UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF THE OUTLET WORKS AT CARTER LAKE RESERVOIR DAM NO. 1 JOINING THE ST. VRAIN CANAL

Hydraulic Laboratory Report No. Hyd-394

ENGINEERING LABORATORIES

COMMISSIONER'S OFFICE
DENVER, COLORADO

January 12, 1955

FOREWORD

Hydraulic model studies of Carter Lake Dam No. 1 outlet works joining the St. Vrain Canal, a part of the Colorado-Big Thompson Project, were conducted in the laboratory of the Bureau of Reclamation at Denver, Colorado, during the period from June to September 1954, after the structure had been built and operated.

The modifications to the structure evolved from this study were developed through the cooperation of the staffs of the Spillway and Outlets Works Design Section, the Canals Branch, and the Hydraulic Laboratory.

During the course of the model studies Messrs. H. W. Tabor, R. W. Whinnerah, F. D. Reed of the Spillway and Outlets Section, Messrs. H. K. Brickey, W. E. Schneider, and M. W. Scrivner of the Canals Branch frequently visited the laboratory to observe the model tests and to discuss the results.

These studies were conducted by G. L. Beichley with the aid of Charles Yang and Philip Chao. The studies were supervised by A. J. Peterka and J. N. Bradley under the Hydraulic Laboratory direction of H. M. Martin.

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Office of the Assistant Commissioner
and Chief Engineer
Engineering Laboratories
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Laboratory Report No. Hyd-394
Hydraulic Laboratory
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Reviewed by: A. J. Peterka

Subject: Hydraulic model studies of the outlet works at Carter Lake Reservoir Dam No. 1 joining the St. Vrain Canal

SUMMARY

Hydraulic model studies of the outlet works at Carter Lake Reservoir Dam No. 1 joining the St. Vrain Canal (Figures 1 through 8) were made after the prototype had been constructed and operated. Unsatisfactory performance of the prototype structure, followed by attempts in the field to improve the performance, resulted in a request for a model study to determine the necessary corrective measures. Studies were made on a 1:16 scale model (Figure 9) to improve the outlet works stilling basin performance, to reduce wave heights in the Parshall flume in order to improve its accuracy as a measuring device, and to reduce the waves in the St. Vrain Canal to prevent overtopping of the canal lining.

For discharges near maximum (625 second-feet) with high heads, the original prototype stilling basin of the outlet works had been found inadequate to hold the jump within the basin (Figures 10 and 11). For lesser discharges (560 second-feet), and for small discharges (230 second-feet), the jump remained in the basin, but the excessive water surface roughness in the Parshall flume prevented obtaining accurate staff gage readings from which discharges are determined (Figures 12, 13, 14, and 15).

As a result of extensive model tests it was recommended that six hook-shaped piers (Figures 21 and 22) be added to the stilling basin to hold the jump within the basin. These piers provided the least amount of water surface disturbance of the several types tested. In addition, a short-tube underpass type of wave suppressor was recommended for use downstream from the stilling basin to decrease the water surface fluctuation in the measuring flume. The wave suppressor was installed in the rectangular flume between the stilling basin and the Parshall flume (Figure 28). The baffle piers and the wave suppressor operating together produced a higher water level in the stilling basin, making

it necessary to extend the basin walls 3 feet upward and to project them inward 1 foot (Figure 22). A short cover over the basin was recommended at the entrance to the wave suppressor to prevent splashing over the walls when the waves struck the vertical face of the suppressor (Figure 22).

The recommended modified basin discharging a range of flows through the recommended underpass is shown in Figures 31 and 32. The jump remained in the basin for all flows up to 625 second-feet and for velocities at the tunnel portal up to 60 feet per second. Wave heights in the measuring flume were reduced for discharges of 100 second-feet and above.

Model wave heights were measured in the original structure (Figure 23), in the original structure modified with baffle piers (Figure 24), and in the recommended structure (Figure 33). Wave heights at the staff gage in the Parshall flume were reduced from over 3 feet to about 4 inches (Table 1, page 15) for 625 second-feet.

Waves generated at the downstream end of the Parshall flume, where it joined the rectangular flume (Figures 12 and 33), were reduced by installing a sill, 1 foot by 1 foot in cross section, on the floor at the entrance to the rectangular flume.

In addition to this hydraulic report, a motion picture in color, approximately 600 feet long, was prepared to illustrate the major portion of this study. The film carries the same title as this report. It is hoped that pictures of the recommended prototype structure in operation can be added when the prototype is completed.

INTRODUCTION

Carter Lake, Dam No. 1, and the St. Vrain Canal are a part of the Colorado-Big Thompson Project. The outlet works at Dam No. 1 discharging water from the Carter Lake Reservoir into the St. Vrain Canal is shown in Figure 1. The reservoir is located in the southwest corner of Larimer County, Colorado, approximately 7 miles west of Berthoud, Colorado, Figure 2.

The Carter Lake Reservoir is approximately 2 miles long, north and south, and approximately 0.6 mile wide, east and west. The reservoir was formed by construction of three earth-fill dams across gaps in the eastern rim of the basin.

Dam No. 1, across the greater of the three gaps, has a maximum height of approximately 200 feet, a 40-foot-wide crest at elevation 5769, and a crest length of approximately 1,235 feet. A concrete outlet works, utilizing a tunnel approximately 895 feet in length, is constructed normal to the axis of the dam near the right abutment with inlet sill at elevation 5618, Figure 3. The outlet works, Figure 4, consists of an inlet structure; a 6-foot 3-inch inside-diameter upstream conduit in open cut; a 6-foot 3-inch inside-diameter tunnel upstream from the gate chamber; a transition section; a gate chamber containing two 3- by 3-foot regulating gates and two 3- by 3-foot emergency gates; a hoist house and 7-foot inside-diameter access shaft; an 8-foot 6-inch-wide by 7-foot 3-inch-high tunnel downstream from the gate chamber; and an 8-foot 6-inch-wide by, approximately, 100-foot-long stilling basin. The inlet structure and stilling basin are shown in Figure 5 and the gate chamber in Figure 6. The outlets are designed to discharge a maximum of 625 second-feet with heads up to 159 feet.

Flow from the stilling basin enters the St. Vrain Supply Canal as shown in Figure 1. The canal extends from Station 10+90.56 at the downstream end of the stilling basin to Station 520+75 at St. Vrain Creek near Lyons, Colorado, Figure 2. The upstream portion of the canal in plan and profile is shown in Figure 7. This portion of the canal contains a Parshall flume and a section of rectangular flume immediately upstream and downstream from the Parshall flume as shown in Figure 8. The rectangular flume between the Parshall flume and the stilling basin is 16 feet 3 inches wide and 31.77 feet long. The Parshall flume is 92 feet long and varies in width up to 30 feet. The rectangular flume downstream from the Parshall is 15 feet wide and 120 feet 9 inches long and is followed by a 15-foot-long transition from the rectangular flume to the trapezoidal canal section 24 feet 6 inches in top width. The rectangular flume and the canal downstream from the Parshall flume contain a series of horizontal bends as shown in Figure 7.

The original prototype structure was operated in June of 1954, prior to this model investigation, for a range of discharges up to approximately 625 second-feet with heads of approximately 100 feet. The stilling basin did not function properly, resulting in unsatisfactory flow through the Parshall flume and in the canal. Attempts were made in the field to smooth out the flow using the log raft shown supported on the stilling basin walls in Figure 1. Since unsatisfactory performance still persisted, a model study was requested.

THE MODEL

The model, Figure 9, constructed in the laboratory was a 1:16 scale reproduction of the outlet works, the Parshall flume, and the rectangular flume joining the Parshall flume. The rectangular flume section downstream from the Parshall flume was added later. The outlet works included the gate chamber and gates, the tunnel from the gate chamber to the stilling basin, and the stilling basin.

Portions of the previously tested Willow Creek Dam outlet works model, including the gate chamber and part of the tunnel downstream, were adapted for use in the Carter Lake model. The gate chamber was constructed of transparent plastic and modified by use of wood inserts so that the portion of the chamber downstream from the gates was an exact geometrical model of the prototype. The gates and the portion of the chamber upstream from the gates were modified to make a satisfactory entrance section but were not exact geometrical duplicates of the prototype. This was not important, however, since the studies were made downstream from the gate chamber.

The outlet works tunnel was 22.56 feet long in the model. Approximately 12 feet of this length was taken from the Willow Creek model which had been constructed of transparent plastic. The remainder of the tunnel was constructed of sheet metal. Alternate sections of transparent and metal tunnel were assembled in the model. A wood floor, treated in oil, was inserted in the tunnel in order to adapt the Willow Creek tunnel for use in the Carter Lake model.

The stilling basin was constructed of marine plywood except for the curved trajectory floor at the upstream end which was of sheet metal. The rectangular flume section and the Parshall flume were also constructed of marine plywood.

Water was supplied to the model by means of a vertical 8-inch pump. A portable 8-inch orifice venturi meter was used to measure the discharge. Two piezometers, each 3 feet upstream from the gates on the outside walls of the gate chamber entrance, 1.5 feet above the floor, were used to measure the head on the gates. A Pitot tube was used to measure the velocity head at the outlet portal of the tunnel and a staff gage was used to measure the wave heights at the gaging station in the Parshall flume. Wave height records were made on the final tests using laboratory developed condenser-type wave measuring probes connected to a two-pen recording oscillograph. No tailwater regulation was necessary since the flow passed through critical depth in the Parshall flume.

THE INVESTIGATION

The investigation was concerned primarily with the outlet works and Parshall flume discharging the design flow of 625 second-feet at maximum reservoir elevation, 159 feet above the floor of the gate chamber. However, the investigation was also concerned with the complete range of operating discharges and reservoir elevations to be sure that the structures performed properly for all operating conditions.

The Original Structure

The original structure, as built in the field, is shown in Figures 3 through 8. On June 17, 1954, engineers from the Denver office observed and photographed the prototype outlets discharging an estimated 625 second-feet. The reservoir elevation was about 70 feet below maximum; therefore, the head was only about 56 percent of maximum. At this flow the jump was swept out of the basin as shown in Figure 10. Shooting flow occurred throughout the Parshall flume and continued into the canal where the flow stabilized with a rather mild jump and some wave action. Just before the jump swept out, waves were sufficiently high to overtop the walls of the stilling basin and Parshall flume as well as the rectangular flume and concrete lined canal. The model discharging 625 second-feet under the same conditions is shown in Figure 11. Action in the model is similar to that found in the prototype.

On the same day, an estimated 475 second-feet was also observed. The jump in the basin was rough, producing waves which traveled through the Parshall flume and on into the canal. These waves caused some bank erosion above the canal lining, particularly at the canal bends.

On June 11, 1954, Denver office engineers observed the outlets discharging approximately 560 second-feet as shown in Figures 1 and 12. The head was about 100 feet, which is approximately 60 percent of maximum. The jump was very rough and was close to sweeping out of the basin. Waves estimated to be 2.7 feet high traveled from the jump through the Parshall flume and into the canal, overtopping the training walls of the Parshall and rectangular flumes as well as the canal lining near bends. A log raft, shown in Figures 1 and 10, had been used in the rectangular flume in an unsuccessful attempt to quiet the waves entering the Parshall flume. Sandbags were placed as shown in Figures 10 and 12 to prevent erosion of the canal banks. The downstream end of the Parshall flume, where it joined the rectangular flume, created a disturbance, Figure 12, which also produced waves in the canal downstream. The model discharging 560 second-feet is shown in Figure 13 and was similar to the prototype operation shown in Figure 12.

On August 17, 1954, personnel from the laboratory and Region 7 observed the outlets discharging 230 second-feet as shown in Figure 14. The reservoir was at elevation 5658.63, and both gates were open 10-1/2 inches. The head, therefore, was 44.63 feet which is 28 percent of maximum.

A jump formed in the basin with the toe immediately downstream from the tunnel portal. The water surface throughout the basin was quite rough and waves over a foot high passed through the Parshall flume. The model discharging 230 second-feet is shown in Figure 15 and was similar to the prototype operation shown in Figure 14.

Model and Prototype Similarity

To obtain the similarity between model and prototype described above, it was necessary to resolve (1) certain discrepancies between the assumed conditions in the prototype tunnel at the time it was being designed and the conditions which actually existed after it was built, and (2) the difference in friction losses when a prototype is scaled down to model size.

In the design of the original basin the Manning formula with a roughness coefficient "n" of 0.014 was used to compute the friction losses in the proposed concrete lined tunnel. For 625 second-feet at maximum head the velocity at the tunnel portal, where the flow enters the hydraulic jump, was found to be 39 feet per second. The hydraulic jump basin was proportioned according to this figure by the usual means.

When the prototype structure was operated, however, the jump swept out of the basin before the maximum conditions of either head or discharge were reached, indicating that the velocity entering the prototype basin was considerably greater than 39 feet per second for 625 second-feet and that the assumed value of $n = 0.014$ used in the computation was too high.

The model could not be used to determine the proper "n" value directly because the roughness of the prototype tunnel could not be measured or modeled. Also, as is always the case for high velocity flow in relatively long and flat model channels, scale heads measured at the gate piezometers were not sufficient to produce true prototype velocities at the entrance to the jump, since actual friction head losses in a model are always greater than the prototype values divided by the model scale. Thus, the true prototype velocity had to be determined by some other means. Direct measurement of the velocity in the prototype tunnel was not possible since at the time of the model tests the reservoir had been drawn down and would not be re-filled until the next year.

Operation of the model, however, for 625 second-feet, with a portal velocity of 39 feet per second measured by Pitot tube, showed the jump to be retained in the basin, indicating that the basin proportions were adequate for the velocity calculated for design purposes. In fact a velocity of 53 feet per second was required at the model portal to cause the jump to be swept completely out of the basin. It was logical therefore that the maximum prototype velocity was greater than 53 feet per second.

Thus, the first problem in the model study was to determine the true prototype velocity at the tunnel portal for the design discharge.

On August 17, 1954, laboratory personnel measured the discharge and portal velocity in the prototype structure. A Pitot tube constructed in the laboratory shops especially for these measurements was used to obtain two measurements, one on the center line and one on the quarter point of the tunnel portal. On that day the reservoir was at elevation 5658.63 with both gates open 10.5 inches. The discharge was 230 second-feet. The average velocity at the portal was found to be 24.9 feet per second.

Using these values, the roughness coefficient "n" for the prototype tunnel was computed to be 0.008. Then, using this value of "n" to determine the portal velocity for 625 second-feet and maximum reservoir elevation, the velocity was computed to be 58 feet per second. This calculated value checked the model performance which showed that a velocity greater than 53 feet per second was required to cause the jump to sweep out. Because of the uncertainties involved it was decided to use 60 feet per second as the prototype portal velocity for the model tests.

Velocities in the model were set by increasing the head upstream from the gates until the average velocity, measured at the model tunnel portal with a Pitot tube at seven points across the tunnel opening, matched the newly computed velocity for the design discharge. The same increased head was also used for the tests involving discharges less than maximum.

Instead of increasing the head in the model it would have been possible, as an alternate method, to reduce the length of the model tunnel sufficiently to obtain the desired portal velocity. This was not done, however, since at the time of model construction it was thought that energy dissipating devices might be used in the tunnel to reduce the portal velocity. Consequently, a geometrically similar length of tunnel was constructed for the model tests.

Tunnel Modifications Tested

Based on the velocity computations described above, it appeared feasible to install energy dissipating devices on the tunnel floor to reduce the velocity at the portal sufficiently to use the basin originally constructed. The stilling basin had been designed for a maximum velocity of 39 feet per second but required a velocity, in the model, of about 53 feet per second to sweep the jump out of the basin. It was thought that since 60 feet per second would be the maximum prototype velocity possible, a velocity reduction in the tunnel of something over say 10 feet per second might make the original basin usable without modification.

Several devices were tested in the model tunnel. The most satisfactory scheme tested consisted of two sets of piers placed at about the third points along the tunnel between the gate section and the tunnel portal. Each set of piers was as shown in Figure 16A and performed as shown in Figure 16B. The upstream set of piers used alone reduced the velocity of flow at the tunnel portal from 60 to 50 feet per second. The combined effects of the two sets of piers reduced the velocity at the portal to about 45 feet per second. This was sufficiently close to the original design velocity of 39 feet per second that the hydraulic jump remained in the basin with some factor of safety. However, the jump produced waves that traveled downstream to overtop the training walls of the Parshall flume.

To help quiet the flow in the basin and reduce the waves, a third set of piers shown in Figure 17A was added at the tunnel portal. Figure 17B shows the performance of the stilling basin with the portal piers and two sets of tunnel piers installed. The portal piers stabilized the flow considerably, but objectionable waves still persisted in the downstream structure. Other devices to quiet the flow in the basin were tested including chute blocks, baffle piers, and trajectory splitter rails, but none of these were entirely satisfactory. In addition to the wave problems, the piers placed in the tunnel caused large jets of water to be thrown upward against the tunnel crown, and it was feared that proper tunnel ventilation would be hampered.

Other devices for reducing the velocity of flow in the tunnel were also tried including two groups of 1-foot cubes. The cubes created much more disturbance to the flow than the piers, throwing large volumes of water against the tunnel crown. Even with the upstream faces of the cubes cut on a 45° slope, much more disturbance was created than with the piers. A small 4-inch angle iron anchored to the floor across the width of the tunnel was tested but deflected much of the flow to the tunnel roof. Variations in the number of piers per set, the height and width of piers, and in the shape of the pier nose were all tested and found to be unsatisfactory. Whether five piers were spaced as shown in Figure 16 or three wider piers were spaced farther apart made very little improvement in the performance. Short submerged piers caused more disturbance than those that were taller than the flow depth. Pier noses that were shaped like the bow of a ship and those that sloped downstream on a 45° angle both offered less resistance to the flow, and, therefore, were not as effective as the vertical nosed piers.

In general, blunt nosed devices introduced into the supercritical flow in the tunnel provided some reduction in velocity, but the large scale disturbances they created either sealed off the tunnel, preventing proper venting of the tunnel, or increased the air flow beyond the capacity of the existing venting facilities. When these same devices were streamlined to reduce the disturbances, there was not sufficient energy loss to warrant their use. To obtain sufficient energy losses, a considerable number of sets of streamlined devices would have been required. It was decided, therefore, to test other devices in the stilling basin.

Stilling Basin Modifications Tested and Recommended

The trajectory curve on the upstream end of the stilling basin floor had been designed for 39 feet per second. With higher velocities up to 60 feet per second entering the basin, it was necessary to check the trajectory surface for the possible existence of negative or cavitation producing pressures. Seven piezometers on the trajectory curve were used to measure pressures for a discharge of 625 second-feet and a velocity of 57 feet per second. Figure 18 shows that the lowest pressure recorded was only about 1 foot of water below atmospheric pressure. The original trajectory curve was therefore considered satisfactory for use with the higher tunnel velocities.

Various types and arrangements of baffle piers were tested in the stilling basin to determine whether the resistance to sweepout created by the baffle piers was sufficient to hold the jump in the basin for the maximum conditions. Tests showed that baffle piers with vertical blunt faces caused the high velocity flow in the basin to be directed upward, causing a very rough water surface in the basin. Streamlined piers did little good. The most effective piers tested had a curved, hook-shaped front face as shown by the dotted curve in Figure 19.

To determine the need for the curved front face of the baffle piers, tests were made with the curve replaced by 5 and also 4 tangents. Five tangents, Figure 19, approximated the curve so closely that no difference in performance could be detected. With only four tangents the piers did not perform as efficiently as before. It was concluded that piers with a curved front face or with the curve approximated by five tangents could be used with equal effectiveness.

The hook-shaped front face of the pier intercepted the high velocity flow and sprayed it sideways from the hook as shown in Figure 20A. Six of these baffle piers were recommended for use in the basin as shown in Figure 21, but in Figure 20A only one pier was used with the tailwater purposely lowered to demonstrate the action. None of the water was thrown upwards from the pier. The flying spray in the photograph is caused by the flow deflected into basin side walls and the lack of covering tailwater.

A single pier near the base of the trajectory curve on center line was sufficient to hold the jump in the basin, but the addition of other piers aided in bringing the jump farther upstream into the basin and in smoothing the water surface downstream. With six piers in the basin, much air was entrained in the flow and flow velocities were lower, therefore the depth of flow in the basin was increased. As a result, the basin walls did not provide enough freeboard to contain all of the surges that developed in the basin. It was evident that the walls would need to be raised. Several arrangements of the piers were tested with two different pier widths, 12 inches and 16 inches. The best arrangement and most effective pier widths, however, are those shown in Figures 21 and 22.

Pressures were checked at critical locations on the upstream center pier for maximum flow with maximum and near maximum head as shown in Figure 19. This pier was chosen for the pressure test because it was exposed to the direct action of the jet emerging from the tunnel. Pressures were only slightly below atmospheric at the two locations thought to be critical. Pressures were not sufficiently low, however, to indicate that cavitation can occur.

At the point where maximum pressure was believed to occur, Piezometer 3, the pressure was about 25 feet of water. This pressure was considered to be reasonable by the designers from the structural viewpoint.

The unbalanced forces tending to tip the pier sideways were also determined by means of piezometers. Two pairs, one near the top of the pier and one near center line, Figure 19, showed only about 4 inches of differential water pressure. Trials with the gate openings unbalanced in various degrees showed unbalanced forces of only about 8 inches of water including the surges that occurred. No lateral stability problem on the baffle piers should occur.

The baffle piers shown on Figures 21 and 22 were superior in performance to any of the other devices or any other arrangement of baffle piers. Without baffle piers, at maximum reservoir elevation, the design discharge of 625 second-feet swept through the basin and shooting flow continued past the staff gage in the Parshall flume. With the baffle piers a satisfactory jump was formed, and the fluctuation in water surface from the maximum crest elevation to the minimum trough at the staff gage was 1.2 feet, Table 1. Performance for 625 second-feet with maximum head is shown in Figures 20B and C. For 550 second-feet the baffle piers reduced the wave heights from 3.2 to 1.7 feet and for 400 second-feet from 2.7 to 2.5 feet. For lower flows the baffle piers were of benefit only in that they improved the appearance of the flow; actual maximum wave heights were as great or greater than without the piers. This was probably due to the fact that flows of 200 or less tended to flow over the tops of the baffle piers whereas higher flows plunged beneath the tailwater.

Complete wave records, obtained in the model with the electronic wave measuring device, are shown in Figures 23 and 24 for the original basin and with the recommended baffle piers installed. Upstream and downstream records were recorded simultaneously using two measuring probes connected to a two-pen recorder. Therefore, a wave passing the upstream station can be identified on the charts at the downstream station a few seconds later. For example, a wave travels from the upstream to the downstream station in 6 seconds for 400 second-feet.

The baffle piers therefore accomplished their intended purpose; that of holding the jump in the stilling basin and of reducing the wave heights for the higher flows. Objectionable waves still persisted in the Parshall flume, however, and the deeper water in the stilling basin caused some surging over the walls. Using the piers, therefore, required raising the walls of the stilling basin and developing a device to suppress the waves in the Parshall flume for all discharges. Testing continued on this phase of the work.

Wave Suppressing Tests

Several types of wave suppressing devices were tested, Figures 25 and 26. The curtain wall in Figure 25 proved effective for the discharge for which it was set. It was less effective for smaller or larger flows and had little effect on long period surges. Rafts, both rigid and articulated, were tried but these were also not too effective on surges. Underpass suppressors in the stilling basin area were totally ineffective since turbulence created by the baffle piers and the hydraulic jump extended beyond the suppressor. Suppressors in the transition section were less effective than those located downstream for the same reason. Various arrangements of underpass suppressors were tested in the downstream location, Figure 26. Those with long sloping surfaces were less effective than those with long horizontal surfaces because the effective length of the underpass was thereby reduced, particularly for the shallow depth flows. Doubling the length of the underpass, Figure 26, reduced the wave heights about 50 percent. Perforating the roof of the underpass increased the tendency to surge in the Parshall flume.

The best device tested was a short-tube type underpass installed in the rectangular section upstream from the Parshall flume. Experiments on the necessary length of underpass showed that longer structures performed better than short ones as shown by the data in Figure 27. Experiments with this type of underpass indicated that it would be possible, by lowering the roof, to make the suppressor effective for flows as low as 100 second-feet. To accomplish this, it was necessary to place the roof of the underpass sufficiently low to be in contact with the water surface for 100 second-feet or to make its height above the floor adjustable. The latter was not practical from a cost standpoint, therefore, the former was recommended.

Recommended Wave Suppressor

The roof of the underpass was placed 4 feet 8 inches above the floor which is about 2/3 of the normal flow depth for 625 second-feet instead of 4/5 previously tested. Test showed that the lower roof was slightly more effective for the high flows, Figure 27, and in addition made the underpass effective for flows as low as 100 second-feet. The low roof produced more turbulence at the underpass exit than was desirable, however. The turbulence was caused by the high velocity flow from the underpass expanding rapidly as it entered the deeper water downstream. To smooth out the flow near the underpass exit, an expanding or draft tube type of exit was used as shown in the recommended design in Figure 28. A comparison of the performance of the draft tube type exit and the square exit is shown in Figure 29. Flow leaving the expanding exit had less visible turbulence in the flow and fluctuations at the staff gage appeared less frequently.

Lowering the roof of the underpass to suppress the waves for 100 second-feet increased the depth of flow upstream from the suppressor as shown in Figure 30. Since the baffle piers in the basin also increased the depth of flow, it was necessary to increase the height of the stilling basin walls 3 feet and add a 1-foot inward projection at the top of the walls as shown in Figure 22. A short cover extending upstream from the underpass entrance was also required, Figure 22, to contain the surges and splash within the basin.

The water surface elevations upstream and downstream from the wave suppressor were used to determine the discharge coefficient "C" of the underpass in the equation:

$$Q = CA\sqrt{2g(h+h_v)}$$

where

Q is the total discharge
A is the area of the flow passage through the underpass
h is the head loss through the underpass
h_v is the velocity head upstream from the underpass

The coefficient "C" is plotted versus the velocity of flow through the underpass, Figure 30. This curve should be useful in estimating the amount of head loss through future underpass type wave suppressor designs.

The Recommended Structure

The structure recommended for use in the field consisted of the structure originally built plus the following additions: six baffle piers in the stilling basin, Figures 21 and 22, the wave suppressor shown in Figures 22 and 28, wall height extensions as shown in Figure 22, and a 1-foot-high sill at the downstream end of the Parshall flume which is discussed later.

The performance of the recommended structure, including flows from 100 to 625 second-feet at any operating head, was considered to be satisfactory. Performance is illustrated in Figures 31 and 32. Some spray occurred at the toe of the model jump. If this condition is objectionable in the prototype, a short cover over the basin will prevent flying spray from wetting the adjacent area.

Maximum water surface fluctuations at the Parshall flume staff gage with the recommended structure are shown in Table 1. The wave height records from which these values were taken are shown in Figure 33. These records show that fluctuations at the staff gage will be less for the higher flows than for 200 second-feet. An interesting comparison of Figure 33 with Figures 23 and 24 shows that waves upstream from the wave suppressor are increased due to reflections from the suppressor entrance while waves downstream from the suppressor were reduced by its use.

The maximum values in Table 1 are determined from maximum crest and minimum trough readings which are not consecutive, Figure 33. The ordinarily observed fluctuation from the mean water surface will be somewhat less than the maximum values, and it is anticipated that satisfactory staff gage readings in the prototype may be obtained by eye or from recorder charts.

Photographs of the prototype in operation and inspection of the model indicated that another objectionable wave producing condition existed in the structure which was wholly independent of the wave problem upstream from the Parshall flume. This condition occurred at the downstream end of the Parshall measuring flume where it joined the rectangular flume. The waves were caused by the converging walls at the downstream end of the flume. Concentrations of flow along the converging walls produced standing waves in the flume, Figure 34A.

In the prototype, waves had also been observed in the flume and in the canal downstream, Figure 12. These waves were particularly objectionable for flows near canal capacity since they were at least partly responsible for the overtopping of the canal lining.

Table 1

WAVE HEIGHTS IN FEET--PROTOTYPE
Maximum Head

Discharge	625		550		400		200		100	
	Up-stream*	Down-stream*	Up-stream	Down-stream	Up-stream	Down-stream	Up-stream	Down-stream	Up-stream	Down-stream
Original structure	-	-	4.7+	3.2	3.2	2.7	1.8	1.6	1.3	1.3
With six baffle piers	2.7	1.2	2.8	1.7	2.7	2.5	2.7	2.3	1.1	1.1
With baffle pier and wave suppressor	3.8+	0.3	4.2	0.3	4.5	0.4	3.6	0.4	1.7	0.3

*Upstream station is just downstream from the stilling basin. Downstream station is at staff gage in Parshall flume. See Figures 23, 24, and 33.

+Recorder needle reached limit of travel.

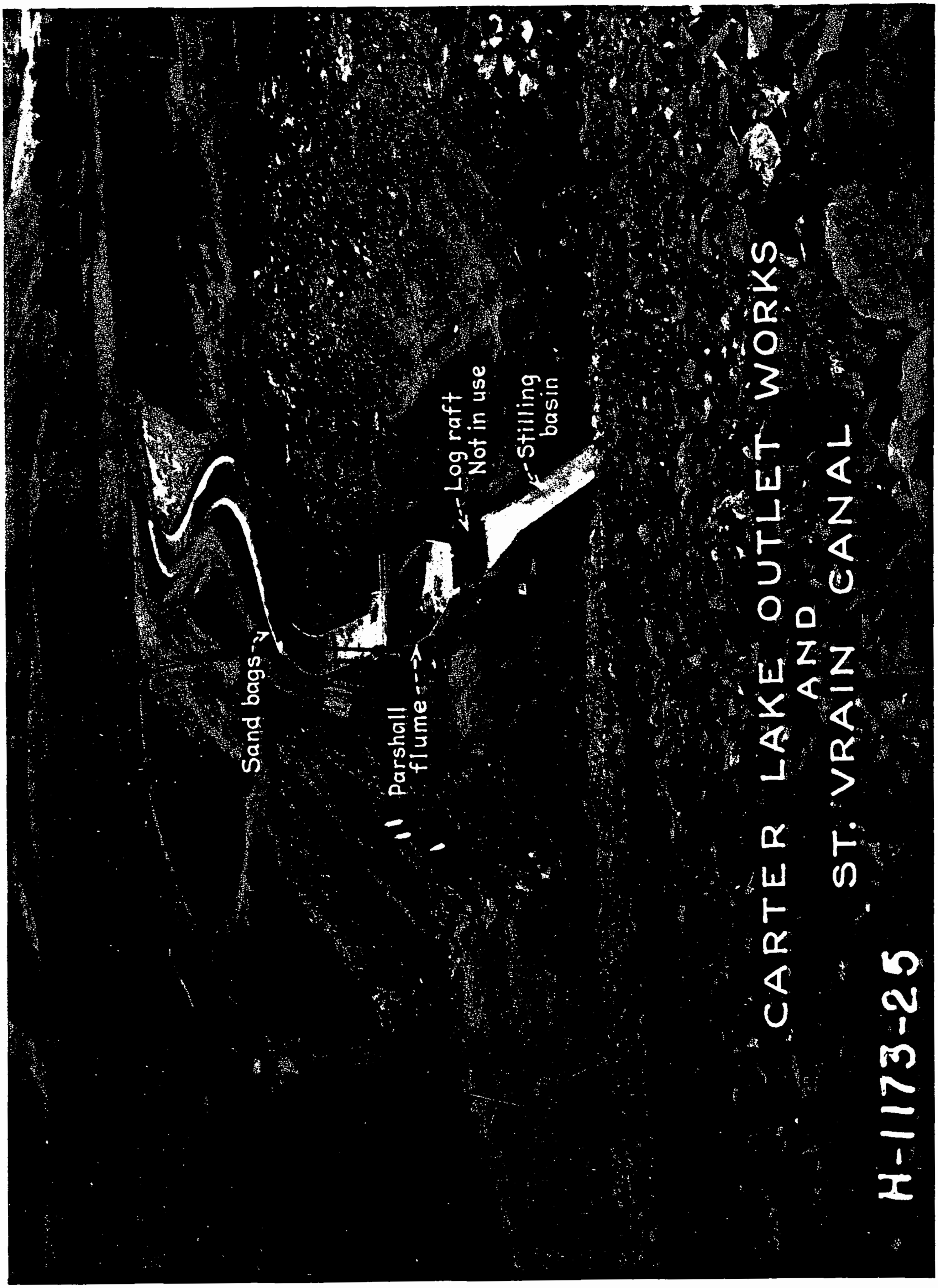
Sand bags
Parshall flume
Log raft
Not in use
Stilling basin
CARTER LAKE OUTLET WORKS
AND
ST. VRAIN CANAL
H-1173-25

FIGURE 2
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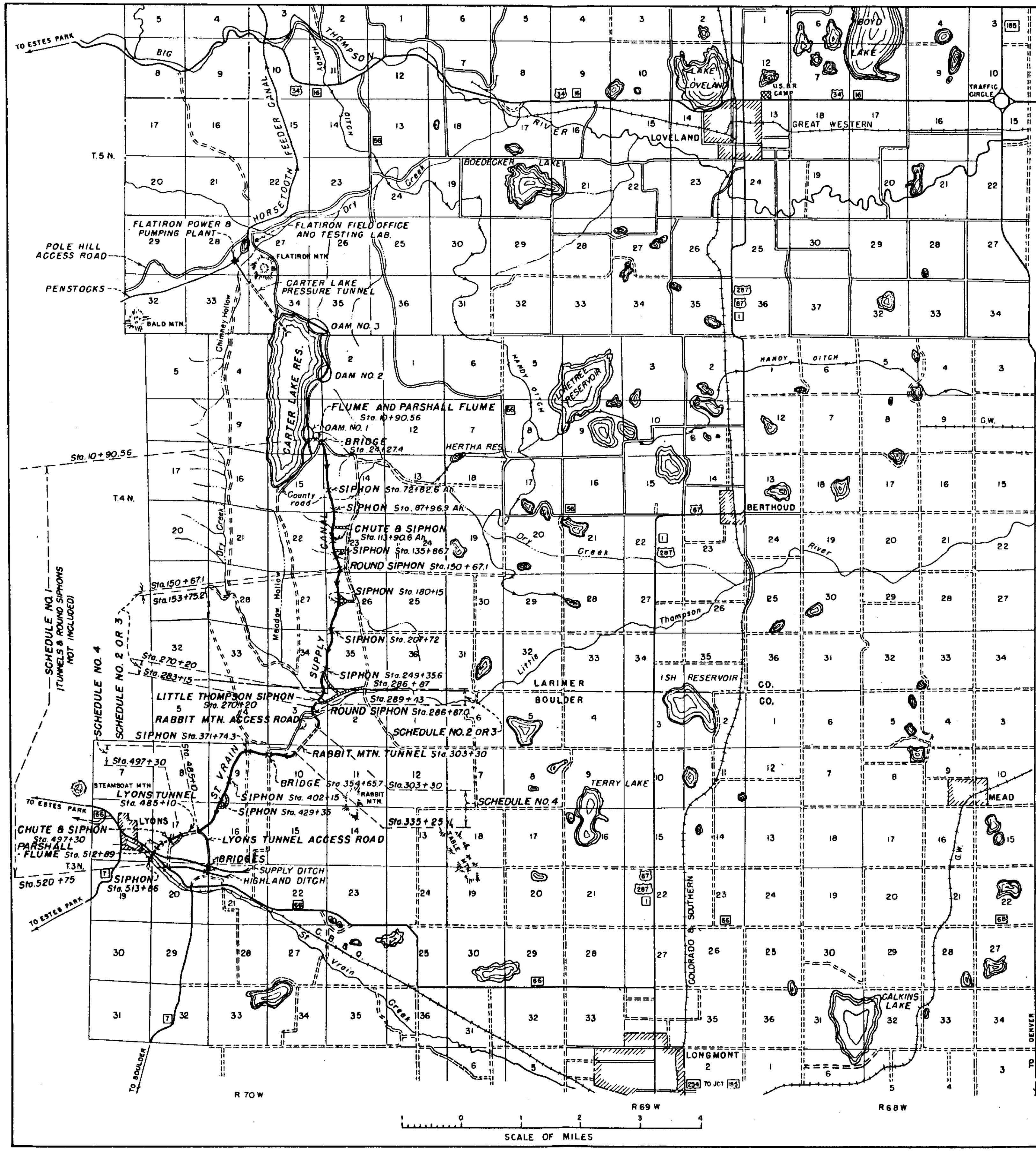

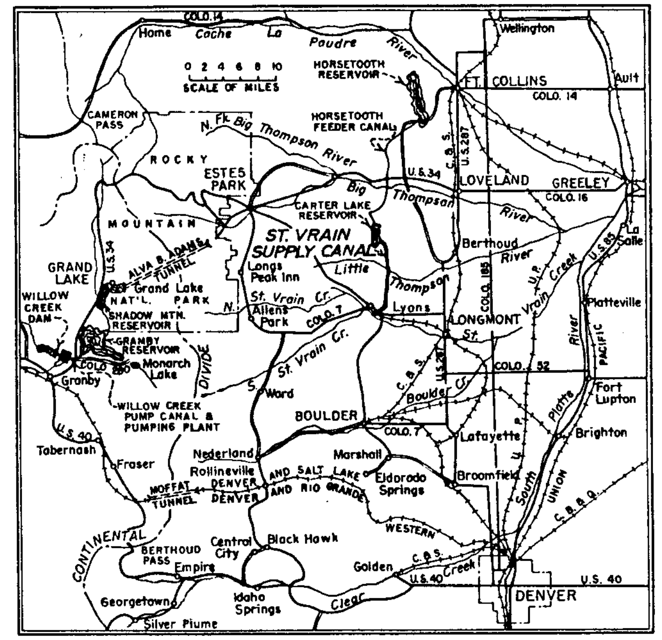

EXPLANATION

Contractor's Access Roads ....

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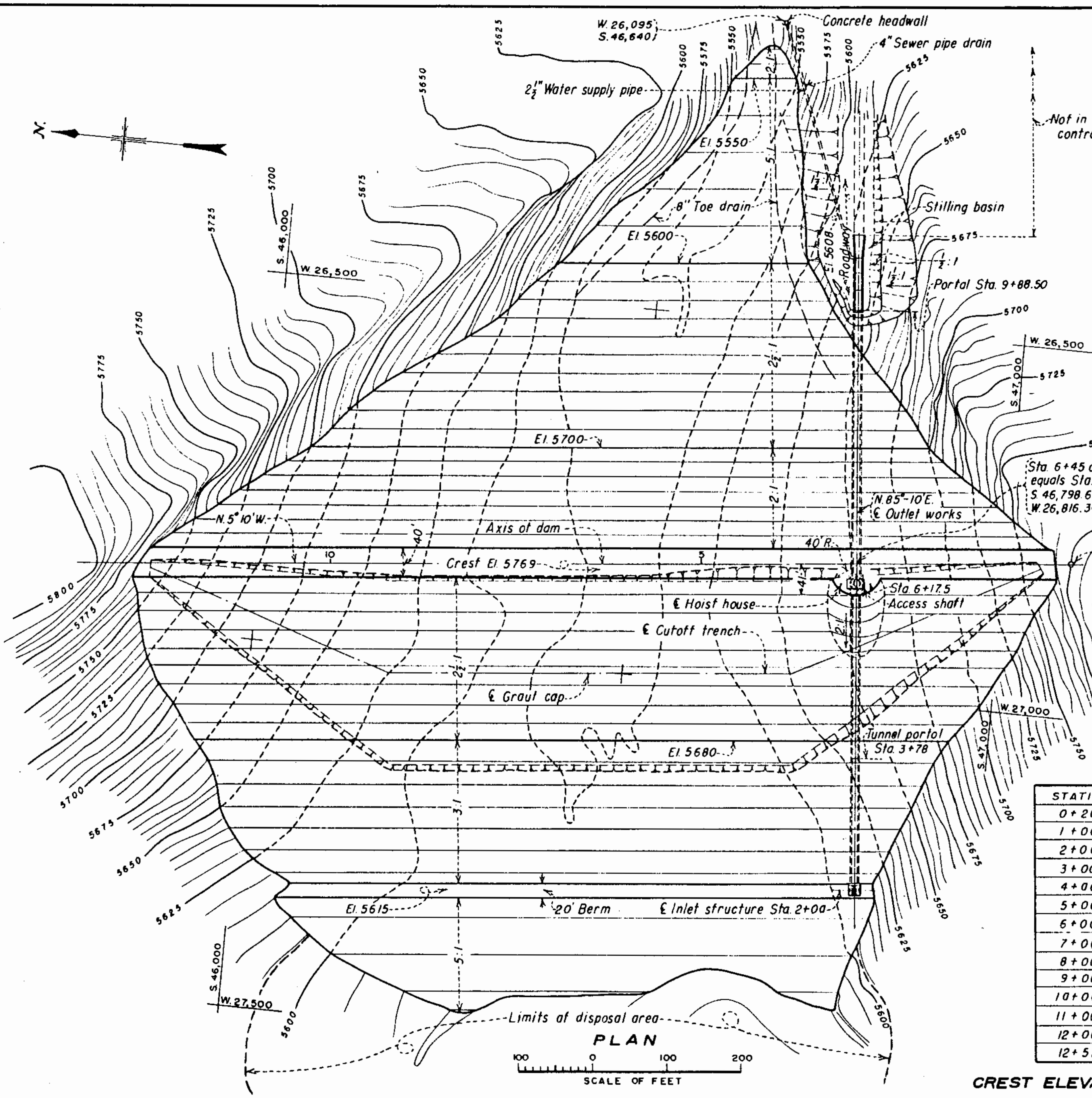
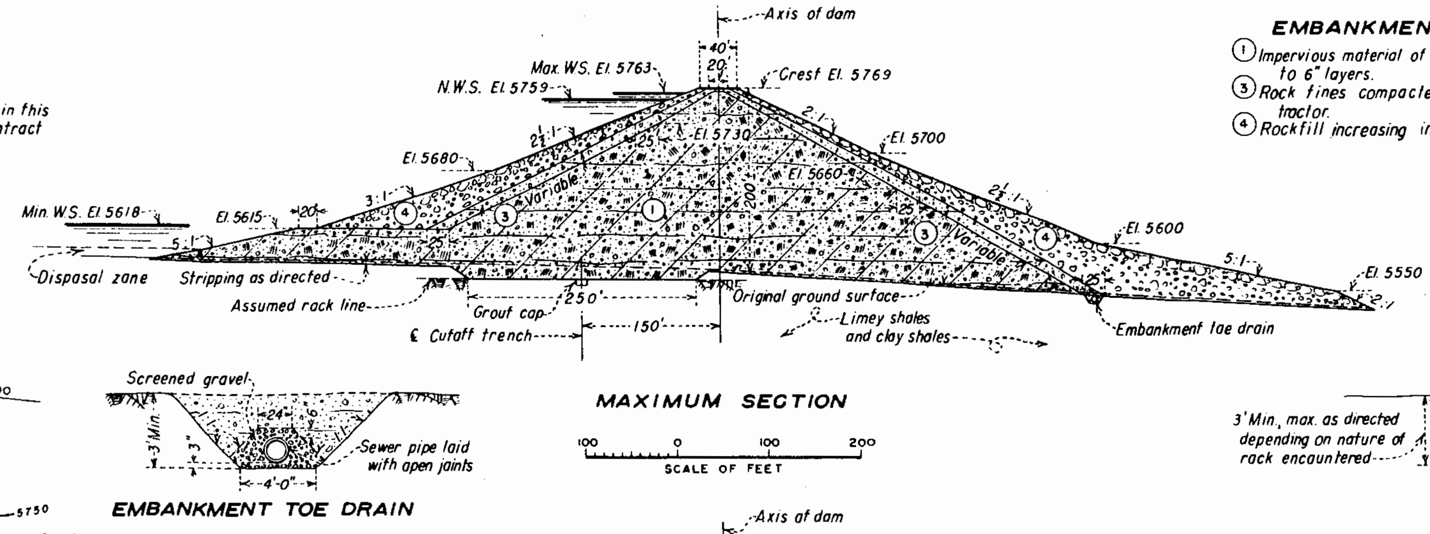
COLORADO - BIG THOMPSON PROJECT - COLO.

ST. VRAIN SUPPLY CANAL

LOCATION MAP

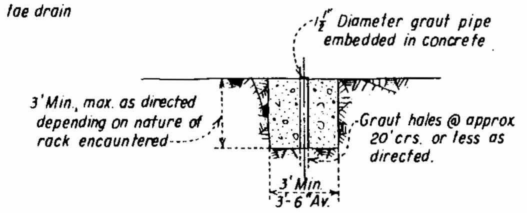
DRAWN A.W.B.	SUBMITTED
TRACED	RECOMMENDED
CHECKED	APPROVED
DENVER, COLORADO, NOV. 15, 1951	245-704-5010

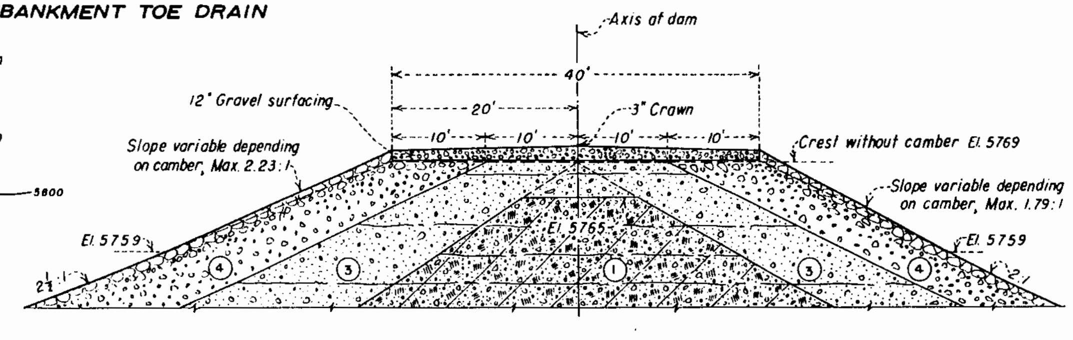


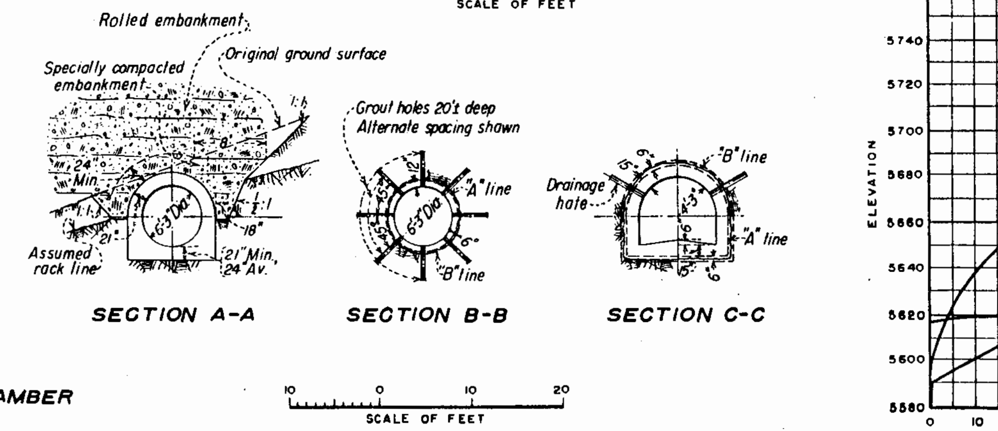


EMBANKMENT EXPLANATION

1. Impervious material of clay, sand and gravel compacted by rollers to 6" layers.
3. Rock fines compacted to 12" layers by crawler type tractor.
4. Rockfill increasing in coarseness toward outer slope.

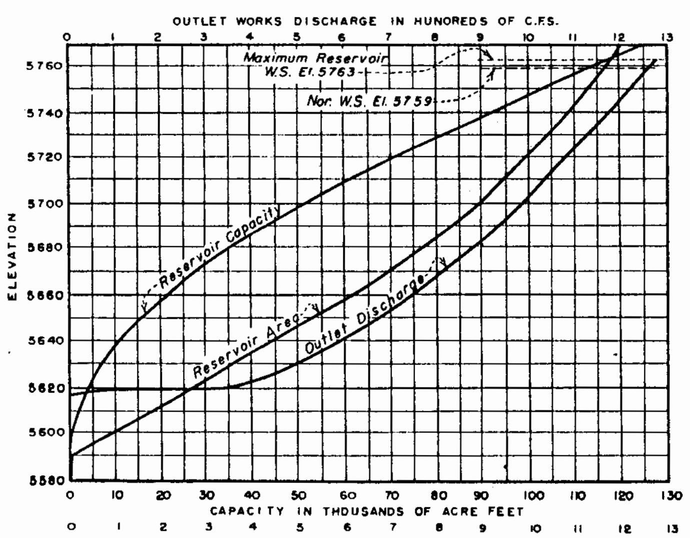




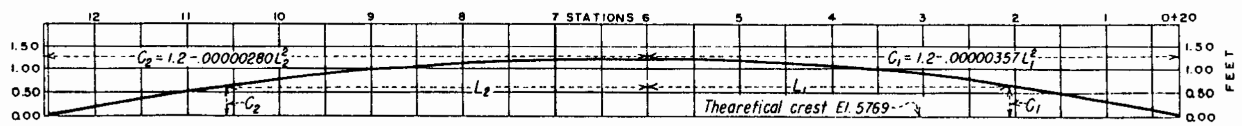


CREST ELEVATIONS WITH CAMBER

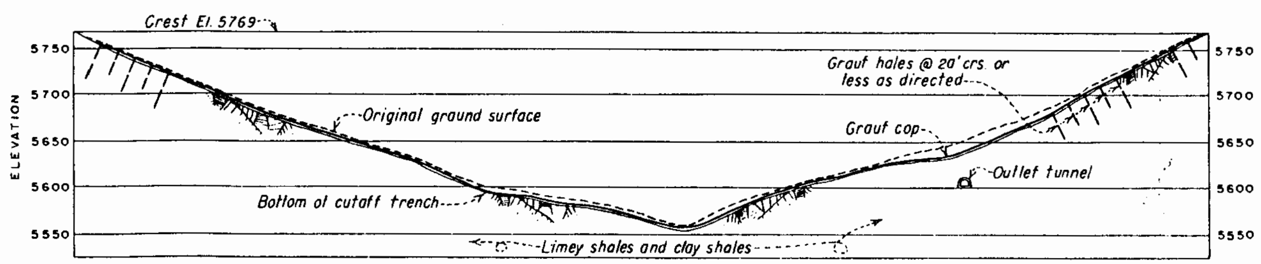
STATION	ELEVATION
0+20	5769.00
1+00	5769.31
2+00	5769.63
3+00	5769.88
4+00	5770.06
5+00	5770.16
6+00	5770.20
7+00	5770.17
8+00	5770.09
9+00	5769.95
10+00	5769.75
11+00	5769.50
12+00	5769.19
12+55	5769.00



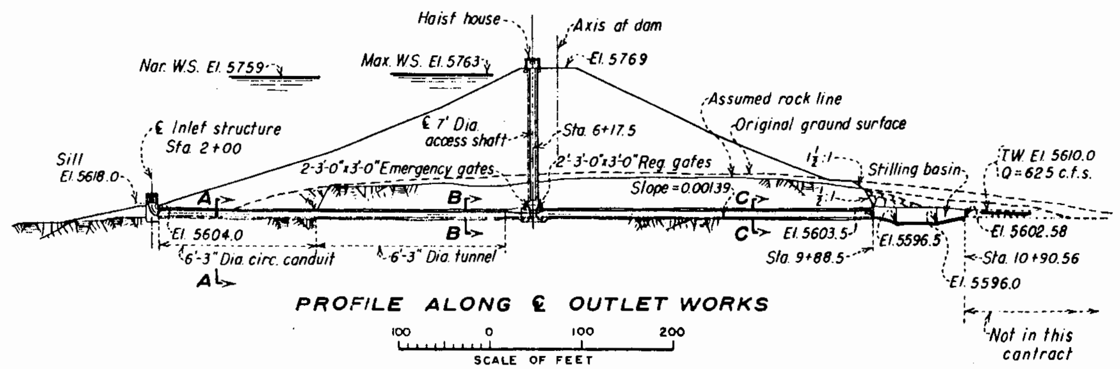
AREA-CAPACITY-DISCHARGE CURVES



CAMBER ON CREST OF DAM



PROFILE ON AXIS OF DAM



PROFILE ALONG ℄ OUTLET WORKS

SCALE OF FEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO-BIG THOMPSON PROJECT-COLO.
CARTER LAKE RESERVOIR

DAM No. 1

GENERAL PLAN AND SECTIONS

DRAWN A.Z. SUBMITTED [signature]
TRACED R.V.S. RECOMMENDED [signature]
CHECKED [signature] APPROVED [signature] CHIEF ENGINEER
DENVER, COLORADO, APR. 3, 1950

245-D-5574

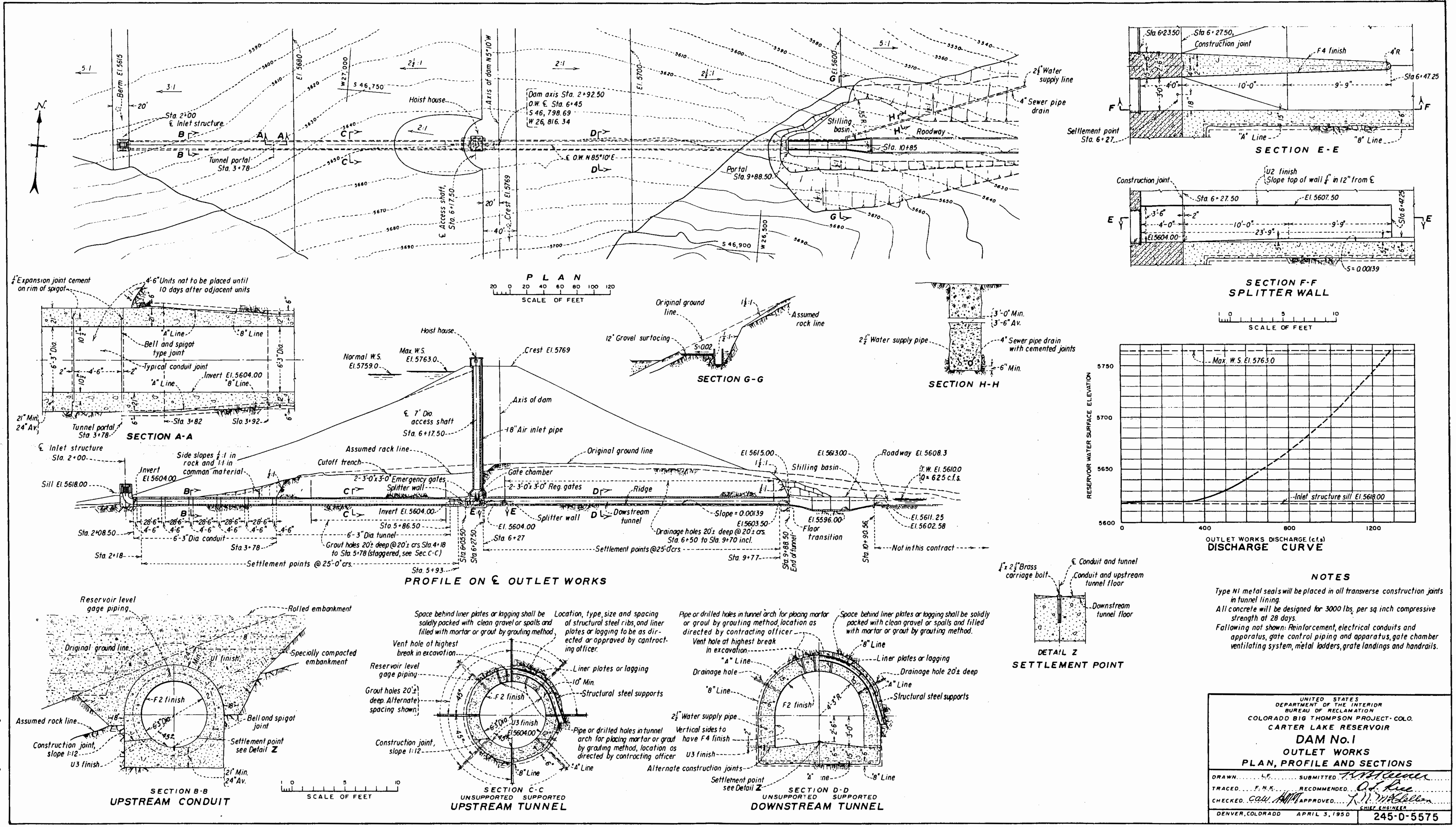
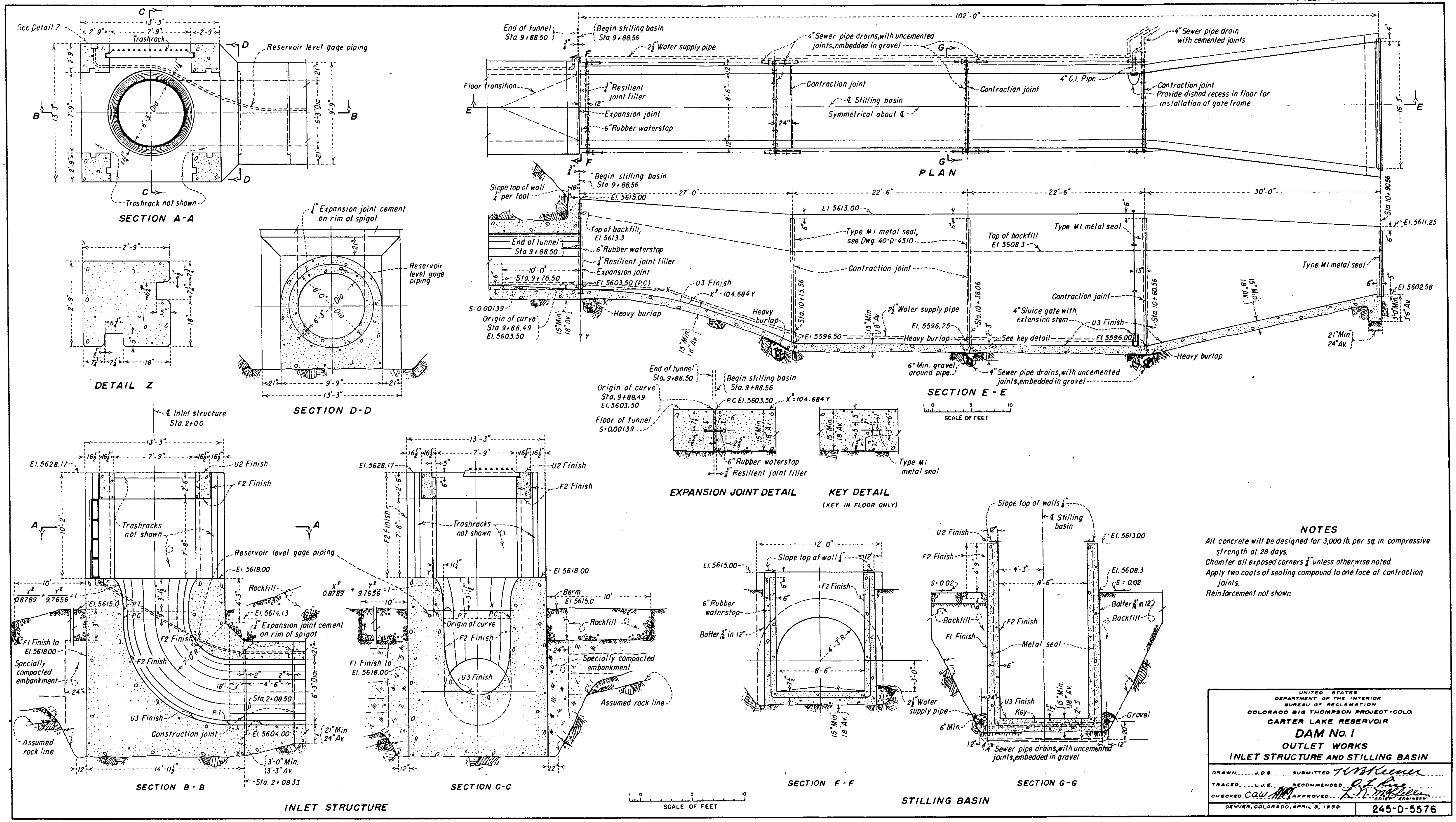
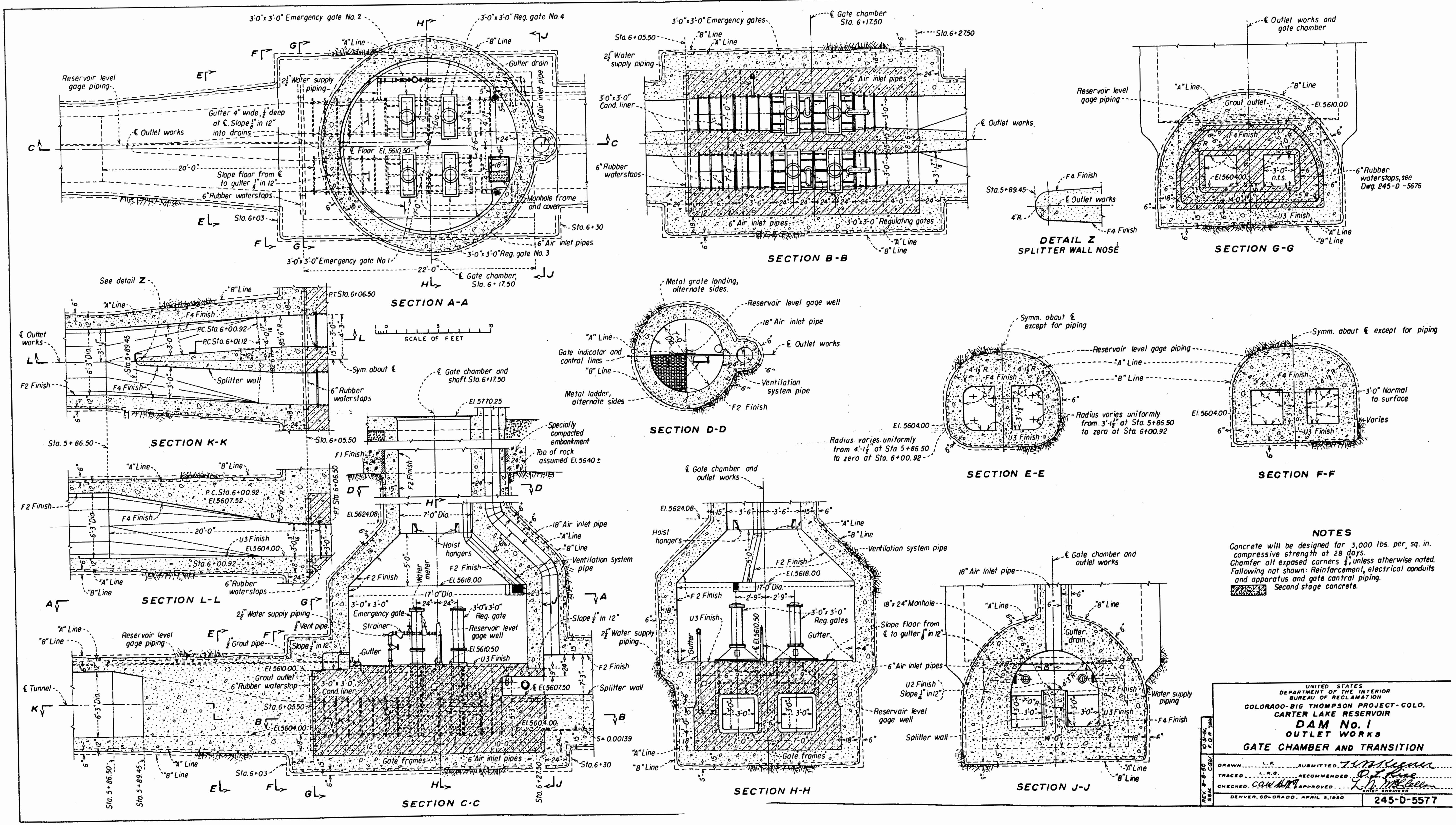
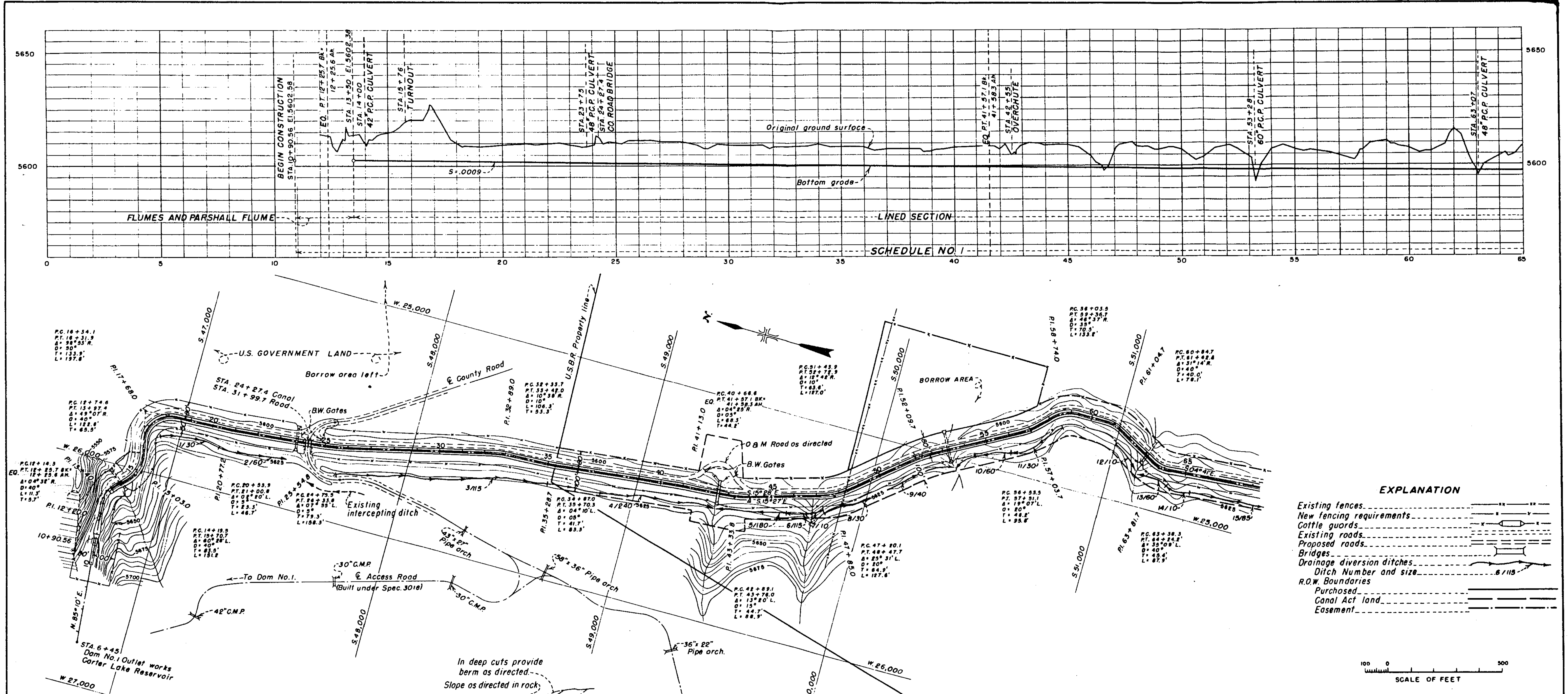
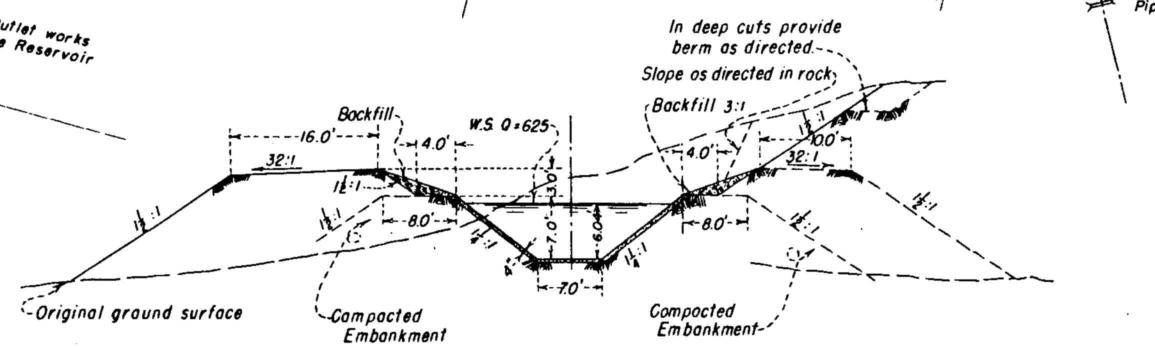

PLAN
SCALE OF FEET
SECTION E-E
SECTION F-F
SPLITTER WALL
SCALE OF FEET
SECTION A-A
SECTION G-G
SECTION H-H
PROFILE ON ℄ OUTLET WORKS
DISCHARGE CURVE
OUTLET WORKS DISCHARGE (c.f.s)
RESERVOIR WATER SURFACE ELEVATION
Max. W.S. El. 5763.0
Inlet structure sill El. 5618.00
SECTION B-B
UPSTREAM CONDUIT
SCALE OF FEET
SECTION C-C
UNSUPPORTED SUPPORTED
UPSTREAM TUNNEL
SECTION D-D
UNSUPPORTED SUPPORTED
DOWNSTREAM TUNNEL
DETAIL Z
SETTLEMENT POINT
NOTES
Type N1 metal seals will be placed in all transverse construction joints in tunnel lining.
All concrete will be designed for 3000 lbs. per sq inch compressive strength at 28 days.
Following not shown: Reinforcement, electrical conduits and apparatus, gate control piping and apparatus, gate chamber ventilating system, metal ladders, grate landings and handrails.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO BIG THOMPSON PROJECT-COLO.
CARTER LAKE RESERVOIR
DAM No. I
OUTLET WORKS
PLAN, PROFILE AND SECTIONS
DRAWN L.F. SUBMITTED
TRACED F.N.K. RECOMMENDED
CHECKED APPROVED
CHIEF ENGINEER
DENVER, COLORADO APRIL 3, 1950
245-D-5575

FIGURE 5
REPORT HYD. 394




SECTION A-A
SECTION B-B
DETAIL Z
SPLITTER WALL NOSE
SECTION G-G
SECTION K-K
SECTION D-D
SECTION E-E
SECTION F-F
SECTION L-L
SECTION C-C
SECTION H-H
SECTION J-J
SCALE OF FEET
NOTES
Concrete will be designed for 3,000 lbs. per sq. in. compressive strength at 28 days.
Chamfer all exposed corners 3/4", unless otherwise noted.
Following not shown: Reinforcement, electrical conduits and apparatus and gate control piping.
Second stage concrete.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO-BIG THOMPSON PROJECT-COLO.
CARTER LAKE RESERVOIR
DAM No. I
OUTLET WORKS
GATE CHAMBER AND TRANSITION
DENVER, COLORADO, APRIL 3, 1950
245-D-5577





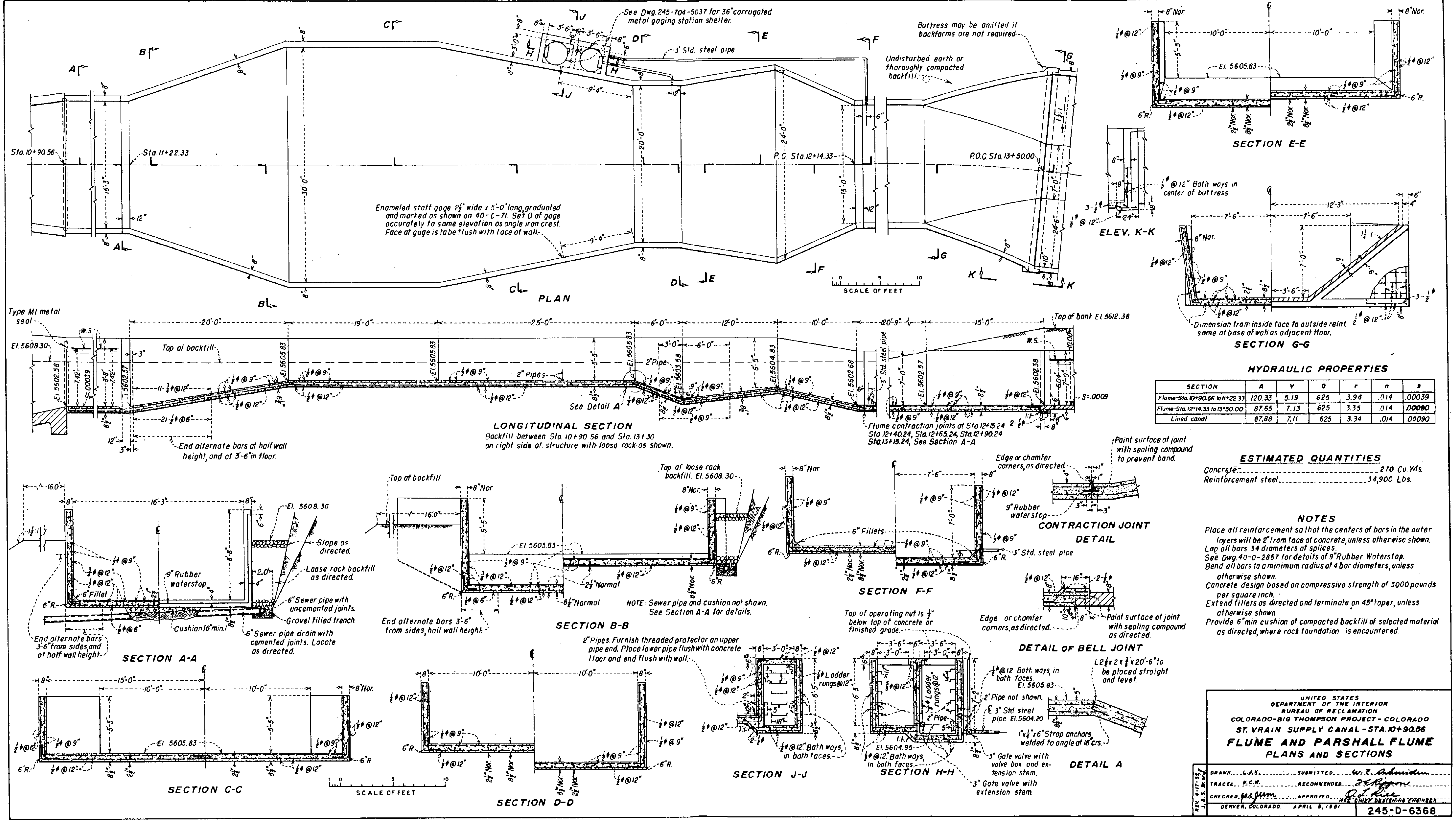
TYPICAL LINED SECTION

HYDRAULIC PROPERTIES						
SECTION	Q	A	V	r	n	s
LINED SECTION	625	87.88	7.11	3.34	.014	.0009

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO - BIG THOMPSON PROJECT - COLO.

ST. VRAIN SUPPLY CANAL
STA. 10+90.56 to STA. 64+00
PLAN, PROFILE, AND SECTIONS

DRAWN T.E.T. - C.T.B. SUBMITTED Harry L. Keve
TRACED R.F.T. (704) RECOMMENDED O. L. Rice
CHECKED APPROVED W. E. Blomgren ACT. CHIEF ENGINEER
DENVER, COLORADO NOV. 15, 1951 SHEET 1 OF 9
245-704-5025



HYDRAULIC PROPERTIES

SECTION	A	V	Q	r	n	s
Flume-Sta. 10+90.56 to 11+22.33	120.33	5.19	625	3.94	.014	.00039
Flume-Sta. 12+14.33 to 13+50.00	87.65	7.13	625	3.35	.014	.00090
Lined canal	87.88	7.11	625	3.34	.014	.00090

ESTIMATED QUANTITIES

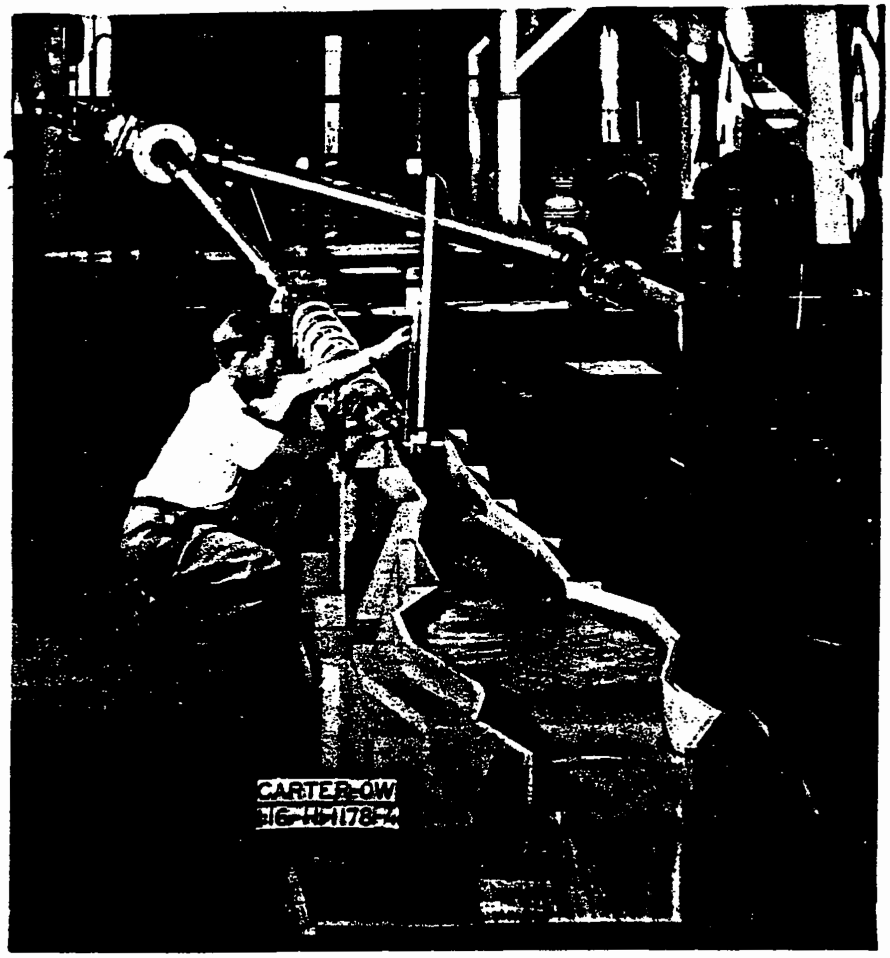
Concrete .... 270 Cu. Yds.
Reinforcement steel .... 34,900 Lbs.

NOTES

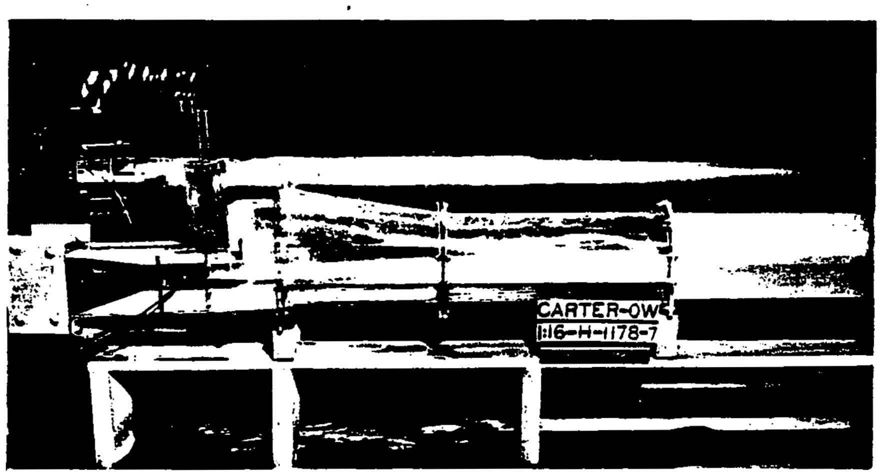
Place all reinforcement so that the centers of bars in the outer layers will be 2" from face of concrete, unless otherwise shown.
Lap all bars 34 diameters at splices.
See Dwg. 40-0-2867 for details of 9" Rubber Waterstop.
Bend all bars to a minimum radius of 4 bar diameters, unless otherwise shown.
Concrete design based on compressive strength of 3000 pounds per square inch.
Extend fillets as directed and terminate on 45° taper, unless otherwise shown.
Provide 6" min. cushion of compacted backfill of selected material as directed, where rock foundation is encountered.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO-BIG THOMPSON PROJECT - COLORADO
ST. VRAIN SUPPLY CANAL - STA. 10+90.56
FLUME AND PARSHALL FLUME
PLANS AND SECTIONS

REV. 4-17-55 J.A.S.
DRAWN L.J.K. SUBMITTED W. E. Schmidt
TRACED W.C.W. RECOMMENDED H.E. Rippon
CHECKED Jed Jum APPROVED O. L. Rice, ACT. CHIEF DESIGNING ENGINEER
DENVER, COLORADO. APRIL 5, 1951
245-D-6368

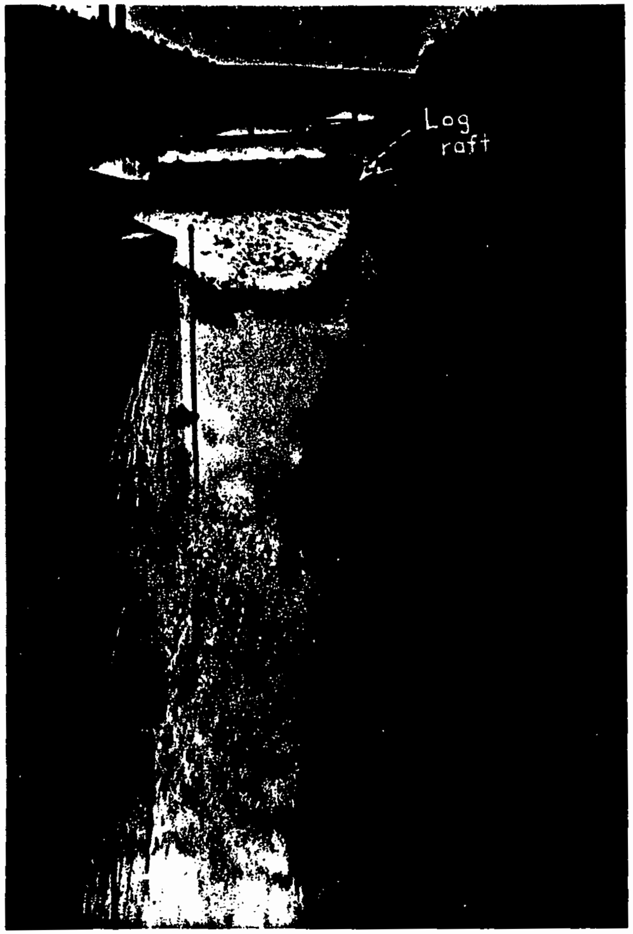


Tunnel, stilling basin, and Parshall flume



Gate Section and Tunnel

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
MODEL VIEWS
1:16 SCALE MODEL



Looking downstream from Parshall flume

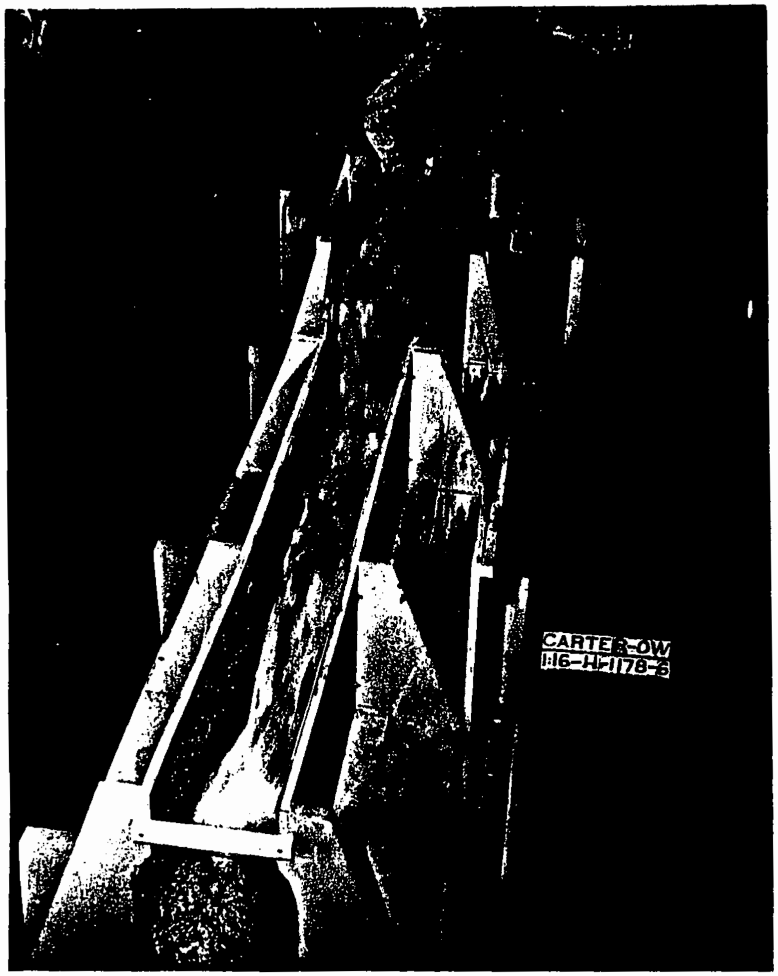
Looking downstream through basin

Looking upstream toward Parshall flume

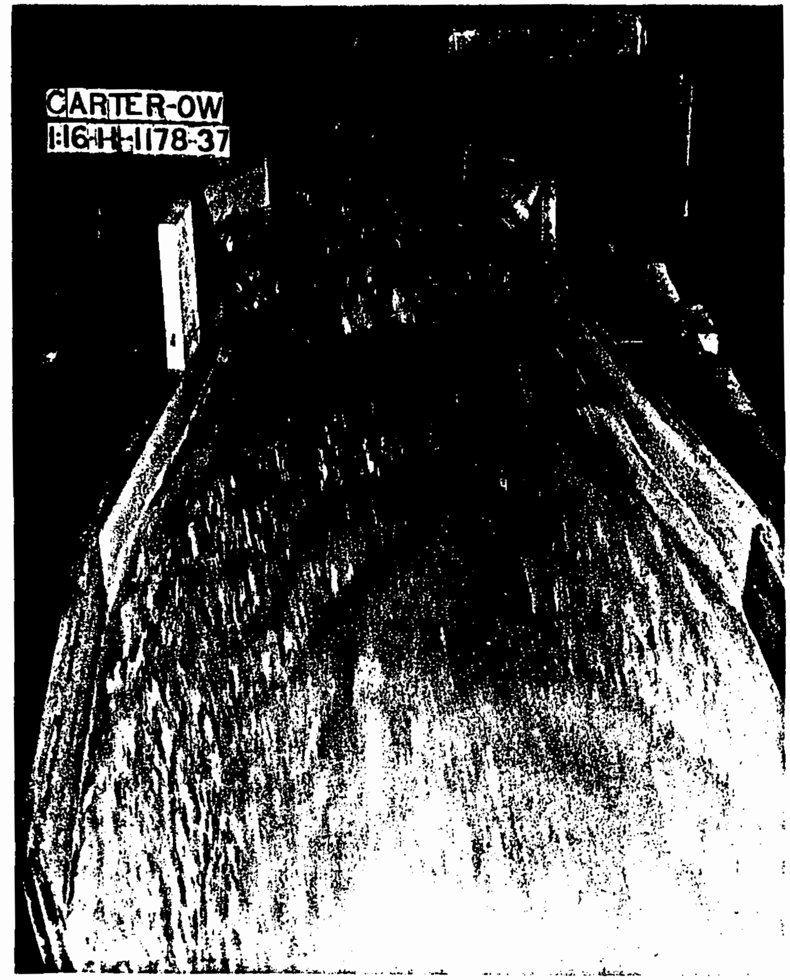
Hydraulic jump has swept out of the basin and through the Parshall flume

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
ORIGINAL STRUCTURE - 625 SECOND-FEET - HEAD 100 FEET
PROTOTYPE VIEWS

Figure 11
Report Hyd 394



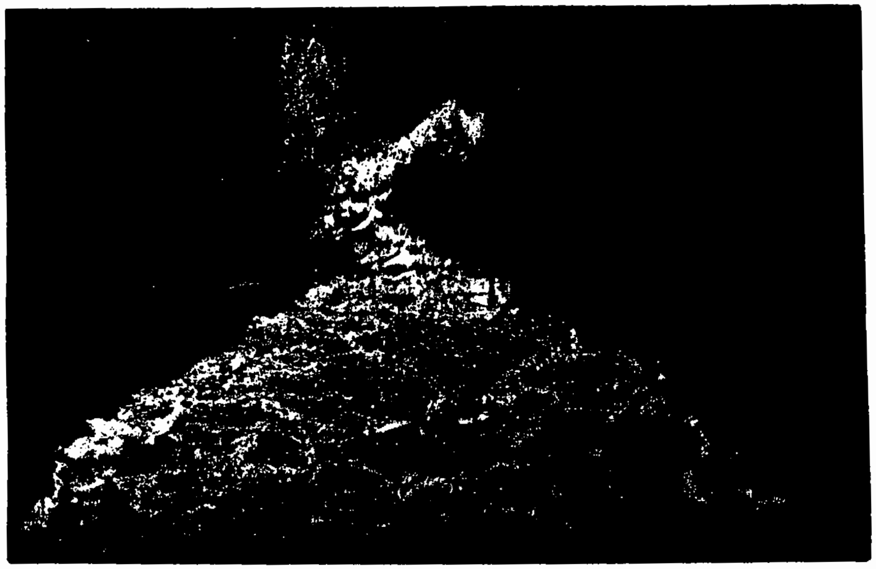
Looking downstream outlet stilling basin and Parshall flume



Looking through Parshall flume

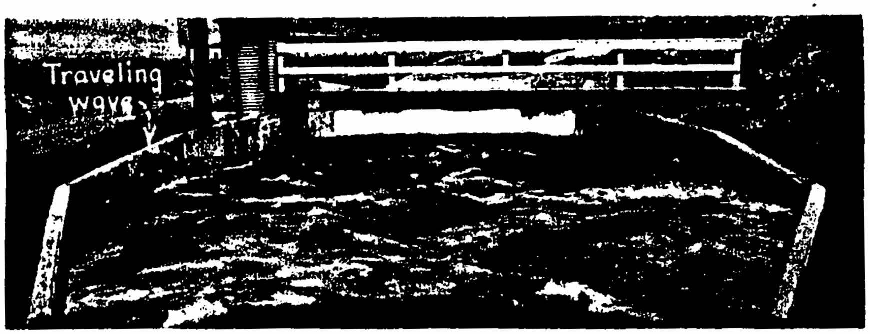
Hydraulic jump has swept out of the basin and through the Parshall flume

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
ORIGINAL STRUCTURE - 625 SECOND-FEET - MAXIMUM HEAD
1:16 SCALE MODEL



Looking upstream into the basin. Hydraulic jump is near the sweep out condition.

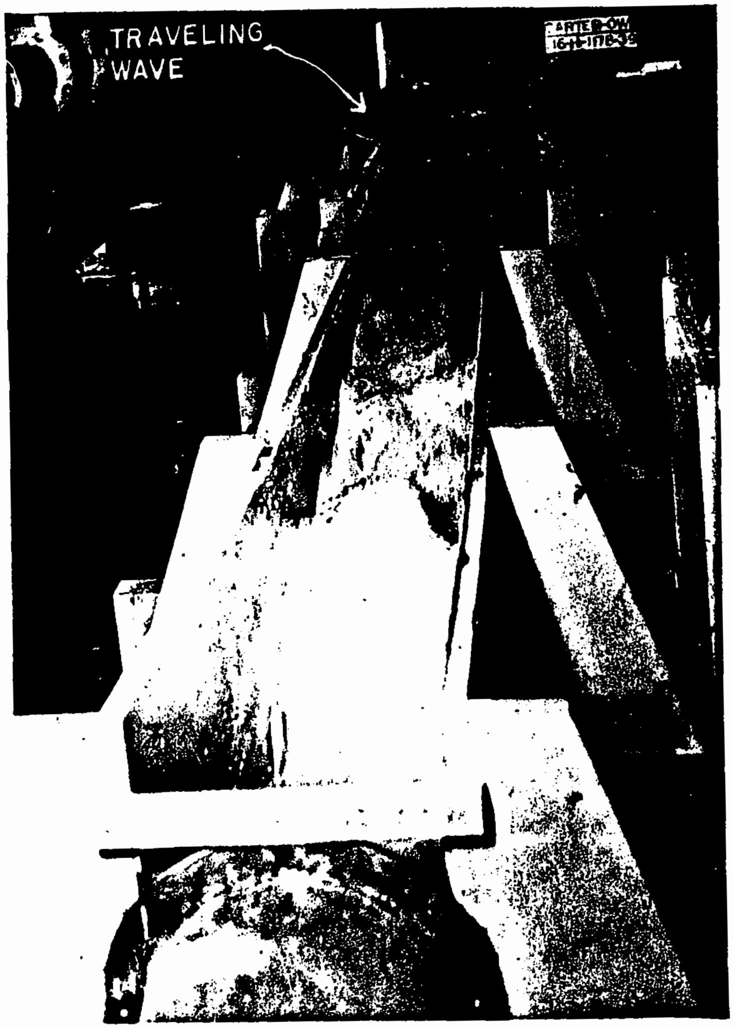
Looking downstream from Parshall flume. Standing waves are developed here.



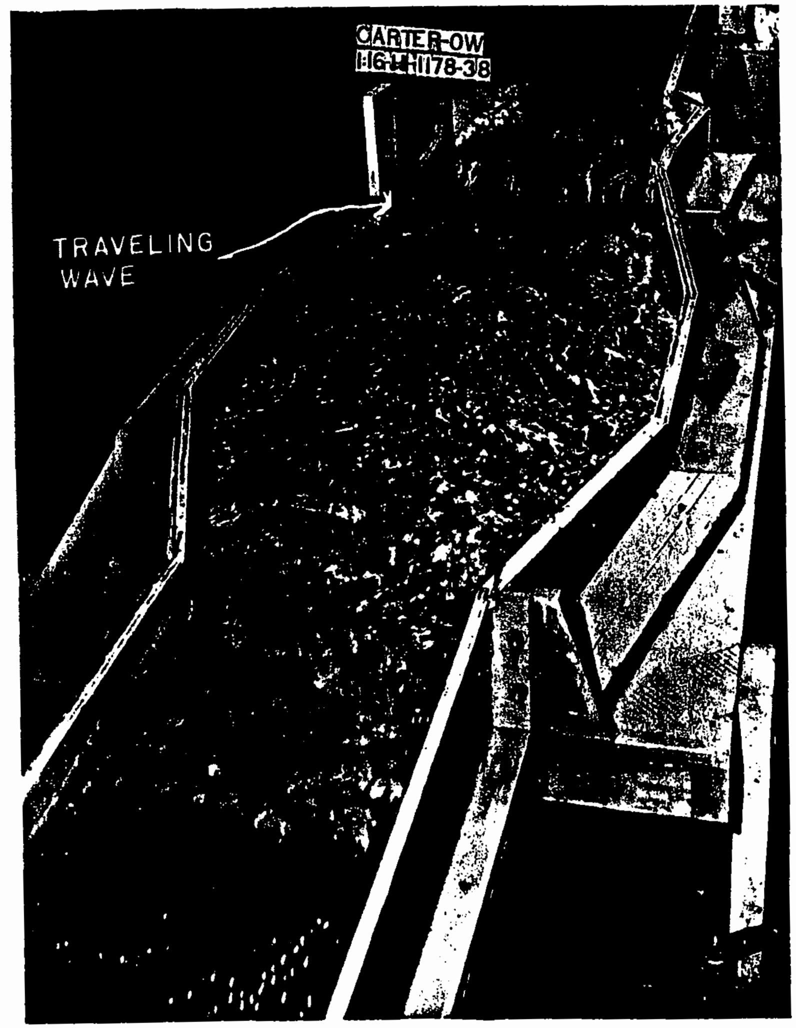
Looking downstream. Waves from the stilling basin travel through the Parshall flume.

Looking downstream. Waves traveling through the canal over top the canal lining at the horizontal bends.

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
ORIGINAL STRUCTURE - 560 SECOND-FEET - HEAD APPROX'LY 100 FEET
PROTOTYPE VIEWS



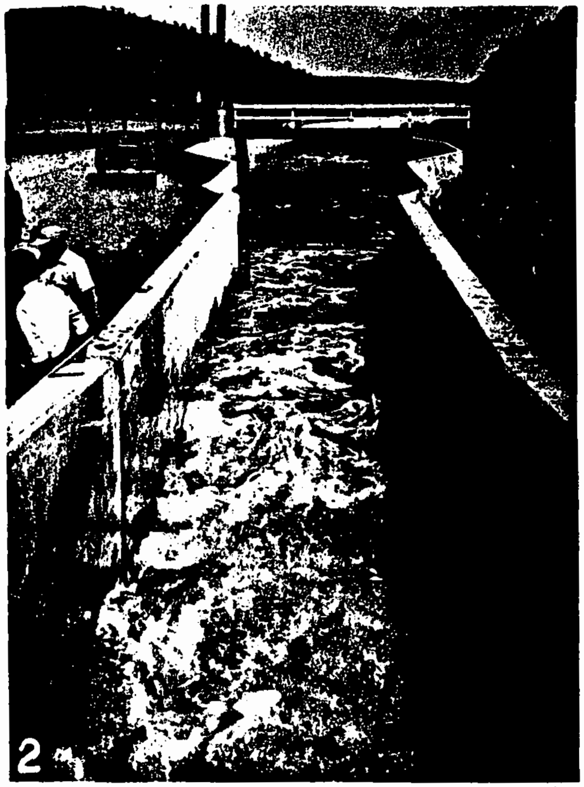
Hydraulic jump is near sweepout in the stilling basin



Waves from the stilling basin travel through the Parshall flume

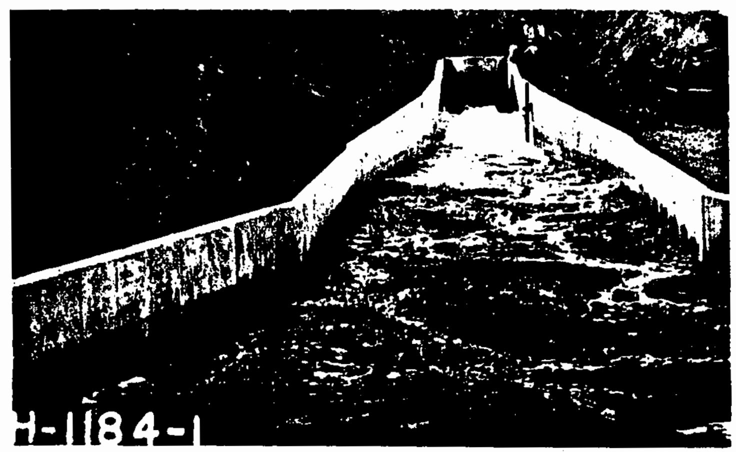
CARTER LAKE OUTLET WORKS AND ST.VRAIN CANAL
ORIGINAL STRUCTURE - 560 SECOND-FEET - AT HIGH HEAD
1:16 SCALE MODEL

Figure 13
Report Hyd 394



Water surface is rough in the stilling basin

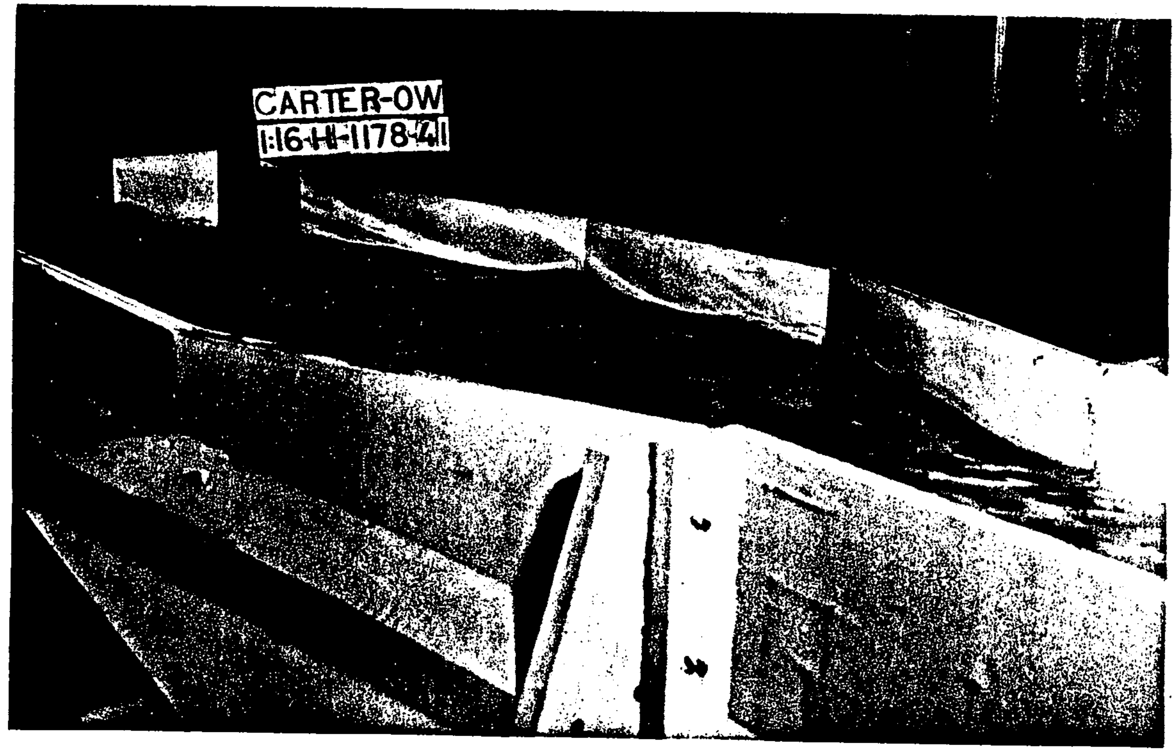
Waves travel through the Parshall flume



Looking upstream into basin

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
ORIGINAL STRUCTURE - 230 SECOND-FEET - HEAD 45 FEET
PROTOTYPE VIEWS

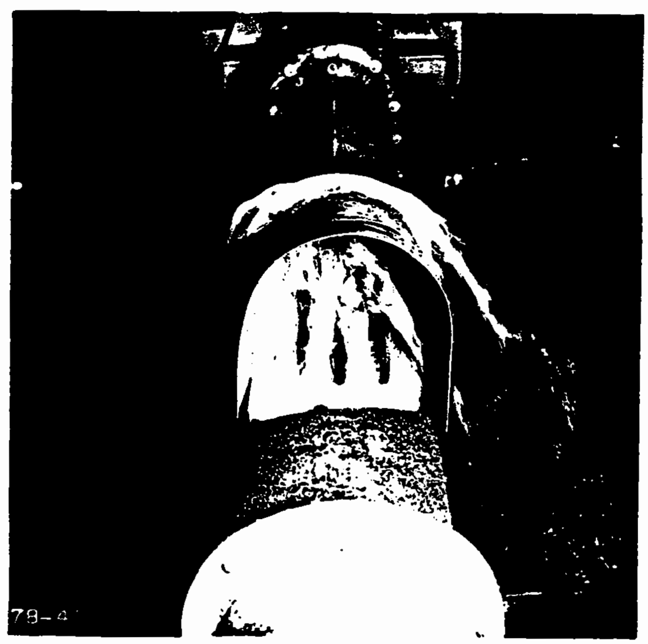
Water surface is rough in the stilling basin



The waves from the basin travel through the parshall flume

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
ORIGINAL STRUCTURE - 230 SECOND-FEET AT HIGH HEAD
1:16 SCALE MODEL

A. Vertical nose piers 2.5 feet high and one foot wide set on tunnel floor



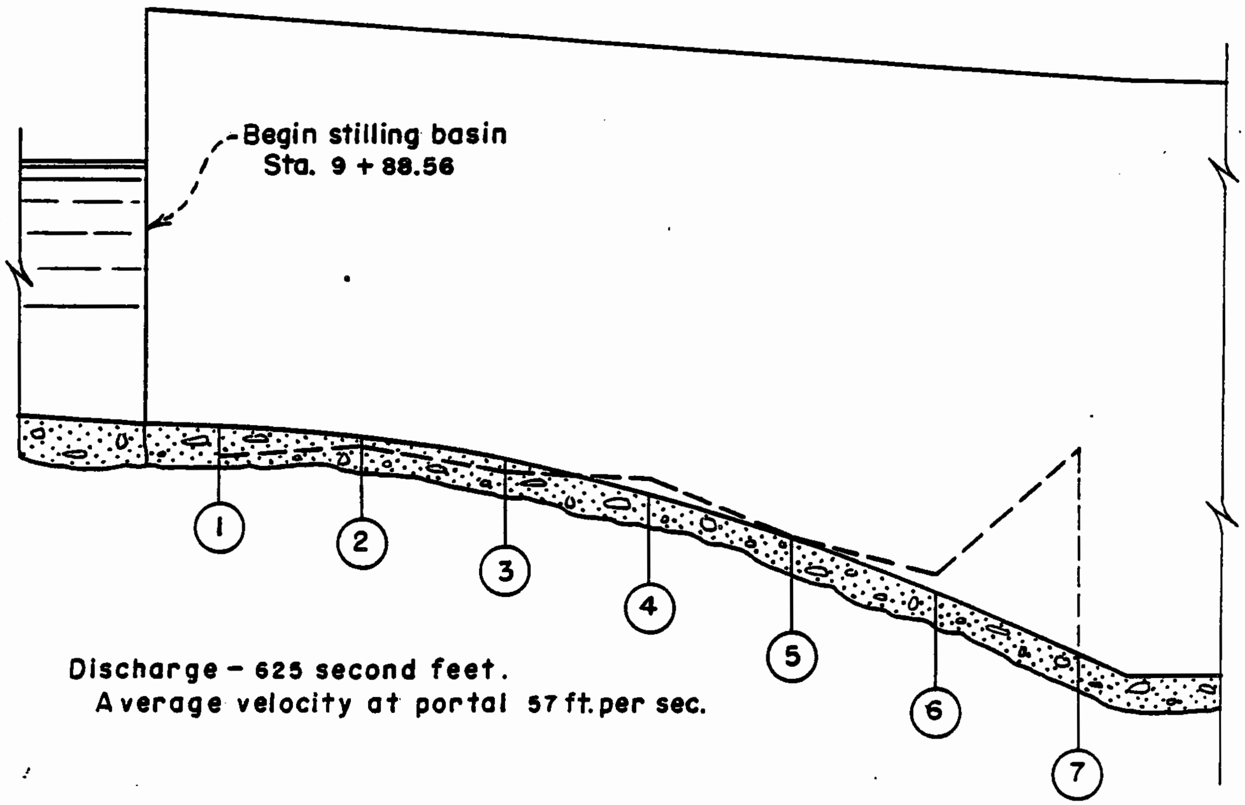
B. Discharge 625 second-feet - Maximum head

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
BAFFLE PIERS IN TUNNEL
1:16 SCALE MODEL

A. Portal Piers are 2.5 feet high and 16 inches wide

B. Two sets of tunnel piers as shown in Figure 16 plus three portal piers shown above. 625 second-feet discharging at maximum head.

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
BAFFLE PIERS IN TUNNEL AND AT PORTAL
1:16 SCALE MODEL



SECTION ON ℄ OF BASIN

Note: Circled numbers indicate piezometer locations.
Pressures above and below atmospheric are plotted above and below the trajectory of the basin floor respectively.

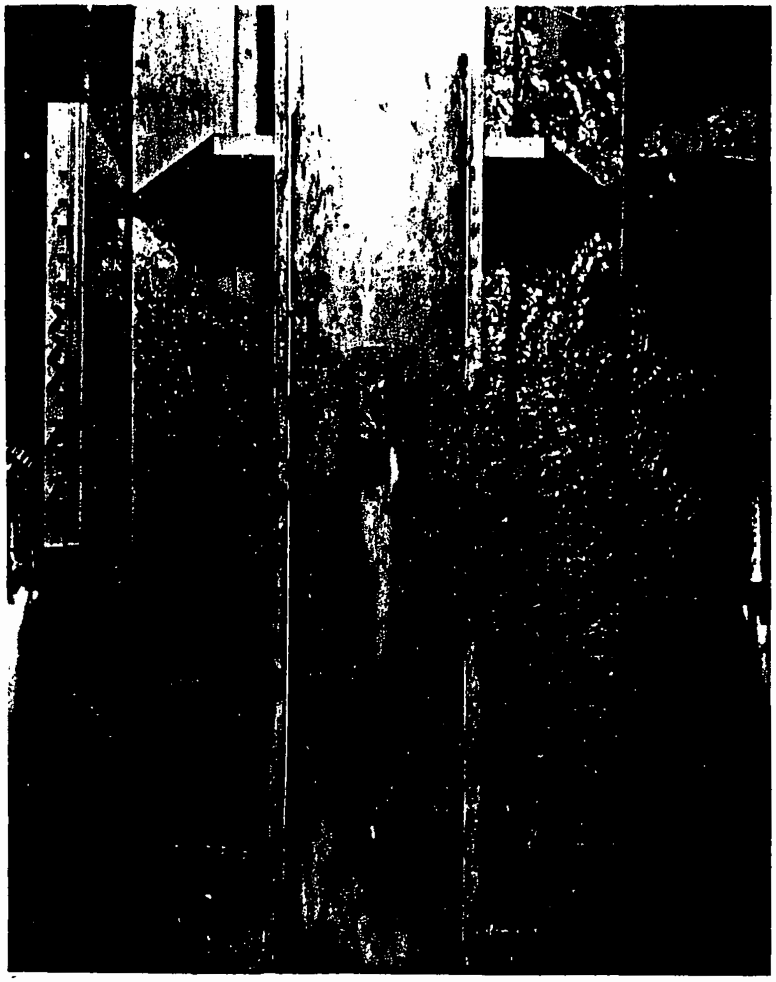
PRESSURE SCALE IN FEET OF WATER

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL

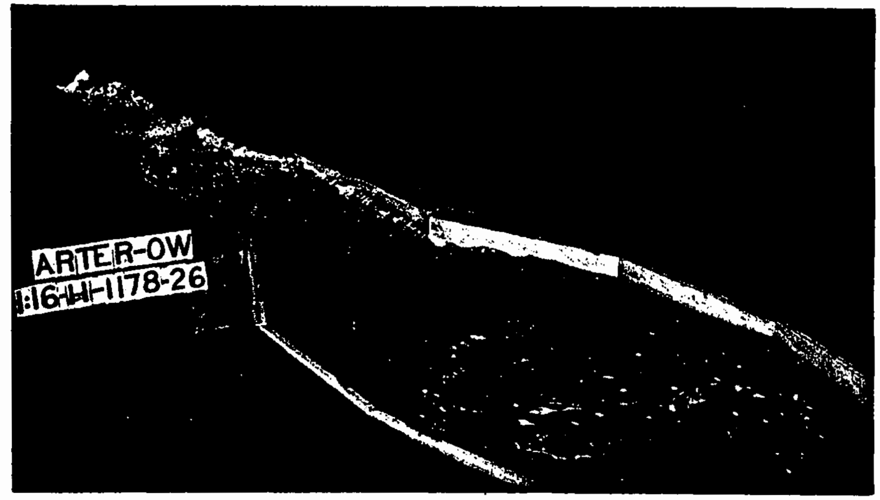
PRESSURE ON CENTER LINE OF STILLING BASIN TRAJECTORY

1:16 SCALE MODEL

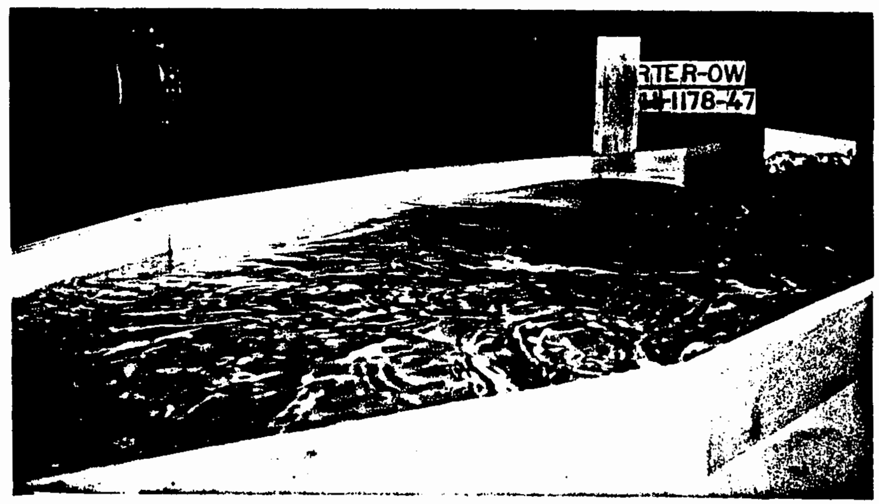
501



A. Performance of baffle piers is demonstrated by one pier with no tailwater cover



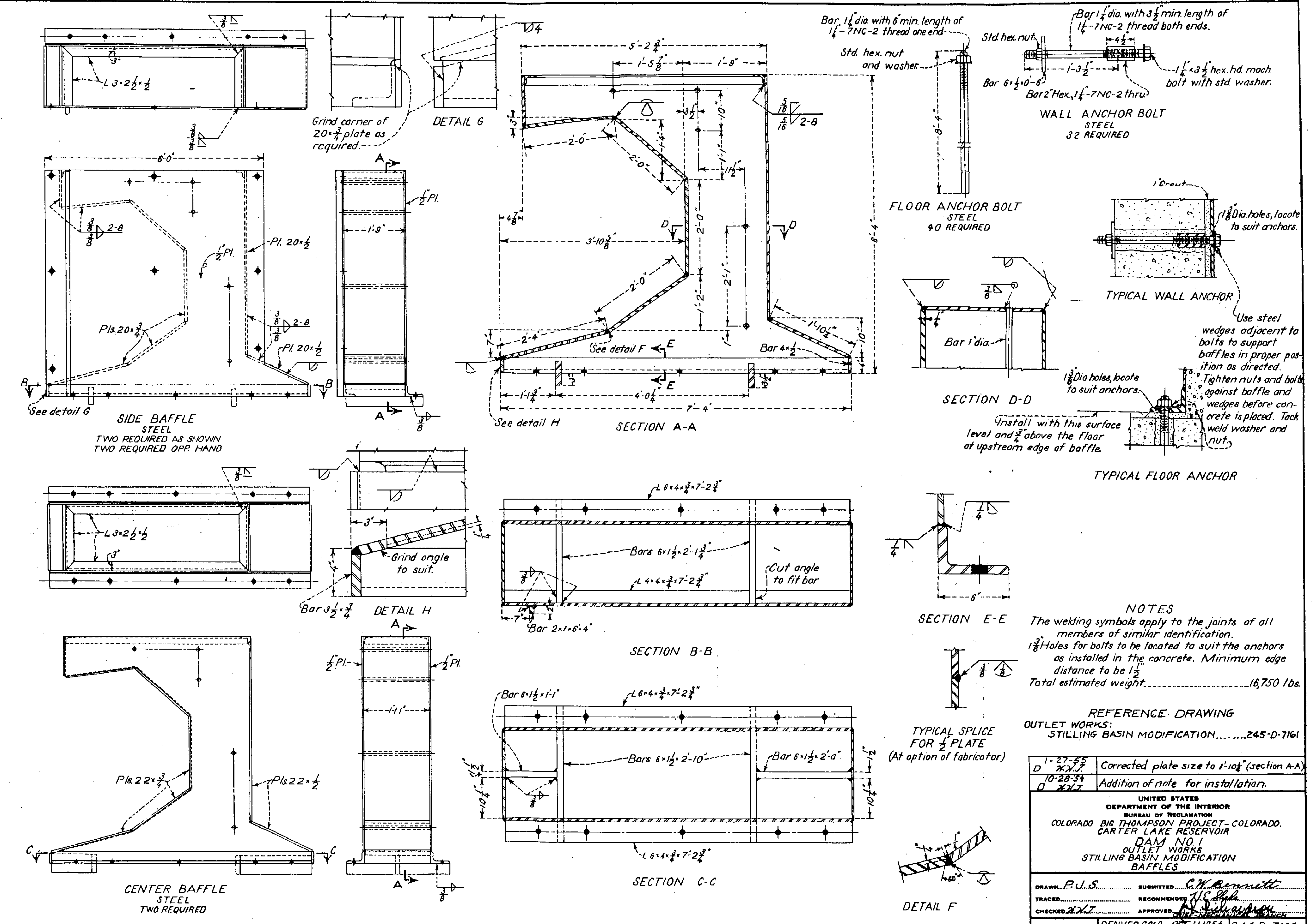
B. Six recommended baffle piers in the stilling basin



C. Water surface in Parshall flume when using six recommended baffle piers

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
RECOMMENDED BAFFLE PIERS IN STILLING BASIN - 625 SECOND-FEET
MAXIMUM HEAD
1:16 SCALE MODEL

Figure 20
Report Hyd 394



NOTES

The welding symbols apply to the joints of all members of similar identification.

$1\frac{3}{8}$" Holes for bolts to be located to suit the anchors as installed in the concrete. Minimum edge distance to be $1\frac{1}{2}$".

Total estimated weight .... 16,750 lbs.

REFERENCE DRAWING

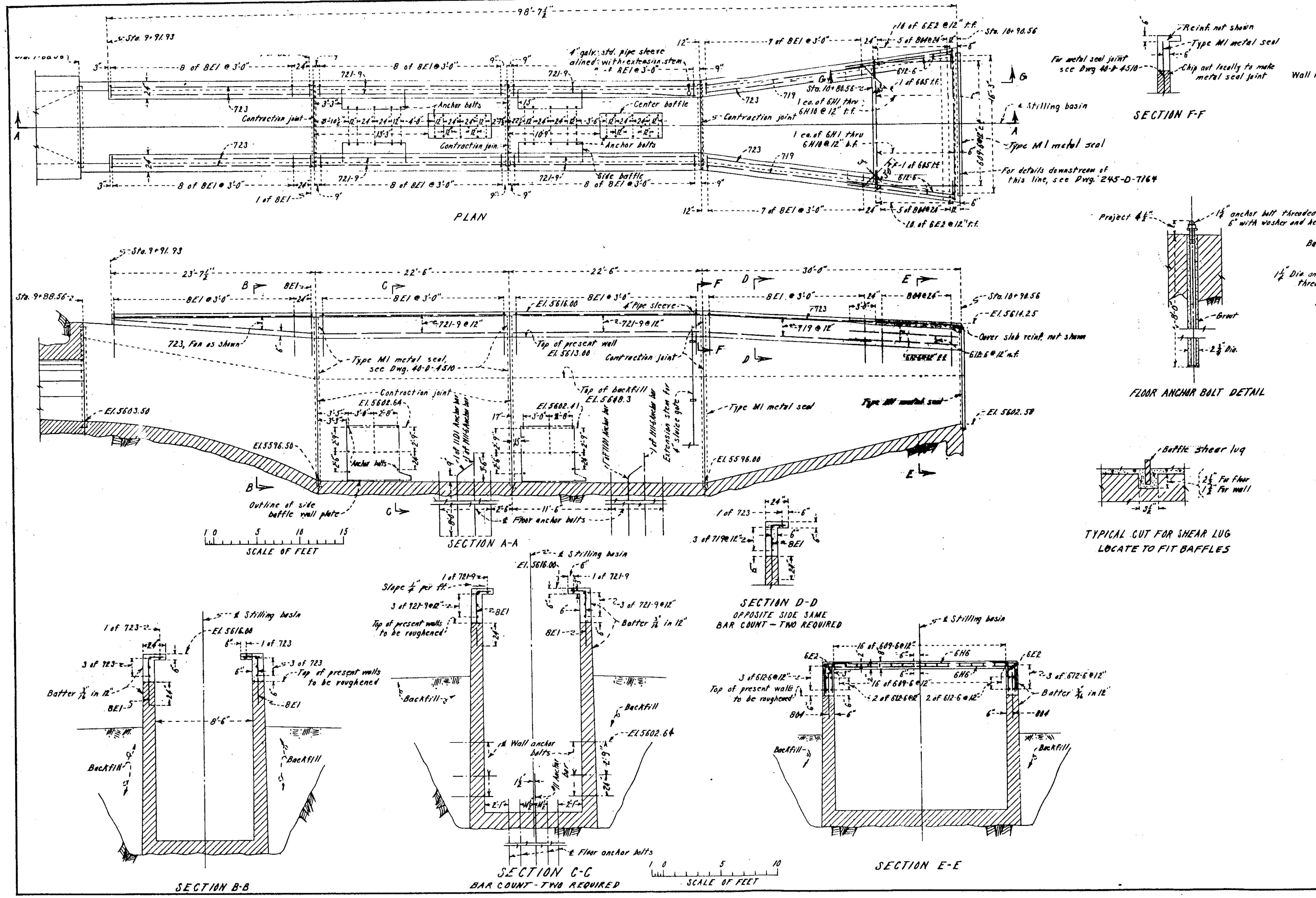
OUTLET WORKS:
STILLING BASIN MODIFICATION 245-D-7161

D	1-27-55	Corrected plate size to 1'-$10\frac{1}{4}$" (section A-A)
D	10-28-54	Addition of note for installation.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO BIG THOMPSON PROJECT-COLORADO.
CARTER LAKE RESERVOIR
DAM NO. 1
OUTLET WORKS
STILLING BASIN MODIFICATION
BAFFLES

DRAWN P.J.S. SUBMITTED C.W. Bennett
TRACED RECOMMENDED
CHECKED
APPROVED

DENVER, COLO. OCT. 1, 1954 — 245-D-7165



NOTES

Design based on concrete of 3,000 lb per sq. in. compressive strength at 28 days.
Chamfer all exposed corners ¾ unless otherwise noted.

f.f. indicates far face; n.f. near face; e.f. each face; t.f. top face; b.f. bottom face.
Drains, water pipes, reinforcement in place, and other misc. metal work not shown.
Apply two coats of sealing compound to one face of contraction joints.
Slope the tops of all walls ¼ per foot.
Designates concrete in place.
Concrete finishes on exposed surfaces F_2 and U_2 unless otherwise noted.
Minimum lap distance of bar splices to be 29 bar diameters of smaller bar unless otherwise noted.
Baffles to be filled with concrete after installation.
Old concrete shall be thoroughly roughened within area covered by steel baffles.

REFERENCE DRAWINGS

OUTLET WORKS
INLET STRUCTURE AND STILLING BASIN 245-D-5576
BAFFLES 245-D-7165

METAL SEAL DETAILS 40-D-4510
REINFORCEMENT LIST 245RL-D-7161-1

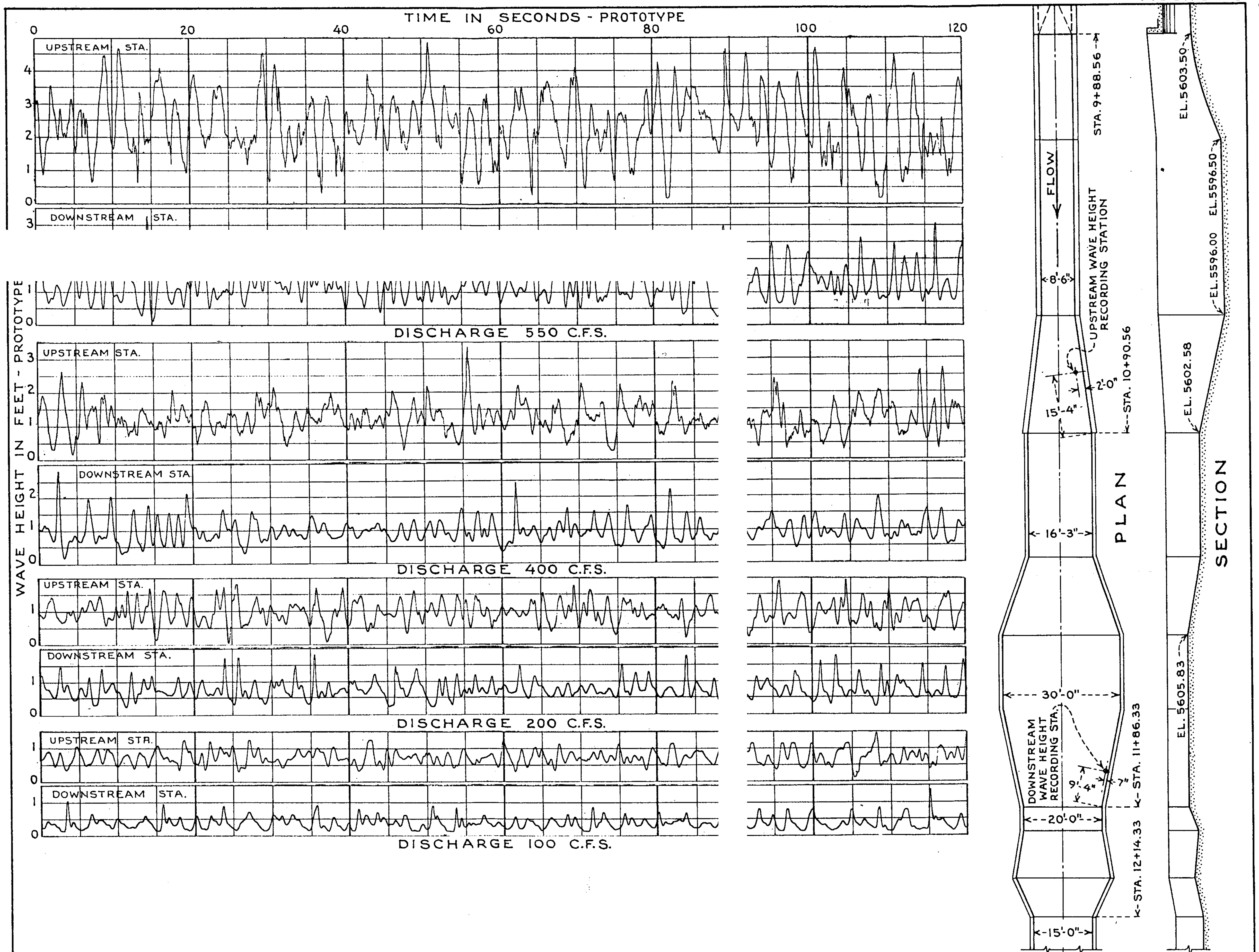
Rev. 11-30-54 KRT — Change of bar identification on Sec. E-E

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO BIG THOMPSON PROJECT - COLO.
CARTER LAKE RESERVOIR
DAM No. 1
OUTLET WORKS
STILLING BASIN MODIFICATION

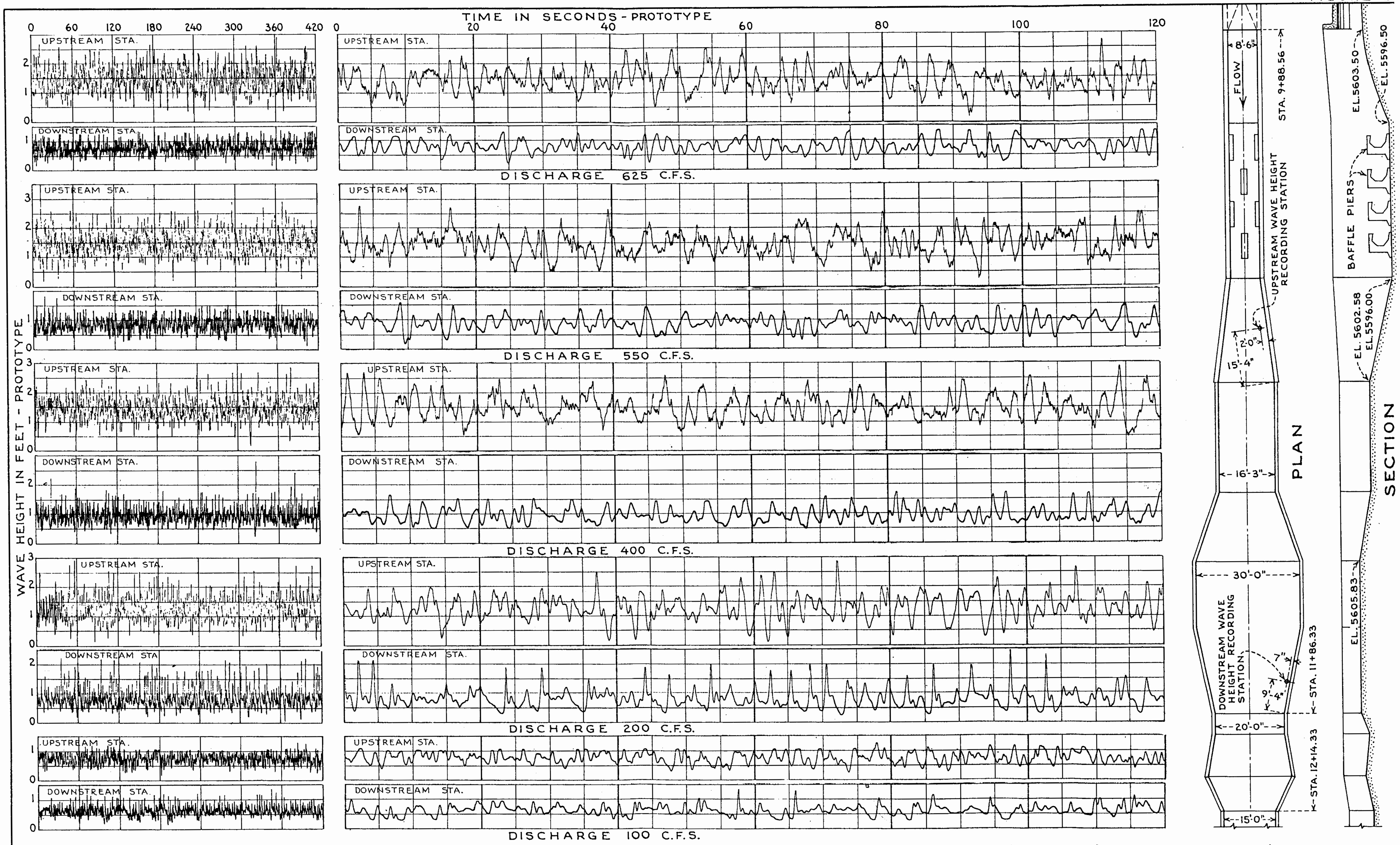
DRAWN KRT SUBMITTED
TRACED RECOMMENDED
CHECKED APPROVED

DENVER, COLORADO SEPTEMBER 29, 1954 245-D-7161

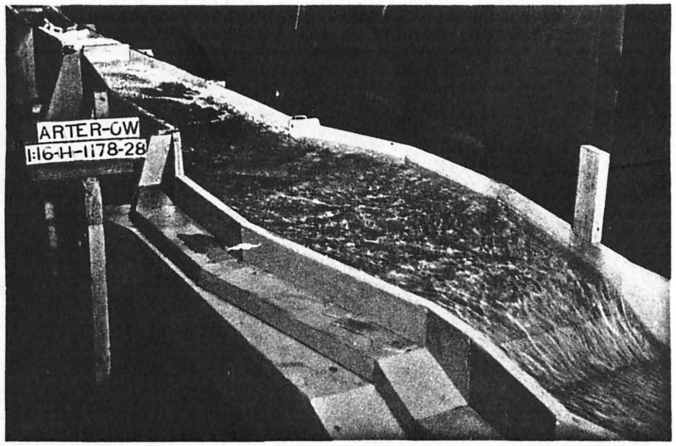
FIGURE 23



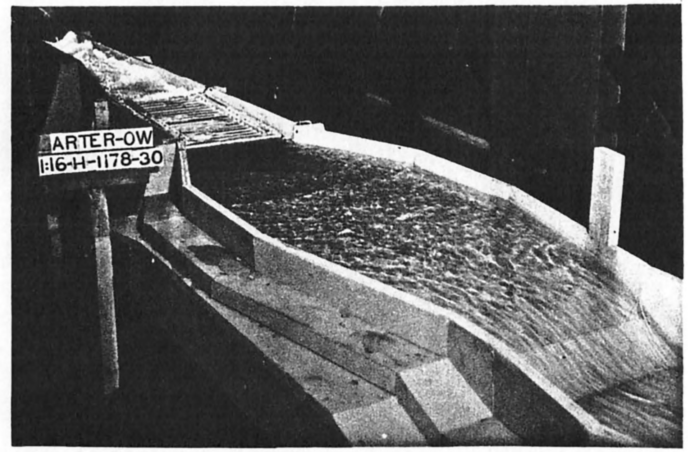
CARTER LAKE DAM NO. 1 • OUTLET WORKS • WAVE HEIGHT RECORD • ORIGINAL STRUCTURE • (MODEL SCALE 1:16)



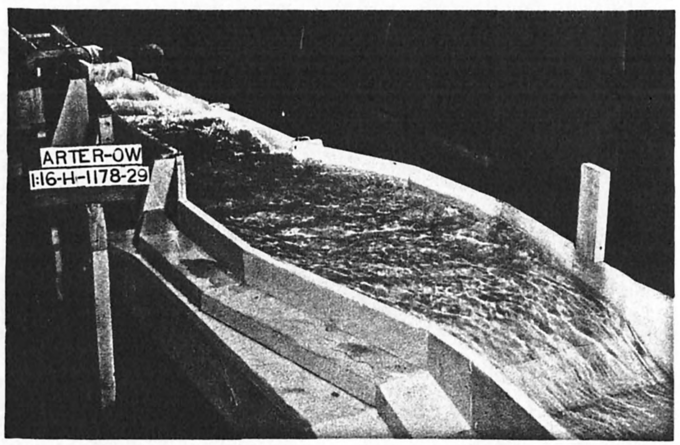
CARTER LAKE DAM NO I • OUTLET WORKS • WAVE HEIGHT RECORDS • ORIGINAL STRUCTURE WITH BAFFLE PIERS • (MODEL SCALE 1:16)



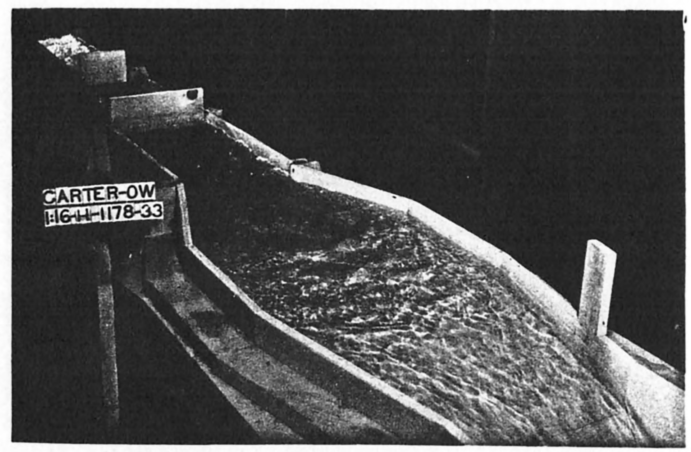
Curtain wall - effective for short choppy waves and a limited range of flows



Rigid raft - reduces waves, ineffective on surges

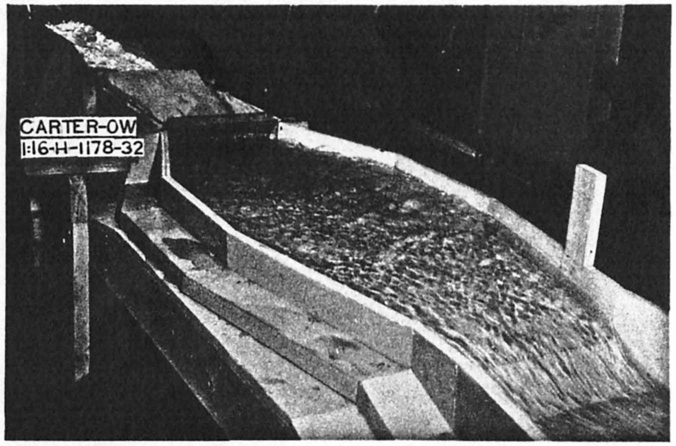


Underpass in stilling basin turbulent flow shoots under the structure

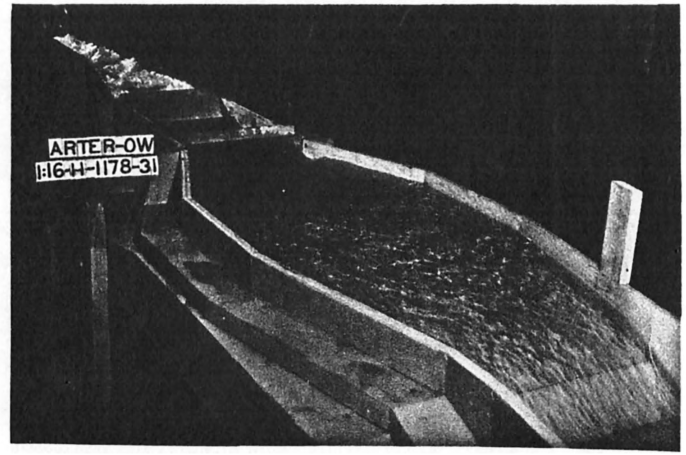


Underpass in transition section. Not as effective as underpass located downstream.

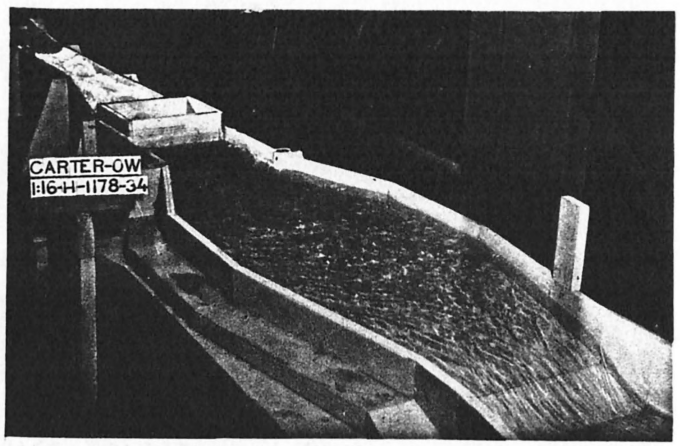
CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
TYPICAL WAVE SUPPRESSORS TESTED Sheet 1 of 2
1:16 SCALE MODEL



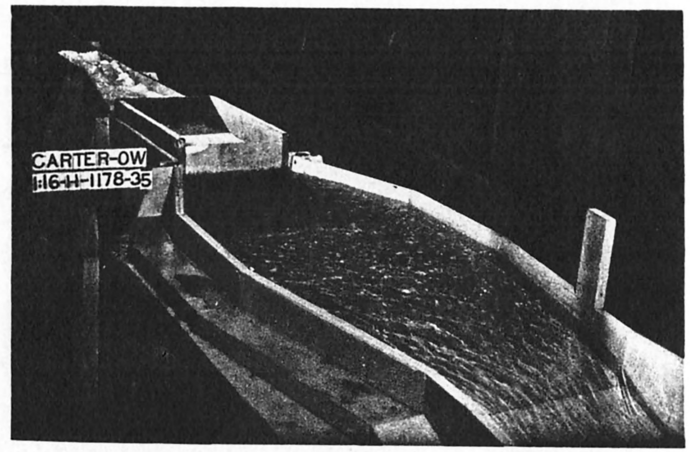
Underpass in rectangular section with 30° sloping entrance and a 45° exit



Underpass with 45° sloping entrance and 30° exit

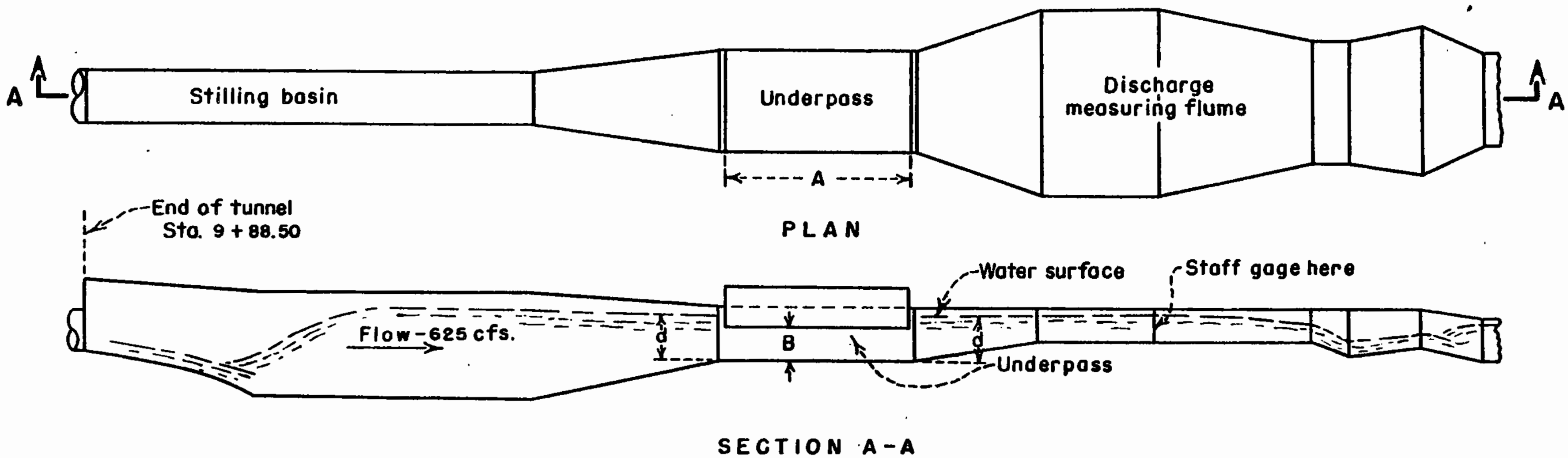


Underpass twice as long as the flow depth, submerged one-fifth of the depth. Fairly effective.



Underpass four times as long as flow depth, submerged one-fifth of the depth. Very effective.

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
TYPICAL WAVE SUPPRESSORS TESTED . Sheet 2 of 2
1:16 SCALE MODEL



MODEL ARRANGEMENT	UNDERPASS LENGTH "A"	HEIGHT "B"	MAX. W.S. FLUCTUATION AT GAGE
Tunnel Piers No Underpass	—	—	2'-8"
Tunnel Piers With Underpass	2 d	4/5 d	1'-0"
Tunnel Piers With Underpass	2 d	3/5 d	0'-8"
Tunnel Piers With Underpass	4 d	4/5 d	0'-6"
Tunnel Piers Plus Portal Piers No Underpass	—	—	1'-4"
Tunnel Piers Plus Portal Piers With Underpass	4 d	4/5 d	0'-3"

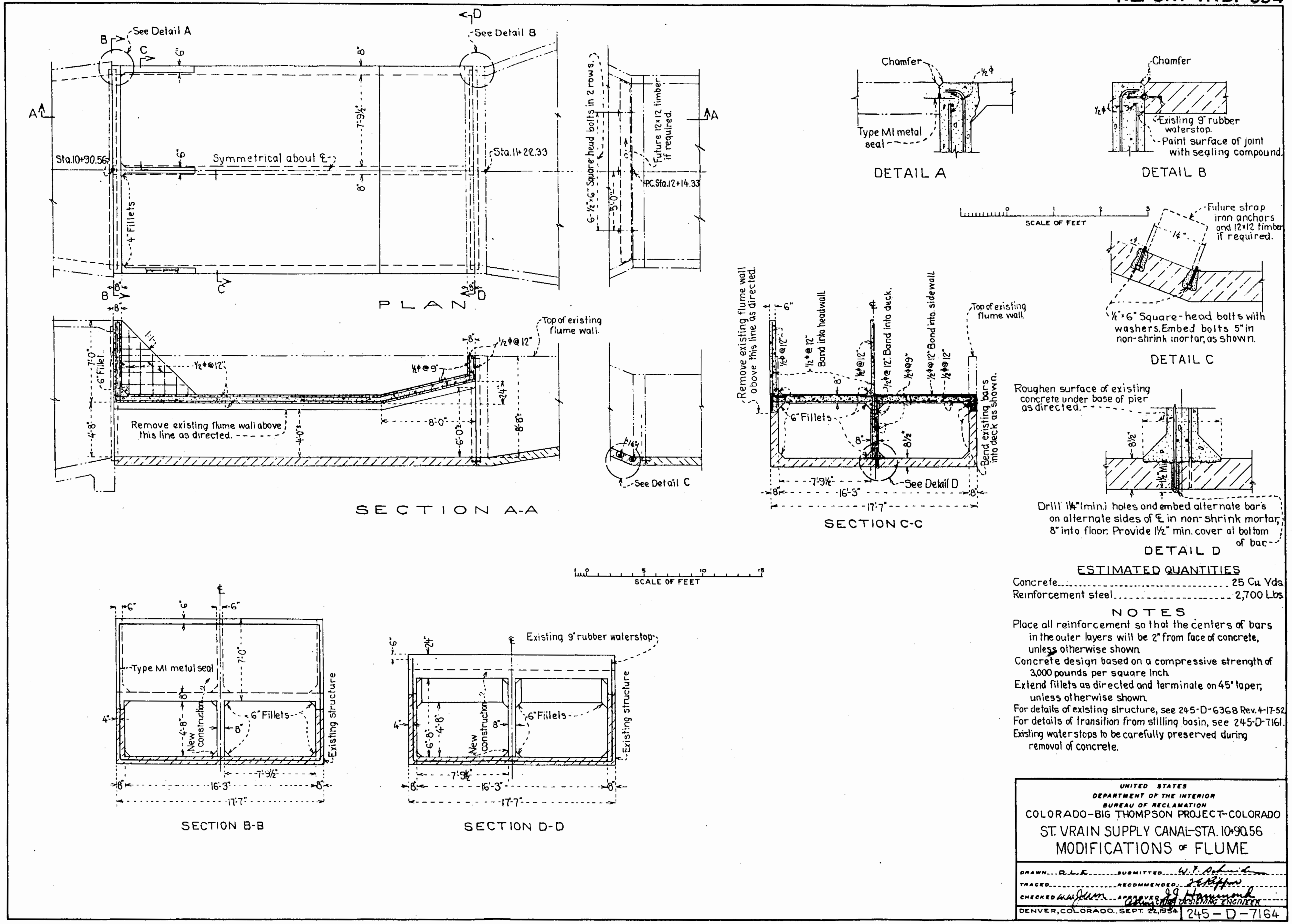
CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL

WATER SURFACE FLUCTUATION FOR UNDERPASS TYPE WAVE SUPPRESSORS

1:16 SCALE MODEL

501

FIGURE 27
REPORT HYD. 394



ESTIMATED QUANTITIES

Concrete .... 25 Cu. Yds.
Reinforcement steel .... 2,700 Lbs.

NOTES

Place all reinforcement so that the centers of bars in the outer layers will be 2" from face of concrete, unless otherwise shown.
Concrete design based on a compressive strength of 3,000 pounds per square inch.
Extend fillets as directed and terminate on 45° taper, unless otherwise shown.
For details of existing structure, see 245-D-6368 Rev. 4-17-52.
For details of transition from stilling basin, see 245-D-7161.
Existing waterstops to be carefully preserved during removal of concrete.

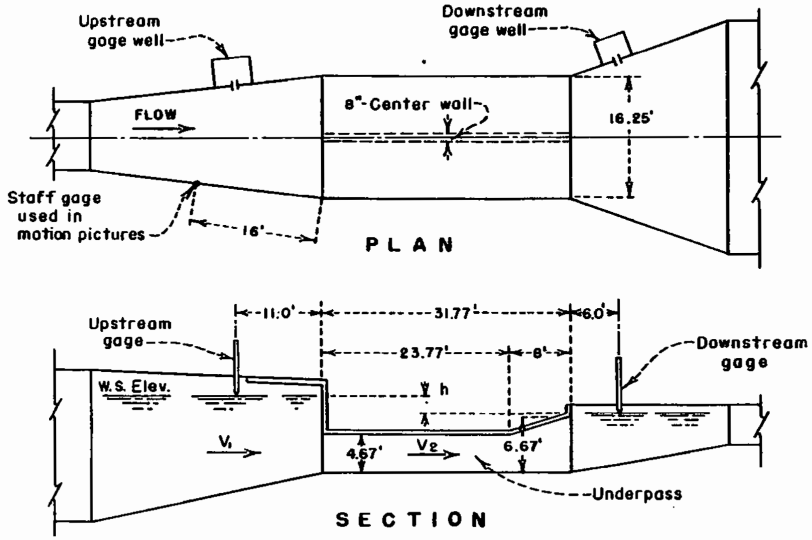
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO-BIG THOMPSON PROJECT-COLORADO
ST. VRAIN SUPPLY CANAL-STA. 10+90.56
MODIFICATIONS OF FLUME

DRAWN B.L.F. SUBMITTED W.T. Schmidt
TRACED RECOMMENDED
CHECKED APPROVED J.J. Hammond
DESIGNING ENGINEER
DENVER, COLORADO, SEPT. 22, 1954 245-D-7164

Abrupt right angle exit

Recommended draft-tube type exit

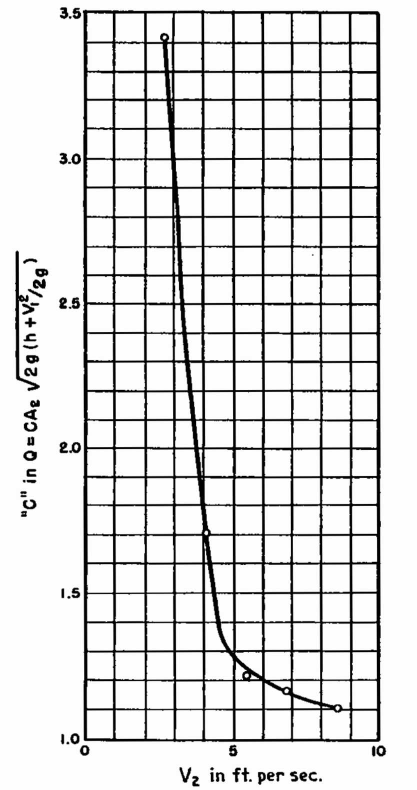
CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
COMPARISON OF TYPES OF OUTLET FOR RECOMMENDED WAVE SUPPRESSOR
1:16 SCALE MODEL



DIS-CHARGE C.F.S.	WITHOUT UNDERPASS		WITH UNDERPASS WAVE SUPPRESSOR							
	W.S. ELEV. AT UP STREAM GAGE (FT.)	W.S. ELEV. AT DOWN STREAM GAGE (FT.)	W.S. ELEV. AT UP STREAM GAGE (FT.)	W.S. ELEV. AT DOWN STREAM GAGE (FT.)	h FT.	V_1* FT./SEC.	$V_1^2/2g$ FT.	$h+V_1^2/2g$ FT.	C**	V_2 FT./SEC.
200	5687.74	5607.74	5607.63	5607.68	-0.05	1.96	0.06	0.01	3.42	2.78
300	5608.06	5608.22	5608.14	5608.17	-0.03	2.76	0.12	0.09	1.71	4.13
400	5608.52	5608.61	5608.73	5608.60	0.13	3.42	0.18	0.31	1.22	5.50
500	5608.96	5609.02	5609.32	5609.04	0.28	4.00	0.25	0.53	1.17	6.88
625	5609.37	5609.44	5609.99	5609.41	0.58	4.67	0.34	0.92	1.11	8.60

* "V_1" is the average velocity of flow at the upstream gage.

** "C" is in the equation $Q = CA_2\sqrt{2g(h + V_1^2/2g)}$



NOTE: $A_2 = 4.67\,(16.25 - 0.67) = 72.9$ sq. ft.

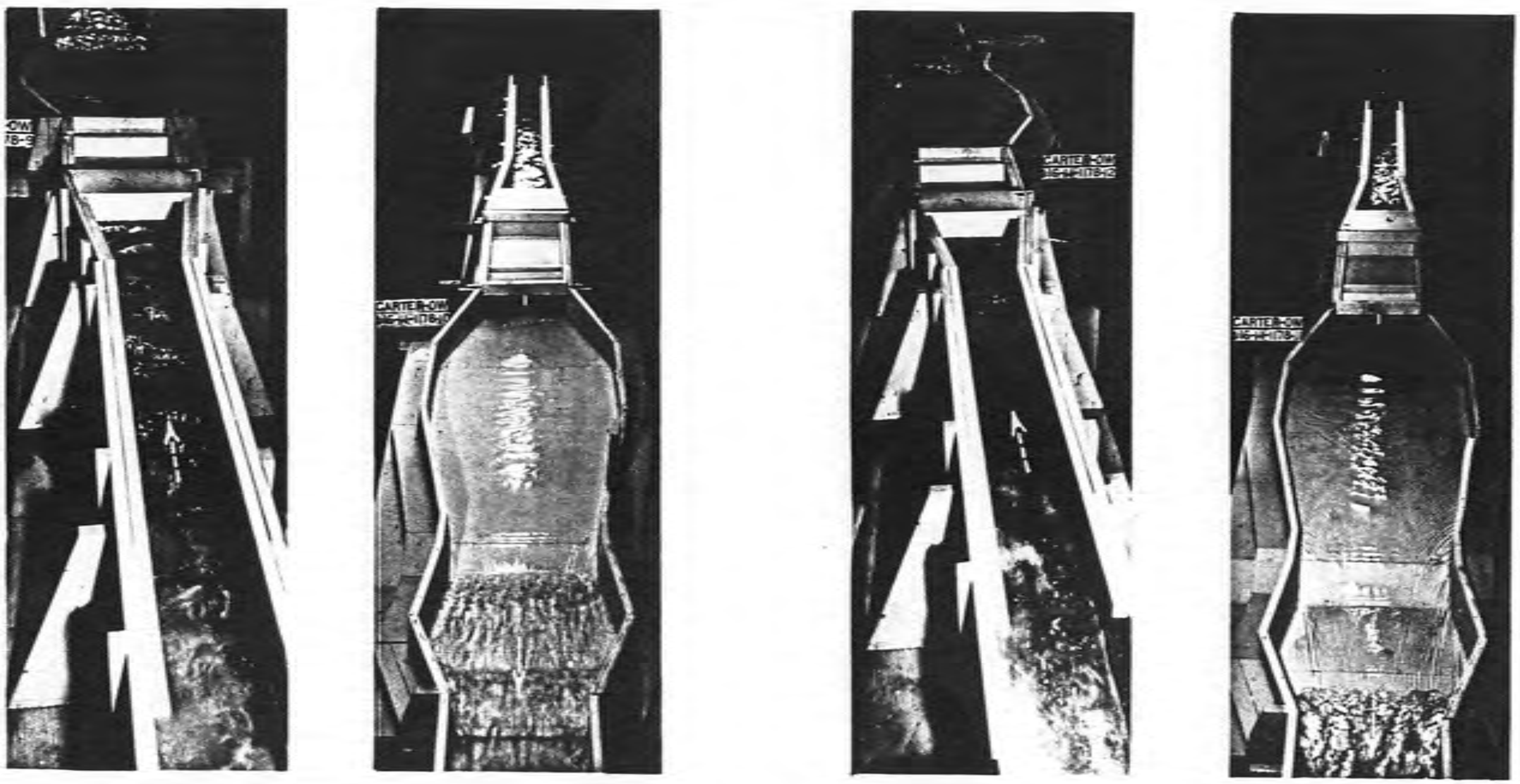
CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL

DISCHARGE CHARACTERISTICS OF THE UNDERPASS

1:16 SCALE MODEL

FIGURE 30
REPORT HYD. 394

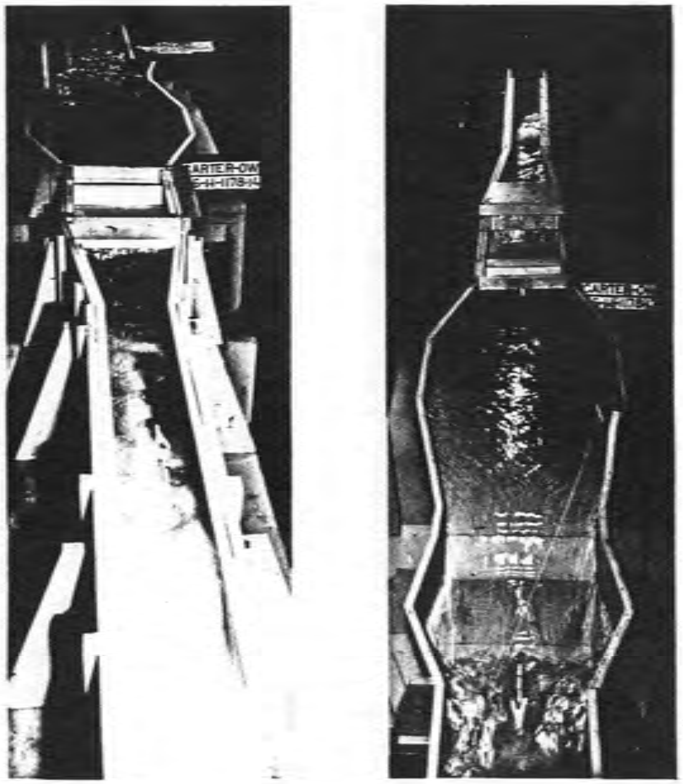
501



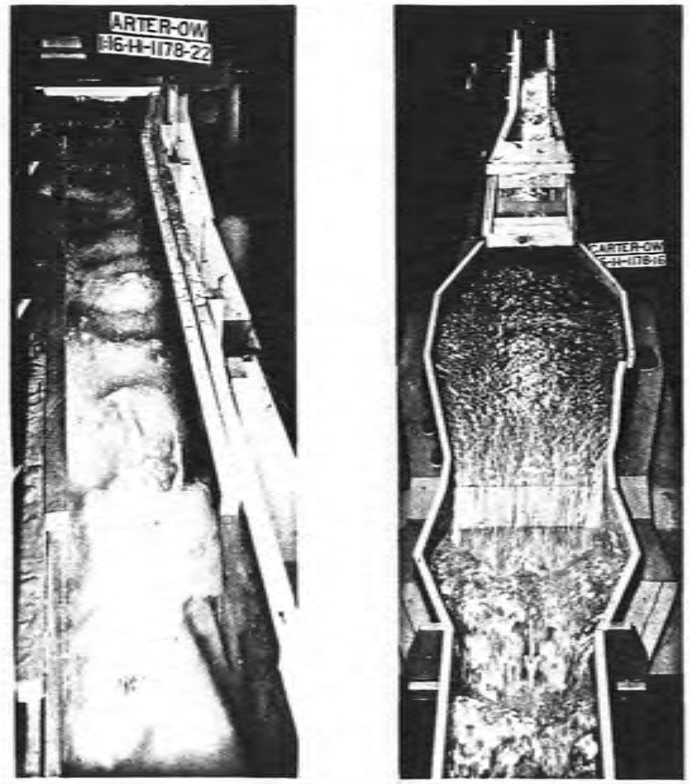
100 second-feet

200 second-feet

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
PERFORMANCE OF RECOMMENDED STRUCTURE - Sheet 1 of 2
1:16 SCALE MODEL

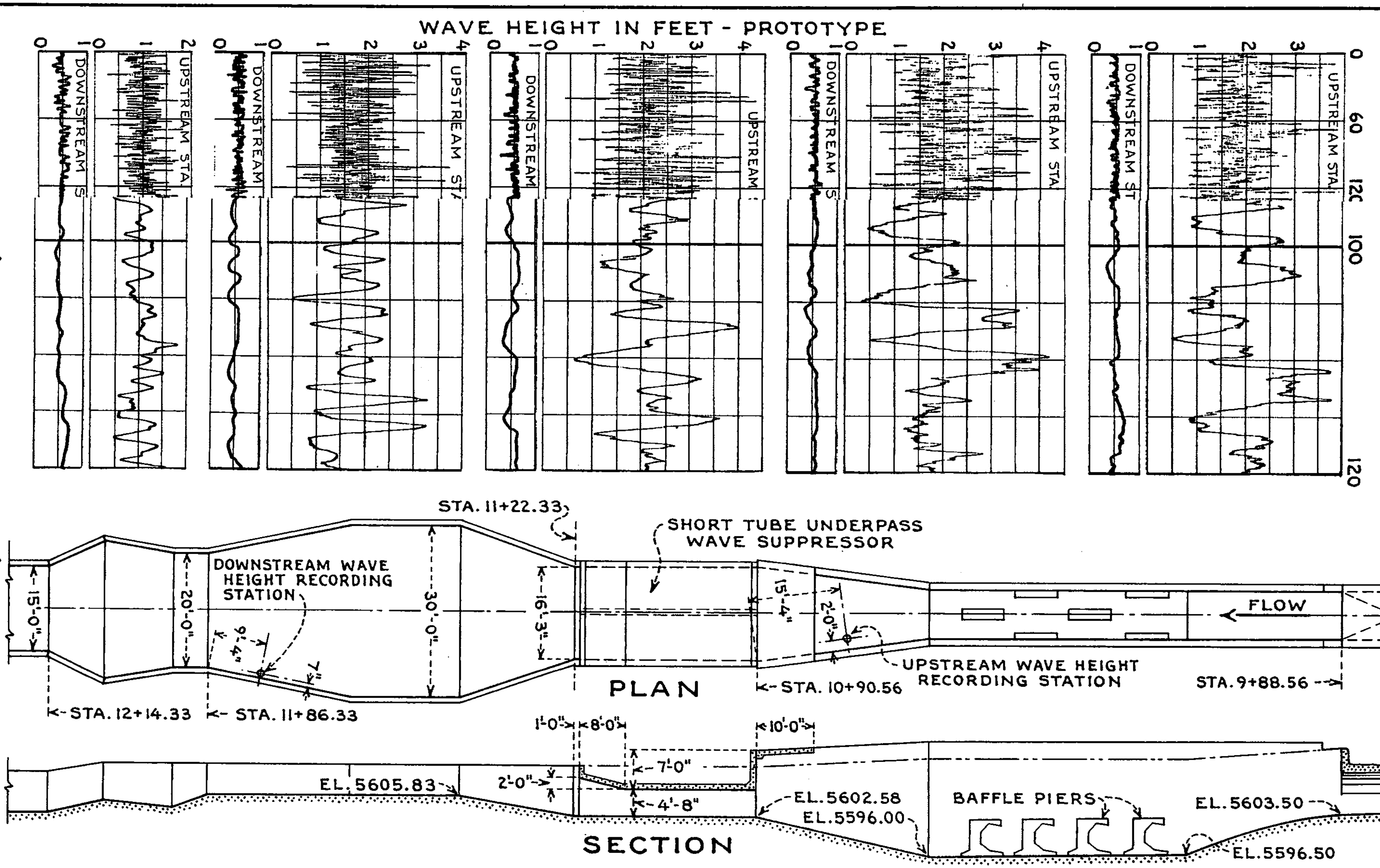


400 second-feet



625 second-feet

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
PERFORMANCE OF RECOMMENDED STRUCTURE - Sheet 2 of 2
1:16 SCALE MODEL



(MODEL SCALE 1:16)

FIGURE 33

Waves occur downstream from Parshall flume without sill

Sill, one-foot square in cross-section, at entrance to rectangular flume reduces wave heights downstream from Parshall flume

CARTER LAKE OUTLET WORKS AND ST. VRAIN CANAL
RECOMMENDED SILL AT ENTRANCE TO RECTANGULAR FLUME
DOWNSTREAM FROM PARSHALL FLUME
1:16 SCALE MODEL