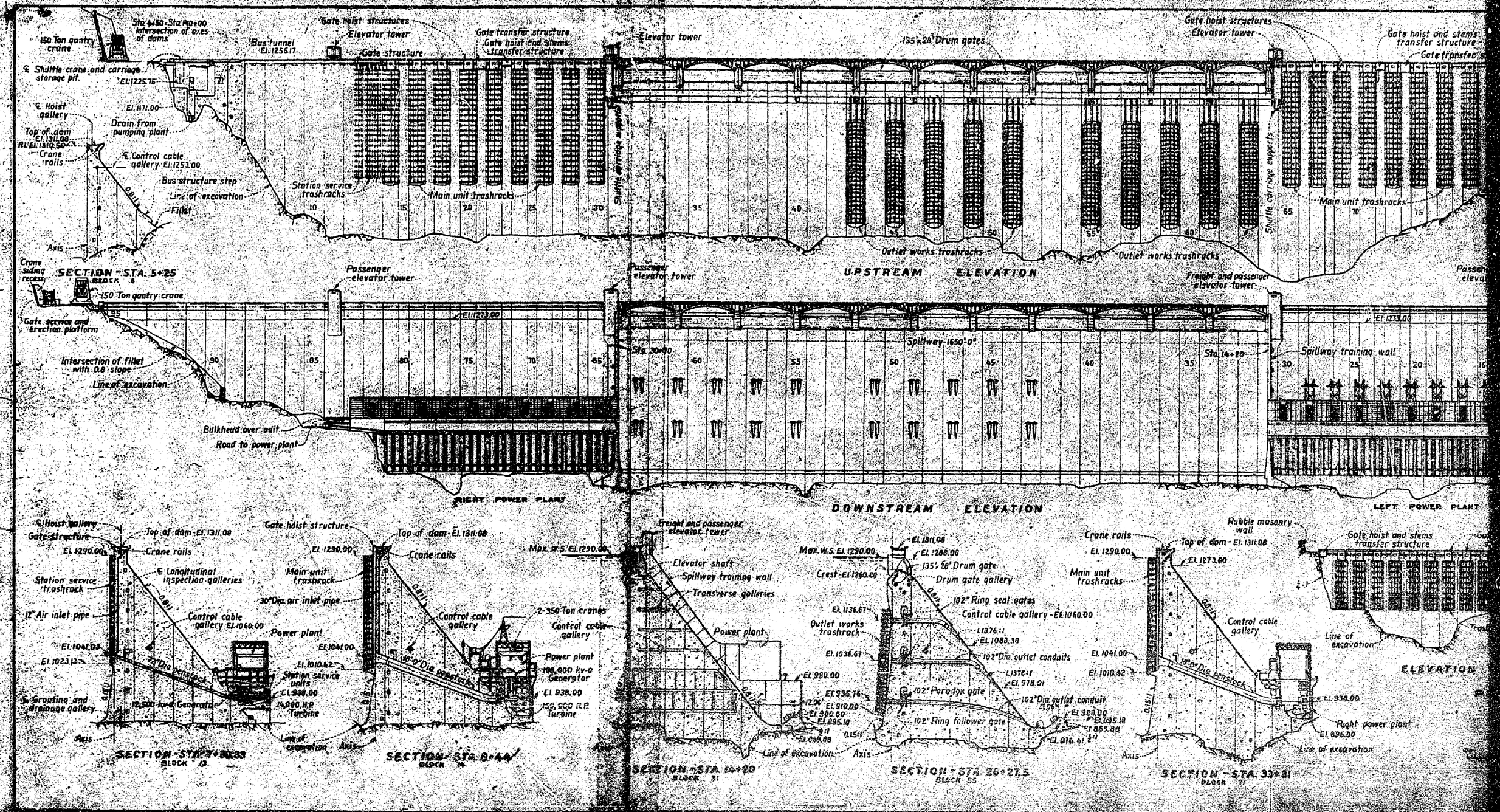
Subject: Hydraulic model studies pertaining to the repair, maintenance, and protection of the spillway bucket and to the protection of the tailrace slopes and downstream river banks at Grand Coulee Dam - Columbia Basin Project, Washington.

I. INTRODUCTION

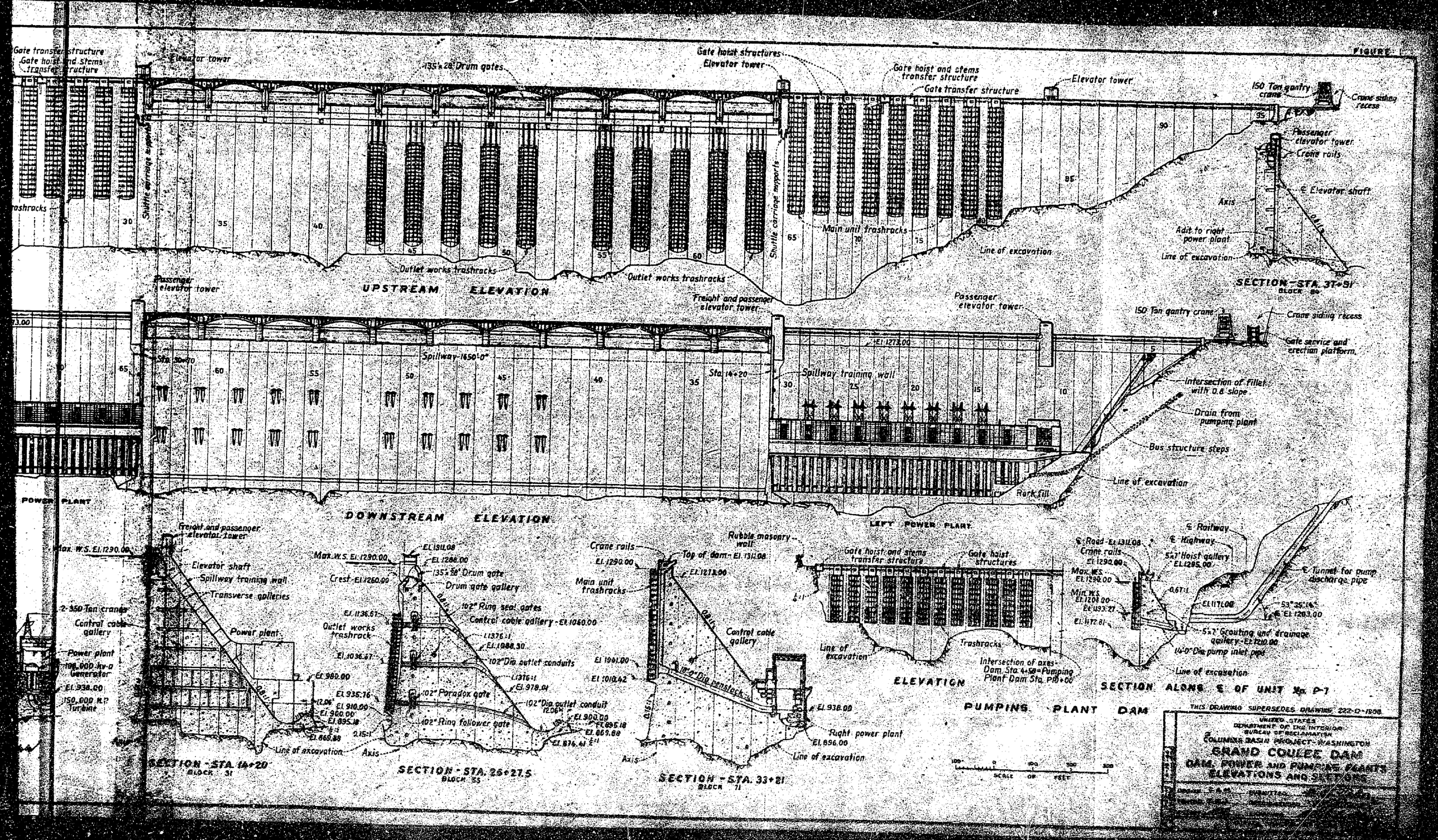
1. Hydraulic features of Grand Coulee Dam. The important hydraulic features of Grand Coulee Dam are its overfall spillway, sixty river-outlet conduits, two powerhouses, and pumping plant, figure 1. The pumping plant will not be considered in this report since it has no direct bearing on the problems to be discussed.

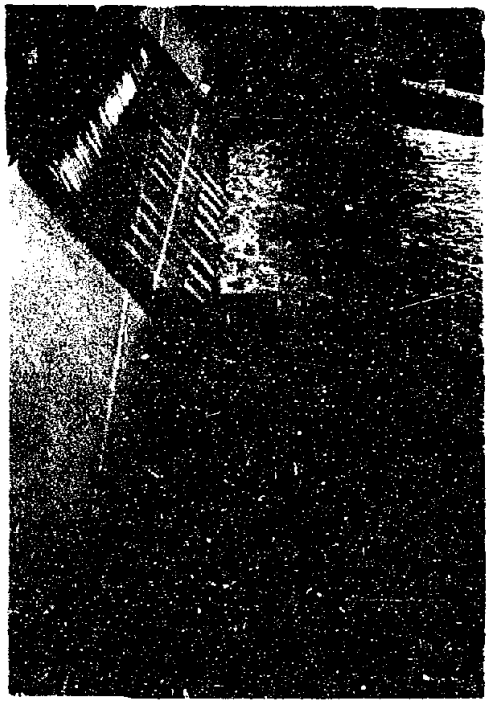
The spillway for conveying surplus flow from the reservoir to the river bed downstream has a gross length of 1,650 feet. Its crest, having an ogee profile, is controlled by 11 hydraulically operated drum gates, 28 feet high and 135 feet long, separated by piers supporting a roadway across the dam. Two training walls on the downstream face of the dam confine the spillway flow to a roller-bucket stilling pool at its toe. The difference in elevation of the reservoir-water surface and tailwater will vary from 350 feet for low flows to 280 feet for the design discharge of 1,000,000 second-feet. Some idea as to the action of this feature under flood conditions may be obtained from photographs of both the model (figure 2), and full-sized structure (frontispiece).

The sixty 102-inch diameter river outlets arranged in three tiers, each differing in elevation by 100 feet, pass through the right two-thirds of the spillway section. Each tier contains 20 outlets, arranged

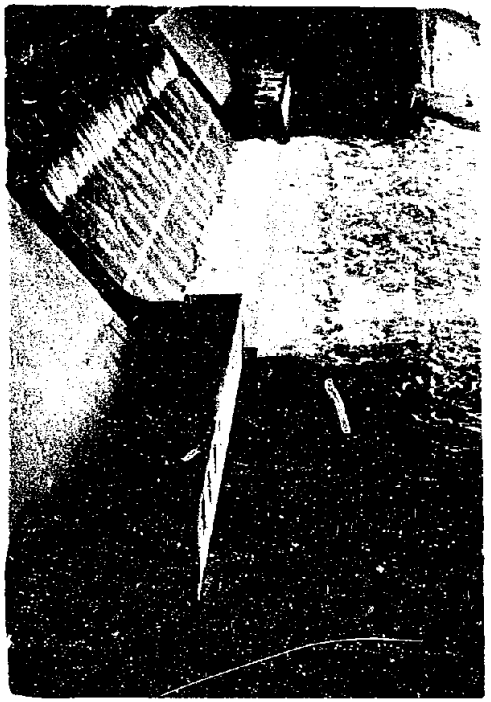

Sta. 4+50 - Sta. 110+00 Intersection of ℄'s of dams
150 Ton gantry crane
Bus tunnel El. 1255.17
Gate hoist structures
Elevator tower
Gate structure
Gate transfer structure
Gate hoist and stems transfer structure
Elevator tower
135'x28' Drum gates
Gate hoist structures
Elevator tower
Gate hoist and stems transfer structure
Gate transfer
℄ Shuttle crane and carriage storage pit
El. 1225.75
℄ Hoist gallery
El. 1171.00
Drain from pumping plant
Top of dam El. 1311.08
R.L. El. 1310.50
Crane rails
℄ Control cable gallery El. 1253.00
Bus structure step
Line of excavation
Fillet
Station service trashracks
Main unit trashracks
Shuttle carriage supports
Main unit trashracks
Axis
Crane siding recess
SECTION - STA. 5+25
BLOCK 8
150 Ton gantry crane
Passenger elevator tower
Passenger elevator tower
UPSTREAM ELEVATION
Outlet works trashracks
Outlet works trashracks
Freight and passenger elevator tower
Passen elevat
Gate service and erection platform
El. 1273.00
Intersection of fillet with 0.8 slope
Line of excavation
Sta. 30+90
Spillway 1650'-0"
Sta. 14+20
Spillway training wall
Bulkhead over adit
Road to power plant
RIGHT POWER PLANT
DOWNSTREAM ELEVATION
LEFT POWER PLANT
℄ Hoist gallery
Gate structure
El. 1290.00
Top of dam - El. 1311.08
Crane rails
℄ Longitudinal inspection galleries
Station service trashrack
12" Air inlet pipe
El. 1041.00
El. 1023.13
Control cable gallery El. 1060.00
Power plant
72" Dia penstock
12,500 kw Generator
℄ Grouting and drainage gallery
Axis
SECTION - STA. 7+80.33
BLOCK 13
Gate hoist structure
El. 1290.00
Top of dam - El. 1311.08
Crane rails
Main unit trashrack
30" Dia. air inlet pipe
Control cable gallery
El. 1041.00
El. 1010.42
Station service units
El. 938.00
14,000 H.P. Turbine
Line of excavation
Axis
2-350 Ton cranes
Control cable gallery
Power plant
108,000 kv-a Generator
El. 938.00
150,000 H.P. Turbine
48" Dia penstock
SECTION - STA. 8+44
BLOCK 34
Freight and passenger elevator tower
Max. W.S. El. 1290.00
Elevator shaft
Spillway training wall
Transverse galleries
Power plant
El. 980.00
El. 935.76
El. 910.00
El. 900.00
El. 895.18
El. 869.88
Line of excavation
Axis
SECTION - STA. 14+20
BLOCK 31
Max. W.S. El. 1290.00
Crest - El. 1260.00
El. 1311.08
El. 1288.00
135'x28' Drum gate
Drum gate gallery
102" Ring seal gates
Control cable gallery - El. 1060.00
El. 1136.67
Outlet works trashrack
El. 1036.67
102" Dia outlet conduits
El. 978.91
102" Paradox gate
102" Dia outlet conduit
El. 900.00
El. 895.18
102" Ring follower gate
El. 816.41
Axis
SECTION - STA. 26+27.5
BLOCK 55
Crane rails
El. 1290.00
Main unit trashracks
Rubble masonry wall
Top of dam - El. 1311.08
El. 1273.00
Control cable gallery
El. 1041.00
El. 1010.42
108" Dia penstock
El. 938.00
Right power plant
El. 896.00
Line of excavation
Axis
SECTION - STA. 33+81
BLOCK 71
Gate hoist and stems transfer structure
Line of excavation
ELEVATION

FIGURE


Gate transfer structure
Gate hoist and stems transfer structure
Elevator tower
135'x26' Drum gates
Gate hoist structures
Elevator tower
Gate hoist and stems transfer structure
Gate transfer structure
Elevator tower
150 Ton gantry crane
Crane siding recess
Passenger elevator tower
Crane rails
℄ Elevator shaft
Axis
Adit to right power plant
Line of excavation
Trashracks
Shuttle carriage supports
Main unit trashracks
Line of excavation
Passenger elevator tower
Outlet works trashracks
Outlet works trashracks
UPSTREAM ELEVATION
SECTION - STA. 37+51
BLOCK 84
Freight and passenger elevator tower
Passenger elevator tower
150 Ton gantry crane
Crane siding recess
Gate service and erection platform
Sta. 30+70
Spillway-1650'-0"
Sta. 14+20
Spillway training wall
El. 1273.00
Intersection of fillet with O.B. slope
Drain from pumping plant
Bus structure steps
Line of excavation
Rock fill
POWER PLANT
DOWNSTREAM ELEVATION
LEFT POWER PLANT
Freight and passenger elevator tower
Max. W.S. El. 1290.00
Elevator shaft
Spillway training wall
Transverse galleries
2-350 Ton cranes
Control cable gallery
Power plant
Power plant 108,000 hv-a Generator
El. 938.00
150,000 H.P. Turbine
Line of excavation
Axis
SECTION - STA. 14+20
BLOCK 31
Max. W.S. El. 1290.00
Crest - El. 1260.00
El. 1311.08
El. 1286.00
135'x58' Drum gate
Drum gate gallery
102" Ring seal gates
Control cable gallery - El. 1060.00
Outlet works trashrack
102" Dia. outlet conduits
102" Paradox gate
102" Ring follower gate
Axis
SECTION - STA. 28+27.5
BLOCK 55
Crane rails
Top of dam - El. 1311.08
El. 1290.00
Main unit trashracks
Control cable gallery
18'-0" Dia. penstock
Right power plant
Line of excavation
Axis
SECTION - STA. 33+21
BLOCK 71
Rubble masonry wall
Gate hoist and stems transfer structure
Gate hoist structures
Line of excavation
Trashracks
Intersection of axes Dam Sta. 4+58 = Pumping Plant Dam Sta. P10+00
ELEVATION
PUMPINS PLANT DAM
℄ Railway
℄ Highway
℄ Road - El. 1311.08
Crane rails
5'x7' Hoist gallery
Max. W.S.
Min. W.S.
℄ Tunnel for pump discharge pipe
5'x7' Grouting and drainage gallery - El. 1210.00
14'-0" Dia. pump inlet pipe
Line of excavation
SECTION ALONG ℄ OF UNIT No. P-7
THIS DRAWING SUPERSEDES DRAWING 222-D-1808
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
DAM, POWER AND PUMPING PLANTS
ELEVATIONS AND SECTIONS
SCALE OF FEET



A. Discharge 250,000 sec.-ft.



B. Discharge 537,000 sec.-ft.

C. Discharge 787,000 sec.-ft.

THE 1 TO 60 MODEL OF GRAND COULEE DAM WITH THE SPILLWAY AND LEFT POWERHOUSE IN OPERATION

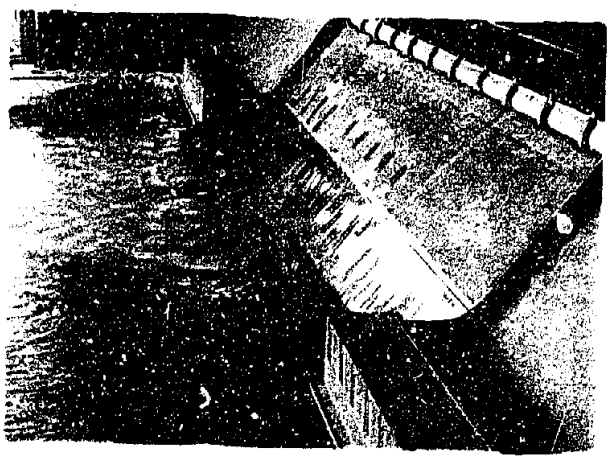
Figure 2

in pairs, located directly above or below those of the other tiers. All emerge from the downstream face of the dam and direct their flow down its face into the stilling pool. An idea as to the flow conditions with the upper, intermediate, or lower tiers operating may be obtained from photographs of the model, figure 3.

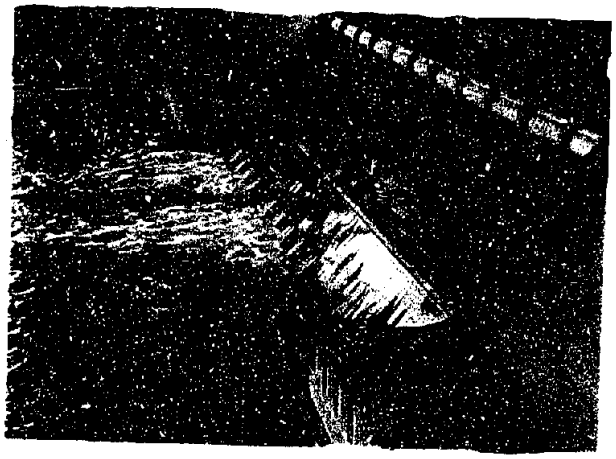
The two powerhouses, situated at the base of the dam, one at each end of the spillway section, having an ultimate capacity of 945,000 kilowatts each, excluding service units, discharge their flow from the turbines through draft tubes into tailraces excavated into the river banks, where it passes into the main river channel downstream from the spillway stilling pool.

The energy in the powerhouse flow has been transformed into electrical power and the water contains little destructive force as it leaves the draft tubes and flows into the tailraces. Unlike this flow, that from the spillway or river outlets attains its maximum energy as it enters the tailwater and this energy must be dissipated or reduced to nondestructive form before it is released to continue on downstream. Otherwise excessive erosion is likely to occur at the toe of the dam. The curved surface or bucket at the toe of the dam was provided to accomplish this transformation of energy which takes place as the sheet of flowing water passes through the bucket and moves downstream through the tailwater, figure 4. The decrease in velocity due to the retardation of flow as it passes through the tailwater to the bucket is responsible for a large portion of the change, but other factors contribute materially to the energy transformation. As the flow emerges from the bucket lip, on a 1 to 1 slope upward, gravity counteracts the vertical component reducing the energy content and the thickening sheet divides. A small portion of the top layer turns upstream to form an elliptical roll over the bucket in which the energy is reduced further, while the remainder moves downstream where the lower-than-tailwater pressure below the bucket lip causes some of the flow at the bottom of the stream to separate and form a ground roller in which the water contacting the river bed moves upstream. This roller, noted in model studies during

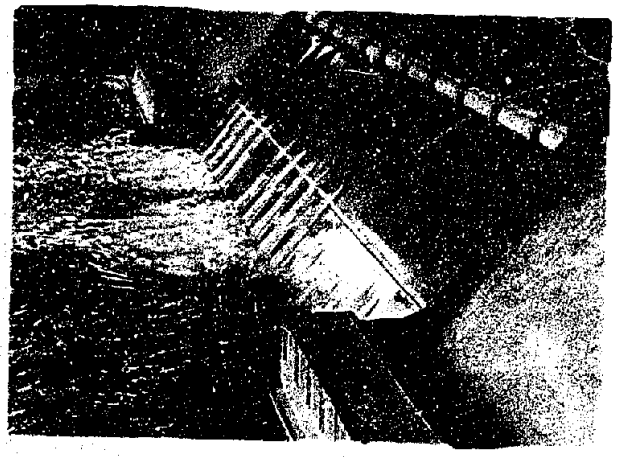
FIGURE 3



A. Lower tier of river outlets in operation

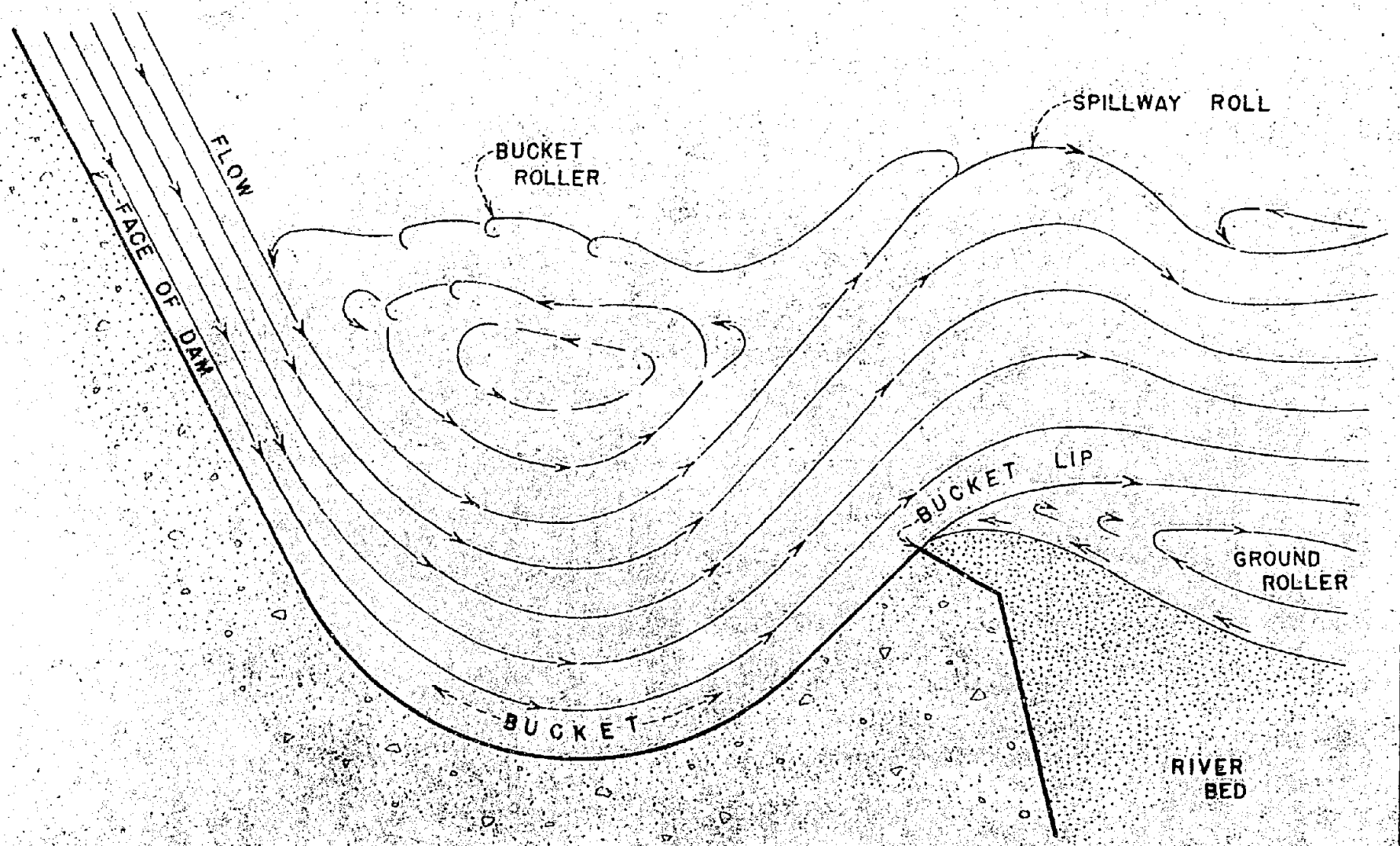


B. Intermediate tier of river outlets in operation



C. Upper tier of river outlets in operation

THE 1 TO 60 MODEL OF GRAND COULEE DAM WITH THE LOWER, INTERMEDIATE AND UPPER TIERS OF RIVER OUTLETS IN OPERATION



FLOW LINES IN ROLLER BUCKET STILLING POOL

FIGURE 4

the development of the bucket design, was observed to move river-bed material upstream and deposit it below the bucket lip, expending additional energy.

It was important that a maximum portion of the energy from the spillway and outlet flow be transformed into nondestructive form particularly when concern was expressed by Bureau engineers that damage to the bucket by moving debris during construction was a possibility which might decrease its efficiency as an energy dissipator.

2. Desirability of inspecting the bucket section of Grand Coulee Dam. The necessity of handling large quantities of water during the seasonal run-off did not permit a uniform distribution of the river flow within the spillway section during the construction of Grand Coulee Dam. As a result, extreme concentrations of flow often existed. High-velocity eddies, capable of transporting materials on the river bottom, formed in the downstream river channel, as was evidenced by noise from the impact of rocks on one another or on boulders of the river bed, which was audible at times from the banks. Sand and gravel were believed to have been transported into the bucket of the spillway by these currents with the possibility that damage to the concrete surfaces by abrasion due to the movement of this material existed. It was considered also likely that various objects had been accidentally dislodged from the numerous trestles, scaffolds, platforms and forms erected in the spillway section during the construction period, and that these had been carried into the bucket where damage to the surfaces by their movement might have resulted.

These conditions were cause for apprehension and were the main factors responsible for the instigation of studies to develop some means of making an inspection of the concrete surfaces of the spillway bucket.

During the period when the structure was nearing completion and it was necessary to release the normal river flow through the spillway section, either via the outlets or low blocks in the dam, inspection by diving was considered infeasible due to the adverse eddy currents in the river adjacent to the toe of the dam. On the other hand, it

was considered imperative that an inspection be made so work was begun on the design of special equipment for that purpose. Records show that the flow of the Columbia River at Grand Coulee Dam approaches 500,000 second-feet occasionally, exceeds 100,000 second-feet from 3 to 7 months of each year, and remains in excess of the left powerhouse capacity during most of the low-water season. The first of these conditions precludes continuous repair operations in the bucket section, the second might interfere with the repair work to some extent, the third would not permit a uniform distribution of the flow over the spillway, and uniform flow could not be obtained by using the river outlets.

In the preliminary stages of the development of the inspection and repair equipment, it was planned to discharge excess water through the river outlets or over the spillway gates farthest from where the repair operations were being performed, thus if the total flow passing the dam exceeded appreciably that of the left powerhouse the conditions below the dam might become so rough as to cause temporary postponement or suspension of the repair work. Moreover, unsymmetrical conditions would prevail similar to those existing during construction and during operation of the spillway immediately following its completion, which were considered the main sources of the damage to the bucket. With operation in this manner for such long periods, repair work might become a continuous operation. To eliminate this possibility and assure against a shortening of the repair season due to choppy water below the spillway, it was planned to provide other facilities for conveying the surplus flow during the low-water season from the reservoir to the downstream river channel. Outlet works, with the water bypassing the dam through tunnels, were considered, but in view of the excessive cost this plan was abandoned and consideration given to using the undeveloped power facilities in the right powerhouse. Diversion through runnerless turbine scroll cases of two units was considered but materials were not available and the hydraulic problems were somewhat complicated, so the plan was abandoned in favor of one involving the installation of 84-inch needle valves between the power penstocks and the unfinished turbine draft tubes. In this arrangement steel cones and elbows were to be

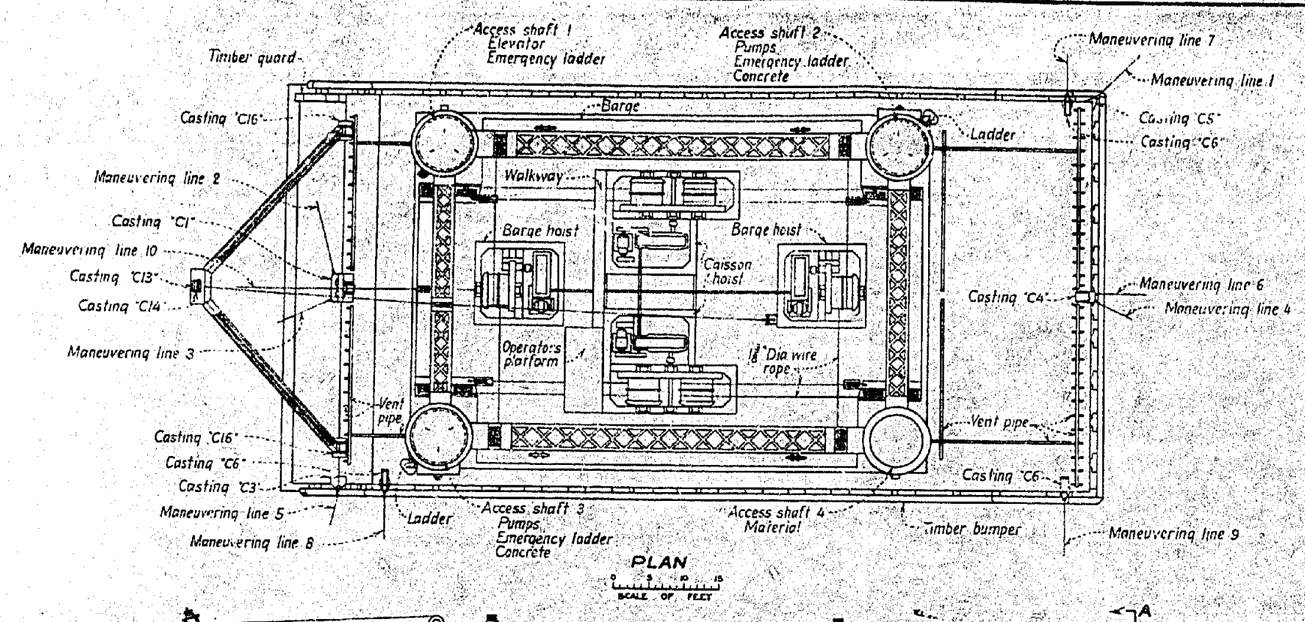
used for connecting the valves to the penstocks while a special steel transition embedded in concrete would complete the passage from the valves to the draft tubes which would discharge into the tailrace downstream. Model studies were made of both designs and variations of them. However, even with the additional outlet capacity the work would have to be seasonal and equipment would have to be removed from the bucket each spring before the flood season, transferred to some location for protection, and returned to the dam as the flood waters subsided. Various types of equipment were proposed, and after careful consideration, a structural-steel floating caisson was designed. This caisson was shaped to fit the bucket profile and was capable of being floated into place over the bucket, lowered into it, the working space or chamber underneath it emptied or filled and then raised.

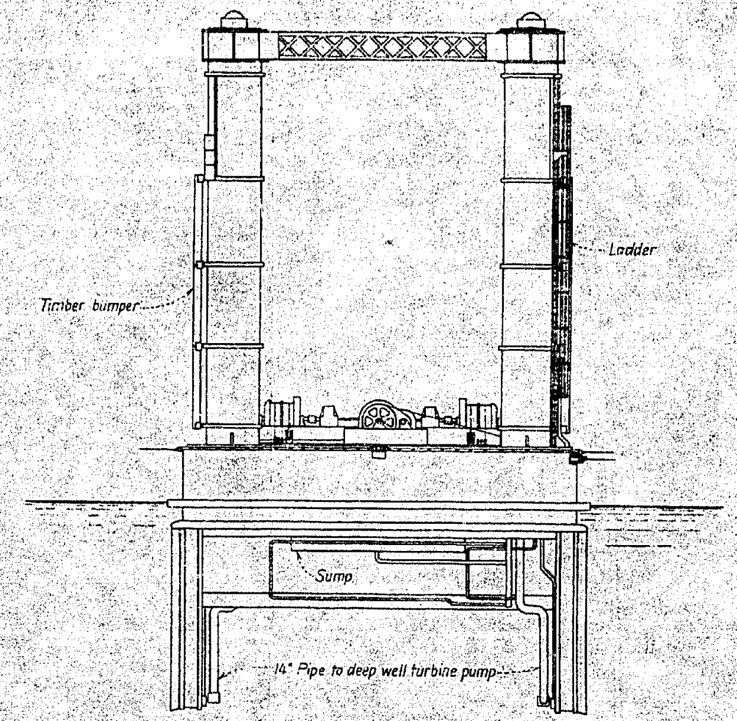
3. The spillway bucket inspection and repair caisson. The caisson is to be a box-like structure, approximately 117 feet long and 59 feet wide, constructed of panels of structural-steel girders and steel skin plates joined by riveting or welding, figures 5 and 6. Although differing somewhat in detail from the final design, the general description of the latter contained in the following paragraphs is applicable.

Two large panels, about 2-1/2 feet thick with the bottom edge shaped to fit the designed profile of the spillway bucket, will form the sides of the structure while panels spanning the ends of these will complete the sides of the box.

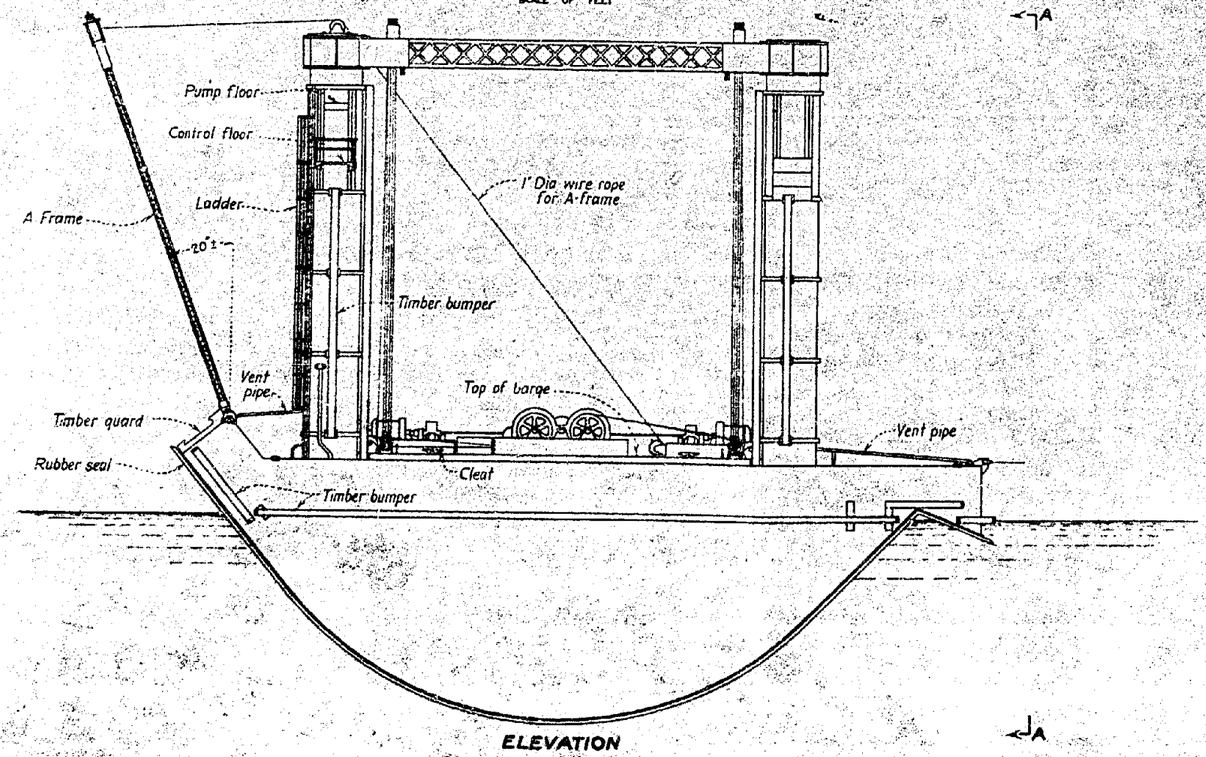
A large cellular panel containing seven cells or compartments about 10 by 6-1/2 feet in cross section through the center portion of the panel and two cells 6-1/2 by 8 feet at the ends, placed between the side girders and centered on them, approximately, with the long axes of the compartments horizontal and their ends closed by the inside skin plates of the side girders, will provide the floating and trim tanks of the caisson as well as the floor for the barge well and a portion of the top of the working chamber. The top of the working space will be completed by panels extending vertically from the ends of the cellular section to the top of the side girders

FIGURE 5





ELEVATION A-A



ELEVATION

REFERENCE DRAWINGS

STRUCTURAL DETAILS	222-D-9494 TO 9524 INCL
ACCESS-SHAFT ELEVATOR	222-D-9530
HOISTS AND SHEAVES FOR CAISSON, BARGE, LINE 10 AND A-FRAME	222-D-9407 TO 9417 INCL, 9459 TO 9462 INCL, 9472 & 2478
PULLERS AND SHEAVES FOR MANEUVERING LINES	222-D-9403, 9436, 9437, 9438, 9463 TO 9468 INCL
PUMPING SYSTEM	222-D-9580 TO 9589 INCL

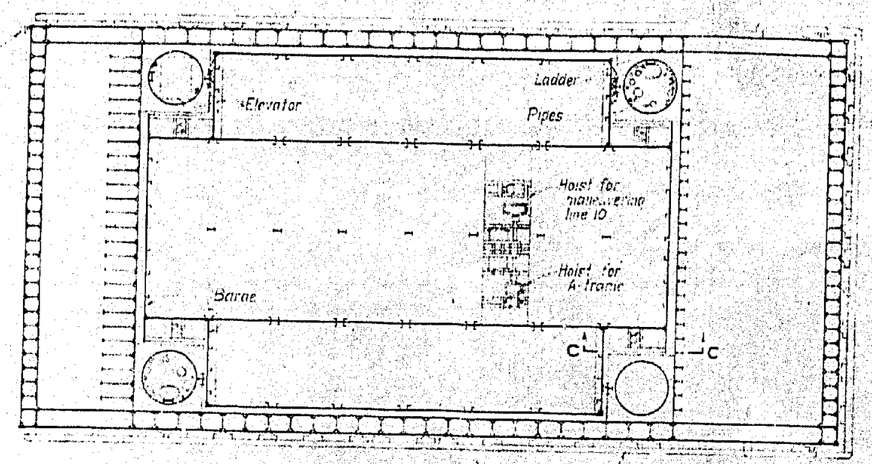
THIS DRAWING SUPERSEDES DWG. 222-D-9268

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASHINGTON
GRAND COULEE DAM
MAINTENANCE OF SPILLWAY BUCKET
FLOATING CAISSON
GENERAL ASSEMBLY
PLAN - ELEVATIONS

DRAWN C.O.L. SUBMITTED
TRACED R.A.S. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO, DEC. 6, 1944

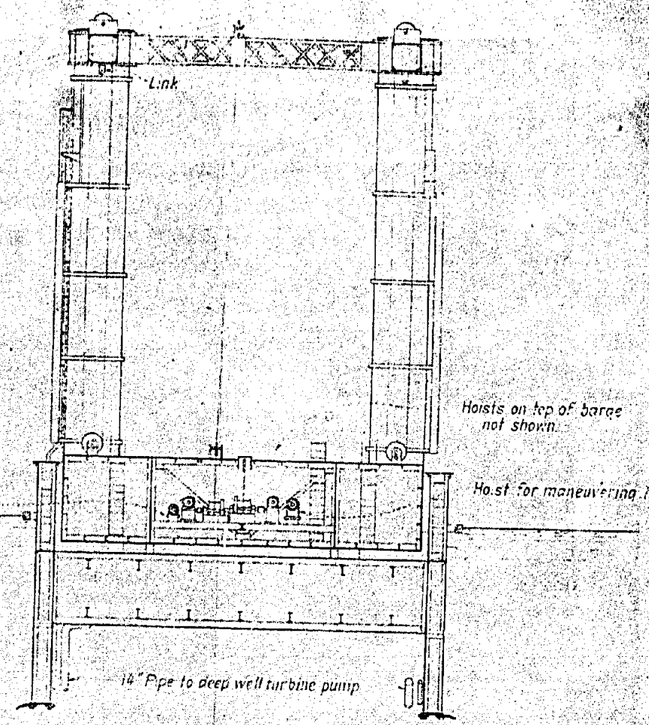
222-D-9492

FIGURE 6

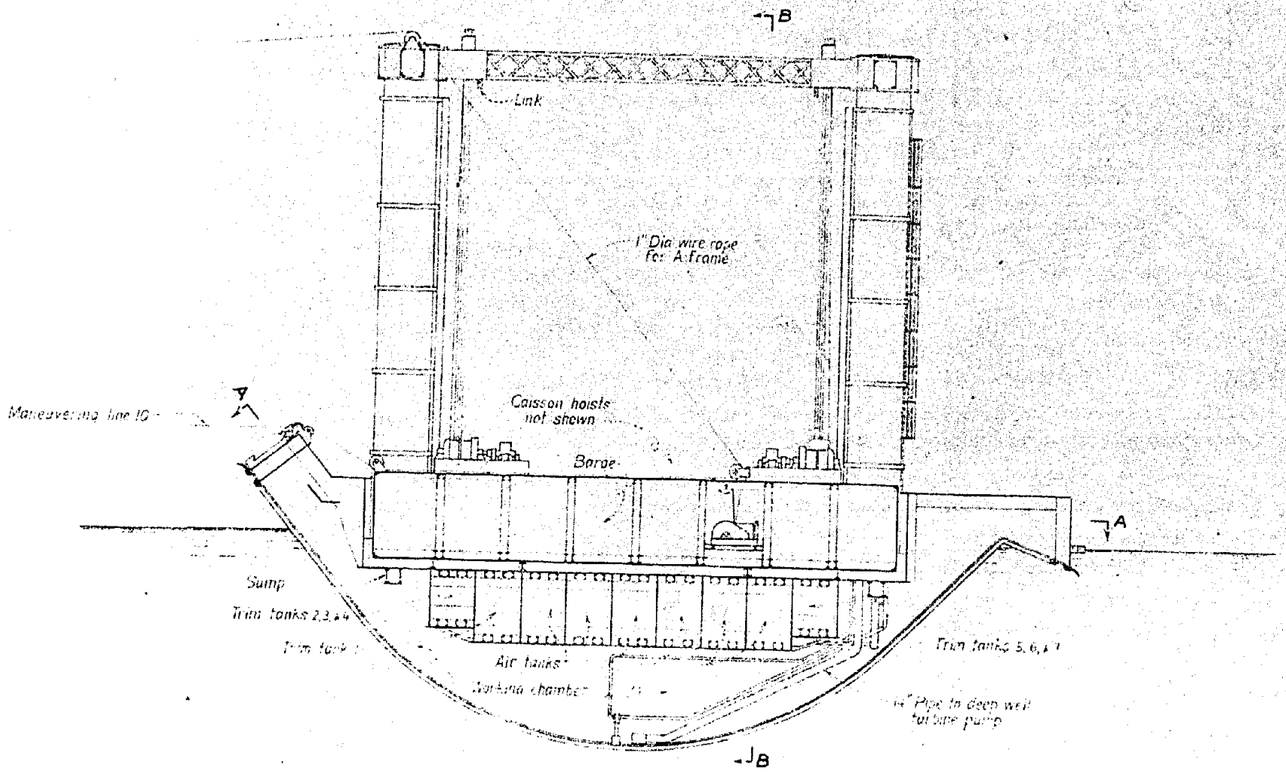


SECTION A-A

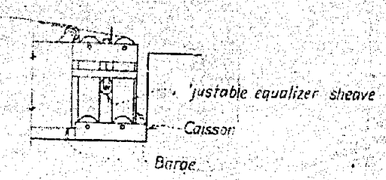
SCALE OF FEET



SECTION B-B



LONGITUDINAL SECTION
(A-frame not shown)



SECTION C-C

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON

GRAND COULEE DAM
MAINTENANCE OF SPILLWAY BUCKET

FLOATING CAISSON
GENERAL ASSEMBLY
SECTIONS

DRAWN G.B.A. SUBMITTED
TRACED W.H.J. RECOMMENDED
CHECKED R.S. APPROVED

DENVER, COLORADO JUNE 5, 1944 222-D-9493

and from these points along the tops of the girders to the ends of the caisson. When in position on the bucket, the working space will be bounded by the bucket surface, the inner surfaces of the side and end girders, and the bottom surfaces of the cellular and top panels. A huge rubber seal will be provided along the bottom of the side and end girders. With this arrangement the repair work can be accomplished under atmospheric pressure.

Four circular shafts, 7-1/2 feet in diameter, through the corners of the cellular panel, will extend vertically upwards approximately 73 feet to provide access for workmen and repair materials when in position on the bucket and to house machinery for pumping water from the working chamber and controlling the floating and trim compartments.

A well 51 feet wide, 75 feet long, and 12-1/2 feet deep on top of the cellular panel and bounded by the side girders and the vertical panels at the ends of the cellular section will harbor the operating barge required to insure stability of the caisson during the sinking and raising operation. The operating barge, a steel structure of similar construction, 72-1/2 feet long, 49-1/2 feet wide, and 12 feet deep, with the corners notched for the access shafts, will be attached to the caisson from the bottom by four 12-part lines, one at each corner, operated from hoists on the barge. Lowering the caisson will be accomplished by filling the space in the barge well surrounding the barge and paying out cable from the hoists of the 12-part lines on the barge, keeping the bottom of the barge below the water surface in the well until water begins to flow over the top of the side girders, when the unbuoyed load of the caisson will be transferred completely to the 12-part lines of the barge and it can be lowered into the bucket by them. The caisson will be refloated by shortening the 12-part lines, forcing the barge into the well and pumping water from the well after the side girders are brought above the tailwater elevation, care being taken not to overload the 12-part lines by forcing too much draft on the barge after the tops of the side girders have emerged from the water. Vertical guides on the four 7-1/2-foot

diameter shafts will serve to control lateral movement of the barge. Four 8-part lines from the top of the barge to the bracing between the tops of the shafts will serve to equalize loads on the corners of the barge and to raise the barge above the water out of reach of waves or other turbulence when the caisson is in position on the bucket or stored in the drydock during large floods, thus preventing uncertain stresses in the shafts or excessive wearing of the guides should the barge be kept in continuous motion from these sources.

A large V-frame or anchor frame attached to the upstream corners of the caisson will provide for holding the caisson in position over the bucket during the lowering and raising operations.

There was no precedent to indicate the coefficient of resistance of the unusual shape of the submerged portion of the caisson against currents of various velocities. The magnitude and direction of the velocities in which it would be necessary to navigate were uncertain. Therefore, the evaluation of the line pull on each maneuvering cable during its transfer from point to point in the downstream river channel, to establish the cable size, was a major consideration. It was equally as important to locate the shore ends of the cables to attain optimum maneuverability and to establish the number of lines required to accomplish the maneuvering satisfactorily as well as the maneuvering procedure for the numerous conditions which might exist during the transfer of the caisson to and from the dam. Hydraulic model studies seemed the only practicable means of obtaining answers to these problems.

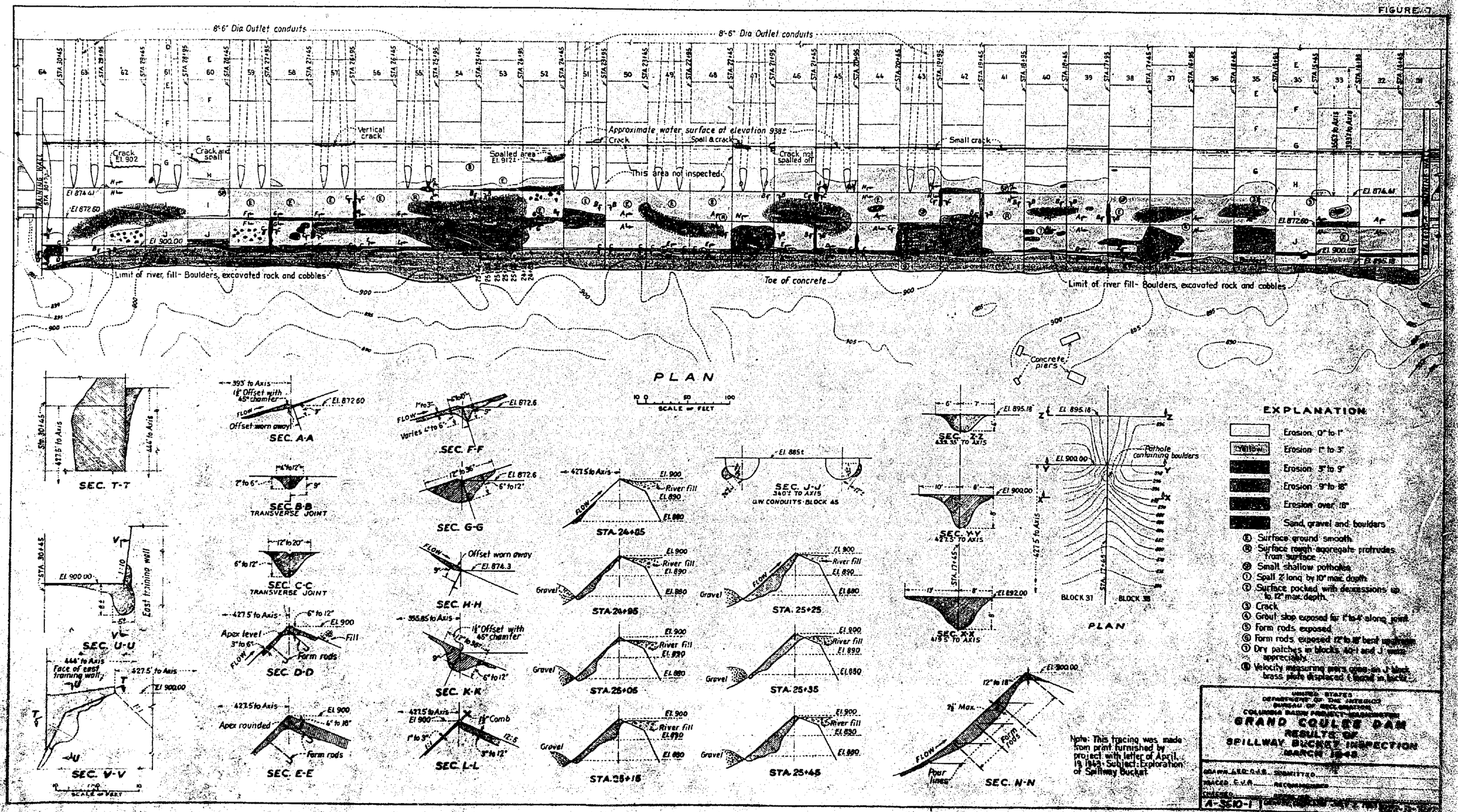
4. Model studies of the inspection and repair caisson. An extensive program was initiated requiring the construction of a 1 to 60 model of Grand Coulee Dam as well as one of the caisson. In view of the complexity of the problem and the large model required, it was proposed to construct the 1 to 60 model at the project in the immediate vicinity of the dam where it would serve for public demonstration after the tests were completed. When the investigation was assigned early in 1941, several locations for the model were considered. The downstream portion

of the parking area at the east end of the prototype structure was selected originally. The model designs and excavation of this site were well advanced when a proposal was made to incorporate the structure with the vista house in the West Vista Park. This required a complete redesign of the model foundation dependent upon the vista house location and architecture, subsequent changes in which prevented the completion of designs and the preparation of construction drawings. By March 1942, plans for the model in its final location were advanced sufficiently to permit construction. The model foundation was finished early in 1943 and the model completed and placed in operation in August of the same year. The tests discussed in this first progress report were conducted between September 1943 and December 1944.

Plans for inspecting part or all of the prototype bucket by divers were formulated during the initial construction on the model. The initial inspection, which concerned the right training wall, the bucket, and the inside of the left training wall, was begun on March 8, 1943, and completed March 28, 1943. The results of this inspection, which indicated damage of sufficient severity to justify rapid completion of the model and an acceleration in preparing plans for the inspection and repair caisson, are contained in a report to the Field Engineer by Engineer L. V. Downs, entitled "Exploration of the Spillway Bucket - Grand Coulee Dam," dated April 15, 1943.

The damage to the spillway bucket reported from the inspection by the divers is shown schematically by colors on figure 7. Briefly, there was slight erosion of the bucket in the construction blocks at the left end of spillway, increasing rapidly toward the center, becoming severe at the junction of construction-block rows 37 and 38, where a deep channel was cut through the bucket lip. Erosion of varying severity was found to the right of this cut with the deepest in construction block rows 52, 53, and 54, between the two banks of thirty outlets on the bucket lip directly downstream of the outlets, and on the inside surface of the right training wall and on the 2-1/2 to 1 slope downstream from the lip adjacent to the training wall. Also, wear exposing pea-sized aggregate was reported below elevation 945 on the downstream

FIGURE 7



Note: This tracing was made from print furnished by project with letter of April 13, 1949. Subject: Exploration of Spillway Bucket.

face of the spillway. This condition was more severe across the central portion than at the ends. The damaged surfaces were reported as having the appearance of being caused by abrasive action rather than by cavitation. Views as to the source of the erosion at various points, based on observations on the 1 to 60 model dam are discussed in subsequent sections of this report.

II. PURPOSE OF MODEL STUDIES

5. Scope of model tests. Though the main purpose of constructing the 1 to 60 model of Grand Coulee Dam, including the downstream river channel and the drydock and floating repair caisson, was to conduct tests for obtaining information relative to maneuvering the inspection and repair caisson from its drydock downstream (shown in the frontispiece) to the dam and return, and to study the behavior of the caisson during the operations of lowering it into and raising it from the spillway bucket, the investigation was expanded to include studies pertaining to important phases of operation of the various hydraulic features of the dam and their relation to conditions inflicting damage to the bucket section. Tests were made also concerning methods for eliminating or minimizing future damage to the bucket section and training walls and to ascertain what measures were necessary to protect the tailrace slopes and river banks from erosion by eddies, high-velocity currents, and wave action to prevent sliding of the banks.

The studies pertaining to the caisson in particular included tests to measure the line forces required in maneuvering the caisson to and from positions over the bucket; to determine the coefficient of resistance of the caisson, moving broadside or head-on into currents of different velocities; to ascertain the positions for the shore ends of the maneuvering lines to give optimum maneuvering conditions with a minimum line pull; to ascertain the magnitude of unbalanced pressures on the caisson should it become necessary to operate the spillway or outlets with the caisson in position on the bucket; to study the stability of the caisson during the sinking and raising operations; and to investigate

operating combinations of spillway gates, outlet conduits, and powerhouse units to produce optimum flow conditions for maneuvering the caisson.

The source of the eroded areas on the spillway bucket and training walls reported by divers in the early spring and late fall of 1943, the measures necessary to eliminate or allay this action, and operating procedures producing optimum conditions for diving operations during the fall inspection were considered in the model tests.

After the feasibility of minimizing the destructive erosion of the spillway bucket and training wall by dredging material from the adjacent river channel had been ascertained, the limits for this work were investigated. The necessity of removing material from the river banks at the entrance of the approach channel to the caisson drydock located about 1,200 feet below the dam, to prevent deposition of bed material and debris in the channel, was investigated also.

A study was made on the 1 to 60 model of the flow conditions in the tailrace and downstream river channel during large flows, to compare the damage in these areas with that indicated in tests made previously on a 1 to 120 scale model and to determine what protective measures would be necessary.

Other investigations on the 1 to 60 model included those concerning the desirability of unsymmetrical operation of the spillway and river outlets; the agreement between certain model and prototype flow characteristics; the rate of recession in the water-surface elevation in the tailrace for power outages of various magnitudes in connection with providing synchronous bypass facilities to prevent sliding of the river banks; the conditions resulting from the operation of the needle-valve outlets installed in the right powerhouse; and the possibility of roughening the face of the model spillway to give the model flow the appearance of the air-saturated flow of the prototype.

Associated problems in preparing the model for studies concerning the caisson included verification tests regarding the river channel roughness required to produce water-surface gradients in the model corresponding to those on the prototype for various discharges;

calibration of the model supply system to obtain data for regulating and ascertaining the flow being supplied to the model; and tests to evolve a means of regulating the river stage (normal tailwater) automatically.

The stability and operating characteristics of the caisson during the floating, submerging, and raising operations were investigated on a 1 to 20 scale model of the caisson and its operating barge.

The feasibility of tipping the huge concrete slab from the gate section of the drydock, where it had been poured to permit the sinking of the drydock structure to its foundation while partially submerged, was investigated on a 1 to 42 scale model of these structures.

A 1 to 60 plaster model of the base of the right training wall and bucket section served to depict the damage in these parts of the spillway and to visualize proposed methods of repair.

The feasibility of various methods proposed for bypassing a portion of the low-water flow at Grand Coulee Dam by the installation of temporary outlet facilities in the right powerhouse turbine bays was investigated using a 1 to 24 model of the Grand Coulee turbine scroll case and draft tube, and a 1 to 30 model of an 84-inch needle valve and the Grand Coulee turbine draft tube.

6. Summary of results. Several tests pertaining to the operation of the 1 to 60 model dam were required to prepare it for the studies concerning the maneuvering of the repair caisson. Among these were the calibration of the model supply system; the establishment of a datum for the various gages; the investigation of the water-surface gradient through the model to determine if roughness and contours of the model river bed gave elevations corresponding to those of the natural channel; and tests to devise some way of regulating the natural stage of the model river automatically.

The calibration of the supply system was accomplished initially by using the model crest as a measuring standard for the preliminary tests and finally by traversing the supply pipes with a standard pitometer to establish an operating curve.

A datum for the gages for measuring the reservoir elevation, the tailwater elevation, and water-surface elevations through the model river channel were established by use of an engineer's level or by simultaneous reading of the gages with water ponded in the river channel.

By accident or otherwise the proper water-surface gradients for natural stages of the river were obtained without the usual alterations and extensive testing.

Automatic regulation of the natural river stage was found impossible through alteration of the tailwater regulating gate provided in the return sump at the downstream end of the model.

The problem of maneuvering the caisson from the drydock, approximately 1,200 feet downstream, to the dam and return by use of maneuvering cables was solved successfully through the investigations on the 1 to 60 model of Grand Coulee Dam. The number of maneuvering lines to transfer the caisson to and from the dam and the most desirable location of the shore ends of these lines to accomplish the transfer, were determined also. The tests indicated that five lines would be required to achieve the maneuvering successfully with minimum line forces. Small torsion dynamometers, designed for the maneuvering tests and attached to the shore ends of the lines yielded data from which the sizes of the prototype lines were established.

The coefficient of resistance of the caisson, head-on and broadside to currents of different velocities, was obtained from the model studies and close agreement found between that for the broadside condition and the value used in initial computations of the forces acting on the structure.

Through a study of the currents existing during operation of the spillway gates, the river outlets, the bypass valves in the right powerhouse, and units in the left powerhouse, it was possible to ascertain the most favorable combination for maneuvering the caisson from the drydock to the dam and returning it to the drydock.

The most favorable currents for this operation, giving minimum line forces, were found to exist when the major portion of the flow

was concentrated through the left powerhouse and over the left end of the spillway, with only three or four of the bypass valves in the right powerhouse discharging. Although this distribution gave minimum velocities against which the caisson would have to navigate, the model indicated little choice between it and the case where the same quantity passed through the powerhouses. Acceptable conditions were indicated on the model for all combinations of flow through the units of both powerhouses. Minimum line forces were indicated for each combination when the caisson was moved from the drydock to the edge of the current from the bypass valves of the right powerhouse, headed directly into this current, ferried across it, and then moved upstream into position over the bucket about 400 to 600 feet inside the right training wall.

Through studies of the currents and the roughness of the water surface on the model, it was possible to establish a program of operation for the spillway to obtain optimum flow conditions for a survey of the bucket during the 1943-44 low-water season. In general, the divers experienced less difficulty during this than on the previous inspection, a fact attributed, in part at least, to the model tests.

Pressures measured on the sides and top of the caisson in the submerged position, with unsymmetrical operation of the spillway and river outlets, indicated that it would be possible to continue work with the caisson only so long as the flows from these sources were kept at an appreciable distance. The flow in the tailrace was quite rough and difficulty might be experienced in transporting and handling repair materials, thus this procedure is not to be followed except in case of emergency.

Moderate unsymmetrical operation of the model spillway indicated only a slight tendency for sand particles to be swept into the bucket by eddy currents in the tailrace. Lesser degrees of symmetry resulted in an increase in the amount and size of sand and gravel entering the bucket. This tendency was accentuated when water-soaked sawdust was placed in the river channel and gave conditions which were believed to be more representative of the prototype action for small flows.

In the opinion of the writer, the source of the damage to the spillway bucket and training walls was indicated clearly in observations

made on the model dam. Tests with the river fill removed for various distances downstream from the spillway toe indicated this type of treatment to be the most feasible and dredging limits for various discharges were established. A study was made of the erosion, by wave action and currents from the spillway, of the tailrace slopes and downstream river banks. Sufficient tests were conducted to indicate the treatment required to reduce this destruction to a minimum, but extensive tests are still to be conducted for determining the extent of this treatment, which involves flattening the slopes and banks and riprapping them heavily.

A tendency for river-bed material to deposit in the entrance channel to the drydock was noted during these tests. Removal of the banks extending into the river from each side of the drydock entrance channel decreased the amount of deposit but did not eliminate the action.

Although several attempts were made to evolve a means of eliminating the condition causing the damage to the spillway training walls, all except that of removing the material from the river bed nearby seemed impracticable.

Attention was focused on degrees of similarity or dissimilarity of model and prototype discovered during the test program. Relatively, the flow conditions below the model spillway for small floods were not as turbulent as the prototype and there was less erosion.

Striking similarity was noted in the behavior of a rock suspended on a string in the vicinity of the left training wall and that of the sounding equipment used on the prototype to determine the river-bed elevation in this region.

Attempts were made to reproduce the inflow of material noted in the bucket of the prototype during diving operations, but it was possible to demonstrate this characteristic only by using silt or very lightweight materials in the river channel of the model.

The rate of recession of the water surface in the tailrace for various power outages indicated that a drop of from 3.0 to 8.0 feet in 38 minutes was to be expected for outages reducing the flow by 19,000 and 37,000 second-feet.

None of the white, air-saturated water so prevalent on the prototype was present on the model. Out-of-scale factors and the small velocities on the model were considered contributing factors.

The injection of air into the spillway flow and roughening the face of the model by fastening hardware cloth of various meshes to it, failed to produce the appearance of the air-saturated water on the prototype. Though roughening the model face gave better results than air injecting, this treatment was considered impracticable and additional tests were not made.

Throughout the test program there were no indications that the needle-valve outlets in the right powerhouse would become a source of trouble insofar as conditions were concerned in the tailrace or downstream river channel.

When the model studies were initiated to develop the caisson for the repair and maintenance of the spillway bucket, it was hoped that tests on a 1 to 60 scale model of the caisson would provide ample information as to the stability of equilibrium of the structure during the operations of maneuvering it to and from the dam, lowering it into the bucket, and refloating it. However, since it was impracticable to attain exact similarity on such a small size, the studies, except in the case of the maneuvering tests, in which only the shape and draft of the vessel were of major importance, were unsuccessful and a 1 to 20 scale model of the caisson, its operating barge, equipped with powered hoists, and a section of the toe of the dam, including the bucket profile, was constructed.

Through the investigations on this model, it was ascertained that the stability of equilibrium of the caisson was good so long as air was not allowed to become trapped in the high points of the working space at each end of the structure as it was submerged, and that it could be lowered into and raised from the bucket satisfactorily when this was the case. Adequate vent systems placed in the high points at the ends of the working space were necessary to prevent the entrapment of air in these regions and assure adequate stability.

Other information obtained from operating this model indicated that the submerging and lowering of the caisson could best be accomplished if the vessel were trimmed level for the initial stages of lowering it into the bucket; retrimmed as it neared the bucket to lower the upstream end into the bucket ahead of the downstream end, preventing a sliding contact between the seal and the jagged concrete surfaces; and the longitudinal axis kept perpendicular to the axis of the dam during this operation to insure optimum conditions for sealing. Conditions were found to be most favorable for raising the caisson to the floating position when this procedure was reversed. Good agreement was obtained between experimental values of the metacentric height and the location of the center of gravity and those computed for the full-sized structure by the design section. It was determined also that the operating barge should under no conditions be raised above the water surface in the barge well in the deck when there is any appreciable amount of water in this well, particularly during the sinking and raising operations.

That it was feasible to tip the reinforced slab from the gate opening of the drydock into an excavated area in the entrance channel adjacent to the structure when the slab was partially submerged seemed assured by tests performed on the 1 to 42 scale model of the slab and drydock. The problem of locating the center of gravity of the model slab experimentally by balancing was undertaken and the force required for tipping the slab obtained (assuming no bond between surfaces of the slab and the gate opening). That it would be possible under certain conditions to free the slab in case there was bond between its surfaces and those of the drydock or to tip it by pumping water into the drydock to raise the surface inside above that in the entrance channel, was indicated also.

The damage to the right spillway training wall and concrete slope below the bucket lip adjacent to it, and proposed repairs were portrayed effectively by the use of a 1 to 60 scale model of the base training wall and bucket section. An idea as to the practicability of proposed repairs in this section of the spillway was obtained by reproducing them on this model.

A satisfactory plan was evolved from studies on a 1 to 24 scale model of the Grand Coulee turbine scroll case and draft tube and a 1 to 30 model of an 84-inch needle valve and the draft tube for providing temporary outlet facilities in the right powerhouse to discharge surplus water downstream from the spillway during the low-water season to facilitate the repair work and prevent damage which would result from unsymmetrical operation of the drum gates or river outlets while repairs were being made.

III. THE MODELS

7. Description of the models. Eight models were used in making the investigations discussed in this report. A 1 to 60 hydraulic model of Grand Coulee Dam, with its appurtenances and approximately one mile of the downstream river channel, principally of concrete, and a model of the bucket inspection and repair caisson of the same scale constructed of transparent plastic were used for the majority of the tests, particularly those concerning the maneuvering of the caisson, the conditions damaging the spillway bucket, the minimization or elimination of the damage, and the protection of the tailrace slopes and river banks downstream.

A 1 to 60 scale caisson model of sheet iron and metal tubing, equipped with piezometers and constructed to fit the spillway bucket of the model dam provided a means for studying unbalanced pressure conditions on the caisson under unsymmetrical operation of the drum gates and river outlets.

A 1 to 20 scale model of the caisson, including all important features to scale, constructed mainly of sheet metal, was used to study the stability and other characteristics of the design during the floating, sinking, and raising operations.

A 1 to 42 model of the caisson drydock below elevation 959.5, including the gate slab, which is to be tipped into the river after the caisson is fabricated and ready for use, constructed of concrete, was used to ascertain the feasibility of tipping the slab into the river when water was surrounding it.

A 1 to 60 scale plaster model of the right training wall and bucket section was constructed to more effectively visualize the damage which has taken place at the base of the wall and to study proposed repairs.

A 1 to 24 model of the Grand Coulee turbine scroll case and draft tube, constructed of transparent plastic, and a 1 to 30 scale model of the draft tube and an 84-inch needle valve, the draft tube of concrete and the model valve of brass, were used in determining the feasibility of proposed methods of providing outlet capacity in which the flow was discharged downstream from the dam.

8. The 1 to 60 model of Grand Coulee Dam. When plans concerning the inspection and repair of the spillway bucket at Grand Coulee Dam had progressed to the point where model studies were considered the only practicable means of establishing the feasibility of the proposed scheme, consideration was given to the construction of an operating model of the dam with its hydraulic features and about a mile of the downstream river channel in the laboratories of the Denver office. A large model was required to assure dependable results and space was not available for the 1 to 60 scale chosen, thus a proposal was made to construct it on the project near the prototype where it would benefit project personnel and could be used for public demonstration after the completion of the test program. Several sites were considered before one in the downstream portion of the parking area at the east end of the dam was selected. Designs for the model were begun and excavation of the foundation started when it was proposed instead to include the structure in the vista house to be constructed in the Vista Park area above the west abutment of the dam.

The model and its foundation, constructed principally of concrete, was completed in this location in August 1943. It consisted of the model dam, figures 8, 9, 10, and 11, a water-supply and recirculating system, figure 12, a section of river channel representing approximately one mile of the prototype, a subterranean storage basin beneath the model, and a downstream sump with a concrete conduit to return the model flow to the storage basin for recirculation, figure 13. The

FIGURE 8

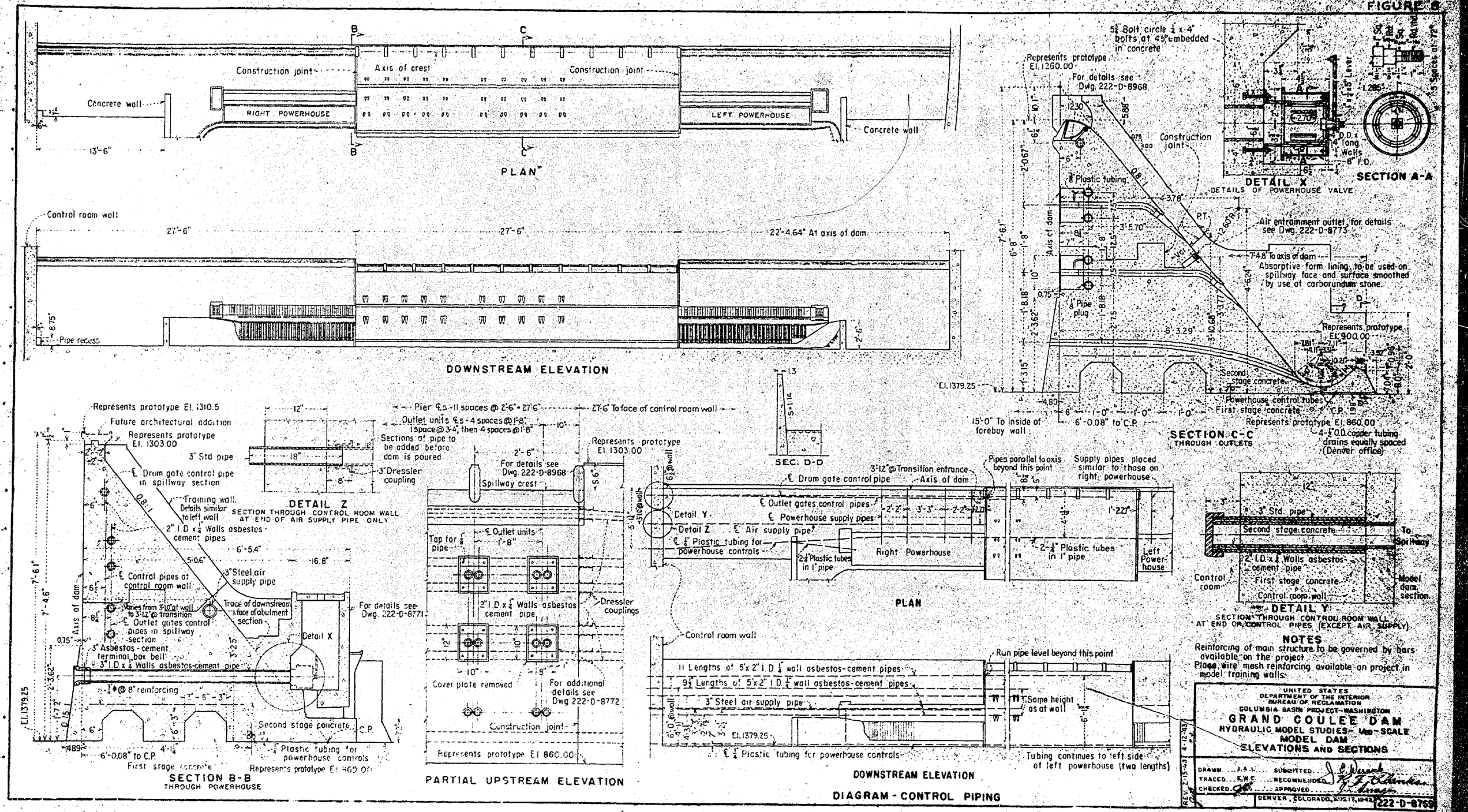

PLAN
Construction joint
Axis of crest
Concrete wall
RIGHT POWERHOUSE
LEFT POWERHOUSE
13'-6"
DOWNSTREAM ELEVATION
Control room wall
27'-6"
22'-4.64" At axis of dam
Pipe recess
DETAIL X
DETAILS OF POWERHOUSE VALVE
SECTION A-A
Represents prototype El. 1260.00
For details see Dwg. 222-D-8968
Air entrainment outlet, for details see Dwg. 222-D-8773
Absorptive form lining, to be used on spillway face and surface smoothed by use of carborundum stone.
Represents prototype El. 900.00
Second stage concrete
Powerhouse control tubes
First stage concrete
Represents prototype El. 860.00
4-1" O.D. copper tubing drains equally spaced (Denver office)
SECTION C-C
THROUGH OUTLETS
SEC. D-D
Represents prototype El. 1310.5
Future architectural addition
Represents prototype El. 1303.00
3" Std pipe
Drum gate control pipe in spillway section
Training wall. Details similar to left wall
2" I.D. x 1/4 Walls asbestos-cement pipes
3" Steel air supply pipe
Control pipes at control room wall
Trace of downstream face of abutment section
Outlet gates control pipes in spillway section
3" Asbestos-cement terminal box bell
3" I.D. x 1/4 Walls asbestos-cement pipe
Second stage concrete
First stage concrete
Plastic tubing for powerhouse controls
Represents prototype El. 860.00
SECTION B-B
THROUGH POWERHOUSE
DETAIL Z
SECTION THROUGH CONTROL ROOM WALL AT END OF AIR SUPPLY PIPE ONLY
Pier ℄s - 11 spaces @ 2'-6" = 27'-6"
27'-6" To face of control room wall
Outlet units ℄s - 4 spaces @ 1'-8"
1 space @ 3'-4", then 4 spaces @ 1'-8"
Sections of pipe to be added before dam is poured
Represents prototype El. 1303.00
Spillway crest
Tap for pipe
Outlet units
2" I.D. x 1/4 Walls asbestos cement pipe
Dressler couplings
Cover plate removed
For additional details see Dwg. 222-D-8772
Construction joint
Represents prototype El. 860.00
PARTIAL UPSTREAM ELEVATION
Drum gate control pipe
Axis of dam
Pipes parallel to axis beyond this point
Supply pipes placed similar to those on right powerhouse
Outlet gates control pipes
Powerhouse supply pipes
Air supply pipe
Plastic tubing for powerhouse controls
Right Powerhouse
Left Powerhouse
PLAN
DETAIL Y
SECTION THROUGH CONTROL ROOM WALL AT END OF CONTROL PIPES (EXCEPT AIR SUPPLY)
Control room wall
11 Lengths of 5' x 2" I.D. 1/4 wall asbestos-cement pipes
9 1/2 Lengths of 5' x 2" I.D. 1/4 wall asbestos-cement pipes
3" Steel air supply pipe
Run pipe level beyond this point
Same height as at wall
Plastic tubing for powerhouse controls
Tubing continues to left side of left powerhouse (two lengths)
DOWNSTREAM ELEVATION
DIAGRAM - CONTROL PIPING
NOTES
Reinforcing of main structure to be governed by bars available on the project
Place wire mesh reinforcing available on project in model training walls
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT-WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES- 1/40-SCALE
MODEL DAM
ELEVATIONS AND SECTIONS
DRAWN J.A.L.
TRACED E.H.C.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO, MAY 17, 1942
222-D-8769

FIGURE 9

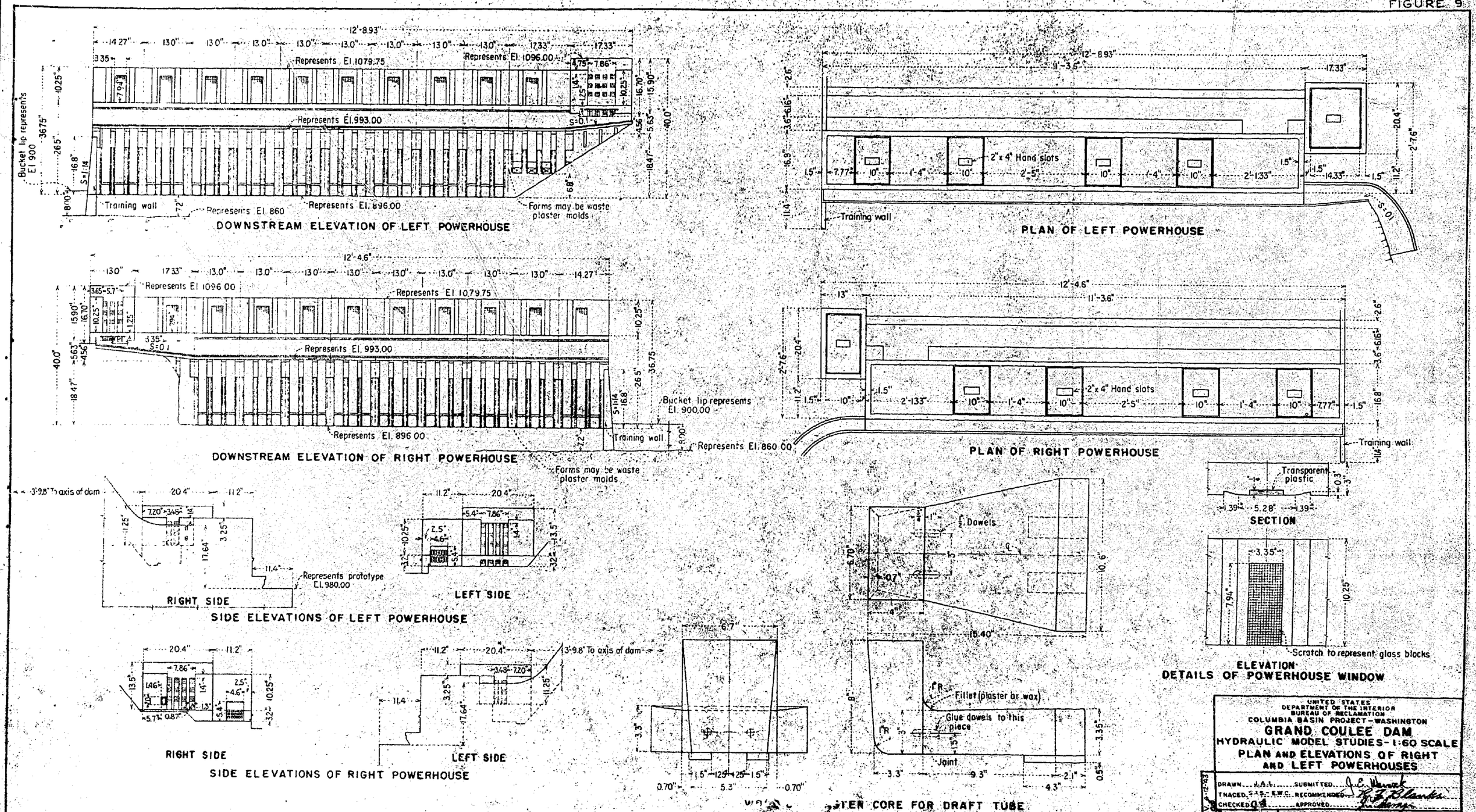
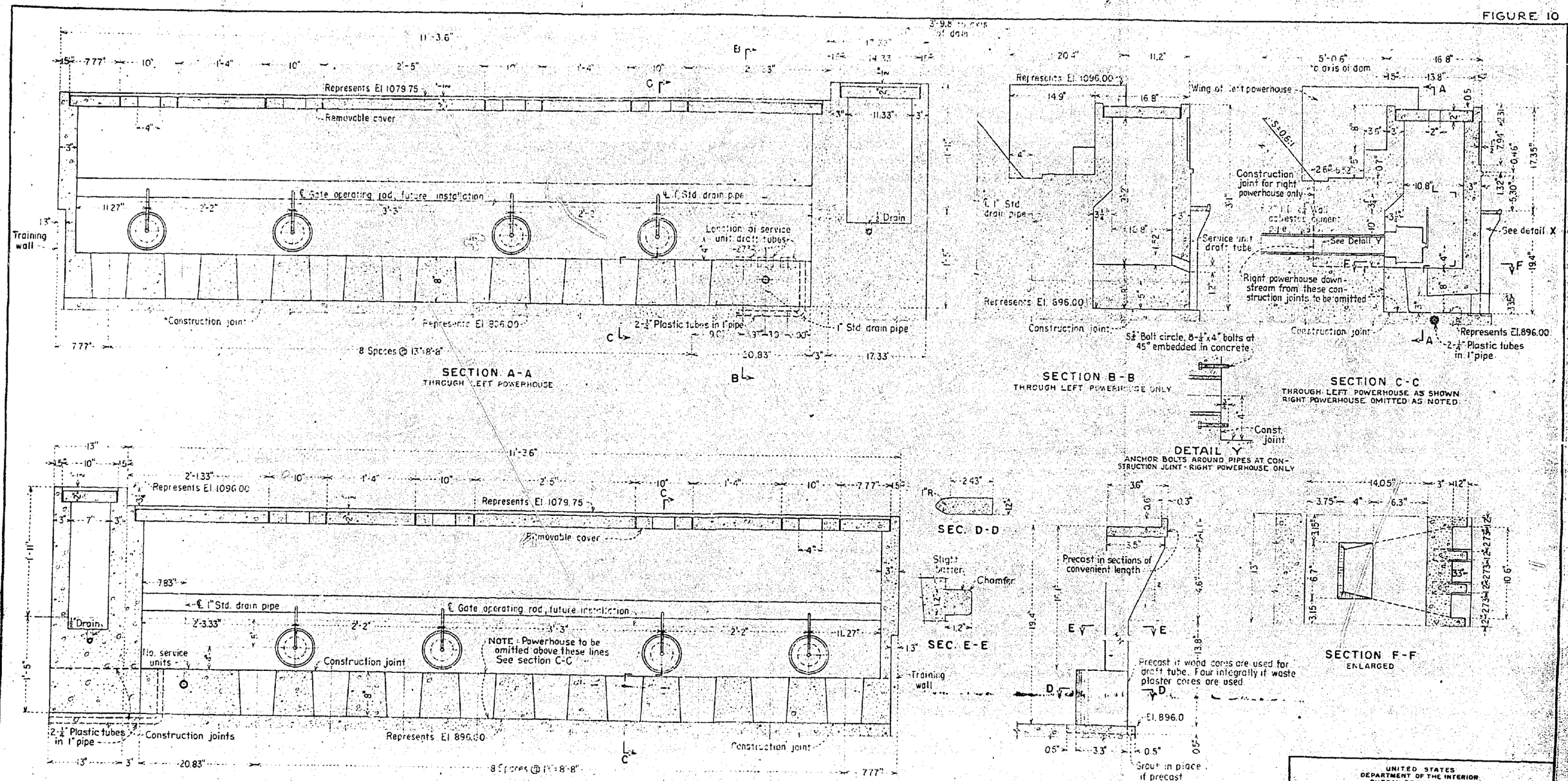

DOWNSTREAM ELEVATION OF LEFT POWERHOUSE
Represents El. 1079.75
Represents El. 1096.00
Represents El. 993.00
Bucket lip represents El. 900
Training wall
Represents El. 860
Represents El. 896.00
Forms may be waste plaster molds
PLAN OF LEFT POWERHOUSE
2" x 4" Hand slots
Training wall
DOWNSTREAM ELEVATION OF RIGHT POWERHOUSE
Represents El. 1096.00
Represents El. 1079.75
Represents El. 993.00
Bucket lip represents El. 900.00
Represents El. 896.00
Training wall
Represents El. 860.00
Forms may be waste plaster molds
PLAN OF RIGHT POWERHOUSE
2" x 4" Hand slots
Training wall
3'-9.8" To axis of dam
Represents prototype El. 980.00
RIGHT SIDE
LEFT SIDE
SIDE ELEVATIONS OF LEFT POWERHOUSE
RIGHT SIDE
LEFT SIDE
SIDE ELEVATIONS OF RIGHT POWERHOUSE
Transparent plastic
SECTION
Dowels
Fillet (plaster or wax)
Glue dowels to this piece
Joint
Scratch to represent glass blocks
ELEVATION
DETAILS OF POWERHOUSE WINDOW
WATER CORE FOR DRAFT TUBE
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES—1:60 SCALE
PLAN AND ELEVATIONS OF RIGHT AND LEFT POWERHOUSES
DRAWN H.A.L. SUBMITTED
TRACED S.A.S. - E.W.C. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO, AUG. 17, 1942.
222-D-8770

FIGURE 10



NOTES

Place wire mesh reinforcing as located on project in all powerhouse walls.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT-WASHINGTON

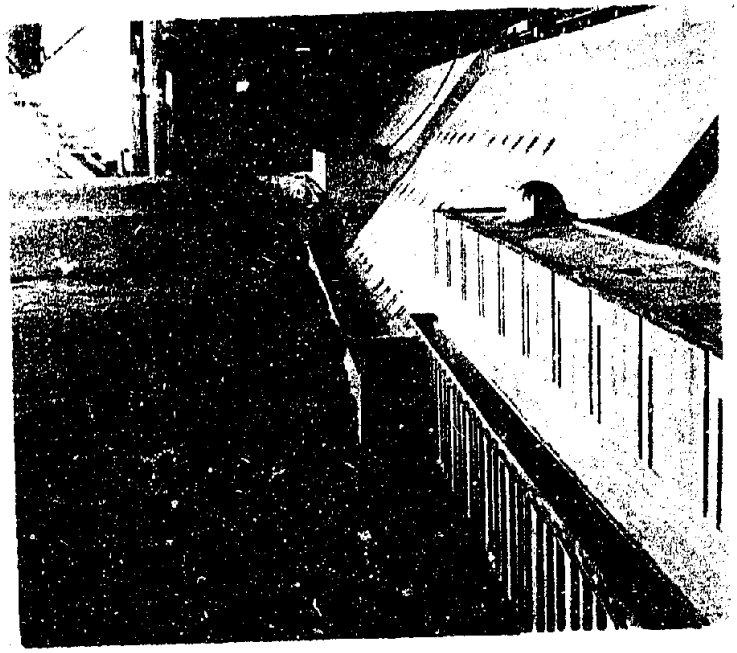
GRAND COULEE DAM

HYDRAULIC MODEL STUDIES - 1:60 SCALE

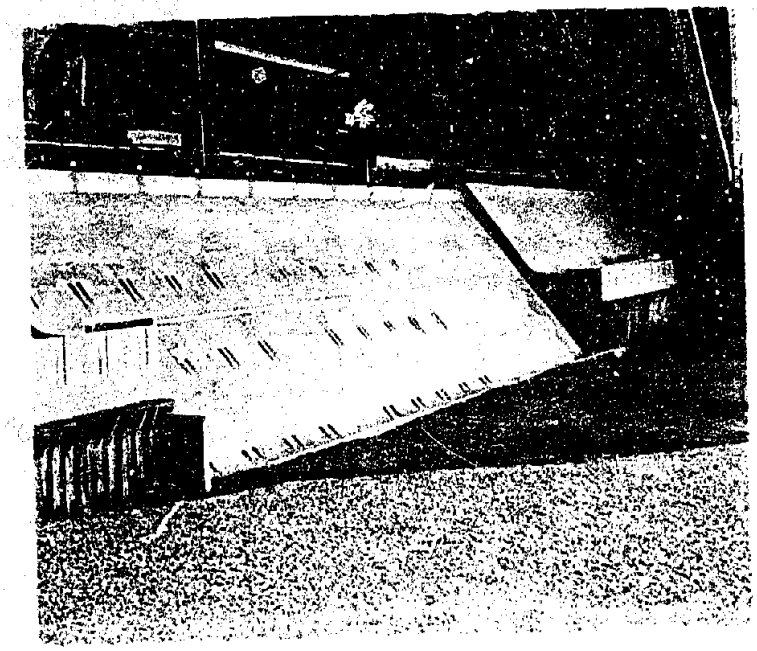
SECTIONS OF RIGHT AND LEFT POWERHOUSES

DRAWN H.R.B. J.A.L. SUBMITTED J. C. Warnock
TRACED E.D.L. E.W.C. H.R.S. RECOMMENDED R. F. Blanke
CHECKED APPROVED
DENVER, COLORADO, AUG. 17, 1942 222-D-8771

REV. 4-12-43 JAN. 6



A. Model dam and river channel from left bank



B. Model dam and river channel from right bank

MODEL OF GRAND COULEE DAM (1:60 SCALE) WITH DOWNSTREAM RIVER CHANNEL PREPARED FOR TESTING

FIGURE 12

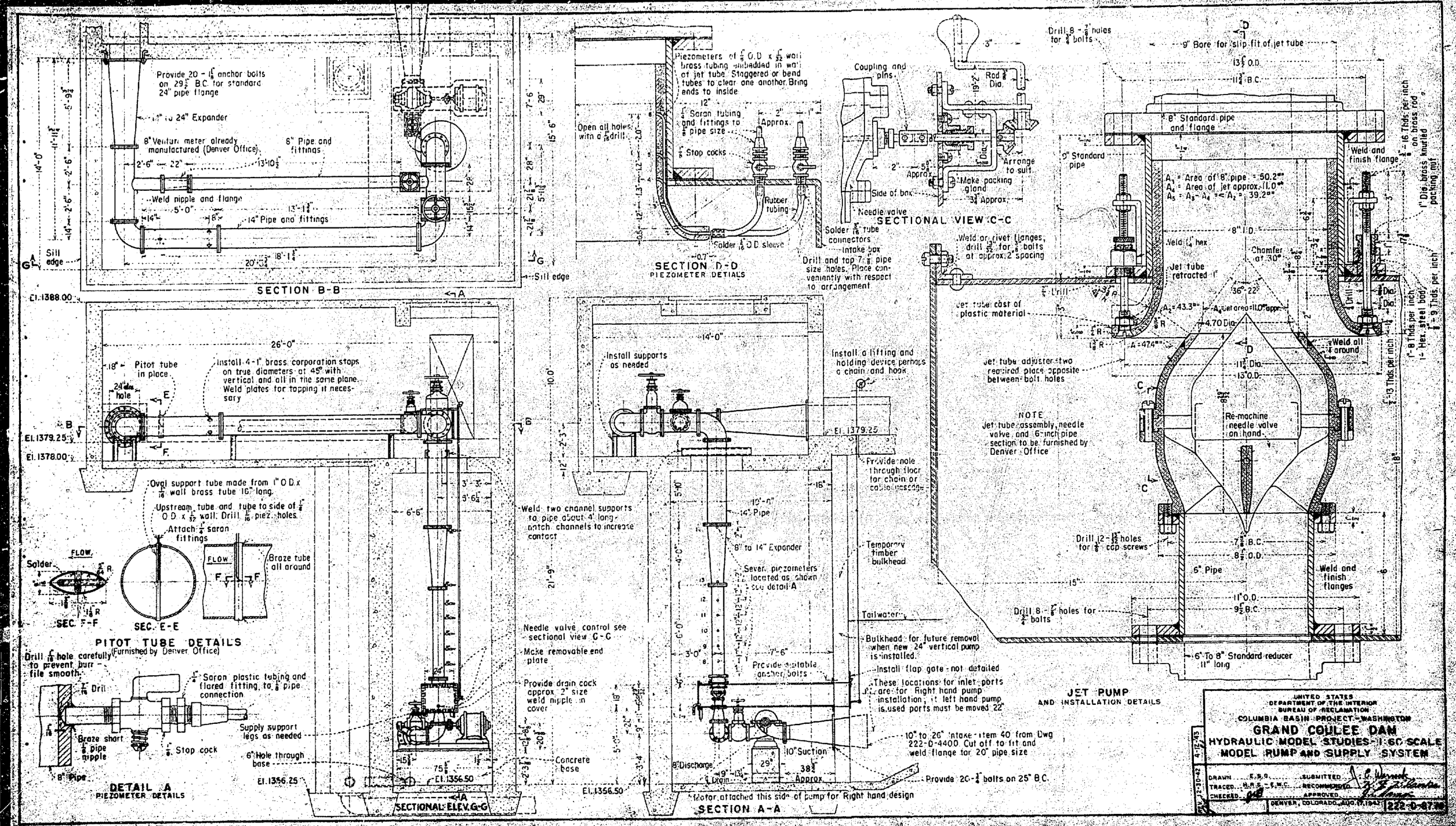
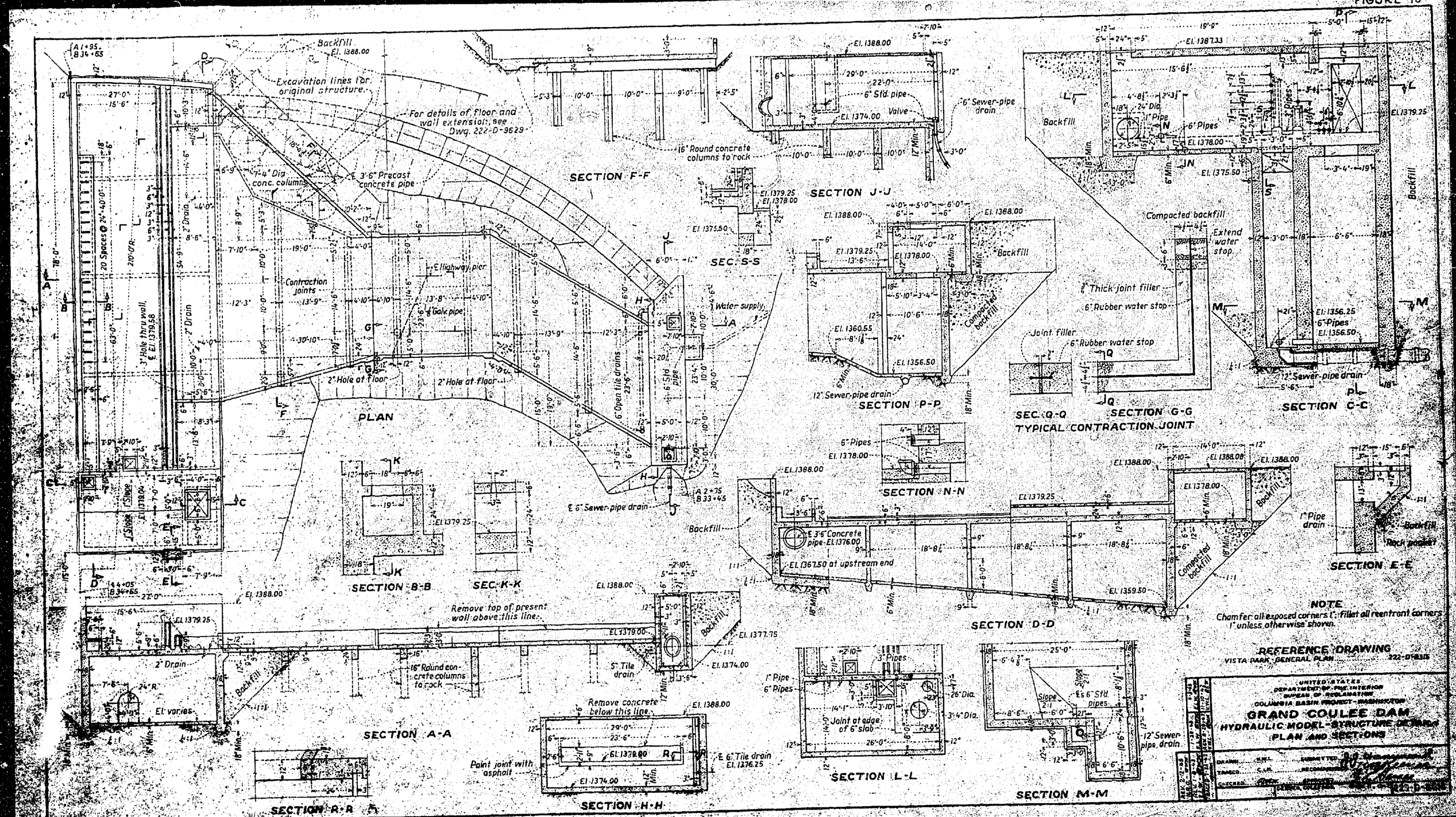

SECTION B-B
Provide 20 - 1 1/8" anchor bolts on 29 1/2" B.C. for standard 24" pipe flange
1" to 24" Expander
8" Venturi meter already manufactured (Denver Office)
6" Pipe and fittings
Weld nipple and flange
14" Pipe and fittings
Sill edge
El. 1388.00
SECTION D-D
PIEZOMETER DETAILS
Piezometers of 1/8" O.D. x 1/32" wall brass tubing embedded in wall of jet tube. Staggered or bend tubes to clear one another. Bring ends to inside
1/8" Saran tubing and fittings to 1/8" pipe size
1/8" Stop cocks
Open all holes with a 1/16" drill
Rubber tubing
Solder 1/16" O.D. sleeve
SECTIONAL VIEW C-C
Coupling and pins
Rod 3/8" Dia.
Arrange to suit
Make packing gland
Side of box
Needle valve
Solder 1/16" tube connectors
Intake box
Drill and tap 7 - 1/8" pipe size holes. Place conveniently with respect to arrangement
Weld or rivet flanges, drill 5/8" for 1/8" bolts at approx. 2" spacing
Jet tube cast of plastic material
Jet tube adjuster two required place opposite between bolt holes
NOTE
Jet tube assembly, needle valve and 6-inch pipe section to be furnished by Denver Office
Drill 8 - 7/8" holes for 3/4" bolts
9" Bore for slip fit of jet tube
13 1/2" O.D.
11 3/4" B.C.
8" Standard pipe and flange
9" Standard pipe
Weld and finish flange
A1 = Area of 8" pipe = 50.2□"
A4 = Area of jet approx. 11.0□"
A5 = A3 = A4 = A2 = 39.2□"
8" I.D.
Weld 1/4" hex
Chamfer at 30°
Jet tube retracted 1"
A2 = 43.3□"
A4 = Jet area = 11.0□" appr.
4.70 Dia.
A = 474□"
Weld all around
Re-machine needle valve on hand
Drill 12 - 13/32" holes for 1/8" cap screws
7 5/8" B.C.
8 1/2" O.D.
6" Pipe
Weld and finish flanges
15"
11" O.D.
9 1/2" B.C.
Drill 8 - 7/8" holes for 3/4" bolts
6" To 8" Standard reducer 11" long
1" Dia. brass knurled packing nut
1" - 16 Thds. per inch on brass rod
1" - 8 Thds. per inch
1" - 9 Thds. per inch
1" - 13 Thds. per inch
JET PUMP
AND INSTALLATION DETAILS
26'-0"
Pitot tube in place
24" dia. hole
Install 4 - 1" brass corporation stops on true diameters at 45° with vertical and all in the same plane. Weld plates for tapping if necessary
El. 1379.25
El. 1378.00
Oval support tube made from 1" O.D. x 1/16" wall brass tube 16" long.
Upstream tube and tube to side of 1/8" O.D. x 1/32" wall. Drill 1/16" piez. holes.
Attach 1/8" saran fittings
Solder
FLOW
Braze tube all around
SEC. F-F
SEC. E-E
PITOT TUBE DETAILS
(Furnished by Denver Office)
Drill 1/16" hole carefully to prevent burr - file smooth
1/8" Saran plastic tubing and flared fitting to 1/8" pipe connection
1/16" Drill
Braze short 1/8" pipe nipple
1/8" Stop cock
8" Pipe
DETAIL A
PIEZOMETER DETAILS
Install supports as needed
Weld two channel supports to pipe about 4' long - notch channels to increase contact
Needle valve control see sectional view C-C
Make removable end plate
Provide drain cock approx. 2" size weld nipple in cover
Supply support legs as needed
6" Hole through base
Concrete base
El. 1356.25
El. 1356.50
SECTIONAL ELEV. G-G
Install a lifting and holding device perhaps a chain and hook
El. 1379.25
Provide hole through floor for chain or cable passage
10'-6" 14" Pipe
8" to 14" Expander
Temporary timber bulkhead
Sever piezometers located as shown see detail A
Tailwater
Bulkhead for future removal when new 24" vertical pump is installed.
Install flap gate - not detailed
These locations for inlet ports are for Right hand pump installation, if left hand pump is used ports must be moved 22"
Provide suitable anchor bolts
10" to 26" intake - item 40 from Dwg 222-D-4400. Cut off to fit and weld flange for 20" pipe size
10" Suction
8" Discharge
9" Drain
Provide 20 - 1/4" bolts on 25" B.C.
Motor attached this side of pump for Right hand design
SECTION A-A
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES - 1/60 SCALE
MODEL PUMP AND SUPPLY SYSTEM
DRAWN E.S.G.
TRACED H.R.S. - F.W.C.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO, AUG. 17, 1942
222-D-4730

FIGURE 13

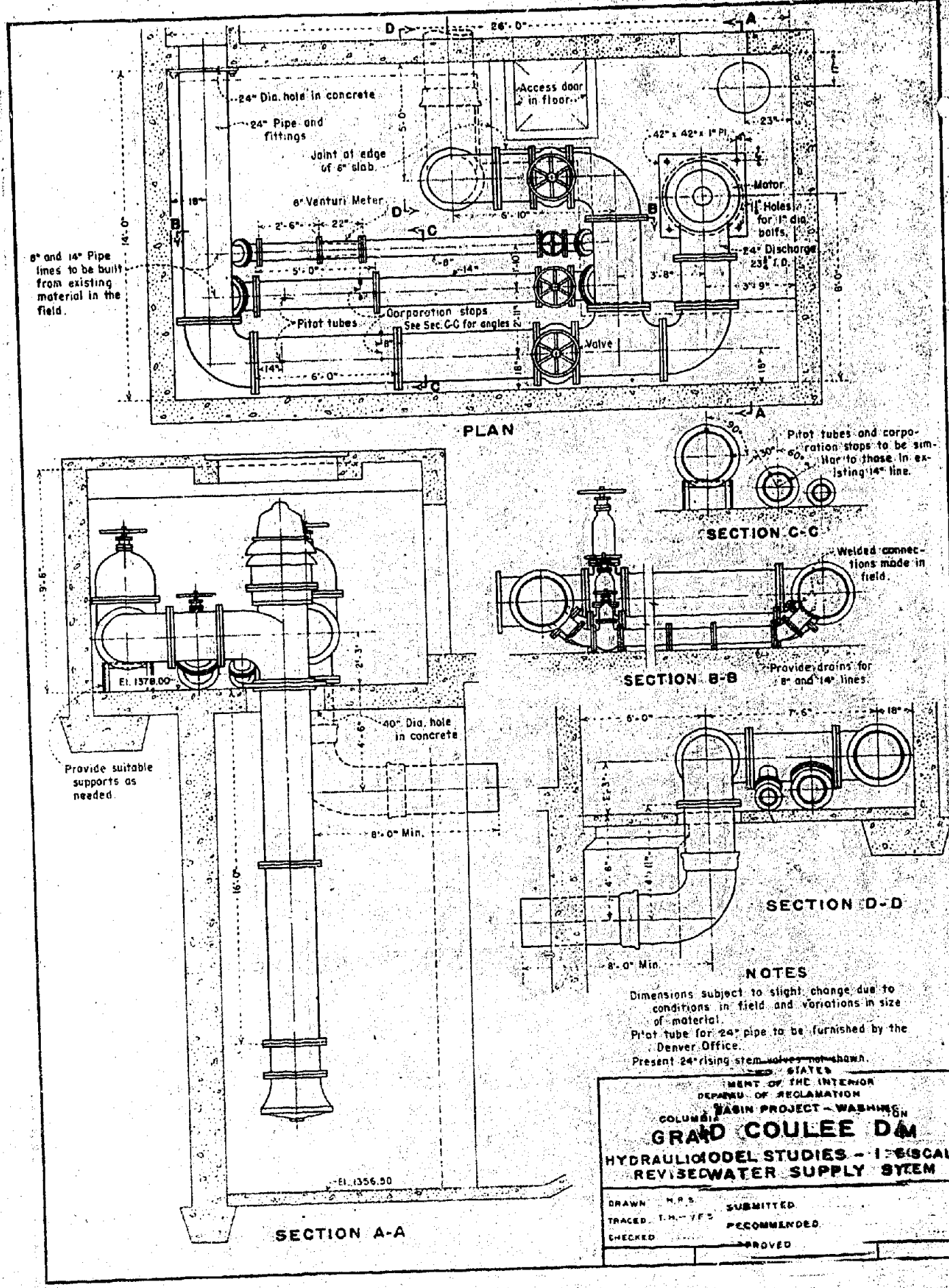

PLAN
SECTION F-F
SECTION J-J
SEC. S-S
SECTION P-P
SEC. Q-Q
SECTION G-G
TYPICAL CONTRACTION JOINT
SECTION C-C
SECTION N-N
SECTION B-B
SEC. K-K
SECTION D-D
SECTION E-E
SECTION A-A
SECTION R-R
SECTION H-H
SECTION L-L
SECTION M-M
NOTE
Chamfer all exposed corners 1"; fillet all reentrant corners 1" unless otherwise shown.
REFERENCE DRAWING
VISTA PARK GENERAL PLAN
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL - STRUCTURE DETAILS
PLAN AND SECTIONS

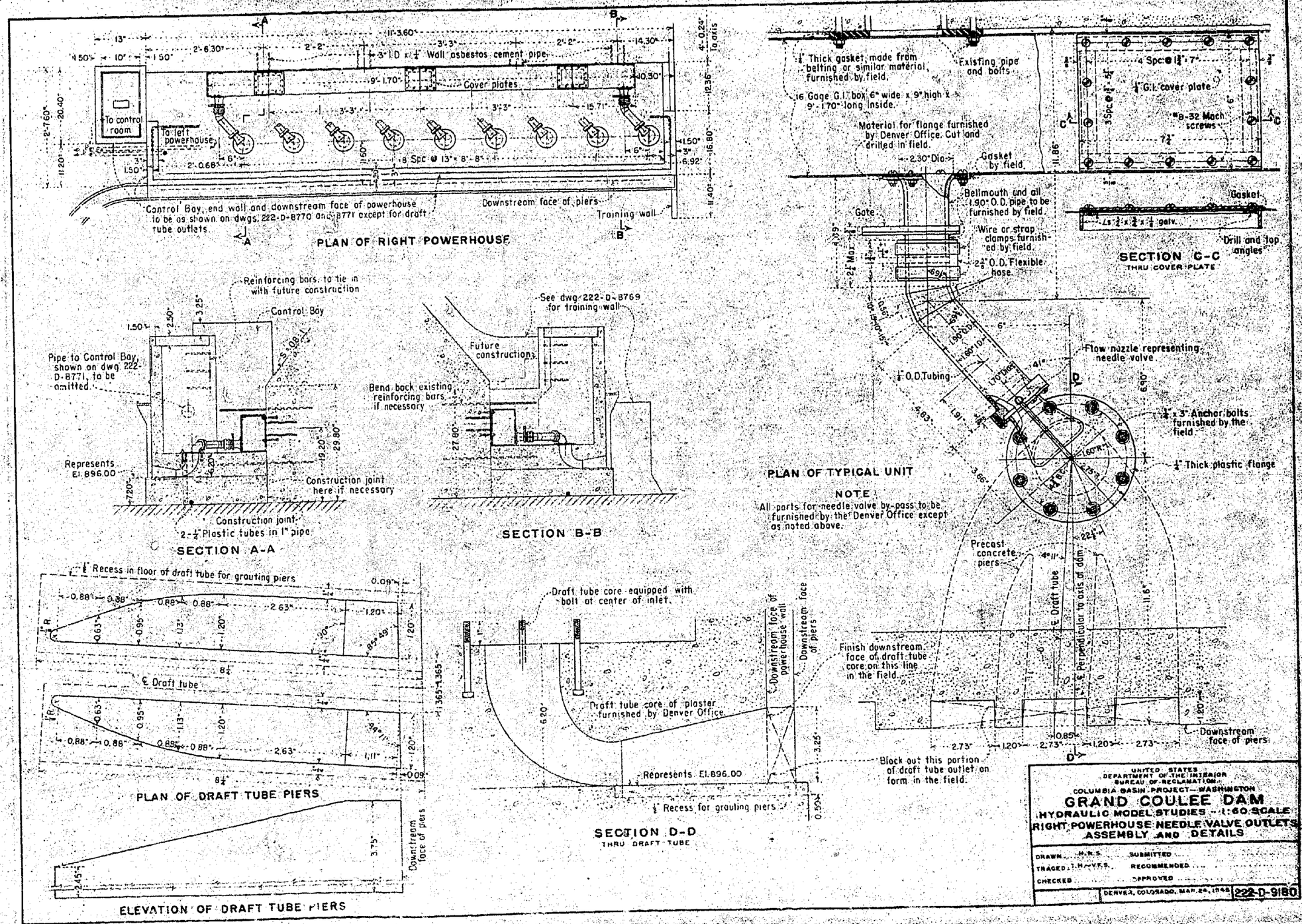
storage reservoir was founded on rock which sloped downward toward the right end of the basin providing good drainage to the pump sump. Three sides of the basin extended to the ground surface to form its walls and the boundaries of the model reservoir, while the fourth or downstream side terminated in the covering slab over the subterranean basin which formed the floor of the reservoir, 8 feet 9 inches below the ground surface.

The wall at one end of the reservoir formed one side of a control room constructed over the pump sump, 6.5 by 10.5 by 21.5 feet deep, which was designed to accommodate a 24-inch vertical propeller pump having a capacity of 35.86 second-feet, a quantity representing a prototype flow of 1,000,000 second-feet. At the time the model was built it was not possible to purchase a pump of this type and a jet pump utilizing available high-head equipment was installed, figure 12. Later, when the jet pump proved inadequate for some of the tests, a second-hand, 24-inch propeller pump was purchased and installed, figure 14. The control room was intended to house the pump, supply piping, and model controls for demonstrating to the public. A 24- by 30-inch concrete conduit having 21 rectangular openings with baffles to quiet and distribute the flow as it was discharged into the reservoir, was placed along the upstream side of the model reservoir.

The dam proper, a concrete structure 7-1/2 feet high, 80 feet long, and approximately 8-1/4 feet thick at the base, containing facilities for controlling the flow from the hydraulic features, was constructed on the storage-basin cover slab where it formed the downstream end of the reservoir. Flow to the model powerhouses was supplied through eight 3-inch asbestos-concrete pipes embedded in the dam. All the pipes were provided with bellmouth entrances to minimize head losses. Cylinder valves of plastic, with deflectors to quiet the water were attached to the downstream end of those leading to the left power plant, while a special header tank with circular slide valves, flow nozzles, and plastic transition elbows representing the prototype needle-valve bypass installation, was placed in the right powerhouse, figure 15. In both cases the water was discharged through draft tubes in the bases of the powerhouses.

FIGURE 14


PLAN
24" Dia. hole in concrete
24" Pipe and fittings
Joint at edge of 6" slab
Access door in floor
6" Venturi Meter
8" and 14" Pipe lines to be built from existing material in the field.
Pitot tubes
Corporation stops See Sec. C-C for angles
Valve
Motor
42" x 42" x 1" Pl.
Holes for 1" dia. bolts
24" Discharge 23¾" I.D.
26'-0"
14'-0"
8'-0"
6'-10"
5'-0"
3'-8"
3'-9"
6'-0"
SECTION C-C
Pitot tubes and corporation stops to be similar to those in existing 14" line.
SECTION B-B
Welded connections made in field.
Provide drains for 8" and 14" lines.
SECTION D-D
6'-0"
7'-6"
8'-0" Min.
SECTION A-A
El. 1378.00
40" Dia. hole in concrete
Provide suitable supports as needed.
8'-0" Min.
16'-0"
El. 1356.50
9'-6"
NOTES
Dimensions subject to slight change due to conditions in field and variations in size of material.
Pitot tube for 24" pipe to be furnished by the Denver Office.
Present 24" rising stem valves not shown.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT – WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES – 1:6 SCALE
REVISED WATER SUPPLY SYSTEM
DRAWN H.R.S.
TRACED T.H.-V.F.S.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED


PLAN OF RIGHT POWERHOUSE
To control room
To left powerhouse
3" I.D. x 1/4" Wall asbestos cement pipe
Cover plates
8 Spc @ 13" x 8'-8"
Control Bay, end wall and downstream face of powerhouse to be as shown on dwgs. 222-D-8770 and 8771 except for draft tube outlets.
Downstream face of piers
Training wall
SECTION A-A
Reinforcing bars, to tie in with future construction
Control Bay
Pipe to Control Bay shown on dwg. 222-D-8771, to be omitted.
Represents El. 896.00
Construction joint here if necessary
Construction joint
2-1/4" Plastic tubes in 1" pipe
SECTION B-B
See dwg. 222-D-8769 for training wall
Future construction
Bend back existing reinforcing bars if necessary
PLAN OF TYPICAL UNIT
NOTE
All parts for needle valve by-pass to be furnished by the Denver Office except as noted above.
1/4" Thick gasket made from belting or similar material, furnished by field.
16 Gage G.I. box 6" wide x 9" high x 9'-1.70" long inside.
Existing pipe and bolts
Material for flange furnished by Denver Office. Cut and drilled in field.
Gasket by field.
Bellmouth and all 1.90" O.D. pipe to be furnished by field.
Wire or strap clamps furnished by field.
2 1/2" O.D. Flexible hose.
Gate
1/4" O.D. Tubing
Flow nozzle representing needle valve.
1/4" x 3" Anchor bolts furnished by the field.
1/4" Thick plastic flange
4 Spc @ 1 1/2" x 7"
3 Spc @ 4"
1/4" G.I. cover plate
8-32 Mach screws
Gasket
Ls 2" x 2" x 1/4" galv.
Drill and tap angles
SECTION C-C
THRU COVER PLATE
Recess in floor of draft tube for grouting piers
℄ Draft tube
PLAN OF DRAFT TUBE PIERS
ELEVATION OF DRAFT TUBE PIERS
Downstream face of piers
Draft tube core equipped with bolt at center of inlet.
Draft tube core of plaster furnished by Denver Office
Downstream face of powerhouse wall
Downstream face of piers
Represents El. 896.00
1/4" Recess for grouting piers
SECTION D-D
THRU DRAFT TUBE
Finish downstream face of draft tube core on this line in the field.
Block out this portion of draft tube outlet on form in the field.
Precast concrete piers
℄ Draft tube
℄ Perpendicular to axis of dam
Downstream face of piers
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES—1:60 SCALE
RIGHT POWERHOUSE NEEDLE VALVE OUTLETS
ASSEMBLY AND DETAILS
DRAWN H.R.S.
TRACED J.H.—V.F.S.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO, MAR. 24, 1948
222-D-9180

The river outlets were constructed from 1-1/2-inch standard brass pipe bored and reamed to 1.7-inch inside diameter and curved to the proper profile (figure 16) by employing a special bending device (figure 17) developed for that purpose in the project shops. For the lower tubes, with a long radius curve, the device consisted of a stationary grooved block curved to a radius about one pipe diameter less than that of the finished dimension, to allow for the tendency of the pipe to straighten when the bending forces were released; a radial beam pivoted at the center of the curve and a floating block placed between the curved block and the radial beam. The tubes were bent by placing them vertically in the stationary grooved block, clamping in that position, placing the floating block, also grooved, between the stationary block and a roller on the radial beam, and pulling the beam upward by block and tackle. This procedure held the floating block against the stationary one, bending the tube to the shape of the latter.

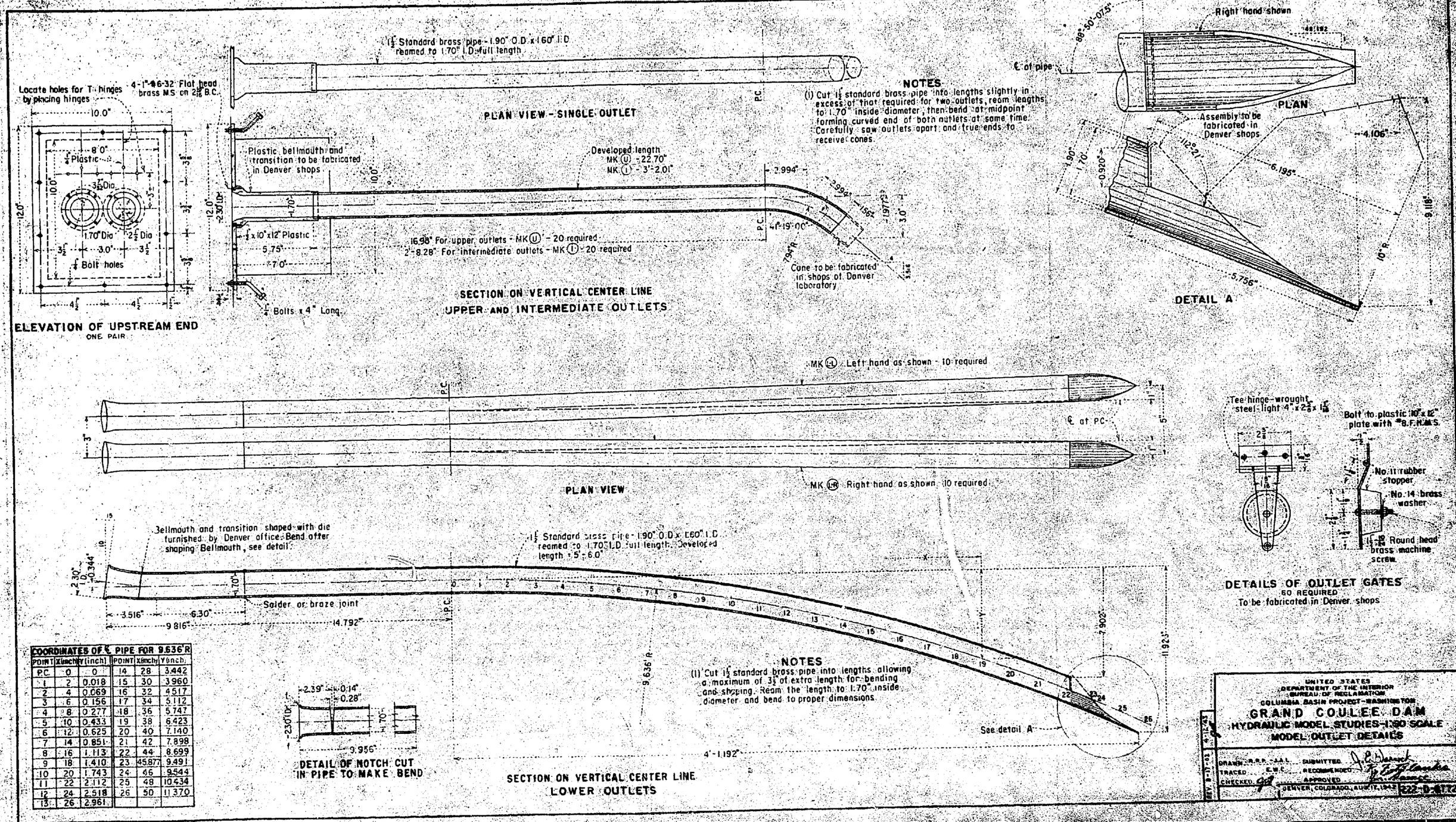
After the bending operation was completed, the downstream end of the tube was placed in a jig and cut to conform to the bucket surface with which it was to coincide. Small brass plates of the proper shape were soldered to this end to complete the outlet exit.

Short lengths of the reamed pipe were placed over a die and swaged to form the bellmouth entrances which were soldered to the upstream ends of the curved sections of the lower outlets.

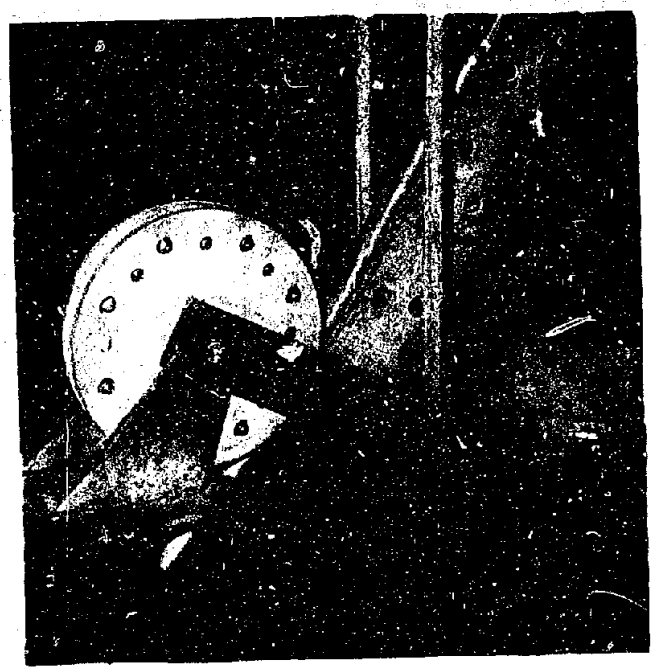
Since the outlets were in pairs and the center lines of the lower pairs diverged, making individual location difficult, an assembly in pairs was accomplished by placing two tubes in a jig and fastening them with special welded split-ring supports.

The upper and intermediate pairs were shaped similarly to the lower tier except that the bending apparatus consisted of a grooved wheel having a diameter producing the proper curve, when the beam pivoted at the center of the wheel, was revolved over the floating block, figure 17. The bend in this case was so sharp as to require the tube to be filled with fine sand and the ends closed before the bending operation was begun. In this case however, two tubes were shaped from one length of pipe. The bend for the downstream ends

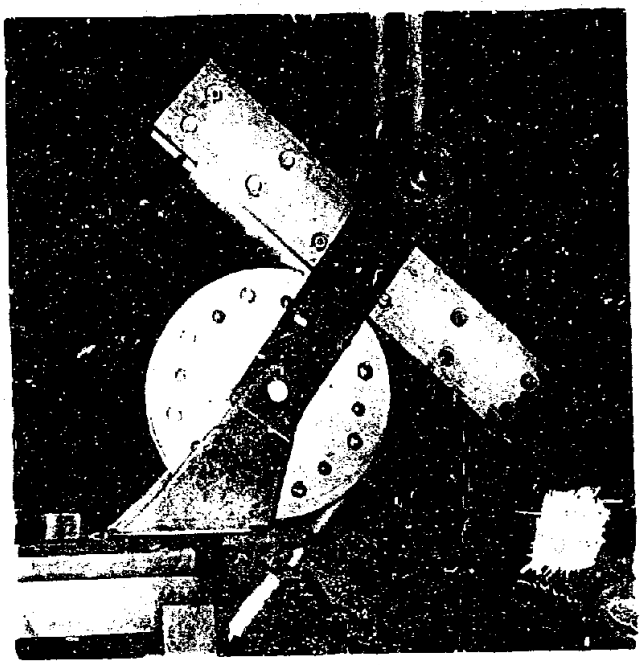
FIGURE 16



COORDINATES OF ℄ PIPE FOR 9.636'R					
POINT	X (inch)	Y (inch)	POINT	X (inch)	Y (inch)
PC	0	0	14	28	3.442
1	2	0.018	15	30	3.960
2	4	0.069	16	32	4.517
3	6	0.156	17	34	5.112
4	8	0.277	18	36	5.747
5	10	0.433	19	38	6.423
6	12	0.625	20	40	7.140
7	14	0.851	21	42	7.898
8	16	1.113	22	44	8.699
9	18	1.410	23	45.877	9.491
10	20	1.743	24	46	9.544
11	22	2.112	25	48	10.434
12	24	2.518	26	50	11.370
13	26	2.961			



A. Sand-filled outlet tube in position ready for bending



B. Sand-filled tube with bending completed

APPARATUS FOR FORMING BENDS IN RIVER OUTLET TUBES OF 1 TO 60 MODEL DAM

FIGURE 17

was made at the center of the length which was then sawed into two pieces.

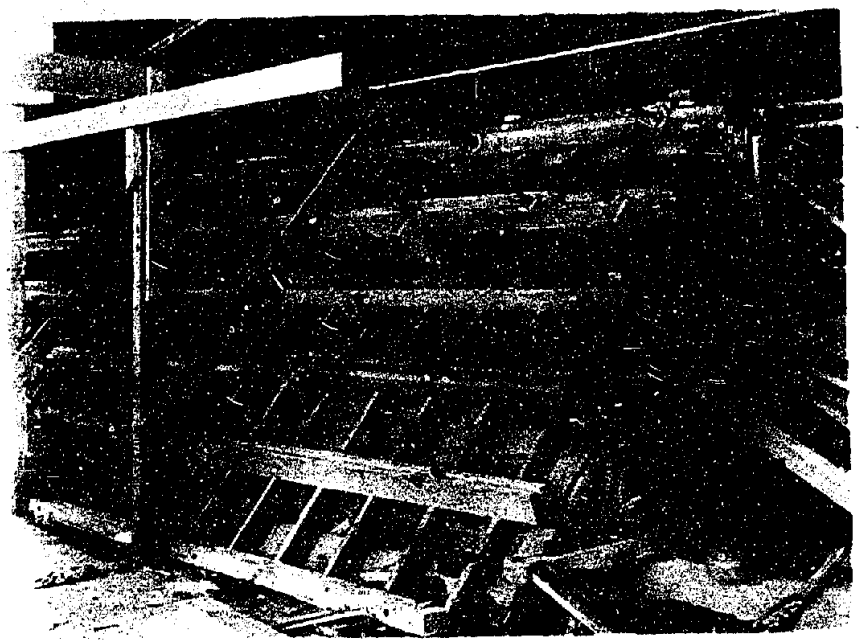
The bellmouth entrances for the upper and intermediate tiers were cast of plastic and made removable to facilitate the future installation of apparatus for automatic operation for public demonstration. Temporary gates for all the outlets consisted of rubber corks attached to hinges which were fastened to the face of the dam to the plastic plates covering the recesses provided for enclosing the contemplated automatic apparatus. The gates were operated by 1/16-inch sash cord attached to the hinges. All outlets were supported by a framework of reinforcing steel before being embedded in the concrete (figure 18A).

Forms for the model dam were constructed accurately, sanded smooth, treated and fastened into place by bracing rods and she-bolts (figure 18B). The alinement of the forms and the position of the materials to be embedded, were checked by the project engineers and necessary adjustments made. Absorptive form lining was not used because test results proved it to be less satisfactory than carefully constructed wooden forms. Surfaces of the completed structure were less than 1/16-inch out of alinement because of the care taken in constructing, setting, and checking the forms prior to and during the placing of the concrete.

The outlet exits with their deflectors and the curved bucket at the downstream toe of the model dam made special forms necessary. From several tests it was found that best results would be obtained by using a hinged form on the downstream face of the dam, carefully built of tongue-and-groove lumber, sanded smooth and treated (figure 19A). Very accurate dimensions were obtained on the face and in the bucket by using this method.

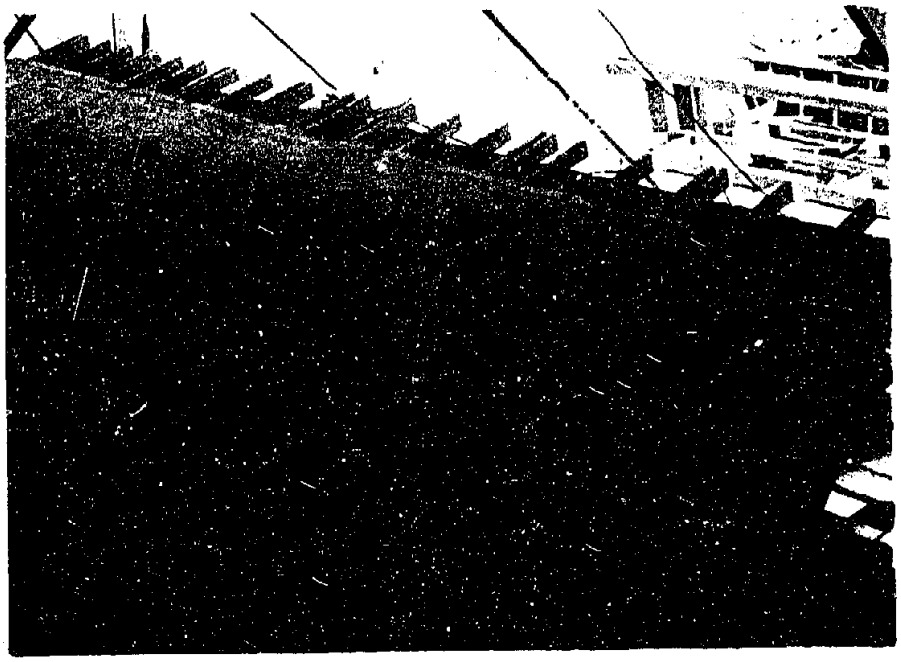
The abutment sections (figure 18B) were poured first, and these sections used as supports for the ends of the hinged form of the spillway section. The abutments were poured in two lifts and the spillway section in three, the third in the latter case being drypack concrete forming the gate recesses and the downstream curved portion of the crest.

A. Lower and intermediate river outlet tubes supported for pouring



B. Forms on right abutment of model dam

RIVER OUTLET TUBES SUPPORTED FOR EMBEDDING AND FORMS ON RIGHT ABUTMENT OF 1 TO 60 SCALE-MODEL DAM



A. Hinged form for downstream face of model dam ready to be tipped into place

B. Completed left abutment of dam

HINGED FORM FOR DOWNSTREAM FACE OF SPILLWAY SECTION AND COMPLETED LEFT ABUTMENT OF 1 TO 60 SCALE MODEL DAM

Supports for the drum-gate seats, the crest section, and the crest piers were embedded in the second pour. The piers and crest pieces made of plastic were bolted to the concrete portion of the dam, and the gates, also of plastic, were hinged at the piers and operated by a shaft and system of levers in the pier and at the back of the gate, figures 20 and 21. Provision was made for the installation of hydraulic valves for operating the gates for public demonstration but levers were to serve the purpose during the test program.

Recesses and blockouts were provided in the pouring of the main structure to facilitate the installation of the training walls and model powerhouses which were constructed of concrete. This type of construction makes it possible to remove and reconstruct these parts of the model without undue damage should it prove desirable to do so in the future. The upstream wall of the right powerhouse was omitted to facilitate any changes which might be required in the bypass-valve installation.

Architectural features were kept at a minimum to conserve labor and materials. Conduits for contemplated hydraulic control apparatus were embedded in the structure. A groove was blocked-out on the face of the spillway section as part of an air-entrainment device which was to serve as a means of producing the white appearance of the prototype flow. The air-entrainment device of plastic contained small holes on the underside to receive the air from the groove and distribute to a horizontal trough in the face of the spillway, figure 22. The device was supplied through a 3-inch pipe embedded in the right abutment and extending through the control-room wall.

Initially no tests were contemplated on the model concerning the erosion of the river channel below the dam, thus it was planned to construct this part of the model using a base of graded rock covered with a 2- to 3-inch layer of concrete shaped to the contours. However, preliminary studies on the needle-valve bypass-outlet installation in the right powerhouse indicated that it would be desirable to construct a portion of the channel immediately below the dam of loose material making erosion studies possible. Moreover, this portion of the model

FIGURE 20

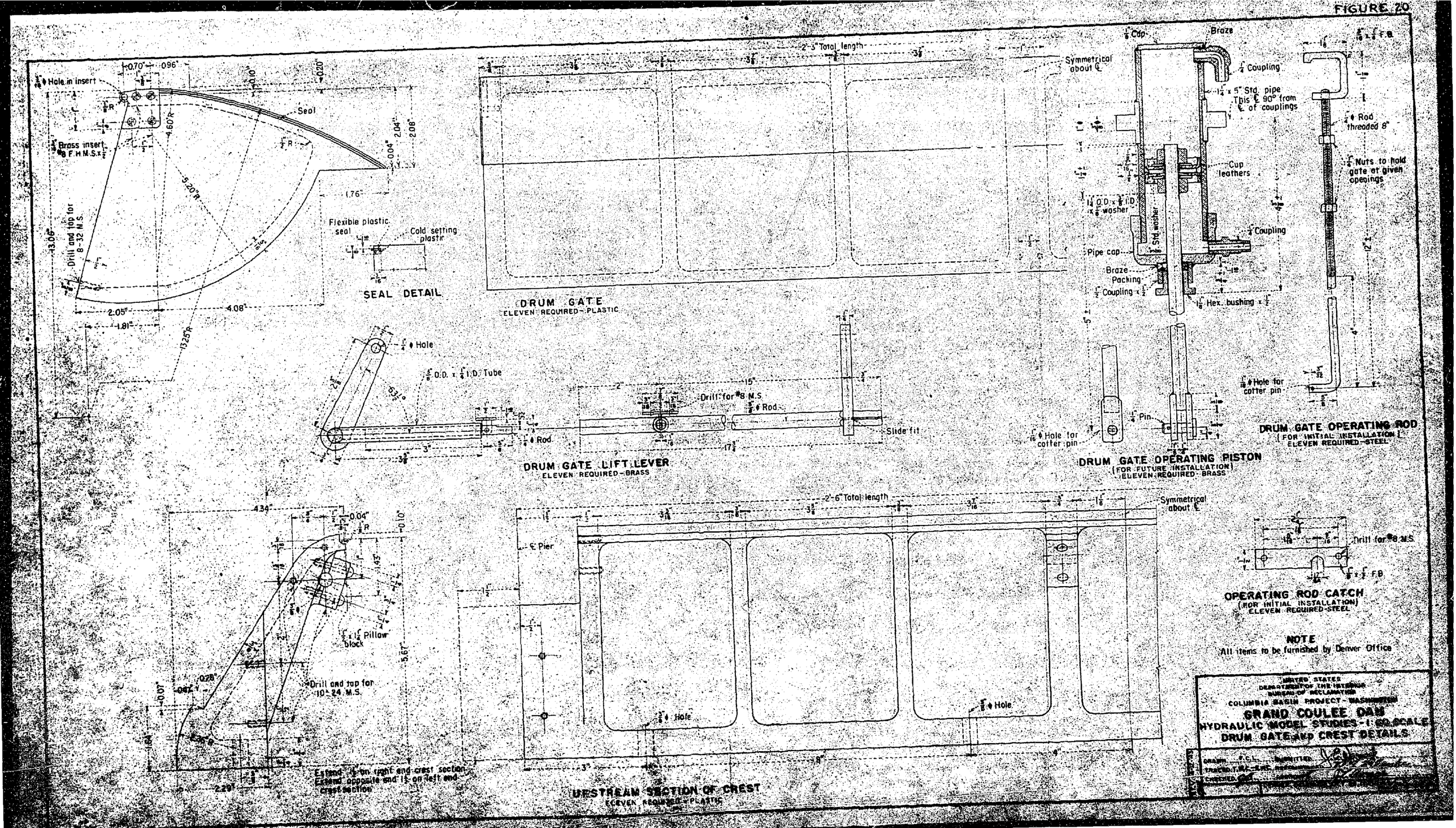
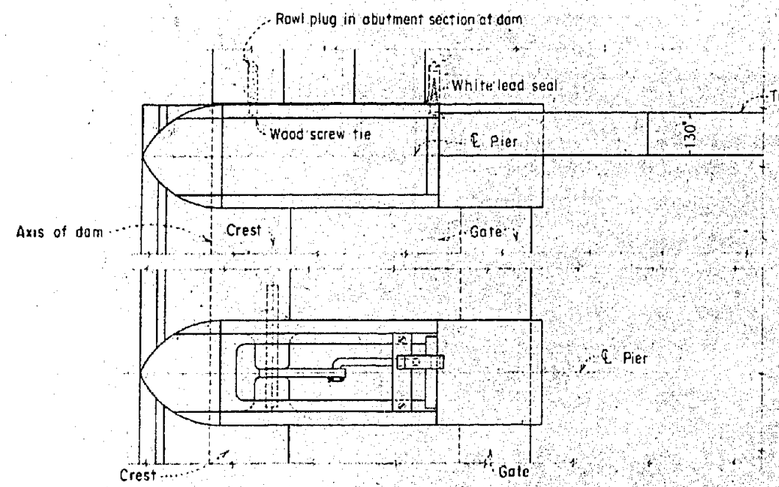
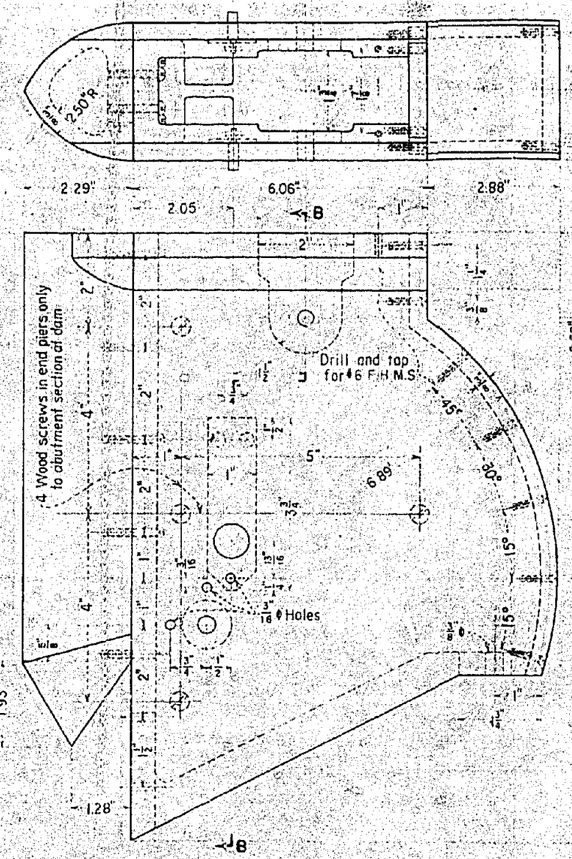
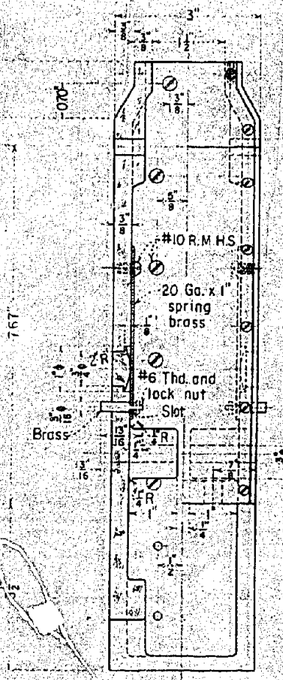

Hole in insert
Brass insert #8 F.H.M.S. x 1/2
Seal
Drill and tap for 8-32 N.S.
Flexible plastic seal
Cold setting plastic
SEAL DETAIL
2'-3" Total length
Symmetrical about ℄
DRUM GATE
ELEVEN REQUIRED—PLASTIC
Cap
Braze
1/4 Coupling
1/4 x 5" Std. pipe This ℄ 90° from ℄ of couplings
Rod threaded 8"
Nuts to hold gate at given openings
Cup leathers
Std. washer
Pipe cap
Braze
Packing
Coupling x 1/2
Hex. bushing x 1/2
Hole for cotter pin
Pin
DRUM GATE OPERATING ROD
(FOR INITIAL INSTALLATION)
ELEVEN REQUIRED—STEEL
DRUM GATE OPERATING PISTON
(FOR FUTURE INSTALLATION)
ELEVEN REQUIRED—BRASS
Hole
0.D. x 1/8 I.D. Tube
Drill for #8 N.S.
Rod
Slide fit
DRUM GATE LIFT LEVER
ELEVEN REQUIRED—BRASS
2'-6" Total length
℄ Pier
Hole
Drill for #8 N.S.
OPERATING ROD CATCH
(FOR INITIAL INSTALLATION)
ELEVEN REQUIRED—STEEL
Pillow block
Drill and tap for 10-24 N.S.
Extend 1/2 on right end crest section. Extend opposite end 1/2 on left end crest section
UPSTREAM SECTION OF CREST
ELEVEN REQUIRED—PLASTIC
NOTE
All items to be furnished by Denver Office
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES—1:60 SCALE
DRUM GATE AND CREST DETAILS

FIGURE 21



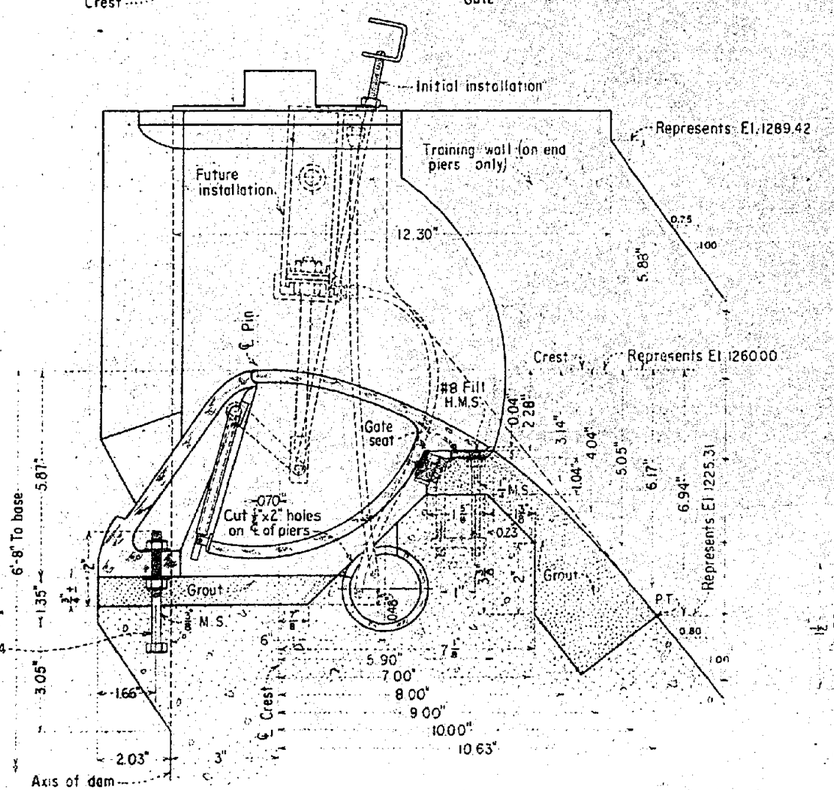


DRUM GATE PIERS
TWELVE REQ'D. - PLASTIC

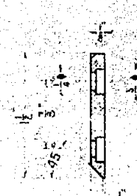


HALF SEC. B-B

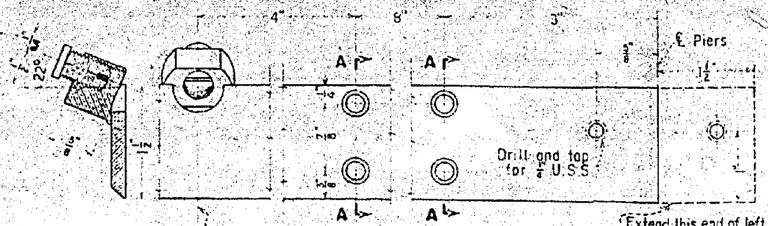
HALF ELEV.



CREST DETAIL



SEC. A-A



Each gate section of 30" symmetrical about this ℄ except end sections

Extend this end of left end sec, extend opp end of right end section.

DRUM GATE SEAT
ELEVEN SECTIONS REQ'D. - BRASS
(Fabricated in Denver shops)

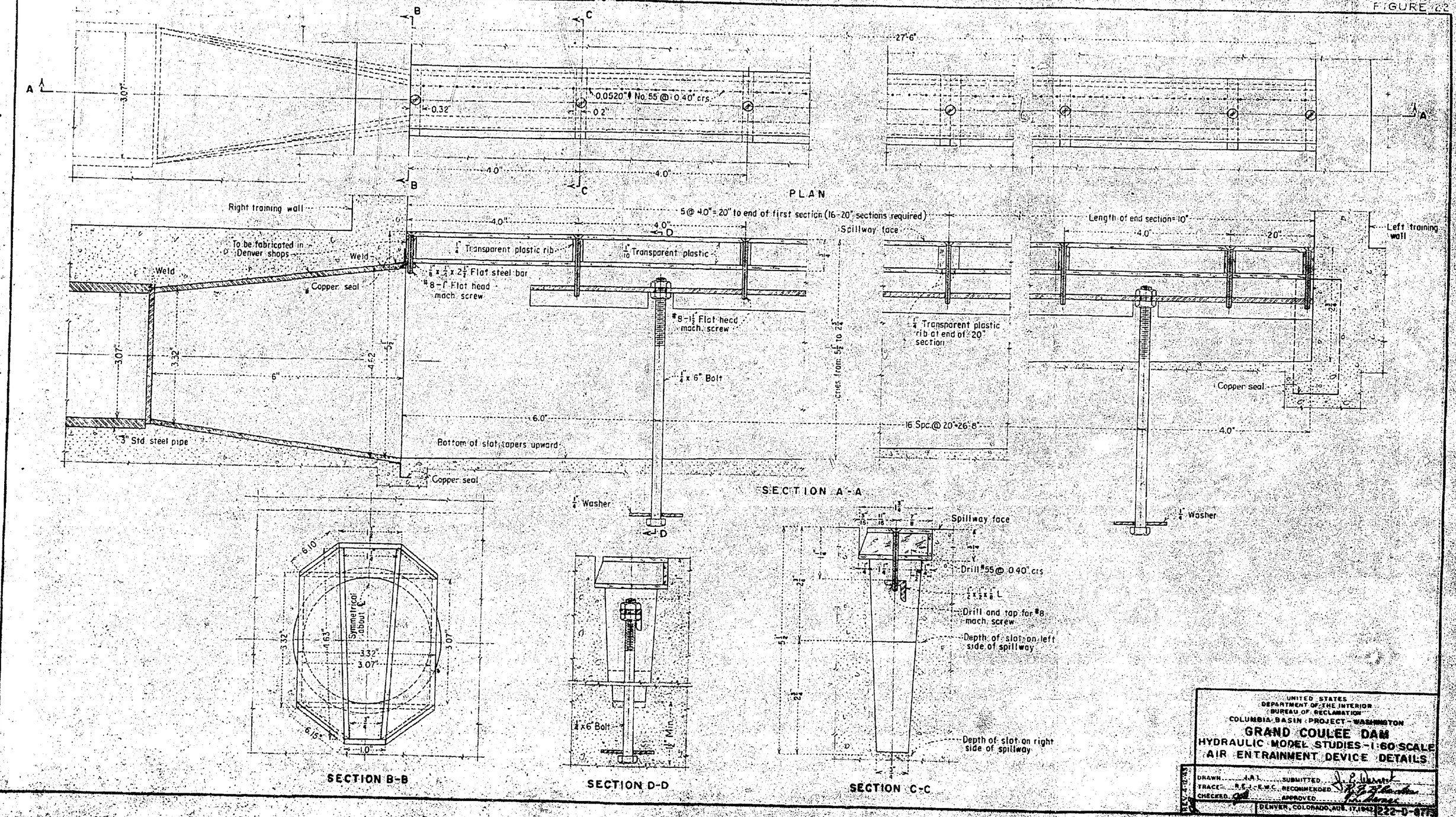
NOTE
All items to be furnished by Denver Office

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON

GRAND COULEE DAM
HYDRAULIC MODEL STUDIES - 1:60 SCALE
CREST AND PIER DETAILS

DRAWN E.C.B. SUBMITTED
TRACED R.A.W. - E.W.C. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO, OCT. 19, 1942 | 222-D-8968
REV. A

FIGURE 62



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON

GRAND COULEE DAM
HYDRAULIC MODEL STUDIES - 1:60 SCALE
AIR ENTRAINMENT DEVICE DETAILS

DRAWN L.A.I. SUBMITTED
TRACE R.E.J. - E.W.C. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO, AUG. 17, 1943 222-D-8775

had not been completed when the first inspection of the bucket was made, and the damage revealed by this inspection appeared to be the result of the movement of loose material on the river bottom, increasing the desirability of providing a movable bed in this region.

Considering these items, the model was constructed with a river channel of loose material for a distance of about 35 feet downstream and of material of a more stable nature beyond this point. Compacted sand covered with a 1-1/2-inch thick blanket of 3/8-inch screened gravel was used to form the movable bed, while porous concrete was used to form the more permanent section, figure 23. The porous concrete section was poured in layers, the first within 6 to 8 inches of the finished surfaces, using 3/4-inch aggregate, figure 24A; the second, using smaller aggregate, screeded between negative templates of the channel to within 1-1/2 to 2 inches of the finished surface; and the third, using small screened aggregate, floated to the configuration of the contours, the elevation and location of which were established by setting nails in a quick-setting concrete mortar at selected cross sections of the river channel, figures 24B and C. A grout coat was brushed into the top surface to prevent excessive percolation of water from the channel through the porous concrete, figure 25.

The highway-bridge piers in the river below the dam were cast of concrete and grouted in place. The river-stage gage similar to the prototype was included in the left pier, but the bridge structure was not included since it had no bearing on the model tests.

A covered concrete sump placed at the downstream end of the river channel collected any sand or gravel carried by the model flow. The sump contained a slide gate for regulating the tailwater elevation, a 42-inch concrete pipe for returning the flow to the storage basin, and a 6-inch drain.

The 42-inch concrete conduit, approximately 100 feet long, curved in plan, sloped downward toward the storage reservoir, which was divided into compartments, the dividing walls supporting the floor slab of the model reservoir and allowing the outside faces of the foundation walls to be constructed vertically to facilitate future construction of the

FIGURE 23

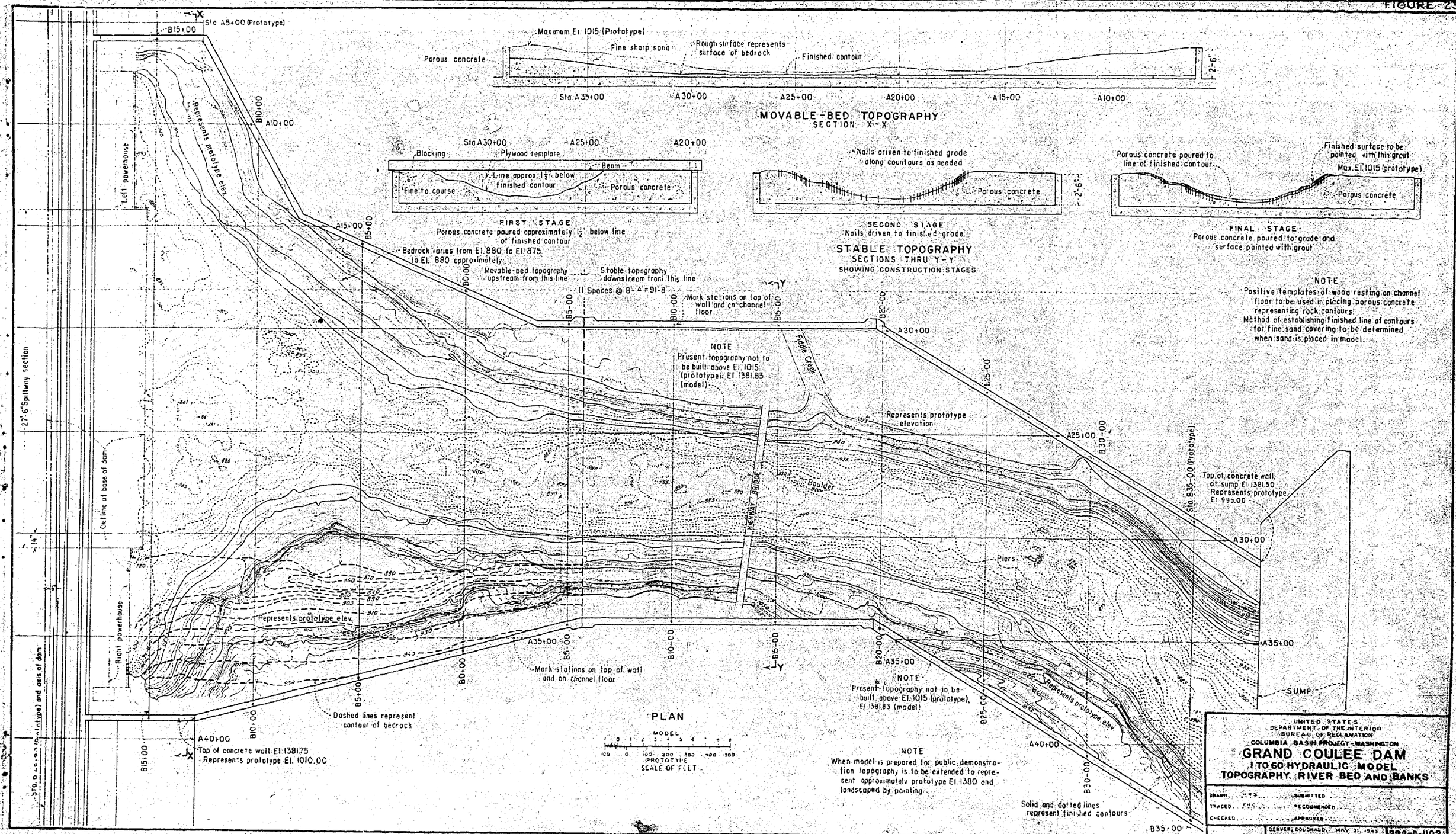
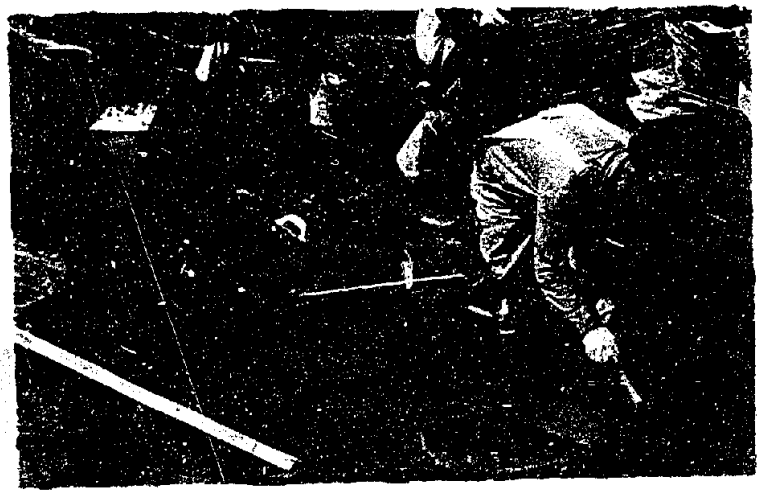

Sta. A5+00 (Prototype)
B15+00
Maximum El. 1015 (Prototype)
Fine sharp sand
Rough surface represents surface of bedrock
Porous concrete
Finished contour
2'-6"
Sta. A35+00
A30+00
A25+00
A20+00
A15+00
A10+00
MOVABLE-BED TOPOGRAPHY
SECTION X-X
Sta. A30+00
A25+00
A20+00
Blocking
Plywood template
Beam
Line approx. 1½" below finished contour
Fine to course
Porous concrete
FIRST STAGE
Porous concrete poured approximately 1½" below line of finished contour
Nails driven to finished grade along countours as needed
Porous concrete
2'-6"
SECOND STAGE
Nails driven to finished grade
Porous concrete poured to line of finished contour
Finished surface to be painted with thin grout
Max. El. 1015 (prototype)
Porous concrete
FINAL STAGE
Porous concrete poured to grade and surface painted with grout
STABLE TOPOGRAPHY
SECTIONS THRU Y-Y
SHOWING CONSTRUCTION STAGES
NOTE
Positive templates of wood resting on channel floor to be used in placing porous concrete representing rock contours.
Method of establishing finished line of contours for fine sand covering to be determined when sand is placed in model.
Left powerhouse
Represents prototype elev.
B10+00
A10+00
A15+00
B5+00
Bedrock varies from El. 880 to El. 875 to El. 880 approximately
Movable-bed topography upstream from this line
Stable topography downstream from this line
11 Spaces @ 8'-4" = 91'-8"
Mark stations on top of wall and on channel floor
B5-00
B10-00
B20-00
A20+00
NOTE
Present topography not to be built above El. 1015 (prototype), El. 1381.83 (model)
Fiddle Creek
B25-00
A25+00
B30-00
Sta. B35-00 (Prototype)
Represents prototype elevation
Boulder
HIGHWAY BRIDGE
Top of concrete wall at sump El. 1381.50 Represents prototype El. 995.00
A30+00
Piers
A35+00
SUMP
27'-6" Spillway section
Outline of base of dam
Right powerhouse
Represents prototype elev.
Mark stations on top of wall and on channel floor
Dashed lines represent contour of bedrock
A40+00
Top of concrete wall El. 1381.75 Represents prototype El. 1010.00
Sta. D 20+00 (Prototype) and axis of dam
B15+00
B35-00
PLAN
MODEL
PROTOTYPE
SCALE OF FEET
NOTE
When model is prepared for public demonstration topography is to be extended to represent approximately prototype El. 1380 and landscaped by painting.
Solid and dotted lines represent finished contours
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT-WASHINGTON
GRAND COULEE DAM
1 TO 60 HYDRAULIC MODEL
TOPOGRAPHY, RIVER BED AND BANKS
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO, MAY 31, 1943
222-D-1102

FIGURE 24

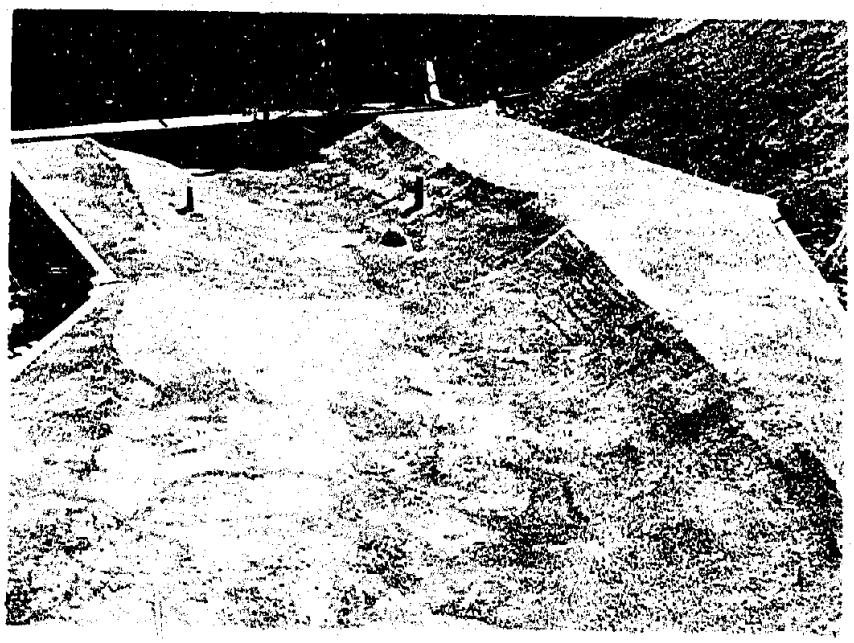
A. Screeding porous concrete river bed to approximate channel shape

B. Sections of river channel formed with nails set at proper elevation in quick-setting concrete

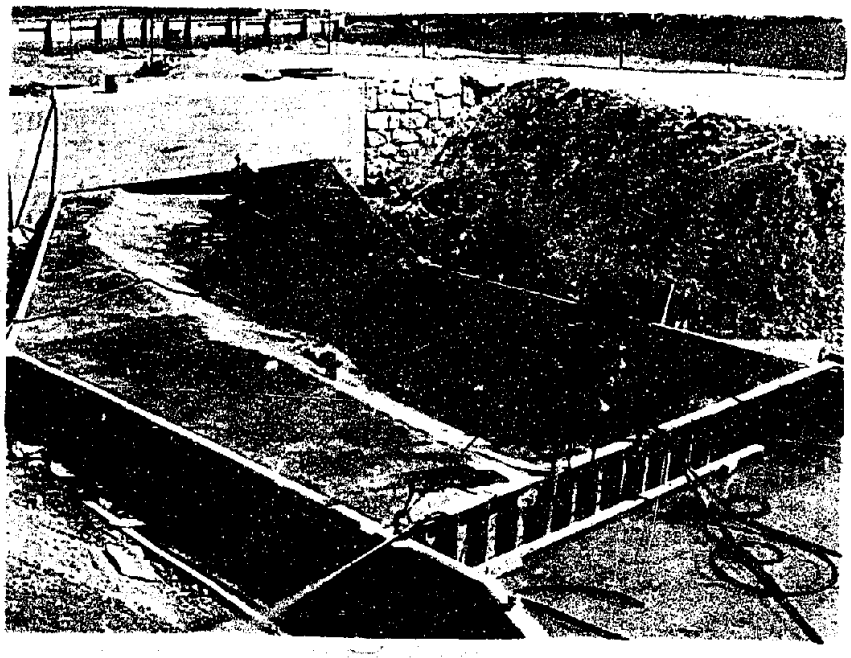


C. Placing top layer of porous concrete to form final river-channel contours

STAGES IN THE CONSTRUCTION OF THE POROUS-CONCRETE TOPOGRAPHY OF THE 1 TO 60 MODEL OF GRAND COULEE DAM



A. Stable section of model river bed, looking upstream toward model dam. Roadway bridge piers in channel



B. Completed river channel of porous concrete looking downstream - Top of prototype structure and reservoir in background

COMPLETED POROUS-CONCRETE SECTION OF THE DOWNSTREAM RIVER CHANNEL OF THE 1 TO 60 MODEL OF GRAND COULEE DAM

foundation for the vista house. Openings through the intermediate walls allowed the water to pass from one compartment to another to reach the pump sump in the opposite end of the basin under the control room. Sufficient valves and drains were provided to empty the system completely, figure 13.

Restrictions on the purchase of new pumping equipment made it necessary to place a temporary bulkhead between the pump sump and the storage basin to provide for the installation of a jet pump designed to make use of high-head equipment available on the project, figure 12.

An 8-inch pump, capable of delivering 5 second-feet of water under a head of 110 feet, was arranged to discharge through a needle valve into a bellmouth entrance on an 8-inch pipe leading to the 14-inch supply line. The intake to the bellmouth was an 18- by 24-inch rectangular conduit, closed at the end, extending through the temporary bulkhead.

The high-velocity jet from the needle valve passed through the bellmouth and 8-inch pipe to an 8- to 14-inch expander, thence through a length of 14-inch pipe to one end of the control room where a 14- to 24-inch expander connected it to the 24- by 30-inch distribution conduit in the model reservoir. The 14-inch pipe was provided with a stationary pitot tube to indicate the flow in it and with facilities for calibrating it using a standard pitometer. An 8-inch bypass pipe containing a calibrated venturi meter was provided for measuring small quantities.

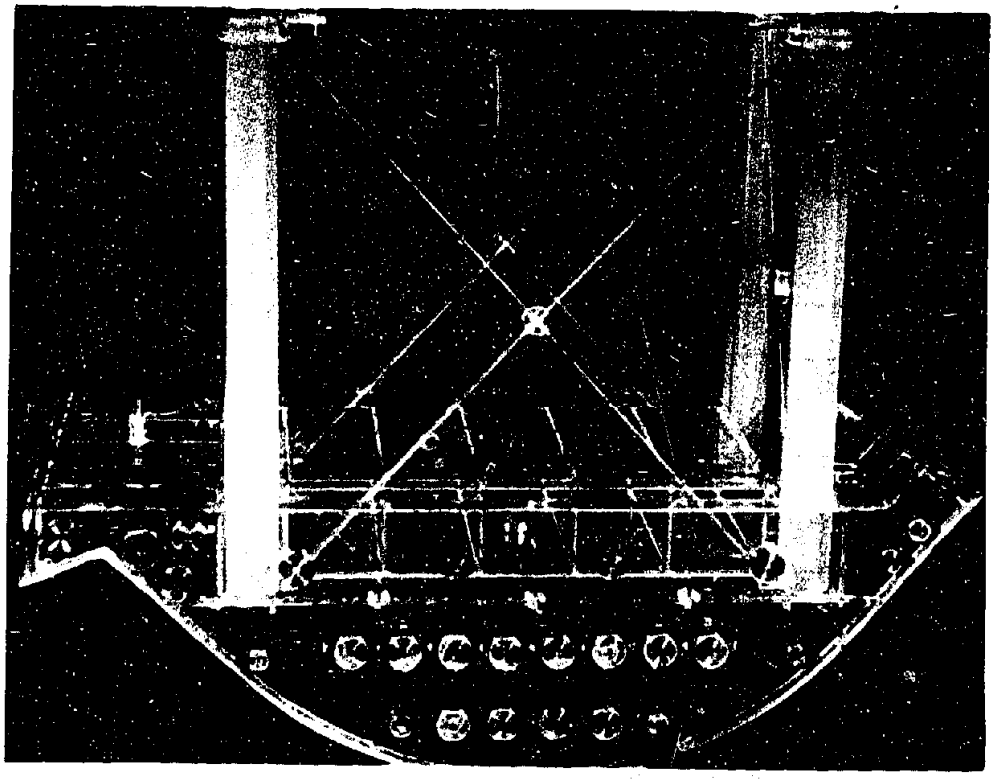
The flow through the system was regulated by changing the needle-valve opening. The pumping equipment was capable of delivering only one-third of the design capacity of the model dam, a quantity at that time believed to be sufficient for all the model tests.

When further testing at higher discharges was considered essential in making a study of protective measures concerning the bucket, tailrace slopes, and river banks downstream, the model supply system was altered and a 24-inch propeller pump provided, figure 14. This installation was completed in September 1944.

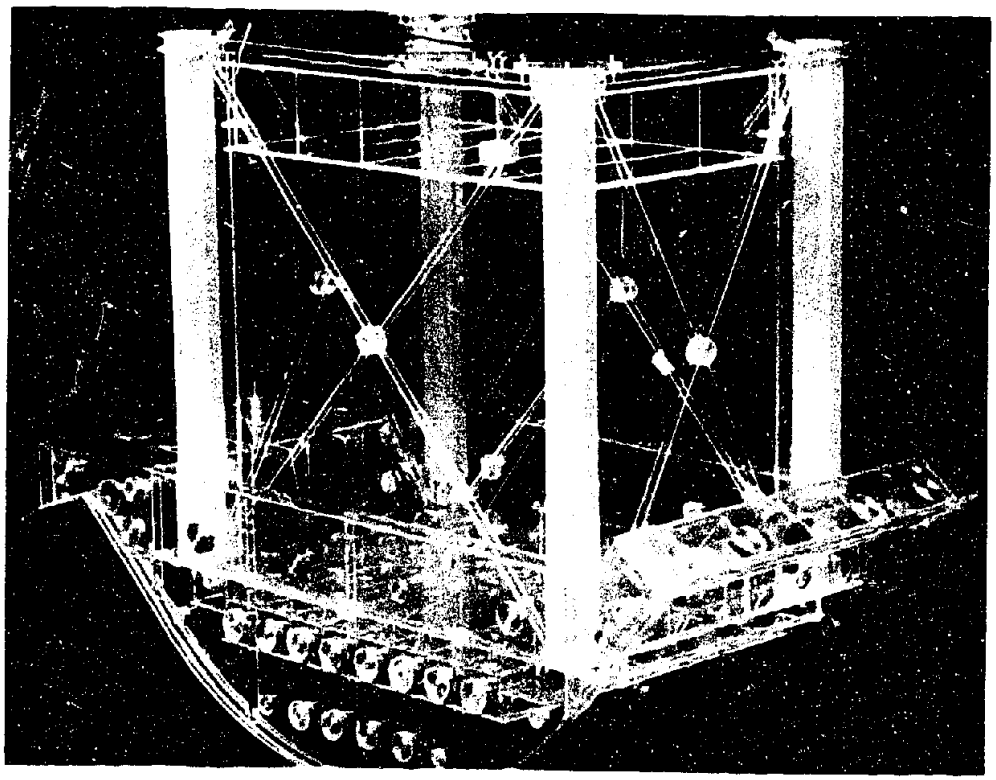
9. The 1 to 60 model floating caisson for maneuvering tests. Since the currents in the river channel below the dam were important factors influencing the maneuvering of the caisson to and from its position over the bucket, it was essential that they be reproduced accurately for the tests pertaining to this phase of the problem. A complete model, including the dam, powerhouses, and downstream river channel in which the maneuvering was to take place, was considered necessary to attain these conditions. Although it was realized that the caisson constructed to the scale of the model dam, described previously in this section of this report, might be too small for many of the tests pertaining to its action during the sinking and raising operations and a larger model of the caisson with a tank, containing the toe of the dam and a section of the bucket, might be required, it was not considered feasible to construct the model dam to a greater size than one-sixtieth of the prototype. Moreover, preliminary tests could be made on the caisson of this scale and the necessity of the larger model established from the results. Furthermore, the size was considered adequate for the maneuvering tests in which the caisson was to be moved from the drydock to a position over the bucket and returned to the drydock by lines from points located on the river banks or powerhouse structures.

Consideration was given to fabricating this model from sheet metal, with all parts to scale, but due to the extremely thin sheet required to keep the weight within proper limits and the difficulty in ascertaining whether or not the compartments were properly filled or emptied, the plan was abandoned and the model, including the main operating barge, constructed of water-resistant, transparent, plastic sheet with low specific gravity and more thickness, figure 26. The weight of each panel was corrected by inserting brass discs at such points as to retain the proper moment of inertia and radius of gyration of each girder and partition.

It was impossible to provide the correct buoyancy due to the thickness of this material, but the fabrication was done in a manner minimizing the variation resulting from the thicker sheets. This was



A. Side view of caisson with operating barge in lower position



B. Oblique view of caisson showing working space, operating barge in raised position

COMPLETED 1 TO 60 PLASTIC MODEL OF THE FLOATING CAISSON TO BE USED IN THE REPAIR AND MAINTENANCE OF THE SPILLWAY BUCKET OF GRAND COULEE DAM

accomplished by constructing the model with all outside dimensions to scale, confining the variations affecting the buoyancy to the inside of the girders and compartments. The effect upon the buoyancy was considered insignificant, particularly in view of the preliminary nature of the design. The Plexiglas or Lucite plastic sheets were fastened together by using a special solvent for the material. Tubing of the same material was used for the four access shafts.

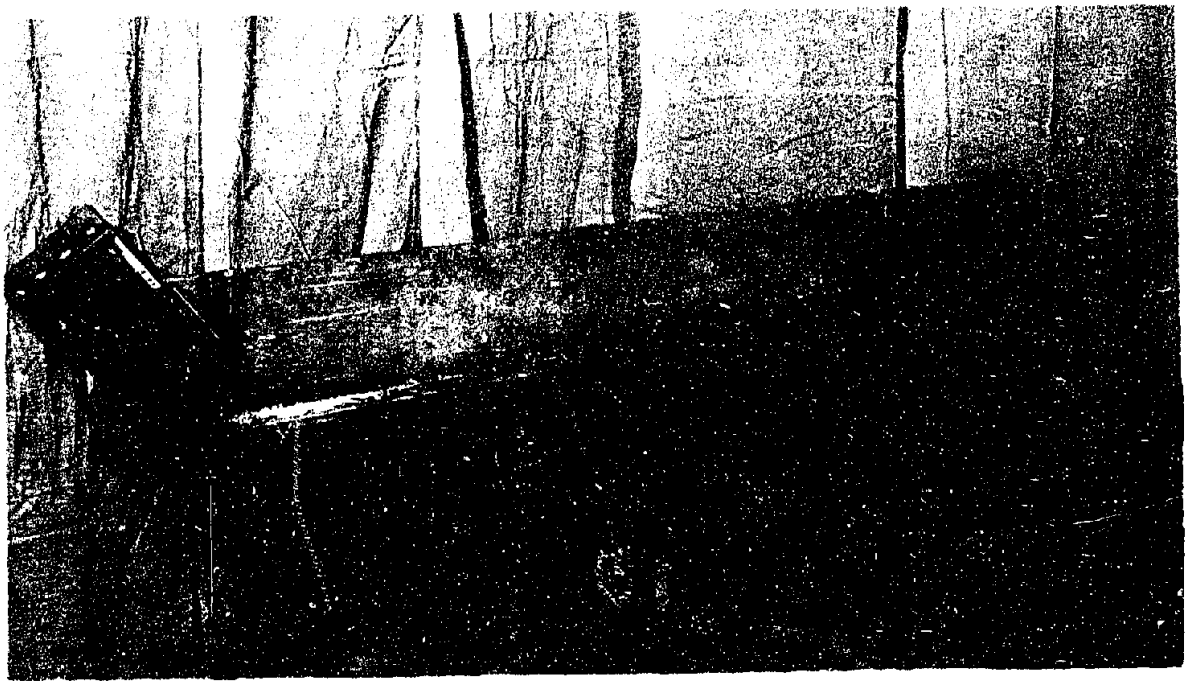
10. The 1 to 60 scale model of floating caisson used for pressure tests. In conducting tests to determine the feasibility of discharging water over the drum gates or through the river outlets as planned in the early stages of development of the repair caisson, it was necessary to study the flow characteristics in the river channel below the dam under various operating combinations of these features with the caisson in position on the bucket. Of particular significance was the nearness of the flow to the caisson, without encountering critical conditions; the roughness of the water surface in the vicinity where repair work was being performed; the velocities in the bucket under which the work must be accomplished; and the side thrust expected on the caisson when in the bucket with flow through the bucket on one side and relatively quiet water on the other. A special caisson model on the same scale as the model dam, equipped with piezometers for measuring this force, was constructed of sheet metal. Since the installation of pressure-measuring equipment precluded using the model in the floating position for maneuvering tests, due to excessive weight, no attempt was made to construct it geometrically similar to the prototype design other than in the shape and arrangement of the outside surfaces. The caisson proper was of 16- and 20-gage sheet metal, the access and elevator shafts of brass tubing, and the shaft bracing of angles made of lightweight sheet metal. All joints were soldered. Piezometer tubes were of 1/16-inch inside diameter copper tubing inserted and soldered in holes drilled in the outside surfaces of the caisson. The outside ends of the tubes were dressed flush with the outside surfaces of the caisson and the other ends threaded through the working space underneath and up the access and elevator shafts where rubber tubing could be attached.

A graduated manometer board, with approximately 45 glass columns with an airtight header across the top from which air could be exhausted to raise the water surfaces in the piezometers to readable levels, was provided for recording the pressures on the surfaces of the caisson. When this device proved cumbersome and unsatisfactory because of the effect upon the meniscus of slight irregularities in the walls of the tubing, a U-tube, equipped with a pressure probe and a lead of rubber tubing to a constant-level reservoir, was used to obtain the pressure data.

Point gages mounted on a temporary wooden beam spanning the spillway of the model dam were used to record the heights of waves forming near the dam.

When measurable, the velocities in the vicinity of the caisson lowered onto the bucket, were recorded by a midget current meter. When not, the surface velocities in the region were obtained by placing confetti on the water surface and timing the travel of the particles over measured courses.

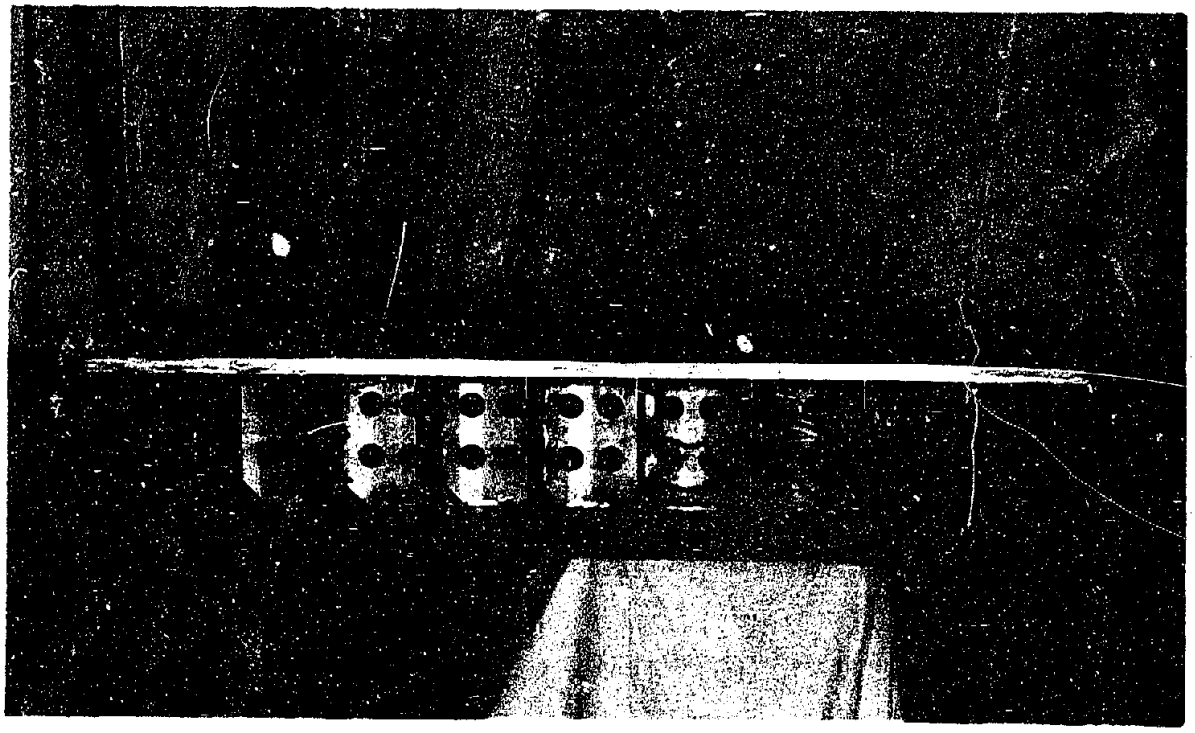
11. The 1 to 20 scale model of the floating caisson used for stability tests. As soon as preliminary tests using the 1 to 60 plastic model of the caisson proved it to be inadequate for studies concerning the stability of equilibrium and the operating characteristics during the sinking and raising operation, plans were made immediately to construct of sheet metal a larger model having the weight, center of gravity, and buoyancy factors to scale. A scale of 1 to 20 was chosen when it was found that sheet metal of standard gages would meet weight and thickness requirements. The girders, compartments, access shafts and bracing were fabricated from galvanized sheets of various gages and soldered together, figures 27, 28, and 29. The top portion of the side girders and the trim and floating compartments were made watertight and provided with lengths of copper tubing for draining and filling by an ejector. For filling, the ejector was connected to the vent at the top of a compartment while a rubber hose from the drain tube was placed in a vessel of water. The evacuation of air from the compartment would permit atmospheric pressure on the water



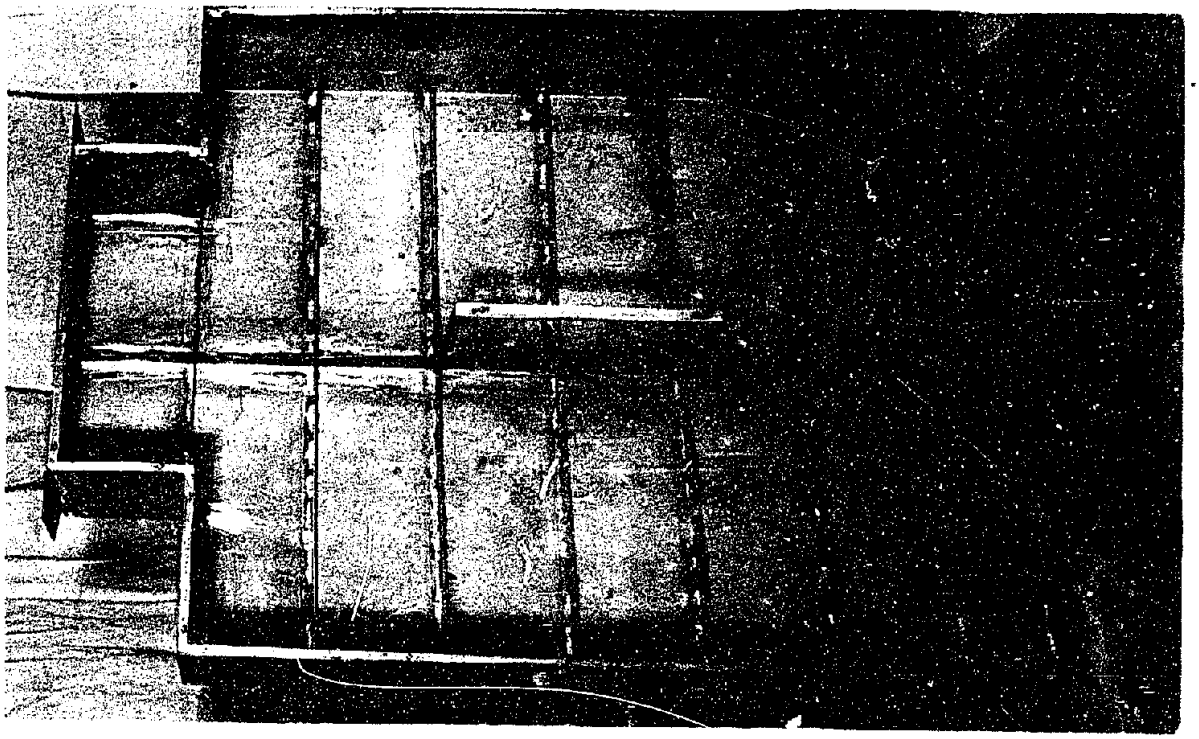
A. Side girder with inside skin plate removed

B. Back and front end sections

FABRICATED SECTIONS OF 1 TO 20 MODEL OF THE FLOATING CAISSON TO BE USED IN THE REPAIR AND MAINTENANCE OF THE SPILLWAY BUCKET OF GRAND COULEE DAM

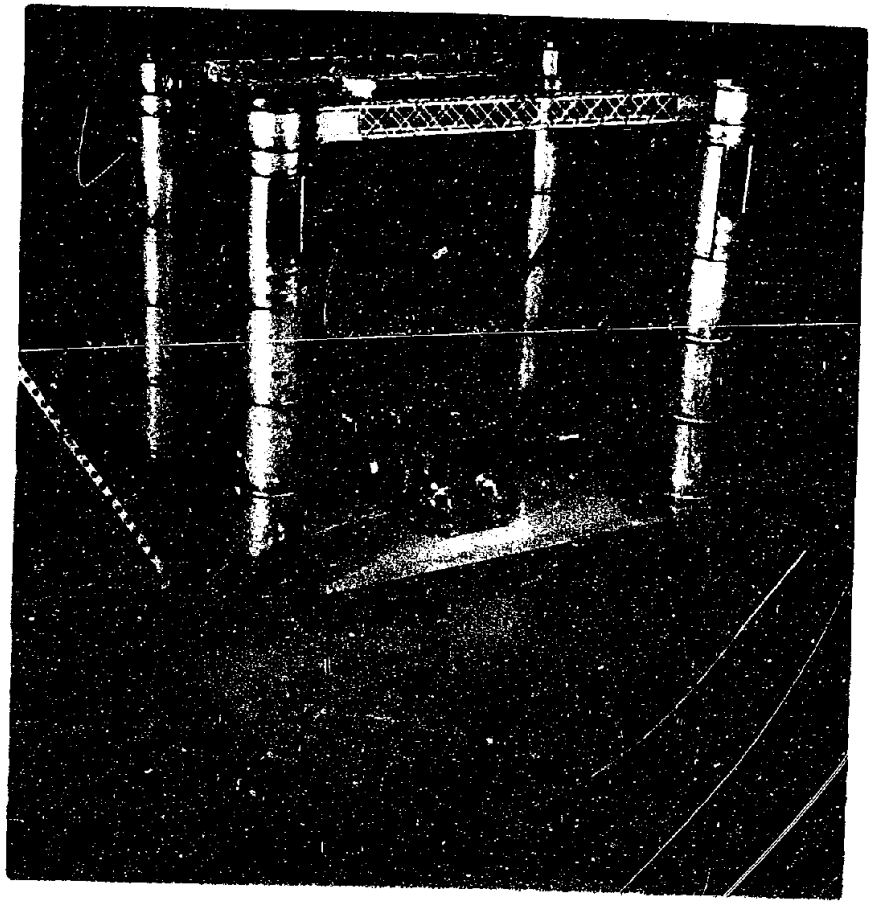


A. Side view of center section showing floating and trim compartments

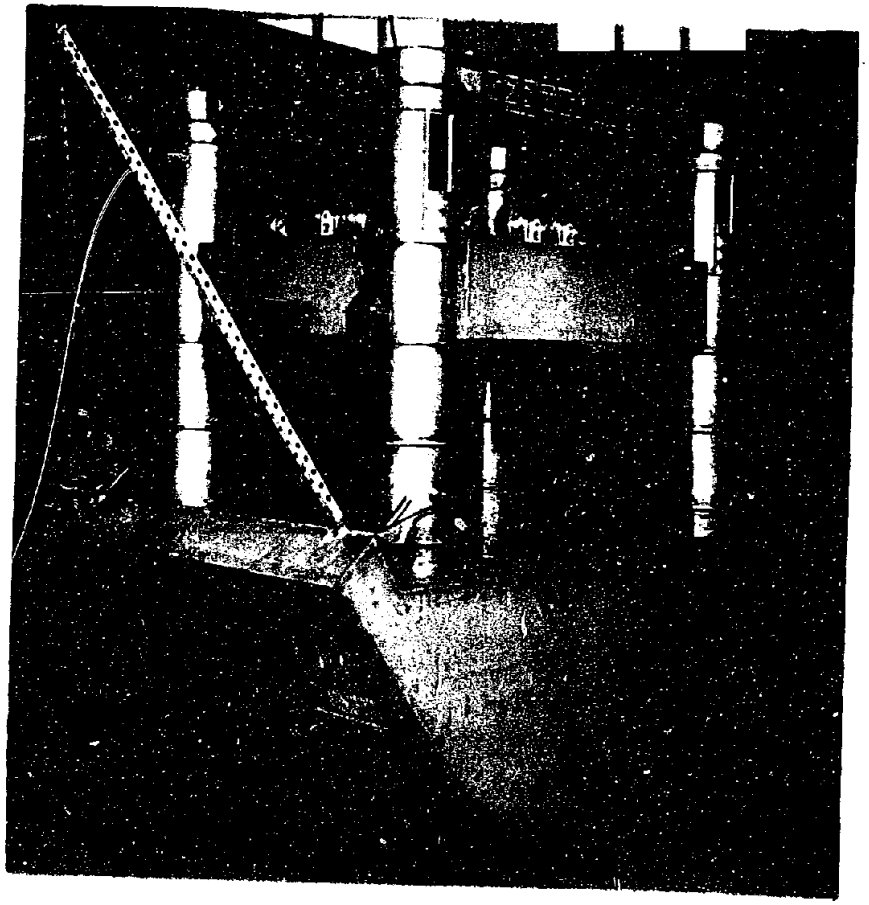


B. Top view of operating barge with deck removed

CONSTRUCTED SECTIONS OF 1 TO 20 MODEL OF SPILLWAY BUCKET, REPAIR-CAISSON AND OPERATING BARGE



A. Side view of caisson-powered-operating-barge in lower position



B. Oblique view of caisson showing working space operating-barge in raised position

COMPLETED 1 TO 20 MODEL OF THE FLOATING CAISSON AND ITS OPERATING BARGE TO BE USED IN THE REPAIR AND MAINTENANCE OF THE SPILLWAY BUCKET OF GRAND COULEE DAM

FIGURE 29

surface in the vessel to force water through the drain into the compartment, filling it. To drain a compartment, the ejector was attached to the drain tube and the vent opened to the atmosphere. An ejector operated by air, made it possible to determine whether the compartments were full or empty by examining the water content of the ejector discharge. Air vents in the ends of the working chamber of the caisson were provided to exhaust the air from this region when the caisson was beginning its descent to the bucket and to allow the air to escape when the compartment was flooded in preparing it for the surfacing operation. Metal cylinders 4-1/2-inch inside diameter, extending into the top of the working space, represented the access and service shafts. A V-frame for anchoring the caisson to the dam and alining it during the sinking and raising operations was constructed of sheet metal.

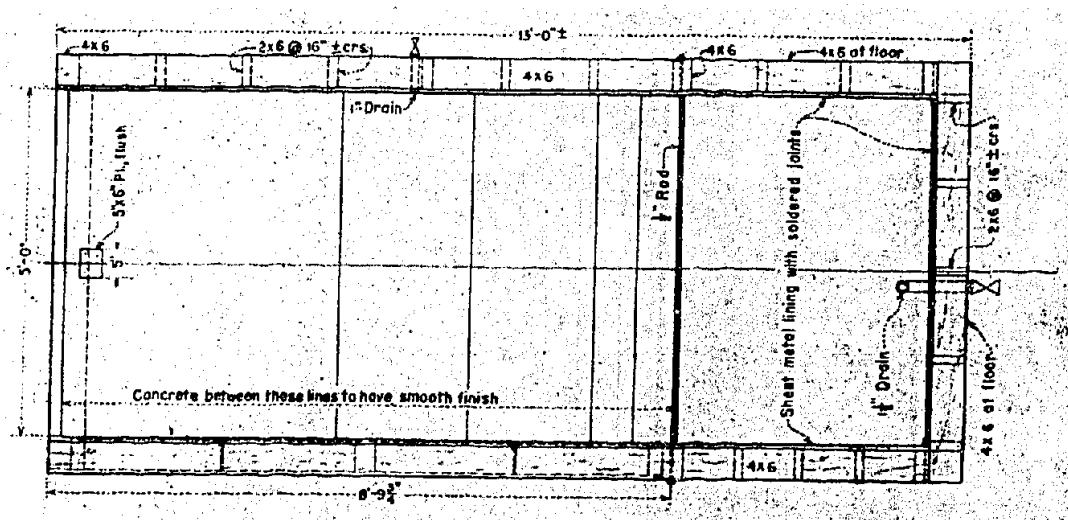
The caisson contained a depressed deck to accommodate the main operating barge, thus it was capable of floating and at the same time transporting the operating barge. By filling or draining the deck or barge well, it was possible to transfer the weight of the caisson to and from the barge which lowered it into position and raised it to the surface. The barge was equipped with hoists, lines, and pulleys of brass to accomplish the sinking and raising operations. An 8-part line between the barge and caisson was provided at each corner to represent the 1-1/4-inch, 12-part lines of the prototype. The weight of the caisson was carried on these lines during the sinking and raising operations and the lines were operated by 2.7-inch diameter drum hoists mounted on the deck of the barge. Four 4-part lines from the barge to the bracing at the top of the access shafts, represented those to be used on the prototype, to raise the barge from the water surface, out of reach of the waves and other disturbances when the caisson was in position in the bucket, or stored in the drydock during high water, and to assist in maintaining equal loads on the four corners of the barge during the sinking and raising operations. Drum hoists for these lines were the same size as those for the 8-part lines and were mounted on the deck. All hoists were operated from a central power unit consisting of a small washing-machine motor, gear reducer, and drive shafts.

Since it was necessary for the caisson hoists to take up line at the rate it was being released by the barge hoists, and vice versa, all were of the same size and were interconnected to operate simultaneously and at the same speed. Aerial wire was first used for the lines since 1/16-inch metal sash cord was not available. It was unsatisfactory due to the excessive amount of stretch. Its size did not permit its being rove on the drums in one layer. Satisfactory operation resulted when it was replaced by 24-gage spring brass wire.

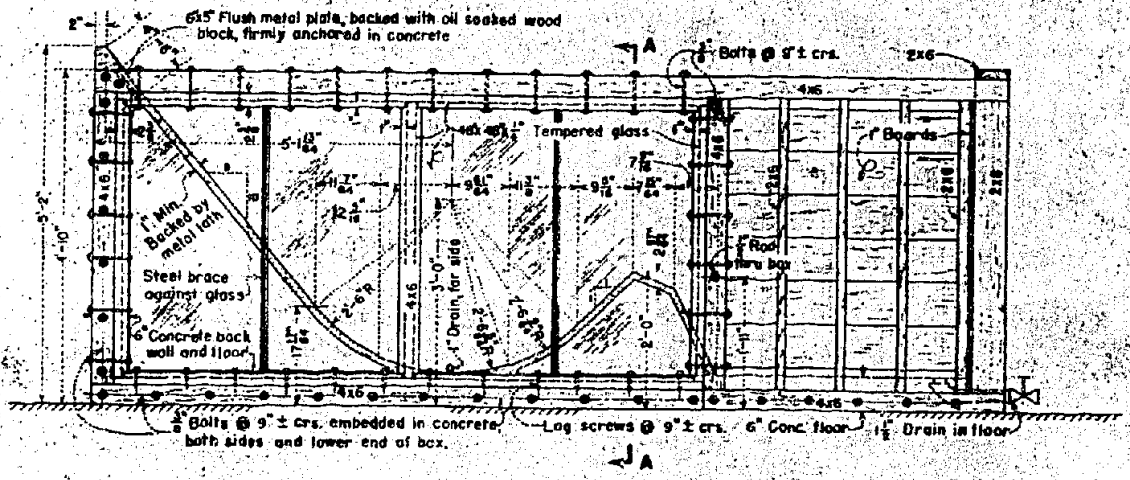
A tank, 5 feet wide, 13 feet long, and 5 feet deep, with the bottom and one end of concrete, and the remaining sides of wooden and glass panels, was constructed for tests on this model, figure 30. A section of the toe of the dam and spillway bucket were placed at the concrete end of the tank. The profile of the dam and bucket was formed by screeding mortar between two positive templates of plywood, bolted to the sides of the concrete end and the bottom of the tank, before the side panels were erected. Metal lath was suspended between these templates approximately one inch below the finished surface and a concrete-plaster mortar plastered on the lath and screeded between the templates with a straightedge to the finished surface as the mortar obtained its set. After this portion had attained sufficient strength the templates were removed and the metal-lined and glass side panels bolted in place. Coal-tar paint and building paper were used to seal the joints. A metal plate, backed with wood to facilitate drilling and tapping to attach the Y-frame anchor of the caisson, was embedded in the concrete face of the model section. A glass panel was installed in one side to aid in observing the caisson when under water. An overhead track and chain-fall were used to transfer the caisson model to and from the tank.

12. The 1 to 42 scale model of caisson drydock. A decision to provide a cutting edge around the bottom rim of the circular drydock and sink it to its foundation by removing material from the center and pouring the wall as the sinking proceeded, made it necessary to construct the structure as a unit to withstand stresses that might be induced during this operation. To make the huge cylindrical structure continuous and rigid, it was planned to pour a reinforced-concrete slab in the

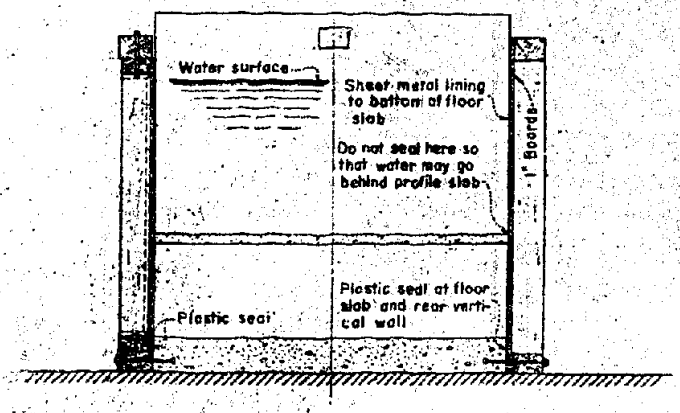
FIGURE 30



PLAN



SIDE ELEVATION



SECTION A-A

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASHINGTON

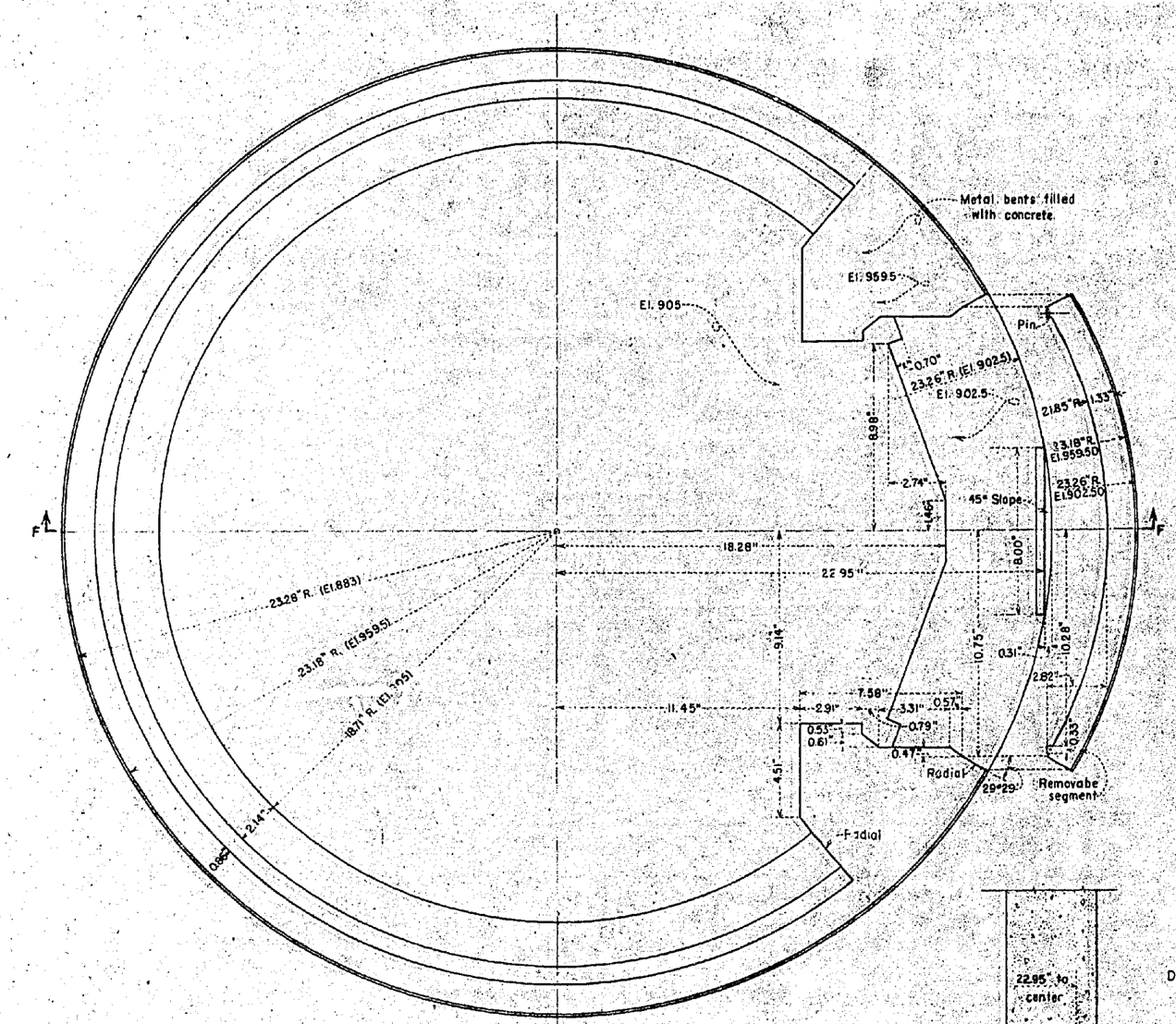
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES
MANIPULATION TANK FOR 1:20 MODEL FLOATING CAISSON

DRAWN H.R.S. SUBMITTED
TRACED H.R.B. RECOMMENDED
CHECKED APPROVED

gate section and remove it after the drydock was in place on its foundation. The fact that the 57-foot high by 80-foot wide by 8-foot thick slab had to be reinforced to make the structure continuous to perform the sinking operation satisfactorily, complicated its removal and disposal. However, after a study of the problem by the project engineer and the design section, a decision was made to tip the slab into an excavated area in the entrance channel adjacent to the drydock where it would remain to form a section of the floor of the channel. To accomplish the tipping it would be necessary to prevent bonding between the slab and the structure, chip the joints, and cut the reinforcing bars passing through them. Paint was to be placed between the surfaces of the joints and a steel tipping edge provided on the outer rim at the center of the slab, figure 31. After chipping the joints and cutting the reinforcing steel, the slab was to be tipped by hydraulic jacks placed near the top of the slab between it and drydock structure. At first it was planned to topple the concrete mass into a dry approach channel before the latter was excavated to the river. No difficulty was expected in accomplishing this operation. However, construction schedules and economical considerations made it desirable to retain the slab as a bulkhead to keep the drydock free of water to enable the fabrication of the floating caisson within it and eliminate the necessity of constructing a watertight crib across the opening. The urgency to make use of the caisson at the earliest date possible made it necessary to complete the excavation of the entrance channel, allowing water to come in contact with the slab, thus requiring it to be tipped when partially submerged. It was to investigate the feasibility of toppling this enormous slab into the excavated area with water surrounding it to the normal tailwater elevation that a 1 to 42 scale model of the drydock and slab to elevation 959.5 was constructed, figures 31 and 32.

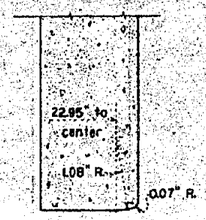
The base of the model was shaped by revolving a straightedge with a vertical offset horizontally about the center pivot. A small amount of molding plaster was added to the concrete to hasten its setting and the shape obtained by building up and scraping off surplus

FIGURE 31

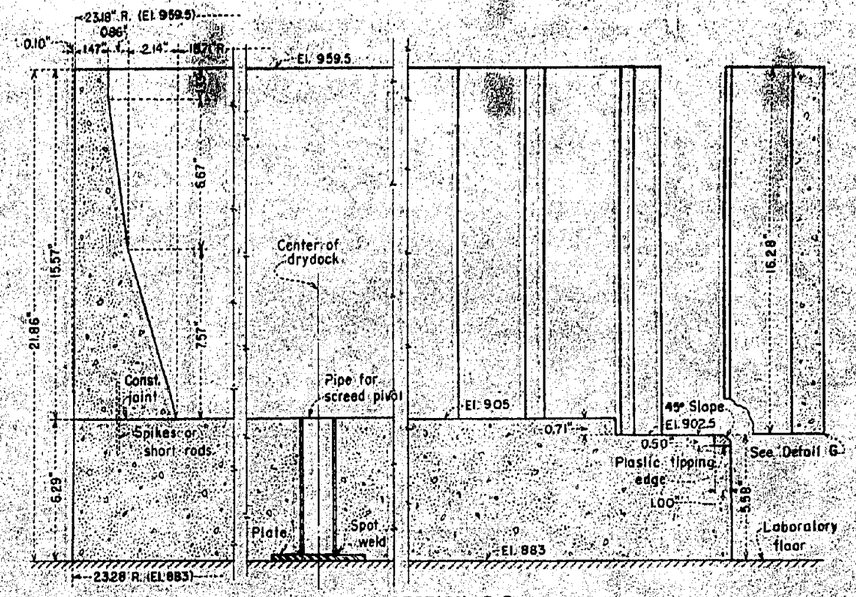


PLAN

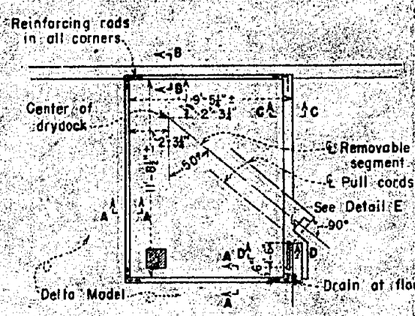
DRYDOCK



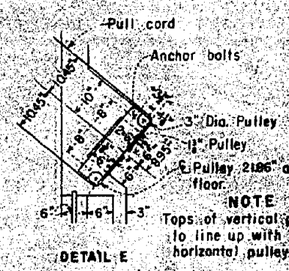
DETAIL G



SECTION F-F



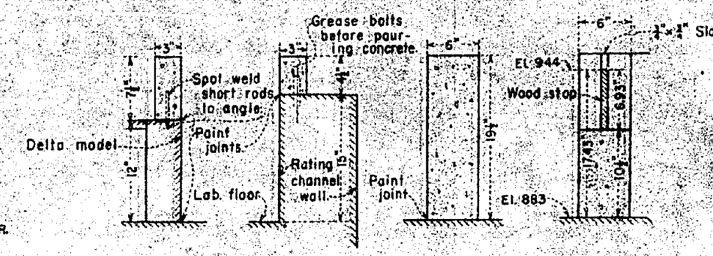
PLAN OF BOX



DETAIL E

NOTE

Tops of vertical pulleys to line up with ℄ of horizontal pulleys.



SEC. A-A SEC. B-B SEC. C-C SEC. D-D

TANK FOR 1:42 SCALE MODEL DRYDOCK

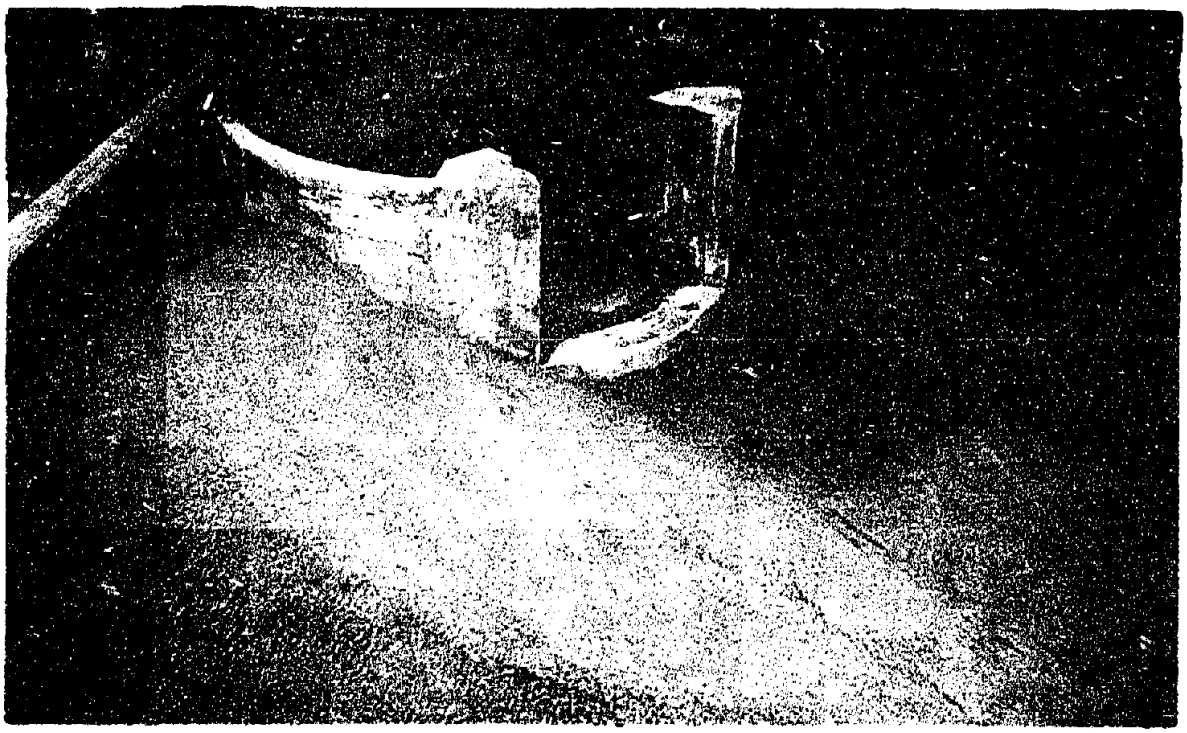
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT-WASHINGTON

GRAND COULEE DAM

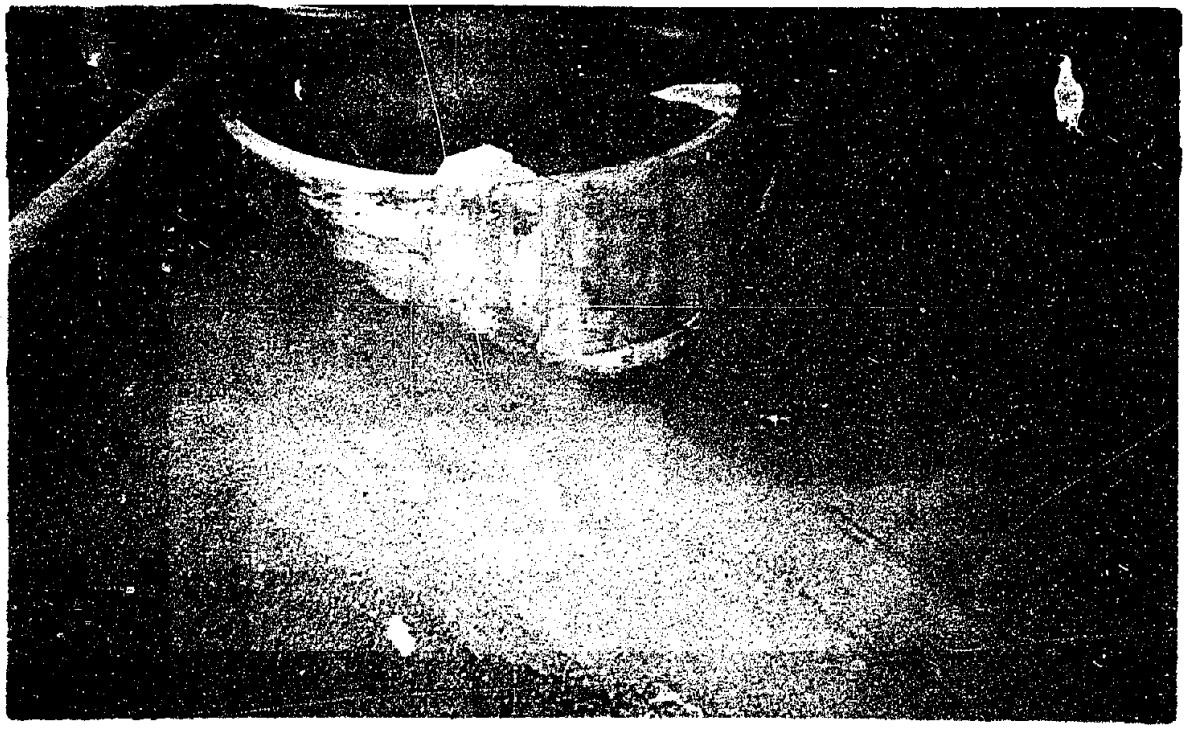
MAINTENANCE OF SPILLWAY BUCKET

DETAILS OF 1:42 SCALE MODEL OF CAISSON DRYDOCK

DRAWN N.R.A. SUBMITTED
TRACED M.L.L. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO



A. Drydock with slab removed from gate section



B. Drydock slab sector in place ready to be tipped

THE 1 TO 42 SCALE MODEL OF THE DRYDOCK FOR THE FLOATING CAISSON WITH GATE SLAB AND ADJACENT TOPOGRAPHY

mortar. The tipping edge shaped of plastic was embedded in the base at the center of the opening. The gate-anchorage columns on each side of the opening were formed by making a framework of sheet metal, supporting them with bolts embedded in the concrete base, filling the frames with a concrete-plaster mortar and smoothing the surfaces by drawing a straightedge across them.

The wall of the model drydock was constructed by swinging a negative metal template of the wall about the center pivot embedded in the base, building up and scraping off surplus mortar as in forming the drydock base.

The gate slab was cast from a cement-plaster mortar in a box mold of plaster, the mold, made by moving a positive template through setting plaster, being treated with a parting medium, filled with the mortar and the exposed surface smoothed by drawing a straightedge across it. Two small nails were embedded, with the heads protruding slightly, in the two upper corners on top of the slab opposite the tipping-jack recesses.

A system of grooved pulleys with ball-bearing mountings rigged with a lightweight fish line, placed on the edge of the concrete tank containing the model, was used to tip the model slab. Loops in the ends of the line were placed over the nailheads and the lines carried to the wall of the tank parallel to the center line of the drydock entrance channel over pulleys through horizontal bends toward the center line, thence over pulleys in a vertical bend to a small pail in which lead shot could be placed to produce the tipping force. The force to tip the slab was obtained by pouring shot into the pail until the slab started to tip. The force was then determined by weighing the pail and shot.

13. <u>The 1 to 60 plaster model of damage at base of right training wall.</u> A short section of the bucket, the base of the right training wall, and the topography in the immediate vicinity was assembled to visualize proposed repairs to the damaged areas at the base of the training wall downstream from the bucket lip, figure 33. The bucket section was constructed by screeding plaster mortar placed between

A. Eroded areas on inner surface of right training wall and bucket lip

B. Pebbles in eroded area representing 1- to 2½-foot boulders

C. Item 1 - Proposed concrete repair

D. Items 1,2, and 3 - Proposed concrete repair

THE 1 TO 60 MODEL OF RIGHT TRAINING WALL AND A SECTION OF THE BUCKET DEPICTING DAMAGE AND PROPOSED REPAIRS TO BASE OF THE WALL AND THE LIP OF THE BUCKET

templates. The training wall was of composition board and the topography of layers of upsom board covered with a papier-mache. The eroded area at the base of the wall was represented by gouging the plaster and composition board surfaces and three proposed stages of repair were represented by removable sections of plaster which had been poured into the eroded cavity.

14. The 1 to 24 and 1 to 30 models of outlets in right powerhouse. The first model used for studies pertaining to the outlet facilities to be provided in the right powerhouse for bypassing small flows around the spillway, was constructed on a scale of 1 to 24 and used previously for efficiency studies of the Grand Coulee draft tubes. The model, which included the scroll case and draft tube, with the wicket gates, but without the runner, was almost entirely of transparent plastic. The outer end of the draft tube was attached to a bulkhead across a 7-foot channel in the laboratory and the scroll case to it and to the supply system. The tailwater was regulated by varying the openings of gate valves in the tank on the water side of the bulkhead.

When it was proposed to use the 84-inch needle valves from Boulder Dam in the auxiliary outlets, the model scale was changed to 1 to 30 to utilize a model valve on hand in the laboratory. The draft tube and transition elbow between it and the model needle valve were constructed by pouring concrete in forms using cores of sheet-metal bents and modeling clay. Additional description, including photographs and a discussion of the results of the tests made concerning the various proposals is contained as appendix I of this report.

IV. THE INVESTIGATIONS

15. Study of bypass outlets in right powerhouse. As transportation of river-bed material and debris by eddy currents created by unequal distribution of flow in the spillway section during construction was considered to be a major factor contributing to the damage inflicted on the spillway bucket, it was deemed advisable to eliminate, or to reduce to a minimum, this undesirable flow distribution. The necessity for intermittent repair work to the face of the dam, repair and maintenance work on the spillway bucket by divers or with the special caisson, and frequent maintenance work on the spillway drum

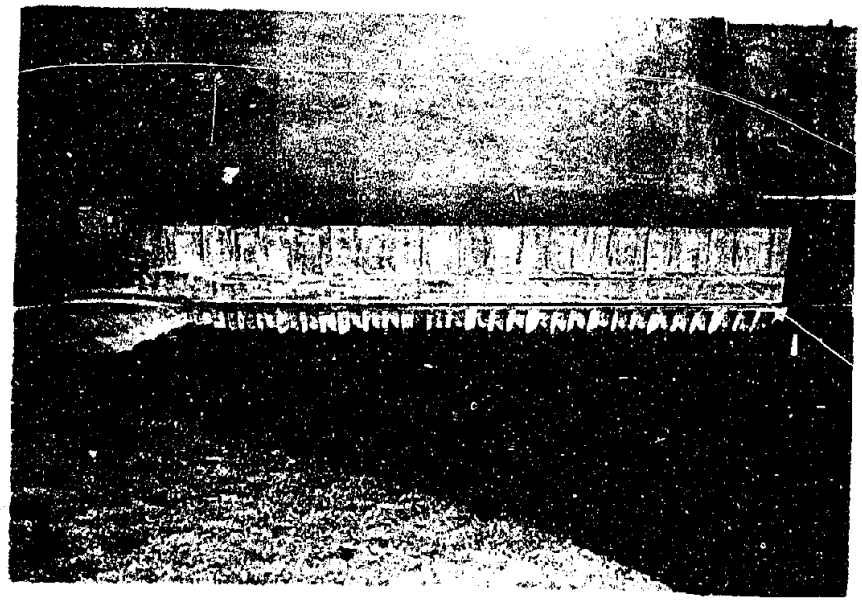
gates made it impossible under normal operation to distribute the flow evenly through the spillway section during most of the low-water season. Thus some means of alleviating the situation was sought. Development of power facilities to ultimate output would have assisted in this respect, but only if the demand for power existed to keep the units in operation during the low-water season. Completion of the irrigation features to pump water from the reservoir would help also; but an enormous amount of work remained to be completed, and this could not be accomplished for many years. Moreover, both these plans would require large quantities of strategic material which was not available for such purposes.

The only other method considered practicable in view of the urgency of performing repair and maintenance work on the spillway bucket was that of providing additional outlet facilities which would discharge in the river channel below the dam. Outlet works with the water bypassing the dam through tunnels were considered, but the cost was prohibitive and the plan was abandoned when others appeared more economical and equally as satisfactory. Consideration was given to developing some of the additional power facilities. Restrictions on the purchase of the materials and the time required for manufacturing and delivery, had the materials been available, made it necessary to abandon this scheme. Several variations, using the 84-inch needle valves from the canyon-wall outlets at Boulder Dam installed at the ends of the power penstocks in the right powerhouse, were investigated and a final arrangement evolved through hydraulic model studies (figure 15). The results of these studies have been presented in Hydraulic Laboratory Report No. 132, which is included as appendix I of this report. The approved plan placed the needle valves between the ends of the penstocks and the turbine draft tubes, with transitions from the penstocks to elbows on the upstream ends of the valves and a special transition elbow from the downstream ends of the valves to the draft tubes. The discharge from the valves was directed into the transition elbows where it was turned vertically downward into the turbine draft tubes.

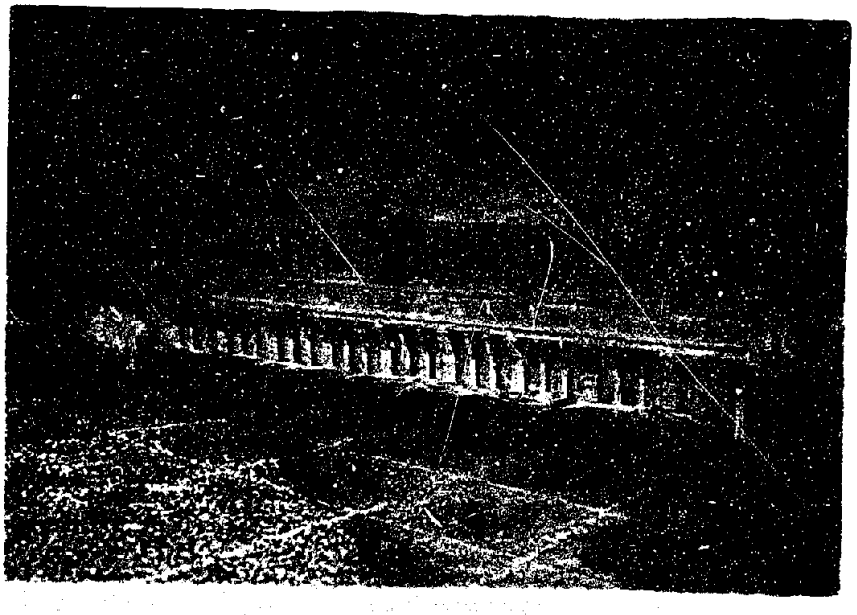
Detailed tests of the various arrangements were not made on the 1 to 60 model; but the final design, as determined by the studies described in Hydraulic Laboratory Report No. 132, was installed and observed under various operating conditions. Nine valves, one for each unit, were represented in the model, it being uncertain whether the prototype installation would include 8 or 9 of the 84-inch valves. The valves were represented on the model by flow nozzles designed to give the proper quantity at velocities corresponding to the prototype. The transition elbows were molded of transparent plastic, with flanges to connect to the valve and the draft tube. No unsatisfactory conditions were noted on the 1 to 60 model for this arrangement, figure 34.

16. Studies to place 1 to 60 model dam in operation. A number of preliminary or tune-up tests was necessary on the 1 to 60 model dam before the extensive program concerning the caisson was begun. Suitable gages for measuring the reservoir and tailwater elevations and for recording the discharge through the supply system were provided before these tests were made.

Among the first of these tests was the calibration of the supply system. An 8-inch calibrated venturi meter had been installed in an 8-inch branch of the model supply system to provide a means of measuring model flows representing prototype flows up to and including 100,000 second-feet (figure 35). Since information concerning the line forces and number of lines to maneuver the caisson to and from the dam was needed urgently, the 14-inch supply line was provided with a special pitot tube, which was calibrated temporarily and quickly by using the model crest as a measuring device, assuming the same coefficient curve as had been obtained in previous studies. The plan proved satisfactory since in all cases where accuracy was essential, the flows could be measured by the venturi meter in the 8-inch branch. The 14-inch line was calibrated later by traversing cross section of the pipe with a standard pitometer and recording e head differential on the special stationary pitot tube. Apparly the crest was more efficient than had been anticipated since alibration by the pitometer indicated more flow for the same

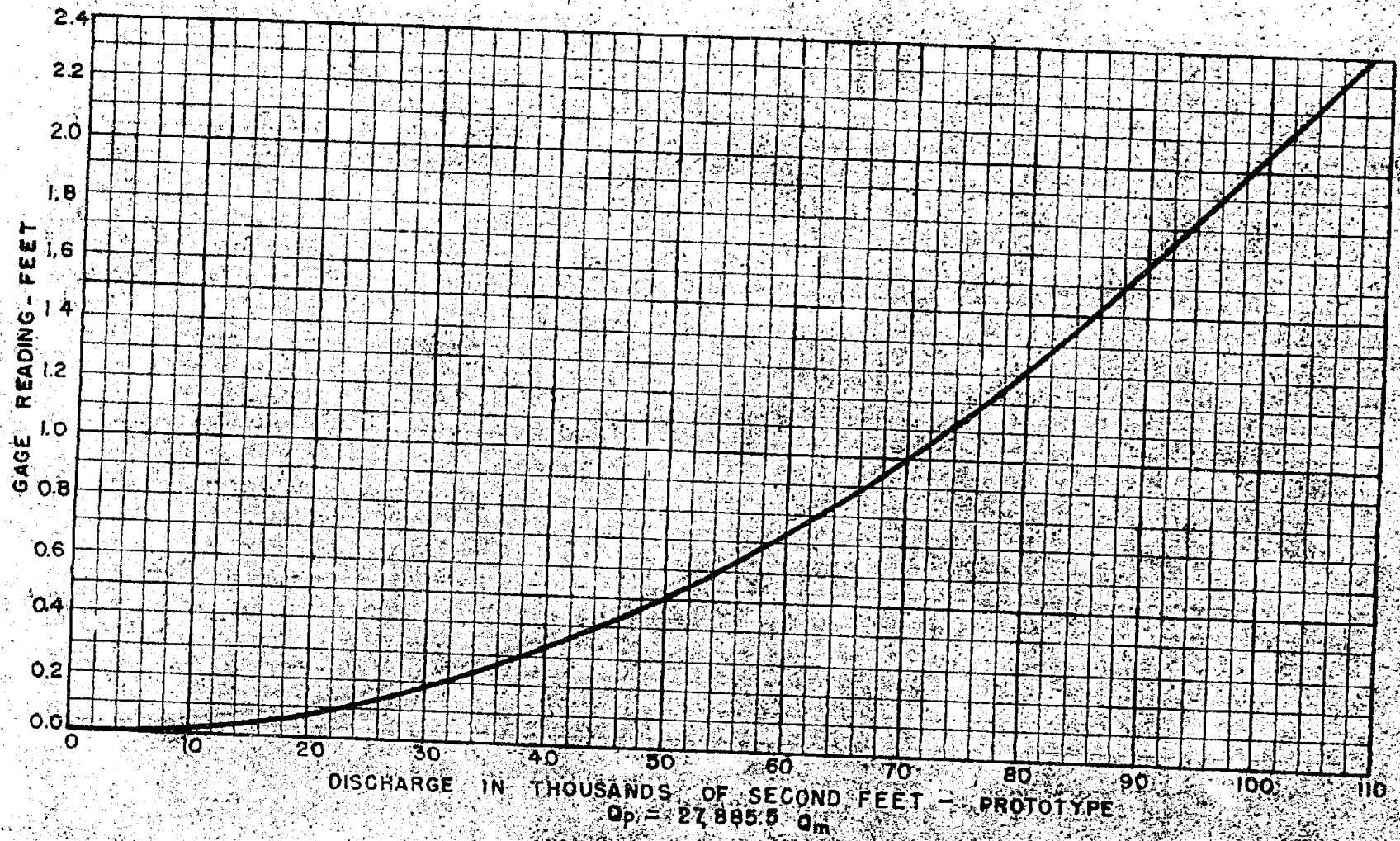


A - Three outlets in right powerhouse operating



B - Nine outlets in right powerhouse operating

NEEDLE-VALVE OUTLETS IN RIGHT POWERHOUSE OF 1 TO 60 MODEL DAM IN OPERATION - FLOW IN RIVER REPRESENTING 73,000 SECOND-FEET.



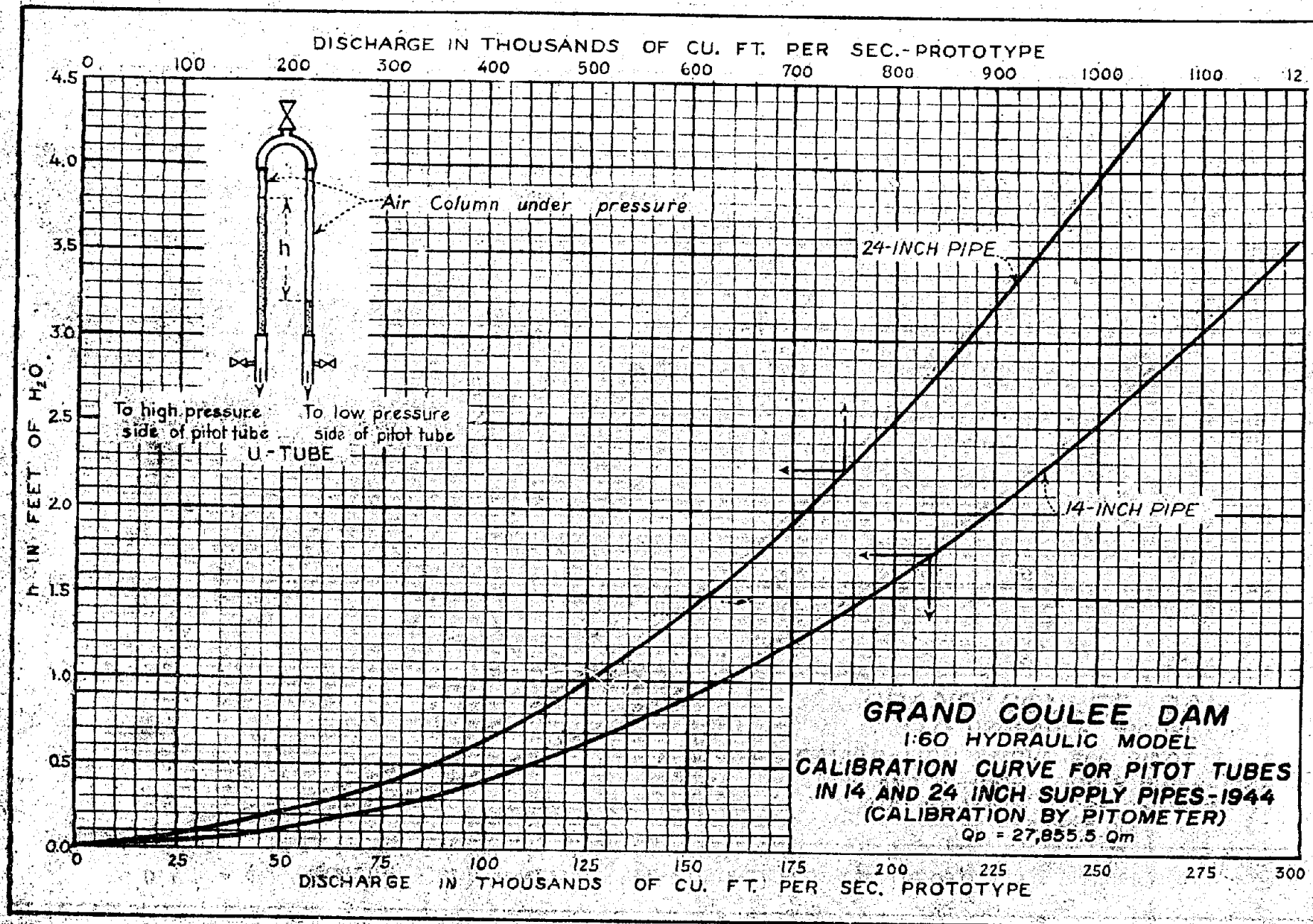
GRAND COULEE DAM
1:60 HYDRAULIC MODEL
CALIBRATION CURVE
8-INCH VENTURI METER IN MODEL SUPPLY LINE

FIGURE 7.35

differential head on the pitot tube indicator than when the quantity was based on the crest coefficients from studies made previously on models in the laboratory.

Data on the operating characteristics and behavior of the jet pump, the first to be used by the laboratory for supplying water to models, were obtained during the calibration tests. Pressures in the bellmouth entrance and in the mixing tube of the pump were measured and the maximum capacity noted. The pump had been designed using the momentum theory which under the operating conditions at the model had given an indicated ratio of total to driving quantity of 2.8. When computed from the data recorded, this ratio was 2.7. The maximum quantity was not as great as computed but the head on the pump was not as much as expected. During the spring of 1944, when tests concerning the deposition of material in the river channel below the dam were being made at the project, a large piece of wood was found lodged in the runner. The capacity of the pump increased notably when the piece was removed, but further tests were not made before the pump was replaced with a 24-inch vertical propeller pump for the 1944 test program concerning measures to minimize damage to the bucket and to study the protection of the tailraces and river banks downstream. From the tests, it was concluded that the momentum theory may be used advantageously in the designing of jet pumps.

The installation of the 24-inch vertical propeller pump and piping to supply more water to the model for continuing the studies, made it necessary to provide means for measuring the increased amount of water being supplied to the model. Pitot tubes were installed in both the 14- and 24-inch lines and traverses of the pipes made with a standard pitometer at various flows. Operating curves for each were prepared from these data but it was found that too few points were obtained to establish a definite curve for the higher discharges in both cases. As these sections of the curves were used very little and the error was small, additional data were not obtained until most of the program had been completed. The final calibration curves for the two pipes are shown on figure 36.



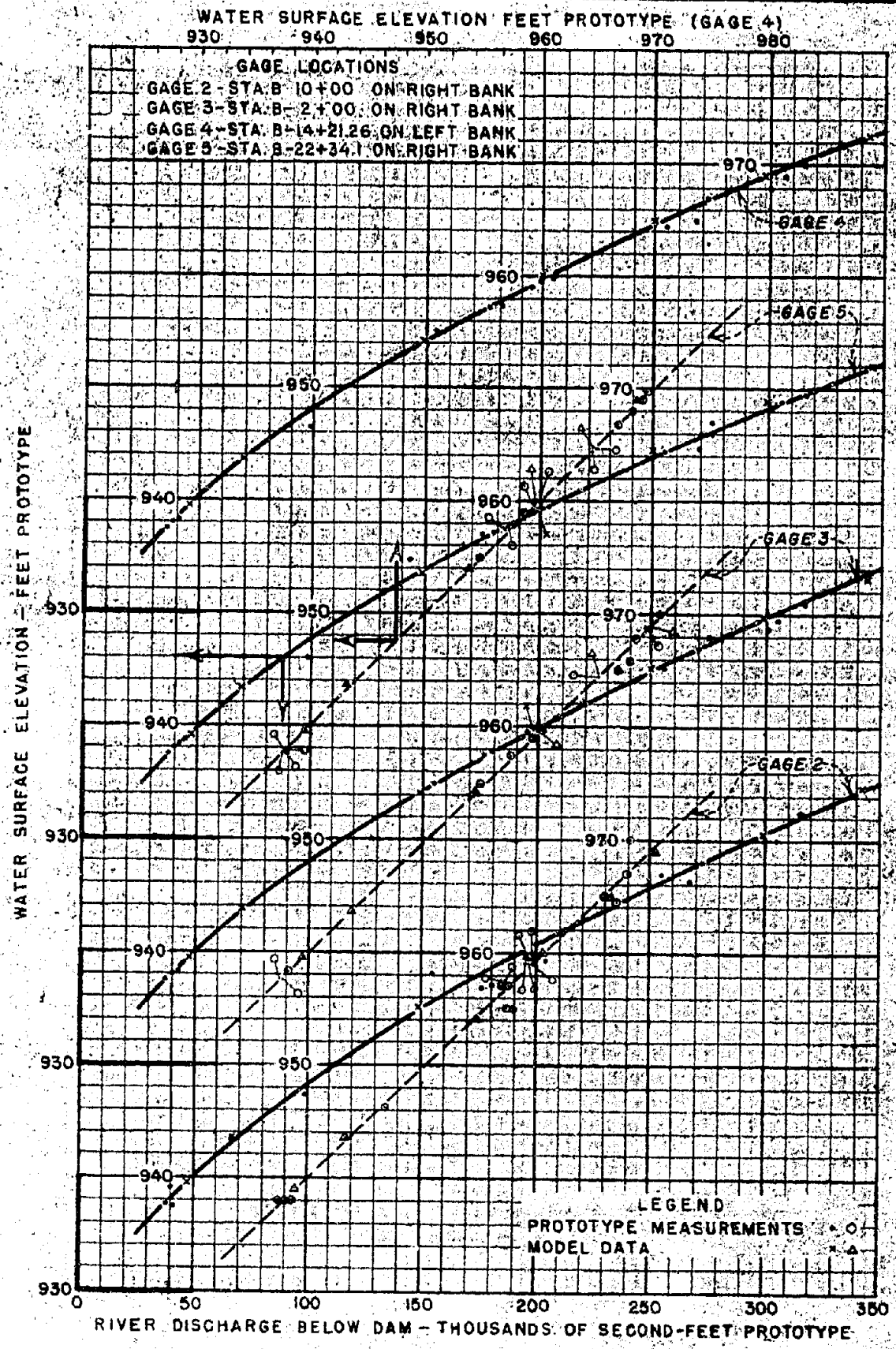
GRAND COULEE DAM
1:60 HYDRAULIC MODEL
CALIBRATION CURVE FOR PITOT TUBES IN 14 AND 24 INCH SUPPLY PIPES-1944
(CALIBRATION BY PITOMETER)
$Q_p = 27{,}855.5\ Q_m$

FIGURE 36

Many of the tests pertaining to the spillway-bucket inspection and repair caisson, particularly those concerning the maneuverability of the caisson, were dependent upon the existence of relative water-surface elevations at widely separated points in the river channel of the model to give proper velocity distribution. Since the configuration of this channel and its roughness were main factors controlling these factors, both were given careful consideration during its construction. It was possible to represent the configuration accurately since a comprehensive survey of the prototype river channel had been completed a few months prior to the construction of the model. The problem of roughness was not so simple. The banks immediately below the dam had been riprapped while those farther down the river had not. It was known that the river bottom and the lower levels of the banks contained a heavy blanket of rock varying from small sizes to huge boulders. The relative roughness would necessarily have to be high. From previous experience with similar models it seemed that a surface of pit-run gravel would be too smooth. The channel was constructed in two sections, one stable to represent the section of river not expected to undergo changes due to releases from the reservoir or to the installation of the bypass system in the right powerhouse, and the other being movable to provide for altering the model and to enable a study of the action in the immediate vicinity of the dam during the operation of its various hydraulic features.

Two sizes of aggregate were chosen. Aggregate of 1/4-inch maximum dimension containing no fine sand was used for the surface layer of porous concrete of the stable bed and a 1-1/2-inch layer of 3/16- to 3/8-inch aggregate was placed over a thoroughly packed sublayer of pit-run gravel to form the movable bed section. When the model was placed in operation and water-surface measurements made at stations corresponding to those used on the prototype for discharges varying from 45,000 to 345,000 second-feet, the elevations were found to be in excellent agreement with the prototype measurements, figure 37. Either the bed material chosen for the model after a casual survey of the prototype had been well-chosen or the configuration of the

FIGURE 37



GRAND COULEE DAM

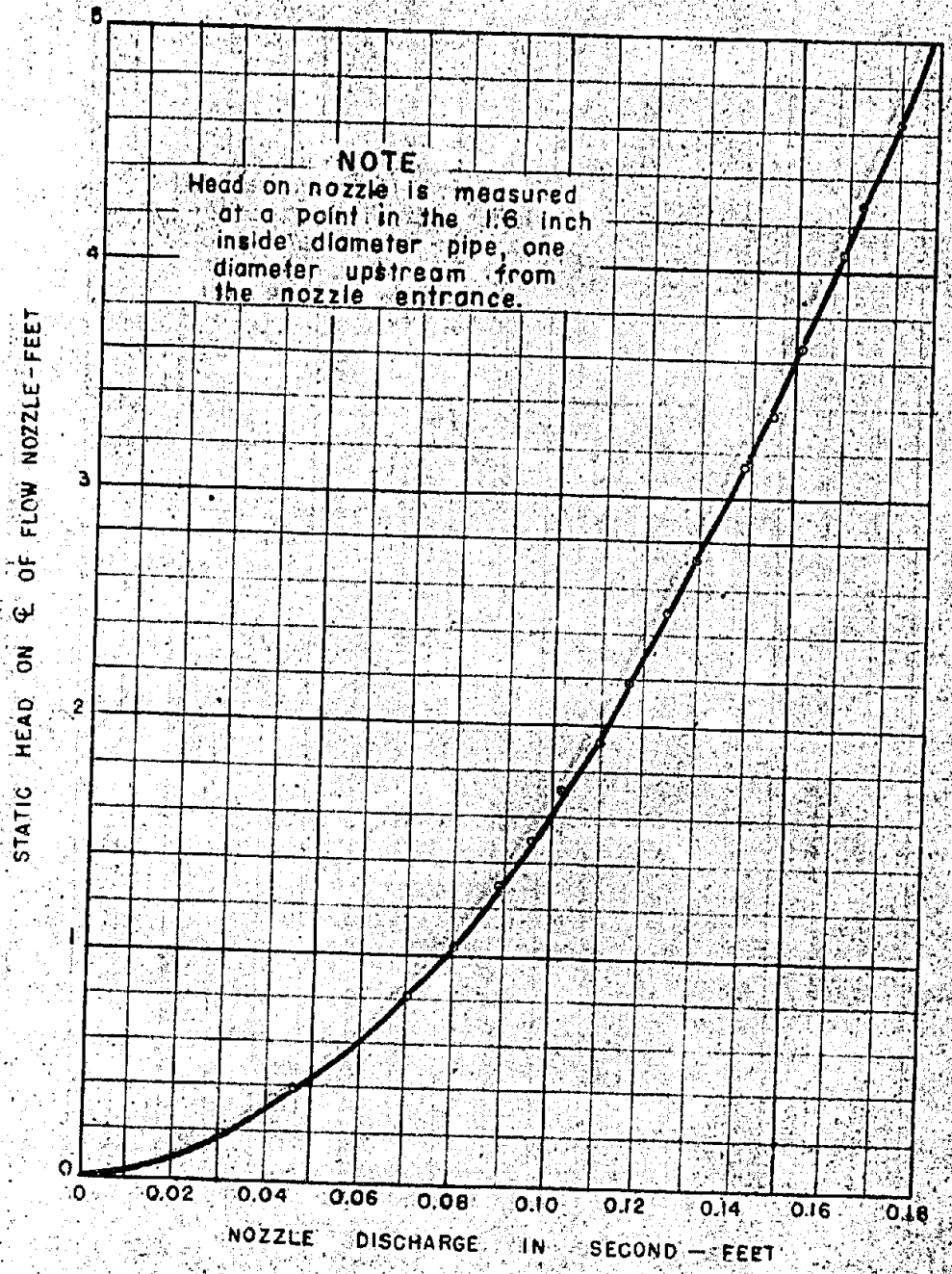
HYDRAULIC MODEL STUDIES 1:60 SCALE

COMPARISON OF WATER SURFACE GRADIENT IN RIVER BELOW DAM-MODEL-PROTOTYPE

channel, built carefully according to current surveys, was the governing factor. All data were within the limits of experimental error. With this agreement established it was possible to begin the extensive program which had been planned in connection with the design, construction, and operation of the caisson for inspecting and repairing the spillway bucket.

The flow from the model powerhouses was to be regulated by cylinder gates designed for that purpose. However, the installation of needle valves in the right powerhouse prevented using them in this part of the model and they were installed in the left powerhouse only. Special valves were used for the bypass system in the right powerhouse, which was represented by flow nozzles and transition bends which directed the nozzle flow downward into the turbine draft tubes. The use of the flow nozzle was possible in this case since the prototype valves were to remain either in the open or closed position. One of the flow nozzles was calibrated in the Denver laboratory and a discharge curve prepared, figure 38. Pressures in each of the nine units were recorded on the model and although the distribution was not equal, the variation in the quantity was negligible. Flow conditions in the transparent bend and those in the tailrace were noted. No adverse conditions were observed although for certain combinations a current formed near the tailrace slope which had velocities representing 3.6 to 6.5 feet per second prototype. It is believed that better operation of both of the model powerhouses might be obtained by improving the scheme used in the right powerhouse and making individual feeders to the turbine draft tubes in the left structure. This should be given consideration if the model is to be used extensively for demonstration.

During the period of testing it was desirable to construct a special tailwater regulator to maintain automatically the natural rating curve of the stream below the dam. Several tests were made to ascertain if alterations to the tailwater gate in the downstream sump would accomplish the desired results. It was decided that automatic regulation could not be obtained in this manner as a weir with a long crest would be necessary. Since it was certain that automatic



GRAND COULEE DAM

HYDRAULIC MODEL STUDIES 1:60 SCALE

DISCHARGE CURVE FOR FLOW NOZZLES IN RIGHT POWER HOUSE OF MODEL

regulation could be attained by constructing a suitable barrier at the end of the river channel as it enters the downstream sump, no further tests were made concerning this problem.

The river outlets, using rubber corks on hinges as gates, were found to operate satisfactorily for the test program. Air vents for the condition where the spillway only operates were provided through transite pipe embedded in the model. This was not the original intention but the vents to the outlets were overlooked until construction progress prevented their inclusion. The air vents may be permanently attached to these pipes and the operating mechanism for the gates, to be installed in the future, operated from submerged lines in the model reservoir.

The action of the model distribution system, with its rectangular conduit, openings, and baffle plates, was noted for the complete range of discharge. There was a minimum amount of turbulence in the model reservoir.

Tests made previous to the construction of the 1 to 60 model dam, using a short length of crest and an 0.8 downstream slope, to investigate the possibility of representing the white cotton-tuft appearance of the prototype flow, indicated that some measure of success could be realized by injecting air into the underside of the spillway flow. The arrangement which gave promising results on the short section failed, however, when placed on the long section of the model spillway. The air would collect at the higher positions of the entrainment device and lift the flow from the face of the spillway. Considerable noise accompanied its operation. When the entrainment device failed, sections of screen of different mesh were fastened to the face, producing different degrees of roughness. The flow appeared somewhat whiter for this arrangement than for the smooth face of the model for thin sheets of water, but they were very ineffective for high flows. Apparently the roughness would have to be of such magnitude as to be impracticable. Further tests will be required to corroborate this conclusion.

In preparing the 1 to 60 model caisson for the maneuvering tests it was found that the draft was too shallow. The condition was

attributed to the difference in weight of the preliminary design, from which the model was constructed, and the current design. Four brass weights of about 180 grams were fastened to the deck adjacent to the side girders to correct this condition. Initially the shallow draft was believed to be the result of increased buoyancy introduced by the out-of-scale thickness of the plastic sheet used in constructing the model. Computations however did not indicate this to be the case. The additional weight required to produce the computed value was too much to be gained from the displacement due to the excess thickness. The small size of the model made it impracticable to study the sinking and raising operations. It was too difficult to keep the caisson under control without introducing errors from outside sources. Moreover, a trace of instability seemed to exist, a condition in itself sufficient to justify the construction of a larger and more complete model. Excellent stability was noted for the maneuvering tests when the entire unit floated similar to a large barge.

17. Studies to determine resistance of caisson to currents of different velocity. The sizes of the maneuvering lines for moving the caisson to and from its position over the spillway bucket were dependent upon the forces exerted on the cables during the maneuvering operation. These forces, in turn, were dependent upon the shape of the submerged portion of the caisson, the velocity of the flowing water, and the angle of the cable with the resultant force on the caisson. The velocities in the tailrace for the various flow combinations which might exist during the maneuvering were unknown as was the coefficient of resistance for the unusual shape of the submerged portion of the caisson. The angles of operation of the various lines were unknown since the position of the shore ends could not be established advantageously except by maneuvering tests conducted on a model in which the magnitude and direction of the currents were represented correctly. Since none of these factors could be determined satisfactorily without model tests and it was necessary to establish criteria for design purposes months in advance of the tests, certain assumptions were made for the preliminary stages of design. The maximum velocity in the

portion of the river channel in which the maneuvering was to be confined was assumed to be five feet per second. A value of 1.5 was selected for the coefficient of resistance and temporary locations for the shore ends chosen. These assumptions resulted in high values for the pull on the various lines, as was established later from the model studies.

As soon as the model was placed in operation and equipment could be assembled for testing, the resistance of the caisson broadside and head-on to currents of different velocity was measured. The caisson model was placed in a section of the model river channel (sta. B-12+00) after the velocities had been determined from current-meter traverses.

A two-line Y-hitch was attached to the top of the upstream girder and the downstream corner of the caisson, and a torsion dynamometer attached to the free end to obtain the magnitude of the forces for holding it against the current. The same hitch was placed at the two corners on top of the front end girder for obtaining the forces to hold it head-on into the current. The list of the caisson produced by the different velocities when it was broadside to the current was measured also.

When the data was first analyzed, the recorded minimum, average, and maximum forces were plotted against the average velocity at the section. The points were scattered, with the deviation greatest for the higher velocities. Due to turbulence in the flowing water, the current in the section was unsteady, the velocity varying from point to point and time to time. Thus it seemed reasonable to assume that the variation in the force on the Y-line was due to the changes in velocity particularly since the submerged area of the caisson was small compared with the area of the flowing water. The maximum force was therefore plotted against the maximum velocity measured in the section, the average against the average, and the minimum against the minima. The results were more consistent.

The caisson listed noticeably with the two-line Y-hitch and uncertainty was felt as to whether or not the resistance coefficient was correct for the shape. The test was repeated using a three-line Y-hitch

which held the side of the caisson more nearly perpendicular to the flow than had been the case with the two lines. The coefficient, taking into account the decreased draft on the upstream side, was the same as for the two-line arrangement.

The resistance coefficient, as determined from the model, for the broadside condition was approximately 1.3, figure 39, while that for the head-on condition was 0.65.

The resistance of the caisson broadside to the current for velocities up to and including 8 feet per second were compared with that obtained from the following expressions using a coefficient of 1.5:

$$P = CAV^2$$

where P = resistance (or total pull on line), pounds

C = the resistance coefficient

A = area of submerged face of caisson, square feet

V = velocity of current, cubic feet per second

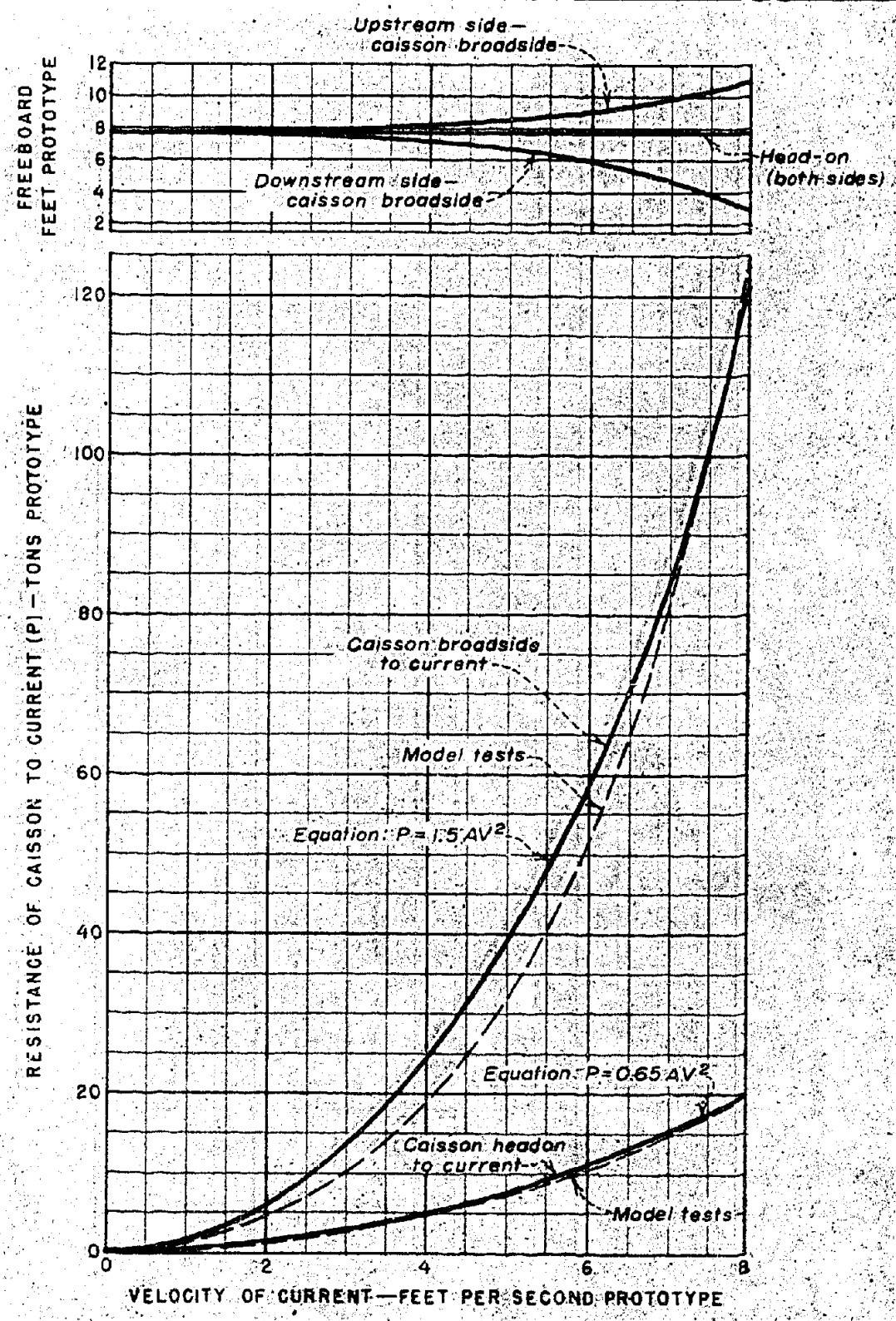
The coefficient of 1.5 gave a greater resistance than the model data for most of the range of velocities considered. The structure listed dangerously for the higher velocities, figure 39.

A resistance coefficient of 0.65, using the above expression, gave values coinciding with those obtained from the model for the head-on condition. The difference in shape of the submerged area for the broadside and head-on conditions are believed the reason for the different coefficients.

As a check on the reliability of the total resistance of the caisson broadside to the current an analysis of data taken during the maneuvering tests described in a subsequent section of this report was made by the design section. Very good agreement was revealed.

18. A study of conditions influencing the maneuvering of the floating caisson. Records show that the flow of the Columbia River approaches 500,000 second-feet occasionally, exceeds 100,000 second-feet from three to seven months of each year, and remains in excess of the present capacity of the left powerhouse during much of the low-water season. These conditions would prohibit repair work in the bucket during high water and make it necessary to remove the floating caisson from the spillway

FIGURE 39



GRAND COULEE DAM

HYDRAULIC MODEL STUDIES—1:60 SCALE

RESISTANCE OF CAISSON TO CURRENTS OF DIFFERENT VELOCITY

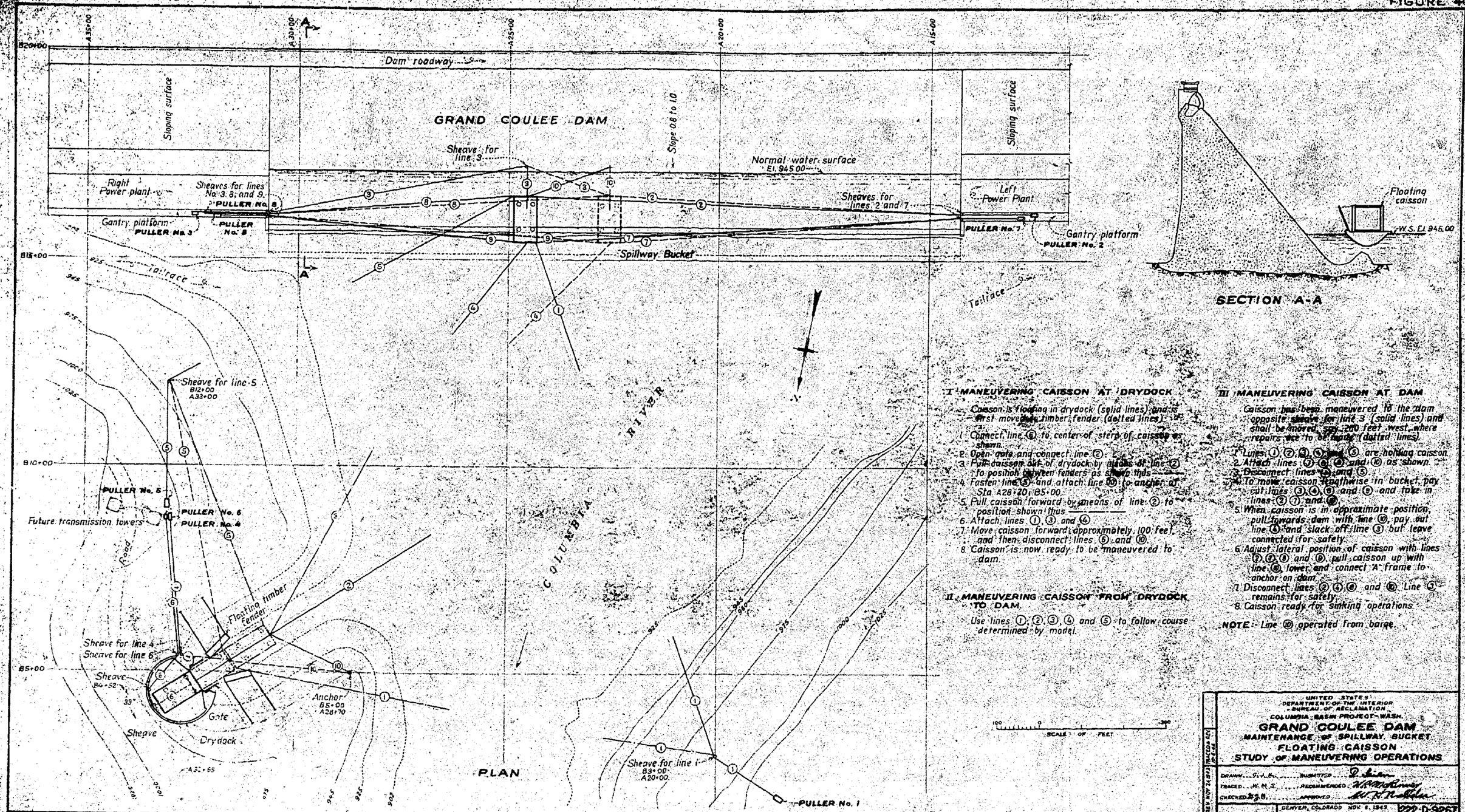
bucket each spring, transfer it to the drydock approximately 1,200 feet downstream for protection during the flood season, and return it to the dam in the fall after the flood waters had subsided. It was proposed that the transfer of this equipment be accomplished by a system of puller machines located at various points on the river banks, the powerhouses, and the dam, with cables leading from them to hitches on the caisson. The final arrangement of this system is shown schematically in figure 40.

There was no precedent for establishing the size, number and arrangement of the lines to accomplish the maneuvering operation, because of the unknown coefficient of resistance of the unusual shape of the submerged portion of the caisson to currents of various velocities and the unpredictable velocity distribution in the river channel immediately below the dam. Tests were conducted on the 1 to 60 model dam using the caisson constructed of transparent plastic to obtain these data.

As it was desired to move the caisson to the bucket at the earliest opportunity each fall and return it to the drydock as late as possible in the spring to obtain maximum working periods for making the bucket repairs, it was necessary to maneuver under conditions with maximum flow through both powerhouses. At the time the tests were made, the installation of nine 84-inch needle valves in the right powerhouse was considered; later the number was changed to eight. With a capacity of 4,000 second-feet each, the outlet flow from the right powerhouse would be 36,000 second-feet. The estimated flow for the left powerhouse was 37,000 second-feet, making a total flow of 73,000 second-feet for the maneuvering operations; no flow through the river outlets or over the drum gates.

The currents in the river channel of the model opposite the entrance to the drydock were not changed noticeably for combinations of five and six valves operating at the right end of the right powerhouse. Less concentration of velocity resulted when those at the left end were operating. However, the best distribution with less concentrated currents resulted when valves at each end were operated and those in the center

FIGURE 40



I MANEUVERING CAISSON AT DRYDOCK

Caisson is floating in drydock (solid lines) and is first moved to timber fender (dotted lines).

1. Connect line ⑥ to center of stern of caisson as shown.
2. Open gate and connect line ②.
3. Pull caisson out of drydock by means of line ② to position between fenders as shown thus
4. Fasten line ③ and attach line ⑩ to anchor at Sta A28+70, B5+00.
5. Pull caisson forward by means of line ② to position shown thus
6. Attach lines ①, ③ and ④.
7. Move caisson forward approximately 100 feet, and then disconnect lines ⑥ and ⑩.
8. Caisson is now ready to be maneuvered to dam.

II MANEUVERING CAISSON FROM DRYDOCK TO DAM.

Use lines ①, ②, ③, ④ and ⑤ to follow course determined by model.

III MANEUVERING CAISSON AT DAM

Caisson has been maneuvered to the dam opposite sheave for line 3 (solid lines) and shall be moved, say, 200 feet west, where repairs are to be made (dotted lines).

1. Lines ①, ②, ③, ④ and ⑤ are holding caisson.
2. Attach lines ⑦, ⑧, ⑨ and ⑩ as shown.
3. Disconnect lines ④ and ⑤.
4. To move caisson lengthwise in bucket, pay out lines ③, ④, ⑧ and ⑨ and take in lines ②, ⑦ and ⑩.
5. When caisson is in approximate position, pull towards dam with line ⑩, pay out line ⑤ and slack off line ③ but leave connected for safety.
6. Adjust lateral position of caisson with lines ⑦, ⑧, ⑨ and ⑩, pull caisson up with line ⑩, lower and connect A frame to anchor on dam.
7. Disconnect lines ②, ④, ⑧ and ⑩. Line ③ remains for safety.
8. Caisson ready for sinking operations.

NOTE:- Line ⑩ operated from barge.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT—WASH.

GRAND COULEE DAM
MAINTENANCE OF SPILLWAY BUCKET
FLOATING CAISSON
STUDY OF MANEUVERING OPERATIONS

DRAWN S.V.R. SUBMITTED
TRACED W.M.S. RECOMMENDED
CHECKED APPROVED

DENVER, COLORADO NOV. 6, 1945 222-D-9267

REV. NOV 12, 1945 TRACED NOV 6, 1945

were closed. The reason for this was not apparent, but it was believed to be due to a redistribution of the currents in the tailrace. Closing one or two valves had little effect on the flow conditions regardless of position, while closing three or four reduced the velocity concentration noticeably along the right bank. From these observations it was concluded that the conditions for any combination would not be more severe than for those existing with all the valves operating at full capacity.

Various flow combinations (various units in both powerhouses operating) were observed on the model to ascertain if some produced more severe conditions than others. Most of the tests were conducted using the maximum powerhouse flow, since the conditions for the other combinations were as good or better. However, the maneuvering characteristics were studied and line forces recorded for some of the combinations most likely to exist during the maneuvering operations and for one in which part of the flow was passed over the left end of the spillway and the current from the right powerhouse eliminated.

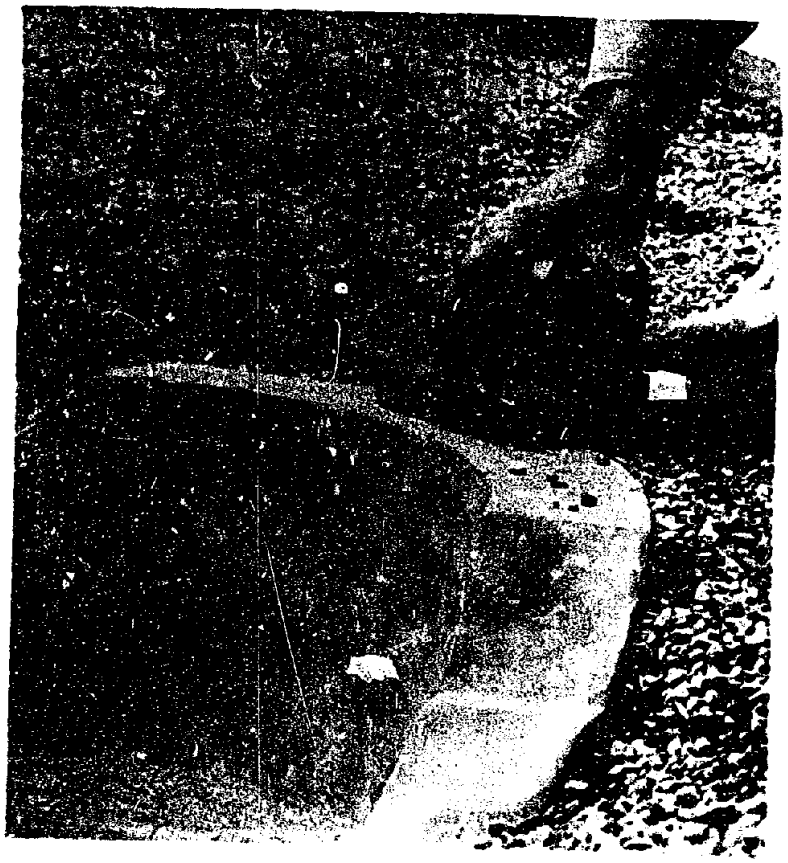
After observing the flow conditions, with both powerhouses operating, and noting the action of the caisson when it was being maneuvered, it was believed that the line forces for maneuvering the caisson could be reduced considerably by manipulating the drum gates. By passing the flow over the gates at the left end of the model spillway instead of through the right powerhouse, it was possible to have the currents, through which the caisson would be towed, move in an upstream direction. With this combination the caisson could be taken upstream to the right of the flow from the spillway and left powerhouse, to a point just below the dam and upstream from the path of flow from the right powerhouse. When in this position, the flow could be transferred from the spillway to the right powerhouse and the caisson then moved to its position over the bucket. However, the flow conditions would be unsymmetrical and material no doubt would be carried back into the bucket unless the dredging below the bucket, discussed subsequently in this report, has been completed. Also, the flow below the dam was rough and might interfere with handling the maneuvering lines and the scheme was not considered seriously even though the line forces were found to be small.

Special drum dynamometers, making use of the angular deflection of a steel wire under torsion to measure the magnitude of the force on each line, were designed for the maneuvering tests (figure 41A). With these instruments, which were equipped with a lightweight fish line that could be reeled in or out to move the caisson through the downstream river channel, it was possible to study the maneuverability for various shore positions of the lines to ascertain the number of lines required and to determine the magnitude of the pull on each line whether the caisson was moving or held stationary.

A system of lines, forming a grid of 20-inch squares, representing transverse and longitudinal stations of the prototype, was painted in yellow on the river bed to facilitate locating and establishing the angular position of the caisson at the time readings were taken.

At first it was believed desirable to record the line forces while the caisson was in motion, but this procedure did not yield data for direct comparison because of the difficulty of coordinating the actions of the operators and obtaining the location, position, and force readings simultaneously. Maximum readings on the drums carrying the major portion of the load were obtained during the maneuvering, but the caisson was maneuvered into position and held stationary with all lines taut for the line-force and position readings. The path through which the caisson was maneuvered, giving its position and direction at the time the readings were taken, was recorded schematically on graph paper with all pertinent data supplied in tabular form on the same sheet.

The direction of the currents and the magnitude of the velocities in the tailrace and river channel below the model dam to station B4+00 were obtained for the various flow combinations investigated. Since conditions with the maximum flow through both powerhouses were the most severe, data for that arrangement only is contained in this report (figure 42). From these data it was possible to check the correctness of the line-force measurements. Computations using these data and a directional position, recorded when line forces were measured, gave values which were in good agreement with those recorded by the drum dynamometers. Maximum velocities through which is was necessary to



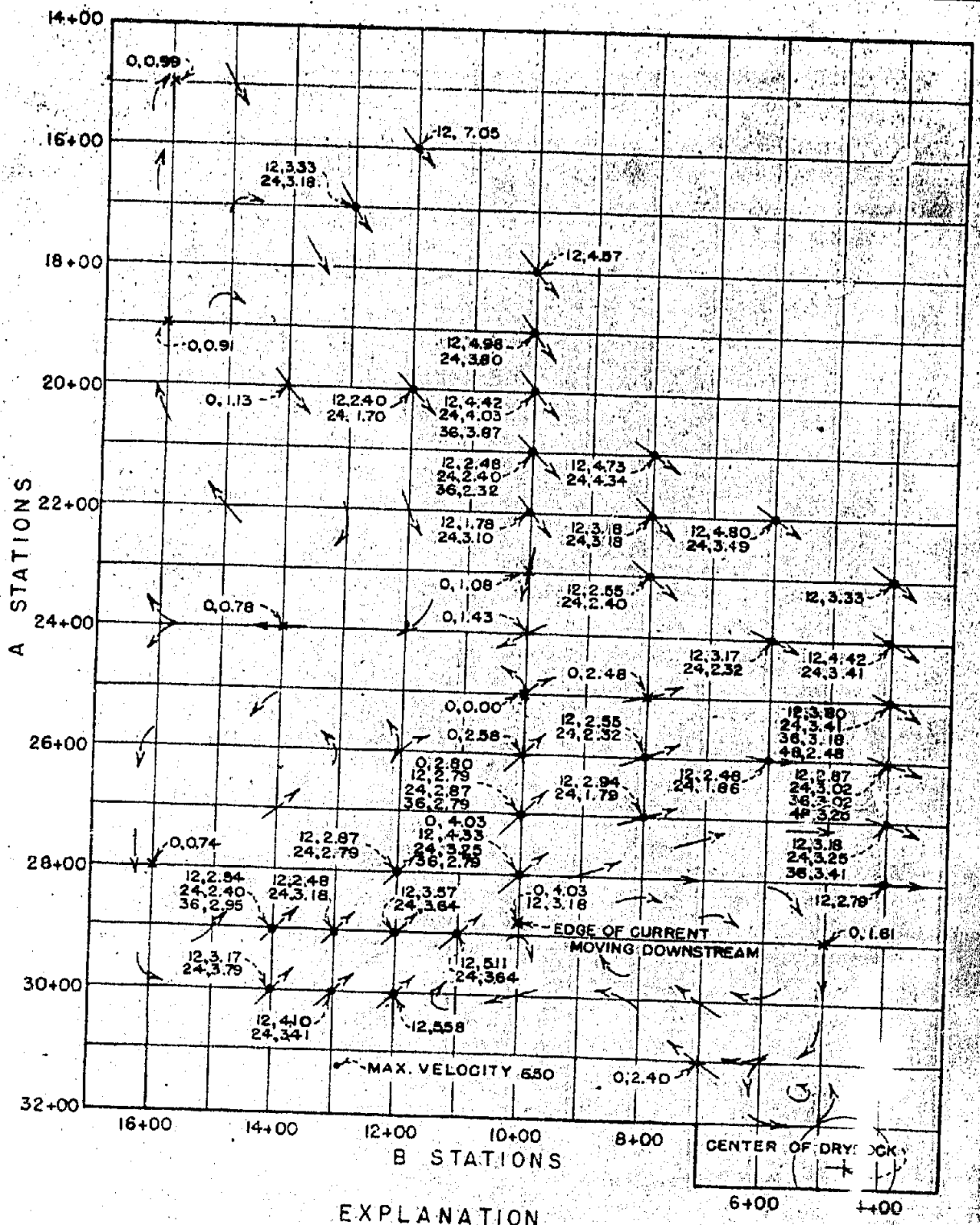
A. Dynamometer for determining pull on maneuvering lines

B. Water-surface recorder to measure rate of fall for power outages

DEVICES USED ON 1 TO 60 MODEL DAM TO DETERMINE PULL ON MANEUVERING LINES OF CAISSON AND TO RECORD DROP OF WATER SURFACE FOR POWER OUTAGES

Figure 41

FIGURE 42



EXPLANATION

- • Current meter stations
- x Surface velocities using confetti
- → Direction of current

Numbers accompanying arrows give depth in feet below water surface and velocity in feet per second at the given depth

Right powerhouse discharge = 3 000 c.f.s. (9 valves)

Left powerhouse discharge = 3 .000 c.f.s.

Tailwater elevation 944.1

All figures given are prototype

GRAND COULEE DAM

1:60 SCALE MODEL

CURRENTS AND VELOCITIES IN DOWNSTREAM RIVER CHANNEL

maneuver the caisson were recorded in the flow from the right powerhouse bypass. A velocity of 4.3 feet per second (prototype) was measured near station A28+00, B10+00.

In general, the flow from the powerhouses continued downstream along the banks, but eddies were formed downstream from the spillway section and in the entrance channel to the drydock. Clockwise eddies formed below the left end of the spillway and in the drydock entrance, while one moving counterclockwise formed below the right end of the spillway. Flow in the upstream direction between the two eddies below the spillway section occurred along station A24+00.

19. Studies to determine the number of lines required to maneuver the floating caisson. Preliminary tests indicated that the axis of the approach channel to the drydock should coincide with the longitudinal axis of line 2 with its shore end in the vicinity of the right end of the left powerhouse when the caisson was being moved from the drydock and fenders to the river. This change was made when the drydock was relocated, before the main part of the test program was begun.

Initially tests were made with maximum powerhouse flow, using four lines which were numbered to facilitate testing and the recording of data. The shore end of line 1 was placed on the opposite bank downstream from the drydock; that of line 2, at a point representing elevation 955 on the face of the dam 50 feet inside the left training wall; that of line 3, at elevation 955 on the face of the dam 300 feet inside the right training wall; and line 4, on the bank just upstream from the drydock. Lines 1 and 4 were attached to the downstream left and right corners of the caisson, respectively, and lines 2 and 3, to the upstream center. Though the arrangement differed somewhat from the final system shown on figure 40, it was similar. It was impossible to keep the caisson headed into the current with this arrangement, and the shore end of line 3 was moved to the top of the right powerhouse in an effort to improve this condition. The maximum line forces which had varied from 0.3 to 0.5 pound (model) were reduced by this change over most of the distance, but they increased rapidly as the caisson neared the dam, reaching approximately the same values, thus demonstrating the necessity of a fifth line to maneuver it to the face of the dam.

The large magnitude of these forces was attributed to two factors. First, the angles of the lines with the resultant force on the caisson approached 90 degrees, causing high tensile stresses in the two lines, and secondly, the caisson could not be headed into the current with the four lines to reduce the force acting against the two lines. Some decrease in these forces was obtained by moving the caisson into the stream opposite and slightly upstream from the drydock and then towing it upstream to the face of the dam. The operation was considered too limited, and the fifth line was considered essential. Moreover, this arrangement of lines made it difficult to maneuver the caisson in the entrance channel near the drydock.

However, to make certain that the maneuvering could not be performed satisfactory using four lines, the tests were continued. The shore end of line 3 was moved again to the point 300 feet inside the right training wall, and that of line 4 to the back of the drydock. Also, the caisson ends of lines 1 and 4 were transferred from the corners to the downstream center, since the arrangement had proven so satisfactory in the previous tests for lines 2 and 3. The maneuverability was improved slightly but the shore positions were not suitable for heading the caisson into the current from the right powerhouse. Moreover, it was not possible to arrange the shore positions to accomplish this without decreasing the maneuverability. The pull on lines 3 and 4 remained high, except when the maneuvering was limited to a definite path in which the caisson was first moved into the river opposite the downstream edge of the entrance channel, allowing lines 2 and 3 to take the load, and then being drawn directly upstream to the dam by these lines. Data for several tests with the shore ends of lines 1, 2, and 4 kept at the same location and that of line 3 at points 90, 210, 410, and 610 feet inside the right training wall indicated the same trend. In these tests the shore end of line 1 was at station A19+00, B2+50; that of line 2 on top of the spillway end of the left powerhouse, and that of line 4 at the back of the drydock opposite the gate opening. For the same maneuvering path the pull on lines 3 and 4 increased as the shore end of line 3 was moved toward the center of the spillway. This was attributed to the

increase in the angle between these lines and the resultant force on the caisson. In all cases excessive forces were produced by improper maneuvering, such as the lines being operated to pull against each other.

20. Tests to establish the optimum location for the shore ends of the maneuvering lines. Since it was not desirable to confine the maneuvering to a definite path to reduce the line pull, and the forces seemed to be excessive when the caisson approached the dam, a fifth line (line 5, figure 40) was added. For the initial tests with this line, its shore end was placed on the right bank at station A32+00, B10+00. With the shore end positions of lines 1, 2, 4, and 5 as indicated above, and that for the No. 3 line 90 feet inside the right training wall, the forces on line 3 remained high, since the components of lines 1, 4, and 5 opposed it. With the shore ends of this line near the right training wall it was necessary to maneuver the caisson in the outer edges of currents from the outlets in the right powerhouse where higher velocity existed near the dam. Moreover, the force distribution on the lines was upset due to the unfavorable angularity of some of the lines. As the shore end was moved toward the center of the spillway, the magnitude of this force decreased and the maneuvering and control of the caisson in the area below the spillway was improved. However, the change resulting from this movement was small for any appreciable distance (200 feet or more) from the right training wall, thus it was concluded that the location need not be fixed exactly. A distance of 610 feet inside the right wall was chosen arbitrarily for all subsequent tests except those in which part of the river discharge passed over the spillway gates, when it became necessary to move it closer to the training wall to prevent interference from the water passing over the spillway. With this arrangement, the load distribution on the lines was most satisfactory when the caisson was maneuvered into position over the bucket directly downstream from the shore end of this line.

With the shore end of line 1 at station A19+00, B2+50; that of line 2 on top of the left powerhouse; that of line 3 at elevation 955 on the face of the dam, 610 feet inside the right training wall; and that for

line 4 at the back of the drydock, the position of line 5 was varied to establish the optimum location for holding the caisson into the current from the right powerhouse. First, it was placed on top of the powerhouse 40 feet outside the right training wall; secondly, 365 feet outside the wall; thirdly, on the right bank at station A36+00, B12+50; and fourthly, on the bank at station A37+25, B13+50.

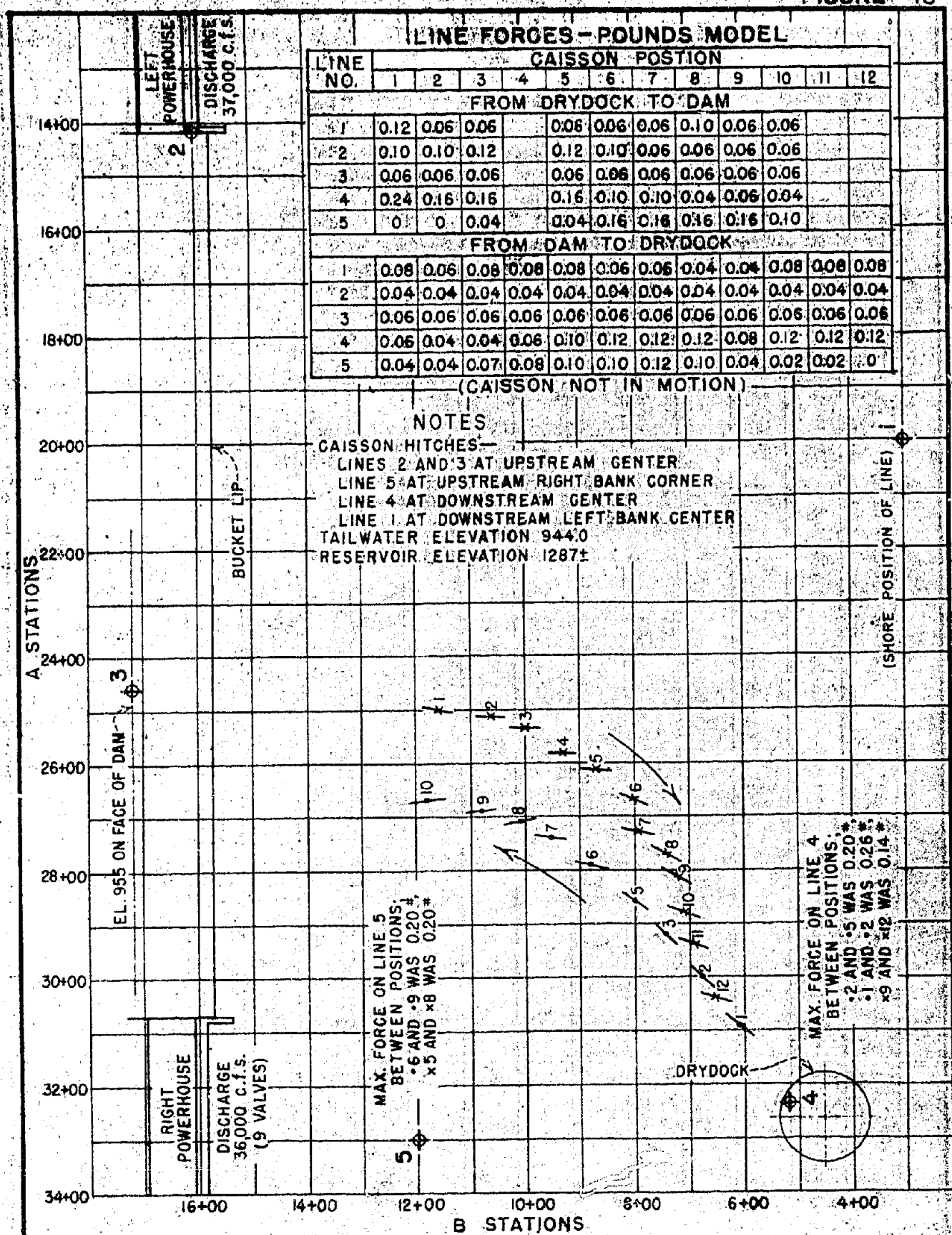
Initially line 5 was too far to the left. Moving it to the second position on top of the powerhouse improved conditions materially, but trouble was experienced with the line contacting the edge of the powerhouse roof as the caisson approached the dam. Maneuvering equipment in this location would have interfered with the future installation of power equipment. Moving the end of the line to the bank below the powerhouse in the third case eliminated this objection and did not seem to alter the maneuverability appreciably. However, it appeared that moving the shore end upstream would facilitate holding the caisson head-on into the current and the fourth position was investigated. As this location seemed to be the optimum, it was retained for subsequent tests in which the shore position of line 1 was varied to obtain further improvement. Tests were made with the end of line 1 at stations A15+00, B10+00; A17+00, B7+00; A18+00, B6+00; A18+00, B5+00; A18+00, B4+00; and A20+00, B3+00. As the end of this line was moved upstream it facilitated the maneuvering of the caisson in its upstream travel, making it easier to keep the caisson head-on into the current from the right powerhouse, but increasing the component acting against line 4 and making maneuvering more difficult as the caisson was being returned to the drydock. Moving the end back downstream decreased the component of force acting against line 4 and improved the maneuverability of the caisson as it crossed the current on its return to the drydock. The maneuverability in the upstream direction was altered very little by this change. Considering these characteristics, the best location appeared to be station A20+00, B3+00. Minor changes in the location of the shore end of line 3 were made to shorten the line and facilitate its handling and protection during the tests in which the optimum position for line 3 was obtained. The maneuverability was not changed noticeably by

slight changes in the shore-end position and the final position chosen, considering the importance of being able to direct the caisson into the current while maneuvering it through the flow from the outlets in the right powerhouse, was station A33+00, B12+00.

Structural and other considerations made slight changes desirable in the shore-end positions for lines 2 and 4, also. Line 4 was placed on the quoin post upstream from the drydock gate opening and line 2 on the deck at the right end of the left powerhouse. From the beginning, three positions for the shore end of line 2 had been considered, namely, behind the left powerhouse near the training wall; on top of the powerhouse on the tower supports for unit 9; and on the draft tube bridge deck near the right end of the powerhouse. From the standpoint of maneuverability the roof position seemed most desirable, however the interference with the installation of unit 9 and the changes required made the deck position more suitable. The danger from ice sliding from the downstream face of the dam during winter operations made the position back of the powerhouse undesirable. No difference in the maneuverability or the line forces was noted when check runs were made with the final shore-end locations (figure 43). The final locations of the shore ends of the lines determined from the model studies, considering the maneuverability of the caisson, structural limitations, and line forces were: line 1, station A20+00, B3+00; line 2, the draft tube bridge deck at left end of left powerhouse; line 3, 610 feet inside of right training wall on face of dam; line 4, upstream side of drydock gate opening; line 5, station A33+00, B12+00 (figure 40).

That the flow from the left powerhouse had little effect upon the maneuvering qualities was shown by data obtained with no discharge from this source. This condition was attributed to the fact that it was always necessary for the caisson to cross the current from the bypass valves in the right powerhouse and the velocity of this current remained nearly constant, thus it was the predominating factor influencing the maneuvering operation.

At various times during the tests for locating the shore ends of the lines and measuring the line forces, the maneuverability of the



LINE FORCES – POUNDS MODEL

LINE NO.	CAISSON POSTION 1	2	3	4	5	6	7	8	9	10	11	12
FROM DRYDOCK TO DAM												
1	0.12	0.06	0.06		0.06	0.06	0.06	0.10	0.06	0.06		
2	0.10	0.10	0.12		0.12	0.10	0.06	0.06	0.06	0.06		
3	0.06	0.06	0.06		0.06	0.06	0.06	0.06	0.06	0.06		
4	0.24	0.16	0.16		0.16	0.10	0.10	0.04	0.06	0.04		
5	0	0	0.04		0.04	0.16	0.16	0.16	0.16	0.10		
FROM DAM TO DRYDOCK												
1	0.08	0.06	0.08	0.08	0.08	0.06	0.06	0.04	0.04	0.08	0.08	0.08
2	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
4	0.06	0.04	0.04	0.06	0.10	0.12	0.12	0.12	0.08	0.12	0.12	0.12
5	0.04	0.04	0.07	0.08	0.10	0.10	0.12	0.10	0.04	0.02	0.02	0

(CAISSON NOT IN MOTION)

NOTES

CAISSON HITCHES—
LINES 2 AND 3 AT UPSTREAM CENTER
LINE 5 AT UPSTREAM RIGHT BANK CORNER
LINE 4 AT DOWNSTREAM CENTER
LINE 1 AT DOWNSTREAM LEFT BANK CENTER
TAILWATER ELEVATION 944.0
RESERVOIR ELEVATION 1287±

GRAND COULEE DAM

1:60 SCALE MODEL

LINE FORCES TO MANEUVER CAISSON FROM DRYDOCK TO DAM AND RETURN

FINAL ARRANGEMENT

caisson over the entire course to and from the dam was investigated. As the network of maneuvering lines approached the final arrangement, it was observed that certain handling of the lines resulted in a ferrying action which carried the caisson across the current either in an upstream or downstream direction, depending upon the manipulation of the lines. On the upstream journey, when the caisson was headed into the current, this action was obtained by slackening lines 1 and 4 and taking-up on lines 2 and 3. On the downstream trip, the action was obtained by reversing the procedure. The length of line 5 was kept approximately constant in either case. However, the caisson had to be headed into the current properly in both cases or the ferrying action would not result. The line forces were very moderate under these conditions.

It was characteristic of line 4 to carry considerable load when the caisson was being moved from the drydock through the entrance channel to the river. This condition was attributed to the fact that this line resisted force components from each of the other four lines, thus it was desirable to locate the shore ends of lines 1 and 5 as far downstream as possible, taking into consideration the maneuverability of the caisson.

21. Measurement of forces exerted on maneuvering lines to establish cable sizes. Line forces with the final maneuvering arrangement were obtained for various combinations of spillway, powerhouse, and bypass flows. The measurements were made using the small drum dynamometers mentioned in section 18 and the combinations limited to those most likely to exist during the maneuvering operations. The line forces were not excessive for any of the combinations tested, and the sizes of the prototype maneuvering lines were based on the forces for maximum flow through both powerhouses. The sizes chosen, with their respective factors of safety, are listed in table I.

Table I

Characteristics of Maneuvering Lines

Line No.	Maximum line pull various flow combinations: Pounds - model	Tons - prototype*	Line size	Factor of safety
1	0.20	21.6	1-3/4"	5.5
2	0.15	16.2	1-3/4"	7.3
3	0.20	21.6	1-3/4"	5.5
4	0.26	28.1	2"	5.5
5	0.20	21.6	1-3/4"	5.5

* Tons prototype = lbs model x $\frac{50^3}{2000}$.

Since all the maximum forces shown above are not for the final line arrangement, in which the forces were reduced somewhat from those listed above, the factors of safety will be greater than shown in the table for some of the lines. Factors of safety for lines 3, 4, and 5 were based on the final arrangement, the others were not.

A test was made also to determine the magnitude of the forces required in moving the caisson parallel with the face of the dam after it was in position over the bucket. Two dynamometers were placed just inside each training wall on wooden platforms clamped to the model walls. The lines from these were fastened to the four corners of the caisson and force readings obtained for various locations of the caisson over the bucket. A maximum force of 0.04 pound model or 8,640 pounds prototype was measured on one of the lines. Relatively more roughness on the prototype might increase this some.

22. Determination of the optimum positions for hitching the maneuvering lines to the caisson. A study was made of the effect of the position of the cable hitches to the caisson upon the maneuverability. In most cases the handling of the model improved when the hitches were placed on the centerline at the upstream and downstream ends of the structure. However, other considerations in some instances made it desirable to attach to the corners instead. Line 5 was changed to the

upstream right corner to prevent its pulling against the upstream right access shaft when the caisson neared its position over the bucket.

Similar trouble was experienced with line 2 for the same position of the caisson. However, the action was not as severe. Since moving the hitch to the corner, as in the case of line 5, would make maneuvering near the fenders in the drydock entrance more difficult, it was suggested that this position be retained andsome method of handling the line be provided to prevent its catching on the rubber seal of the side girder or coming in contact with the access shaft, when the caisson approached the dam.

Line 4 was transferred from the downstream center of the caisson to the downstream right corner to facilitate directional control while maneuvering through the approach channel. This change was necessitated because of the new location placing the shore end of this line on the upstream side of the drydock gate opening.

In one test the caisson was returned to the drydock with lines 1 and 2 attached to the downstream center of the caisson and lines 3, 4, and 5 to the upstream center. The caisson was maneuvered easily into the entrance channel with very small forces, but it was not possible to direct it into the drydock using this arrangement.

In general, the maneuvering operation transferring the caisson from the drydock to the dam was accomplished most satisfactorily when the caisson was moved in a path leading from the drydock through or downstream from station A28+00, B8+00, headed directly into the current from the right powerhouse, ferried across it, then turned and moved upstream to the face of the dam directly downstream of the sheave for line 3 attached to the face of the dam. Much the same path was used in returning the caisson to the drydock. However, it seemed more advantageous to allow it to float backwards downstream directly below the line 3 sheaves to station A10+00, head it into the current from the right powerhouse, ferry it to station A28+00, B8+00 (approximately), then turn it with the back end toward the drydock and move it through the entrance channel into the floating fenders.

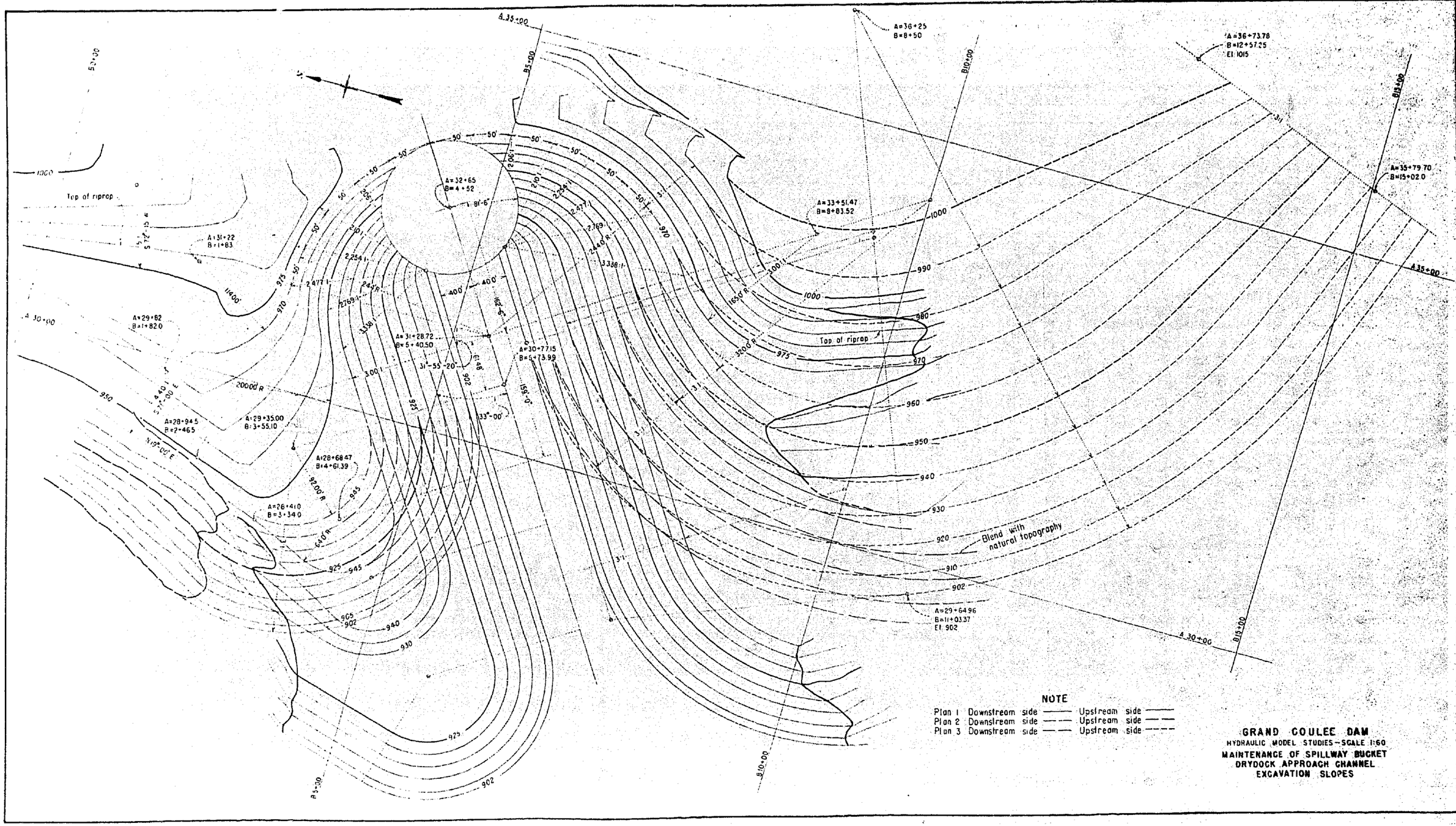
23. Effect of drydock entrance channel on the maneuvering of the floating caisson. When the topography in the vicinity of the drydock was reestablished on the 1 to 60 model for tests concerning the limits for dredging below the bucket to keep materials from entering it and to study various methods of protecting the tailrace slopes and downstream river banks, using contours giving a minimum of excavation in the vicinity of the drydock, (plan 1, figure 44) the ground elevation at the entrance of the drydock approach channel differed appreciably from that used for the maneuvering tests discussed in the foregoing paragraphs obtained from a preliminary drawing. In general, the banks below elevation 940 extended 200 feet farther into the river, increasing the length of the drydock approach and choking the river channel below this elevation.

The long, narrow entrance channel, in which it was necessary to accomplish the maneuvering of the caisson, and the choking of the river channel made it desirable to re-examine the maneuvering of the caisson and study the conditions in the river at the channel entrance for various flows.

The model was operated at a flow representing 69,000 second-feet (37,000 through the left powerhouse and 32,000 through the eight bypass valves of the right powerhouse) for the maneuvering tests and then increased by increments to 787,000 second-feet (37,000 through the left Powerhouse and 750,000 over the spillway) for the river-channel studies.

Five small drums rove with a lightweight fish line, located above the center of the spillway, were used in the maneuvering tests. The lines passed fom the drums through eyebolts in a small wooden beam above them, then through eyebolts set in concrete blocks placed at points representing the shore ends of the lines from the caisson. The maneuvering was accomplished by winding or releasing line from appropriate drums. A river flow representing 69,000 second-feet (37,000 through the left powerhouse and 32,000 through the eight bypass valves in the right powerhouse) was maintained for the tests. The long channel produced no adverse currents in the maneuvering area; in fact, the flow

FIGURE 44



NOTE

Plan 1 Downstream side ——— Upstream side ———
Plan 2 Downstream side — — — Upstream side — — —
Plan 3 Downstream side — — — Upstream side — — —

GRAND COULEE DAM

HYDRAULIC MODEL STUDIES—SCALE 1:60

MAINTENANCE OF SPILLWAY BUCKET DRYDOCK APPROACH CHANNEL EXCAVATION SLOPES

seemed smoother than for the previous tests with the shorter channel. However, the maneuvering was confined to such a narrow path that it was difficult to move the caisson between the drydock and river channel without grounding it on the side slopes of the approach channel. It seems imperative that every precaution should be taken to prevent grounding of the prototype caisson since the seal on the side girders might become damaged. Suitable markers should be provided in the prototype to guard against such occurrences.

In an effort to make maneuvering in the long approach channel less difficult, the caisson end of line 2, from the left powerhouse, was changed from the upstream left corner to the center of the caisson, and the shore end of line 4 was placed at the back center of the drydock. Both changes resulted in improvement, but the change in line 2 seemed more important as it was easier to keep the axis of the caisson parallel to the channel. Locating line 4 at the back center of the drydock caused it to come in contact with the upstream side of the opening in the drydock as the caisson neared the dam.

Marked improvement in handling the caisson resulted when the tests were repeated, after the points extending into the river from both sides of the approach channel were removed to make the topography similar to that used for the initial tests. As before, the same changes in lines 2 and 4 gave noticeable improvement. In this test the shore end of line 4 was changed to the quoin post on the opposite side of the drydock gate opening, also. While this change gave improvement for most positions of the caisson, difficulty was experienced in alining it with the channel near the drydock to place it in position to enter the fenders. It seemed that the shore end of line 4 could not be relocated advantageously without introducing complications.

During the test described above, it was found necessary at times for line 1, 3, 4, and 5 to drag on the points extending into the river from both sides of the approach channel. Line 1 passed over the point on the downstream side when the caisson was moved in or out of the channel near the drydock. Lines 3 and 5 passed over the point upstream when the caisson was moving in or out of the channel near the drydock

and line 4 over the same point as the caisson neared the dam. Protection on both points to prevent damaging these lines would be necessary if the ground surface were not lowered considerably or removed. Protection on the right bank for line 5 had been considered necessary from the beginning, but the extent of this protection would be reduced considerably by removal of the points. It was believed that the protection for line 5 might be reduced further by placing an open sheave on the shore near station A31+30; B9+25 or providing some other means to eliminate the necessity of line 5 sliding up and down the ridge between the main sheave and the approach channel when the caisson is near the drydock. A piece of 1- by 2- by 6-inch board with a nail driven in it at an angle, placed flush with the ground surface on the model at this station, indicated this to be correct. The control of the caisson at the drydock for moving it in and out through the gates and fenders, seemed to be improved by this arrangement. However, other factors, such as the use of line 3 for maneuvering the caisson in this region, might make it desirable to handle line 5 in some other manner.

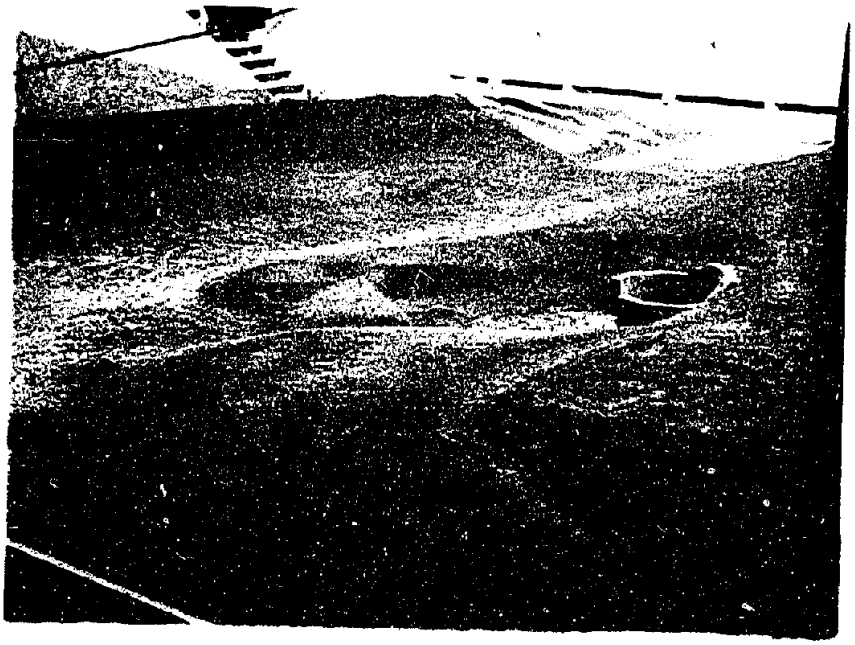
The flow conditions in the tailraces, drydock approach channel, and in the river channel, except over the point extending into the river from the downstream side of the approach channel, with the greatest portion of the points on both sides of the approach channel removed, when only the powerhouses were operating, were very good. A high-velocity, turbulent stream passed over the point on the downstream side of the entrance channel to form a large return eddy against the bank. This condition became more severe as the flow was increased to represent 300,000 second-feet, then became less severe at larger flows as the depth of water over the point increased. In some cases, small particles of sand moved downstream over the point. There were no high-velocity currents directed toward the left bank, but the disturbance acted to decrease the flow area and thus increase the velocity. The velocity change was not measurable on the model.

Operation of the model for short periods at flows, representing 69,000; 87,000; 137,000; 187,000, and 237,000 second-feet, and for one hour at 287,000 second-feet disclosed that pumice and screened coal

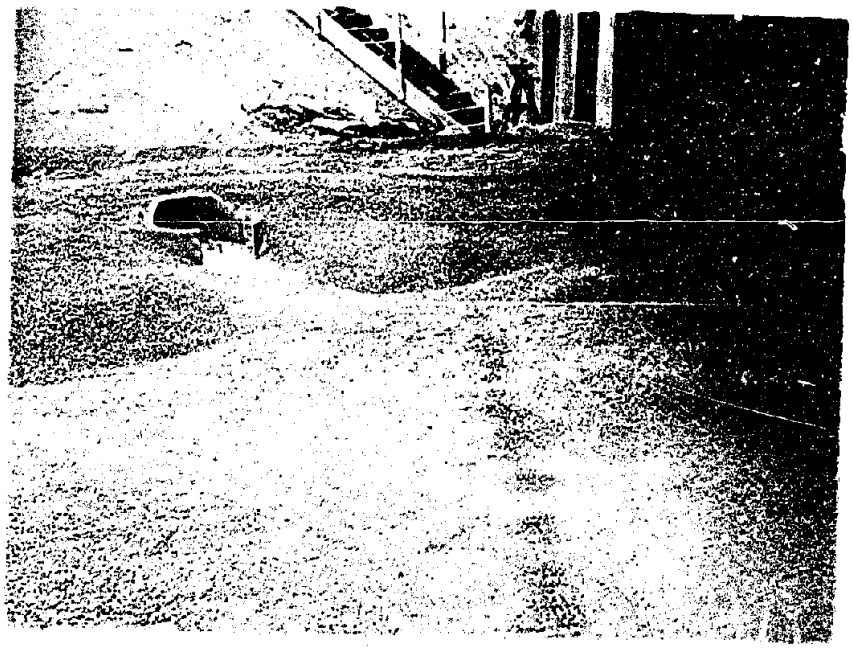
used to study the movement of bed load in two areas downstream from the bucket near the center of the spillway, were being moved downstream toward the right bank. After operating for three hours at the equivalent of 537,000 second-feet, a large quantity of pumice and coal had been deposited in the entrance of the drydock approach channel. This material moved into the channel to form a bar across it 6 inches deep (30 feet prototype) at 787,000 second-feet (figure 45A). Apparently the relatively quiet water in the bottom portion of the channel formed a natural settling basin for this material. To correct this condition and reduce the turbulent flow across the downstream side of the approach channel, the points extending into the river on both sides were removed and the test repeated (plan 2, figure 44).

The change resulted in smoother conditions downstream from the entrance of the drydock approach, but a ridge of pumice and screened coal still collected in the outer portion of the channel. The bar contained about the same amount of material but was much different in shape with less depth (figure 45B). The improvement gained did not seem to justify the added expense of additional excavation and the tests were discontinued. However, it was considered important to shorten the channel to at least the limits used in the tests just described. This would reduce considerably the protection needed for the maneuvering lines; it would improve measurably the maneuvering in the drydock approach channel, and would lessen the depth of the deposit in this channel.

24. <u>Operation of drum gates and river outlets with caisson in bucket.</u> During the initial stages of the development of possibilities for the repair and maintenance of the spillway bucket, it was believed that flow in excess of the left powerhouse capacity would have to be discharged over the spillway drum gates or through the river outlets when the caisson was in position on the bucket and repair work was being performed. Such operation would result in unbalanced forces tending to slide the caisson toward the flow from the spillway or river outlets if they were operated too near the caisson, thus investigations were made on the model to determine the distance the caisson should be



A. Deposit of pumice in long approach channel to drydock



B. Deposit of pumice in shortened approach channel to drydock

EFFECT OF TOPOGRAPHY ON DEPOSIT OF MATERIAL IN ENTRANCE CHANNEL TO DRYDOCK

from this flow to keep the forces within satisfactory limits. A model caisson of sheet metal with piezometer openings installed at various points on all its outside surfaces was constructed for this purpose. Since the caisson was not to be used for tests other than those in which it was in position on the bucket, only the outside surfaces were kept geometrically similar to the prototype design. Other than supplying sufficient weight to hold the caisson in position on the bucket no consideration was given to this factor. The copper piezometer tubes, 1/16-inch inside diameter, passed through the walls and up the access shafts to rubber-hose connections. The tubes were soldered in the caisson walls and dressed flush with the outside surfaces.

Initially a manometer board with approximately 45 glass tubes, 6 mm. in diameter, attached to an airtight manifold from which air could be exhausted to raise the water surfaces to readable levels, was used to record the pressure differences. Besides being cumbersome, the irregularities in the glass tubes gave inconsistent results for such small differentials. To minimize any inconsistency due to the irregularities in the tubes, a two-leg differential manometer operating on the same principle, but having a probe (flattened piece of tubing with end closed and pressure opening in the side) attached to one leg, was devised. The results were more consistent but the differential was still small and very difficult to read accurately. Though pressure data were obtained in this manner for most of the combinations, it was believed that the results would be more reliable if based on velocities measured in the bucket near the caisson, thus data for determining these velocities were recorded for most of the combinations. Measurements were made for different positions of the caisson for each flow combination at first but after a few tests, only one position near the sheet of water passing through the bucket was considered. The results are presented in tabular form (table II). Wave heights and velocities over the bucket outside the flow from the spillway or river outlets were measured. A point gage mounted on a beam spanning the spillway on the training walls back of the powerhouses was employed to obtain the wave heights while the velocities were measured

TABLE II

FLOW CHARACTERISTICS FOR EMERGENCY OPERATION OF SPILLWAY ON RIVER OUTLETS WITH CAISSON IN BUCKET

Caisson position Block	Flow distribution c.f.s.- prototype: Drum gated	River outlets	HPM	LPE	Total	Wave height over bucket: Blk.	Ft. per proto.	Velocities in bucket prototype: Blk.	Ft. per sec.	Blk.	Ft. per sec.	Blk.	Ft. per sec.	Blk.	Ft. per sec.	Pressure differences on sides of model caisson (low pressure on flow side)
47	None	U. 43,45 L & R	20,000	37,000	73,000	35 58	1.4 2.2	33 36	1.2 Re 2.5 Re	39 48	2.2 Re 3.6 L	50 53	4.3 L 3.3 L	56 59	2.5 Le 2.2 Le	0.25 ft. of water.
54	None	U. 43,45, 47,49,51 L & R	20,000	37,000	97,000	35 55	2.0 1.9	38	2.9 Le	54	2.9 to 4.7 R	58	1.4 Le			0.30 ft. of water.
56	None	"	20,000	37,000	97,000	-	-	34	1.8 Re	36	2.5 Re	56	2.2 to 2.9 R			-
36	None	U. 61,63 L & R	20,000	37,000	73,000	35 55	1.1 2.0	36 40	0.5 Re 0.6 Re	44 47	0.7 Re 1.1 Re	50 56	3.3 Re 2.5 R			0.12 ft. of water.
51	None	U. 55,57, 59,61,63 L & R	20,000	37,000	97,000	35 47	1.3 2.6	33 36	1.2 Re 1.0 Re	39 42	1.1 Re 0	45 48	0.9 Le 2.7 Le	52	4.3 to 5.2 L	0.30 ft. of water.
48	None	"	20,000	37,000	97,000	-	-	49	3.5 L	to	5.2 L					
56	None	U.59 L 61,63 L & R	20,000	18,000	80,000	34 46 56	2.5 2.5 2.9	38 39	0.5 Re 0	44 48	1.5 Le 0.8 Le	50 55	3.0 Le 2.9 Le	56	3.4 L to 7.0 L	-
50	None	U. 47 R 43, 45 L & R	20,000	18,000	80,000	34 40 50 60	2.8 2.8 3.4 2.4	36 39 40	3.0 Le 3.3 Le 4.5 L	50	2.6 R to 4.1 R	55 54	1.0 Re 0	56 60	0.9 Le 0.8 Le	0.25 ft. of water. 0.25 ft. of water.
58	None	None	20,000	37,000	57,000	50	0.3	36 42	0.6 Re 1.0 Re	47 50	0 0.4 Le	58 62	1.0 Le 0			Not measurable.
53	50, 11 10,000	None	20,000	37,000	67,000	47 59	0.7 0.3	34 38	0.5 Re 0.6 Re	42 46	0.5 Re 0.5 Re	50 54	0 0.8 Le			Not measurable.
56	10 & 11 10,000 ea.	None	20,000	37,000	77,000	47 56	1.1 0.7	38 41	0.5 R 1.1 R	45 48	0.3 R 0.2 R	52	0			Not measurable.
53	9, 10,11 10,000 ea.	None	20,000	37,000	87,000	47	0.6	36 40	0.5 Re 0.9 Re	43 46	1.0 Re 1.1 Re	50	0			-
50	9,10,11 10,000 ea.	None	20,000	37,000	97,000	50	0.6	35 36	0 0.9 Re	38 42	1.6 Re 1.6 Re	45 47	1.6 Re 1.2 Re	50	1.1 Re	Not measurable
47	7-7,000 8-9,000 9-11,000 10-13,000 11-15,000	None	20,000	37,000	112,000	38 47	1.5 1.3	34	1.2 Re	37	1.5 Re	41	1.4 Re	42 47	0 1.6 Le	-
46	None	None	36,000	37,000	73,000	36 55	0.4 0.3	36 40	0.7 Re 1.2 Re	43 46	1.4 Re 1.0 Re	50 56	0 0.3 Le	58 60	0.5 Le 0.5 Le	Not measurable.
60	11-10,000	None	36,000	37,000	83,000	35 55	1.1 1.7	36 40	1.0 Re 0.9 Re	44 47	1.0 Re 0.9 Re	51 56	0 0.8 Le	58 59	0.6 Le 2.5 L to 2.8 L	Not measurable.
57	12-10,000 11-10,000	None	36,000	37,000	98,000	35 58	0.7 1.4	34 37	0.5 Re 0.6 Re	41 45	0.9 Re 1.3 Re	49 53	0.6 Re 0			
52	9,10,11 10,000 ea.	None	36,000	37,000	103,000	35 47	0.5 0.4	37 40	1.0 Re 1.4 Re	45 48	1.5 Re 1.4 Re	53	2.6 R			Not measurable.
50	8-5,750 9-7,750 10-9,750 11-11,750	None	36,000	37,000	103,000	35 47	0.6 0.9	35 38	0.6 Re 0.6 Re	42 46	0.9 Re 0.6 Re	50	0			Not measurable.
47	7-7,000 8-9,000 9-11,000 10-13,000 11-15,000	None	36,000	37,000	128,000	36 47	1.2 1.0	34 37	0.7 Re 1.3 Re	40 43	1.0 Re 1.2 Re	46	0.7 Re			Not measurable.
36	1-10,000	None	20,000	37,000	67,000	55	1.3	36	2.3 R to 2.4 R	38 40	0.6 Re 1.3 Re	45 48	1.3Re 0.5Re	51 55	0 0.5 Le	Not measured.
39	1, 2 10,000 ea.	None	20,000	37,000	77,000	47 62	0.8 0.9	42 45	0.5 Re 0.9 Re	47 52	0.4 Re 0	54 57	0.5 Le 0.5 Le	58 61	0.6 Le 0.5 Le	Not measured.
45	1-12,000 2-11,000 3- 9,000 4- 7,000	None	20,000	37,000	87,000	47 62	0.2 0.6	36 53	0.9 Le 0	50 47	1.2 Re 1.6 Re	45 45	2.2 Re 1.6 R to 2.0 R			Not measured.
46	1-16,000 2-12,000 3-11,000 4- 9,000 5- 7,000	None	20,000	37,000	112,000	47 62	1.3 0.9	61 59	0.8 Le 0.8 Le	66 61	0.6 Le 0	49 48	0.8 Re 1.9 Re to 2.8 R	47	1.6 R	Not measured.
48	"	None	36,000	37,000	126,000	47 62	1.5 1.1	58 56	0.9 Le 0.8 Le	54 51	0.7 Le 0	48	1.6 R to 2.4 R			
48	1-12,000 2-11,000 3- 9,000 4- 7,000	None	36,000	37,000	113,000	47 62	1.4 1.1	60 57	0.9 Le 1.4 Le	54 51	0.8 Le 0	47 46	0.8 Re 1.2 Re	45	1.9 R to 2.9 R	0.12 ft. of water.
42	1, 2, 3 10,000 ea.	None	36,000	37,000	103,000	47 62	1.0 1.2	61 58	0.6 Le 0.6 Le	58 52	0.4 Le 0.3 Le	50 42	0 1.7 L			-
39	1, 2 10,000 ea.	None	36,000	37,000	93,000	47 61	1.0 1.3	61 56	0.9 Le 1.6 Le	53 51	0.9 Le 0	46 43	0.8 Re 0.8 Re	39	1.7 R to 2.6 R	0.12 ft. of water. 0.12 ft. of water.
39	1,2,10,11 10,000 ea.	None	36,000	37,000	118,000	47	1.6	55 50	1.6 Le 0	47 45	1.6 Re 1.2 Re	39	(1.5 R to (2.9 R			0.15 ft. of water.
39	1,2,3,9, 10, 11 10,000 ea.	None	36,000	37,000	123,000	47	1.2	50 47	1.4 Re 1.2 Re							0.15 ft. of water.

* V-density obtained by using floating aerofoil

either by a small current meter or by scattering confetti on the water surface and timing it over a given distance of travel. Confetti was used mainly in regions where the velocities were too small to be recorded by the current meter.

During pressure tests using the probe, the differential pressures could be partially or completely relieved by increasing the distance between the flow and the caisson. From observations on the model it appeared that the caisson should be kept at least 50 feet from the spillway or outlet flow. However, the tendency of the flow down the face of the prototype to spread more than on the model might make it desirable to increase this, perhaps to even as much as three 50-foot blocks, or 150 feet.

Velocities in the bucket with flow passing over some of the drum gates were too small to be measured with the small current meter and as a rule the surface velocities were greater than those in the bucket below the lip at elevation 900, thus the stresses on the caisson or the tendency for the caisson to slide sideways based on these data would be on the safe side.

From the tests it was concluded that flow through the river outlets or over the spillway should be permitted only in case of emergency when the caisson is in place and that under these conditions the discharge should be kept as remote as possible from the repair operations.

Water-surface conditions with flow through the outlets or over the drum gates will be rougher than for the same flow confined to the powerhouses. Conditions will be more tranquil when the flow passes over the drum gates than through the river outlets, when emergency operation is necessary, due to the concentration of energy in the outlet flow.

The decision to install eight 84-inch needle valves in the right powerhouse to convey water in excess of the capacity of the left powerhouse when the river discharge is favorable for conducting the repair operations, made it unnecessary to discharge water over the gates or through the outlets, except in case of emergency. The data in table II is presented to guide in establishing flow combinations giving optimum

conditions for continuing the repair work should emergency operation in this manner become necessary.

Care should be exercised in any case to keep the spillway or river-outlet flow as far away from the caisson as possible. Some manipulation of the prototype gates or outlets might be necessary to attain optimum conditions for conducting the repair and maintenance operation should an emergency arise.

25. Investigation on the 1 to 20 model concerning stability and operation of floating caisson. As soon as the 1 to 60 scale plastic model had proven inadequate for investigating the behavior of the caisson under operating conditions, plans were prepared for a larger working model containing the hoists, lines, and other important equipment which would enable a detailed investigation of the stability and operating characteristics. A scale of 1 to 20 was chosen for this model in order to utilize standard-gage sheet metal corresponding to the scaled thickness of the skin plates of the structure and to provide sufficient size to permit the installation of the operating equipment and keep within weight limits which vary as the cube of the scale, or 8,000 to 1 for the 1 to 20 model. Galvanneal sheet was used because of its rust-resisting qualities, since the model was to be in contact with water much of the time.

Since the caisson was to be operated as a freely floating vessel while being transferred to and from the dam and during the sinking and raising operations, it was important that it have properties similar to ships, barges or other floating vessels. Most important among these is the stability of equilibrium or the tendency of the vessel to remain right side up under pitching and rolling displacements. With the length of the caisson being about twice the width, the stability under rolling displacements was by far the more important, thus comprehensive model tests were restricted mainly to characteristics concerning the lateral stability.

A floating body is in equilibrium when its weight is equal to the weight of the liquid it displaces and the centers of gravity of the body and the liquid are in the same vertical line. If the body is

is displaced slightly from the equilibrium position so that the line joining its center of gravity and that of the displaced liquid (center of buoyancy) is no longer vertical, the forces acting may tend to do one of three things; to restore the body to its original position; to move it still further from that position, overturning it; or to maintain it in equilibrium. The first is defined as the stable condition, the second as the unstable condition, and the third the neutral condition. These conditions and the properties influencing them can be demonstrated best by the use of a diagram. Let C (figure 46A) be the center of gravity of the vessel; B the center of buoyancy when in the equilibrium position; and D the center of buoyancy for the displaced position. Through D draw a vertical BM to meet BC at M.

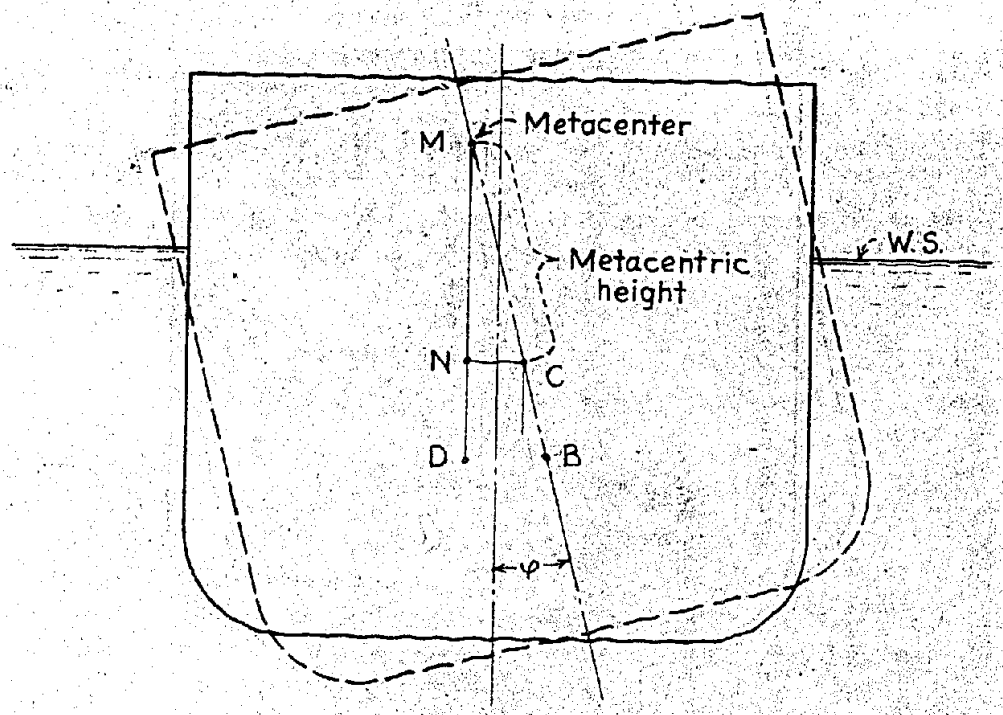
The weight of the body now acts vertically downward through C and and the equal force of buoyancy vertically upward through D, each being equal to the weight of the vessel, W, forming a couple with arm CN, the perpendicular from C to DM.

From figure 46A, it is observed that if M is above C the couple tends to restore the vessel to the equilibrium position; if below, it tends to overturn the vessel; and if coincident with it, the equilibrium is neutral.

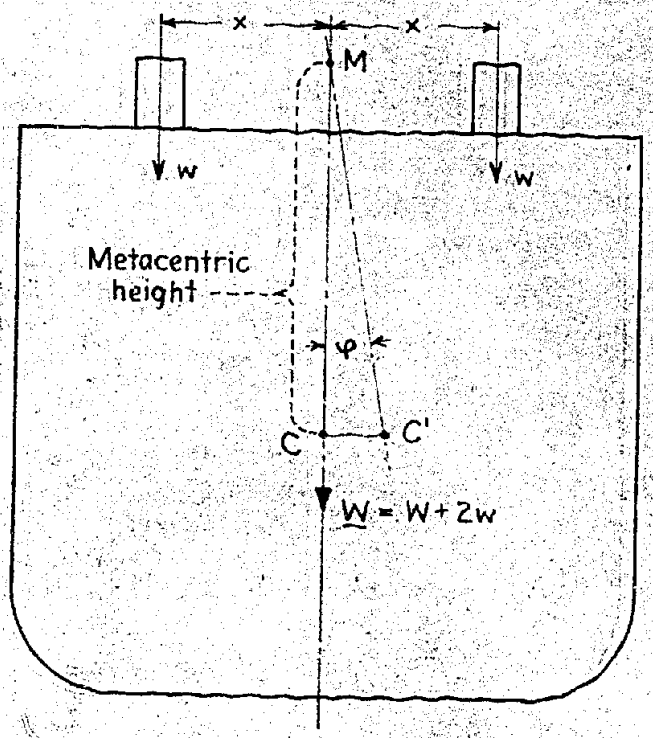
If the angle of displacement = ϕ, the magnitude of the righting moment for the stable condition is W(CN) or W(CM) sin ϕ.

As ϕ is increased, the position of the intersection M of the verticals through BD will in general move and approach or recede from C. The point M for an infinitely small angle of displacement is called the metacenter of the vessel and the distance CM the metacentic height. In ship design it is of the utmost importance that the metacenter be above C under all conditions of loadin, and under any circumstances of rolling. This is true also of the repair caisson if the maneuvering, sinking, raising and operation is to be accomplished satisfactorily.

Though it is possible to tell immediately upon floating a vessel whether the metacenter is above, below, or coincides with the center of gravity it is desirable in many instances to know the exact location. Some idea as to the nearness of the metacenter to the center of gravity



A. METACENTER OF FLOATING VESSEL, ROLLING DISPLACEMENT



B. ARRANGEMENT FOR EXPERIMENTAL DETERMINATION OF METACENTRIC HEIGHT

THE METACENTRIC HEIGHT OF A VESSEL

can be gained from noting the rapidity with which the vessel rights itself after being displaced and the displacing force removed. The greater the distance or metacentric height the more rapid will be the restoring action. It is possible to determine the metacentric height of a vessel by extensive computations or by experiment. The first method was of necessity used in the design of the prototype structure but it was possible to determine that of the model easily and quickly by experiment. Also, it was desirable to use the experimental method in the case of the model since the value obtained in this manner would serve to check the prototype computations, an operation which a computed value for the model could not be expected to accomplish. The method employed in determining the metacentric height of the model is that used extensively for vessels used in water transportation, outlined on pages 31 to 35 in Gibson's "Hydraulics and its Applications." It is somewhat as follows:

Place two equal weights w at equal distances x from the centerline of the vessel when floating on an even keel in its equilibrium position. (figure 46B).

Let $\underline{W}$ be the weight of the vessel, including the weights.

Then if both weights be moved through a distance dx to the right the center of gravity of the vessel will move through a distance CC', where $CC' = \frac{2P}{W} dx$

Also CC' will be parallel to the direction in which P is moved; i.e., will be perpendicular to CM, since if any portion of a body be moved in a given direction, the center of gravity of the whole moves in the same direction.

If ϕ be the angle of keel produced by the shift of the weights

$CC' = CM \tan \phi$

or $CM = CC' \cot \phi = \frac{2P}{W} dx \cot \phi.$

The angle ϕ can be measured by observing the change in inclination of a long pendulum as the weights are moved. The experiment should be repeated for different values of dx, measured both to the right and left and a curve of ϕ vs. CM drawn. By extrapolation the value of CM when $\phi = 0$ can be determined.

The results of this experiment and that to determine the location of the center of gravity of the unusual shape of the caisson are given in subsequent paragraphs of this section of this report.

Air trapped at the high points of the working space at the ends of the caisson as it was being lowered below the water surface, during preliminary tests, would not exhaust satisfactorily through the vents provided for that purpose when the vent system was constructed to scale and arranged according to prototype plans. The 3/16-inch copper tubes were considered too small insofar as the model was concerned. The head loss in these long tubes was appreciable, and the head available to produce the flow of air was small; thus the resistance of a drop of water should it enter the model line, would prevent air from leaving the working space through the vent until sufficient pressure was developed by submergence to force the water from the tubes. The detrimental influence of the air on the stability of the caisson would be effective before this condition was reached. Obviously, a change in the model vent system was required.

The caisson developed a distinct list during the tests, making it difficult to conduct the studies. Investigation disclosed a sizable leak in the air compartment of one of the side girders and small leaks into some of the floating and trim tanks.

Difficulty was experienced with the hoists and the lines during the lowering and raising of the caisson. The aerial wire used initially for the hoist lines stretched and tended to remain in whatever position it was bent, loosening the lines and hindering the operation. The fact that the wire was too large to be rove on the drums in one layer added to the difficulties. The lines would tighten as the barge neared the upper and lower limits of its travel and would become extremely loose at the center.

Although a satisfactory caisson design had been indicated by these tests, it was considered essential to make certain alterations to the model and to obtain more detailed information concerning its operating characteristics before it was given final approval.

The air and water compartments between the deck and the working space and in the side girders were checked for leaks and sealed, a new

vent system was installed at the top of the working space in each end of the caisson, and the aerial wire cable was replaced with 24-gage spring-brass wire.

Twenty-six 1/4-inch diameter holes were drilled through the skin plate at the downstream end of the caisson, and through that at the high point in the working space on the upstream end. A semicircular manifold made by splitting a length of 7/8-inch brass tubing and soldering the sections over the rows of 1/4-inch holes in the skin plates served as ducts to collect the air and convey it to risers of 5/8-inch brass tubing at the closed ends of the manifolds. Rubber stoppers were used to control the vent system during the test program, but plans were to connect them to the access shafts after the adequacy of the system had been established.

The spring-brass wire proved very satisfactory for the operating lines, no readjustment being necessary during the remainder of the test program.

Since it was important that the position of the center of gravity of the model caisson be in the location corresponding to that of the prototype, its position was determined experimentally by weighing. The position for that of the barge and for the caisson was determined separately as well as collectively. Each of these structures was suspended on knife edges of known distance apart on two weighing scales. The reactions were determined from the scale readings for different inclinations of the structures. With these data the center of gravity was determined as follows:

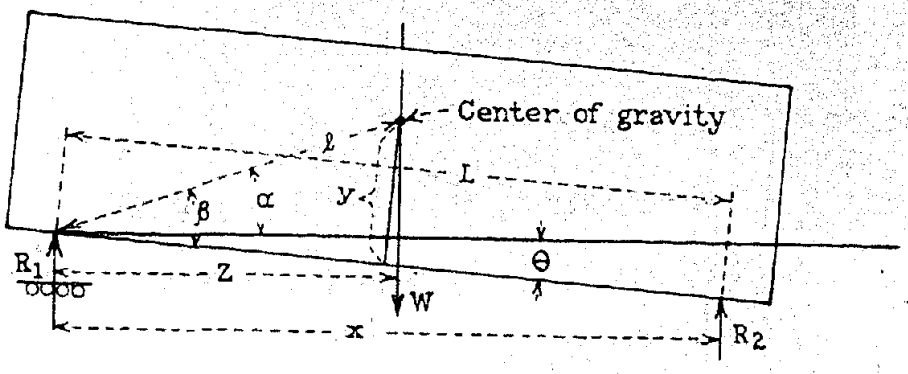


Figure 47. Relation of variables for experimental location of center of gravity of model operating barge.

From figure 47

$$\Sigma M_{R_1} = Wz - R_2 x = 0$$

$$Wz = R_2 x$$

W = total weight, or $R_1 + R_2$ (the reactions at the knife edges)

$$z = \ell \cos(\beta - \theta)$$

$$x = L \cos \theta$$

then

$$W \ell \cos(\beta - \theta) = R_2 L \cos \theta \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

or

$$R_2 = \frac{W \ell}{L} \frac{\cos(\beta - \theta)}{\cos \theta} = K(\cos \beta + \sin \beta \tan \theta)$$

since

$$\cos(\beta - \theta) = \cos \theta \cos \beta + \sin \beta \sin \theta$$

Cos β always positive (first and fourth quadrant). Sin β positive or negative (positive in first and negative in fourth quadrant).

And letting

$$\frac{W \ell}{L} = K,$$

As

β is a constant,

let

$$K \cos \beta = A$$
$$K \sin \beta = B$$

Therefore

$$R_2 = A + B \tan \theta$$

which is the equation of a straight line in which R_2 and $\tan \theta$ are the variables, A the intercept and B the slope, which may be positive or negative as determined from a plot of the recorded values of R_2 and $\tan \theta$.

The sign of B signifies whether or not the center of gravity is above or below a line between the supports. If negative, the center of gravity is above, if positive, it is below. This can be shown by assuming the position above or below and setting up the equation, paying

particular attention to the signs of the angles α and θ. They should be positive when measured above the horizontal line and negative when below. A and B are obtained from the plot of the data taken for the different inclinations of the structure, figure 48.

Using these values for A and B and dividing the expression for B by that for A or

$$\frac{B}{A} = \tan \beta$$

Knowing β it is possible to compute δ from the same expressions.

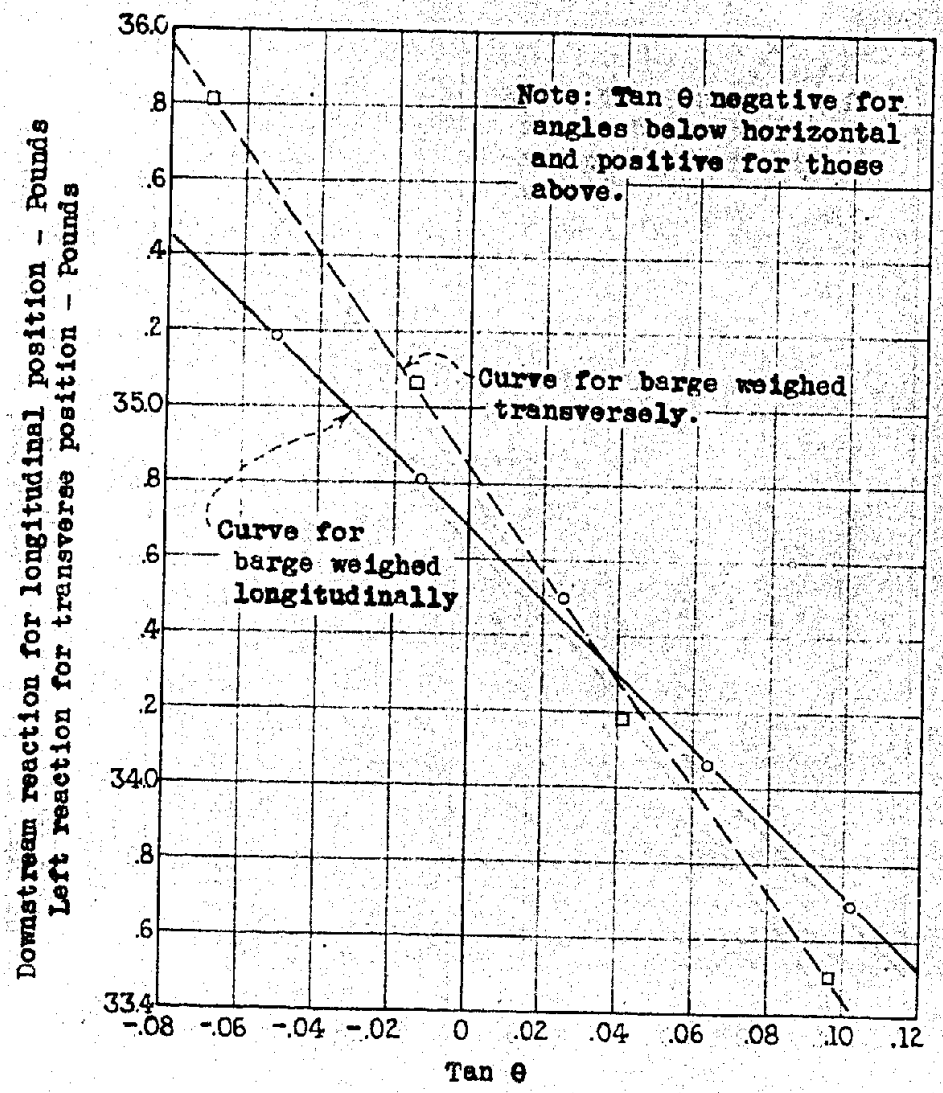


Figure 48. Curve tan θ vs. reactions for model operating barge.

With these data, y and z can be computed for $\theta = 0$, which establishes the center of gravity in one plane. If the structure is unsymmetrical in the third dimension, its position in this plane can be established by weighing also. When a structure can be weighed conveniently in three planes the procedure explained above need not be followed but one weighing in each plane will be required.

The position of the center of gravity of the caisson and the barge obtained by weighing in one plane only, assuming it to be on the longitudinal centerline, checked closely with that computed by the design section. With the similarity in this respect established between model and prototype, the testing was resumed. The freeboard was measured for different degrees of trim, the stability of the apparatus was investigated for the sinking and raising operation, and the action noted for different procedures of filling the barge well above the caisson deck. The tendency for the caisson to right itself when tipped was taken as a measure of its stability.

With the center of gravity on the longitudinal center line 1.66 feet downstream from the vertical center line of the bucket curve and 5.6 feet below the top of the side girders and all trim and side-girder compartments full of air, the freeboard opposite the upstream and downstream ends of the barge well was 10.73 feet and 7.42 feet, respectively.

With both forward trim compartments filled with water, the freeboard was 6.20 feet and 8.95 feet, respectively.

With both front trim compartments filled with water the stability was good for all positions during the descent and the ascent. Although the trim of the caisson resulted in a steep inclination of the structure which transferred a large portion of the load to the upstream cables between the barge and caisson at the end. The caisson settled into the bucket very satisfactorily and the ascent was accomplished just as easily.

Because of the unequal loads on the caisson hoists, the caisson was trimmed level and the test repeated. The load distribution on the lines supporting the weight of the caisson from the barge was very good, and the trim remained nearly constant during the sinking and raising

operation. However, difficulty was experienced in placing the caisson on the bucket. The rubber seal rubbed along the slope upstream from thebucket for considerable distance before the caisson became seated on the bottom. This action was undesirable, since it might result in damage to the seal or the caisson might come in contact with the lip before reaching the bottom, suspending its weight between this point and the V-frame anchor at the face of the dam, inducing stresses in the structure and possibly tearing the V-frame from its anchorage.

To study this action further, the trim of the caisson was changed, by adding water to the compartments, placing the upstream end below the downstream end only slightly. The seating on the bucket was improved measurably and, although the weight distribution on the hoists was unequal, it was much better than with the two front compartments filled completely.

From these observations it seemed that the most satisfactory operation would result if the caisson remained near the level position except when it neared the bucket, where the front trim compartments should be filled to prevent the seal, or bottom of the caisson, contacting the bucket lip as it settled into place on the bucket. In view of the results discussed above, it seemed desirable to divide the front trim compartment into several smaller ones preventing the shift of the water load inducing list. However, this factor did not warrant increasing the number beyond two in the model and no change was made. After viewing the operation of the caisson under the various degrees of inclination, it was considered practicable to move the structure from one position to another on the bucket without raising it completely and unwatering the barge well for each operation. In other words the caisson could be raised to a point near the water surface, where the operating lines between it and the barge would not be overstressed by forcing the barge into the well of the caisson, and then maneuvered laterally to the new location and again lowered into place. Also, it seemed desirable when raising the caisson completely or refloating it, to force the barge into the well as far as possible without overstressing the cables, before the pumps were started to empty the well. Though not essential, this would

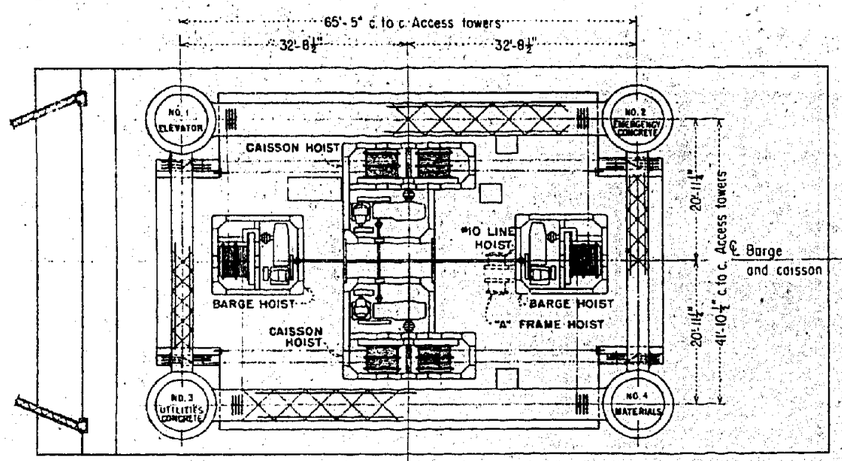
raise the side girders sufficiently to assure against water passing over them into the well, a condition requiring extra pumping during the raising operation.

Since the operation of filling and emptying the barge well during the time the caisson is being submerged or raised above the water surface has to be performed so slowly and in many steps, due to the small space between the barge and the walls of the well, if the barge were to be kept in contact with the water in the well to prevent overstressing the operating lines, due to excess draft on the barge, the question arose as to what would happen if the barge were raised some distance above the deck before the filling of the well for the sinking operation was begun, or if the water surface in the well were lowered below the bottom of the barge between lowerings of the barge during the refloating operation. Tests were made to demonstrate the undesirability of this procedure. The caisson was trimmed level for these tests.

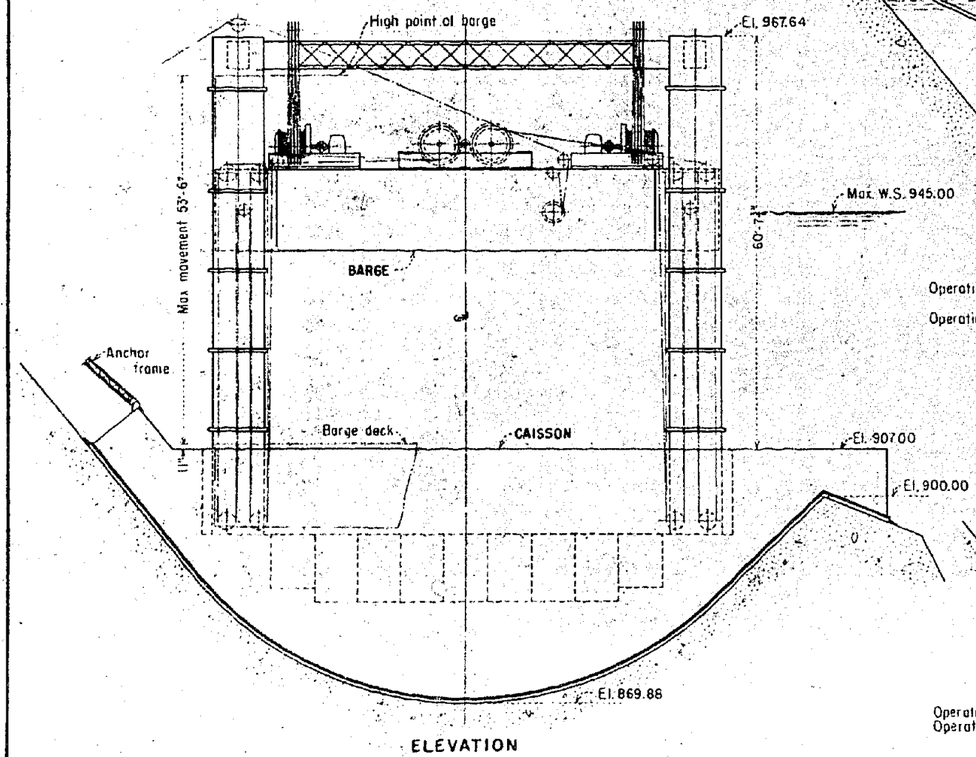
The barge was raised until the bottom was two inches (model) below the tops of the side girders and water was placed slowly into the well. As the water attained depth, a very slight rocking of the caisson caused it to run to the low side and to remain, giving the structure a noticeable list. Additional water served only to worsen this condition and decrease the stability. The water, in moving to the low side, shifted the center of gravity in that direction, producing the list. Lowering the barge into the water until its entire bottom surface was in contact caused the structure to right itself. It was construed that operation in this manner would be undesirable as well as dangerous.

A second test was conducted to determine the minimum submergence of the barge to maintain stable conditions. The well was filled partially with the barge in the lowered position; the barge was then raised gradually until unstable conditions were obtained by tipping it. The structure was stable for all drafts of 3/8-inch and greater, indicating that the draft of the prototype operating barge should never be less than 8 inches. An operating diagram was prepared by the design section for the sinking and raising operation based on these findings (figure 49). No difficulty was experienced with the new system of working-space

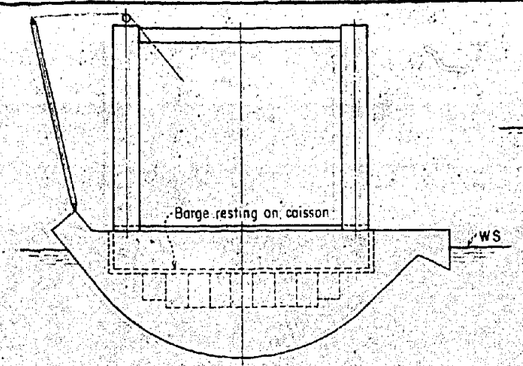
FIGURE 49



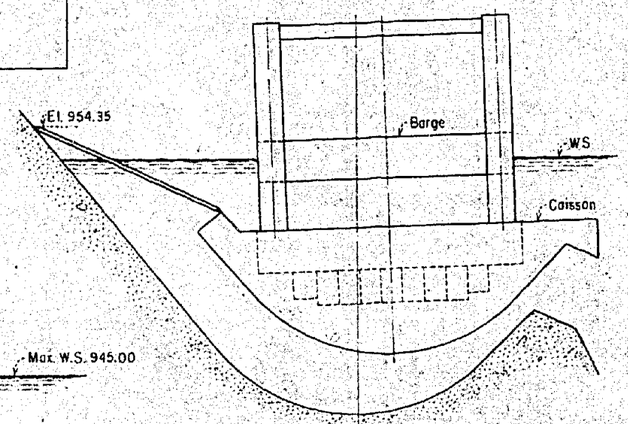
PLAN



ELEVATION



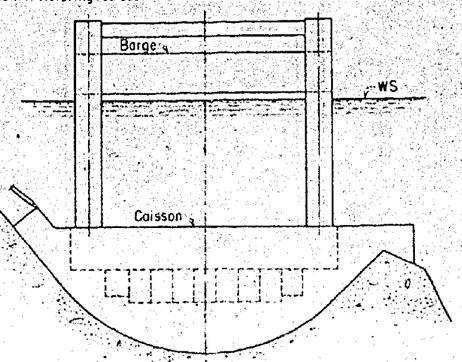
CAISSON IN FLOATING POSITION



CAISSON OVER BUCKET OF DAM

Operation 1. Lowering caisson to bucket of dam. The overhauling load must be lowered under control.

Operation 4. Hoisting caisson from bucket of dam to surface of water.

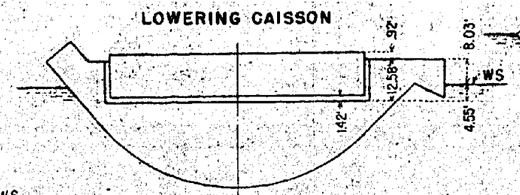


CAISSON LANDED ON BUCKET OF DAM

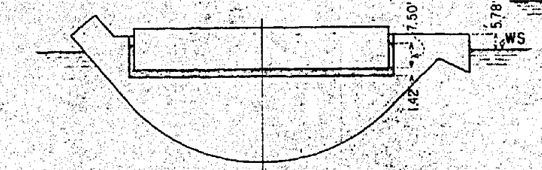
Operation 2 Raising barge clear of water.

Operation 3 Lowering barge to surface of water. The overhauling load must be lowered under control.

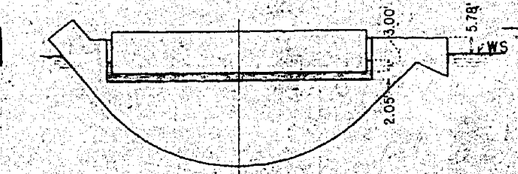
LOWERING CAISSON



CAISSON FLOATING
Barge well and side girders empty

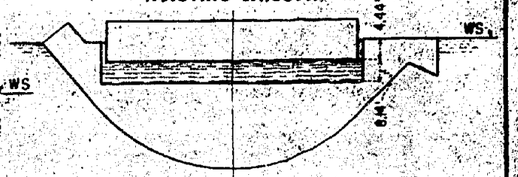


CAISSON FLOATING
After 1st. filling of barge well

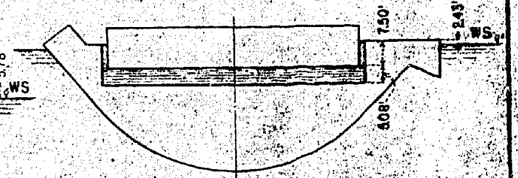


CAISSON FLOATING
After lowering caisson 0.63' with reference to barge

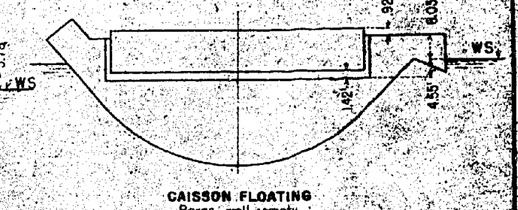
HOISTING CAISSON



CAISSON HOISTED TO WATER SURFACE
Side girders empty



CAISSON HOISTED
Barge drawn into well until head of water on barge equals 7.50'



CAISSON FLOATING
Barge well empty

INSTRUCTIONS FOR LOWERING CAISSON

Set the control for "CAISSON LOWER – BARGE WIND"

When filling the barge well, the maximum head of water on the bottom of the barge at the center must not exceed 7'-6". This depth of water will cause a load of 1,000,000# on the caisson hoist.

When lowering the caisson, the lowering motion must be discontinued when the head of water on the bottom of the barge has been reduced to 3'-0".

After filling the barge well and lowering the caisson about 12 times successively, the top of the caisson will be even with the water surface and the caisson can be lowered to the spillway bucket.

INSTRUCTIONS FOR HOISTING CAISSON

Set the control for "CAISSON HOIST – BARGE PAYOUT."

Hoist caisson until it reaches the surface of the water and continue hoisting until the head of water on the bottom of the barge at the center equals 7'-6".

Then discontinue hoisting and pump water from the barge well until the head of water on the bottom of the barge has been reduced to 3'-0".

Repeat the caisson hoisting and pumping-out operations until the barge is within 6" of its seat in the barge well and while in this position pump the barge well dry.

Set the control for "BARGE LOWER", then seat the barge in the caisson.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT – WASHINGTON

GRAND COULEE DAM

MAINTENANCE OF SPILLWAY BUCKET

FLOATING CAISSON

SINKING AND RAISING OPERATION DIAGRAM

DRAWN F.A.J. SUBMITTED
TRACED C.L.H. V.E.A. RECOMMENDED
CHECKED N.P. APPROVED CHIEF ENGINEER

DENVER, COLORADO, MAY 26, 1944 222-D-9406

vents during the testing described above. The prototype arrangement was made similar to that of the model.

In the foregoing tests under normal operation the sinking and raising operation was accomplished very easily and there were no indications of instability. The caisson righted itself readily when tipped to the side at any time during its descent and ascent, although it appeared more sluggish at some points than at others. The most sluggish point seemed to be just after the caisson became submerged when the barge was supporting the full load. Detailed final tests on this arrangement were planned but were postponed due to the urgency of completing other work. During the interim the plans for the pumping equipment and piping of the prototype had progressed sufficiently to establish accurately the total weight and the final location of the center of gravity of the caisson. Computations showed a substantial increase in weight and an upward shift of the center of gravity, decreasing the distance between it and the metacenter, tending to produce instability. Agreement on the model with the newly computed position was attained by adding four 2-pound bricks of heavy metal to the caisson, one in the top of each access shaft 5/8-inch downstream from the center line. The brass supports for these pieces increased the total weight of the added metal to 8.2 pounds, which was that required by computation to correct the center of gravity when placed 36 inches above the top of the side girders and 3/4-inch downstream from the center line of the bucket curve. Limited space in the model access shafts did not permit placing the bricks 3/4-inch downstream, but the 5/8-inch proved ample when the location of the center of gravity of the model was obtained by weighing.

The front trim compartments were filled sufficiently to level the caisson and the structure studied through the sinking and raising cycle. The stability was good throughout the range, but the caisson was slightly more sluggish due to the lesser distance between the center of gravity and the metacenter caused by the high position of the added weight. Interest as to the amount of weight which could be added to the tops of the access shafts before insufficient stability would result prompted the addition of more 2-pound metal bricks in 8-pound increments.

The first additional eight pounds made the caisson more sluggish, but the stability was good. The second eight pounds made the righting movement very slow, but the stability seemed to remain satisfactory. When the third eight pounds were added, the caisson listed noticeably but did not overturn. However in this case air compartments in the side girders acted as stabilizers when the list became sufficient to lift the top of one side girder out of the water and prevented overturning. The buoyancy of the barge assisted in keeping the caisson upright also, the structure tending to right itself when the high edge of the barge reached the water surface. Surface tension at the bottom of the barge no doubt had some influence on this action. This was evidenced when the structure was tipped enough to raise the bottom edge of the barge out of the water and the caisson tended to overturn unless restrained.

The caisson, with its operating barge, having the position of the center of gravity representing closely that of the final design, was investigated thoroughly. Its behavior with various degrees of trim, throughout the sinking and raising cycle, was studied in particular. The freeboard of the caisson while floating, and that of the barge with the caisson below the water surface, was recorded. The position of the center of gravity in the vertical plane was determined experimentally and data were obtained for determining the metacentric height.

With all compartments free of water, the freeboard of the caisson was 10.4 feet opposite the upstream end of the barge well and 7.0 feet opposite the downstream end. With the caisson submerged, the freeboard of the barge was unequal because the inclination of the structure was such that air was trapped in the upstream end of the dome in the working space at the downstream end of the caisson. Additional vents along the top of the caisson downstream from the back access shafts would prevent this action. Vents were included in this area on the prototype. The stability with the side girders only slightly under water was not so good, for severe tipping caused the caisson to tend to overturn until the buoyancy of the barge acted to prevent it. When tipped moderately, the structure righted itself, but the action was

slow and sluggish. Sinking with the trim compartments free of water is not recommended. Moreover, the curved part of the side girders contacted the bucket lip as the caisson neared the lower limit of its descent, which might damage the seal or tear the anchor from the face of the dam if an attempt were made to lower it onto the bucket.

Filling the second trim compartment on the upstream end made the caisson float nearly level. The stability was good at all times, but the seal passed very close to the bucket surface as the caisson settled into position and seemed to drag on the 1: 1 slope of the bucket lip just before becoming seated. Lowering the caisson into the bucket when it is trimmed level is not recommended, due to the danger of damaging the seal.

The freeboard of the caisson at points opposite the front and back ends of the barge well, when floating, was 8.0 feet and 7.9 feet, respectively, which was in good agreement with the 8.2 feet computed for the prototype. That for the barge with the caisson submerged was 8.1 feet at the front corner just back of the front shafts and 8.4 feet at the back just forward of the rear shafts. The results were almost identical with the front compartment full and the second free of water.

With both of the front trim tanks filled, the stability of the caisson was very good throughout the sinking and raising operation. As the caisson settled into the bucket, the upstream end contacted the face of the dam first and the structure pivoted downward, "rolling" into position on the bucket. Some difficulty might be expected in placing the caisson below the water surface, due to the front end being so low, if the trim tanks were filled prior to this operation.

The freeboard of the caisson when floating was 6.0 feet at the front end of the barge well and 8.7 feet, at the back end, and that for the barge with the caisson submerged was 5.7 feet at the front end and 9.4 at the back end.

Considering the results of the tests for various degrees of trim, it seems that operation of the caisson will be most satisfactory if all the trim compartments are free of water during the maneuvering of the caisson to and from the dam to reduce its resistance, thus lowering the

line pull; that the trim be level when the caisson is in the process of being submerged or while its top is being raised above the water surface during the refloating operation; that the upstream end be substantially lower than the downstream end when the caisson is entering or leaving the bucket; and that the longitudinal axis of the caisson be kept perpendicular to the axis of the dam when it is being seated in or raised from the bucket. If the caisson is level or if it is trimmed to slope downward toward the back there is danger of dragging the seal across the rough bucket lip and damaging it as the structure settles into position on the bucket.

With the final model weight, the longitudinal and vertical location of the center of gravity of the caisson and the barge as a unit was found to be 3.37 feet (prototype) below the top of the side girders and 1.43 feet (prototype) downstream of the vertical radius of the bucket curve at the bottom of these girders. The total weight of the model caisson and barge was 345.5 pounds or 1,382.0 tons (prototype). The position of the center of gravity determined by weighing (listed above) and the weight checked closely with the values of 3.58 feet, 1.35 feet, and 1,347.5 tons computed for the prototype structure.

Experimental determination of the metacentric height, which is a measure of the stability of floating structure, was made by sliding two kilogram-weights set in a piece of plywood on the deck of the barge to each side of the longitudinal center line of the caisson and recording the angle of heel and the position of the weights. The method used in making the determination was that given in Gibson's "Hydraulics and its Applications," pages 31-35, discussed previously. The average metacentric height from six trials, determined in this manner, was approximately 11.8 feet prototype. This value was in good agreement with that of 12.3 feet computed for the prototype by the design section. The model was trimmed level for this test.

Computation of the distance between the center of buoyancy and the metacenter using $d = \frac{I}{V}$ where I is the moment of inertia of the displacement area at the water line about the axis of rotation, and V is the volume displaced by the vessel. The distance of 23.8 feet

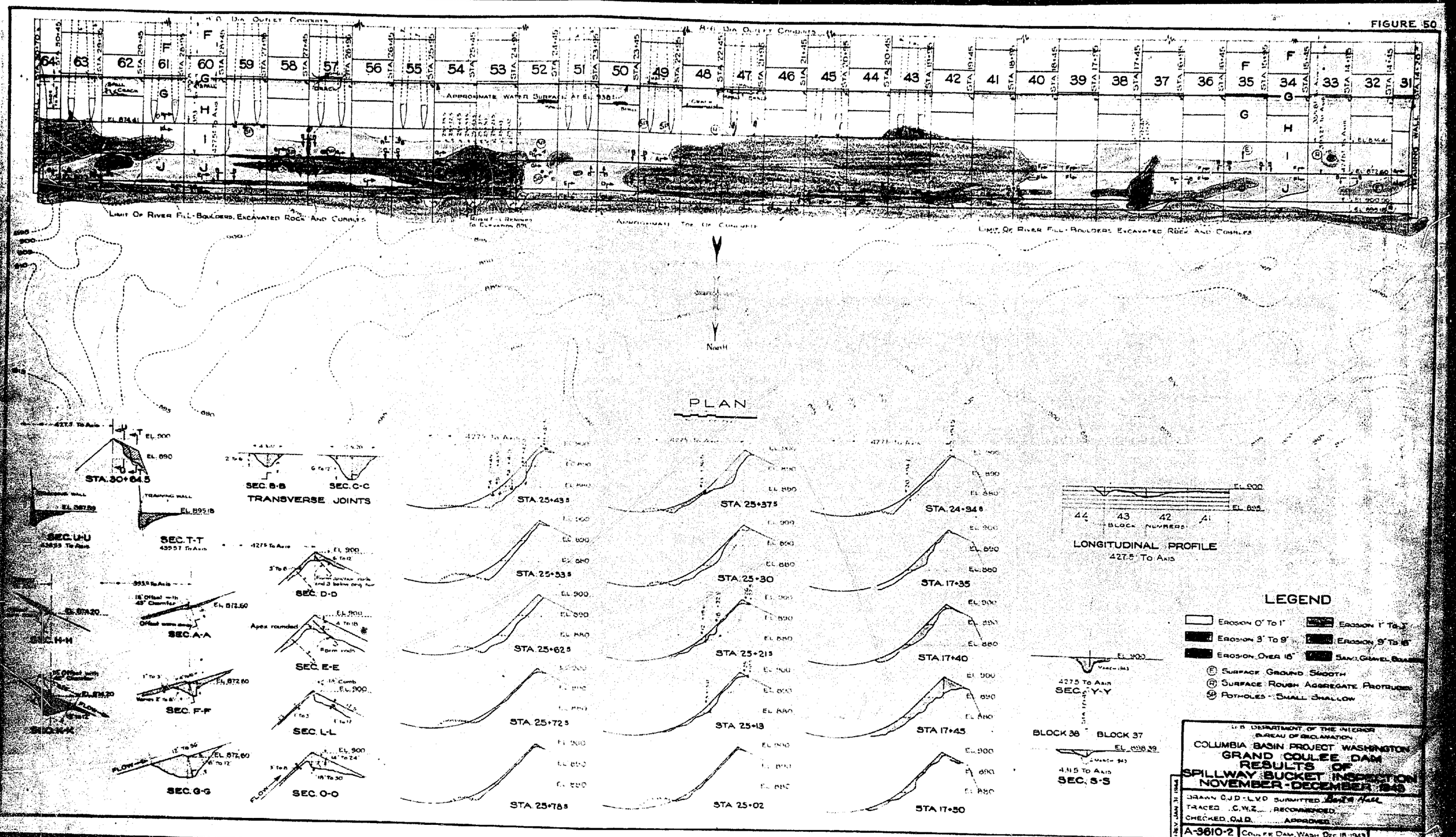
obtained in this manner was a fair check with that of 22.4 feet computed for the full-sized structure.

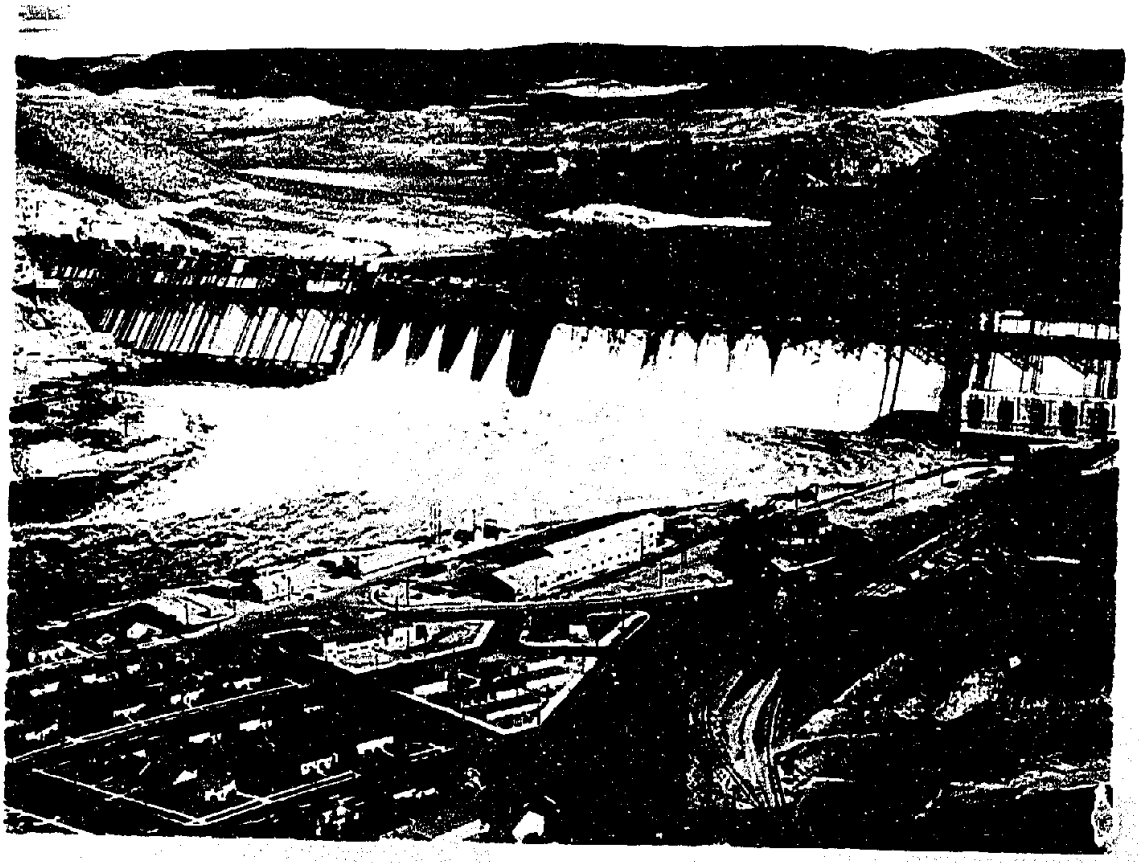
26. Discussion of damage to spillway bucket reported by divers. During tests on the 1 to 60 model concerning the inspection and repair caisson and other related tests discussed in this report, particular attention was focused on flow conditions in and about the spillway bucket to ascertain, if possible, the actions responsible for the damaged areas reported by divers in their inspections in the early spring and late fall of 1943. The observations made during these tests were enlightening, and, insofar as the writer is concerned, indicated clearly the source of most of the damage. An interpretation of the model results and their relation to the damaged areas reported by divers in November and December 1943, and shown on a drawing contained in a report to the Field Engineer by Engineer L. V. Downs dated December 18, 1943, is contained in the following paragraphs.

It was believed originally that the greatest portion of the damage had occurred during the construction period when extremely unbalanced conditions and concentration of flow existed, and that conditions causing it would not be present during normal operation, but after observing the model in operation, this viewpoint was altered somewhat, for it was indicated clearly that although the damage had started during the construction period, many of the conditions causing it were present under normal operation, thus its continuation could be expected unless the sources were removed. Only one of the areas could be attributed solely to conditions existing during the construction period; that in the joint of the No. 37-38 rows of construction blocks (figure 50).

During the summer flood of 1940, the flow of the river passed through the river outlets and flood passages over low sections of the uncompleted structure (figure 51). During most of the period the flow was concentrated to the right of the eroded channel and material was carried into the bucket by the severe counterclockwise eddy formed below the left end of the spillway, a fact indicated by observations by project personnel, described to the writer. It was reported that in

FIGURE 50


PLAN
North
Approximate Water Surface At El. 938±
Limit Of River Fill-Boulders, Excavated Rock And Cobbles
Approximate Top Of Cofferdam
Training Wall
STA. 30+64.5
SEC. B-B
SEC. C-C
TRANSVERSE JOINTS
SEC. U-U
SEC. T-T
SEC. D-D
SEC. A-A
SEC. H-H
SEC. E-E
SEC. F-F
SEC. K-K
SEC. L-L
SEC. G-G
SEC. O-O
STA. 25+43.5
STA. 25+37.5
STA. 24+94.5
STA. 25+53.5
STA. 25+30
STA. 17+35
STA. 25+62.5
STA. 25+21.5
STA. 17+40
STA. 25+72.5
STA. 25+13
STA. 17+45
STA. 25+78.5
STA. 25+02
STA. 17+50
LONGITUDINAL PROFILE
427.5 To Axis
Block Numbers
SEC. Y-Y
SEC. S-S
Block 38
Block 37
LEGEND
Erosion 0" To 1"
Erosion 1" To 3"
Erosion 3" To 9"
Erosion 9" To 18"
Erosion Over 18"
Sand, Gravel, Boulders
Surface Ground Smooth
Surface Rough Aggregate Protruded
Potholes - Small Shallow
U.S. Department Of The Interior
Bureau Of Reclamation
Columbia Basin Project Washington
Grand Coulee Dam
Results Of Spillway Bucket Inspection
November - December 1949
Drawn O.J.D.-L.V.D. Submitted
Traced C.W.Z. Recommended
Checked O.J.D. Approved
A-3610-2
Coulee Dam, Wash. Dec. 18 1949



Flow distribution on prototype during 1940 flood season

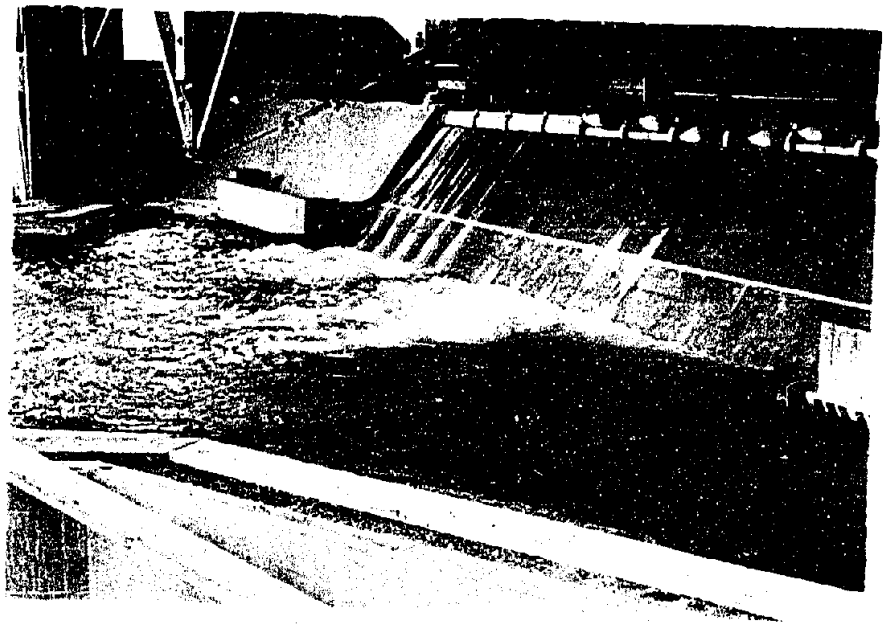
FLOW CONDITIONS AT GRAND COULEE DAM
JUNE, 1940

many instances, noise caused by the impact of rock rolling on the river bottom was audible to persons on the river bank.

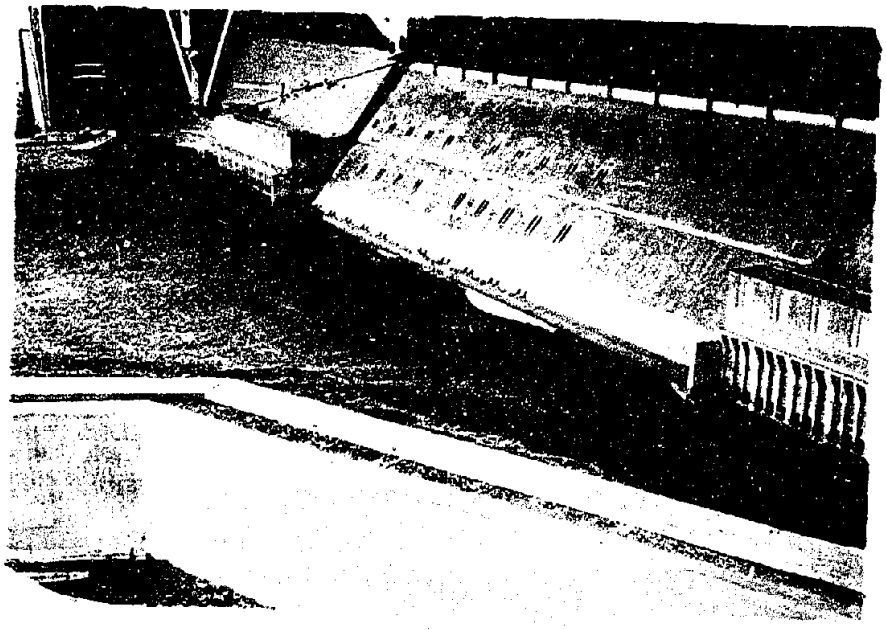
Although it was not possible to represent correctly the 1940 prototype flow because the model was of the completed structure and not a contruction stage and the proper head on the outlets with the correct distribution of water over the crest could not be maintained, the flow conditions below the dam induced by opening the corresponding outlets and placing sand bags on the crest to obtain the approximate flow distribution were similar to those on the prototype in 1940, figure 52A.

Sand particles, varying from silt to 1/2-inch, were moved about, the smaller particles being carried over the lip into the bucket where they were transported along the invert toward the concentrated flow to the right side of the spillway. As they approach the jet plunging through the bucket they moved up the 1 to 1 slope over the lip onto the river bottom downstream. The heavier material near the concentrated flow was moved from an area adjacent to the toe of the bucket and deposited a short distance downstream. A large hole was dug to the porous concrete representing the bedrock in the river channel (figure 52B).

After observing these conditions on the model it was concluded that much of the river-bed material on the prototype was carried upstream over the bucket lip to the left of the eroded channel, rolling toward the invert of the bucket, where it was carried along the bucket axis until it approached the flow from the outlets or flood passages. From this point it was carried out over the bucket lip over a relatively small area, eroding a channel through the lip by abrasive action. The shape of this channel, which veers to the right as it leaves the bucket invert, seems to corroborate this conclusion. It is believed that the damage has increased since the spillway has been placed in operation, but the action is different from that which caused the initial damage. It is believed that rock enters this area from downstream possibly through the channeled lip and is not swept out during normal operation of the spillway, but moved about in the area to produce additional erosion. Repair of this area, or the removal of sufficient



A. Flow of model distributed to represent 1940 flood conditions



B. Erosion of river bed below bucket for flow representing 1940 flood

REPRODUCTION ON 1 TO 60 MODEL DAM OF FLOW CONDITIONS EXISTING DURING 1940 FLOOD AND RESULTING EROSION

material from the river bed to prevent its entering the region, seems to be the only sure means of arresting the action.

Erosion of the construction joints might be attributed to two actions, the movement of material along the joints by currents in them (during unsymmetrical operation) or the rolling of material in the joints by water passing over them at right angles, or the damage might have resulted from a combination of these. To the writer the latter seems a more likely explanation.

The hole eroded in the base of the right training wall and in the slope just below the bucket adjacent to the wall resulted from the abrasive action of rock carried into this region and whirled violently about in a counterclockwise conical-shaped eddy, having a vertical axis with apex at the top, which formed just above the area. This eddy varying in intensity from time to time, turning slowly for awhile, then turning with increasing speed until the motion was extremely violent, was present on the model when either the spillway or the adjacent river outlets were in operation. Sand particles up to 3/8-inch near this region were picked up and whirled violently about, assimilating a high-speed abrasive wheel contacting the concrete surfaces. Damage to this part of the structure could be expected so long as material was supplied to this region. Although severe when the spillway operated normally, the action was intensified when the outlets adjacent to the wall were placed in operation, a condition which existed on the full-sized structure for long periods during 1939, 1940, and 1941. Too, the region was supplied much of the time from a 24-inch pipe with sand and debris from galleries within the dam. It was considered that damage to this region could be checked by eliminating the whirling action of the water or by preventing material from entering it. Tests to determine the practicability of both were made, and the results are discussed in section 29 of this report. The removal of material from the region appeared most practicable.

The excessive erosion of the concrete surfaces of the bucket in construction-block rows 52, 53, and 54 is attributed to the abrasive action of river-bed materials carried into that area by whirling eddies

that formed between the two sets of outlet tubes in rows 51 and 55. Material from the river bed of the model moved about violently in this area, being washed in over the lip and moved about aimlessly in the area but toward the outlet jets which carried it out over the lip and deposited it where it might be picked up again by the whirling eddy and carried back into the bucket. During the tests, this same action, but to a much less degree, was noted between other pairs of outlets. The less severe action was attributed to the smaller distance between the other pairs which prevented the development of such severe eddies. It was concluded that this action could be eliminated either by discontinuing the use of the river outlets or by preventing the material from reaching the area. Considering the entire problem, removal of the river fill downstream seemed the most feasible. Tests relating to this phase of the problem are discussed in section 30 of this report.

In some instances the divers reported wear on the face of the dam below elevation 945 (section 30, block 50, report to Field Engineer by Engineer L. V. Downs, dated December 18, 1943). After observing the flow conditions on the model, for the higher discharges in which sand and gravel were carried by the roll over the bucket lip to the water surface and upstream into the bucket, it was concluded that the wear was of a "sandblast" nature, caused by the abrasion of these materials on the concrete surfaces. It was believed that material transported in the jet and upstream in the roll was thrown spinning against the spillway flow entering the pool, thus causing it to penetrate the sheet and come in contact with the spillway face to produce erosion. There seemed only one practical means of eliminating this action, that of preventing materials from reaching the bucket lip.

A study of the conditions presented in the report mentioned above (figure 50) shows considerable wearing away of the bucket lip below the spillway section containing the river outlets, particularly near the center of the spillway. While a great deal of this destruction occurred when the river outlets were operated for extended periods during construction, the destructive action has continued as a result of normal spillway operation. The model revealed that material, including 3/8-inch rock, moved onto the slope below the bucket lip by the outlet

or spillway flows, was kept continuously in motion and that periodically quantities of this material would be washed into the bucket as if by whirlwinds, where it would move about on the 1 to 1 slope of the bucket before being carried out and deposited again on the river fill downstream, perhaps to be carried back to the slope and reenter the bucket. It is believed that this action explains the source of the damage shown on typical sections D-D, E-E, L-L, and O-O (figure 50). That this type of damage is less severe at the left end of the spillway may be explained by the absence of material below the bucket in this region. It is characteristic of the flow to move the river-bed material from this region toward the center of the spillway section, thereby eliminating the deposit of rock on the slope adjacent to and downstream from the bucket lip. This condition was present for all flow combinations except when the flow over the spillway gates was very small or it was sufficient to produce a steady current of high velocity at the bucket lip (about 250,000 second-feet indicated by the model).

The opinion is that the damage to the base of the left training wall on the powerhouse side has been caused by the movement of rock and gravel across it by the currents in this region. Material placed in the model in the vicinity of this damage was moved aimlessly about by turbulent currents and finally into the main current around the end of the wall, there to be carried under the spillway jet to where some of the particles entered the eddy just inside the wall below the bucket lip, and the rest were carried along the river bottom adjacent to the toe of the bucket toward the center of the spillway.

That material remained in the spillway bucket, with the original profile before damage was inflicted, during large floods, a view expressed by many engineers, does not seem possible. However, after the erosion produced by the movement of materials back and forth over the bucket lip had progressed to give sufficient irregularities to provide space for the formation of eddies under the spillway jet in these eroded areas, it is conceivable that some material might remain in these regions to grind on the concrete throughout the flood season. This

possibility makes it imperative that material below the bucket be removed in accordance with the model findings, section 30 of this report, as soon as practicable. The removal of the river fill will serve to eliminate or allay the erosion to all parts ofthe structure.

27. Optimum flow conditions for inspection of spillway bucket by divers. In view of the desirability of making a second inspection of the spillway bucket as soon as possible after the 1943 summer flood to ascertain if the erosion was progressive and of a rate sufficient to justify a request from the War Production Board for immediate purchase of the necessary strategic materials for construction of the repair equipment, tests were made on the 1 to 60 model to determine the maximum river flow at which diving operations could be undertaken satisfactorily and to determine combinations giving the most favorable flow conditions for the inspection. The prototype flows through the powerhouse andover the various drumgates for each of the test runs and the maximum observed velocities noted for certain of the runs were recorded (table III). Footnotes include references to undesirable conditions relative to tailwater surges or disturbances in the river fill downstream from the spillway bucket. At the outset, tests were conducted to determine what combination of gate settings for flows of 50,000 and 60,000 second-feet, approximate discharges at which it was desired to make the inspection, produced the least tailwater disturbances and gave minimum velocities in the eddy currents along the spillway bucket. After determining the general trend of such manipulations, observations were made of the action of the river fill downstream from the spillway bucket during various flows. For this purpose a small transparent plastic boat was used. To detect tendencies toward disturbances of the river fill or toward material entering the spillway bucket during the tests, fine sand was sprinkled over the river bed adjacent to the spillway bucket. Sawdust was used in observing subsurface flows, and confetti was used for observation of surface eddy current patterns and velocities. Numerous photographs, time exposures, of confetti in motion on the water surface were taken during the progress of the test to record the eddy current patterns and check the velocities in critical regions.

Table III

Summary of Flow Distribution, Velocities and Flow Conditions for Studies pertaining to Diving operations.

Discharge from powerhouse c.f.s.	Gates in operation	Flow per gate c.f.s.	Total flow below dam c.f.s.	Maximum velocity of eddy current; feet per second (prototype) In bucket along face of dam	Upstream along right bank	Remarks
30,000	None	0	30,000	---	1.4	
30,000	1,2,3,4,5	4,000	50,000	1.6	1.7	
30,000	1,2,3,4	5,000	50,000	1.4	1.3	
30,000	1,2,3	6,670	50,000	0.4	0.9	
30,000	1,2	10,000	50,000	1.0	1.8	*
30,000	7,8,9,10,11	4,000	50,000	1.4	---	
30,000	8,9,10,11	5,000	50,000	1.0	---	
30,000	9,10,11	6,670	50,000	1.2	---	
30,000	10,11	10,000	50,000	---	---	*
20,000	1,2,3,4	7,500	50,000	1.3	1.9	
20,000	1,2,3	10,000	50,000	0.8	0.9	
20,000	9,10,11	10,000	50,000	1.0	---	
20,000	9,10,11	13,330	60,000	---	---	*
20,000	8,9,10,11	10,000	60,000	1.6	---	
30,000	9,10,11	10,000	60,000	1.4	---	
30,000	1,2,3	10,000	60,000	0.5	---	
30,000	11	8,500	38,500	Particular note		**
30,000	11	12,500	42,500	was made of mat-		**
30,000	11	14,500	44,500	erial entering		**
30,000	11	20,000	50,000	the bucket and	*	**
30,000	11	30,000	60,000	no velocity mea-	*	**
30,000	7	5,000	35,000	surements were		
30,000	7	10,000	40,000	taken.		**
30,000	7	20,000	50,000		*	**
30,000	7	30,000	60,000		*	**
30,000	7,8	15,000	80,000	No velocity ob-	*	**
30,000	6,7,8,9	15,000	90,000	servations were		**
				made.		
30,000	9,10,11	10,000	60,000	Particular note		**
30,000	all	0 to	30,000 to	was made of mat-		
		25,730	315,000	erial entering		
30,000	9,10,11	5,000	45,000	the spillway buck-		
30,000	9,10,11	6,000	48,000	et and no velocity		
30,000	9,10,11	7,000	51,000	measurements were		**
30,000	9,10,11	8,000	54,000	taken.		**
30,000	9,10,11	9,000	57,000			**

Table III (Cont'd.)

Discharge from powerhouse c.f.s.	Gates in operation	Flow per gate c.f.s.	Total flow below dam c.f.s.	Maximum velocity of eddy current, feet per second (prototype) In bucket along face of dam	Upstream along right bank	Remarks
30,000	9	3,200				
	10	5,000				
	11	6,800	45,000	---	---	
30,000	9	5,000				
	10	6,700				
	11	8,300	50,000	---	---	
30,000	9	6,000				
	10	8,000				
	11	10,000	54,000	---	---	**
30,000	9	7,600				
	10	10,100				
	11	12,300	60,000	1.3	---	**
30,000	8	2,700				
	9	3,900				
	10	5,700				
	11	7,700	50,000	0.9	---	
30,000	8	3,600				
	9	4,900				
	10	6,800				
	11	8,700	54,000	---	---	
30,000	8	4,600				
	9	6,200				
	10	8,500				
	11	10,700	60,000	1.3	---	**
30,000	1	7,700				
	2	5,700				
	3	3,900				
	4	2,700	50,000	1.1	1.2	
30,000	1	8,700				
	2	6,800				
	3	4,900				
	4	3,600	54,000	---	---	
30,000	1	10,700				
	2	8,500				
	3	6,200				
	4	4,600	60,000	1.2	2.0	**

* Water surface very rough.

** Tendency for material to enter spillway bucket exists, or violent movement occurs on river bottom downstream from the bucket lip. These conditions are considered unfavorable and the prototype structure should not be operated with such flow distribution.

At the time these tests were made the flow through the left powerhouse was slightly above 20,000 second-feet, but installation of another unit was expected before the inspection, which would increase the quantity to about 30,000 second-feet, however, the conditions below the dam were observed for both discharges.

The total model flow was varied to represent from 30,000 to 80,000 second-feet with the flow in excess of the powerhouse capacity distributed over various of the drum gates set at unequal, as well as equal elevation. In the initial tests it appeared that optimum conditions for diving would result when the spillway flow was confined to the end gates so the program was continued on this basis. Increased roughness and higher velocities were noted as more water was discharged over the drum gates and indications were that diving operations would not be feasible for flows much larger than 50,000 second-feet, thus quantities in excess of that representing 60,000 second-feet were seldom investigated.

Flow conditions with the gates at different levels, the elevation increasing and the flow diminishing toward the center of the river, were improved over those for the same quantity with the gates at equal levels. In general, smoother flow resulted when the water passed over the right instead of the left end of the spillway. In the latter case the extreme concentration produced higher velocities in the currents over the bucket.

The tendency for materials to be carried into the bucket by currents adjacent to the flow from the drum gates was intensified as the discharge increased regardless of whether the gates were kept at the same level, and the action was more pronounced with the gates at the same level than with the elevation increased progressively toward the center of the structure. The tendency decreased as the flow was distributed over more gates.

A spiral-like current revolving about a horizontal axis was observed in the bucket adjacent to the unconfined edge of the spillway flow for most of the gate combinations. Though the extent of this current was not recorded for every combination, it generally extended 250 to 500 feet (prototype) along the bucket. Suspended material caught

in this current moved with increasing velocity toward the drum gate discharge where it was carried away.

Throughout these tests surges in the water surface, where work in the bucket was expected, varied considerably and as a general rule, the magnitude of the surge increased with the concentration of flow. With the additional turbine in operation, increasing the flow through the left powerhouse, it was believed that diving operations could be conducted safely in the spillway bucket with considerably more total discharge as compared to that existing during the operations in March 1943. From the model tests it seemed that 54,000 second-feet approached the limit of total river discharge at which operations in the spillway bucket should be undertaken, this being predicated on a flow of 30,000 second-feet through the left powerhouse, attainable only after Unit L-5 was placed in operation.

Although the total discharge varied from 57,000 to 46,000 second-feet during the diving operations, in general, they were performed with less difficulty than for the spring inspection. This fact was attributed in part to the more favorable river flow conditions resulting from operating the drum gates in accordance with the model findings and to higher water temperatures which allowed the divers to stay down for longer periods.

28. Sand and gravel found in spillway bucket during 1943 fall inspection. The second inspection of the spillway bucket by divers was begun on November 15, 1943. The flow was distributed over the drum gates in accordance with the model findings discussed in the foregoing section of this report and operations begun at the left end of the spillway. No material was found in the bucket until construction-block row 40 was reached, where considerable sand and gravel had been deposited. Tests were conducted on the 1 to 60 model in an effort to ascertain its source.

A fine sand was placed on the river fill adjacent to the bucket lip since it was certain that the material used to represent the river bed was too coarse to be transported at such small flows. The sand was placed in the model river channel with a flow representing 85,000

second-feet passing over the eleven drum gates, all of which were at the same level. During the placing of the sand, the fine grains, up to approximately 1/64-inch, were carried upstream to the lip where some were raised upward by the flow from the bucket. A portion of these were carried upstream into the bucket while the rest were moved downstream and deposited on the river bottom. At points along the lip where the velocity of the water leaving it was unsteady, the grains of sand crossed and fell into the bucket. This material moved about but was never swept completely from the bucket. Although many grains were swept out by the currents, others were dislodged from the lip to take their place.

When the flow over the crest was increased to that representing approximately 120,000 second-feet, all the sand grains were swept from the bucket and deposited on the sand ridge at the lip where small eddies often dislodged them, carried them over the lip onto the 1 to 1 slope and moved them about vigorously before again depositing them on the sand ridge. At this point it was desired to ascertain the action of particles placed in the bucket. Sand was scattered along the roll for ten or twelve feet of bucket length and its movement observed through a transparent-bottomed vessel. Much of the sand was caught in the spillway roll where it was carried upstream into the bucket roll, held in it for several turns of the roll, then deposited either on the ridge below the lip or on the 1 to 1 slope of the bucket where it moved up and down the slope and in and out over the lip. After a few minutes only the larger grains, approximately 1/16 to 1/8-inch, remained in the bucket. Since these were moving slowly from the bucket the flow was increased until that passing over the drum gates represented approximately 155,000 second-feet. The particles were soon swept from the bucket and deposited on the ridge of sand at the lip where they were intermittently carried into the bucket and again deposited on the ridge of sand. As soon as the bucket was free of sand except from the action just described, the flow over the spillway was decreased slowly to that representing 95,000 second-feet. The action was similar but less steady as the flow decreased. Approximately twenty small grains remained in the bucket when the flow was decreased to a point where no movement

could be detected at the lip. Since the gradual decrease in flow did not appear capable of depositing the quantities of material reported in the bucket during the diving operations, the gates were operated unsymmetrically.

The drum gates were operated using combinations of gates 1 to 4, inclusive, 1 to 5, 1 to 6, 1 to 7, and 1 to 8. A few grains of sand were found in the bucket to the right of the discharge in each case but the quantity did not exceed that deposited by the symmetrical operations. Though the quantity of sand remaining in the bucket during the tests was small, it is reasonable to assume that the same action exists on the prototype structure and that it is more severe in nature. At this point consideration was given to factors which, under normal conditions, do not permit exact similitude between the model and the prototype. These are minor as a rule and do not alter the final results but at times become very important. The viscosity factor is not to scale when the fluid in both model and prototype is the same and neither is the surrounding pressure when both operate under atmospheric pressure. These factors influence the turbulence, both tending to damp it, thus it might be expected that there would be less movement of material for small flows on the model than for corresponding flows on the prototype. Moreover the relation between the transportation of material on the model and that on the prototype is uncertain, thus the model material may not have been of the proper size and density. To overcome these conditions to some extent, silt instead of sand was used and similar observations made.

With all drum gates at the same elevation and small discharges (less than 65,000 second-feet) silt placed several inches downstream from the bucket moved upstream to the lip and was carried to the surface, part turning upstream into the roll over the bucket and the rest moving on downstream. Some of that entering the roll, which formed near the face of the dam, was deposited in the bucket downstream from the invert. Much of it was then carried out by the unsteady currents passing over the lip.

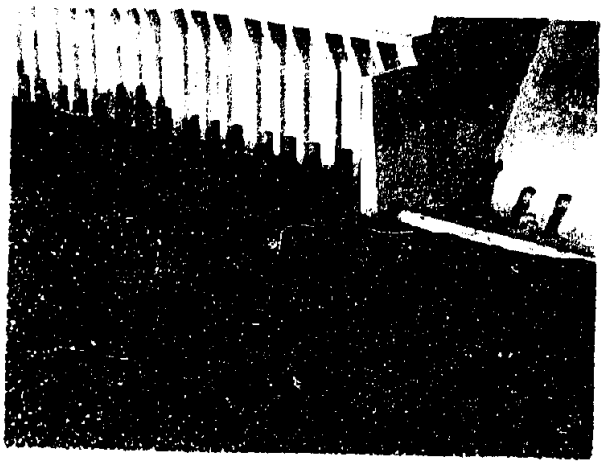
With combinations using gates 1 to 4, inclusive, 1 to 5, 1 to 6, 1 to 7, and 1 to 8, silt placed several feet downstream in the eddy,

which moved upstream toward the bucket below the right end of the spillway, was carried upstream over the bucket lip where a thin blanket was deposited on the 1 to 1 slope.

From the silt tests it seems that the deposit of sand and gravel found in the bucket of the prototype structure during the fall inspection might have resulted from either symmetrical or unsymmetrical operation of the spillway gates or both. In the opinion of the writer, the deposit resulted from both types of operation, the greatest portion being deposited during the unsymmetrical arrangement. The gravel was deposited in greater quantities near the central section of the spillway than at the ends since it is characteristic of the spillway to form a higher ridge of sand and gravel in this region when the spillway operates.

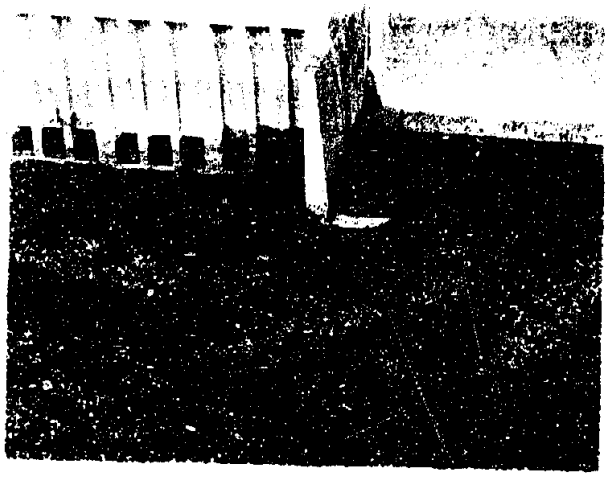
These tests seem to emphasize the necessity of the dredging operation mentioned previously.

29. Study of flow conditions and erosion at ends of traning walls. In initial studies on a 1 to 120 scale model of Grand Coulee Dam, conducted in 1935, it was disclosed that holes would be formed in the river fill at the ends of the spillway training walls when flood waters were discharged by the spillway. Tests were initiated in the laboratory at that time to study the action and to evolve some means of eliminating or minimizing it. Extensions of different shapes and sizes were made to the right wall of the model and although some success was indicated, the improvement did not seem to justify the additional cost and the studies were discontinued. The results of the tests are discussed in an abstract, by the writer, "Erosion at Downstream Ends of Training Walls, 1 to 120 scale model of Grand Coulee Dam", Appendix II of this report. Conditions similar to those observed on the 1 to 120 model were disclosed when the 1 to 60 model was placed in operation in 1943 (figures 53 and 54, A and B). In view of the damage to the base of the right training wall reported by the divers in the spring and fall inspections of 1943, studies were made on the 1 to 60 model to ascertain if the conditions at the ends of the walls noted on both models were contributing factors.

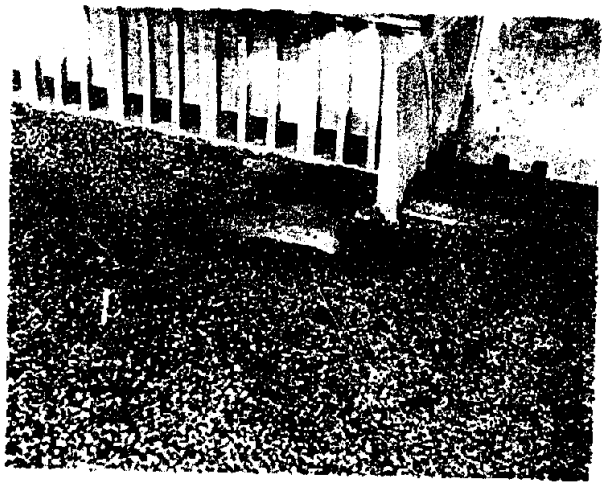


A. Pumice and screeded coal at right wall for erosion tests

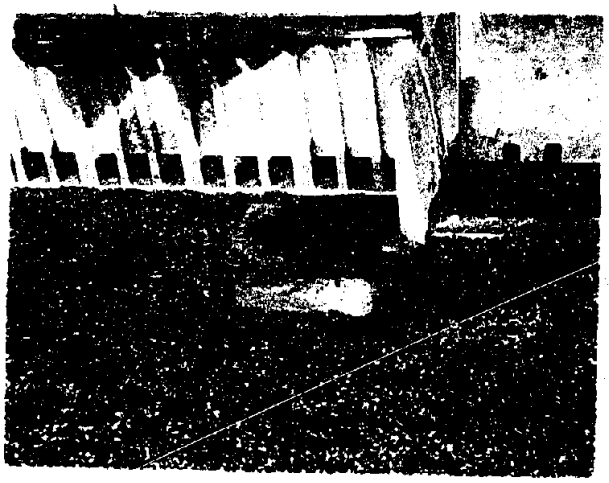
B. Condition at base of wall after flow of 787,000 second-feet



C. Conditions with concrete block in crevice at end of wall after flow of 287,000 second-feet

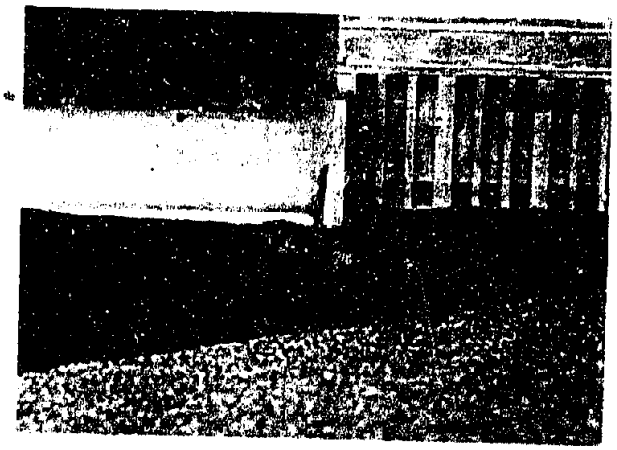


D. Conditions at end of wall after flow of 537,000 second-feet

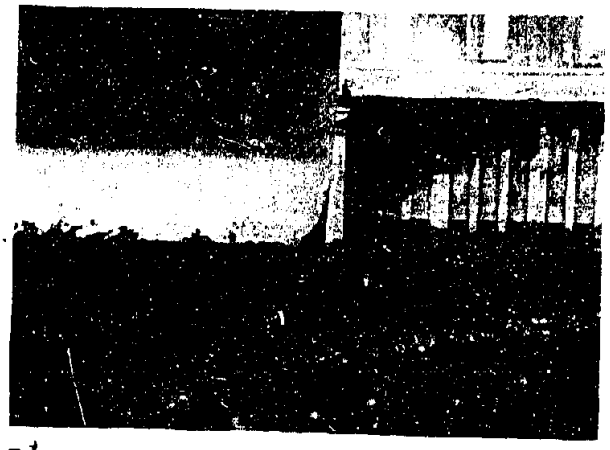


E. Conditions at end of wall after flow of 787,000 second-feet

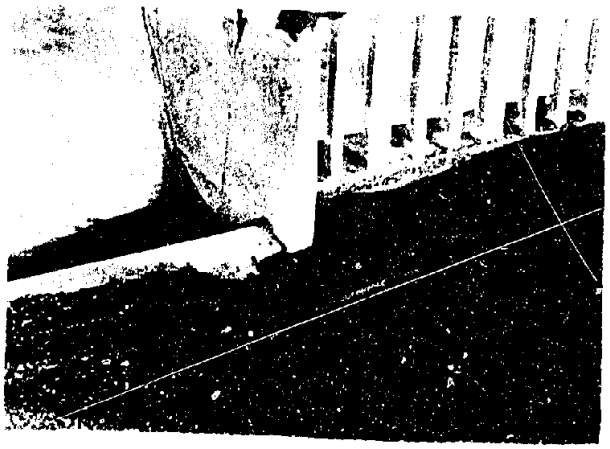
EROSION AT END OF RIGHT TRAINING WALL WITH AND WITHOUT CONCRETE BLOCK IN CREVICE BETWEEN WALL AND ROCK TOPOGRAPHY



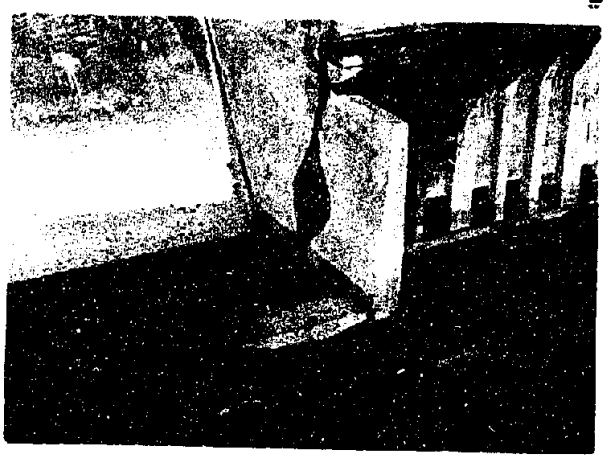
A. Gravel at end of left training wall for erosion tests



B. Conditions at base of left training wall after flow of 787,000 second-feet



C. Bonded rock and concrete cap at end of left training wall



D. Abrasive erosion of concrete block after flow of 787,000 second-feet for three hours

E. Abrasive erosion of concrete block after flow of 787,000 second-feet for 9 hours

EROSION AT END OF LEFT TRAINING WALL WITH AND WITHOUT BONDING THE ROCK AND ADDING THE CONCRETE CAP AT END OF WALL

In the studies it was found that a strong current persisted downstream adjacent to the powerhouse side of the wall, around its end and upstream along the base on the spillway side to the bucket lip where the flow divided, part continuing along the wall into the bucket and the rest turning at right angles to the wall and moving along the river bottom parallel and adjacent to the bucket lip. There was a counterclockwise whirling cone-shaped eddy, with a vertical axis, the apex at the top and the base on the 2-1/2 to 1 slope below the lip, formed just downstream from the lip adjacent to the wall. This action was present at all times at the right wall when there was any appreciable quantity over the adjacent gate or through the adjacent pairs of upper, intermediate, or lower river outlets. Pebbles transported from the river fill at the end of the wall, by the current around it, were carried upstream along the base on the spillway side, some to be moved toward the center of the spillway by that portion of the current moving along the bucket in that direction and the rest to be moved on upstream into the base of the whirling eddy just described.

A portion of those entering the eddy, which was of a spasmodic nature, were forced out of it onto the slope below the bucket lip where they might remain for a short time before being picked up either by the current moving toward the center of the spillway or be carried again into the eddy and whirled. At times some of the whirling pebbles would be lifted to the apex of the whirling cone adjacent to the wall and carried upstream into the bucket. Material entering the bucket appeared to be washed out almost immediately under most conditions, however, particles entering at the left end of the spillway were observed to remain in the bucket for several minutes moving up and down the slope downstream from the bucket invert before being carried out over the bucket lip. This action was attributed to an eddy which formed in this region when the high-velocity flow through the bucket could not follow the boundary due to the flare resulting from the intersection of the upward slope of the bucket surface and the batter on the inside surface of the training wall.

The action at both walls was similar but the whirls were of opposite hand and that at the right wall was more severe. The more severe

conditions at the right wall were attributed to the absence of flow from the right powerhouse.

Measurements of the velocity of the current toward the center of the spillway downstream from the end of the right training wall of the model were made. The velocity increased with the spillway discharge, varied from 1.9 to 9.0 feet per second prototype for model flows representing prototype discharges of 100,000 to 250,000 second-feet, and became so great at 500,000 second-feet, that the revolutions of the current meter could not be counted.

The reason for the damage to the right training wall of the prototype structure was shown clearly by this test.

As there seemed but two practicable ways to prevent further damage to the wall, namely, eliminate the whirl, or keep material from entering it, the tests on the 1 to 60 model concerning the conditions at the training walls were made accordingly.

Extensions to the existing walls, bonding the adjacent riprapped surfaces, and removing the overburden in the vicinity of the wall were the treatments studied and, although satisfactory action was indicated for some of the wall extensions, the removal of material to prevent its entering the region seemed the most practicable and economical treatment.

A treatment of the damage at the base of the right wall and the placing of a concrete block in the crevice at the end of the wall, proposed by project engineers, was studied on the model dam.

The concrete block, filling the crevice between the end of the right wall and the bedrock downstream, did not effect appreciably the flow conditions or action adjacent to the wall, in fact, a pronounced whirl formed at the end of the wall on the spillway side of the concrete block, a condition which would most certainly result in erosion of the wall and the block.

The tendency of material to be transported from the powerhouse side of the wall was lessened by the filling of the crevice (figures 53, D, and E). An extension of the base of the right wall for an equivalent of 20 feet downstream with its top at elevation 920 was made. The eddies on the inside of the wall were not eliminated but their

formation was intermittent and they remained nearer the end of the wall than before, seldom moving upstream to the bucket lip. In general, the material reaching the $2\frac{1}{2}$ to 1 slope below the bucket lip was moved back and forth in an upstream and downstream direction and not whirled. Occasionally a violent whirl formed at the end of the extension and moved upstream along the inside of the wall then disappeared. Raising the top of the extension had no appreciable effect on the action described above. The sharp corners at the downstream end of the wall were the main factors inducing the whirling eddies. A low wall extension of approximately $22\frac{1}{2}$ feet with its top at elevation 920 curved bankward at the downstream end, would eliminate this characteristic. With this extension there was very little disturbance on the inside of the original wall, the currents being in an upstream direction near the base of the wall. Very little movement of the material was noted and that carried from the end of the extension was deposited below the bucket adjacent to the wall. However, a region of violent turbulence was formed on the inside of the extension near its downstream end. Further refinement of the shape of the end of the extension would improve conditions but the solution did not seem practicable because of the cost and the construction difficulties involved, thus testing was discontinued on the wall extensions. The more feasible solution seemed to be the removal of material from the end of the wall and from regions in which it would be transported to this area under normal operation of the spillway and powerhouses.

Simultaneous tests, but with different treatment of the area adjacent to the wall, were made on the left training wall. These tests for the main part involved the bonding of the river fill and riprap adjacent to and downstream from the wall and the removal of material in this area. Of the treatments tested, that of removing the material to approximately elevation 885 and bonding the rock on the river bed seemed most promising.

For the initial test on this wall, the rock fill on the river bottom, bounded by the downstream edge of the powerhouse base, the training wall, the spillway bucket, and stations A14+40, B15+00, and

A13+40, was bonded by grouting and a concrete cap to support the toe of the tailrace slope added to a portion of this area downstream and to the right of the training wall (figure 54C). This arrangement was proposed by project engineers.

For small discharges a lightweight material, pumice, was dropped into the water behind the wall to study the action in this region. For flows up to and including that representing 200,000 second-feet on the prototype (37,000 through the powerhouse and 163,000 over the spillway) water-soaked pumice placed on the base at the back of the wall was carried around the wall and upstream, some entering the whirl over the bucket lip and moving into the bucket, but most of it being carried toward the center of the spillway by the transverse current under the spillway roll and deposited on the river fill. A very indistinct whirl on the concrete cap at the base of the wall was noted. As the flow was increased, the action became more pronounced and materials including sand particles 3/8-inch in size were whirled violently about on the concrete surface of the cap. The action was so violent for a flow representing 750,000 second-feet that the comparatively soft surface of the block was eroded as if by sandblasting. Photographs were taken of the erosion after the model had been operated for periods of three and nine hours (figures 54 D and E).

From the foregoing test it seems certain that conditions damaging the base of the left wall would be induced by a treatment of this type, thus it should not be considered for the prototype. During this test, movement of material to the left of the wall and immediately downstream from the left powerhouse was observed, with and without unit 9 operating. Without unit 9 there was a tendency for materials in this region to move aimlessly about on the base of the wall. In moving about, those entering the region near the end of the wall were picked up by the current passing around the end where they were transported as explained previously. With unit 9 in operation, any material entering the region was carried outward and into the current passing around the end of the wall. It is believed that materials have been carried into this area by currents induced by certain combinations of spillway and powerhouse

discharge and that the damage to the base of the wall on the powerhouse side has resulted from the action just described.

For a subsequent test the area surrounding the wall was excavated bowl-shape with the bottom at elevation 885 and the materials on the river bottom in the region bonded. Pumice and sand placed on the powerhouse side of the wall were carried around the end and upstream as with the bonded area and concrete cap described previously, but no eddy formed near the end of the wall and most of the material was carried by the current moving toward the center of the spillway downstream from the bucket. A few particles entered the bowl-like excavation and settled to the bottom near the base of the wall, where very little motion was noted for flows up to and including 537,000 second-feet. However, a few of the smaller particles moved upstream to the $2\frac{1}{2}$ to 1 slope below the bucket lip adjacent to the wall where they would remain for a short time vibrating, as if on a vibrating table of high frequency, until a whirling eddy formed adjacent to the wall to lift and move them about violently, carrying them into the bucket or dropping them on the slope as the eddy disappeared to reform again a few moments later. However, this treatment was considered a substantial improvement over that in which the area was bonded and a concrete cap added.

In view of the encouraging results obtained with the bowl-shaped excavation, the area bounded by the powerhouse, the training wall, the bucket, the stations A14 + 70, B14 + 50 and A13 + 50, was rearranged. The area was sloped downward toward the center of the spillway from elevation 896 at station A13 + 50 to bedrock (about elevation 885) at station A14 + 70. With discharges up to and including 337,000 second-feet the action was similar to that for the bowl-like excavation. At higher flows an eddy similar to that for the bonded area and concrete cap formed at the end on the inside edge of the base of the wall, and violent movement of material occurred, being carried from the other side of the wall where it was placed. In view of the results of these tests and the decision to dredge the river fill below the bucket to prevent damage to it by the movement of the river bed materials, it

seems that dredging near the training walls would prove most satisfactory, particularly when cost, construction difficulties and effectiveness are considered.

30. Studies concerning the dredging of river fill to prevent or allay damage to the spillway bucket. The serious nature of the damage to the spillway bucket of Grand Coulee Dam, ascertained from inspections of that part of the structure by divers, and the disclosure by tests on the 1 to 60 model that the destruction incurred by the bucket resulted from the movement of river bed material by currents in the eroded areas made it imperative that steps be taken to eliminate or allay the action; otherwise the repair and maintenance work would be continuous. Studies were conducted therefore to evolve satisfactory corrective measures. There seemed but two methods which might prove practicable, that of stabilizing the river fill in the immediate vicinity below the bucket, to prevent material from being transported to the bucket to cause erosion by abrasion, and that of dredging the loose material from the river bed increasing the area of flow of the ground roller, thereby reducing its ability to transport materials and decreasing the amount of material available for transporation by the ground roller.

The river fill on the model was excavated to a level representing elevation 890 on the prototype for various distances downstream from the bucket and a one-inch covering (elevation 895.0) of 1/4- to 1-inch rock added. The behavior of the river bed material was studied for flows representing from 50,000 to 337,000 second-feet on the full-sized structure. Consolidation in this manner for a distance of five inches downstream from the model spillway appeared ample from station A14 + 00 to station A16 + 00 and from station A29 + 00 to A31 + 00. At other points the 5-inch strip of heavy rock was covered completely with fine sand transported upstream along the river bottom by the ground roller below the bucket. A 1-foot width near the center of the spillway appeared to be satisfactory for the small discharges but fine sand was carried upstream to cover the rock as the discharge approached that representing 300,000 second-feet. The results indicated that it would be necessary to stabilize the river fill for more than sixty feet downstream if

material was to be kept from the bucket during floods of this magnitude and possibly for a much greater distance for larger floods. At this point in the program it seemed more logical to dredge the material from the river and omit the covering of heavy rock, thus tests concerning the consolidation of material below the bucket were discontinued.

Sand was removed from the model to levels representing elevations 880 and 885 for distances downstream varying from six inches adjacent to the training walls to 20 inches at the center of the spillway (30 to 100 feet on the prototype), and the action observed for flows representing up to 337,000 second-feet, the capacity of the model supply system or about one-third the design discharge of the structure. The fine sand in the excavated area moved gradually upstream to the toe of the 1/2 to 1 slope downstream from the bucket where it was deposited, the depressed area acting as a catchment. At the end of two hours' operation at the discharge representing 337,000 second-feet, no material had reached the bucket lip. There seemed to be very little transportation of the material from the downstream edge of the excavated region, particularly where the width was 20 inches. From this test it seemed that dredging, without replacing the material with heavy rock, would be the most practicable of the two treatments, and that the limits for the dredging below the spillway bucket would extend at least 100 feet downstream from the center portion of the structure and possibly for a lesser distance at the training walls.

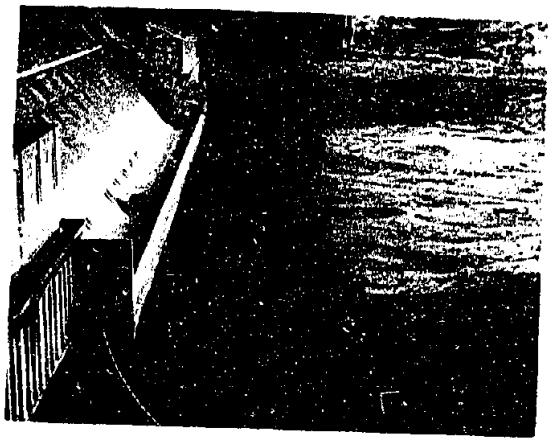
Since floods approaching 500,000 second-feet are expected occasionally and it is possible that larger floods will occur, it was considered imperative that investigations be expanded to include model discharges representing these quantities. The distance downstream from which transportation of the loose bed material took place seemed to vary as the discharge, thus tests to establish limits to which the dredging would have to be carried for the higher discharges were desirable and the program was postponed to permit the purchase and installation of an adequate pump and supply system.

The tests were resumed in 1944 when the installation of a 24-inch vertical propeller pump with a capacity representing the design discharge

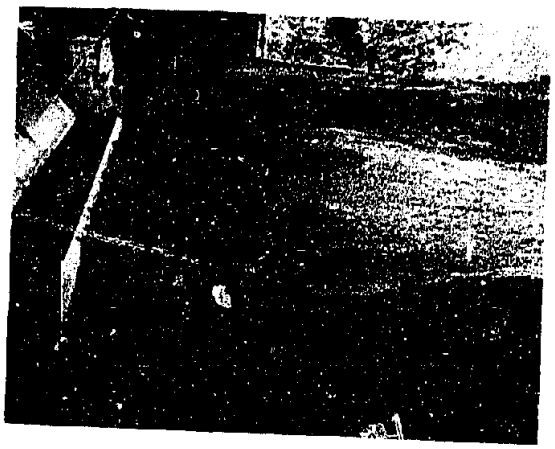
of 1,000,000 second-feet, was completed. The tests pertained principally to the establishing of the limits for dredging the downstream river fill. The limits used in the first test with the increased model capacity were based on the configuration of the river channel below the spillway section after the model had been operated through a flow cycle representing from 69,000 to 787,000 second-feet. The material in an area bounded by a line running through a point on station A14 + 20, 75 feet prototype below the bucket, to station A16 + 50, 220 feet from the bucket lip, to station A28 + 50 at the same distance downstream, and then through station A30 + 70 at a point 75 feet downstream from the bucket lip, was removed to a level corresponding to elevation 885 for this test (figure 55A). The model was observed with a flow representing 300,000 second-feet. As considerable material was moved from the bedrock surface and deposited against the 1/2 to 1 slope at the toe of the dam (figure 55B) and difficulty was experienced in determining if the limits were satisfactory, the material was swept from the bedrock surface and slightly more excavation made at the ends of the spillway (figure 55E). The limits were made to intersect stations A14 + 20 and A30 + 70 at 100 instead of 75 feet prototype, downstream from the bucket lip; otherwise the excavation was the same. The model was operated again at 300,000 second-feet and attempts made to measure the upstream velocity of the ground roll on the river bed just below the bucket. Too much turbulence was present, and the currents were so spasmodic that the results were not satisfactory. One velocity measurement indicated 5.8 feet per second, but the velocities were much higher at times and in some instances reversed.

Although a small amount of material was deposited along the toe of the bucket in the central portion of the spillway (figure 55D) it was believed to originate from pores in the bedrock surface which could not be swept free of all sand particles rather than from the river fill downstream from the excavated area. The deposited material was removed, and the flow increased to that representing 537,000 second-feet (500,000 over the spillway and 37,000 through the left powerhouse).

After five hours' operation at this flow, a bar with an equivalent maximum width of 70 feet and depth of 10 feet had formed below the



A. River fill removed to elevation 885 for 220 feet downstream



B. Condition of excavated area after flow representing 300,000 second-feet for 5 hours

C. Excavation extended 25 feet further downstream at end of spillway and all material swept from area

D. Condition of excavated area after flow representing 300,000 second-feet for 5 hours

E. Condition of excavated area after flow representing 537,000 second-feet for 5 hours

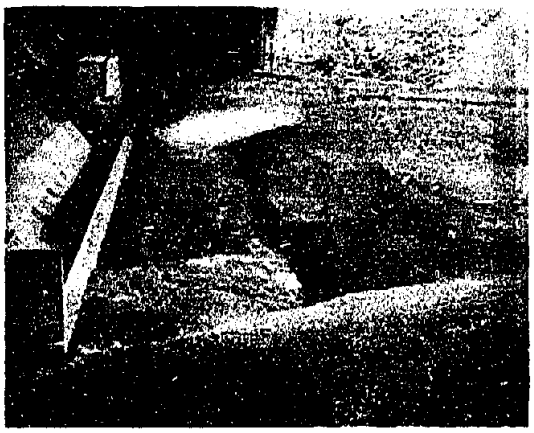
MOVEMENT OF RIVER FILL BELOW BUCKET OF 1 TO 60 MODEL DAM FOR VARIOUS DEPTHS AND LIMITS OF EXCAVATION AND FLOWS REPRESENTING 300,000 AND 537,000 SECOND-FEET

central section of the spillway between station A17 + 50 and A27 + 50. From these stations, the ends extended in an arc to the tailrace slopes farther downstream (figure 55E). Although conditions indicated the tailrace slopes to be the main source of this material, it was considered possible that some had originated from the slopes along the downstream side of the excavated area.

To ascertain the source or sources of this material, sections along the downstream boundary of the excavated area were sprayed with a thin paint. Three colors were used - white at the ends, red on sections bounding the excavated area between stations A15 + 50 and A20 + 50, and A29 + 50, and blue in the central portion between stations A20 + 50 and A25 + 50 (figure 56A).

A study of the results showed that a small proportion of the material in the bar had originated from the ends; that some of it had moved from the downstream edge of the excavated area where the river bed was low; and that the greatest part had been transported upstream from the slope where the river bed was high downstream from the left end of the spillway. This was considered as an indication that the limits should be extended, so material was removed from the bedrock for an additional 40 feet prototype downstream, based on measurements made upstream from the section where the river bed downstream was low following the previous test. The new boundary passed through station A14 + 20, a distance equivalent to 100 feet downstream from the bucket lip, to station A15 + 40, 260 feet downstream from the bucket lip, to station 29 + 50 at the same distance, then through station A30 + 70 at a point 100 feet downstream from the bucket lip.

Using colors, as in the previous test, a five-hour run at 537,000 second-feet was made. There was very little material moved upstream from the ends and from the boundary where the river bed was low, but a bar formed upstream from the region where the river bed was high below the left end of the spillway (figure 56B). To determine how the higher ground contributed to the amount of material moved upstream, the mound in the river channel below the left end of the spillway was excavated to a level representing elevation 900 (figure 56C).

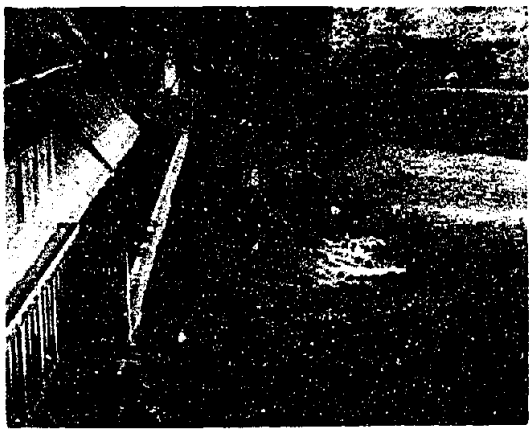


A. Excavated slopes painted to study movement of material

B. Conditions of excavated area after flow of 537,000 second-feet for 5 hours

EXCAVATED AREA EXTENDING 220 FEET PROTOTYPE DOWNSTREAM

C. Excavated area extended 250 feet downstream and mound below left end of spillway removed to elevation 900



D. After flow representing 537,000 second-feet for 5 hours

E. After flow representing 787,000 second-feet for 5 hours

CONDITION OF EXCAVATION AREA BELOW BUCKET
MOVEMENT OF RIVER FILL BELOW BUCKET OF 1 TO 60 MODEL DAM FOR
VARIOUS DOWNSTREAM LIMITATIONS OF EXCAVATION AND FLOWS OF
537,000 AND 787,000 SECOND-FEET

Because the quantity of material moving upstream from the boundary of the excavated area at the right end of the spillway was small, it was not considered practicable to extend the limits in this region.

The relative position of the spillway and tailrace on the left side of the river made the fixing of excavation limits difficult in that region as they were dependent upon the treatment of the tailrace banks. Thus the excavation boundary in this area should not be established definitely until the nature of the treatment has been determined and tests for this purpose are in progress.

For comparable conditions, the removal of the mound reduced the amount of material carried upstream, there being about half the amount in this test for a discharge of 537,000 second-feet as for the same flow in the previous test (figure 56D). Indications were that the downstream boundary of the excavated area should be extended, possibly to the equivalent of 300 feet on the prototype, if transportation upstream was to be eliminated. However, it does not seem practicable to excavate the additional material since the limits were satisfactory for flows in excess of 300,000 second-feet. Floods of such magnitudes are infrequent, thus material from this source could be removed when conditions adjacent to the bucket demanded it. There was some cutting at the toe of the left tailrace slope downstream from the end of the training wall and some transportation of material upstream at the ends of the spillway toward the center just downstream from the bucket.

In order to determine whether or not the flow conditions, erosion and transportation of material would be worsened with an increase in discharge, the model was operated for five hours at a flow representing 787,000 second-feet. The results were much more severe than for the lower flow (figure 56E). Although there was some transportation of sand from the downstream edge of the excavated area, the greater portion of the bar deposited below the bucket seemed to originate from the tailrace slopes just downstream from the ends of the training walls. The slopes were eroded by the whirling eddies just outside the spillway flow at the ends. These whirls, counter-clockwise at the left and clockwise at the right, carried material from the slopes, moving it

upstream under the spillway flow, where it was picked up by the current moving under the spillway jet toward the center of the river and deposited as a bar downstream from the bucket. The top of the bar was at the elevation of the bucket lip and the width varied from 60 to 80 feet prototype. The axis of the bar was parallel to the bucket between stations A17 + 15 and A27 + 75. The ends of the bar curved gradually downstream from these stations until their upstream edges were about 100 feet (prototype) from the bucket lip at points directly downstream from the training walls.

Because of the remote possibility of the occurrence of floods of 750,000 second-feet or more, it was not considered practicable to provide for greater discharges and the test was therefore discontinued.

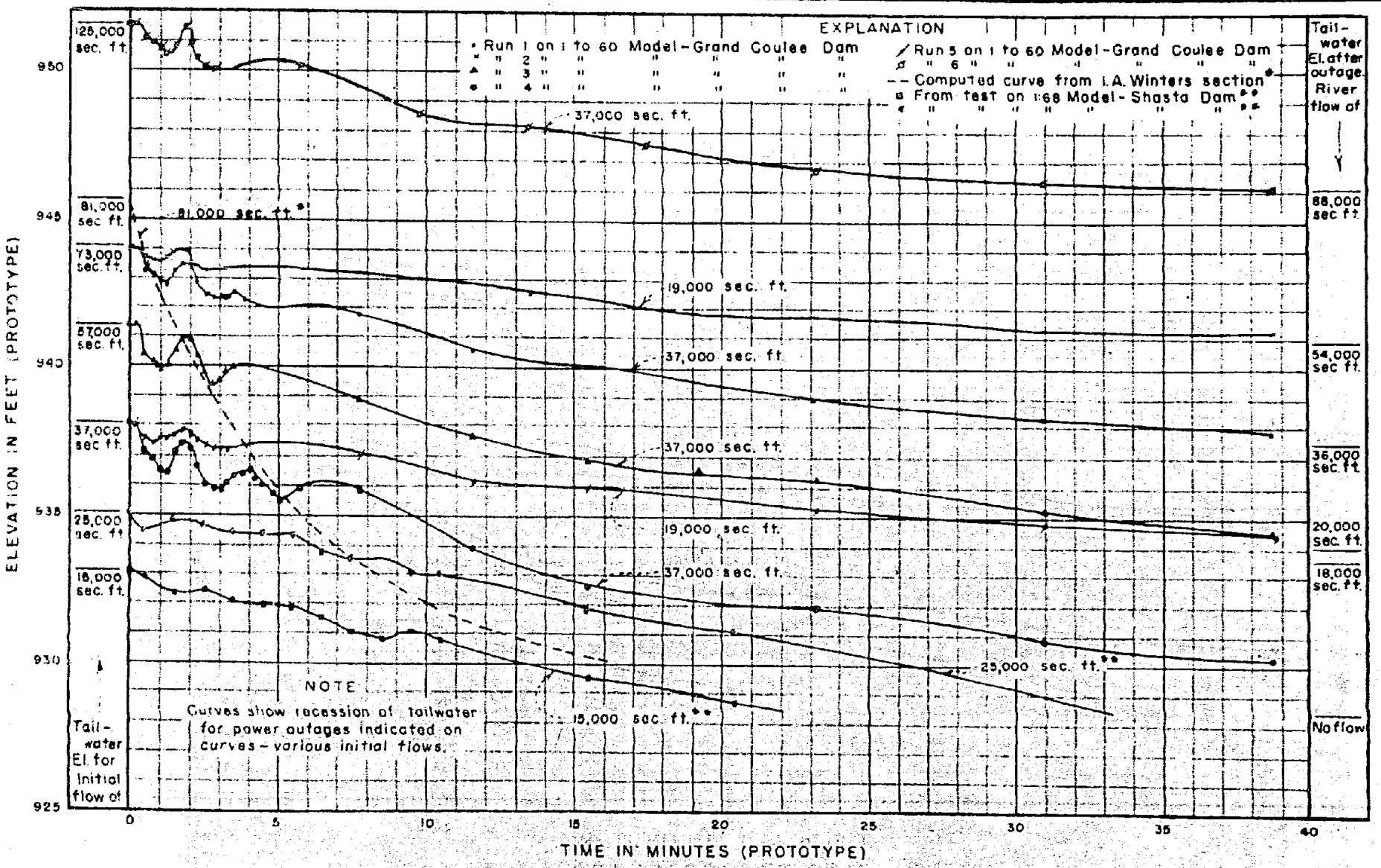
The amount of excavation by dredging and the limits will depend upon the flows for which protection is desired. It is doubtful if it will be practicable to remove all material to bed rock. The more logical and economical procedure would seem to be that of excavating the small material on the river bed to an approximate elevation (about elevation 885) leaving all heavy rock, and dredging any material which might be deposited below the bucket during the summer flood, after the seasonal run-off.

After dredging to the elevation within the limits determined for the flood expected it is believed that very little damage by material transported upstream would result in the short time the larger floods would exist.

31. Study of the effect of power outages on the tailwater elevation. In the early stages of construction consideration was given to the possibility of slides occurring in the tailrace slopes due to a sudden fall in the tailwater elevation resulting from a complete or partial outage of power, but it was not until 1940 that tests were initiated to determine the rate of fall in the water surface to establish some means of ascertaining the speed with which the river outlets or other controls would have to be set in operation to maintain a reasonably constant water surface in the river below the dam. At that time models of Grand Coulee Dam were not available. However, the left

tailrace and part of the downstream river channel were represented on a 1 to 68 model of Shasta Dam. The results from those tests are shown on figure 57. For a flow and complete outage of 15,000 second-feet, the drop was 3 feet in 13 minutes and 7 feet in 40 minutes. The same was noted for 25,000 second-feet. These figures are not in agreement with those obtained on the 1 to 60 model where the tailwater elevation was very close to normal for the initial and final flow without regulation of the tailwater control (figure 57). Neither agreed with the computed values (figure 57), but some of the difference might be the result of different discharge quantities. Although it is more likely that the assumptions made in computing the prototype were incorrect and the tailwater regulator of the 1 to 68 model of Shasta Dam was not suitable.

A special piece of apparatus (figure 41B), was improvised for recording these data. A plastic disc was attached to the drive shaft of the oscillator of an electric fan which was mounted on the top of a short section of 3½-inch pipe containing a float with a rod supporting a recording pen, capable of being placed in contact with the paper or removed from it as desired. The apparatus was placed in the river channel just below the lip of the bucket about one-fourth the length of the spillway from the left training wall. A clean sheet of paper, cut to the size of the disc, was attached to it with paper cement. The pen was placed in contact with the paper and held firmly near the outer edge of the revolving disc, to form a base circle after which it was retracted. The disc was then set in motion and the contact between the pen and paper made at the instant the flow in the left powerhouse was cut off. The pen was allowed to remain in this position until the crossing of the lines due to surges in the tailrace made the record confusing when it was lifted from the paper to be replaced again just long enough to record the water level at predetermined intervals. The amount of recession of the water surface was obtained by subtracting the radial distance from the base circle to the starting point from that from the base circle to the pen line at any desired interval. The results were plotted in prototype terms (figure 57). A recession of from 3 to 8



GRAND COULEE DAM
HYDRAULIC MODEL EXPERIMENTS 1 TO 60 SCALE
RECESSION OF TAILWATER CAUSED BY
POWER OUTAGE

feet in 40 minutes may be expected for outages reducing the flow by 19,000 and 37,000 second-feet, depending upon the initial and final tailwater levels.

From 50 to 60 percent of the total drop in water surface occurs within the first 12 minutes, and for all practical purposes the change from the normal elevation for the initial to the normal elevation for the remaining flow occurs in the first 40 minutes.

Since the approximate natural stage of the river existed for the normal flow before and after the outages when the tailwater was regulated to give the correct elevation for the initial discharge, the data is believed to be reliable as a guide to establish the rate at which synchronous bypass facilities would have to be placed in operation to prevent the formation of static pressure in the banks causing them to slide.

32. Observations concerning similarity of model and prototype. It is seldom possible to obtain perfect similitude in fluid motion phenomena on hydraulic models; thus in some cases the laws governing it must be accepted with reserve and the model operated to minimize the influence of these factors. As a result, the hydraulic engineer who uses models as a means of obtaining solutions to problems dealing with fluid flow is on the alert for data establishing the limitations of these laws. For this reason, observations of this nature were made on the 1 to 60 model of Grand Coulee Dam during the studies for the development of the repair and maintenance caisson.

The flow conditions of model and prototype were compared for a limited range of discharges many times during the test program, particular note being made of the turbulence, erosion, and surface action downstream from the spillway, for small floods. The model, being near the prototype, afforded an excellent opportunity for comparison in this respect.

In general, the conditions on the model for small flows were relatively smoother than for the prototype. The turbulence and wave action in the tailraces and the river channel below the model were less pronounced than those on the ptototype, and the roll over the spillway

bucket of the prototype seemed to be rougher and higher than that indicated by the model. The erosion of the tailrace slopes by currents and wave action was relatively more severe on the prototype than on the model, the slopes on the prototype being attacked vigorously during a flow of 300,000 second-feet. Some idea of the roughness of flow in the vicinity of the spillway bucket on the full-sized structure may be visualized from the following incident. On one occasion, when Engineer L. V. Downs, of the project field office, and the writer were observing the prototype flow conditions from the bull ring of a crane mounted on the left powerhouse deck above the training wall at elevation 1000, the impact of the roll from the spillway bucket on the training wall was so severe that a heavy sheet of water was thrown onto the crane, drenching both men. Similar roughness was obtained on the model at higher discharges only. The roughness for a model flow representing 500,000 second-feet seemed to be more representative of that for a prototype flow of 300,000 second-feet than did the corresponding model discharge. It is believed that the out-of-scale fluid viscosity and atmospheric pressure of the model influenced materially the model action at small flows and that the influence of these factors became negligible for the higher discharges. From observations on the model, it seemed that the critical point was at a flow representing approximately 400,000 second-feet, for the roll over the spillway bucket became more pronounced when the model flow approached the equivalent of this quantity. From these observations, it was concluded that conditions of turbulence and erosion will be relatively more severe on the prototype than indicated by the model, for small discharges, and that the degree of variation will change with the model scale.

None of the white, air-saturated water so prevalent on the prototype was present on the model. Its absence was attributed, in part, to the out-of-scale viscosity and the atmospheric pressure, and to the fact that under test conditions the model velocities were insufficient to entrain any appreciable amount of air.

The fact that sand and gravel were observed to be in motion on the model in the areas and on the surfaces where erosion had occurred on the

prototype is indication of hydraulic similarity. To what extent is not understood completely since no attempt was made to represent the prototype material to exact scale and it was not known what sizes of prototype material had inflicted the damage. However, if the model action is an indication, rock approaching two feet in size might have been moved about by the prototype flow.

Dissimilarity at small discharges was indicated also when the movement of material into the bucket by unsymmetrical operation of the prototype drum gates could not be demonstrated except by placing fine silt on the model river bed. Failure to obtain correlation in this respect was due, no doubt, to a combination of out-of-scale factors, the complexity of which does not seem to justify discussion in this report. An increase in the model size or the use of a lightweight material would prove advantageous in this type of study. From observations on the 1 to 60 model of the movement of river-bed materials, it seems that any action observed on the model, using the scaled particle size and the same density as for the prototype, is certain to take place on the full-sized structure. However, the model may fail to reveal the action without further refinement in instances similar to those described above.

Another observation which might be considered in the light of similitude was that made to ascertain if the flow conditions existing on the prototype during the 1940 flood season were responsible for the channel eroded through the bucket lip at the joint between construction-block rows 37 and 38. When similar conditions were reproduced on the model (exact similarity not being possible because the model represented the completed stage and not a construction state of the prototype), a small amount of material was observed to move into the bucket, along the bucket invert toward the flow through the bucket, and then out over the bucket lip as it neared this flow. This action, although shown indistinctly by the model, was considered sufficient indication that the greatest part of the eroded channel was formed during the 1940 flood. The shape of the channel curving to the left from the invert to the lip of the bucket seems to offer proof that this was the case.

Perhaps one of the most interesting observations relative to hydraulic similitude, made on the 1 to 60 scale model, was the action noted when sounding equipment was being operated at the left training wall of the full-sized structure. During these operations which were made from a crane mounted on the draft tube bridge deck of the left powerhouse over the training wall, it was found that occasionally the large granite boulder attached to the cable of the crane moved around the wall and upstream into the bucket, adjacent to the wall. Prior to this occurrence, which happened a few weeks before the 1 to 60 model was placed in operation, it was not considered likely that any river-bed material would find its way into the bucket when flows were distributed evenly across the spillway section. A test was made on the model to verify this occurrence and to establish that the action did not result from the handling of the sounding equipment. It was not considered advisable to make investigations on the prototype to determine the limits of this action since the crane was not arranged to sustain large forces in the direction of application when the boulder moved into the bucket. Although the writer did not see the action mentioned above, it was surprising to note how that of the small rock suspended on a string at the left training wall of the model checked with the description given by project engineers directing the operations. For the model tests the rock was lowered to different elevations on the left side and near the end of the left training wall.

When suspended above the level representing elevation 930 of the prototype, the rock was moved around the end of the wall, into the spillway flow adjacent to it, and carried downstream.

Between elevations 910 and 930, approximately, the rock passed around the end of the wall and upstream, adjacent to it, into the bucket. Apparently the current at this level around the end of the wall near its base continued upstream along the wall into the diverging area produced by the intersection of the bucket curve and the inside batter of the training wall, for later investigation disclosed a current upstream into this area.

When lower than elevation 910, the rock was moved around the end of the wall, upstream along the inside to a point a short distance downstream from the bucket, and then toward the center of the spillway section.

The actions below elevation 910 and above elevation 930 were observed by the writer on both the full-sized structure and the model. They appeared to be identical.

33. Investigation concerning protection of tailrace slopes and river banks. In the original plans only the excavated slopes of the powerhouse tailraces below Grand Coulee Dam were to receive a protective armor of riprap. That this protection would not be adequate to prevent severe damage to the tailrace slopes and river banks during large floods was indicated by tests on a 1 to 120 scale model representing the construction stage in which the flood waters were diverted through the west cofferdam area. The indication of inadequacy by this model made it desirable to investigate conditions for large floods over the completed structure, so the study was continued in 1936, using a 1 to 120 scale model of the dam and downstream river channel. A study of the protection provided in the original plans corroborated the results on the diversion model and in addition proved the thickness and extent of the riprap established by this model to be inadequate for larger floods.

In view of the results of this test and the abundance of rock available on the project during construction, the thickness and extent of the riprapped surfaces were increased over that tested on the model. The actual treatment was never tested nor was consideration given to moving the toes or the tailrace slopes bankward (downstream from the training walls) to prevent erosion by the return eddies at the ends of the spillway, or flattening these slopes to give them added protection against wave action. Operation of the spillway and experience with slides in the river banks below the full-sized structure in the past few seasons have served to emphasize the importance of providing adequate protection to these areas, so much so, that as many pertinent tests as time would permit were made on the 1 to 80 model dam during the 1944 test program.

The model arrangement for the first of these tests is shown on (figures 58A and B). The results of these tests, many of which were, of necessity, preliminary in nature, are presented in the following paragraphs. Because of the nature of the data it should not be taken as a solution to the problem, but used as criteria for establishing the limits and characteristics of protection, which should undergo thorough investigation on the 1 to 60 model before being adopted for the prototype. These investigations are now in progress.

Severe damage to the tailrace slopes and river banks below Grand Coulee Dam by waves formed during operation of the spillway at large flows, was indicated by tests made in 1936 on a 1 to 120 scale model of the dam and downstream river channel. The action was described and photographs of the erosion contained in a memorandum to the Chief Designing Engineer: "Limits of Riprap on River Banks Below Tailraces of Grand Coulee Dam", by J. E. Warnock and J. W. Ball, dated August 10, 1936 (Appendix III). The report was submitted to the consulting board for consideration and a copy should be in the project files.

Operation of the 1 to 60 model under conditions comparable to those used in making the tests on the 1 to 120 scale model gave similar results.

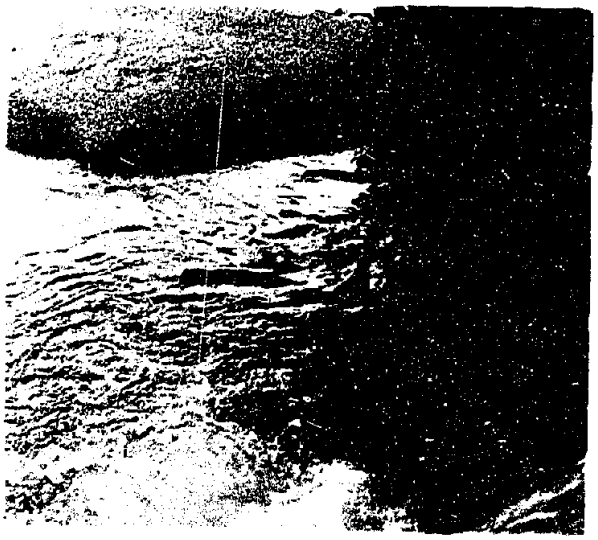
Initial damage to the slopes resulted from wave action (figures 58C and D) which forced water into the riprap blanket, plucking and moving material until the blanket of riprap was ruptured to allow transportation of the smaller particles by currents adjacent to the banks (figures 59A, B, C, and D). During the test, in which the flow was increased by increments to that representing 787,000 second-feet, it was noted that the action was less severe for the flatter slopes (figure 59B) and that the beaching action of the waves flattened the slopes to 8 to 1, approximately.

Using these data as a guide and considering that surfaces riprapped properly should remain on a slope steeper than 8 to 1, two areas, chosen arbitrarily, one in each tailrace, were prepared for tests to obtain some indication as to what slope would be required to resist the destructive action of the waves. An area of the left tailrace bank was rebuilt

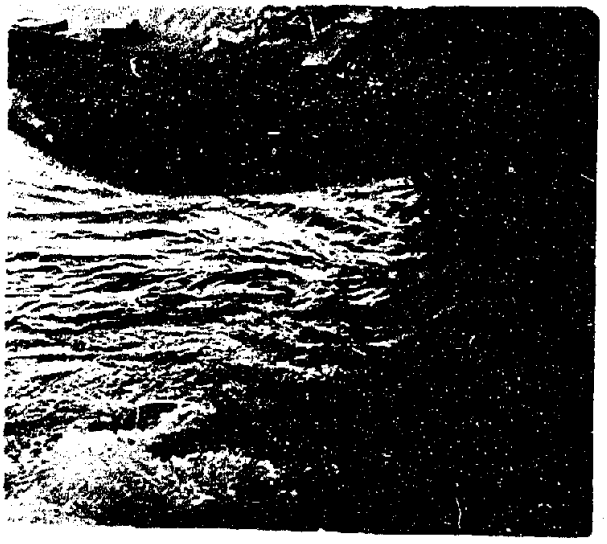
A. Right powerhouse and river bed

B. Left powerhouse and river bed

1 TO 60 MODEL READY FOR TESTS CONCERNING PROTECTION OF TAILRACE SLOPES AND DOWNSTREAM RIVER BANKS



C. Flow representing 537,000 second-feet

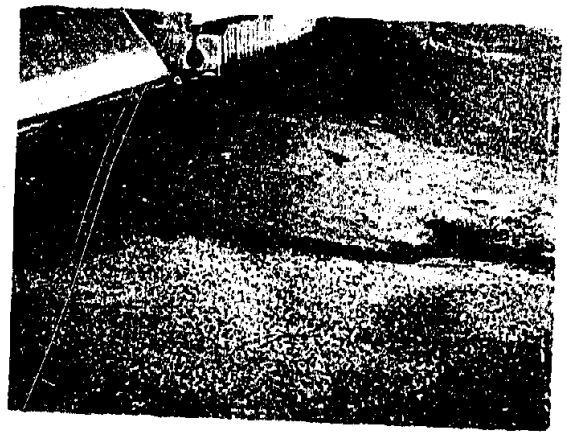


D. Flow representing 787,000 second-feet

WAVE ACTION ON RIGHT TAILRACE SLOPE

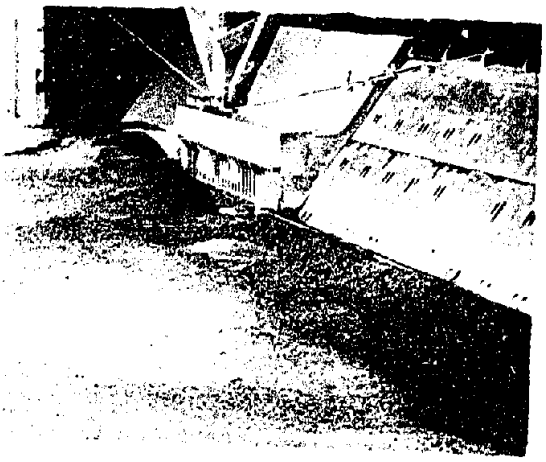
DOWNSTREAM RIVER CHANNEL ON 1 TO 60 MODEL DAM BEFORE TEST AND WAVE ACTION ON RIGHT TAILRACE SLOPE

A. Right bank



B. Left bank

After flow representing 537,000 second-feet



C. Right bank

D. Left bank

After flow representing 787,000 second-feet

E. Flattened right tailrace slope after flow representing 787,000 second-feet

EROSION BY WAVE ACTION OF TAILRACE SLOPES AND DOWNSTREAM RIVER BANKS

on a 6 to 1 slope and covered with gravel, representing 6- to 26-inch rock (figure 60A). The small material was placed adjacent to the excavated surface and the size increased gradually to the surface which was formed with a layer one diameter in depth of material representing 18 to 26-inch rock. The slope of the right tailrace was treated in a similar manner, but using a 4 to 1 instead of a 6 to 1 slope (figure 60C). A flow representing 787,000 second-feet (750,000 over the spillway and 37,000 through the left powerhouse) was chosen arbitrarily for this test since the wave action for this flow was similar and seemed as severe as that for higher discharges. The model was operated at this flow for five hours. Movement on the 4 to 1 slope occurred immediately. Some of the large rocks were rolled down the slope by the waves and there was some settling at the waterline. The cutting of the bank continued throughout the test, but the riprap blanket was never ruptured (figure 60D).

Slight settling only was noted for the 6 to 1 slope, and very slight movement of material occurred during the test (figure 60B). From the test it appeared that slopes of 6 to 1 would be required in areas attacked severely by waves and that the slopes could be steepened as the size of the waves decreased. These data were used in establishing contours for subsequent tests made to study the effect of such protective measures relative to the location of maneuvering facilities for the floating caisson on the right bank.

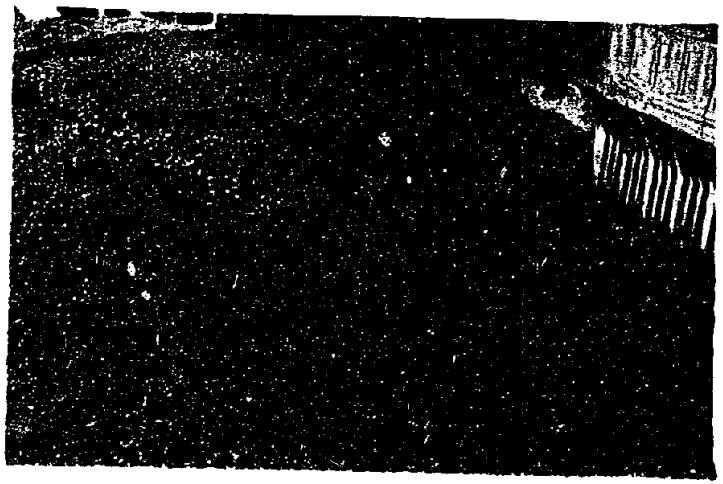
Cutting of the toes of the tailrace slopes directly downstream from the training walls by large spillway discharges was noted during the test program. The condition was more severe at the toe of the left than at the right tailrace slope, since it projected farther into the river and into the current from the spillway flow. Whirling eddies between the outer extremities of the spillway flow and the tailrace banks contained sufficient velocity to move material from and roll it down the slopes where it was moved either upstream or downstream. This action eroded the slope to change it from a convex to a concave surface. Contours of these surfaces were obtained after several hours of operation at flows varying from 70,000 to 787,000 second-feet (figure 61A),

A. Area of left tailrace on 6 to 1 slope before test

B. After flow representing 787,000 sec. ft. for 5 hours

AREA OF LEFT TAILRACE ON 6 TO 1 SLOPE PROTECTED WITH GRADED RIPRAP REPRESENTING 6- TO 26-INCH ROCK

C. Area of right tailrace on 4 to 1 slope before test

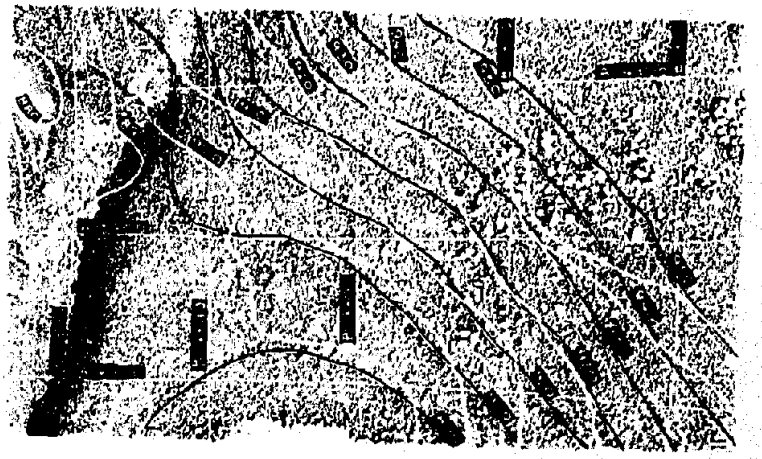


D. After flow representing 787,000 sec. ft. for 5 hours

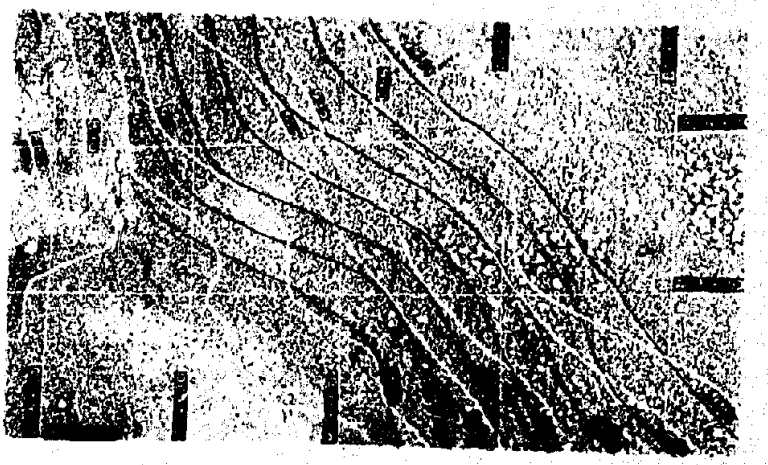
AREA OF RIGHT TAILRACE ON 4 TO 1 SLOPE PROTECTED WITH GRADED RIPRAP REPRESENTING 6- TO 26-INCH ROCK

EROSION BY WAVE ACTION OF 6 TO 1 AND 4 TO 1 SLOPED AREAS OF TAILRACE SLOPES BLANKETED WITH GRADED RIPRAP REPRESENTING 6- TO 26-INCH ROCK

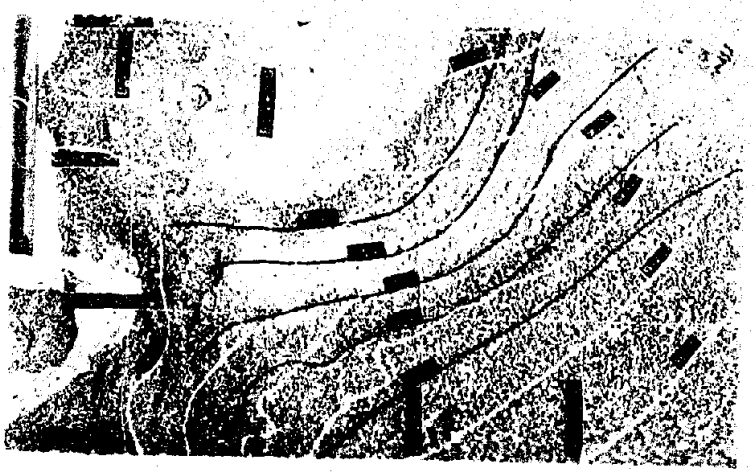
FIGURE 60



A - Left end of spillway after flow representing 787,000 second-feet for 5 hours.



B - Left end of spillway after flow representing 1,000,000, 925,000, 775,000, and 700,000.



C - Right end of spillway after flows representing 1,000,000 925,000, 775,000, and 700,000 second-feet for 1 hour each

CONTOURS OF EROSION BELOW ENDS OF SPILLWAY OF 1 TO 60 MODEL DAM AFTER FLOODS OF VARIOUS MAGNITUDE

and after operating at flows of 1,000,000, 925,000, 775,000, and 700,000 second-feet (approximately) for an hour each (figures 61B and C). In the latter case, the tests were made for calibrating the model supply system and no attempt was made to maintain normal tailwater. As a result, the river elevation downstream was too low. Though the erosion might have been more severe than with normal river elevations, the period of operation was short, and it is believed that the data is sufficiently representative to be used in determining the treatment of the tailrace slopes for additional model tests. The contours of the erosion in the right tailrace obtained in these tests were used in establishing excavation limits of the river bank between the dam and drydock for studies concerning the location of the maneuvering facilities for the floating caisson and protection of the tailrace slopes and river banks below the dam.

Assuming that a 6 to 1 slope of the tailrace surfaces, where the wave action was severe, would be satisfactory for floods of about 750,000 second-feet and that the slope could be steepened as the flow decreased, a surface parabolic in cross section, passing through elevation 902 on a 3 to 1 slope and elevation 1000 on a 6 to 1 slope was chosen for certain areas, and alterations made to the right tailrace surfaces and river bank (plan 3, figure 44). The toe of the tailrace slope downstream from the training wall was moved bankward an amount indicated by the contours obtained from the river after the calibration runs mentioned above, to prevent cutting by the flow from the spillway. The excavation limits adjacent to the upstream side of the approach channel to the drydock were based on those used to study conditions in the vicinity of the drydock after it was discovered that the ridge between the drydock and the river extended into the river much farther than shown in the preliminary plans and the points had been removed. The topography below the drydock was rebuilt (plan 2, figure 44) while that above the drydock was made to conform with the new proposal using the variable slope (plan 3, figure 43).

The flow on the model was increased gradually to represent 787,000 second-feet and allowed to remain at this quantity for five hours, making it comparable with previous tests.

There was considerable beaching action by the waves at first. The action was most severe, however, on the tailrace slope downstream from about the center of the right powerhouse where the surface warped from the section of the parabola to a 3 to 1 slope (figure 52E).

As the test progressed, the beaching action became less severe. This was noticed particularly where the slope approached 6 to 1.

The waves attacked the slope downstream from units 2, 3, and 4 of the right powerhouse severely, and penetrated the 4-1/2-foot blanket of 6- to 26-inch material after about 1-1/2 hours' operation of the model.

The conditions were less severe than with the steeper slopes in previous tests and it seemed that more improvement would result by an additional decrease of the slope to as near 6 to 1 as possible in the critical areas, particularly in the regions where the wave action was most severe.

There was no cutting at the toe of the new tailrace slope downstream from the right training wall. In fact, there seemed to be a tendency for fine material to deposit on the lower portion of the toe in this region.

34. Study relating to the concrete arch from the gate section of the drydock. An arched slab of reinforced concrete was poured in the gate opening of the drydock to give that structure strength and rigidity during the sinking operation. It was necessary to remove and dispose of this concrete segment before the floating caisson could be moved from the drydock to the dam for repair work to the bucket. It was proposed to remove this segment by tipping it with jacks from the gate opening into an excavated area in the entrance channel adjacent to the drydock, where it would remain as part of the floor of the entrance channel.

Under normal construction schedules in which the permanent miter gates would have been available for installation at the appropriate time, the tipping could be accomplished before the entrance channel was cut through to the river and water admitted to the channel. Due to unavoidable changes in schedules, economic considerations, and the urgency of placing the repair caisson in operation at the earliest date possible,

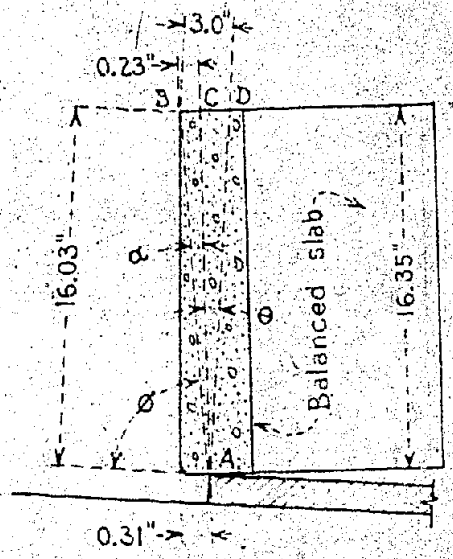
it was desirable to retain the slab as a bulkhead for the purpose of keeping the drydock free of water to permit fabrication of the floating caisson within it, thereby eliminating the necessity for constructing watertight cofferdams across the opening prior to the fabrication of the caisson, and possibly later for installing the miter gates. This plan permitted the completion of the excavation of the entrance channel to the drydock to allow immediate use of the caisson upon its completion but made it necessary to tip the slab in water having a depth equal to the normal stage of the river. It was planned to admit water into the drydock to the same elevation as that in the entrance channel to equalize the pressure on both sides, reducing the force required for the tipping operation.

It was to investigate the feasibility of tipping the gate slab under these conditions that the 1 to 42 model of the drydock and entrance channel was constructed. The tests performed were mainly of a qualitative nature concerning the behavior of the arched slab during the tipping operation, with the water surrounding it at elevations within the range expected. Since the location of the center of gravity, the weight, and the force required to accomplish the tipping of the slab were important factors relating to the tipping operation, data concerning these were taken.

The tests were performed with the use of two models of the arched slab, the second being constructed of a concrete plaster mortar reinforced with strands of heavy wire after the first, of unreinforced cement-plaster, was damaged beyond repair when it was tipped onto a flat bench of dry sand in front of the drydock model. Although the feasibility of tipping into the water was indicated by tests performed on the first model before it was broken, more detailed records were desired, so the second was constructed. Moreover, the first was not constructed to the proper weight scale, N^3 where N is the model scale, but was too light. The second, somewhat heavier than the first, was still too light to meet the weight requirement for similarity. However, this out-of-scale factor was considered unimportant, since the lighter model would give results on the safe side. With the lighter weight,

the resistance against sliding would be less and it would come to rest nearer the drydock when tipped than one constructed with the correct weight ratio.

The position of the center of gravity of the model was found by tipping it over a straight edge to a balanced position and measuring the angle of inclination, first, right side up, then upside down. Equations were developed for these conditions, and a simultaneous solution gave the location of the center of gravity. The procedure was as follows:



From figure 62

$$AB = \sqrt{16.03^2 + 3.00^2} = 16.31$$

$$\sin \alpha = \frac{0.23}{16.31} = 0.01410$$

$$\alpha = 0^\circ\ 49'$$

$$\tan \phi = \frac{16.03}{3.00} = 5.34333$$

$$\phi = 79^\circ\ 24'$$

$$\text{But } \theta = 90 - \phi - \alpha = 9^\circ\ 47'$$

Figure 62-Center of gravity of arched gate slab of drydock by balancing.

The angle for the upside-down position with the fulcrum 0.31 inches back of the outer edge, determined in the same manner, was 8° 41'. Supplying coordinates for the two sloping lines AD and A'D', the center of gravity is at their intersection (figure 63).

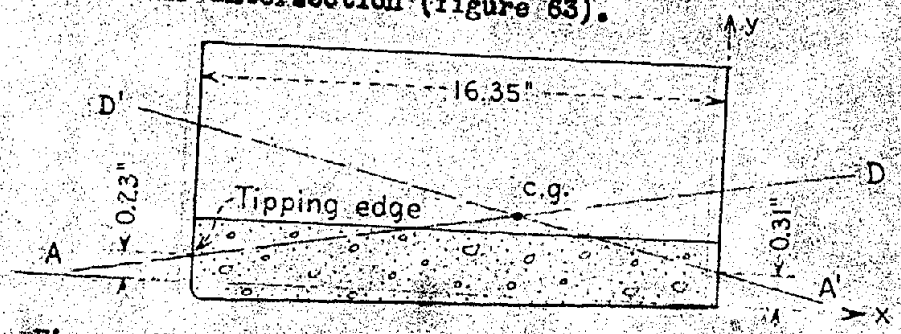


Figure 63 - Graphical location of center of gravity of arched gate of drydock

From the figure, the equations for the lines are:

$$(AD) \qquad y = 0.1724x + 3.05$$

$$(A'D') \qquad y = -0.1527x + 0.31$$

Subtracting the second from the first

$$0.3251x + 2.74 = 0$$

$$x = -8.43$$

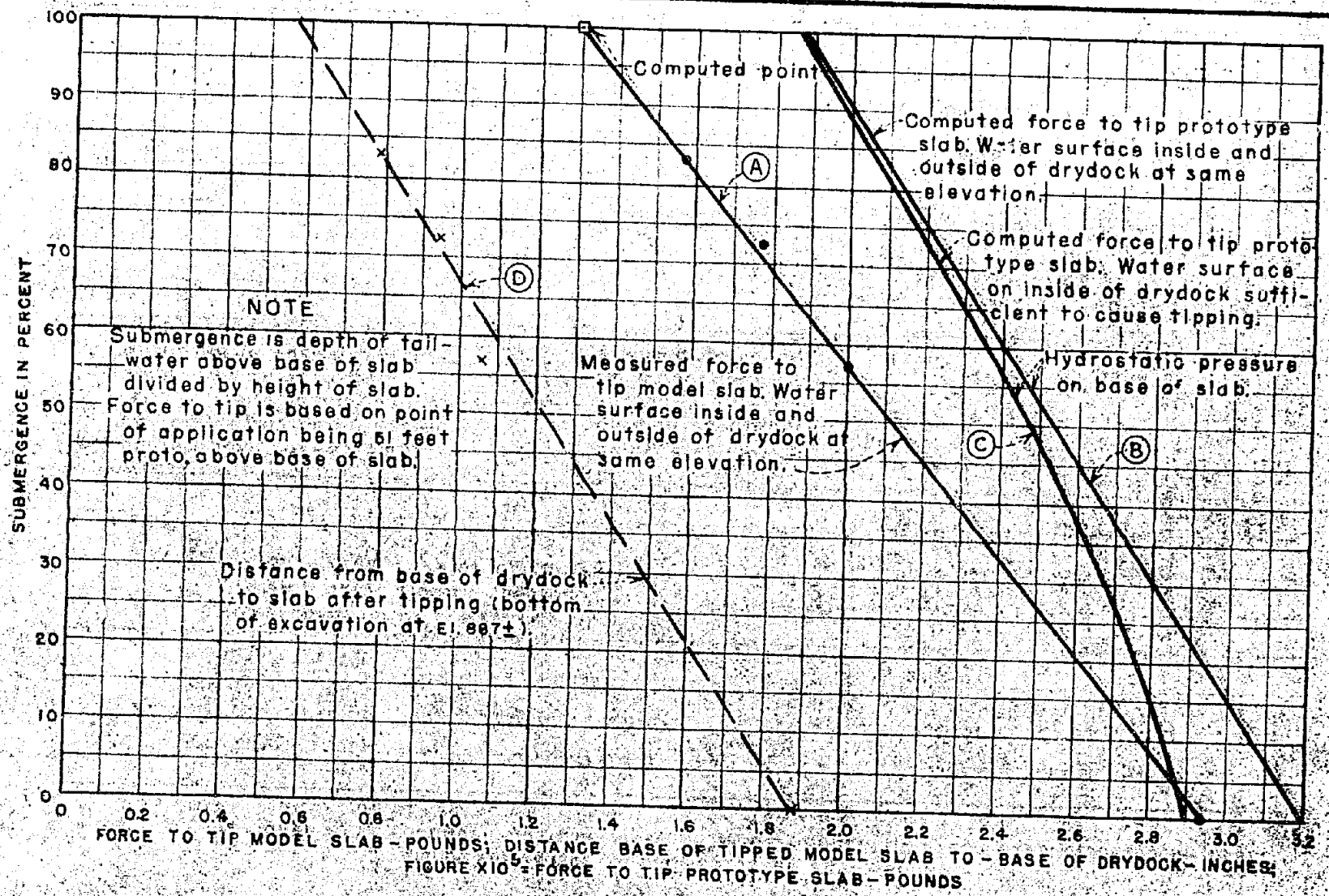
$$y = 1.60$$

The center of gravity, was therefore, 16.35 - 8.43 or 7.92 inches above the base, and 1.60 - 0.23 or 1.37 inches back of the tipping edge; or 27.7 and 4.80 feet, respectively, for the prototype. This checked closely the computed values of 28.3 and 4.72 feet, respectively.

The force required to tip the slab was determined by use of the system of lines and pulleys described under section 12 of this report. The weight of the model was recorded to the nearest ounce. Lead shot was added slowly to the pail suspended on the ends of the lightweight fishline fastened to to the top of the arched slab until sufficient force was obtained to start the tipping and keep the slab in motion. The total weight of bucket and lead was recorded. These data were obtained for a condition where the slab was not surrounded with water and for the surrounding water at levels representing elevations 935, 944, and 950.

The force to tip for complete submergence was computed using the location of the center of gravity found as described above. Plotting this value and those obtained from the model for various degrees of submergence against percentage of submergence (percentage of submergence being defined as the distance of the water surface above the base of the slab divided by the total height of the slab) resulted in a straight line (figure 64A).

It is reasonable to assume that the force necessary to tip the prototype structure varies similarly. Therefore, using the position of the center of gravity obtained from the physical properties of the prototype, the force required to tip the slab in the dry and that required to tip under a condition of complete submergence was computed.



FIGURE

GRAND COULEE DAM

FORCE TO TIP DRYDOCK GATE SLAB AND DISTANCE GATE SLAB TO DRYDOCK BASE AFTER TIPPING

HYDRAULIC MODEL STUDIES - SCALE 1:42

the values plotted and a straight line drawn through the points (figure 64B). From this straight-line relationship, the force for any intermediate elevation of the water surface can be obtained assuming that no bond exists between the arch and the drydock proper.

When making the tests to ascertain the force required for tipping, observations were made of the behavior of the model in its fall to the bottom of the channel. The distance between it and the drydock structure after it had been tipped into the entrance channel, which varied with the degree of submergence, was recorded in each case (figure 65). A plot of this distance against the percentage of submergence gave a straight line (figure 64D). For the dry condition and small degrees of submergence the fall was rapid. The action became slower as the depth of water increased. This characteristic together with the fact that the shallower the water the farther the distance between the tipped slab and the edge of the drydock, indicates that for the same degree of submergence the heavier the structure the farther will be this distance. The inertia force will be greater. Since the tipping operation proved satisfactory on the model with a slab that was too light, it seems reasonable that the operation would be equally as satisfactory with a model that more nearly represented that of the prototype structure and that conditions on the prototype might be improved from those indicated by the model. Some disturbance, with the possible formation of turbulent currents, bores and waves in the immediate vicinity that might prove dangerous for small boats, will be present as the slab descends.

As the water elevation was increased on the model the slab seemed to pause for an instant just after its center of gravity passed beyond the tipping edge. This characteristic was not noticeable with small submergences but was evident with the water at the higher elevations. It is believed that a heavier structure than that used for these tests would have caused the action to be less pronounced in each case.

During the test program the question arose as to what would happen should the water surface in the drydock be raised above that in the river for the purpose of breaking any bond at the joints, aiding the jacks in breaking it, or making use of such a method to accomplish

Water surrounding gate slab at start of tipping operation

Model slab in position after tipping with surrounding water at level representing elevation 935.0

Model slab in position after tipping in dry.

Arched gate slab of 1 to 42 drydock model ready for tipping and in position on entrance channel floor after being tipped.

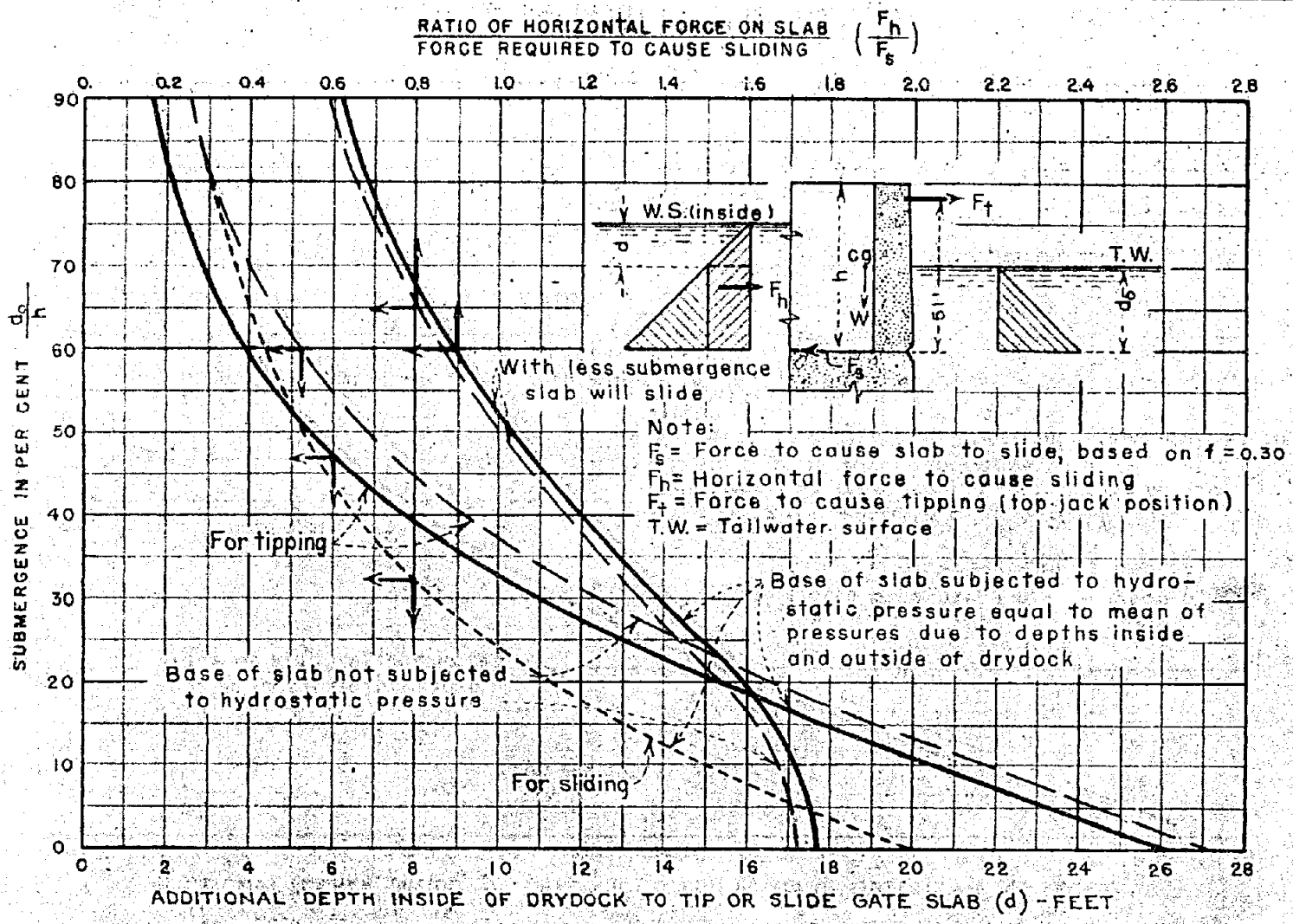
the tipping of the slab. Tests in which a small pump was used to raise the water surface in the drydock above the normal elevation of that in the channel were made to study the feasibility of this method. When the flow into the drydock was small and the slab tipped outward about the tipping edge until sufficient water had flowed from the drydock to reduce the force against its inside surface, the structure did not return to its original position but moved back and forth as if hinged about the tipping edge. This action was caused mainly by irregularities on the base of the model slab or on the floor of the drydock gate section and would not take place on the prototype. Once started the prototype slab should move out to complete the tipping operation unless the pressure variations due to wave action are sufficient to start the oscillating motion. A substantial increase in the inflow to the model drydock to overcome the effect of flow outward through the joints, resulted in the slab being tipped quickly.

The model slab was held in place until the water surface inside the drydock was higher than that on the outside by as much as 2 inches (model) to represent a condition wherein a bond existed between the slab and the surfaces of the gate opening. When the additional depth inside was small, the tipping action occurred quickly with no perceptible sliding. As the additional depth increased there appeared to be a slight sliding action before the tipping occurred. The distance between the edge of the drydock and the base of the slab after the slab had been tipped decreased as the additional depth inside the drydock increased, possibly due to this sliding action. The cases in which no sliding was apparent were considered to represent those wherein the additional depth inside the prototype drydock would not produce sufficient horizontal forces to slide the structure before the bond was broken. Those where sliding was apparent were considered cases in which the additional depth was sufficient to cause sliding at the time the bond was broken. Computations were made to investigate this presumption. The additional depth within the drydock to balance the arched section such that any additional outward force would cause it to tip (force to tip), the additional depth to induce sliding, the horizontal force tending to

slide the slab, and the force required to induce sliding for various degrees of submergence were determined.

The additional depth to cause tipping, that to induce sliding, and the ratio of the horizontal force tending to induce sliding to that required to produce sliding, were plotted against percent submergence for conditions with and without hydrostatic pressure developed on the base of the slab (figure 66). The reason for the two actions noted on the model is shown clearly by these data. For submergences having a ratio of horizontal force tending to induce sliding to that required to produce sliding greater than unity, sliding will occur before tipping. When the ratio is less than unity tipping will occur before sliding. The change from one condition to the other occurs at a submergence of approximately 54 percent when a friction coefficient (against sliding) of 0.3 is used and it is assumed that hydrostatic pressure equal to the mean of the pressures inside and outside the drydock is developed on the base of the slab. The additional depth to tip the slab and that to induce sliding are equal when the tailwater is at elevation 934 (approximately 54 percent submergence). With the tailwater below this elevation the depth to tip exceeds that necessary to induce sliding, while above this level the conditions are reversed. The difference between the additional head to tip and that to induce sliding is very small for levels above elevation 934. With the tailwater at elevation 944, the additional depth to cause tipping is 2.6 feet, while that to induce sliding is 3.4 feet, or a difference of 0.8 foot. The depths for a tailwater elevation of 950 are 1.9 and 2.8 feet respectively or a difference of 0.9 foot.

In view of the uncertain friction coefficient, the unpredictable dynamic forces, and the characteristics shown by figure 66, the procedure of filling inside the drydock, either to break the bond between the slab and the drydock, and aid jacks in breaking the bond or tipping the slab, or to tip the slab from the gate opening, is not recommended.



GRAND COULEE DAM
DEPTH OF WATER IN DRYDOCK TO TIP GATE SLAB
EFFECT OF SUBMERGENCE ON SLIDING

FIGURE 66

V. CONCLUSIONS AND RECOMMENDATIONS

35. Operation of 1 to 60 model dam. The flow nozzles, with individual control gates, used in the right powerhouse of the 1 to 60 model to represent the bypass outlets were superior to the cylinder valves controlling the left powerhouse, thus this type of control is recommended for both structures if the model is to be used for public demonstration.

From data taken concerning the jet pump used in the initial studies on the model dam, it was concluded that this type of pump, though noisy, is a very satisfactory means of obtaining large quantities under low heads from high-head pumping equipment. The capacity of this pump, the design of which was based on the momentum theory, with a ratio of total to driving quantity of 2.7, indicates that the momentum theory may be used to advantage in investigating the adequacy of jet pump designs. The 24-inch vertical propeller pump, installed when it was desired to make tests using flows in excess of the capacity of the jet pump, will provide the quantity representing the design capacity of Grand Coulee Dam only if the storage reservoir is filled to maintain the minimum head on the pump after the model is in operation. Flows approaching that representing 900,000 second-feet are obtainable without raising the water level in the storage reservoir under the model dam.

Accurate measurement of the wide range of discharges required in the model investigation was made possible by the installation of two branches in the supply system, using the jet pump and three branches in the case of the 24-inch vertical propeller pump. An 8-inch venturi meter in the 8-inch branch and pitot tubes in the 14- and 24-inch branches proved satisfactory as recording devices.

The water entering the model reservoir was free of undesirable turbulence for all flows. The 21 diverging rectangular openings in the distributing conduit and the baffles placed over and three inches from them gave very good distribution.

The careful reproduction of the river channel contours and the proper selection of surface materials for the movable and stable

sections of the model river channel made it possible to establish water surface gradients on the model which were in good agreement with those for equivalent flows on the prototype without the usual extensive testing. That the velocity distribution in the model river channel below the dam was reasonably correct is indicated by the agreement between the water-surface gradients of the model and prototype.

Drom data obtained concerning the operation of the tailwater regulating gate in the downstream sump of the model, it was determined that automatic control of the river stage could not be attained by altering the slide gate but that more extensive changes to the control apparatus would be necessary.

The air-saturated condition so prevalent on the prototype structure could not be represented satisfactoryily by the air-entrainment device installed in the model. Roughening the spillway surface would assist to produce the white appearance but it is considered impracticable to provide a roughness which would give satisfactory results for the entire flow range.

The torsion dynamometers used to measure the pull on the maneuvering lines of the model caisson to transfer it to and from the dam were suited excellently for that purpose.

36. Bypass outlets in right powerhouse. From the tests concerning the bypass outlets in the right powerhouse it was concluded that there would be no damage to the tailrace slope due to currents from the draft tubes into which the 84-inch needle valves discharged.

A current of relatively high velocity from the outlet flow will be formed along the tailrace slope and continue across the entrance channel of the drydock. It will be necessary to navigate this current in transfering the floating caisson to and from the dam for repairing the spillway bucket. Its presence was the main factor necessitating the use of a fifth line for maneuvering the caisson. The velocity and extent of this current varied for different valve combinations, but the change was not sufficient to influence the maneuvering operation when more than two valves were discharging.

Though there will be eddy currents moving upstream to the face of the spillway section of the dam and then bankward when there is flow

from both powerhouses, it was believed that the velocities will be low and that little or no transportation of river fill material will result. Inasmuch as the river fill is to be removed from the channel below the bucket to allay damage by the movement of river-bed materials during normal operation of the spillway, no detrimental action should result from this source after this operation has been completed.

In view of the model results it seems that the turbulence in the tailrace below the right powerhouse due to the operation of the bypass outlets should not exceed materially that which would result from the normal operation of the units in the left powerhouse.

37. Resistance of caisson to currents. That the force to hold the floating caisson head-on into a current of a given velocity will be less than for the broadside condition is shown by data obtained from the model for both conditions. In view of this, it is recommended that the floating caisson be held head-on into the current whenever possible during the operations of maneuvering it from the drydock to the dam and return, particularly when it is moved through the current from the right powerhouse.

Because of the high position of the line hitches to the caisson, it is possible, when holding it broadside against high-velocity currents, to produce sufficient list to cause water to flow over the upstream side girder and enter the space between the barge and the caisson with the possibility of overloading and damaging the 12-part lines between the barge and caisson or breaking the maneuvering lines. The caisson should not be maneuvered in such velocities unless it is kept headed into the current and there is every assurance that the maneuvering lines are adequate.

The use of 1.5 for the coefficient C in $P = CAV^2$ for the broadside condition will give forces in excess of those measured on the model for velocities of less than 7 feet per second. The model data did not include velocities much in excess of 8 feet per second and measurements with these velocities were difficult, thus the data for velocities of this magnitude are considered unreliable. For velocities to 8 feet per second for the head-on condition a coefficient of 0.65 will give forces that are identical to the model results for all practical purposes.

38. Maneuvering lines for caisson. Five maneuvering lines must be provided to transfer the caisson to and from the dam if the line forces are to be kept within reasonable limits and satisfactory latitude in the maneuvering path obtained. It would be possible to maneuver the caisson to and from the river channel a short distance below the dam using four lines, but the operation would require so many limitations to hold the line pull within reasonable limits that it was not considered advisable to operate in this manner. Moreover, a fifth line would be required to move the caisson from this position to the face of the dam to eliminate excess stresses in the lines and larger cables would be required for emergency cases if the procedure were not adhered to strictly.

The 1 3/4- and 2-inch lines based on the model forces measured by the torsion dynamometers, should prove adequate for maneuvering the caisson under all conditions in which the flow does not exceed the capacity of the two powerhouses. Care should be taken, however, to prevent the lines pulling directly against one another as it would be possible to produce excessive stresses in this manner and break the lines.

Though the shore-end locations of the various lines to give optimum maneuvering were:

Station A20+00, B3+00, for line 1;
The right end of the draft tube bridge deck of the left powerhouse, for line 2;
The face of the dam at approximately elevation 955, 610 feet inside the right training wall, for line 3;
On the center line of the gate opening at the back of the drydock, for line 4; and
At station A33+00, B12+00, for line 5.

Some variation in any of the positions should not produce an appreciable change in the maneuvering qualities. However, substantial changes, 50 feet or more, might impose additional limitations on the maneuvering operations, such as restricting the maneuvering to a definite path to prevent overstressing certain of the maneuvering lines or reducing the maximum river flow through which the maneuvering could be accomplished safely. The transfer of the shore end of line 4 to the quoin post on the upstream side of the drydock gate section prevented the use of

this line for maneuvering the caisson in and out of the drydock, but had very little influence on the maneuverability on the model for any of the three shore-end positions of line 2 used. Improvement in the maneuvering of the caisson in the entrance channel resulted from placing the drydock gate section on a line between the center of the drydock and the shore end of line 2 at the left end of the left powerhouse.

Though maneuvering of the caisson seemed easier with all lines attached to the front or rear center of the caisson, the transfer of lines 5 and 1 to the corners made no appreciable change in the maneuverability. Maneuvering of the caisson in the vicinity of the drydock is accomplished with the minimum difficulty when line 2 from the draft tube bridge deck of the left powerhouse is attached to the front center of the caisson. The transfer of this line to the upstream left corner of the caisson to prevent its contacting the upstream end of the side girder when the caisson nears the dam, is not recommended since it makes the handling of the caisson in the entrance channel more difficult, tending to cause it to move toward the upstream side of the entrance channel.

It should be possible to move the caisson across the current from the outlets in the right powerhouse, either in the upstream or downstream direction, by a ferrying action if the maneuvering lines are manipulated properly.

Best conditions for maneuvering from the drydock to the dam should be encountered if the caisson is taken from the drydock toward station A28+00, B8+00, or downstream from it, then headed into the current from the right powerhouse, ferried across it to the river below the center of the spillway, and then moved upstream to the face of the dam directly downstream from the shore end of line 3. For the return trip, the maneuvering should be facilitated by moving the caisson downstream along station A25+00 (approximately) to about station B10+00, heading it into the current from the right powerhouse and ferrying it across through station A28+00, B8+00 (approximately), then turning and towing it backward into the fenders leading to the drydock. Additional

studies, using auxiliary lines to handle the caisson when it is in the fenders or the drydock, might give useful information in the procedure for handling these lines.

The tendency of the caisson to be carried toward the face of the dam by currents in the river channel is so slight that little difficulty should be experienced in preventing it from contacting the concrete surface when it is being moved from one setting to another. This action might be prevented by using a short line attached from the downstream end of the caisson to the work barge or a suitable motor launch. Relatively low forces will be required in moving the caisson transversely from point to point in the bucket when the entire flow is through the powerhouses.

39. Excavation adjacent to drydock entrance channel. The excavation of the points on both sides of the entrance channel to the drydock thereby shortening the length of this channel, will improve the maneuverability of the caisson and eliminate the necessity of providing extensive protection on the banks for lines 1, 3, 4, and 5.

Since there is little use for line 5 in the entrance channel when the caisson is moving in or out, the protection for this line might be reduced further by placing an open sheave on the shore or making some arrangement to prevent its dragging over the point between the channel and the main sheave.

The removal of these points also decreases the tendency for material to deposit in the entrance channel to the drydock. This deposit will be decreased as the material is removed from below the bucket by dredging. Because of the lightweight material used in making the tests concerning the deposit in the approach channel, the amount deposited may be exaggerated over that which might be expected on the prototype. However, this may not be true since the model was operated for such short periods. It is reasonable to assume that the same action will exist on the prototype and that some material will be deposited making dredging of the channel necessary after a large seasonal run off.

Removal of the point extending into the river on the downstream side of the entrance channel will minimize the formation of undesirable

eddies along the right bank below and have little or no effect on the velocity and direction of currents along the left bank opposite the drydock.

The danger of grounding the floating caisson and damaging the seal would be lessened by the removal of these points. It seems desirable to provide adequate marking of the channel for the manouvering operation in either case.

40. Excess flow over spillway. Except for an undesirable eddy below the right end of the spillway, when the flow in excess of the left powerhouse passes over gates at the left end of the spillway, instead of through the right powerhouse, conditions for maneuvering the caisson are as favorable as with the entire flow distributed through both powerhouses. The current in which the caisson must be maneuvered in this case is in the upstream direction, and the fifth maneuvering line is not so essential.

41. Operation of drum gates and river outlets with caisson on bucket. Insofar as hydraulic conditions are concerned, it will be possible, but not desirable, to operate the river outlets when the caisson is in place for repair work on the bucket. If operation of these features is ever necessary due to the necessity of releasing water in excess of the powerhouse capacity, during the low water season, the flow should be kept as remote as possible from the repair operations. It will be possible in case of necessity to pass water over some of the drum gates when repair work is being performed on the bucket. In this case too, the flow should be kept as far away from operations as possible. Conditions are less severe than for the river outlets. Although the action on the model seemed satisfactory with the caisson the equivalent of 50 feet from the flow, it is recommended that the minimum distance be 150 feet as there is a tendency for the prototype flow to spread relatively more than on the model. Some idea as to the relative velocities in which the repair operations would have to be performed under emergency operation of the spillway or river outlets may be obtained from table III of this report, which lists the velocities for various combinations of outlets and drum gates with the caisson at different positions on the bucket.

42. Stability and operation of floating caisson. The deficiency in draft of the 1 to 60 plastic model of the floating caisson in the initial tests was not the result of the increased buoyancy induced by the out-of-scale thickness of the plastic sheet used in its construction but to the fact that it had been constructed according to an earlier design in which the total weight was less than that of the later design. The model was corrected satisfactorily for the maneuvering tests, but was inadequate for tests concerning the stability and operation of the caisson. The model caisson was so small as to be too sensitive to outside influences, preventing accurate interpretation of test results. Proper manipulation of the operating barge of the model was not practicable on such a small scale. Satisfactory results concerning the stability and operation of the caisson could not have been obtained without the use of the larger and more complete 1 to 20 scale model.

Proper venting of the working space of the floating caisson is important since air trapped in the top of it will produce a detrimental effect on the stability of the structure, causing it to list or preventing proper trimming and making the sinking and raising operation more difficult. The vent system of the manifold type adopted for the prototype, which is similar to that found satisfactory on the model, should prove ample.

The upstream end of the caisson should be lowered into the bucket ahead of the downstream end to prevent the seal on the bottom of the side girders from contacting the rough concrete surfaces and to prevent the side girders from "hanging-up" on the bucket lip to damage the seal or tear the anchor frame from the face of the dam. The downstream end should never be kept level (top of side girders level) or allowed to incline downward from the front end when the caisson is being placed in the bucket.

The floating compartments below the barge deck and in the side girders should be kept reasonably free of water to prevent listing of the structure during the skinking, raising, and floating operation. The arrangement of these compartments and the manner in which the

barge is attached to the caisson serves to stabilize the caisson when it is tipped due to the shifting of the center of buoyancy to counteract such movements. The caisson will not be overturned easily because of this characteristic when the trim and floating compartments are handled properly.

The operating barge should never be raised above the water surface in the barge well when water is being admitted during the sinking operation, nor should the water be filled to such depth between the barge and caisson as to overload the 12-part hoisting lines. In pumping water from between the barge and caisson during the refloating of the caisson, care should be taken to keep the bottom of the barge in contact with the water in the well to prevent shifting of the water load, for any slight tipping will cause listing when this condition exists. A minimum draft of one foot is recommended. In raising the tops of the side girders above the tailwater during the refloating operation, care should be taken to keep the loads on the 12-part lines within safe limits by controlling the draft. (Do not force the barge too far into the well in the caisson to remove water from it.)

If the floating compartments are kept reasonably free of water and the structure is trimmed level or with the upstream end low during the sinking and raising operations the caisson should be stable at all times during the floating, sinking, and raising operations.

Sinking or raising the caisson with all trim compartments free of water is not recommended as the stability is a minimum for this condition.

From the stability tests it seems feasible to move the caisson from one position on the bucket to another without refloating it completely.

Indications were that considerable weight might be placed on the deck of the operating barge or caisson before the stability would be changed materially. A weight equivalent to 64 tons prototype placed at the top of the access shafts of the model was required to produce instability.

The freeboard of the caisson, when floating and trimmed level, should be approximately 8.0 feet.

Accurate location of the center of gravity of an object experimentally by weighting, using two supports, is possible when the object can be supported in two of the three planes and its inclination in these planes varied. Location in two planes is possible when the object can be supported in one plane only and the inclination in this plane varied. It is much simpler, however, to determine the exact location when it is possible to support and weight the object in all three planes. One weighing to determine the reactions in each plane is all that is required for this condition.

Because of the good agreement between experimental values of freeboard, the position of the center of gravity, the metacentric height, and the distance from the metacenter to the center of bouyancy, determined from the model and those computed for the prototype, the results obtained from the operation of the 1 to 20 model caisson are accepted with the utmost confidence.

43. Source of damage to spillway bucket. Though most of the damage to the spillway bucket is believed to have started during the construction period when extreme concentrations of flow existed, many of the conditions causing it are present under normal operation. Except for a few small areas reported by divers as having the appearance of pitting due to cavitation, the damage is considered the result of abrasive action of materials being moved about by currents or eddies.

Though the greatest portion of the channel eroded through the bucket lip between construction block rows 37 and 38 is believed to have occurred during the 1940 flood season, it seems certain that erosion be materials moving in the damaged area continues under normal operation of the spillway. The excessive erosion of the bucket between the two banks of river outlets, construction block rows 52, 53, and 54, has resulted from the movement of sand and gravel in this area by eddies formed during the operation of the adjacent pairs of outlet conduits. The greater distance between adjacent outlets at this point permits the formation of more severe eddy currents, thus the increased erosion.

Erosion on the slope downstream from the bucket lip has resulted from the continuous movement of river-bed material in this region during the operation of the spillway and river outlets. Is is believed that once the bucket surface upstream from the lip is eroded, the conditions producing the erosion exist under normal operation. Eddies forming in these eroded pockets whirl materials entering them to incur further damage.

Erosion of the training walls has resulted from the movement of materials in eddies forming in and near the damaged areas.

The erosion of the downstream face of the dam below elevation 945 (approximately), is due to the impact and movement of smaller materials picked from below the bucket lip and carried to this part of the structure by the bucket roll.

Damage near the center of the spillway is greater than at the ends, both on the bucket lip and upstream from it, since it is characteristic of the spillway to move the river fill below the bucket from the ends toward the center of the spillway section where it accumulates and is kept in motion almost continuously. Removal of the source of the damage to the various parts of the bucket and training walls by dredging material from the river bed to prevent its entering the eddies or currents in these areas seems the only practicable method of eliminating or allaying the damage. Without treatment of this nature, the repair work will be a continuous operation.

44. Flow distribution for optimum diving conditions. With the river flow confined to the spillway and the left powerhouse, the maximum discharge giving satisfactory conditions for the inspection of the bucket by divers will be about54,000 second-feet. Optimum flow conditions for diving operations will be present under these conditions when the spillway flow is concentrated at the end opposite that at which the divers are working and the flow is distributed unequally over three, four, or more gates, with their elevations higher progressively toward the center of the structure. Increasing the number of gates operating will decrease the turbulence but move it closer to the diving operations, which may or may not be desirable, a fact

ascertainable only during diving operations. Conditions will be worse when the entire flow is on one side of the river since the eddy below the opposite end of the spillway will be more severe.

The whirl present at the unconfined edge of the spillway flow when only part of the gates are discharging will become more severe as the discharge increases, with the possibility that the smaller material on the river bed will be transported into and swept from the bucket. Removal of the river fill downstream from the bucket should alleviate this condition.

That less difficulty was experienced in the diving operations conducted in the fall of 1943 than those made during the spring, is attributed in part to the more favorable flow conditions resulting from operating the drum gates in accordance with the model findings.

45. Source of material in bucket. Though the tests to ascertain the source of the sand and gravel found in the spillway bucket during the 1943 fall inspection did not indicate the action clearly, it is believed that a portion of the material was carried into the bucket by currents existing below the spillway during unsymmetrical operation of the gates when the diving operations were started and that the remainder was deposited during low flows by conditions which dislodged material from the lip where it had accumulated during larger floods.

Dredging of the river fill downstream from the bucket lip in accordance with the model tests discussed in section 30 of this report should eliminate, in part at least, the objection to unsymmetrical operation of the drum gates. It is believed, however, that operation in this manner should be avoided.

46. Flow and erosion at ends of training walls. Flow conditions at the ends of the spillway training walls which pick up material, transport it from the powerhouse side to the spillway side of the walls and whirl it about violently below the bucket lip adjacent to the walls are responsible for the damage incurred by these parts of the structure. The difference in the nature of the damage to the two walls is attributed to the operation of the left powerhouse and to the extremely severe eddy esisting at the right wall when the river outlets at that

end of the spillway operate. Though some success in correcting these conditions to prevent further damage to the walls was indicated through the use of wall extensions on the model, the difficulty and cost of construction made their use prohibitive, particularly since the material is to be dredged from the contributing areas as a general protective measure. To eliminate the whirl on the side of the right wall below the bucket lip would require an extension of considerable size to the present wall, the construction of which would be difficult and costly. It is possible by extending and curving the training walls outward to eliminate the whirling eddies forming adjacent to the walls and downstream from the bucket lip. The construction of a concrete block in the crevice at the end of the right wall is not recommended nor is the placing of a concrete cap on the river bottom near the end of the left wall.

From the tests concerning the conditions damaging the training walls it seems that consideration should be given to repairing the right training wall and to protecting it against further damage. Placing the concrete by the Prepakt or similar method might prove advantageous in making the repairs. The best method of protection seems to be that of removing the material from the vicinity by dredging. The removal of this material would prevent its reaching the whirling eddies to cause damage.

47. Dredging of river fill to allay damage to bucket. Damage to the spillway-bucket surfaces was the result of abrasive action caused by the movement of rock and gravel by currents in the vicinity of the damaged areas. The only practicable method of eliminating further damage would be to prevent material from entering or moving about in these eddies. This might be accomplished by bonding all materials for some distance downstream, or by dredging to remove material from regions on the river bottom from which it might be carried into the eddies, or a combination of both. Dredging the river fill from the channel below the spillway and adjacent to the training walls seems the more practicable treatment.

Due to the severe cutting of the tailrace slopes by the spillway flow, particularly the left, it is believed that further tests to

determine the final limits for dredging the river fill in these regions should not be performed until some plan has been adopted for protecting the tailraces and river banks. Both are interdependent and the two studies should be carried on simultaneously after the model river channel on the left side below the dam has been extended for the protective studies. In the meantime the results of the tests made thus far can be used to establish tentative boundaries for the river dredging. The limits for dredging indicated by the model are as follows:

For 300,000 second-feet, a line defining the limits should intersect station A14+20 at 100 feet downstream from the bucket lip, continue to a point 220 feet downstream from the lip at station A16+50, for the same distance downstream to station A28+50, and then upstream to intersect station A30+70, 100 feet downstream from the bucket lip. Treatment outside the training walls will depend on the method used in protecting the tailrace slopes and river banks downstream and should be determined during tests which are planned for 1945.

For 500,000 second-feet the limits should be defined by a line intersecting station A14+20, at a point 100 feet downstream from the bucket lip, and continuing to station A15+40, 260 feet downstream from the lip, then station A29+50 at the same distance, and upstream to intersect station A30+70, 100 feet downstream from the bucket lip. Treatment adjacent to training walls to be established as for 300,000 second-feet.

For flows in excess of 500,000 second-feet, it is believed more practicable and economical to do whatever dredging appears necessary below the bucket after such floods have occurred. In this case, the limits would be as defined for 500,000 second-feet.

During the tests there was indication that the river fill should be removed to bedrock in the area dredged. However, this may not be necessary depending on the nature of the material. If after dredging to elevation 885 the rock covering consists mainly of clay or large boulders (2 feet or more in diameter) it is believed that such excavation would not be necessary and should not be done unless operation of the prototype indicated it to be desirable. Should most of the

material be small rock, it is almost certain to be carried upstream to the bucket. In this case dredging of the material after it had moved upstream to the toe of the bucket might prove advantageous. Removal of the high mound below the left end of the spillway would decrease the amount of material moving upstream to the bucket during large floods, but this does not seem justified except to improve conditions in the river channel and improvement was not apparent on the model. Because of the tendency for material below the bucket to move toward the center of the river during operation of the spillway, it is believed that dredging should begin at the ends and progress toward the center.

48. Recession of tailwater for power outages. From the model tests it seems that a power outage reducing the river flow by about 37,000 second-feet will result in a recession of tailwater elevation of about 2 feet in the first 7 minutes after the outage occurs. This will be true for all initial flows from 37,000 to 125,000 second-feet. The same recession for an outage reducing the flow by about 19,000 second-feet will take place in about 17 minutes. From these date it seems that outages of short duration would not require synchronous bypass to keep the tailwater from excessive recession resulting in sliding of the river banks. There would be ample time to place such facilities in operation should they be provided. Synchronous bypass facilities should not be a major importance if the tailrace slopes and river banks are flattened materially.

It is believed that a complete outage reducing the flow by 81,000 second-feet will not result in as rapid a recession as indicated by the curve computed in Mr. Winter's section. The use of either this computed curve or the data obtained from the Shasta model for establishing the adequacy and the speed with which the facilities would have to be placed in operation, would have given satisfactory results.

49. Similarity of model and prototype. The flow conditions for model discharges representing small floods were smoother relatively on the model than the prototype, thus the turbulence and wave action in the tailraces will be more pronounced on the prototype at small flows

than indicated by the model. Little difference should result once the roll in the spillway bucket develops similarly to that on the prototype. This seemed to occur at a flow representing 400,000 second-feet. The influence of the out-of-scale fluid viscosity and atmospheric pressure for the model is more pronounced at the lower flows, becoming negligible for the higher discharges.

The movement of river-bed material noted on the model will occur similarly on the prototype but that on the prototype will be more severe than indicated by the model.

Although not shown conclusively by the model, the movement of material on the river bottom and in the bucket which resulted in the damage to the bucket lip between construction-block rows 37 and 38 was indicated when flow conditions approximating the prototype flood of June 1940 were represented.

That the action of the small rock attached to a string, when lowered into the water in the vicinity of the left training wall, was almost identical to that of the sounding equipment used on the prototype was attributed to the similarity between the currents in the model and prototype river channels.

50. Protection of tailrace slopes and downstream river banks. That there will be severe damage to the tailrace slopes and river banks immediately downstream of Grand Coulee Dam from wave action was indicated by the results of tests on the 1 to 60 model. This corresponds to the results of similar tests conducted on a 1 to 120 scale model of Grand Coulee Dam discussed in a memorandum to the Chief Designing Engineer, "Limits of Riprap on River Banks Below Tailraces of Grand Coulee Dam," by J. E. Warnock and J. W. Ball, dated August 10, 1936. The observation of wave action on the slopes of the left tailrace of the prototype structure during a flood of 315,000 second-feet in 1943, seems to corroborate this conclusion. It is believed that the action might prove more severe than indicated by the model, particularly for the smaller spillway discharges. The erosion by wave action increases as the slopes are steepened. If, with steep slopes, the riprap protection is not provided to sufficient height above the water surface, the beaching

action will undermine the riprap layer, lower it, and attack the banks, washing the fine material rapidly.

The movement of material by wave action can be reduced by decreasing the slope of the riprapped surfaces. Graded riprap will prove more satisfactory than dumped riprap in which smaller sizes are near the surface. Using the model results as a guide, it seems that a 6 to 1 slope covered with graded riprap would be required in critical areas near the water surface during large floods, particularly on the tailrace slopes and banks where the action is severe. With less severe conditions at lower flows, due to smaller waves, a steeper slope would be satisfactory. It is believed that satisfactory protection against wave action could be provided by using a 3 to 1 slope to elevation 940 (approximately), then gradually decreasing the slope to 6 to 1 at elevation 1000 (approximately), and extending this slope to about elevation 1030. The adequacy and extent of such protection should be tested extensively on the 1 to 60 model before a definite plan is adopted. The nature and extent of the protection to be provided would depend upon the magnitude of the flow for which the protection is intended. It seems that adequate protection in all areas should be provided for the maximum flood which might reasonably be expected to occur and that moderate protection be provided certain areas and adequate protection others for larger floods. Protection of the tailraces and transmission-line towers should be adequate at all flows. Under present conditions, the most critical areas appear to be the left tailrace and river bank between the dam and approximately station B6+00, where sliding has been evident at all times.

The erosion of the tailrace slopes downstream from the ends of the training walls on the model at flows representing 537,000 and 787,000 second-feet seems cause for alarm since disturbing the toe of the slope of the left tailrace, which projects much farther into the spillway flow than the right, would increase the tendency for sliding. Operation of the power units in the left powerhouse might become difficult or impossible if a slide should move sufficient material into the tailrace. A slide of the left tailrace slope would no doubt endanger four transmission towers located in slide areas. To safeguard against such

occurrences would necessitate the relocation of the toe of the tailrace slope and excavating it on a variable slope similar to that employed for the special test on the right tailrace. Removal of material in the slide area above the river on the left bank and resetting of the four transmission towers might be necessary. These tests are now in progress. Protection against erosion by wave action and cutting at the toes of the tailrace slopes seems essential. It is believed that protective measures should be determined and provided as soon as practicable.

The excavation, a section of which was constructed with a parabolic profile, tested on the right tailrace of the model, should prove satisfactory for 787,000 second-feet. However, the slope where the contours join those of the tailrace seems too steep. Improvement would result if the slope in this region were flattened.

The excavation at the toe of the tailrace slope downstream from the right training wall, investigated with the resloped surface mentioned above seems sufficient to prevent cutting of the toe of the tailrace slope by the spillway flow.

The limits for dredging the river fill below the bucket at the left end of the spillway will depend somewhat on the treatment of the tailrace slope, so both must be considered in future tests.

51. Tipping of arched slab from drydock gate section. The arched slab poured in the gate section of the drydock to give rigidity while the dock was lowered to its foundation as an open caisson, can be tipped successfully into the approach channel by jacks when the water surfaces inside and outside are equal or correspond to the natural stage of the river. The higher the water surface the less the force required to tip the slab. The force to tip varies linearly with the submergence and can be computed by considering the force acting on the projected area of the slab, the weight concentrated at the center of gravity and taking moments about the tipping edge correcting for the buoyant force due to submergence. The shallower the water, the farther the slab will be from the drydock as it comes to rest on the entrance channel floor. The model indicated this distance to vary from 2.0 to 6.6 feet depending on the

degree of submergence but the weight of the model slab was light, and the prototype distance should be comparatively greater due to the additional momentum.

Turbulent currents, bores, and waves of sufficient size to capsize small boats are likely to be present in the entrance channel when the slab is toppled into the channel.

Though the model indicated that it would be possible to tip the arched section from the gate opening of the drydock by increasing the elevation of the water inside above that in the entrance channel, the difference between the additional head required to tip the slab and that necessary to induce sliding, computed for the prototype, is so small that this method of disposing of the arched segment is not recommended.

Should the coefficient of friction against sliding be slightly less than that of 0.3 used in the computations, the additional head to induce sliding of the arched slab might be less than that to tip for all degrees of submergence.

Though the tipping and sliding forces were computed assuming the same distribution over the projected area as at the centerline of the slab and it is unlikely that this distribution is correct, the values obtained are considered indicative of the actions which would result from increasing the head inside the drydock above that on the outside, whether for the purpose of breaking the bond between the slab and the drydock assisting hydraulic jacks in breaking the bond or tipping the slab, or to tip the slab without the use of jacks.

APPENDIX I

HYDRAULIC REPORT NO. 132

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
July 9, 1943

Laboratory Report No. 132
Hydraulic Laboratory

Compiled by: J. E. Warnock

Subject: Hydraulic model studies on repairs and maintenance of Grand Coulee Dam spillway bucket.

1. A number of experiments have been completed during the past month to investigate the feasibility of various methods proposed for bypassing a portion of the low-water flow at Grand Coulee Dam by the installation of temporary additional outlet facilities in the right-powerhouse turbine bays. This supplemental outlet capacity will make it possible to avoid operation of the spillway during low-water season, thus permitting maintenance of the spillway gates, face, and bucket, and avoid unequal gate discharges which have caused eddy currents and excessive bucket erosion in the past. This report is a description of the various methods studied and the results obtained from the hydraulic-laboratory experiments.

2. Five methods of releasing the additional flow were considered, as follows:

(a) The first proposal was to secure advance shipment of standard Grand Coulee turbines, complete with spiral cases, wicket gates, servo-motors, oil pumps, and storage tanks, but without runners. These empty turbine cases would be installed and the steel bulkheads at the upper end of the penstocks removed, using one of the regular penstock gates for unwatering the upper end of the penstock. The capacity of each turbine as determined by model studies would be approximately 11,000 second-feet as compared to 5,200 second-feet with the turbine runner installed. A high degree of energy dissipation would be effected by the vortex flow out of the scroll case into the draft tube and by reduction of velocity in the draft tube due to sudden expansion. This plan would utilize equipment already on order, but the delivery of which has been deferred by a War Production Board stop-work order. By installing these units in the bays nearest the spillway, no difficulty would be experienced from the resulting draft-tube velocities of about 20 feet per second. These installations could be controlled either as ordinary outlet works or as synchronous bypasses.

(b) A second proposal, similar to the first, would be to install oversize turbine units, without runners, with individual capacities of 15,000 second-feet.

(c) A third proposal would be to borrow 84-inch needle valves from the canyon-wall outlet works at Boulder Dam for installation in the right powerhouse, connecting them to the power penstocks already in place. One of the regular penstock gates would be used for unwatering the upper end of the penstock and removing the steel bulkhead. In the first variation of this proposal, the needle valves would be placed as a horizontal continuation of the turbine penstock, discharging into energy absorbers similar to those on the turbine synchronous bypasses at Boulder Dam. The water at a greatly reduced velocity and head would flow through connecting elbows into the draft tubes, where an additional reduction of velocity would be obtained by the draft-tube action.

(d) A fourth scheme would utilize the 84-inch Boulder needle valves attached to the exposed ends of the turbine penstocks in the right powerhouse as in (c), but conduits attached to the downstream ends of the valves would be placed to discharge above the tailwater surface. A transition from a circular to a rectangular cross section and a bend would disperse the jets of water over a large area with very little disturbance in the tailwater.

(e) In a fifth scheme--considered most practicable because of the minimum amount of critical material required, the minimum of time required for installation, and the simplicity of design--the 84-inch Boulder needle valves would be connected to the turbine penstocks in a horizontal position but with the center lines of the valves intersecting the vertical center lines of the draft tubes. An elbow attached to the outlet end of each needle valve and embedded in a concrete block over the draft-tube opening would divert the jet from the valve downward into the draft tube. The energy largely would be dissipated in the turbulence in the draft tube and the flow conditions in the tailrace would be similar to those from a regular turbine installation.

3. To study the hydraulic behavior of the first and second proposals, the turbine model used in the study of the design of the Grand Coulee draft tubes was reinstalled without the turbine runner and dynamometer. Preliminary tests indicated the feasibility of the scheme and also indicated the need for comprehensive model studies to provide information on the torque on the wicket gates and pressure distribution within the draft tube. The model was revised by shifting the speed ring until the wicket gates and stay vanes were in line when the wicket gates were 60 degrees from the closed position. The conical section at the top of the draft tube was replaced by a cylindrical section so as to provide a sudden expansion below the scroll case, and a filler block

was installed in the horizontal portion of the draft tube to provide a second sudden expansion. A sudden expansion is one of the best methods of dissipating energy in a closed conduit without damaging the conduit due to cavitation and pitting. One wicket gate was remounted in ball-bearing supports and attached to spring balances to provide means of determining the magnitude of torque at different gate openings. The shape of the wicket gates was altered slightly to conform to the design in the Grand Coulee turbines. The fairwater cone with a vent pipe to the atmosphere and a ring representing the runner shroud were installed. With these changes, the capacity of the scroll case under a head of 345 feet was 11,000 second-feet when the wicket gates were 60 degrees from the closed position. The maximum designed opening as determined by the stroke of the servomotor was 60 degrees. The discharge when the wicket gates were at 70 degrees, or 10 degrees beyond the limit of the servomotor, was 13,000 second-feet and, when radial, or 80 degrees open, the discharge was 12,600 second-feet. In terms of the prototype, a vent pipe with a diameter of approximately 33 inches was required through the fairwater cone to prevent cavitating pressures in the draft tube. Four 9-inch pipes were also needed to relieve excessive subatmospheric pressures at the sudden expansion below the scroll case, but no aeration was needed at the filler block in the horizontal portion of the draft tube. If aeration was not provided, the vibration of the scroll case and draft tube was excessive and the pressures in the prototype would have reached vapor pressure and caused cavitation and pitting. With aeration as described, there was slight vibration at all openings of the wicket gates, it being least severe when the gates were radial, or 80 degrees, from the closed position. At 60 degrees the torque on the wicket gates when operating without the turbine runner was six times the torque at the position of maximum operating capacity when the turbine runner was installed; at 80 degrees the torque was 50 percent greater. To have completely explored the feasibility of this scheme, it would have been advisable to conduct tests on a model operating with prototype heads, which would require 8 to 10 months. The uncertainty of knowledge of the behavior without high-head tests, the excessive time involved in installation of the scroll cases, and the excessive amount of critical material required led to favorable consideration of another scheme. The principal disadvantage was that the installation could not be completed available for use prior to the low-water season of 1945-46.

4. The hydraulic behavior of the third proposal, in which the Boulder 84-inch needle valves would have been installed at the lower end of the turbine penstocks so as to discharge into energy absorbers, was obvious from the satisfactory performance evolved from model studies made several years ago and since verified in the prototype installation on the turbine synchronous bypasses at Boulder Dam. The scheme, however, would have required an excessive amount of critical material and the installation might not have been available for use during the low-water season of 1944-45.

5. In the fourth scheme, where the Boulder 84-inch needle valves would have discharged above the tailrace through conduits attached to the downstream end of the valves, the model indicated no adverse behavior insofar as the valves and conduits were concerned. The spray from the jets of water would have been objectionable and considerable excavation of concrete would have been required in the downstream wall of the powerhouse in placing the conduits and later in removing them when the turbines were installed. Most of this excavation would require caissons on the downstream face of the powerhouse to permit excavation of the concrete and installation of the conduits during high-water season.

6. The objectionable features of the fourth proposal were the excessive excavation of concrete in the downstream wall of the powerhouse in placing the conduits and later in removing them and the caissons required to permit work during the high-water season. To utilize the draft tubes, which were already equipped with steel bulkheads at the downstream end, consideration was given to the discharging of the needle valves directly into the draft tubes. To study the feasibility of this idea, a model needle valve with a discharge diameter of 2.8 inches, which had been used previously as a 1:30 model of the 84-inch Alcova valves, was mounted in a vertical position above a 1:24 model of a Grand Coulee draft tube. With the jet from the valve tangent to the back face of the elbow of the draft tube, the performance was excellent. The flow was well-distributed through the three passages of the draft tube and the disturbance at the tailrace water surface did not greatly exceed that with flow from normal turbine operation. In the prototype, however, the vertical mounting was not structurally feasible, so, in the subsequent test, the model was mounted in a horizontal position with a 90-degree elbow between it and the draft-tube opening. A number of positions were tried as to the angle between the center line of the turbine penstock and the needle valve and as to the point of introduction of the jet into the draft tube. In the final design, the angle between the center line of the penstock and the valve was 41 degrees and the center line of the needle valve intersected the vertical center line of the draft tube. The elbow between the needle valve and draft tube had a transition from the circular section 84 inches in diameter downstream from the valve to a section 15 percent larger in area composed of a circle on the back coinciding with the diameter of the draft tube at the point of connection and a straight line opposite. The circle and the straight line were connected by circles of lesser diameter. The 15-percent increase in area was introduced to create low pressures in the elbow which would cause a flow of air through a wind box between the needle valve and the elbow. The air demand was created to start turbulence and energy dissipation in the jet as it issued from the valve and to prevent local cavitation due to irregularities on the surface of the metal lining. With the valve and elbow in place, trials were made as to the quantity and point of introduction of air. Through cooperation with the mechanical section, a

combined air box and expansion joint was developed on the top of which will be connected an air-vent pipe at least 18 inches in diameter. An additional air-vent pipe at least 6 inches in diameter will be installed extending vertically upward through the concrete block over the draft-tube opening. This second pipe can be either extended upward to the necessary height or can be connected across to the 18-inch vent pipe. With these vent pipes in place, the lowest pressure measured was -8.0 feet of water at a point on the under side of the elbow near the point of tangency to the draft tube. Without these vent pipes, the pressures in the elbow will be of the magnitude which will cause vibration, cavitation, pitting, and ultimate destruction of the under side of the elbow and the adjacent concrete in the supporting block. The model of the final design of this installation and the conditions on the surface of the tailrace are shown in figure 1. The 1:24 model of the Grand Coulee draft tube used in the preliminary tests of this installation is shown in the background of figure 1a.

- - -

Figure 1

a. 1:30 model of a single by-pass outlet for right powerhouse - Draft tube of 1:24 model

b. Flow conditions in tailrace with by-pass outlet operating.

MODELS USED IN TESTING OF BY-PASS OUTLET FOR RIGHT POWERHOUSE OF GRAND COULEE DAM

APPENDIX II

ABSTRACT OF TEST RECORDS

Erosion at downstream ends of training wall, 1 to 120 scale model of Grand Coulee Dam. by: J.W. Ball.

APPENDIX II

-- ABSTRACT OF TEST RECORDS

Erosion at downstream ends of training walls, 1 to 120 scale model of Grand Coulee Dam. by: J. W. Ball.

Excessive scour of the river bottom adjacent to the downstream ends of the training walls was noted during the initial tests on the 1 to 120 scale model of the dam, its appurtenances, and the downstream river channel. An investigation of the flow conditions in these regions disclosed the presence of high-velocity currents moving from the tailraces transversely across the ends of the walls into the spillway flow. A part of this flow continued toward the center of the river under the spillway sheet carrying sand from the eroded areas with it. Apparently the velocity of this current decreased toward the center of the stream for the sand was piled above the elevation of the bucket lip in this region with the height diminishing gradually toward the walls where substantial areas of the bedrock were swept clean. The size of these areas varied directly as the spillway discharge. Large quantities of water overtopped the walls and flowed into the spillway bucket.

The transverse flow at the ends of the walls and along the bucket was attributed to a pressure differential between the tailrace and the region immediately below the bucket. Although no measurements of the velocities were made, the pressure difference taken from pressures measured on the wall of the 1 to 120 model and the bucket of the 1 to 40 sectional model indicated them to be 23 to 28 feet per second prototype or 2.1 to 2.55 feet per second on the 1 to 120 model. According to Gibson(1) the model velocities are capable of transporting pea gravel. A study of the photographs of the training wall tests corroborate this action. With the prototype velocities large size rocks are certain to be transported from these regions. The velocities at the base of the wall might be higher than indicated by the pressure data, since the pressures on the bucket were taken at the center line of the

(1) "Hydraulics and its Applications," by A. H. Gibson, P. 341.

1 to 40 sectional model, and the transverse velocity of the current below the bucket which tends to lower the pressure in this region was negligible.

Several extensions to the right wall varying in length and height, were investigated to ascertain their effect on the scouring action (figures 1 and 2).

With extensions of 10 to 20 feet (prototype) to the original wall, which had a slope of 1/2 to 1 on the downstream end and its top at elevation 935.0, there was very little change in the depth or size of the eroded area from that for the original design. No improvement was recorded for the shorter extension when observations were made at flows representing 250,000 and 500,000 second-feet. With the longer extension the scour decreased on the tailrace side and increased on the spillway side of the wall. The change was slight, however, indicating that an extension of appreciable length would be required to reduce the scour materially. Moreover, the height of these extensions seemed overadequate.

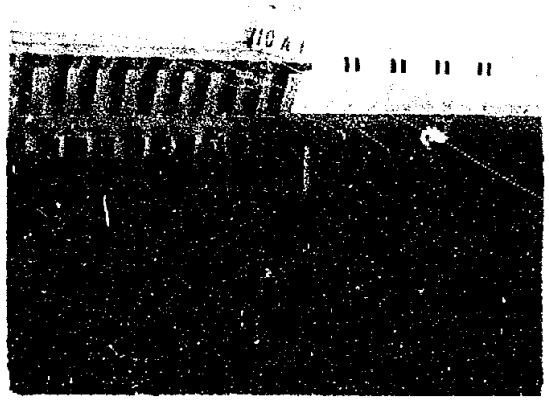
Extensions of 50, 75, and 100 feet (prototype) with their tops at elevation 900.0, so as not to constrict the flow from the tailraces, were used for the next series of tests.

The erosion with the 50-foot extension was less than with the shorter ones of the previous test but was considered excessive. Further improvement resulted with the 75-foot extension and conditions with the 100 foot length were better than for any of the others. Additional improvement seemed possible, but the solid nature of the foundation rock did not seem to justify the cost of the protection, so the study was discontinued.

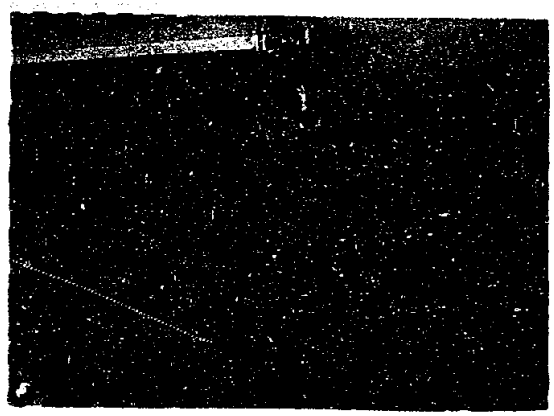
The top of the wall was raised to elevation 980 and the downstream end placed 444.0 feet from the axis of the dam and made vertical to improve flow conditions at the corners of the powerhouses. The erosion at the base of the wall, however, showed no improvement over that of the original design.

On one occasion during the tests a whirling eddy was observed downstream from the bucket lip adjacent to the training wall. Apparently the seriousness of this whirl was not realized at the time for it was

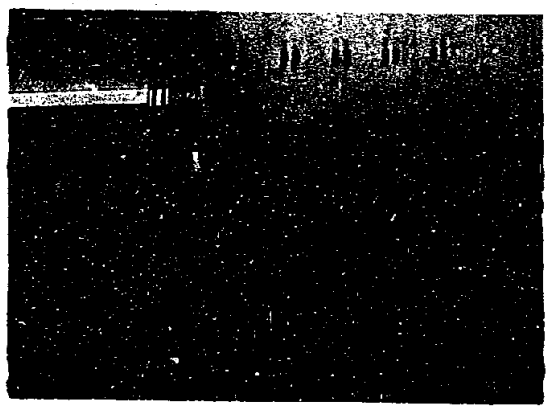
not mentioned in the notes, but was indicated on a sketch showing the currents in the vicinity of the wall. No doubt the small size of the sand particles used in the model and the small number of grains reaching this region after the flow conditions were established were responsible for this misinterpretation.



A. Before test- 10-foot extension.

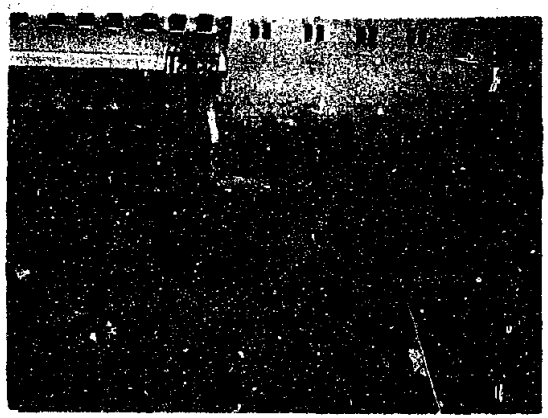


B. Erosion for 10-foot extension and 500,000 second-feet.

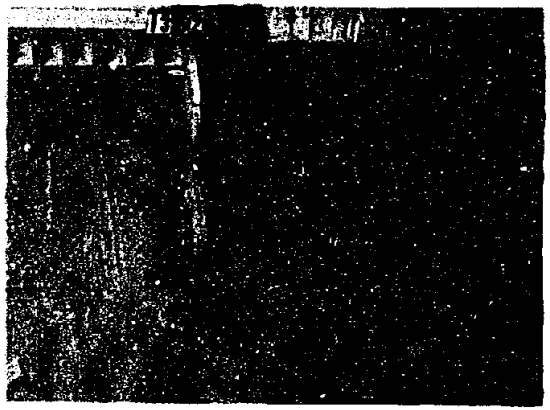


C. Erosion for 20-foot extension and flow of 500,000 sec.-ft.

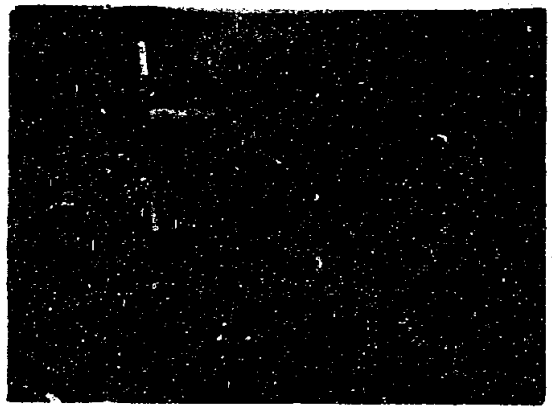
TRAINING WALL EXTENSION WITH TOP AT ELEVATION 935.0



D. 50-foot extension



E. 75-foot extension

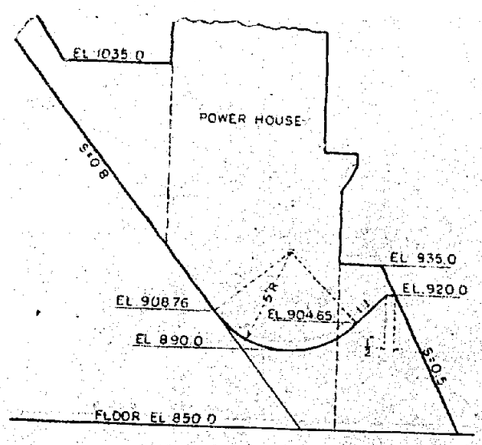


F. 100-foot extension.

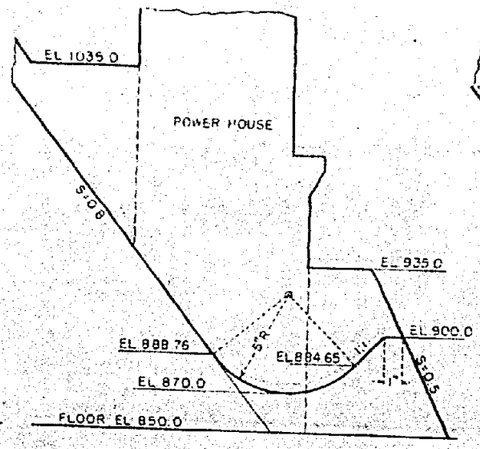
Erosion at end of training wall extensions with tops at elevation 900.0 for a flow of 500,000 c. f. s.

EROSION AT END OF TRAINING WALL EXTENSIONS
1:120 MODEL OF GRAND COULEE DAM

Figure 1



TEST NO. 8



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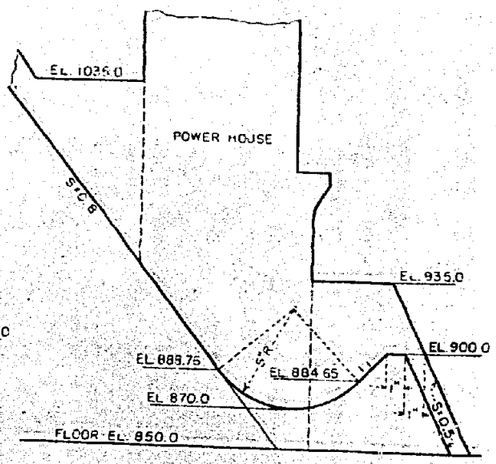

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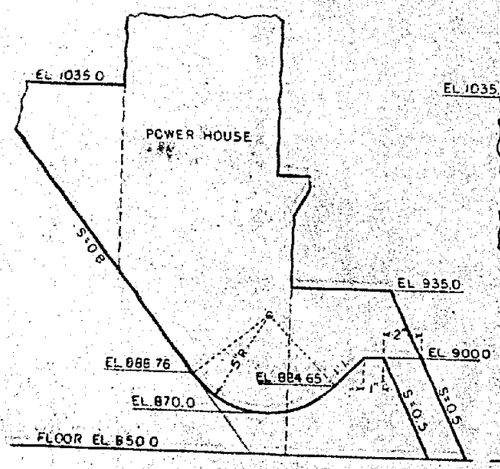

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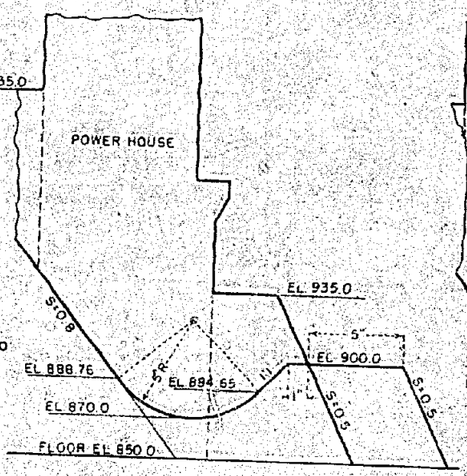

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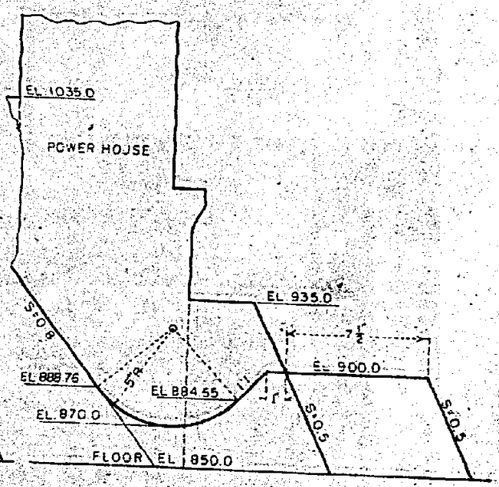

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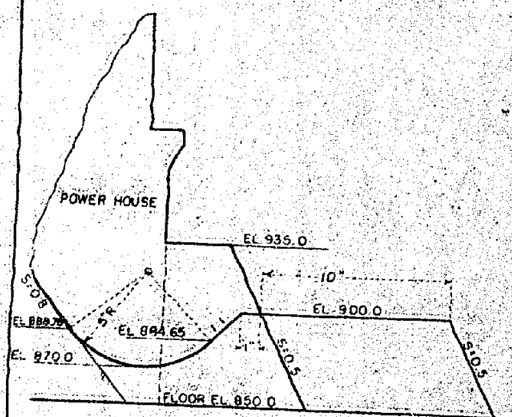

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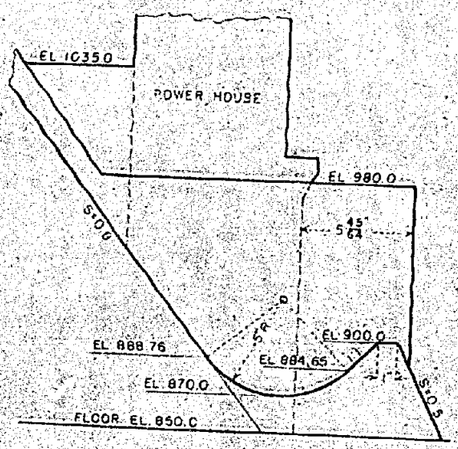
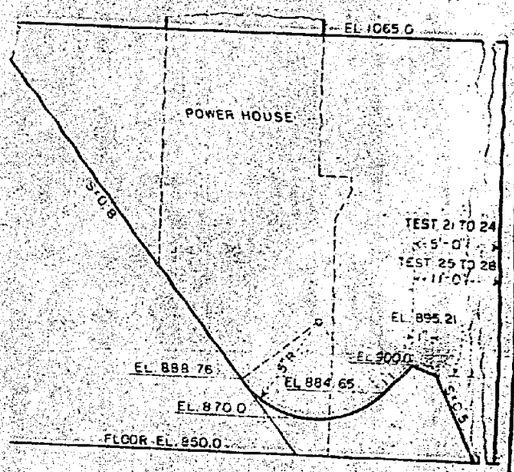
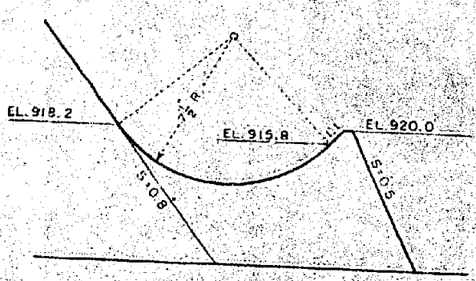


TESTS NOS. 15 AND 20

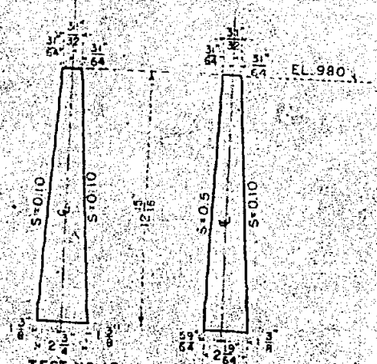


TESTS NO. 21 TO 35 INC.



75 FOOT RADIUS BUCKET

NOTE
Sill tested on 75 foot radius bucket similar to sill used in test No. 9 — 1 to 40 model.



TEST NO. 15 TEST NO. 20
UPSTREAM ELEVATION
LEFT TRAINING WALL

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON

GRAND COULEE DAM

HYDRAULIC MODEL EXPERIMENTS
SPILLWAY STUDIES — SCALE RATIO 1 TO 120
DETAILS OF BUCKETS AND TRAINING WALLS

DRAWN D.G.W. SUBMITTED
TRACED H.A.L. 45. RECOMMENDED
CHECKED C.W.L. APPROVED
DENVER, COLO. MAR. 30, 1945.

222-D-3284

FIGURE 2

APPENDIX III

HYDRAULIC REPORT NO. 10

(With only most important drawings and photographs included)

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

FORT COLLINS, COLORADO

August 10, 1936.

MEMORANDUM TO CHIEF DESIGNING ENGINEER
(J. E. Warnock and J. W. Ball)

Subject: Limits of riprap on river banks below Tailraces of Grand Coulee Dam, Columbia Basin Project.

1. In compliance with your verbal instructions of June 5, in which you referred to item 8 of the report of the board of consulting engineers of April 6, 1936, the model of the ultimate development of the Grand Coulee Dam was reinstalled in the Fort Collins laboratory and actual testing was commended on July 21.

2. The April 16 report of the board on this subject is as follows:

"Item 8 - Limits of Riprap on River Banks below Tailraces. In our study of this item, we have looked over, in such detail as time has permitted, the results of the Fort Collins model experiments bearing upon this problem. We have also inspected on the ground, the general condition of the banks between the dam and the highway bridge, especially on the west side of the river, taking note of the more recent fills along this bank, and their general condition as regards character of material and slope.

In the light of the results of these model experiments and in the exercise of our best judgment otherwise, we recommend as follows:

(1) On the west side of the river, riprap protection three feet in thickness, placed generally between the limits of elevation of 950 and 1000 and extending for a minimum distance of 1500 feet downstream from the end of the present tailrace riprap.

(2) As a protection to the bank near and just above the highway bridge piers on the east side of the river, and especially as a safeguard against any cutting which might endanger pier No. 1, we recommend the present placing on the east bank, of riprap of the same thickness and between the same limits of elevation, from the bridge pier for a distance of 100 feet both upstream and downstream.

(3) We recommend the setting up of a model similar to that with which the studies thus far have been made, but of such length as to indicate the character of the flow for a distance of some 2500 feet downstream from the present riprap limit; and the conduct of such experiments as shall serve to indicate the character of the flow over a river stretch of this length, or approximately between the dam and the highway bridge. We recommend further that consideration be given to such additional riprap protection on either or both sides of the river as the results of these tests may indicate to be needful.

In connection with this riprap protection, we may note that, with the large amount of scaling which has developed as necessary in the east cofferdam and abutment areas, and from other excavations yet to be made, there will be available an abundance of rock for such purpose and for the placing of which it appears probable that arrangements can be made with the contractor at a low cost."

3. The model of the ultimate development of the Grand Coulee Dam to a scale ratio of 1:120 had been originally constructed to assist in the development of the protection at the toe of the dam, but had been dismantled to provide space for the study of the diversion of the Columbia River through the west cofferdam area. In the new installation about 5,000 feet of river bed downstream from the dam has been represented with the topography up to elevation 1075. Highway bridge piers 2 and 3 (fig.1), constructed of sheet metal, were included in the first two tests. The topography (plates 1 and 8) was molded in fine sharp-grained sand of which 100 percent passed a ten-mesh screen. The topography was replaced at the beginning of each test by the use of the

profilograph which was especially designed and built in the laboratory for that purpose. The riprap, when used, was a hard crushed shale, screened and mixed to conform as nearly as possible to the rock size specified in a letter from Construction Engineer Banks to the Chief Engineer, dated October 5, 1935, which says:

"In response to the request in your letter of September 20 on the above subject, we would advise that the gradation in size of the riprap actually being used for the protection of the downstream side of the tailrace from the west powerhouse, is estimated as follows. Two estimates are given, one for rock excavated from the west abutment, and the other of rock excavated from the floor of the west foundation. The rock from the west abutment is broken up to smaller sizes, possibly because the breaking up effect of blasting is supplemented by a similar effect due to falling from the steep and high abutment.

	Rock from West Abutment	Rock from floor of West Foundation
Dust to 1/4 inch	5%	2%
1/4 inch to 1 cu. ft.	40%	20%
1 cu. ft. to 1 cu. yd.	50%	70%
Over 1 cu. yd.	5%	8%

The riprap material used in the model had a net density of 151 pounds per cubic foot, with an analysis as shown on figure 2. A concrete slab was placed immediately below the bucket extending 360 feet downstream to represent the rock at the damsite.

4. Preliminary tests on this model were witnessed at the time of its completion by Engineers Hammond and Hornsby of the Denver office. In these preliminary tests, no riprap was placed on the slopes of the tailraces. With a flow of 250,000 second-feet over the spillway there was no movement of the sand along the river banks other than a slight settling caused by wave action along the shore. As the flow was increased, a large, slow-moving surface eddy developed in each tailrace resulting from an "ejector action" produced by the high-velocity water at the downstream ends of the spillway training walls. The eddies were

were responsible for severe erosion of the tailrace slopes at or near the water surface. These could be partly eliminated by operating all turbines in the powerhouses, but, since this is not practical it was not considered a feasible solution.

5. Upon the completion of the preliminary tests, the topography of the river bed was restored to its initial shape (plate 1). The tailrace slopes were covered with riprap ten feet thick at the base and three feet thick at the top near the powerhouse roadway. This is according to the original recommendations of the Board. Riprap limits for test 1 are shown on figure 3 (the downstream limit in the west tailrace was that determined by tests on the 1:120 Grand Coulee diversion model). The tailwater-discharge relationship was maintained during any increase or decrease in discharge. The gates were operated to maintain an approximate reservoir elevation of 1288. The tailwater elevation, observed immediately upstream from the highway bridge, was regulated according to the rating curve on figure 4, which was also used for previous tests on this same model. Each test was divided into four runs with discharges of 250,000, 500,000, 750,000 and 1,000,000 second-feet. The following table was adhered to in making the tests and the river bed was not disturbed between runs.

Discharge Sec. Ft.	Model time to attain Discharge *		Duration of Discharge				
			Model		Prototype		
	Hours	Minutes	Hours	Minutes	Days	Hours	Minutes
250,000	0	22	2	0	0	21	55
500,000	0	40	3	0	1	8	52
750,000	0	52	5	0	2		48
1,000,000	1	0	7	0	3		41

* The same times were used in reducing to zero discharge.

6. Cross-sections were taken at various points along the river for all runs except that with a flow of 250,000 second-feet. These sections are shown on figures 5 to 12 inclusive. Briefly, the results of test 1 are as follows:

(a) Flow - 250,000 second-feet (plate 7A). During a run of two hours on the model the riprap was not disturbed or the banks noticeably eroded. The usual rearranging of the river bed occurred exposing the rock surface at a point 180 feet downstream from the right end of the spillway. The top of the mound in this vicinity (figure 3) was moved a short distance downstream.

(b) Flow - 500,000 second-feet. After a run of three hours the erosion increased downstream from the bucket. The wave action attacked the river banks but soon reached a stage of stability, ceasing to do further damage. The tailrace riprap was disturbed but was not carried away (plates 4B and 5B) and slight erosion occurred at the bridge piers (plate 6B).

(c) Flow - 750,000 second-feet. The wave action became more severe and considerable erosion occurred along the river banks, particularly where they were steep (plate 2). The continuity of the tailrace riprap was broken by the wave action and a return flow along the bank deposited very fine sand on the slopes in front of the powerhouses (plates 4C and 5C). A portion of the riprap at the foot of the tailrace slopes was disturbed and deposited immediately downstream from the lip of the bucket (plate 2). Erosion at the bridge increased exposing the top of the rectangular footing of pier 3 (plate 6C). The length of the run was five hours (model time).

(d) Flow - 1,000,000 second-feet. Very severe erosion occurred for this discharge. The run at this discharge was terminated after a seven-hour period, as movement of the bed material, except that caused by wave action (plate 7B) practically ceased. The tailrace slopes were seriously eroded and large piles of sand were deposited in front of the powerhouses, (plates 4D and 5D) but no sand was deposited in the draft tubes. Erosion of river banks near the highway bridge piers was severe but did not expose the tops of the two footings (plate 6D).

7. The thickness of tailrace riprap according to the present design is insufficient for floods greater than 500,000 second-feet. To protect the river banks immediately below the tailraces for floods of this magnitude, it appears to be necessary to extend the riprap at

least 500 feet downstream. Riprapping of the river bank behind highway bridge pier 2 is necessary for the protection of pier 1. After careful study of the results obtained from test 1, particularly those for the maximum discharge, Engineers Hammond and Hornsby recommended increasing the thickness of the riprap as well as an extension downstream. The riprapping of both tailraces (figure 3) was extended to the end of the 2-1/2:1 excavated slope and a uniform thickness of six feet was used between elevation 960 and the edge of the roadway and to a limit of elevation 1050 at the lower end. The other limits of this thickness were the point of curvature of the 3:1 slope and the end of the 2-1/2:1 excavated slope. From the point of curvature on the 3:1 slope, the space occupied by the added rock was triangular in shape reaching the original thickness of three feet near the abutment end of each tailrace as indicated on figure 3. The east powerhouse roadway was placed on a four percent grade to protect it from the wave action during large floods. The right bank at the highway bridge was riprapped between elevations 930 and 1050 for a distance of 100 feet upstream and downstream from the nose and tail of pier 2. When these changes were incorporated in the model (plate 8) the same procedure was followed as for test 1. The results of test 2 indicated that the riprap was still insufficient to withstand the wave action at the higher discharges. Better conditions were prevalent, however, for discharges up to and including 750,000 second-feet (plates 9 to 14, incl., and figures 13 to 20, incl.). It is believed that additional thickness along the upper limits of the tailrace slope and the extension of the riprap to a point farther downstream would aid in resisting this erosion, but would not completely eliminate it. The three feet of rock on the right bank near pier 2 is sufficient protection for discharges up to 750,000 second-feet. Indications were that with a flow of 1,000,000 second-feet, erosion would endanger the end bridge piers (plate 14). It is intended to verify this belief by tests using a complete set of bridge piers. Design details of the end piers were not available until after the completion of test 2 thus preventing the inclusion of that data in this memorandum.

8. As a protection against the severe scour conditions and the high wave action prevalent in the stilling pool at the higher discharges, as indicated by the model, the following suggestions are made:

(a) Place riprap protection approximately five feet in thickness on the west river bank, between elevations 975 and 1030 extending approximately 1500 feet downstream from the end of the 2-1/2:1 excavated slope.

(b) Increase the thickness of the riprap on the west tailrace slopes, used in test 2, from six to ten feet, between elevations 975 and 1030, extending from a point 300 feet upstream from the curve connecting the 3:1 and the 2-1/2:1 slopes to the downstream end of the latter slope. It is believed that riprap thus placed should be bonded by mortar or some asphalt compound to make it more resistant to the severe wave action.

(c) Place riprap protection five feet in thickness on the east river bank between elevations 975 and 1030, extending approximately 500 feet from the downstream end of the 2-1/2:1 excavated tailrace slope. A small amount of additional excavation would be necessary to properly place this riprap.

(d) Increase the thickness of the riprap on the east tailrace slopes, used in test 2, from 6 to 10 feet, between elevations 975 and 1030, extending from a point 300 feet upstream from the curve connecting the 3:1 and the 2-1/2:1 slopes to the downstream end of the latter slope. It is believed that this riprap should also be bonded.

(e) Place riprap protection five feet in thickness in the vicinity of bridge piers 2 and 3 between elevations 950 and 1030, extending 100 feet upstream and 100 feet downstream from the extreme limits of the pier base.

9. All the features in paragraph 8 are being incorporated in the model at the present time including a complete set of bridge piers. The results will be forwarded to you before the conclusion of the meeting of the Board.

10. It is deemed advisable that you, and if possible the members of the Board, visit the laboratory in the near future to inspect the model in operation.

J. E. Warnock

J. W. Ball

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Fort Collins, Colorado.

August 12, 1936.

MEMORANDUM TO CHIEF DESIGNING ENGINEER
(J. E. Warnock and J. W. Ball)

Subject: Limits of riprap on river banks below tailraces of Grand Coulee Dam, Columbia Basin Project.

1. Test 3, containing the features noted under paragraph 8 in our memorandum dated August 10, 1936, has just been completed and part of the data is presented herewith as a supplement to the progress report contained in that memorandum. Data corresponding to that taken for previous tests was obtained. Time does not permit its entire completion so as to reach you before the end of the board meeting and thus a short discussion, with pictures, will be presented at this time. Cross-sections of the river bed will be available in the near future and if requested can be furnished each member of the board.

2. The features, except parts (b) and (d), outlined in paragraph 8 of our memorandum dated August 10, 1936 were followed explicitly (plate 15). In parts (b) and (d) the thickness of riprap between the point of curvature of the 3:1 slope and a point 300 feet upstream was made variable instead of a uniform 10-foot thickness, figure 21, plates 19A and 20A. As this difference occurred within the region of bonded riprap this change had no effect on the results. Furthermore, if bonding is used the 10-foot thickness of riprap will not be necessary.

3. The results of test 3 are contained in the following discussion.

(a) Flow 250,000 second-feet. No movement of material was noted other than the usual rearranging of the river bed just downstream from the bucket.

(b) Flow 500,000 second-feet. The erosion immediately downstream from the bucket was increased. Some erosion due to the wave action was

noted on both banks and immediately downstream from the sections of bonded riprap (plates 16, 19B, and 20B). In general the conditions were the same as in tests 1 and 2 (plate 16).

(c) Flow 750,000 second-feet. With this discharge the riprap along the lower limits of the 2-1/2:1 excavated tailrace slopes was disturbed such that the bonded sections were undermined (plates 19C and 20C), otherwise the conditions along the tailrace slopes were much improved (plate 17). Slight improvement was also noted along the banks below the tailraces and in the vicinity of the bridge piers. A small slide, which was probably due to the steep slope, was noted near pier 4 and the west approach to the bridge (plate 21C). Extension of the area of bonded riprap on the tailrace slopes would serve to prevent undermining at the lower edge.

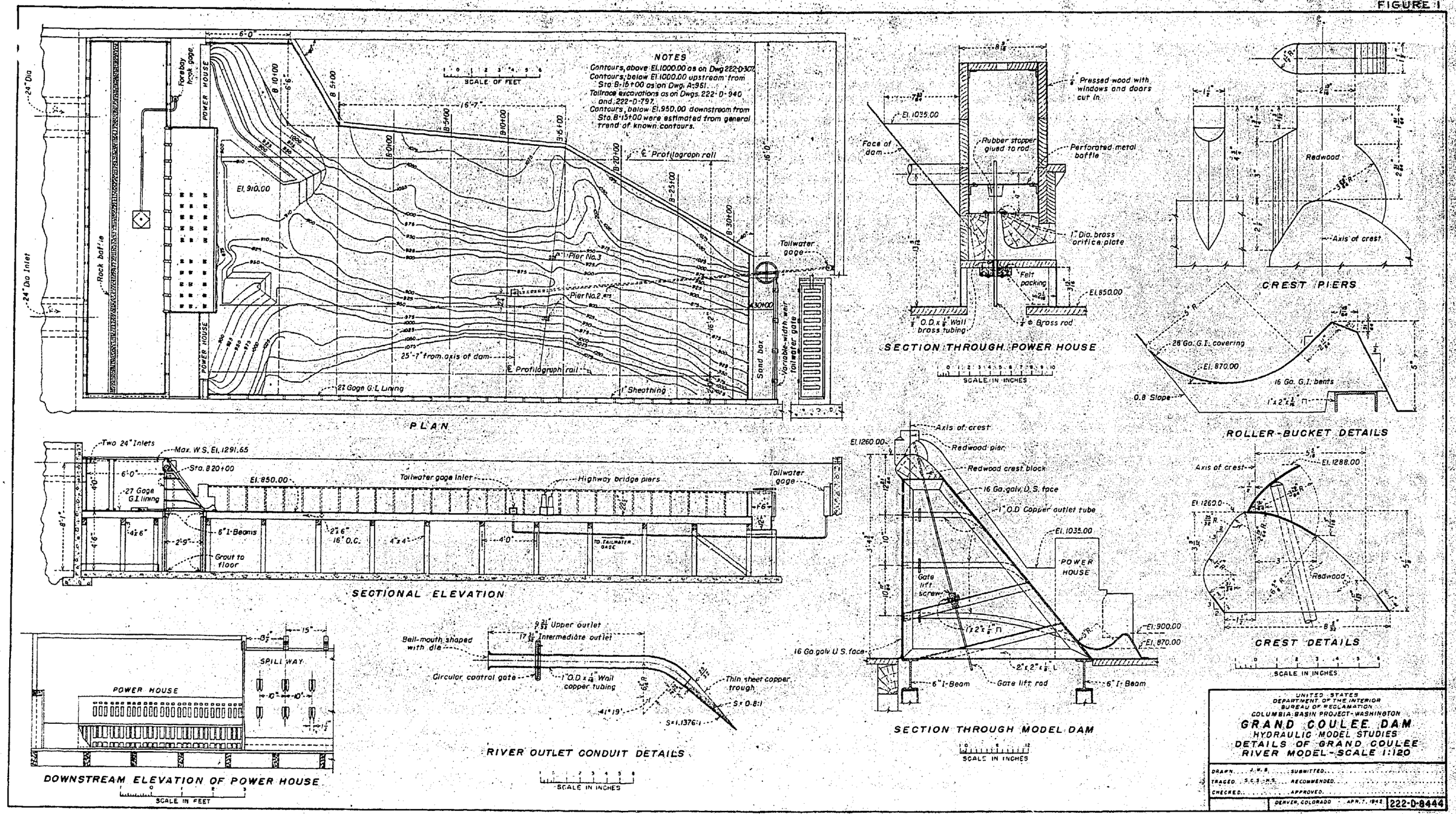
(d) Flow 1,000,000 second-feet. Continued undermining of the bonded riprap sections was noted (plates 19D and 20D). The erosion became more severe but did not reach proportions comparable with those of tests 1 and 2, (plate 18). Some sand was deposited on the tailrace slopes immediately downstream from the spillway end of both powerhouses (plates 19D and 20D). The riprap on the banks of the river at the ends of the highway bridge was not washed away. That near pier 1 settled slightly but was not overtopped (plate 21D). Near pier 4 and the approach considerable settling was noted. The riprap settled below the water surface, sand was washed away and the footing exposed. The top of the rectangular base on pier 3 was also exposed.

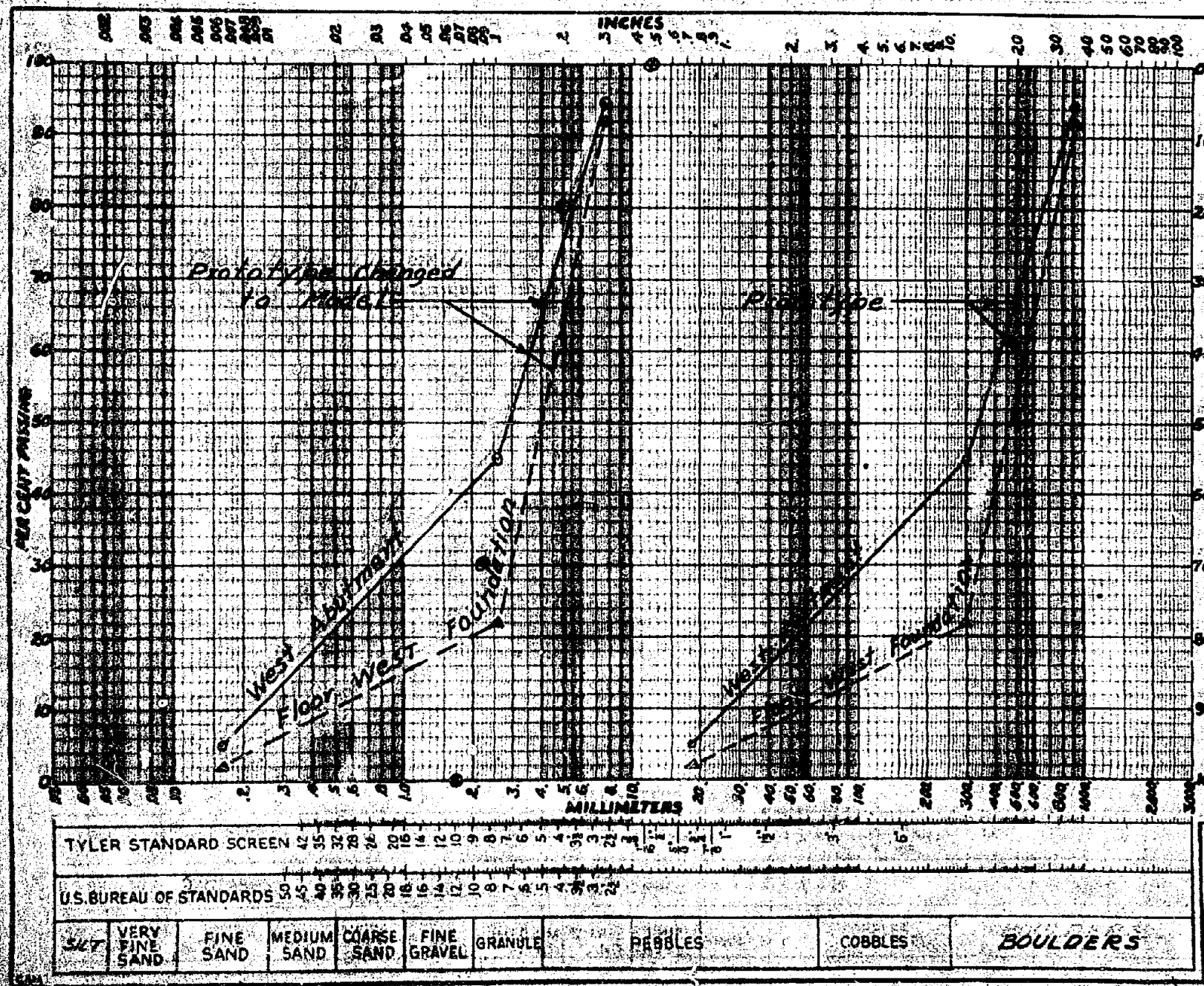
- - -

J. E. Warnock

J. W. Ball

FIGURE 1


NOTES
Contours, above El. 1000.00 as on Dwg 222-D-307.
Contours, below El. 1000.00 upstream from Sta. B-15+00 as on Dwg. A-951.
Tailrace excavations as on Dwgs. 222-D-940 and 222-D-797.
Contours, below El. 950.00 downstream from Sta. B-15+00 were estimated from general trend of known contours.
SCALE OF FEET
PLAN
24" Dia Inlet
Rock baffle
Forebay hook gage
POWER HOUSE
El. 910.00
Pier No. 3
Pier No. 2
25'-7" from axis of dam
℄ Profilograph rail
22 Gage G.I. Lining
1" Sheathing
Tailwater gage
Sand box
Variable-width weir
tailwater gate
SECTION THROUGH POWER HOUSE
Face of dam
El. 1035.00
Pressed wood with windows and doors cut in
Rubber stopper glued to rod
Perforated metal baffle
1" Dia. brass orifice plate
Felt packing
El. 850.00
O.D. x Wall brass tubing
Brass rod
SCALE IN INCHES
CREST PIERS
Redwood
Axis of crest
ROLLER-BUCKET DETAILS
26 Ga. G.I. covering
El. 870.00
16 Ga. G.I. bents
0.8 Slope
SECTIONAL ELEVATION
Two 24" Inlets
Max. W.S. El. 1291.65
Sta. 8 20+00
El. 850.00
Tailwater gage inlet
Highway bridge piers
Tailwater gage
27 Gage G.I. lining
6" I-Beams
Grout to floor
16" O.C.
TO TAILWATER GAGE
SECTION THROUGH MODEL DAM
El. 1260.00
Axis of crest
Redwood pier
Redwood crest block
16 Ga. galv. U.S. face
1" O.D. Copper outlet tube
El. 1035.00
POWER HOUSE
Gate lift screw
El. 900.00
El. 870.00
16 Ga. galv. U.S. face
6" I-Beam
Gate lift rod
CREST DETAILS
El. 1288.00
El. 1260.0
Redwood
SCALE IN INCHES
DOWNSTREAM ELEVATION OF POWER HOUSE
SPILL WAY
POWER HOUSE
SCALE IN FEET
RIVER OUTLET CONDUIT DETAILS
Upper outlet
Intermediate outlet
Bell-mouth shaped with die
Circular control gate
1" O.D. x Wall copper tubing
Thin sheet copper trough
S = 0.8:1
S = 1.1376:1
SCALE IN INCHES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES
DETAILS OF GRAND COULEE RIVER MODEL - SCALE 1:120
DRAWN J.W.B. SUBMITTED
TRACED S.C.S. - H.S. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO - APR. 7, 1942
222-D-8444



REMARKS

NOTE

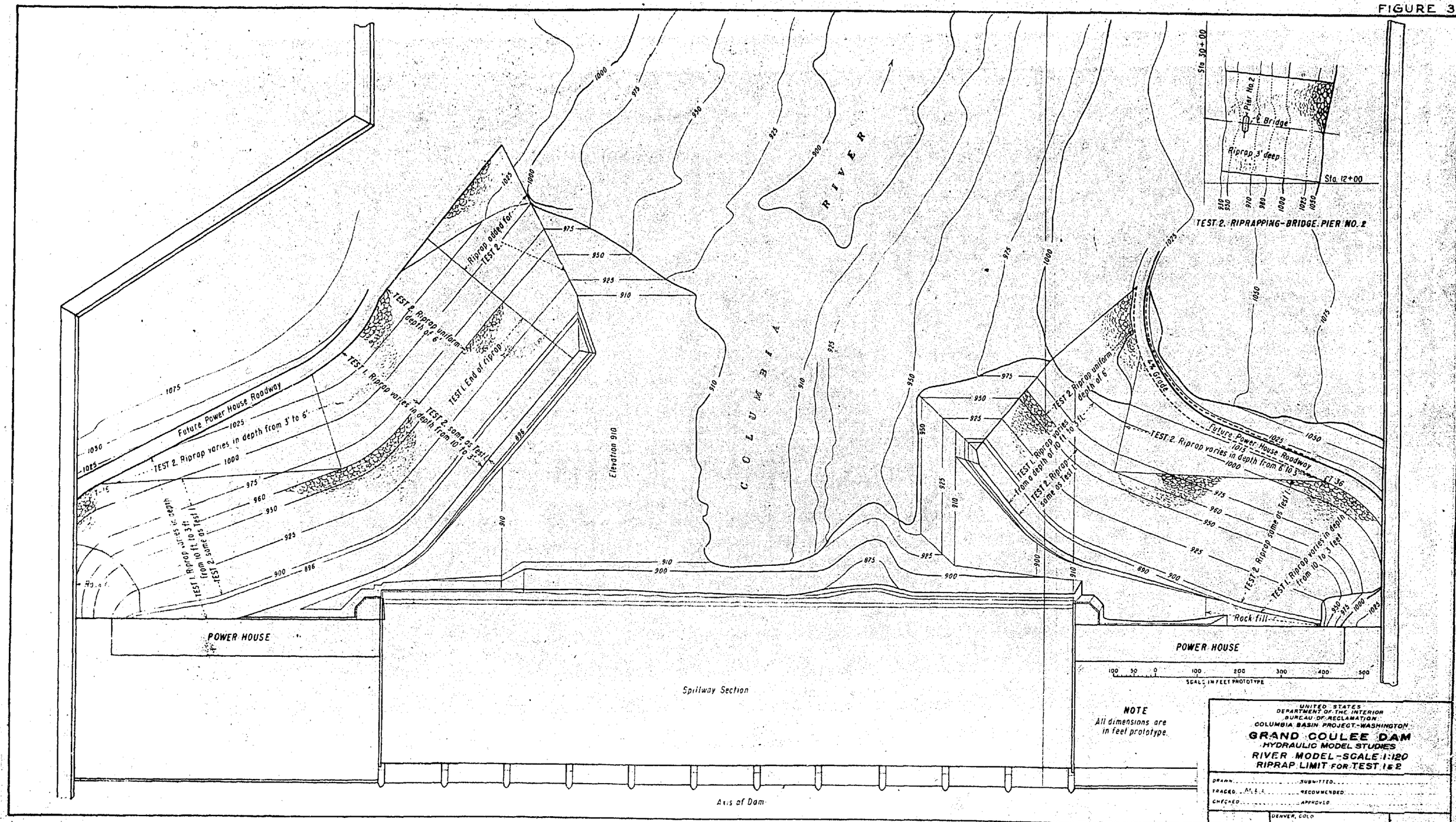
● Rip-rap as Tested on Model

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

ANALYSIS CHART
GRAND COULEE DAM
RIP-RAP
(Material)

DATE
DRAWN W.O. CHECKED R.G. X-D-524

FIGURE 3


COLUMBIA RIVER
Elevation 910
Future Power House Roadway
TEST 2. Riprap varies in depth from 3' to 6'
TEST 1. Riprap varies in depth from 10 ft. to 3 ft.
TEST 2. same as test 1
TEST 1. Riprap varies in depth from 10 to 3'
TEST 2. same as test 1
TEST 1. End of riprap
TEST 2. Riprap uniform depth of 6'
Riprap added for TEST 2
POWER HOUSE
Spillway Section
Axis of Dam
Rock fill
Future Power House Roadway
TEST 2. Riprap varies in depth from 6' to 3'
TEST 1. Riprap varies from a depth of 10 ft. to 3 ft.
TEST 2. Riprap same as test 1
TEST 2. Riprap uniform depth of 6'
TEST 2. Riprap same as test 1
TEST 1. Riprap varies in depth from 10 to 3 feet
POWER HOUSE
Sta. 30+00
Pier No. 2
℄ Bridge
Riprap 3' deep
Sta. 12+00
TEST 2. RIPRAPPING-BRIDGE PIER NO. 2
SCALE IN FEET PROTOTYPE
NOTE
All dimensions are in feet prototype.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT-WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES
RIVER MODEL-SCALE 1:120
RIPRAP LIMIT FOR TEST 1 & 2
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
DENVER, COLO

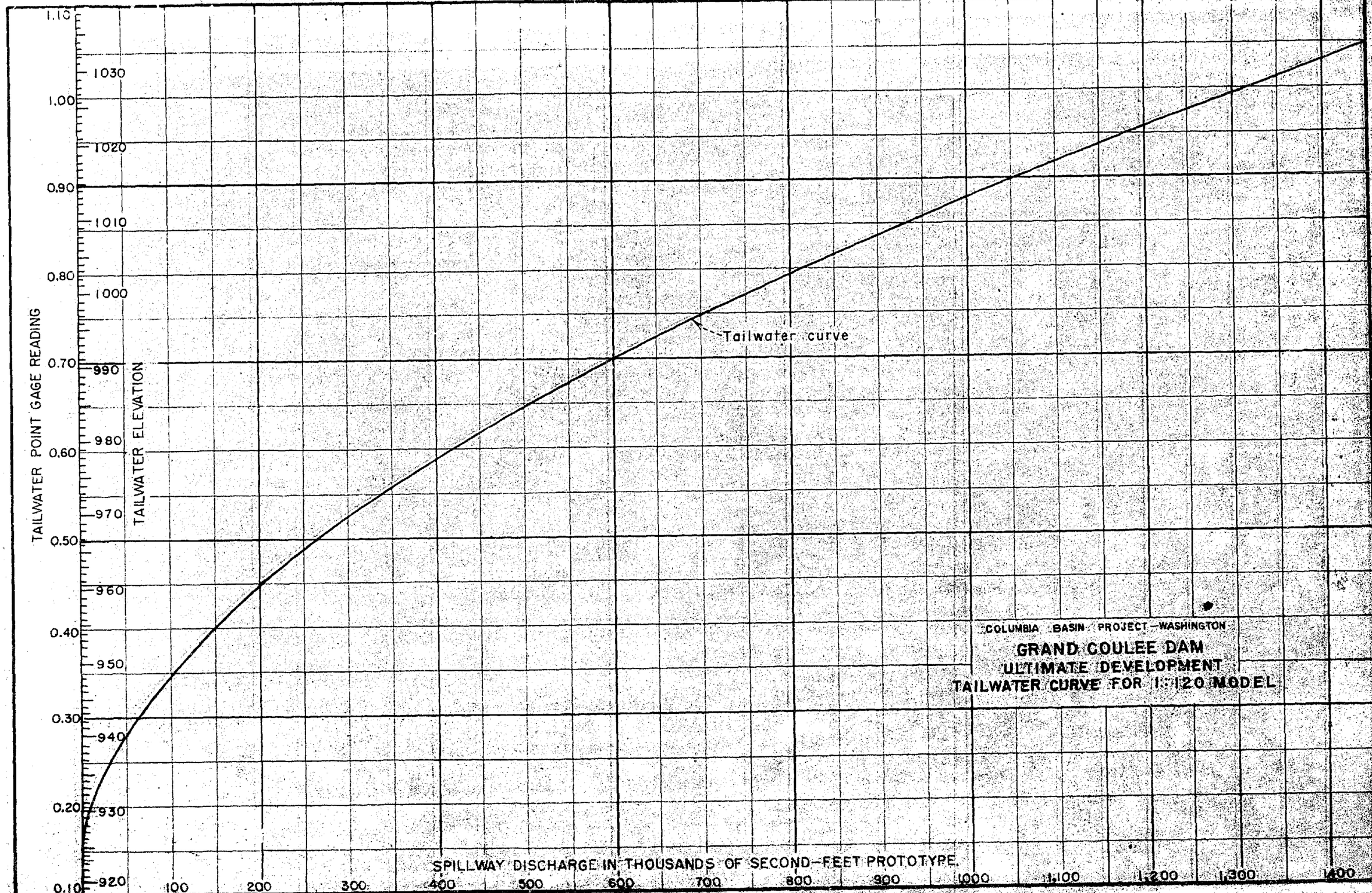
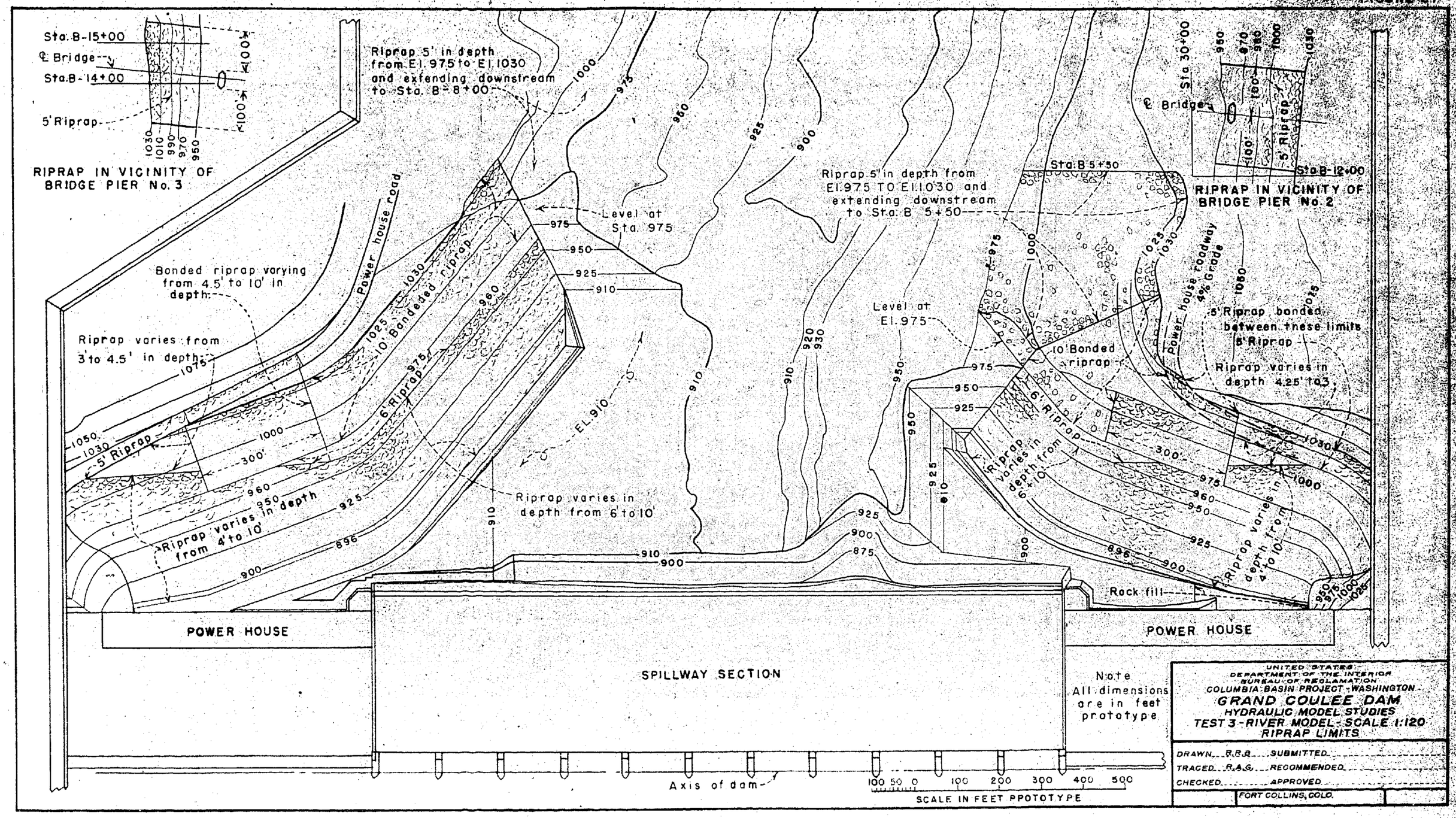

TAILWATER POINT GAGE READING
TAILWATER ELEVATION
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
1030
1020
1010
1000
990
980
970
960
950
940
930
920
Tailwater curve
SPILLWAY DISCHARGE IN THOUSANDS OF SECOND-FEET PROTOTYPE.
100
200
300
400
500
600
700
800
900
1000
1,100
1,200
1,300
1,400
COLUMBIA BASIN PROJECT—WASHINGTON
GRAND COULEE DAM
ULTIMATE DEVELOPMENT
TAILWATER CURVE FOR 1:120 MODEL

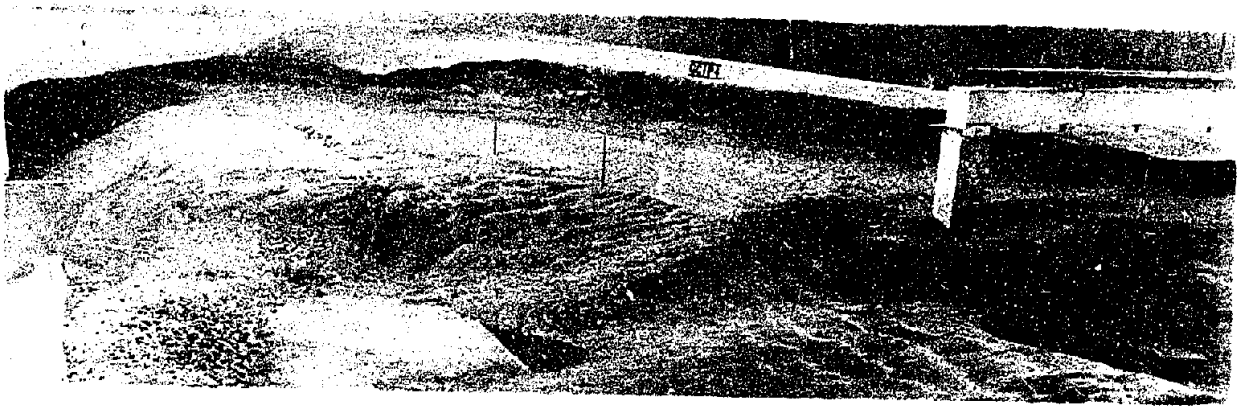
FIGURE 21


Sta. B-15+00
℄ Bridge
Sta. B-14+00
5' Riprap
100'
RIPRAP IN VICINITY OF BRIDGE PIER No. 3
Riprap 5' in depth from El. 975 to El. 1030 and extending downstream to Sta. B-8+00
Power house road
Bonded riprap varying from 4.5' to 10' in depth
Riprap varies from 3' to 4.5' in depth
10' Bonded riprap
6' Riprap
5' Riprap
300'
Riprap varies in depth from 4' to 10'
Level at Sta. 975
EL. 910
Riprap varies in depth from 6' to 10'
Riprap 5' in depth from El. 975 TO El. 1030 and extending downstream to Sta. B 5+50
Level at El. 975
10' Bonded riprap
Riprap varies in depth from 6' to 10'
Sta. B 5+50
Sta. 30+00
℄ Bridge
5' Riprap
100'
Sta. B-12+00
RIPRAP IN VICINITY OF BRIDGE PIER No. 2
Power house roadway 4% Grade
5' Riprap bonded between these limits
5' Riprap
Riprap varies in depth 4.25' to 3'
Riprap varies in depth from 4' to 10'
Rock fill
POWER HOUSE
POWER HOUSE
SPILLWAY SECTION
Axis of dam
100 50 0 100 200 300 400 500
SCALE IN FEET PROTOTYPE
Note
All dimensions are in feet prototype
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
HYDRAULIC MODEL STUDIES
TEST 3 - RIVER MODEL - SCALE 1:120
RIPRAP LIMITS
DRAWN R.R.B. SUBMITTED
TRACED R.A.G. RECOMMENDED
CHECKED APPROVED
FORT COLLINS, COLO.

BEFORE TEST.

AFTER DISCHARGE OF 500,000 SECOND-FEET.

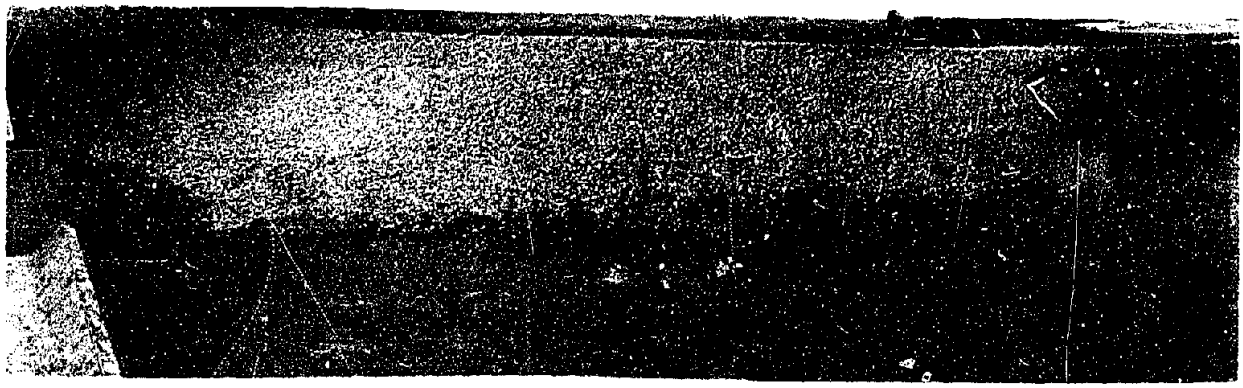
AFTER DISCHARGE OF 750,000 SECOND-FEET.



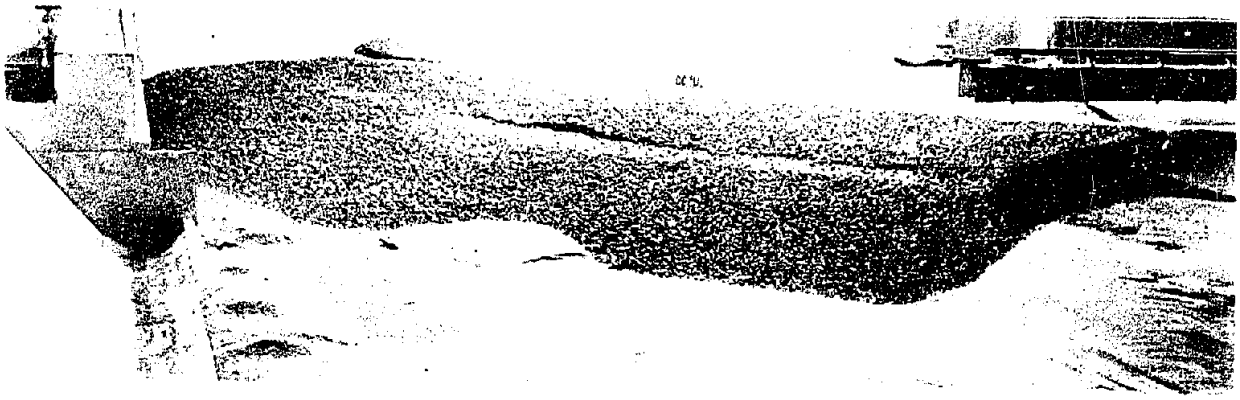
AFTER DISCHARGE OF 1,000,000 SECOND-FEET.

TEST 1

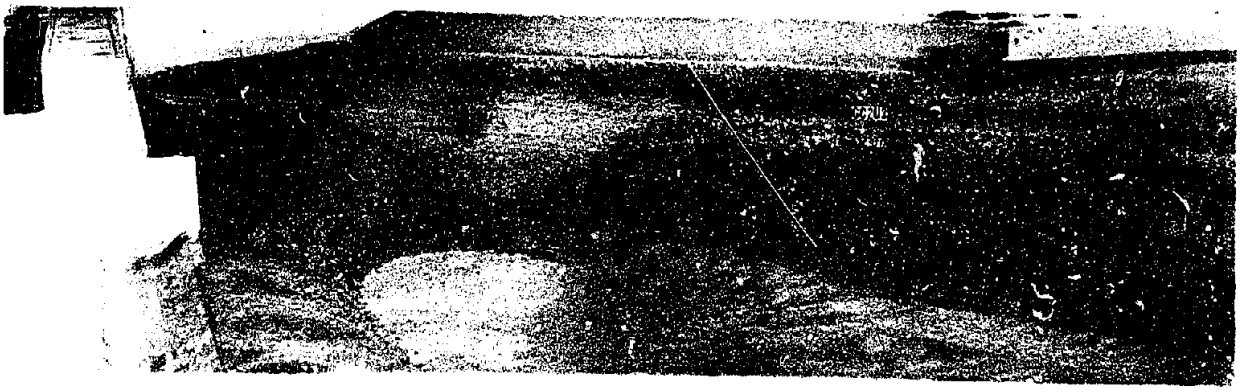
EROSION AT LEFT TAILRACE.



BEFORE TEST



AFTER A DISCHARGE OF 500,000 SECOND-FEET.

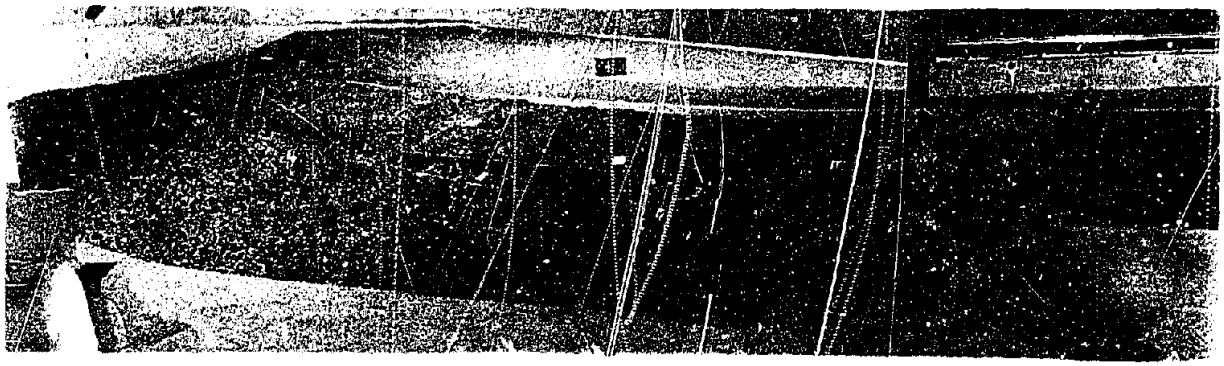


AFTER DISCHARGE OF 780,000 SECOND-FEET.

AFTER A DISCHARGE OF 1,000,000 SECOND-FEET.

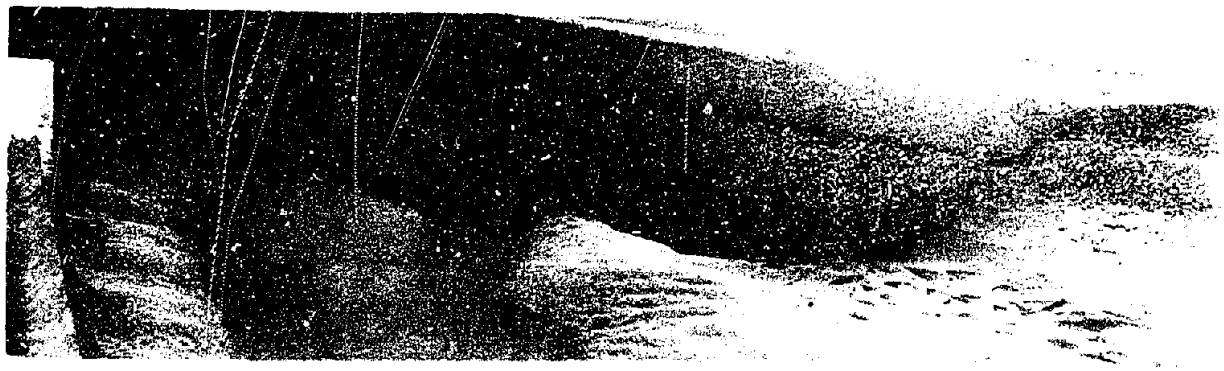
TEST 2

EROSION AT LEFT TAILRACE.



BEFORE TEST.

AFTER DISCHARGE OF 800,000 SECOND-FEET.



AFTER DISCHARGE OF 980,000 SECOND-FEET.

AFTER DISCHARGE OF 1,000,000 SECOND-FEET.

TEST 3 - EROSION AT LEFT TAILRACE.

APPENDIX IV

LIST OF PERTINENT CORRESPONDENCE

List of Pertinent Correspondence

Letter from Chief Engineer to Supervising Engineer, Coulee Dam, Washington, "Resloping of right tailrace slope - Grand Coulee Dam - Columbia Basin Project" August 23, 1945.

Letter from Supervising Engineer to Chief Engineer, "Resloping of right tailrace - Grand Coulee Dam - Columbia Basin Project," August 10, 1945.

Letter from Supervising Engineer to Denver Office, "River and tailrace excavation between right powerhouse and drydock, Grand Coulee Dam, Columbia Basin Project," June 19, 1945.

Letter from Supervising Engineer to Denver Office, "Maintenance of spillway bucket - Drydock construction article for publication - Grand Coulee Dam - Columbia Basin Project," June 12, 1945.

Letter from J. L. Savage, Consulting Engineer to Chief Engineer, "Inspection and conferences - Grand Coulee Dam," June 6, 1945.

Letter from Chief Engineer to Supervising Engineer, "Repair and maintenance of spillway bucket - Suggested method for effecting caisson seal - Grand Coulee Dam - Columbia Basin Project," May 10, 1945.

Letter from Supervising Engineer to Denver Office,"Repair and maintenance of spillway bucket - Suggested method for effecting caisson seal - Grand Coulee Dam - Columbia Basin Project," April 28, 1945.

Letter from Acting Supervising Engineer to Denver Office, "River and tailrace excavation between right powerhouse and drydock - Grand Coulee Dam - Columbia Basin Project," April 17, 1945.

Letter from Chief Engineer to Supervising Engineer, "Repairs and maintenance of spillway bucket - Suggested method for effecting caisson seal - Grand Coulee Dam - Columbia Basin Project," April 17, 1945.

Letter from Acting Supervising Engineer to Denver Office, "Repair and maintenance of the spillway bucket - Suggested method for effecting caisson seal - Grand Coulee Dam - Columbia Basin Project," March 7, 1945.

Letter from Assistant Chief Designing Engineer to Supervising Engineer, Coulee Dam, Washington, "Operation of face caisson for repairs to spillway face above elevation 902 - Grand Coulee Dam - Columbia Basin Project", February 19, 1945.

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