FIELD EVALUATION OF DROP-CHECK STRUCTURES FOR FARM IRRIGATION SYSTEMS

March 1971 ARS 41-180

Agricultural Research Service

UNITED STATES DEPARTMENT OF AGRICULTURE

CONTENTS

FIELD EVALUATION OF DROP-CHECK STRUCTURES FOR FARM IRRIGATION SYSTEMS[1]

by

A. S. Humpherys and A. R. Robinson[2]

Soil and Water Conservation Research Division

INTRODUCTION

The slope of irrigated lands varies greatly, requiring gradient control and energy-dissipating structures in most irrigation systems. Water-level control structures are also required for good water management on irrigated farms. Combination check and drop structures are often used where both types of control are needed in the same ditch. The headwall extension length and cutoff wall depth requirements for these structures vary for different soils and site conditions. Most commercial structures of a given hydraulic capacity have fixed dimensions and do not provide sufficient headwall length or cutoff wall depth for many field conditions. They also have stilling basins that are generally too narrow or too short or both. Washed-out structures and eroded ditches are a problem on many farms.

A common standard design for a stable structure at a given site is a formed, cast-in-place, concrete structure. However, because of economy and convenience of installation, prefabricated structures are frequently used. Additional design and site limitation information is needed to reduce failures of prefabricated structures. Structures are needed that provide adequate erosion and water control, economy, ease of installation, and ready acceptance by the farmer.

This study was made to obtain information to improve the design of drop and check structures for farm irrigation systems, and for use by the USDA Soil Conservation Service and other agencies in evaluating small structures. The study involved the evaluation and comparison of the field performance of various conventional and standard structures as they are commonly used. Several experimental structures were also included. The hydraulic and structural characteristics, together with installation and maintenance requirements, were observed. The amount of scour occurring upstream and downstream from each structure, together with bank erosion and undercutting, were observed through four seasons of operation.

PREVIOUS WORK

There are many designs of drop structures, and dissipation of the energy below the drop is accomplished in various ways. Some structures consist only of a bulkhead or cutoff wall with a scour hole below the notch. The scour hole may be filled with gravel or lined with small riprap to

[1]Contribution from Soil and Water Conservation Research Division, Agricultural Research Service, USDA, and the Idaho Agricultural Experiment Station, cooperating.

[2]Agricultural Engineer, Snake River Conservation Research Center, Kimberly, Idaho; Research Agricultural Engineer and Director of the Research Center until January 1969 when he became Director, USDA Sedimentation Laboratory, Oxford, Miss.

form a stilling basin. Other structures have an apron with and without cutoff walls and end sills below the drop. One commercial structure has a cofferdam drop preceding a chute or apron. The various methods of energy dissipation used by the structures generally determine their relative effectiveness in reducing the exit velocity and erosion hazard.

There has been little systematic study of the erosion-control and hydraulic characteristics of small drop-check structures for irrigation water control. Various designs have been developed from experience or from construction and prefabrication requirements rather than from research. Several publications give limited information on the design of small structures *(2, 4, 5)*[3]. Organizations such as the Soil Conservation Service have developed design manuals and charts based primarily on experience. Generally, the structures proposed are more than adequate for erosion control, since a factor of safety has been included in the design. However, due in part to the over-design, they are very expensive to construct.

Numerous studies have been made on large drop structures, spillways, and stilling basins. Some of the results from these studies apply to the smaller structures *(3, 6)*. However, studies involving structures for flows in the range of 1 to 3 cubic feet per second (c.f.s.) apparently do not exist.

This report covers the initial phase of the study, which was conducted in the field. Laboratory studies are planned for improvement and modification of the structures.

TESTING PROCEDURE

To evaluate cast-in-place and commercially available structures, a study site was obtained near Jerome, Idaho. A ditch approximately 3 feet across the top and 1 1/2 feet deep was constructed with a V-ditcher. The ditch was about 1,000 feet long and had a drop of about 16 feet. The size and shape of the ditch as originally constructed did not conform well to some of the structures. The ditch was reshaped and widened by hand after the second season. Soils were classed as loams and sandy loams, with a slight hardpan at various depths over part of the area. This hardpan may have restricted the scouring potential over part of the area. Because of livestock damage, the test area was fenced after the first season.

Thirty-one structures, representing 16 designs, were installed in the ditch with a drop of approximately 0.5 feet between each structure. Water-measuring flumes equipped with stage recorders were used to measure and record the flow. Additional flow entered the ditch at its midpoint, requiring a second flume at this location. Continuous flows at varying rates existed during each of the four seasons of observation. The maximum flow was in the range of 3 to 4 c.f.s., with an average flow between 1 to 2 c.f.s. recorded for much of each period. Since the ditch conveyed irrigation water, it was in operation from May to November each year and was dry for the balance of the year.

A description of the structures used in this study follows:

Structure No.	*Structure description*
1	Concrete, precast headwall with riprap-lined stilling basin.
2, 3	Concrete, cast-in-place, rectangular basin without end sill, standard design.
4, 5	Concrete, cast-in-place, rectangular basin with end sill, standard design.
[4]6, 17	Concrete, precast with end sill.
[4]7, 18	Concrete, precast, without end sill.
8, 9, 10	Concrete, precast, with upstream cofferdam and downstream liner section, small size.
11, 12	Concrete, precast headwall with fiber glass stilling basin.

[3] Italicized numbers in parentheses refer to Literature Cited on page 19.

[4] Because of nearby rock outcroppings, structures 17 and 18 were removed from service after the first year and, therefore, were not included in the evaluation.

Structure No.	Structure description
13, 14	Concrete, precast headwall with steel-lined stilling basin.
15, 16	Concrete, precast headwall with riprap-lined stilling basin.
19, 20	Concrete, cast-in-place, trapezoidal basin, standard design.
21, 22	Concrete-block, rectangular basin, standard design.
23	Commercial, modular steel design.
24	Commercial, modular aluminum, special design.
25, 26	Concrete-block headwall with cast-in-place trapezoidal basin, standard design.
27, 28, 29	Concrete, precast, with upstream cofferdam and downstream sidewalls, large size.
30	Commercial, modular aluminum, local design.
31	Wooden, standard design.

Photographs of each design of structure are shown in Appendix figures 9 to 26. Those structures called "standard design" were standard Soil Conservation Service structures with the designs and dimensions as specified in the SCS Engineering Handbook. The precast concrete structures were all commercially available and were widely used in the area. The commercial metal structures were also commonly used and were available in a range of sizes. Most of the structures were installed in duplicate, with the remainder installed in triplicate.

Since structure cost is important, the expense was recorded. For the prefabricated structures, the cost included the structure and the expense of installation. For those constructed in place, the cost included part of the cost of forms and all labor and materials required to complete the structure.

Because the structures were combination check and drop structures, they were operated as such. During the first year of operation, 1966, they were operated only as drop structures. With the insertion of check boards, structures in the lower one-half section of the ditch were operated as check structures during the second season. During the third season, the structures were operated alternately, 1 week as drop structures and 1 week as check structures. During the fourth season, the check boards were removed and the structures were used as drops throughout the entire season.

During the 4-year test period, observations and measurements were made both visually and physically. A complete photographic record was made periodically. Elevations were checked on each structure to determine movement and stability. Measurements were made at the beginning and end of the study to determine weighted-creep ratios and the change in creep ratios caused by erosion around the structures. Observations were also made of the durability of the structures, since they were exposed to a wide range of weather conditions. The effect of freezing and thawing on the concrete and on the concrete block structures was of particular interest.

The ditch-bottom profile was measured at periodic intervals. Cross-section measurements were made both upstream and downstream from the structures. The downstream cross-section measurements were made at 6, 18, and 30 inches in order to determine bed degradation and bank widening. These measurements were made in detail at the end of each operating season to determine relative changes.

Velocities were measured at different discharges, using a small propeller current meter to determine the relative effectiveness of each structure. Point measurements using the 0.6 depth method were made 1 foot upstream and at the end of the stilling basin. Additional measurements were made at 1-foot increments downstream from the end of the stilling basin. The measurements were used as a reference in determining which structures reduced velocities to a safe level and which structures promoted high exit velocities.

The structures were rated for economy and performance, based on measurements of cost; stability and durability; creep ratios; flow velocities; downstream bed degradation, bank widening and sloughing; and general operation. These ratings can be used as a measure for determining the adequacy and desirability of using the different types of drop and check irrigation structures.

EXPERIMENTAL DATA AND OBSERVATIONS

COSTS

One of the most important variables in the study was structure cost. Installed costs of the individual structures ranged from $13.81 to $75.00 (based on 1966 prices) and are shown in table 1. The most expensive structures, in general, were those that required forming and construction in place. Both labor and material costs of these were high. Labor costs included hand excavation and backfilling, placing and stripping of forms, pouring concrete, assembly and fabrication of the metal modular and wooden structures, and installation. Those having the highest material costs were the metal and concrete-block structures. Machine costs were incurred in excavating for the cast-in-place structures and for handling the concrete precast structures, which were too heavy to place manually. The precast concrete structures cost less because they required less material and less labor to install; also manufacturing costs were less for quantity production. The labor cost of installation was higher for most of the structures than it would be in normal practice because rocks were sometimes encountered during excavation and the field crew was inexperienced. This was particularly true with the cast-in-place structures because these were the first installations of this type made by the workers and they had not developed the techniques used by experienced personnel.

STRUCTURAL STRENGTH AND DURABILITY

Performance of the structural material was satisfactory for most structures. The following weather damage and deterioration was observed during the test period:

Structure 5: Vertical crack in wingwall.

Structure 7: Longitudinal cracking in the corners of the basin at the base of the sidewalls.

Structures 8, 9, and 10: Cracking and spalling on the top of the center section.

Structures 11 and 12: Separation of the fiber glass basins from the concrete headwalls.

Structure 21: Block cracking, some joint separation and longitudinal block cracking near the base of the concrete-block sidewalls.

TABLE 1.–Itemized cost of individual drop-check structures tested (based on 1966 prices)

Structure No.	Structure description	Materials	Cost per structure: Machine	Labor (man-hours)	Labor cost @ $2.50/hour	Total cost	Cost rank[1]
1	Concrete headwall with gravel basin	$ 7.56	$1.25	2	$ 5.00	$13.81	16
2, 3	Concrete, cast-in-place without sill	[2] 18.51	4.40	13.25	33.12	56.03	7
4, 5	Concrete, cast-in-place with sill	[2] 18.51	4.40	13.25	33.12	56.03	6
6, 17	Concrete, precast with sill	12.00	1.25	2	5.00	18.25	15
7, 18	Concrete, precast without sill	19.73	1.25	2	5.00	25.98	12
8, 9, 10	Concrete, precast with cofferdam basin	11.00	–	3.67	9.18	20.18	14
11, 12	Concrete headwall with fiber glass basin	22.00	1.25	2	5.00	28.25	11
13, 14	Concrete headwall with metal basin	23.90	1.25	2	5.00	30.15	10
15, 16	Concrete headwall with gravel basin	7.56	1.25	2	5.00	13.81	16
19, 20	Concrete, cast-in-place trapezoidal basin	[2] 18.51	4.40	12.25	30.62	53.53	8
21, 22	Concrete-block	31.01	4.40	14.5	36.25	71.66	2
23	Commercial steel, modular	47.72	–	5	12.50	60.22	4
24	Commercial aluminum, modular	62.50	–	–	[3] 12.50	75.00	1
25, 26	Concrete-block headwall, trapezoidal basin	[2] 23.30	2.20	14	35.00	60.50	3
27, 28, 29	Concrete, precast with cofferdam basin	12.85	–	3.5	8.75	21.60	13
30	Commercial aluminum, modular	41.15	–	–	[3] 8.23	49.38	9
31	Wooden	22.40	–	14	35.00	57.40	5

[1] Based on a scale of 1 (highest cost) to 16 (lowest cost).
[2] Includes prorated form cost of $2.50 per structure each time form was used.
[3] Installation cost was 20 percent of the material cost.

Structure 22: Block cracking, some joint separation and longitudinal block cracking near the base of the concrete-block sidewalls, transverse cracking of the basin floor at each end, separation of the basin floor from the sidewalls, and end-sill break.

Structure 25: Longitudinal cracking of the basin and headwall, caused by settling of the backfill material beneath the sidewalls.

Structure 26: Transverse cracking of the basin floor where it joins the headwall, and separation of the basin from the block headwall.

Structures 27, 28, and 29: Concrete spalling on the downstream face of the cofferdam.

Structure 31: Checking of the wood.

Prefabricated structures 6 and 17 were reinforced with steel. The concrete headwalls used in structures 1 and in 11 to 16 were made with an unusually thin steam-cured section, also reinforced with steel. This resulted in a headwall that was relatively lightweight and yet appeared satisfactory for the strength requirement. The only other structures using steel reinforcing were the concrete-block structures. Although no material deterioration was observed on the metal and fiber glass structures, they were less rigid than the concrete structures and, therefore, subject to greater distortion from unequal settling and compaction of the backfill.

STRUCTURE STABILITY

Piping

A weighted-creep distance is often used to evaluate the stability of a structure and its resistance to piping. Lane's weighted-creep distance is defined as the sum of all the vertical distances plus one-third of the horizontal distances along the shortest seepage path at the interface between the structure and the soil from headwater to tailwater. This value was determined for each structure and is shown in table 2 together with the structure's basic dimensions. The shortest seepage path in all cases was horizontally around one end of the headwall and along the stilling basin sidewall. This was also where all failures occurred.

Lane's weighted-creep ratio is the weighted-creep distance divided by the seepage head. A

TABLE 2.–Basic dimensions, weighted-creep distance and creep ratio for drop structures

Structure	Head-wall exten-sion	Crest length (opening width)	Up-stream cutoff depth	Toe-wall cutoff depth	Still-ing basin length	Stilling basin bottom width	Wing-wall length	End-sill height	Weighted-creep values[1]			
									Initial		End of study	
									Distance	Ratio	Distance	Ratio
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*		*Inches*	
1	20	18	[2][6]	–	[30]	[24]	–	–	9	1.5	8	1.3
2, 3	21	18	12	12	32	24	18	–	35	5.8	28	4.6
4, 5	21	18	6	6	32	24	18	3	35	5.8	30	5
6	17	18	7	–	20	18	–	3	15	2.4	11	1.9
7	18	15	8	6	19	15	–	–	18	3	7	1.1
8, 9, 10	–	[3]19	–	–	50	12 [4][18.5]	–	–	23	3.8	20	3.4
11, 12	20	18	6	6	24	[5]12	–	4	11	1.9	10	1.6
13, 14	20	18	5	6	30	[5]14	–	6	11	1.8	9	1.5
15, 16	20	18	[6]	–	[36]	[24]	–	–	9	1.5	6	1.0
19, 20	16	24	8	8	42	[5]12	16	–	41	6.8	39	6.6
21, 22	32	16	10	10	40	16	32	4	53	8.9	47	7.9
23	20	20	12	6	36	24	18	8	37	6.2	29	4.9
24	22	20	[10]	[10]	25	23	21	3/4	36	6.0	29	4.9
25, 26	32	18	4	12	36	[5]12	–	2	37	6.2	33	5.6
27, 28, 29	–	[3]22.5	–	–	24	14	–	–	22	3.7	22	3.7
30	22	15	[10]	–	20	22	–	–	18	3.0	8	1.3
31	24	18	–	–	36	18	24	6	47	7.8	42	7.0

[1] Creep path assumed horizontal around outside of headwall and stilling basin walls; seepage head = 6 inches. [2] Numbers in brackets are approximate. [3] Crest length of cofferdam; opening width is smaller. [4] Width of liner below headgate.
[5] Trapezoidal stilling basin.

minimum value of 4 *(7)* is commonly used for design purposes for loam and sandy loam soils having a clay content of 15 percent or greater. Creep ratios at the beginning and at the end of the test period are shown in table 2 for a seepage head of 6 inches. The structures constructed in place and the prefabricated structures with wingwalls had the highest creep ratios. Most of the precast concrete structures had a low creep ratio, and structures 1, 7, and 11 to 16 failed at least once during the study. These washout failures, initiated by piping around one end of the headwall, occurred several times on some structures. None of the cast-in-place structures failed by piping, whereas the only prefabricated and modular structures that did not fail at least once were 6, 8, 9, 10, 24, 27, 28, 29, and 31. None of the headwalls on the prefabricated concrete structures were of adequate length. Some of these, where failures had occurred, were lengthened with metal modular panels in 1968 to prevent further piping.

The concrete cast-in-place structures provided adequate headwall length and also provided good contact along the sides of the stilling basin between the soil and the concrete. When prefabricated basins were attached to concrete headwalls (structures 11 to 14) or precast structures were used, it was not possible to obtain good soil-to-structure contact, and thus there was little resistance to seepage flow along this interface. When wingwalls were used, it was easier to compact the backfill to obtain seepage resistance along the length of the structure. Even though the backfill was compacted around structure 23, it washed out three times during each of the seasons 1966 and 1967. Livestock damage and rodent holes were contributing factors in the failures. No failures occurred after the beginning of the 1968 season, when the structure was removed, modified, and reinstalled.

Structures installed by driving metal modular panels into the ground (structures 24 and 30) are normally very resistant to piping failures. Adequate hydraulic design, however, is still necessary to prevent failure by undermining and erosion around the structure. In rocky or gravelly soils, it may not be possible to drive the panels deep enough to prevent piping. A washout failure initiated by piping was experienced with structure 30. The headwall panels on one side of the structure were not driven deep enough and this, combined with erosion from the downstream side, caused failure.

The prefabricated structures were relatively unstable immediately after installation and were subject to piping failures because of the loosened soil condition. Even though the backfill was compacted and replaced with care, it was still more subject to piping and erosion than soil that had not been moved. Most piping failures occurred during the first two seasons. Livestock trampling around some structures contributed to the failures during the first year. Failures during subsequent years resulted almost entirely from piping caused by rodents. In the spring of each year the ditch banks were extensively damaged by mice. The structures most seriously affected were those that were prefabricated and had inadequate headwall length. These structures were also more vulnerable because of the loosened condition of the soil after installation.

Piping around prefabricated structures 8, 9, and 10 and 27, 28, and 29 was noticeably absent. These structures were installed with a backfill mixture of sand and fine gravel next to the structure. This backfill, although quite erodible, appeared to discourage rodent activity. The backfill was puddled to consolidate it and to obtain good soil-to-structure contract. If the backfill can be protected from stream erosion, this installation method might be used to advantage with other structures. The backfill material was protected from downstream erosion by rocks on structures 8, 9, and 10 and with concrete sidewalls on structures 27, 28, and 29.

Overtopping

Because of the loosened soil condition following installation, the prefabricated structures were more subject to damage from streamflow overtopping than those constructed in place. Structures are usually designed with enough freeboard that overtopping is not a problem. During a period of high flow (2.98 c.f.s.) in 1967, structures 27 and 29 were partly washed out when they were overtopped while being operated with their mated check boards adjusted to check the water 6 inches. If overtopped, these structures are subject to failure because of the erodibility of the sand and fine gravel backfill mix. Unusually large flows, however, are not common in well-regulated systems. When the cofferdam structures are used as checks, the check boards must be removed from the bottom structures first to prevent water "pileup" and overtopping by the large flows that would otherwise occur.

Structure Movement

Elevations were taken at different points on the structures in November 1966, and again in July 1969, after the structures had been in place for three winters. Elevation differences during this period indicate the amount of settling, tilting, or frost heave. The changes in elevation are shown in table 3. Except for structures 24 and 30, which show a slight rise in elevation, the amount of structure movement is negligible. With the exception of structure 24, there was no significant difference in movement between the prefabricated, cast-in-place, or metal modular structures.

STREAMFLOW

Streamflows through the test channel varied during each irrigation season. Flows in the range from 2.5 to 4 c.f.s. occurred once or twice each season but were usually of short duration, lasting only from 1 to 4 days. The average daily flow and the number of days each month that a measurable amount of water flowed in the test ditch for the 1966 to 1968 seasons are shown in table 4. Flow measurements during 1969 were not made; however, the flow was approximately the same as during the three previous seasons. Because the test ditch was a waste channel at the end of a distribution lateral, the flow varied considerably with intermittent periods during which no flow occurred.

TABLE 3.—Changes in structure elevations between November 1966 and July 1969[1]

Structure No.	East headwall or cofferdam	West headwall or cofferdam	Basin upstream	Basin downstream
	Feet	*Feet*	*Feet*	*Feet*
2	-0.01	-0.02	-0.01	-0.01
3	- .03	- .02	- .01	- .02
4	- .01	+ .01	- .02	- .02
5	+ .01	.00	.00	- .01
7	- .04	.00	.00	+ .01
8	+ .01	+ .01	.00	.00
9	+ .01	- .02	.00	.00
10	+ .02	+ .03	+ .01	.00
11	.00	.00	+ .07	+ .04
12	+ .02	+ .03	+ .03	03
13	+ .02	+ .02	–	.02
14	+ .01	+ .02	–	+ .02
15	+ .02	+ .03	–	–
16	+ .04	+ .03	–	–
19	+ .02	+ .02	+ .01	.00
20	+ .01	+ .01	.00	.00
21	- .04	+ .05	+ .01	+ .01
22	- .06	- .03	+ .01	+ .01
24	+ .14	+ .11	+ .14	+ .12
25	.00	- .01	+ .01	+ .01
26	+ .02	- .02	+ .02	+ .03
27	- .01	+ .02	+ .01	.00
28	+ .02	+ .01	+ .02	.00
29	.00	+ .01	+ .01	+ .01
30	+ .01	+ .04	+ .02	+ .06
31	+ .02	+ .03	+ .01	+ .01

[1] Data for structures 1, 6, 17, 18, and 23 are not shown because these were removed or repositioned during the test period.

TABLE 4.–Average daily flow and number of days each month that water flowed through drop-check structures for three seasons

Month	1966		1967		1968	
	Average daily flow	Days of flow	Average daily flow	Days of flow	Average daily flow	Days of flow
	C.f.s.	*No.*	*C.f.s.*	*No.*	*C.f.s.*	*No.*
May	0.61	9	–	–	0.37	18
June	1.13	22	1.57	30	1.63	30
July	.30	21	.68	30	.46	27
August	.43	17	.82	21	.88	25
September	1.42	24	.78	27	.91	24
October	1.51	31	.94	30	1.65	31
November.	0.55	5	1.50	26	1.50	5

VELOCITY MEASUREMENTS

Velocity measurements were made at the center line near one structure of a pair or group and are representative of the flow through that particular type structure. Because the structures were normally used both with and without check boards, measurements were made for each condition. In general, the measurements were made 1 foot upstream from the structure, at the end of the stilling basin or liner section, 1 foot downstream, and 2 feet downstream. The measured velocities at three different flow rates for each type structure are shown in table 5. These measurements were made to indicate, qualitatively, the relative performance of the structures.

The downstream velocity at the end of an effective stilling basin or liner section should be low and generally less than that approaching the structure. In actual practice, these structures are used as checks with the tailwater checked to an increased depth by the next structure downstream, only when the ditch is serving as a head ditch for direct irrigation. Therefore, they normally operate for longer periods of time as

TABLE 5.–Velocity measurements above and below drop structures for different discharges and check board conditions

Structure No.	Measured velocity											
	Without check boards Flow = 0.84 c.f.s.				Without check boards Flow = 1.40 c.f.s.				With check boards Flow = 1.59 c.f.s.			
	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)
	Ft./sec.				*Ft./sec.*				*Ft./sec.*			
2	–	–	–	–	1.51	2.54	1.62	1.60	–	–	–	–
3	1.29	3.05	1.42	1.39	–	–	–	–	–	–	–	–
5	1.57	1.53	0.83	0.93	1.59	1.94	1.48	1.09	–	–	–	–
6	–	–	–	–	1.80	1.18	2.00	–	–	–	–	–
7	–	–	–	–	2.22	6.01	3.91	2.78	–	–	–	–
9	1.48	3.66	2.78	2.80	1.84	5.18	2.90	3.36	0.87	0.80	1.14	1.13
11	1.31	0.58	0.57	0.31	2.04	0.61	0.52	0.33	–	–	–	–
14	1.27	0.43	0.33	0.26	1.76	1.06	1.16	0.59	–	–	–	–
15	1.17	–	[5] 1.59	[6] 0.79	1.46	–	[5] 2.19	[6] 1.17	–	–	–	–
20	1.25	1.59	1.25	1.17	1.67	1.35	1.39	1.39	0.85	1.82	1.22	0.90
21	1.55	1.39	1.39	0.95	2.01	2.16	2.31	1.16	0.93	0.93	2.07	1.17
23	1.65	0.47	1.46	0.41	2.05	0.63	0.56	0.46	0.85	0.53	0.34	0.50
24	0.86	1.06	–	0.55	1.20	2.80	1.59	1.11	0.71	0.74	1.52	0.93
25	1.54	0.76	0.57	0.34	1.67	1.48	1.04	0.69	0.89	0.50	0.49	0.31
28	1.42	3.19	2.65	1.20	1.85	3.77	2.54	1.51	0.57	0.63	1.48	1.02
30	–	–	–	–	2.63	–	2.04	–	–	–	–	–

[1] Measurement taken 1 foot upstream. [2] Measurement taken at the end of basin over end sill. [3] Measurement taken 1 foot downstream. [4] Measurement taken 2 feet downstream. [5] Measurement taken 3.8 feet downstream. [6] Measurement taken 5 feet downstream.

drops, without check boards. When operating as drop structures, the downstream water depths are shallower, resulting in higher exit velocities that are more conducive to degradation of the stream channel. There was a reduction in velocity through most of the structures as noted in table 5. At the lower flow without check boards, structures 3, 9, and 28 all had high exit velocities. At the higher flow without check boards, structures 2, 7, 9, 15, 21, 24, and 28 all had high exit velocities; those for 7, 9, and 28 were very high at the end of the basin or liner section. None of the structures with high exit velocities had end sills, except for 21. At both flows without check boards, the velocity below structure 9 remained high for a distance of at least 2 feet downstream. The high velocity below structure 28 did not persist for as great a distance and was greatly reduced within the first 2 feet. The high velocity below structure 15 occurred near the end of the basin where some of the gravel tended to accumulate. The soil at this location did not erode, however, because of protection provided by the gravel material. When the structures were operating as checks with an increased depth downstream, the exit velocity from most structures, at the indicated flow rate, was reasonably low. For all conditions and flows, the velocity 1 foot downstream from structure 21 remained relatively high. Because of turbulence over the end sills, it was difficult to obtain an accurate measurement of average velocity at that point. The measured velocity downstream from some structures having end sills was higher than the measured velocity over the sill, due to the hydraulic conditions.

The maximum permissible or nonerodible velocity is the highest velocity that will not cause erosion of the channel bed. It varies with soil texture, soil structure, and other factors. The following tabulation showing maximum permissible velocities for selected soil conditions is given by Chow *(1)*:

Soil	*Maximum permissible Velocity, Ft./sec.*
Fine sand	1.50
Sandy loam	1.75
Silt loam	2.00
Clay	3.75
Shale and hardpan	6.00

For soils at the test site, velocities of more than 2.0 feet per second should not be allowed, particularly near a control structure. Without a hardpan condition, velocities greater than 2.0 feet per second would result in downstream erosion, and the use of riprap and gravel for channel stabilization would be necessary. Many of the structures evaluated were questionable from an erosion-control standpoint because of high exit velocities.

HYDRAULIC PERFORMANCE

Stilling Basins

The purpose of a stilling basin is to dissipate the excess energy of flowing water for downstream channel protection. If water leaves the basin with a high velocity, energy dissipation continues over unprotected soil in the downstream channel and contributes to channel degradation. The exit velocity from a stilling basin is influenced by the basin geometry. Although data were not obtained to design the length of a stilling basin, it was observed that, in general, those structures in the study having stilling basins less than 30 inches in length, did not provide adequate stilling of the water before it left the structure.

Structures with narrow stilling basins tend to confine the flow, and this results in high exit velocities. This was particularly noticeable for those structures that did not have end sills or were not set below grade to provide extra tailwater depth (structures 7, 8, 9, 27, and 28). The effect of high velocity downstream from these structures and from structures 21 and 22 was indicated by the absence of moss growing in the bottom of the ditch for some distance downstream. Structures 10 and 29 operated with a greater tailwater depth and consequently had a lower exit velocity than their counterpart structures. The high velocity through the cofferdam structures (structures 8, 9, 10, and 27, 28, 29) was caused by the narrow throat width below the cofferdam; this contracted the flow and accelerated it as shown in figure 1. At the higher flows within the design range of these structures, the control shifted from the cofferdam crest to

Figure 1.–Photograph showing high-velocity, accelerated flow through the narrow opening of a cofferdam-type structure at 91 percent (1.65 c.f.s.) of design flow.

the narrow opening. A considerable part of the total drop in water surface occurred as the water flowed through this opening rather than in the cofferdam drop. Therefore, the relatively long cofferdam crest was nullified by the contracted opening downstream, and the effective length of the structure was reduced for stilling purposes. Sidewalls were used on structures 27, 28, and 29 to protect the downstream channel banks and to prevent erosion of the sand- and-gravel backfill material next to the structure walls. The sidewalls increased the effective length of the basin; however, the channel bed between the sidewalls was still exposed to high velocities when the structures were operated as drops with low tailwater levels. The sidewalls need to be installed relatively deeply to prevent their being undermined. Gravel or rock placed between them would help protect the bed.

Certain conditions seem to favor the occurrence of the wall attachment or Coanda effect. This occurs in structures having relatively narrow openings between two confining walls in which a high velocity fluid flows. Because of the high velocity near the wall, a pressure differential exists across the jet that tends to attach the stream or hold it close to the wall. This effect was observed under certain conditions (fig. 2) with structures 8, 9, and 10, which have relatively narrow openings and a liner or basin long enough for the effect to develop. This is an undersirable feature in a drop or check structure because the high-velocity flow leaves the structure on one side or the other. This tends to accelerate bank undercutting and erosion more than if the maximum velocity were confined to the center of the channel. In addition to undercutting by the high-velocity stream impinging on one bank, a secondary circulation or reverse current is established near the opposite bank. The stream is usually directionally unstable and may be changed from one side to the other by a disturbance in the channel deflecting it to the opposite side. The Coanda effect was not observed with structures having wide basins and low exit velocities.

Figure 2.–Photograph showing the Coanda or wall attachment effect.

Trapezoidal and rectangular stilling basins each have advantages. A trapezoidal basin without a headwall does not obstruct ditch-cleaning equipment, and a tractor-mounted ditcher can pass through it. The basin can also be constructed without expensive forming by placing concrete directly on the soil in the ditch. With a lower tailwater, however, the performance of a trapezoidal basin may be poor because the narrow bottom contracts the flow, resulting in high exit velocities. Structure 19 performed poorly and had high exit velocities because of shallow tailwater, while structure 20 operated with a higher tailwater and gave very good performance. Both were effective in stilling the flow when operated as checks with adequate tailwater. As water entered the basin, the nappe clung to the wide crest and water spilled down each sloping side towards the center. Flow meeting in the center of the basin resulted in a strong mixing action and turbulence near the upper end of the basin. This effectively stilled the water before it left the structure. The downstream channel must be shaped to match the basin to prevent bank undercutting.

The stilling basins of structures 1, 15, and 16, which were lined with coarse gravel or small riprap, were effective in providing good water control and energy dissipation. There were also the most economical of any structures installed. This type of basin requires relatively flat side slopes for the rock to be stable. Because of this, a wide basin immediately below the structure is needed. The basins of these structures were shaped with 2:1 side slopes. At this slope, the rock stayed in place if it was not disturbed; however, when disturbed by livestock, it tended to move to the bottom and end of the basin. One disadvantage of this basin is that it is subject to damage when livestock are present. This structure requires an extra long headwall with a deep cutoff to prevent piping and to obtain a satisfactory creep ratio.

Considerable turbulence occurred at the downstream end of all structures having end sills where a horizontal roller formed. Because of the roller, the water surface elevation directly over the sill and immediately downstream was higher (fig. 3). This altered the velocity distribution

Figure 3.—Turbulence and lateral velocity currents at the downstream end of a structure having an end sill.

across the channel and caused the water to flow laterally and to impinge upon the sides of the ditch resulting in bank undercutting and erosion. Thus, channel widening occurred immediately downstream from those structures that had end sills. Visual observations indicated that turbulence over the sill persisted even with an increase in tailwater depth when the structures were used with check boards. Structures of this size may possibly be more effective without end sills if some other means is provided to prevent high exit velocities from the basin. Relatively wide basins without end sills performed well when the tailwater was great enough to dissipate the excess energy by providing a pool or by creating a hydraulic jump within the basin, (structures 2, 3, 8, 9, 10, 19, and 20). When there was not sufficient tailwater within the liner of structures 8, 9, 10, the flow discharged onto the soil surface downstream with a high velocity that had considerable erosion potential. The liner appeared to provide sufficient length to contain the jump when one formed. The short basin of structure 7, however, was not long enough to contain the jump which usually occurred directly over the streambed.

End sills varying from 2 to 8 inches in height were used. The 2-inch sill on structures 25 and 26 appeared as effective as higher sills on other structures and often exhibited less turbulence than the 6- and 8-inch sills. The upstream side of one sill (structure 6) was sloping rather than vertical. This gave a "flip bucket" effect to the flow as it left the basin and tended to increase the amount of turbulence.

Metal modular structure 23 (see Appendix, fig. 21) originally had a narrow opening and a narrow stilling basin. At the beginning of the 1968 season, it was removed and rebuilt to provide a wider and longer basin with a longer crest. The scour hole that had developed downstream during the first two seasons was filled. Extending the crest length and basin width did not entirely correct its faulty performance. Flanges on the vertical edges of the modular components protruded into the stream and apparently altered the pressure distribution near the end of the basin. At some discharges, the water was observed to flow almost 90° laterally at the downstream end of the structure. The high end sill in structure 23 also appeared to contribute to strong turbulence, which persisted even with an increased tailwater depth. Because of the lateral velocity currents, considerable erosion and channel widening continued downstream after the original scour hole was refilled. This structure washed out several times during the course of the study, and considerable maintenance was required around the ends of the wingwalls.

The stilling basins for metal modular structures 24 and 30 were too short to provide adequate energy dissipation. The apron of structure 30 was extremely short, and strong turbulence occurred downstream over the unprotected soil; however, an apparent hardpan in this area reduced the scouring. This structure was designed and installed by the local dealer as it is normally used in the area. Consequently, the apron was very short and wingwalls were not provided. Erosion occurred beneath the basin and behind the sidewalls extending upstream to the headwall. This caused the structure to fail by undermining one side of the headwall. A construction flange at the end of the basin on the bottom and sides of structure 24 appeared to contribute to the formation of eddy currents similar to those at the end of structure 23. Vortices and eddy currents tended to form at the downstream corners of all basins having 90° wingwalls, but they were stronger and more pronounced with basins having sills and protruding flanges. Because the metal panels are driven into the soil, structures similar to 24 and 30 are normally resistant to piping and washouts.

After the study was initiated, a culvert was installed downstream from structure 31. This resulted in an increased tailwater depth, and the structure operated partly submerged except when the flow was very small. Even though partly submerged much of the time, the narrow basin, combined with turbulence over the end sill, caused considerable erosion and stream channel widening immediately downstream.

Crest

The channel bottom upstream from the cofferdam structures must be as wide as or wider than the crest. Otherwise, water flows into the cofferdam from the sides, and the channel banks upstream become undercut and slough into the ditch. When these structures are used as checks, the cofferdam is not used and stilling occurs within the short throat section or the liner section. As noted previously, the cofferdam was not very effective at the higher range of flows for which these structures were designed, because the control shifted from the crest to the narrow opening downstream.

Structures with a very wide opening or long effective crest length, such as trapezoidal structures 19 and 20, have a smaller head or water depth over the crest. This results in the water entering the basin at relatively low velocities, requiring a shorter stilling basin. Because the nappe of these structures normally was not aerated, the water plunged into the stilling basin close to the upstream headwall. For this condition, it appeared that greater stilling was accomplished than for flow having a higher velocity and a nonaerated nappe entering the midportion of the basin. The nonaerated nappe observed with structures 21 and 22, which have narrow basins, may account for the good stilling in these structures. The nappe entered the basin close to the headwall, resulting in a longer effective basin length. An undesirable condition may exist, however, if the nappe should become alternately aerated and nonaerated. The nappe on the above structures was always observed nonaerated. Normally, a nonaerated nappe is undesirable; however, for small structures the disadvantages of a nonaerated nappe may not be significant. Because of the relatively low heads and velocities involved, negative pressures should not be great enough to cause cavitation.

SCOUR AND EROSION

Where soils are uniform, scour volumes downstream from a series of structures depend on the exit flow velocities and the effectiveness of the control structures. Channel cross-section measurements were made at three locations below the structures at the beginning of the 1968 season and again at the end of the 1968 and 1969 seasons. The stream channel cross sections 18 inches downstream from each structure are shown in Appendix figures 27 to 31. The volume of soil eroded between 6 and 30 inches downstream from each structure is shown in table 6. The average scour depth and change in channel width for the three downstream measuring stations are also shown in table 6. Cross-section measurements made during the first 2 years were not used because of the variability in channel size in relation to some of the structures. The narrow ditch resulted in an unusually large amount of erosion and bank undercutting downstream from structures having extra-wide stilling basins. Also, considerable bank undercutting occurred upstream during the first season from those structures having cofferdam stilling basins. Consequently, large pieces of soil were dislodged from the sides of the ditch. After the channel was reshaped, bank undercutting upstream did not occur and undercutting downstream was not so severe. During the second

TABLE 6.—Erosion volume, average scour depth, and average increase in channel width between 0.5 and 2.5 feet downstream from drop-check structures during a 2-year period

Structure No.	Erosion volume	Average scour depth	Average increase in channel width
	Cu. ft.	*Ft.*	*Pct.*
1	1.72	0.17	23
2	2.89	.49	0
3	2.32	.37	10
4[1]	2.28	.06	34
5[1]	3.31	.21	42
6[1]	3.48	.19	51
7	3.83	.44	17
8	3.70	.44	23
9	3.19	.44	26
10	2.99	.47	25
11[1]	3.91	.24	58
12[1]	4.04	.26	37
13[1]	4.18	.18	47
14[1]	3.55	.16	34
15	0.85	.00	15
16	2.00	.00	15
19	5.75	.55	34
20	2.20	.10	45
21[1]	4.17	.28	65
22[1]	5.25	.40	65
23[1]	7.74	.58	60
24	6.14	.65	30
25[1]	3.67	.29	44
26[1]	5.66	.43	48
27	4.67	.51	20
28	4.18	.44	24
29	2.41	.26	21
30	4.47	.47	45
31[1]	5.81	.20	19

[1] Structures with end sills.

season, when structures 19 to 31 were operated as checks, considerable sediment accumulated upstream from each structure. When the structures were operated alternately as checks and drops the next year, these deposits did not occur.

Two general scour patterns, related to the presence or absence of an end sill, were observed. Downstream channel widening was generally associated with those structures that had end sills, while degradation of the channel bed was associated with those structures which did not have end sills. Structures without sills generally had the greatest scour depth and usually had the highest exit velocities, as shown in table 5. A typical channel cross section 18 inches downstream from a structure having an end sill is shown in figure 4 and for a structure without an end sill in figure 5. Corresponding photographs of these structures are shown in figures 3 and 6.

Disregarding those structures having a riprap-lined basin and structures 17 and 18, which were removed from service, the remainder of the structures were equally divided between those that had end sills and those that did not. They were randomly intermixed throughout the length of the ditch so that the effect of soil variations was minimized. The sum of the scour volumes and scour depths for both groups of structures was determined. The average scour per structure for each group at the end of the 1968 and 1969 seasons is shown in the following tabulation:

	Structures with end sills		*Structures without end sills*	
	1968	*1968-69*	*1968*	*1968-69*
Average scour volume per structure . . . . cu. ft.	3.9	4.4	3.0	3.8
Average scour depth per structure . . . . . . ft.	.24	.27	.39	.43

The above data indicate that, during the 2-year period, about 16 percent more total soil loss occurred in the first 2.5 feet downstream from structures having end sills. On the other hand,

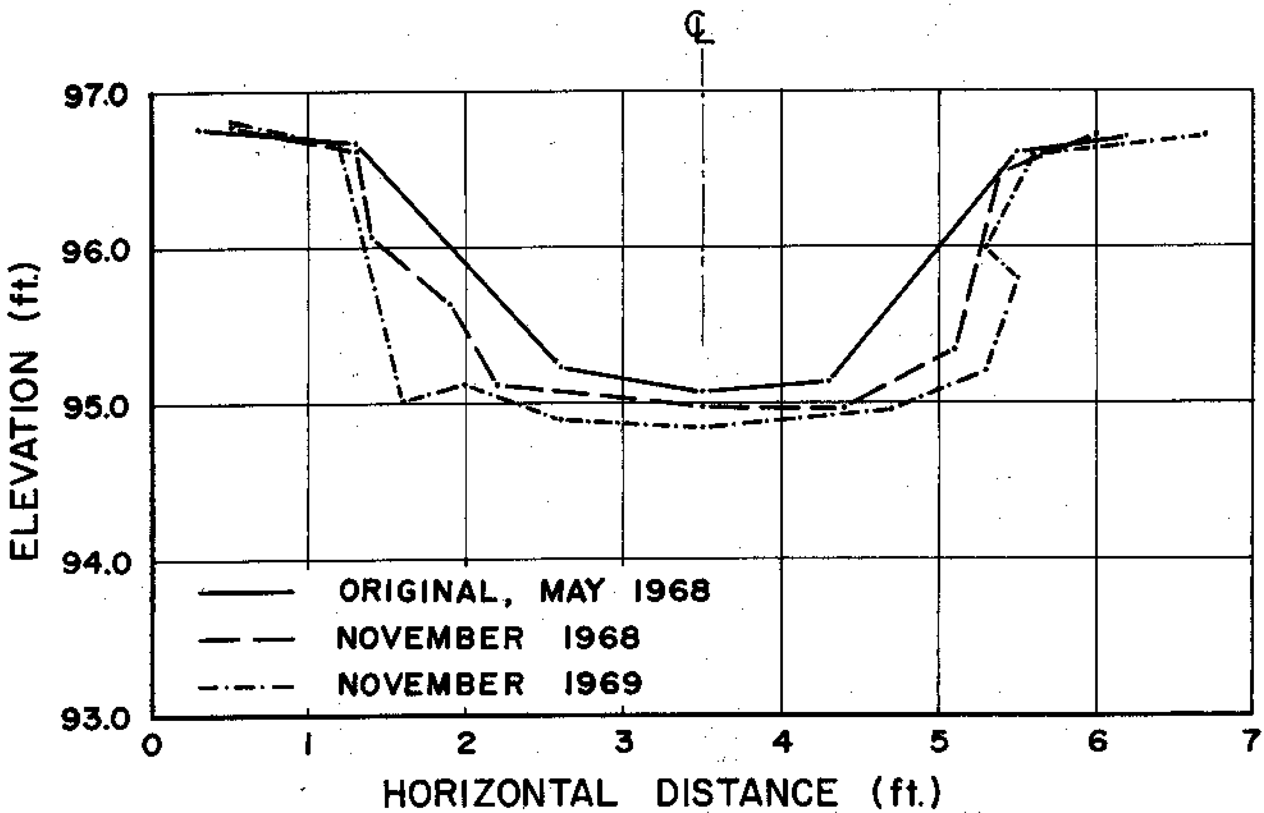


Figure 4.–Channel cross section 18 inches below a structure with an end sill, No. 6.

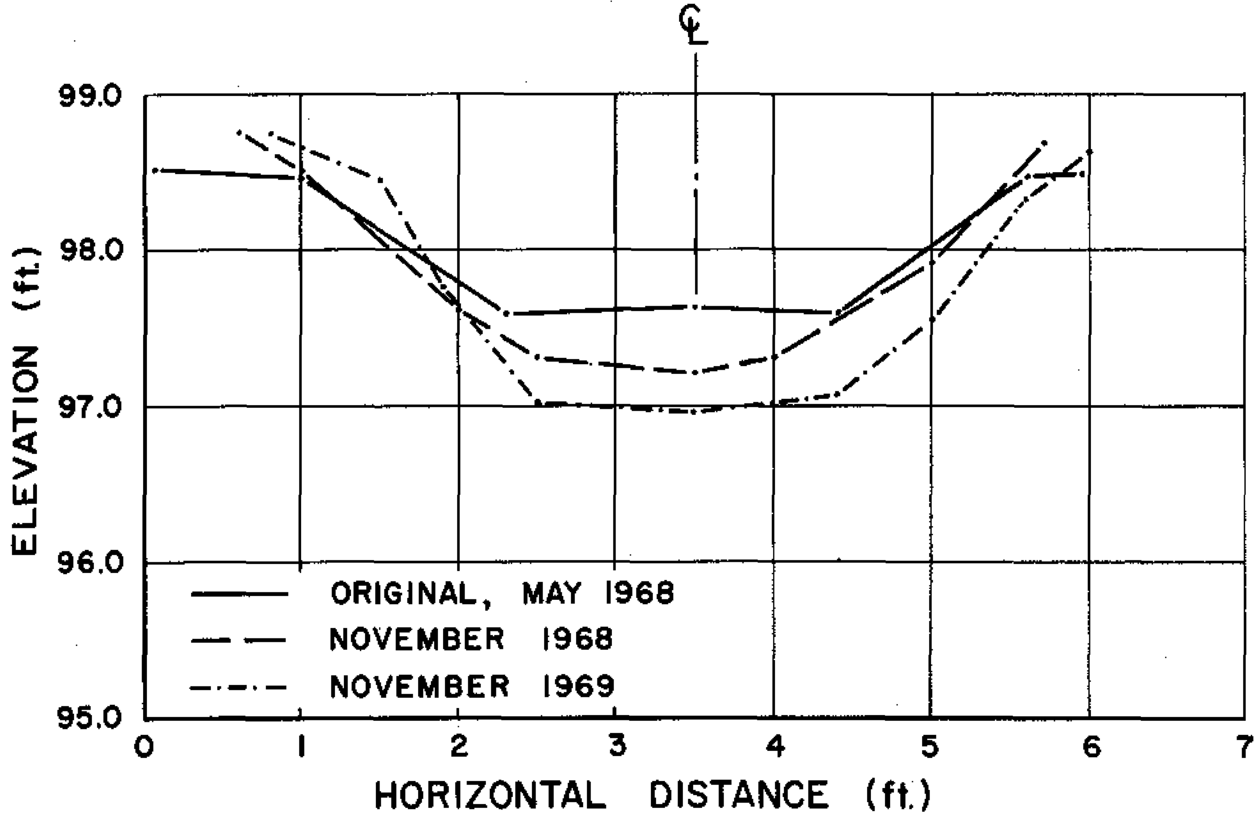


Figure 5.–Channel cross section 18 inches below a structure without an end sill, No. 2.

Figure 6.–Flow in a standard rectangular structure without an end sill.

the data show that the average scour depth downstream from structures without end sills was about 62 percent greater. For those structures without an end sill, there was a direct qualitative relationship between scour volume and the average velocity at the end of the basin, as represented by the average velocity measured with a flow of 1.4 c.f.s., shown in figure 7. Scour volume and average velocity at the end of the basin for structures that had end sills were only slightly related.

The difference in tailwater depth noted previously between structures 19 and 20 accounts for the large difference in scour depth and erosion volume for these structures, which are of the same design. The difference in scour volume and performance is significant and emphasizes the importance of adequate tailwater depth for this type of structure. Channel degradation below structure 19, which had shallow tailwater, is shown in figure 8.

Erosion upstream from the crest of most structures was observed. This was sometimes attributed to the loosened condition of the soil during installation. However, erosion also occurred upstream from the crest of structures 24 and 30, where metal modules or panels were

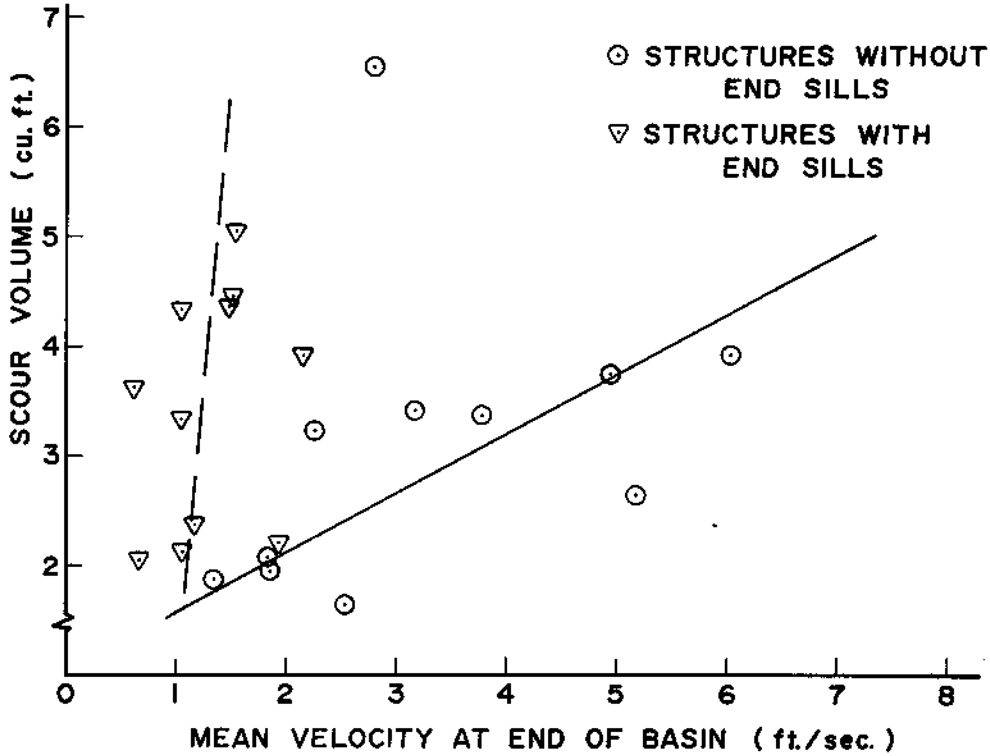


Figure 7.–Qualitative relationship between scour volume and average velocity at the end of the basin for structures with and without end sills. The average velocity shown is that measured with a flow of 1.4 c.f.s.

Figure 8.–Channel degradation below a trapezodial structure that operated with a very shallow tailwater depth.

driven into the ground without disturbing the soil.

Those structures having riprap-lined stilling basins had the least amount of scour of all structures in the test study. Some soil movement and widening of the riprap basins resulted from livestock trampling along the sides.

DISCUSSION

The hydraulic and erosion-control performance of many of the structures included in this study needs to be improved. The structures performed better when used as checks with extra tailwater depth than when used as drops without additional tailwater. This was particularly true with those structures that did not have end sills. Setting a structure below grade or increasing the effective tailwater depth is a possible alternative to the use of an end sill when relatively wide basins are used. It may be possible, by changing the stilling basin design, to overcome some of the disadvantages of end-sill turbulence. The stilling basin of most structures tested was too short to contain turbulence created by the drop. There did not appear to be a consistent relationship between the height of an end sill and scour volume; however, turbulence over the highest end sills (6 and 8 inches) appeared greater than that over low sills. The cast-in-place structures were more stable. None of these failed by washing out, whereas many of the prefabricated and modular structures failed during the study.

The costs reported are those incurred at the time of installation (1966) and are not adjusted for subsequent price changes. There appears to be a tendency to underdesign metal structures to keep their cost more competitive with prefabricated concrete structures. Because of this, the metal structures are sometimes poorly designed.

Cost and performance ratings for each type of structure are shown in table 7. The performance ratings were determined from visual observations and experimental data collected during the study. The ratings, based on a scale from 1 (unsatisfactory) to 10 (excellent), were made on the structures with the dimensions as shown in table 2. In some cases, a particular structure might possibly perform better and be rated higher if it had more adequate dimensions than those used in this study. For example, the concrete-headwall structures 11 to 16 rated very low for stability because inadequate headwall length and cutoff wall depth resulted in a low creep ratio. Because of this, washout and piping failures occurred. Longer dimensions would have resulted

TABLE 7.—Ratings of drop-check structures[1]

Structure No.	Cost	Structural adequacy	Stability	Hydraulic performance		Erosion control	Practical aspects	Average rating	
				Drop	Check			Drop	Check
2, 3	3	9	9	7	8	7	6	6.8	7.0
4, 5	3	9	9	8	8	7	6	7.0	7.0
6	8	8	5	5	7	6	7	6.5	6.8
7	7	6	4	3	6	5	5	5.0	5.5
8, 9, 10	7	7	7	5	8	6	8	6.7	7.2
11, 12	7	7	3	6	7	5	5	5.5	5.7
13, 14	6	7	3	7	8	5	5	5.5	5.7
1, 15, 16	9	7	3	8	9	9	7	7.2	7.3
19, 20	4	9	9	5	9	5	7	6.5	7.2
21, 22	2	6	10	7	8	4	5	5.7	5.8
23	3	7	7	6	7	2	5	5.0	5.2
24	2	8	8	5	7	3	6	5.3	5.7
25, 26	3	6	9	7	9	4	5	5.7	6.0
27, 28, 29	7	8	7	5	8	5	8	6.7	7.2
30	5	6	3	4	6	5	6	4.8	5.2
31	3	6	8	7	7	3	6	5.5	5.5

[1] Rating Scale: 1.—Unsatisfactory. 2.—Very poor. 4.—Poor. 6.—Fair. 8.—Good. 10.—Excellent.

in better performance and a higher rating. The cost rating assigned was determined from the cost range within which each structure fell. The total structure costs from the lowest to the highest were divided into ten price ranges, with a scale number assigned to each range. The scale number for a given structure's price range is the rating for that structure.

Structural strength and durability was determined qualitatively by taking into consideration such things as the amount of structural maintenance required, steel reinforcing, durability of the materials, the amount of freeze-thaw damage, and the susceptibility of a structure to such damage.

The rating for structure stability was based on creep ratios. The total creep ratio range for all structures as given in table 2 was divided into 10 segments, with a scale number assigned to each. The scale or rating number for each structure was then adjusted up or down depending upon its field performance. Consideration was given to whether or not the structure washed out and its susceptibility to failure by piping and overtopping.

The hydraulic performance rating was determined with the structure used as a drop and as a check. The velocity 1 foot downstream from each structure was used as a base for this rating. The range of measured velocities was divided into 10 segments, the same as with previous parameters; the scale number for the velocity associated with each structure was used as the rating. This was then adjusted up or down depending upon the structure's field performance, using visual observations and considering the stilling basin design with the dimensions shown in table 2. For example, structures 21 and 22 had a high downstream velocity, which gave a low rating. However, because of the extra-long stilling basin and the good stilling that normally occurred, the rating was adjusted upward; conversely, the rating for structure 30 was adjusted downward because of the very short apron.

The same procedure was followed in rating the structures for their erosion-control performance. The erosion volume, as determined from downstream channel cross-section measurements, was used as the base for assigning the rating numbers. The rating for practicality was somewhat arbitrary and was based upon the amount of maintenance required; obstruction to ditch-cleaning equipment; utility of operation; and farmer acceptance based on the availability, convenience, and ease of installation.

The overall rating is an average of all the ratings for the various parameters given in table 7. Generally, a low rating in one category is offset by a higher rating in another category, so that the average structure rating falls within a rather narrow range.

These small structures usually operate for longer periods of time as drops than as checks. For this reason, more emphasis should be placed on the average rating as drops as given in table 7. All of the structures rated lower than good; most rated from fair to poor.

With the cost rating excluded, the concrete cast-in-place structures have the highest overall performance rating. The concrete-headwall structures with gravel-lined stilling basins have a high overall rating; however, this would have been higher had the headwall been longer with a deeper cutoff. Most of the prefabricated structures would also receive a higher rating if they had longer and wider stilling basins and longer headwalls to insure against piping.

In applying the ratings, those in the individual cost and performance categories will normally receive the greatest attention. The average rating gives equal weight to each individual rating. However, in a particular field situation one category may be more important than another and may be the dominant criterion. The hydraulic performance will usually be very important; however, in some cases structure cost or convenience aspects may be the deciding factors between alternative structures; or, soil conditions may require that more consideration be given to the stability rating.

CONCLUSIONS

Based on observations and field data, some general conclusions may be drawn:

(1) The commercial prefabricated structures did not generally provide adequate stilling basins for energy dissipation. They were less stable than cast-in-place structures and tended to wash out more easily. They also required more maintenance and, because of their smaller size, were generally less efficient hydraulically. The prefabricated structures, however, cost less and were easier to install.

(2) End sills caused turbulence that affected the downstream scour pattern and, in a small ditch, increased the total erosion volume by undercutting and eroding the banks immediately downstream. Although there did not appear to be a consistent relationship between the amount of scour and end-sill height, visual observation indicated that there was a greater degree of turbulence over the high sills.

(3) Structures having relatively wide basins performed better than those with narrow basins. The narrow basins contracted and accelerated the flow, resulting in higher exit velocities. The wide basins provided a larger flow area and thus a lower velocity. With adequate tailwater depth, relatively wide structures without end sills performed quite well.

(4) The cofferdam-type structures gave fairly good hydraulic performance when used as checks with sufficient tailwater depth. At high flow rates, the narrow opening below the cofferdam restricted the flow and caused high exit velocities.

(5) With adequate tailwater depth, a trapezoidal stilling basin gave good hydraulic performance. Without sufficient tailwater, the performance was poor and high velocity caused excessive downstream erosion.

(6) For the relatively small structures and water depths in the study, a nonaerated nappe contributed to good stilling within the structure.

(7) Structures installed by "puddling" with a sand and fine gravel backfill mix were resistant to rodent damage and piping failures, even though the mix was quite erodible when the structures were overtopped.

(8) With adequate cutoff depth and headwall length, headwall structures with a gravel-lined basin or plunge pool were the most economical and the most effective structures tested.

Laboratory studies are needed to investigate different methods of improving the hydraulic design. The effects of using flared wingwalls, rounded corners, different sill configurations and placement, nonaerated nappe, and protruding flanges should be studied, as well as how to eliminate the wall-attachment phenomenon.

LITERATURE CITED

(1) Chow, V. T.
1959. Design of channels for uniform flow. *Open-Channel Hydraulics,* pp. 157-191. McGraw-Hill Book Co., Inc., New York.

(2) Code, W. D.
1957. Farm irrigation structures. Colo. (Fort Collins) Agr. Expt. Sta. Bulletin 496-S, 60 pp.

(3) Donnelly, C.A., and Blaisdell, F. W.
1965. Straight drop spillway stilling basin. Amer. Soc. Civ. Engin. Proc., Hydraul. Div. J. 91 (Hy3): 101-131.

(4) Gilden, R. O., and Woodward, G. E.
1952. Low-cost irrigation structures. Wyo. (Laramie) Agr. Ext. Ser. Cir. 122, 16 pp.

(5) Herpich, R. L., and Manges, H. L.
1959. Irrigation water control structures. Kans. (Manhattan) Agr. Expt. Sta. 12 pp.

(6) Peterka, A. J.
1964. Hydraulic design of stilling basins and energy dissipators. U.S. Bureau of Reclamation, Engineering Monograph No. 25, 114 pp.

(7) U.S. Department of Agriculture, Soil Conservation Service
1958. Engineering design standards, Far Western States.

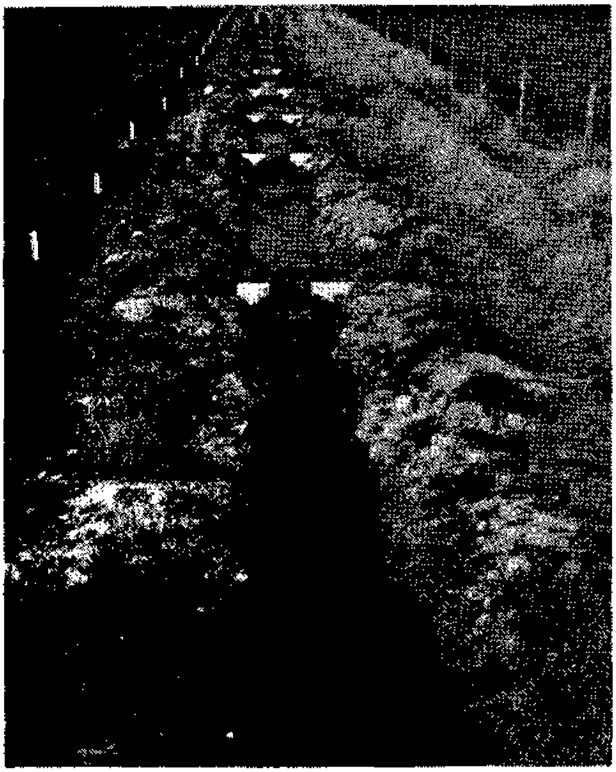


Figure 9.–General view of test ditch looking upstream from structures 16 to 1.

Figure 10.–General view of downstream section of ditch looking upstream from structures 30 to 19.

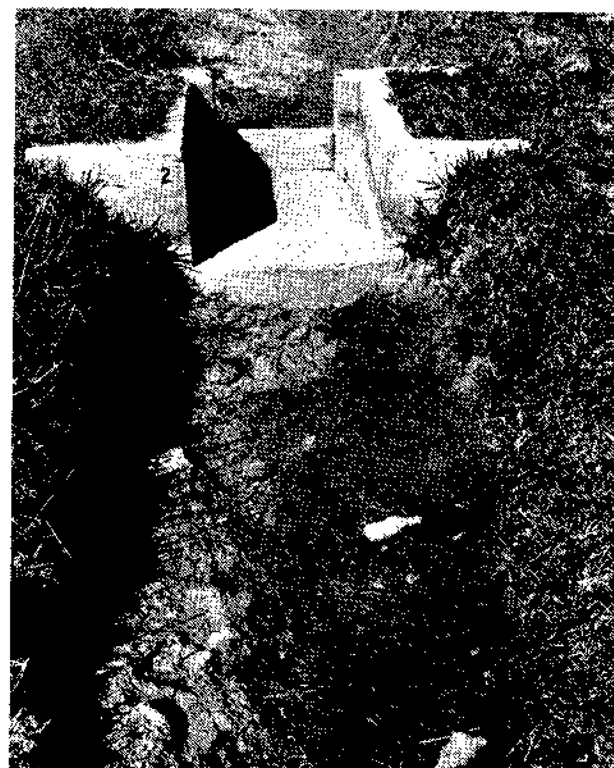


Figure 11.—Standard design, cast-in-place concrete, rectangular-basin structure without an end sill, Nos. 2. 3: A, In operation. B, At the end of the study.

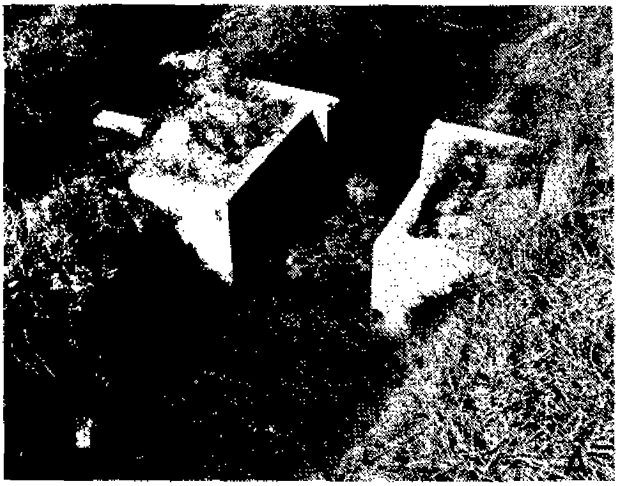


Figure 12.–Standard design, cast-in-place concrete, rectangular-basin structure with an end sill, Nos. 4, 5: A, In operation. B, At the end of the study.

Figure 13.—Precast concrete structure with an end sill, Nos. 6, 17: A, In operation. B, At the end of the study.

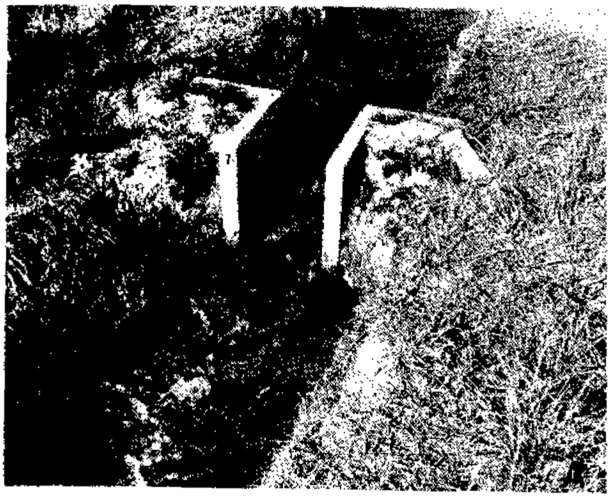


Figure 14,–Precast concrete structure without an end sill, Nos. 7, 18: A, In operation. B, At the end of the study.

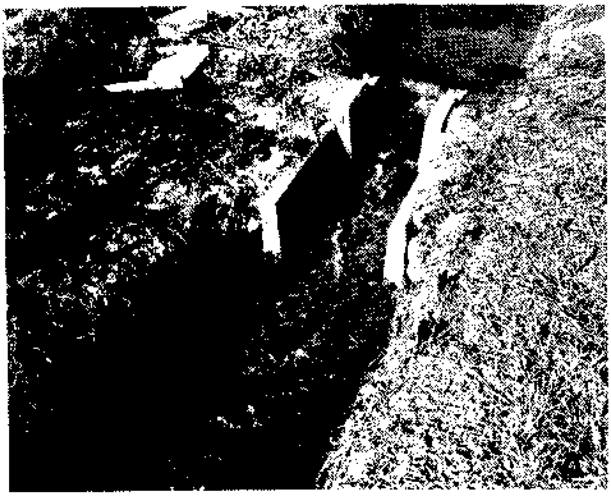


Figure 15.–Small-size precast concrete structure with upstream cofferdam and downstream liner section, Nos. 8, 9, 10: A, In operation. B, At the end of the study.

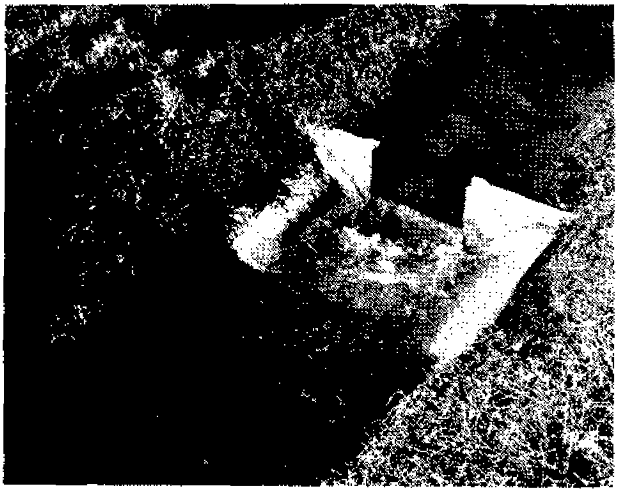


Figure 16.—Precast concrete-headwall structure with fiber glass stilling basin, Nos. 11 and 12: A, In operation. B At the end of the study.

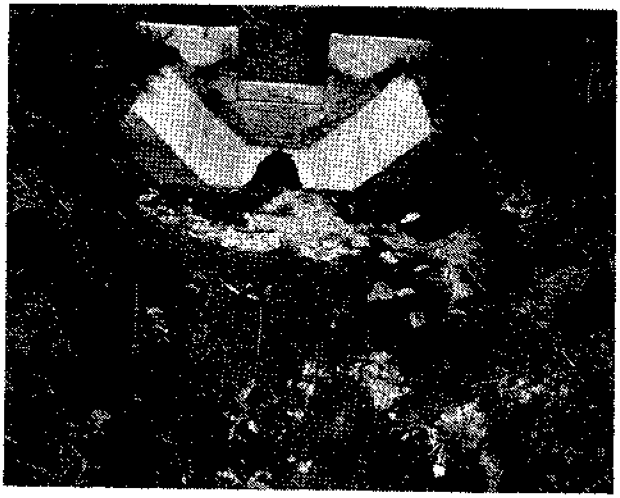


Figure 17.–Precast concrete-headwall structure with steel-lined stilling basin, Nos. 13, and 14: A, In operation. B, At the end of the study.

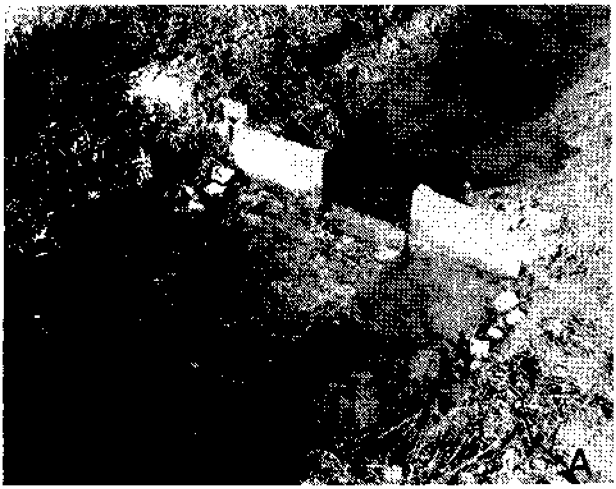
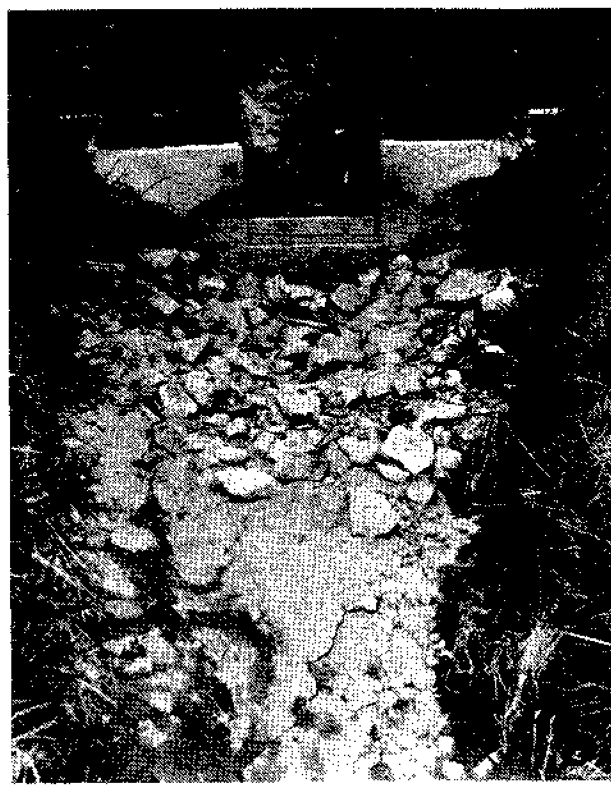


Figure 18.–Precast concrete-headwall structure with small-riprap stilling basin, Nos. 1, 15, and 16: A, In operation. B, At the end of the study.



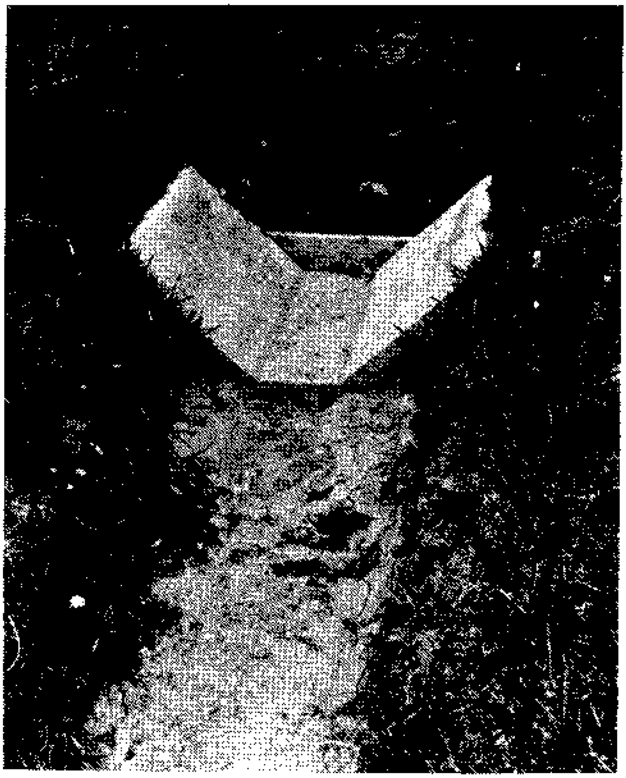


Figure 19.–Standard design, cast-in-place concrete structure with trapezoidal basin, Nos. 19 and 20: A, In operation. B, At the end of the study.

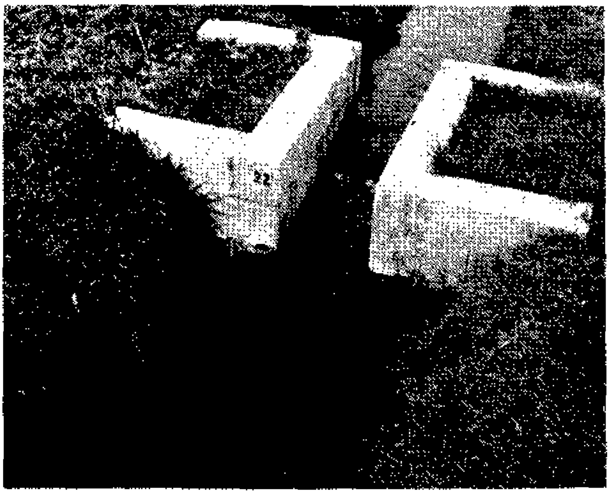


Figure 20.--Standard design, concrete-block structure with rectangular basin, Nos. 21 and 22: A, In operation. B, At the end of the study.

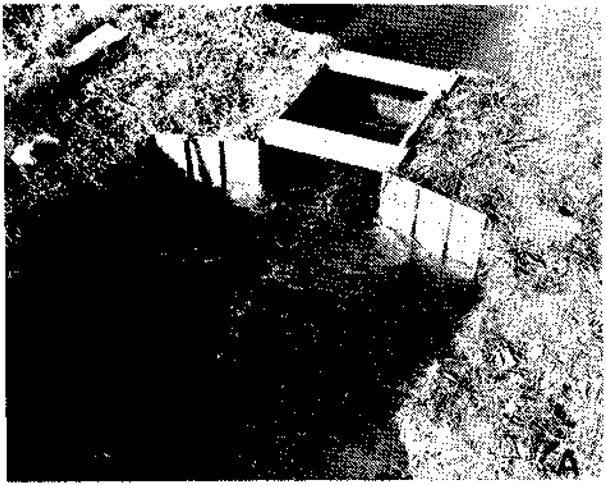


Figure 21.–Commercial, modular-steel structure with water checked to give increased tailwater depth, No. 23: A, In operation. B, At the end of the study.

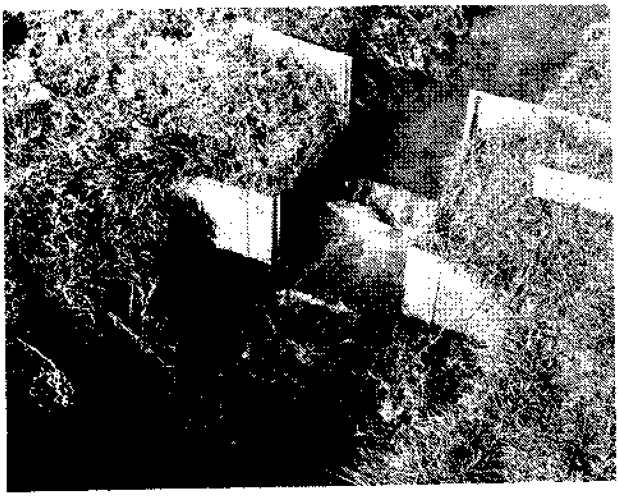
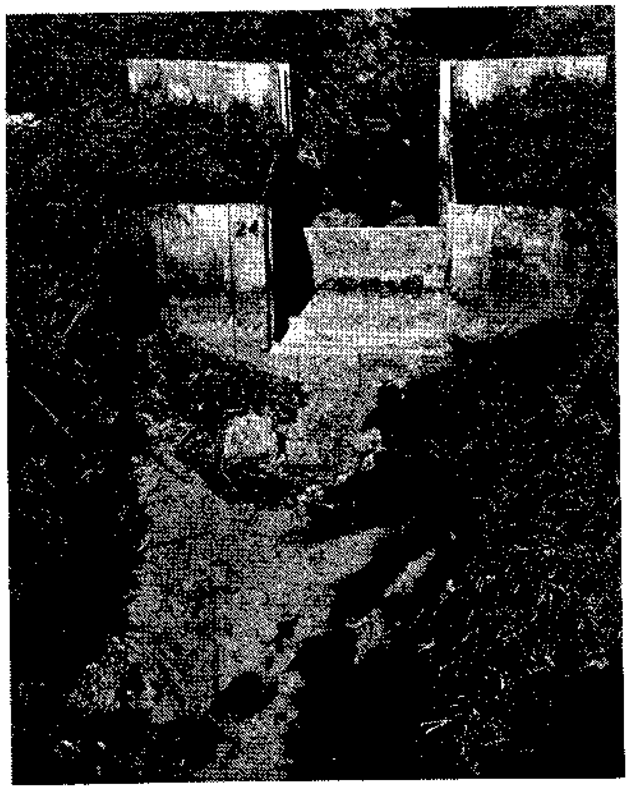


Figure 22.–Commercial, modular-aluminum structure, special design, No. 24: A, In operation. B, At the end of the study.



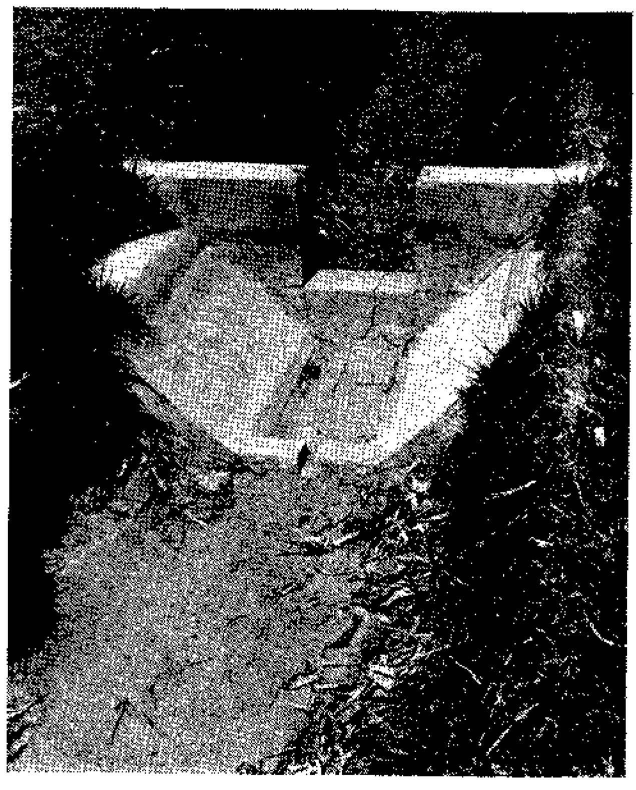


Figure 23.–Concrete-block headwall structure with formed trapezoidal basin, Nos. 25 and 26: A, In operation. B, At the end of the study.

Figure 24.–Large-size, precast concrete structure with upstream cofferdam and downstream sidewalls, Nos. 27, 28, and 29: A, In operation. B, At the end of the study.

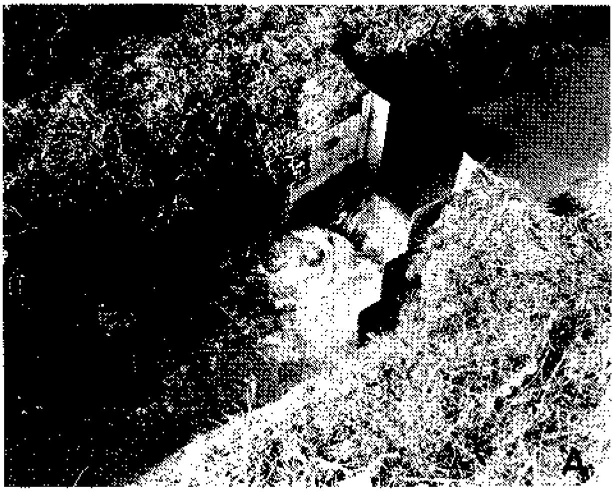


Figure 25.–Commercial, modular aluminum structure, local design, No. 30: A, In operation. B, At the end of the study.

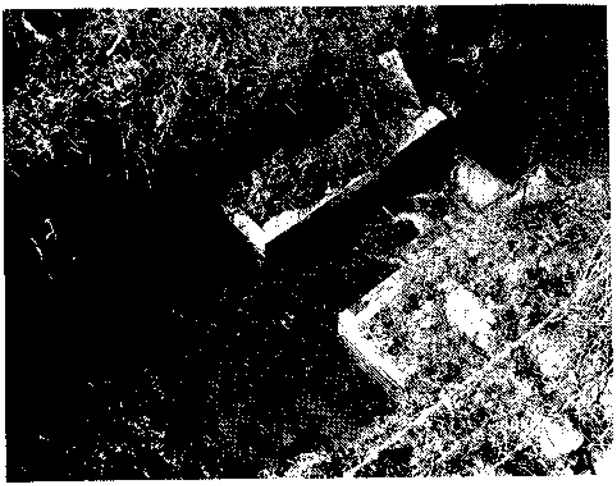
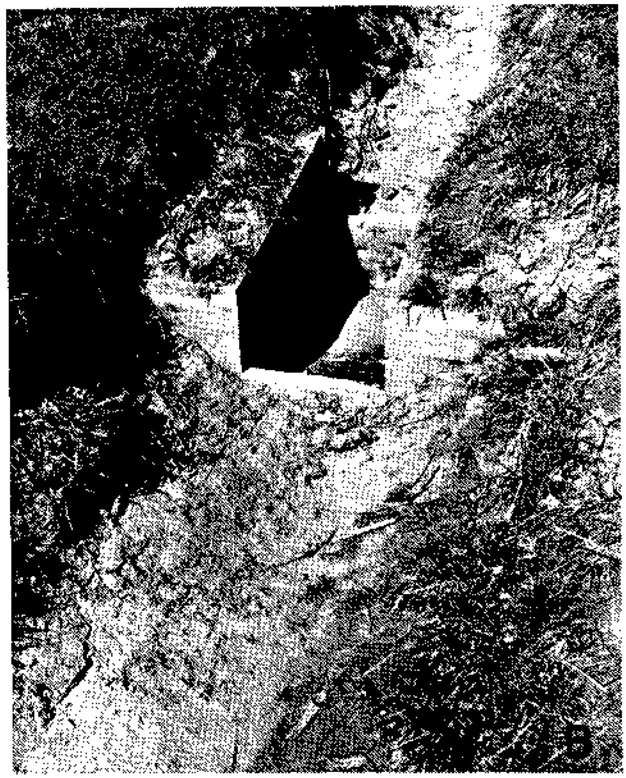


Figure 26.–Standard design wooden structure, No. 31: A, In operation. B, At the end of the study.



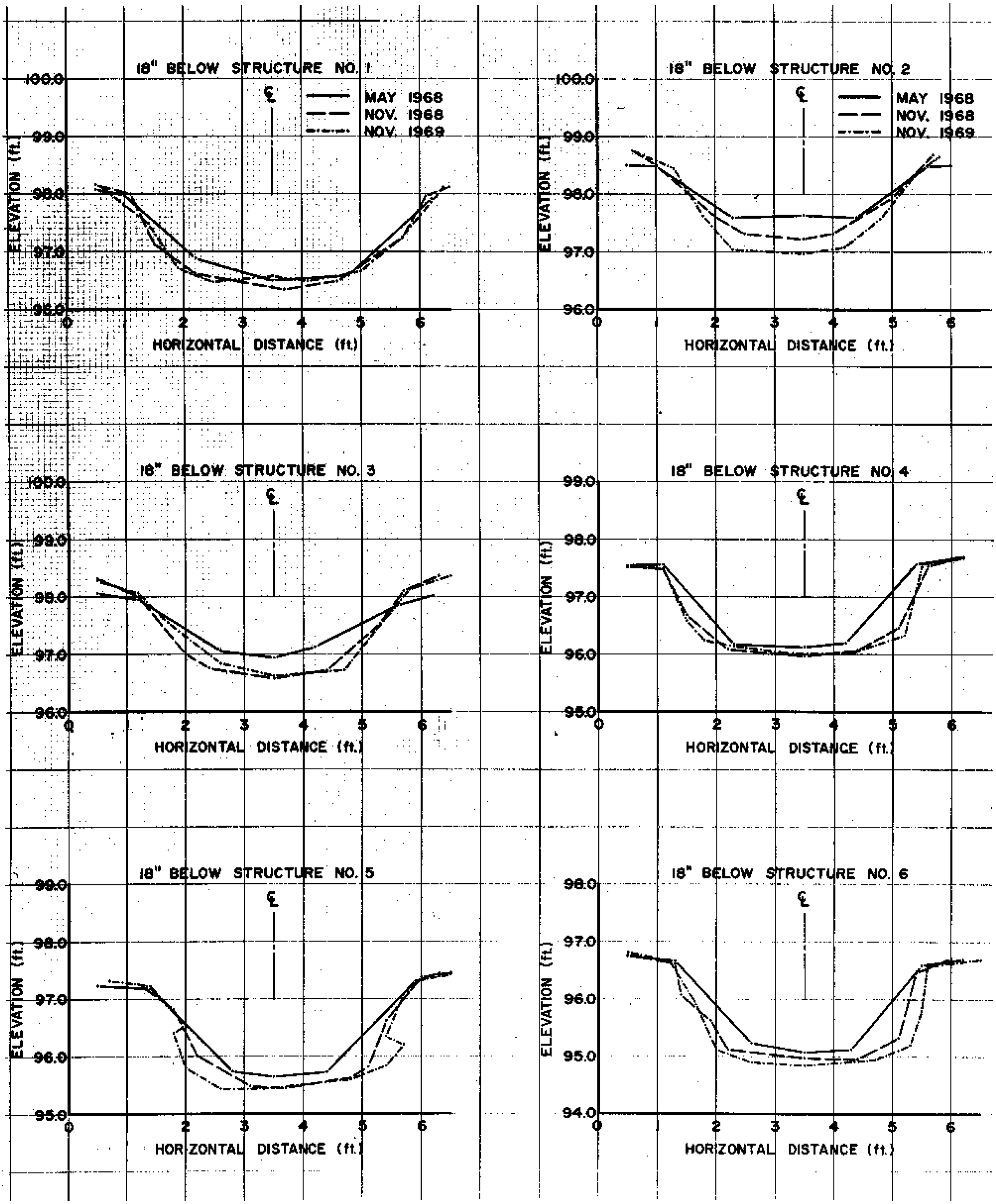


Figure 27.–Channel cross sections 18 inches downstream from structures 1 to 6.

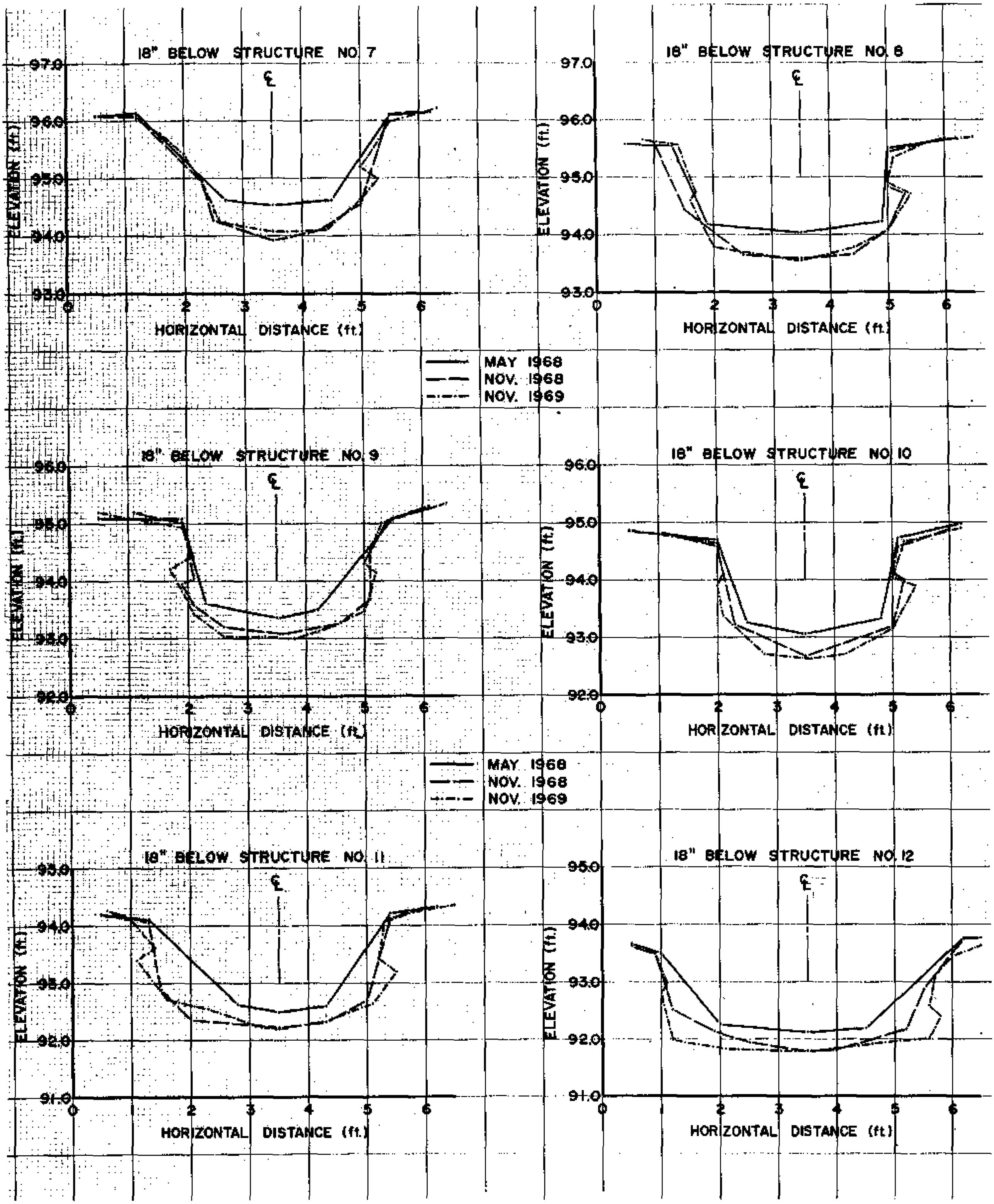


Figure 28.–Channel cross sections 18 inches downstream from structures 7 to 12.

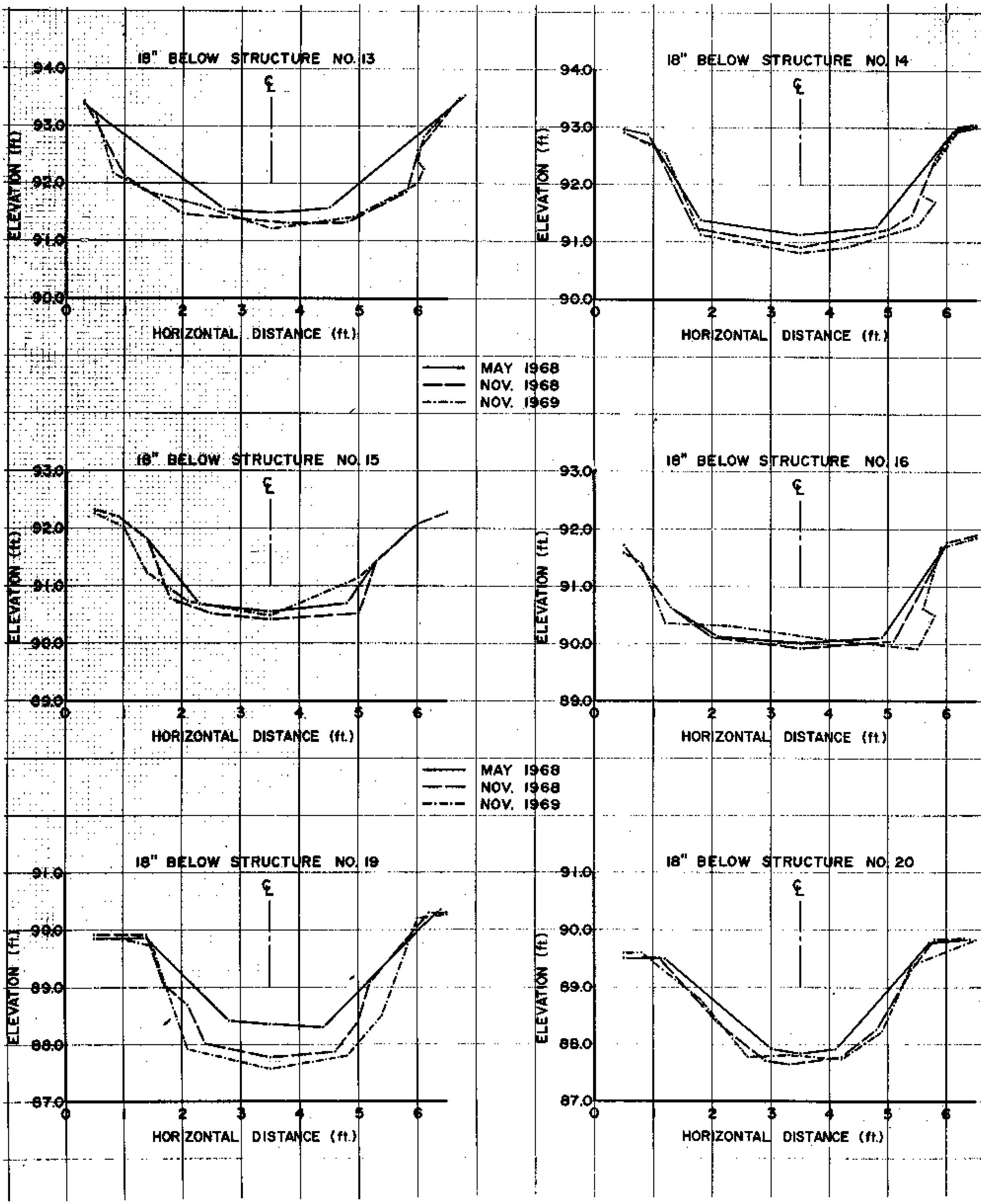


Figure 29.–Channel cross sections 18 inches downstream from structures 13 to 20.

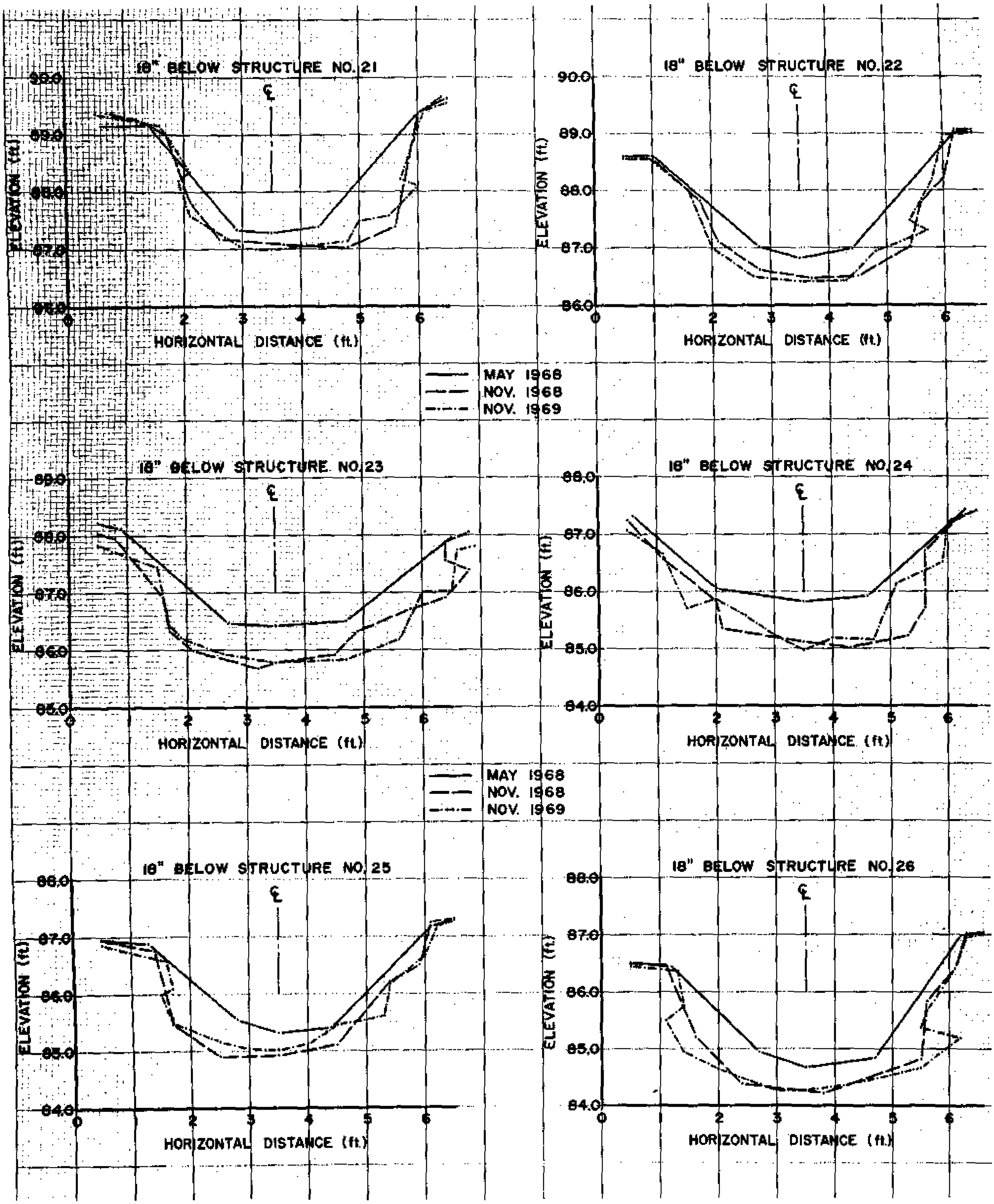


Figure 30.—Channel cross sections 18 inches downstream from structures 21 to 26.

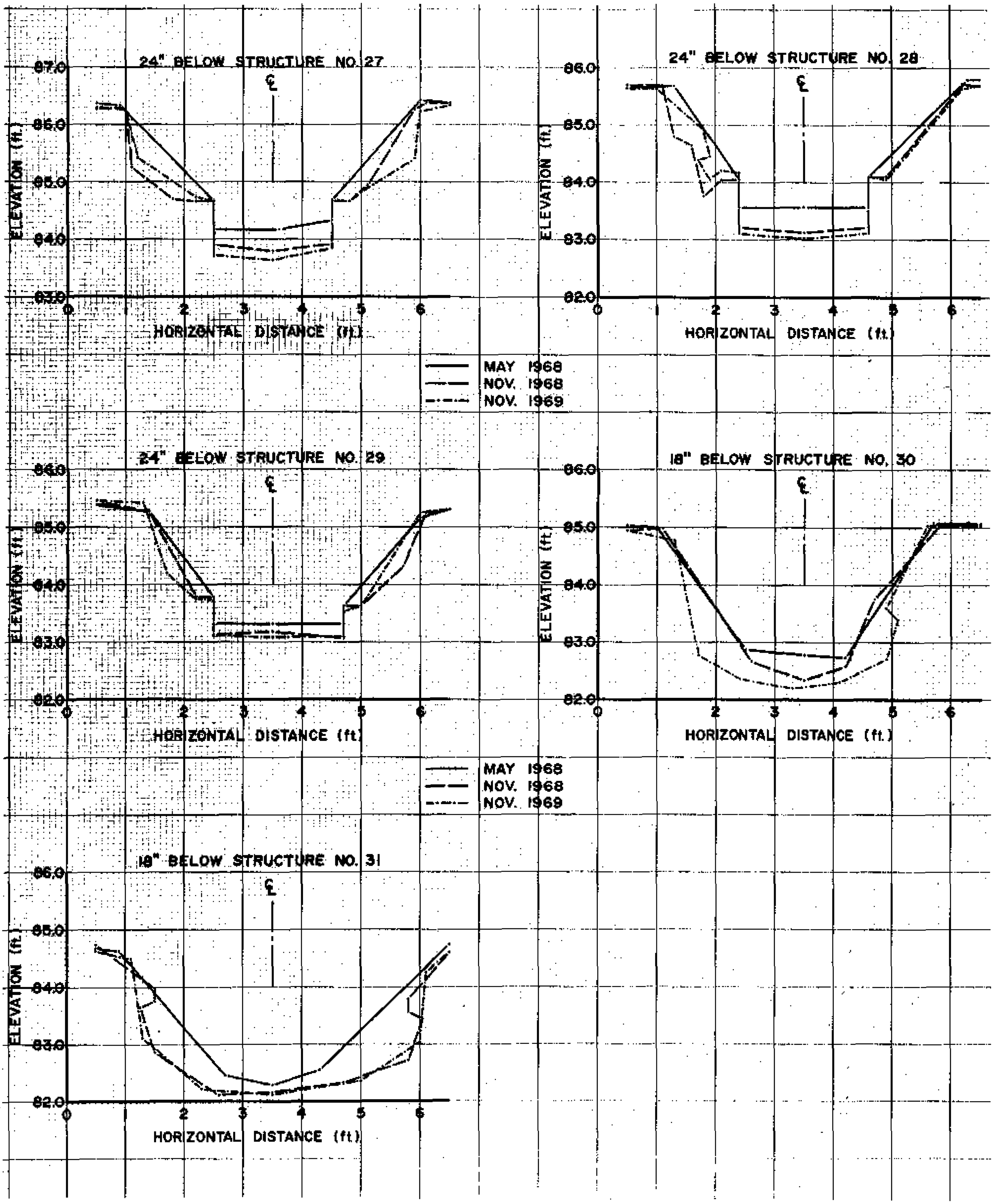


Figure 31.–Channel cross sections 18 inches downstream from structures 27 to 31.