UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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HYDRAULIC LABORATORY REPORT NO. 111

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MODEL STUDIES OF THE COACHELLA CANAL WASH OVERCHUTES ALL-AMERICAN CANAL SYSTEM - CALIFORNIA

By

Fred Locher

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Denver, Colorado

April 20, 1942

FOREWORD

In March 1941, model tests were proposed to determine the adequacy of the combination inverted siphon and wash overchute structures being designed for the Coachella Canal in the All-American Canal system of the Imperial Valley, California. The tests from which a satisfactory design was evolved, were started in April 1941 and were completed the following August, in the hydraulic laboratory of the Bureau of Reclamation, Denver, Colorado.

The wash overchutes discussed in this report were designed under the direction of H. R. McBirney, Senior Engineer. The laboratory investigation was conducted and this report prepared by the writer under the direction of J. E. Warnock, engineer in charge of the hydraulic laboratory. Particular credit is due to H. C. Curtis, Senior Engineer; J. W. Ball, P. W. Terrel, and J. N. Bradley, associate engineers; and J. A. Lindsey, Junior Engineer, who contributed much to the development of the final designs.

All laboratories of the Bureau of Reclamation in Denver, Colorado, are in the Materials, Testing and Control Division, under the supervision of Arthur Ruettgers and R. F. Blanks, senior engineers. All design work is under the supervision of J. L. Savage, Chief Designing Engineer, and all work of the Bureau of Reclamation is directed by S. O. Harper, Chief Engineer. The activities of the Bureau of Reclamation are directed by John C. Page, Commissioner.

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CHAPTER I - INTRODUCTION AND SUMMARY

INTRODUCTION

General. The Coachella Canal, a part of the All-American Canal System, has its headworks at Drop No. 1 on the main canal about 17 miles west of Yuma, Arizona. After crossing East Mesa, the canal passes northwest between the Salton Sea and the Chocolate Mountains to a point near Indio, California, where it turns southwest across the Coachella Valley and then south to the Riverside-Imperial county line, as shown on figure 1. The canal will provide water for irrigation on the East Mesa and in the Coachella Valley.

Due to rough terrain, the canal will intersect numerous washes which empty into the Salton Sea, and which, though ordinarily dry, flow heavily laden with detritus during severe rains. The estimated maximum run-off from the drainage areas adjacent to the canal varies from a few second-feet for the smallest to 20,500 second-feet for the largest. Since only the flows from very small areas can be handled by drainage inlets into the canal, it was necessary to provide some means for conveying the floods from the larger drainage areas across the canal without damaging it. It is planned to use overchutes for this purpose, which are to be of two types: a combination inverted siphon and overchute, and a flume bridging the canal. The latter type, used principally for small isolated washes, was not a part of this investigation, so the term "overchute" in this report will apply only to the 52 combination structures which will be located where large washes intersect the course of the canal, or where several washes can be combined into one by training dikes and diversion channels.

Original design of overchutes. The original or basic designs for the combination overchute structures were of three general types: (1) a narrow structure concrete lined between head walls designed to serve a small wash or combination of small washes, the canal flow passing under the crossing through a concrete trapezoidal siphon (figure 2); (2) a wide structure unlined between head walls designed to serve a large wash or combination of washes, the canal flow passing under the crossing through a concrete rectangular siphon, the top of which was placed approximately one foot below the elevation of the lowest point of the wash between the head walls (figure 9); and (3) a wide structure unlined and consisting of training walls of earth embankment protected by riprap designed to serve large washes or a combination of washes, the canal flowing through a concrete round barrel siphon buried approximately 6 feet beneath the wash crossing protected by adjacent rock-fill and sheet piling (figure 17). The structures of (1) and (2) are protected with rock-fill placed adjacent to the slope paving and siphon barrel, and by a 10-foot row of sheet piling connecting the ends of the wing walls at the outlet.

Economical considerations limited the overchute width to considerably less than the width of the natural wash channel. The maximum discharge and the difference in elevation of the canal and siphon grades were the criteria used for determining the width. For the structures of (1) and (2), the width was made equal to a distance obtained by dividing the maximum anticipated flood by an assumed discharge per lineal foot of curb on the downstream edge of the siphon barrel (section A-A, figure 9), plus a distance necessary to connect the horizontal part of the siphon to the canal transitions. The width of the structure in (3) was established by providing sufficient width between the intersections of the slopes of the earth embankments protecting the siphon head walls with the natural wash bed, to pass an assumed discharge for the particular wash in question, limiting the discharge per foot between the intersections to a reasonable amount.

Need for model tests. When the regimen of a watercourse is disturbed by a constriction such as a wash overchute, the stream will adjust itself to fit the new conditions. The accompanying phenomena governing the behavior of the channel downstream did not lend themselves to a reliable rational analysis, so hydraulic model tests were instigated to study these phenomena and to revise the overchute designs to correct for any unfavorable conditions at the structures resulting from channel adjustment.

Scope of model tests. The designs for the structures included in any one of the three types of wash overchute crossings, as shown on figures 2, 9, and 17, differed, in general, in the distance between head walls and the variation in elevations of the component parts necessary to make each structure fit its particular wash slope. For purposes of the model tests, the types were subdivided into groups where the structures in each group had approximately the same wash slope and similar differences in elevation of the component parts. Since the laws of similitude would take into account structures of different width, it was considered necessary to test only a representative structure from each group.

The first tests were conducted on a 1:24 model arranged so there was approximately 120 feet of the prototype channel upstream and 168 feet downstream from the overchute structure. This length, which was limited by available laboratory space, was not sufficient to permit a thorough study of the channel above and below the structure, but it was sufficient to disclose the faults of the original designs and allowed expedition of the laboratory testing. Tests were made with this model on the original designs of the overchutes with trapezoidal, rectangular, and round barrel siphons, located in washes having slopes ranging from 0.013 to 0.058, and on remedial measures for the weaknesses inherent to the designs.

When more laboratory space became available, another model was built to study the effect of changes in the regimen of the watercourse and consequent degradation, and their bearing on the design. This part of the investigation was conducted on a model to a scale of 1:36 having the equivalent of .936 feet of prototype channel downstream from the structure and 435 feet upstream. Some of the more important tests made on the 1:24 model were repeated on the 1:36 model, and in addition, tests of new designs were made, one of which was selected for the final design.

The final design evolved from the 1:36 model tests was without precedent, so a 1:15 sectional model was built to obtain more reliable data concerning local scour, arrangement of the baffles, their size and shape, and the pressures on them at various flows.

At the time these model investigations were in progress some of the overchutes as originally designed had already been built in the field according to figure 9, and others of the same type were still under construction. Since tests on the 1:24 and 1:36 models showed this original design to be inadequate, it was necessary to develop another design whereby the completed structures could be revised and made safe with a minimum expense. Accordingly, additional studies of the 1;24 scale model were made on the original design, but embodying features of the final design evolved from the 1:36 model.

SUMMARY

Inadequacy of original designs. Tests on the overchutes of the original designs as shown on figures 2, 9, and 17, showed that the wash inlets of the structures constructed with concrete wing walls were satisfactory, whereas the inlets constructed of earth embankment and riprap were inadequate, and that the outlets, constructed in accordance with the original designs, were inadequate at the design discharge of 35 second-feet per lineal foot of barrel curb in combination with the steep wash slopes and the new conditions caused by changing the regimen of the watercourse. In each test, failure of the rock-fill and sheet piling, and the subsequent undermining of the structure, were caused by degradation and channelization, both of which were accelerated by the concentrated flow at the structure.

As testing proceeded, it became evident that the overchute constituted a definite control which divided the wash channel into two separate parts, both of which were effected by the presence of the structure and by the training dikes which diverted the flow upstream and concentrated it at the crossing. As a result of these factors, the rate of change in the slope and the characteristics of both parts of the channel during the periods of flow were much different than those which existed previous to the presence of the structure. The regimen upwash was seen during the model tests to change by retrograding to a stable slope, for a given discharge, beginning at the top of the siphon barrel and extending upstream. Consequently there was little change in the elevation of the washbed immediately upstream from the siphon barrel due to retrogression. Therefore, it was reasonable to assume that the inlet to the overchute which is upstream from the siphon barrel was adequate, provided it was properly proportioned and protected against local scour.

Downstream from the structure, however, the regimen of the wash changed by retrograding to a stable slope beginning at the lower end of the wash and extending upstream to the structure. As a result, the change of elevation of the washbed immediately downstream from the structure was more than at any other point between the overchute and the lower end of the wash in the model, which in the field would actually be the mouth of the wash at the Salton Sea. Accordingly, even though the outlet of the structure was properly proportioned and protected against local scour, the retrogression rendered the designs inadequate by undercutting the rock-fill at the sheet piling. The subsequent failure of the sheet piling allowed the retrogression to work back to the siphon barrel, thereby causing the entire structure to fail.

After the phenomenon of retrogression had been clearly understood from observations of the model and from certain facts concerning it in literature, it was apparent that an adequately designed outlet of an overchute would be one that either prevented local degradation due to retrogression entirely, or one that would not have its stability affected by such changes in the slope of the downstream portion of the wash. The first alternative would obviously be economically unfeasible, so the model tests were continued by studying solutions to the second.

Final designs. Many tests were made before an adequate outlet design could be obtained, these being discussed in chapter II. As evolved from model tests, the final designs were similar to the original designs with the exception of the outlet for each type of overchute.

The narrow type of overchute as first shown on figure 2, was redesigned as shown on figure 8. It can be seen that the original outlet consisting of diverging wing walls protected with rock-fill was replaced by a concrete chute on a 2:1 slope to which concrete baffles or dentates have been attached.

The wider type of overchute was similarly revised as can be noted by comparing figure 9, the original design, with figure 24. The round barrel siphon design shown on figure 17 was abandoned in favor of the rectangular or box siphon of figure 24, because the box-type construction was less expensive and better adapted to the concrete chute and baffle arrangement in the outlet of the crossing.

Since retrogression and the accompanying lowering of the washbed at the outlet cannot be prevented, the safety of the structure is assured by the chute and baffle design. By this arrangement, the gradual increase in energy of the water, caused by degradation increasing the total drop at the outlet, is dissipated on the chute, thus allowing the water to pass into the channel without causing undue local scour. Initially, the depth to which the chute must extend below the top of the siphon box depends upon the wash slope, greater depths being required for the less stable or steeper wash slopes. If, due to successive major floods, the degradation causes the wash to be lowered such that the stability of the chute will be impaired by subsequent floods, the chute may be extended to some greater depth. However, the maximum design flood is of rare occurrence and it probably will be many years before an extension to the chute will be necessary.

For those structures of the original designs already completed, or under construction in the field, a revised design was evolved by testing the design of figure 9, but including an adaptation of the chute and baffle arrangement. Results of this study produced a design similar to figure 24, but with the outlet of different shape as shown on figure 30. That this revision would be required is shown by figure 27, which illustrates the scour developed in the field at wash overchutes constructed according to the original design and where the discharge was approximately 25 percent of the design capacity.

CHAPTER II - MODEL TESTS

LABORATORY PROCEDURE

Model construction and test method. As shown on figures 4 to 6, and others, the models used in the overchute studies were placed in large metal-lined boxes. At the upstream end, a general baffle was added to quiet the water entering the model from a pipe line, which was connected to a venturi meter for measuring the discharge furnished by a centrifugal pump. The structural parts of the overchute were the siphon and the vertical headwalls and wing walls which were subjoined by slope paving. The vertical walls and siphon were constructed of redwood, while the slope paving was made of sand-cement mortar, or as in the case of the round barrel siphon, the siphon proper was constructed of sheet metal and the earth embankment consisted of sand covered with gravel as riprap. The wash itself consisted of a deep sand bed of sufficient length and on the proper slope to permit observation of the channel movement as required. When a sectional model was needed, it was installed in a glass-sided flume to permit observation of the flow as shown on figure 25.

In general, each model was tested at several flows varying as percents of the designed discharges expressed in terms of second-feet per foot of length of siphon curb. The design discharges of the overchutes were based on an average intensity of precipitation over the broad area through which the canal is located. Local intensities in several small areas, however, will vary widely from the average and, since the narrow overchutes will serve small drainage areas, it was deemed advisable to test such models at flows double the design capacity. The wash slopes were also varied in the models to cover the range of slopes provided for in the design, this changing the elevation between the component parts of the overchute. For any set of conditions, observations and notes were made of the scour and flow, with photographs being taken to complete the record; quantitative measurements concerning the state of the wash were included only as a basis for comparing the results of the model tests.

At the time the laboratory tests were begun, it was not anticipated that the unforeseen conditions caused by changes in the regimen of the watercourse and the unstable state of the channel would be the major factors controlling the design. Consequently, the designs were based on the assumptions that the channel would be fairly stable and that changes in the regimen of the watercourse would cause only local scour and undesirable flow conditions at the structure. Because of these assumptions, the first models were constructed chiefly for studying flow and scour at the structure only. In these models, the tailboard was but a short distance from the outlet of the structure. This constituted a control which stabilized the channel but did not affect the local scour at the structure, so if the prototype channel were stable, this condition was satisfied in the model. Later on, the length of the model channel was increased to permit the washbed to adjust itself as governed by the operating conditions.

NARROW OVERCHUTES - TRAPEZOIDAL SIPHONS

Initial tests. The tests of the narrow overchutes with trapezoidal siphons as shown on figure 2, were made on a 1:24 scale model. A representative width of 34 feet was selected and tested for several wash slopes at discharges up to 200 percent of the designed capacity, the elevation between component parts of the structure being changed for each slope. The first test (test S30A2) was made to study the adequacy of the original design of the narrow overchute, from which it was found that a tendency toward excessive scour developed in a short time sufficient to indicate failure by undermining, as shown on figure 3A. In this test, the wash slope was 0.031 and the drop from the top of the siphon to the sheet piling was 1.37 feet (prototype), a sketch of the general arrangement for this and other initial tests being shown on section A-A, figure 4.

Another test of the original design was then made with a wash slope of 0.047 but with no drop between the top of the siphon and the sheet piling (test S32A2). The scour was again excessive both upstream and downstream from the sheet piling as indicated by figure 3B. When this test was repeated using a three-foot blanket of rock-fill between the wing walls of the outlet upstream from the sheet piling (test S32A0), the scour was reduced somewhat between the wing walls, but there was still deep erosion downstream from the sheet piling.

Two additional tests (tests S35A0 and S50A0), using the rock-fill blanket, but with a wash slope of 0.042 and a drop of 4.61 feet, and with a wash slope of 0.034 and a drop of 0.99 feet (figures 5 and 6), indicated that the scour increased considerably as the drop increased. As a result of these initial tests, it was evident that the original design of the narrow overchutes would be inadequate, since the scour at the structure would eventually cause the sheet piling and the structure itself to be undermined and eventually destroyed, regardless of the total drop or whether rock-fill was added to reduce the erosion.

Double pool. To eliminate the excessive scour obtained in the initial tests, it was evident that the velocity of the water in the outlet of the overchute should be reduced and the flow made to spread laterally between the wing walls. Previous experiments of the Fortuna wasteway on the Gila main canal indicated that, when a flow was suddenly introduced into the waste channel containing little or no water, the velocity of the waste water could be reduced by placing a weir across the channel within the wing walls just downstream from the wasteway. This weir formed a wide pool of water deeper than the flow in the channel immediately downstream and thus provided a stilling pool in which the higher velocity of the water from the wasteway was reduced more nearly to the normal velocity of the water in the waste channel. As a result the scour in the vicinity of the structure was nearly eliminated.

Applying this principle to the outlets of the narrow overchutes, the unlined portion of the outlet within the wing walls was paved and two weirs were added to form a double pool as shown on section B-B, figure 4. Two pools were considered necessary because of the short length available for reducing the velocity in this type of design. For the first test of this arrangement (test S35C0), the two pools were each made 14 feet long with a weir 4 feet high separating the pools, and a weir 2.50 feet high at the end of the second pool; the wash slope was set at 0.042 and the total drop from the siphon to the downstream weir was 4.11 feet. The upstream pool proved to be too short, and the flow swept out for the higher discharges. This was corrected by increasing the length of the first pool to 18 feet and by adding a sill 10 inches high at the top of the slope entering the stilling pools (test S35E0). Even with this arrangement there was some scour of the rock-fill below the downstream weir for the design discharge, and considerably more for a flow of 200 percent of the design discharge, yet the scour was much less than that observed during the initial tests of the original design.

Since the double-pool arrangement in the outlet was promising, it was studied next for the total drop reduced to 3.50 feet, but at the same wash slope of 0.042 (test CS1A0). For this combination, a general arrangement of which is shown on section C-C, figure 4, the pools were adequate for discharges up to and including 200 percent of the design capacity, the scour being negligible. Further improvement in the flow conditions in the pools could be obtained, however, by reducing excessive boiling and high surface velocities caused by the weirs. This was accomplished by placing 2-foot openings on 10-foot centers in the upstream weir (test CS2A0), as shown on figure 7A; the flow conditions and scour are shown on figures 7B and 7C.

Additional tests of this same arrangement were made to ascertain its adequacy for other conditions besides the slope of the wash and the discharge. For example, when the tailwater was raised two feet, there was no movement at all of the rock-fill in the model and the scour to the channel downstream was negligible (test CS3A0); when the gravel of the rock-fill was decreased from 3/8 inch to No. 4, there was some movement of the gravel, but nothing serious (test CS4A0); when the stilling pools themselves were filled with No. 4 gravel for determining if the flow would remove any debris that might collect in the pools, the gravel was quickly scoured out by the first flood (test CS4C0); but when two vertical training walls were placed in the outlet so that the flow in the pools was confined to the width between headwalls (34 feet), the erosion to the rock-fill was considerably increased (test CS5A0).

Although the double pool worked satisfactorily for a width between headwalls of 34 feet, it was desirable to observe the performance of the double pool when used for narrow overchutes of even less width, since the transverse distance at the end of the apron entering the pools would be much smaller. As shown on section D-D, figure 4, for test S34A0, the double pool evolved from tests with an overchute 34 feet wide between headwalls (section C-C, figure 4), was adapted to an overchute width of 25 feet. It was immediately seen that the upstream pool was inadequate when built to the same proportions as the upstream pool for the overchute 34 feet wide between headwalls. By increasing the slope of the paving entering the upstream pool, the length of this pool along the bottom was increased from 3.76 feet to 8.90 feet (test S34C0, section D-D, figure 4). This arrangement worked satisfactorily for flows up to and including the maximum design discharge.

Bucket design. As a final test on the narrow overchutes, a bucket-type stilling pool was studied in place of the double stilling pool. The use of a bucket as a stilling pool, in which a roller is formed for dissipating the energy of the flow, has proved advantageous wherever the use of a hydraulic jump would require a long apron because of a deficiency in tailwater at high discharges and an excess at low discharges. Although a hydraulic jump cannot be obtained at the overchute, even in the double-pool arrangement which depends on the weirs to build up some tailwater in the pool itself, the bucket-type pool was considered as another method of obtaining energy dissipation with adverse tailwater conditions. The bucket stilling pool studied in this case is shown on section E-E, figure 4, test CS6A0. This design was unsatisfactory, however, because of the small drop and a lack of tailwater, it was impossible for a roller to form in the bucket. Instead, the flow left the bucket as a jet traveling at a relatively high velocity which produced excessive scour of the rock-fill. To obtain more drop and sufficient tailwater by use of weirs would have been unfeasible and would not have been any improvement over the double pool, so further study of a bucket was abandoned.

Final design. At this point of the investigation, the double-pool arrangement of section C-C, figure 4, and as shown on figure 7, was considered satisfactory for overchutes having a width of approximately 34 feet between headwalls; for narrower overchutes in this group, having a width of approximately 25 feet between headwalls, the design of section D-D, test S34C0 on figure 4, was considered adequate.

After more extensive model tests had been made on the wider overchutes with rectangular-barrel siphons, it was found that regardless of the adequacy of their outlet to prevent local scour, retrogression of the downstream washbed would render several of the outlet designs inadequate. As a result, it was evident that the double-pool outlet would also be rendered ineffective by retrogression. The tests leading to this conclusion and the results obtained to overcome the effects of retrogression, are presented in the discussion immediately following, from which it will be shown that the final design of the narrow overchutes with trapezoidal siphons was made similar to the arrangement evolved for the wide overchutes. No separate tests were required of this design for the narrow overchutes, since the width was the only factor changed and this was found to have no effect on the adequacy of the design for the wide overchutes. Accordingly, the final design of the narrow overchutes with trapezoidal siphons has been evolved as shown on figure 8.

WIDE OVERCHUTES - RECTANGULAR SIPHONS

Initial tests. Unlike the narrow overchutes, the wide overchutes shown on figure 9 were designed to serve large drainage areas where the average run-off from the catchment area will not exceed the assumed designed capacity of 35 second-feet per lineal foot of siphon curb. It was, therefore, unnecessary to test these structures for flows exceeding the design capacity, but this was done in a few tests as a precaution.

The initial tests of the original design for the wide overchutes were also made on a 1:24 scale model following the procedure used in tests of the narrow overchutes, namely, to determine the adequacy of the original design and to try minor revisions before attempting studies of major changes to the structure itself. A width of 120 feet (prototype) between head walls was used in these tests, which was the limit that could be conveniently handled in the model. Since the discharge per foot is the same regardless of the width, the results obtained from this width would be applicable to wider or narrower overchutes. As before, the discharge was varied for tests covering different wash slopes of from 0.013 to 0.058.

As previously found for tests on the narrow overchutes with trapezoidal siphons, the original design of the wide overchute was inadequate. This was true regardless of whether a gravel blanket was placed between the wing walls upstream from the sheet piling, or whether the sheet piling was removed or lengthened, or whether the siphon curb was in place or not. On each test the rate of scour indicated that the structures would eventually fail and that the flow in the outlet must be spread laterally and be reduced in velocity. Figure 10 shows the scour developed by the original design, while figure 11 indicates that the use of a gravel blanket is inadequate. Sketches of the general arrangement for these initial tests are given on figure 12, sections A-A, B-B, and C-C, where the difference in each arrangement was the extent of the gravel blanket. An analysis of the performance in any of these tests showed that at a discharge of 25 percent of the design capacity the rock-fill immediately below the siphon and sheet piling began to move downstream, and as the discharge increased, the flow in the outlet plunged under the shallow tailwater at the siphon and at the sheet piling causing deep scour at both places. As the flow continued, the newly formed channel downstream aggravated the conditions which already existed, so that when the flow reached 100 percent of the design capacity, it had scoured a deep pool at the siphon, and the sheet piling was beginning to fail, as shown on figures 10C and 11B.

Since each test indicated the same failure over a wide range of wash slopes and corresponding drops, two facts seemed to be evident: (1) that primary failure in each case was caused by the same natural phenomenon, which was not clearly understood at this point of the investigations; and (2) since the failure of the design was so similar over a wide range of tests, then if the effect of this phenomenon were nullified, a design adequate for one wash channel would be suitable for all the wash channels involved.

Double and single pools. Since the double-pool arrangement for the narrow overchutes was still considered adequate up to this point, the retrogression effect not being fully understood at this time, it was logical to apply this design to the wide overchutes to spread the flow in the wing walls and reduce the velocity of flow in that area. Adapting this design to the wider structures was not particularly convenient, but the acceptance of this major revision would be dependent on its ability to prevent the serious erosion involved with the original design.

The first double pool studied is shown on section D-D, figure 12, for test CS46-1A0. The drop in this design was 8.00 feet, the two pools being separated by a 4-foot weir with a 2.5-foot weir at the end of the second pool. This arrangement was not correctly proportioned since the first pool was too short, which caused insufficient energy dissipation and excessive erosion downstream from the second pool, as shown by figure 14B. Removing the sloping apron into the pool and the weir at the end of the second pool, as shown by section E-E, figure 13, was even less satisfactory. Improvement of the flow conditions and some lessening of the scour was obtained by adding to this last arrangement a horizontal apron 10 feet long, as shown on section F-F, figure 13. In this design a form of hydraulic jump was obtained, in spite of unfavorable tailwater conditions. When the upstream weir of this design was removed (test CS46-4A0, figure 13), the performance was still improved although there was some scouring as shown on figure 15C. Thus it was found that a single pool was more satisfactory for the wider overchutes.

RETROGRESSION OF WASH CHANNEL

Effect on outlet. It became quite evident at the end of this series of tests that some additional factor was causing the scour at the outlets even though a suitable stilling pool could be evolved. It was noticed that the scour below each outlet increased if the tailboard at the end of the model wash was lower than the apron of the stilling pool, indicating that this acted as a control which influenced the slope of the washbed downstream from the outlet. A more careful analysis of this fact disclosed that when scour occurred, it was due to the unstable slope of the channel. From the formula developed by Lacey[1], it was possible to compute the approximate value of a stable slope in this material for the

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[1]These and all following notes in this report refer to numbers listed in the bibliography in their order of appearance.

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maximum discharge and continuous flow. By comparing the computed slope according to Lacey with the slopes at the wash crossings in the field, it was seen that the computed value was much flatter and that the retrogression during periods of flow would be the major factor controlling the design. In the model, the flow caused the channel to retrograde to a stable slope for a given discharge, the depth of scour below the apron being dependent on the elevation of the tailboard, the lower the control the greater the depth of scour.

Consequently the adequacy of the outlet developed for a particular structure was wholly dependent upon a downstream control. In the field such a control might be a rock barrier or other natural impediment whose character and permanent elevation would be known. When such a control exists, the channel will retrograde to a permanent slope as shown on figure 16A, and there will be no further retrogression during future floods between the structure and the control. If there is no fixed physical control between the structure and the mouth of the wash at the Salton Sea, the elevation of the bed downwash will gradually lower due to successive floods, until the elevation of the bed of the wash approaches the elevation at its mouth, as indicated on figure 16B, the new slope of the bed being asymptotic to the original slope and dependent on the hydraulic properties of the stream itself. If this is to be the condition in the prototype channel, and there is every reason to believe it will be, then the stilling pool designs evolved in the preceding tests with the proviso of a stable channel would not be adequate for this new condition. The inlets, on the other hand, would be satisfactory provided they were properly proportioned and protected against local scour, since retrogression upstream from the structure would start at the control produced by the siphon. As a result, the elevation of the wash channel immediately upstream would change very little.

Since it would be entirely impractical to prevent retrogression, the model tests were continued to evolve an outlet which would prevent the progress of the retrogression from affecting the stability of the overchutes.

Verification tests. To verify this analysis of the effect of retrogression on the scour at the outlets, a 1:36 scale model was used in which the wash was extended downstream from the structure an equivalent of 936 feet in the prototype, and upstream an equivalent of 435 feet. In the previous tests on the 1:24 scale model, the downstream distance was 168 feet, while the upstream distance was 120 feet.

The first test with the longer wash channel was made on the original design of a wash crossing in which a round-barrel siphon was buried beneath the crossing, as shown on figure 17. This round-barrel design was selected for this test because it had not been studied previously, and because it could be compared with a test on the 1:24 scale model, in which a rectangular siphon had been buried beneath the crossing (test CSOAO, section G-G, figure 13).

Figure 18A shows the model of a wash crossing detailed on figure 17, with a distance between embankments of approximately 120 feet and a wash slope of 0.028. The training walls were earth embankments (sand in the model) protected by riprap, the round-barrel siphon being buried beneath the crossing. When the flow was introduced in the model, it cut a channel which gradually enlarged but never approached the normal width of the downwash channel (figure 18B). Since there was still a control at the end of the model wash, although considerably farther downstream than heretofore, retrogression was simulated by lowering this control as the test progressed. If retrogression was not a factor, then this slight and gradual lowering of the control would not influence the scour; however, the scour was increased considerably and continued to do so as the retrogression developed (figure 19).

It was evident that for each vertical position of the control, retrogression eventually produced a stable wash slope which began at the control and sloped upstream to the siphon barrel, the new wash bed at the siphon being well below the top of the barrel. For each successive lowering of the control, retrogression continued to lower the wash bed until a new condition of equilibrium occurred for the given discharge and vertical position of the control. In effect, this simulated moving the control farther and farther downstream until finally it would have reached a position corresponding to the elevation of the wash mouth at the Salton Sea.

This test was the only one made on the round barrel siphon, it being an alternate design and one considered to be more expensive than the rectangular siphon, so it was given no further consideration. In addition, wing walls constructed of earth embankment protected by riprap were considered inadequate as a result of the erosion shown on figure 19B.

To complete the study of retrogression, a model was installed with a rectangular siphon and an outlet arranged as shown on figure 20 and by section A-A, figure 21 for test CS44-1AO. This arrangement, being quite similar to the outlet of test CS46-4AO on figures 13 and 15, made with the short wash channel and tailboard control, also permitted a comparison of the effect of retrogression. By comparing figure 15 with figure 20, it is evident that degradation due to retrogression has greatly increased the scour in the outlet.

CHUTE AND BAFFLE DESIGN

<u>Purpose</u>. Since it was now definitely established that all of the previous outlet designs heretofore considered adequate would be eventually rendered useless by degradation due to retrogression, it was necessary to consider new methods of preventing this scour from affecting the

stability of the overchutes. It was evident that the continual lowering of the downwash channel due to successive major floods could not be prevented, so it was logical to consider some type of outlet which would minimize local scour and which would, at the same time, provide protection as the wash channel lowered. Accordingly, tests were made on a sloping apron or chute which could be extended downward as retrogression progressed. Attached to the chute were numerous baffles or dentates which would dissipate the energy of the water and reduce its velocity to minimize local scour. As the wash channel lowered and exposed more of the chute, the baffles would continue to dissipate the increased energy developed by the increase in total drop.

Development of final design. The first test of the chute and baffle arrangement was made using a serrated sloping apron to form a series of cascades, as indicated by section B-B, figure 21. In this series of tests the sloping apron was completely covered by backfill (figure 22A). As the flow continued, however, the degradation would rapidly uncover the apron, so the duration of the flood was made sufficiently long to give some indication of the length of apron required for a particular wash slope. It was found in this first test that the cascades were too small to provide effective energy dissipation, so the slope of the apron was changed from 1-1/2:1 to 3:1 and larger notches or steps were used as shown by section C-C, figure 21. This change proved to be quite adequate, but for the flat slope of 3:1 the chute would be excessively long for the total drops anticipated. Returning to the 1-1/2:1 slope, a type of bucket was placed at the bottom of the chute as shown by section D-D, figure 21, but this was unsatisfactory because it would be impossible to establish a fixed elevation of the bucket due to a continual lowering of the channel by retrogression. This again demonstrated the need of a design sufficiently flexible to be extended as degradation increased over a period of time.

Final design. Another test was then made using a 2:1 sloping apron or chute with baffles spaced as shown by section E-E, figure 21. This arrangement proved to be entirely adequate as indicated by figures 22 and 23, and was selected as the final design. The continued lowering of the wash channel is evident, but the stability of the structure is not affected (figure 23B). The resulting scour shown by this latter figure was developed in 5-1/2 hours in the model, corresponding to 33 hours in the prototype, for the design capacity of 35 second-feet per foot of siphon curb. Such a severe continuous flood is not expected in the field, so it is apparent that the drop from the top of the siphon to the end of chute, 24 feet in this test, would be adequate for several years, particularly since the model erosion was exaggerated because tests were made with clear water, no bed load being added. This drop would be different for various wash slopes and was computed by determining the rate of bed-load movement for each wash slope in a manner similar to that which is explained in the following chapter.

To further study the size, shape, and spacing of the baffles, a 1:15 scale sectional model was constructed as shown on figure 25 in which the effectiveness of the baffles is evident. It was found that the factors developed on the 1:36 model were substantially correct. Piezometers were installed to determine the pressures on several baffles as an aid in their design. The average pressure difference between the upstream and downstream faces was 7.5 feet of water per square foot, prototype.

Figure 24 shows the prototype details of the final design for the wide overchutes with rectangular barrel siphons. As mentioned in section 11, the narrow overchutes with trapezoidal siphons were also finally designed using the chute and baffle arrangement, as shown by figure 8. It will be recalled from the previous discussion that the double-pool arrangement, as shown on figure 7, was considered adequate for the narrow overchutes until it was realized that retrogression would eventually render this design useless by lowering the wash channel below the elevation of the pools.

Comparison of designs. That the final design in figure 24 was superior to those in which the original design was altered by minor revisions to the outlet, was readily seen from test CS45-1A0, section F-F, figure 21. In this test the original design of figure 9 was changed by increasing the vertical length of the sheet piling from 10 to 30 feet; the original 2- by 10-foot blanket of riprap adjacent to the sheet piling was made 3 feet thick and placed on a 3:1 slope extending downstream to a depth of 20 feet below the top of the piling, the wash slope being set at 0.04. In terms of the prototype, the riprap had a maximum diameter of 27 inches.

Although the purpose of this test was originally an attempt to find a design more economical than the final design, it was quickly shown that this or any similar designs would be a failure, as may be seen from figure 26. The superiority of the final design may be fully appreciated by comparing the conditions at the end of 5-1/2 hours (figure 23B), with the conditions in the revised original design at the end of only 15 minutes (figure 26C).

REVISIONS TO COMPLETED OVERCHUTES

Recommended revisions. Since some overchutes of the original design as shown in figure 9 were already constructed or under construction at the time these model investigations were made, studies were necessary to devise methods of alteration to prevent failure in a manner similar to that in figure 10C. That this may well occur in the field can be seen by a comparison of the scour in the model (figure 10C) and the prototype (figure 27).

To perform these tests, a 1:24 scale model was again constructed of the original design, studies being made to adapt the chute and baffle arrangement. It will be recalled that the original design had diverging wing walls in the outlet consisting of vertical retaining walls and concrete paving on a 2:1 slope with rock-fill in the area between the walls and adjacent to the siphon, as shown on figure 9, while the final design had parallel training walls on each side of the concrete chute as shown on figure 24. The chute and baffle arrangement was installed in the unpaved area in the outlet with additional baffles on the slope paving of the existing structure as shown on figure 28. The first test showed this to be satisfactory as shown on figures 28B and 28C, the light colored portion of the model representing the additions to the existing structures. However, it was deemed structurally impractical to place baffles on the old paving and when they were removed, the flow along each side of the chute was not sufficiently baffled, so objectionable scour developed in the channel. To alleviate this scour condition, low training walls were placed at the end of the first three rows of baffles and additional baffles were added to the new lower portion of the paved training walls, as shown at the right of figure 29C. The other side of the model in figure 29C was made to conform with the final design in figure 24, which is recommended for use in future construction. By arranging the model in this manner a comparison of the scour was readily made between the two designs from which it was found that the scour was less for the final design as arranged at the left of figure 29 C. The improvement, however, was not sufficient to warrant the additional cost of changing the existing structures to conform with this design, so it was recommended that the existing overchutes be revised according to figure 30, which shows the details of the arrangement in the model at the right of figure 29C.

CHAPTER III - RESULTS AND CONCLUSIONS

ANALYSIS OF RESULTS

Narrow overchutes. The scour occurring at the outlets in the initial tests of the original design of the narrow overchutes was more severe upstream than downstream from the sheet piling. This fact can be attributed to two conditions: (1) the ratio between the length of the sheet-piling cross-weir and the distance between head walls at the siphons; and (2) the tailboard in the model acting as a control. The flow, in passing from the siphon to the cross-weir, spread laterally over a length approximately three times greater than that at the siphon. In the process of spreading, much of the erosive power was dissipated in the area enclosed by the wing walls and the cross-weir, since the flow spread considerably and moved at a lower velocity. As a result, the scour downstream from the sheet piling was usually considerably less than the scour in the area upstream.

The double pool was developed to prevent scour in the area upstream from the sheet piling and to spread the flow more uniformly across the wash channel downstream from the outlet. The double-pool arrangement of figure 7 appeared satisfactory because of condition (2), whereby the tailwater elevation at the double pool was fairly stable for a given discharge; that is, the tailboard prevented the wash channel and the tailwater from lowering. Accordingly, when it was realized that this tailboard acted as a control and prevented retrogression of the wash channel, the double-pool design was discarded and the final design was selected as shown on figure 8, being made similar to the design evolved from the wide overchute studies which took the effect of retrogression into account.

Wide overchutes. Tests of the original design of the wide overchutes with rectangular siphons showed that there was considerable scour both upstream and downstream from the sheet piling because the discharge per foot at the sheet piling was not much less than that at the siphon. This smaller difference in the discharge per foot occurred at the wider overchutes, since the transverse length of the sheet-piling cross-weir at the end of the flared wing walls was not considerably greater than the width between head walls at the siphon, and because the flow did not spread uniformly across the outlet at the end of the wing walls. As a result, even though there was considerable energy remaining in the flow at the sheet piling to cause excessive erosion immediately downstream.

The single stilling pool in which a form of the hydraulic jump was developed, as shown on figure 15, was satisfactory in the model only when the tailboard was at or above the elevation of the pool. Here again the tailboard acted as a control and retarded normal retrogression in the channel downstream, so that this design was apparently adequate. When it was realized that retrogression would affect the adequacy of this or a similar design, it was necessary to evolve a chute and baffle arrangement which would not permit failure of the overchute as retrogression progressed, the final design for wide overchutes with rectangular siphons being shown on figure 24. The design with round-barrel siphons was abandoned as being inferior from practical considerations, so was not revised to conform with the chute and baffle arrangement. For those structures already constructed in accordance with the original design, which was found to be unsatisfactory as shown on figure 10, the chute and baffle arrangement was adapted to revise them as shown on figure 30.

Retrogression. When a compilation of the test results was made after the single pool had been developed for the wide overchutes, it was apparent that degradation due to retrogression in the wash channel downstream would eventually cause this and similar designs to fail. An analysis of this phenomenon explained the process of degradation and the manner in which it affects the design of the structures.

The washbeds intercepted by the Coachella Canal contain dune sand or other sandy material lying on slopes varying from 0.013 to 0.0716, slopes which are much steeper than those considered to be stable for this material when in a stream channel. During heavy rains, large quantities of this material will be carried away and the washbed will be scoured to an elevation lower than that which existed previous to the storm. If the regimen of the channel bed is not controlled in any way, retrogression will begin at the elevation of the mouth of the wash at the Salton Sea and work towards the source, the greatest change in elevation of the washbed occurring somewhere in the upper reaches of the channel. When an overchute is built in the wash and when the run-off of other washes is diverted by training dikes and diversion channels through the relatively narrow opening of the overchute, the natural regimen of the channel is disturbed. The structure acts as a control in the channel. The part upstream from the structure will change its regimen by retrograding from the control towards the source. The elevation of the downstream end of this part of the channel will be the top of the control, which in this case is the siphon box; hence the degradation due to retrogression immediately upstream from the structure will be slight and will have very little, if any, effect on its stability. Downstream from the structure, the regimen will change by retrograding from a natural control towards the overchute structure. A longitudinal profile of the downstream channel would show the greatest change in elevation of the washbed with respect to a previous grade to occur immediately downstream from the overchute as shown on figure 16B, this having a direct bearing on the stability of the overchute.

The depth of this scour during a flood is a matter of conjecture because the range of wash slopes for which the structures were designed is outside of the limits of the empirical equations and the experiments pertaining to bed-load. The work of Lacey[1] was based on data obtained from rivers and canals flowing in alluvium and with relatively flat slopes. The experiments of Gilbert[2] on which the revised Schoklitsch Bed-Load Formula[3] is based, were conducted with slopes of 0.02 or less, whereas the slopes of the Coachella Washes vary from 0.013 to 0.072. In order to obtain an approximation of the rate of scour in the field, it was necessary, with the aid of this formula, to extrapolate the curve to find values for the quantity of bed-load that would be transported during a flood. In addition, it was impossible to estimate the bed-load which will be transported across the structure or the additional load acquired downstream from the structure. However, it is known that the rate of scour based on the assumption of clear water at the structure would be greater than that which will actually occur in the field. This is evident because the flow passing the structure is carrying bed-load and the additional amount that it will pick up after it leaves the structure is less than that which can be picked up by a similar flow which is without bed-load at the overchute. Accordingly, if the design is based on assumptions for clear water, it will be reasonable to assume that it will be safe for a flood lasting as long as and probably longer than the computed time necessary for the channel to retrograde below the structure.

For the chute and baffle arrangement, the depth to which the chute should extend below the top of the siphon depends upon the wash slope, because the rate of degradation, all other factors being equal, is a function of the wash slope. For example, assume clear water flowing through an overchute situated three miles from the mouth of a wash channel, the channel width being taken as the distance between head walls of the overchute. Assuming wash slopes of 0.013, 0.025, and 0.058, then from the curve on figure 31, the bed-load being transported, computed from the revised Schoklitsch formula, for each slope would be 23, 64, and 230 pounds per second per foot of channel width, respectively, for a given gradation of bed material at the maximum design discharge of 35 second-feet per foot of curb. Then if the longitudinal scour pattern is assumed to be triangular over the three miles of channel, the respective channels in the area adjacent to the structure will retrograde at the rate of approximately 0.104, 0.290, and 1.026 feet per hour for each hour of flow at maximum design discharge. If allowance is made for six feet of local scour, then a chute and baffle arrangement, the toe of which is thirty feet below the top of the siphon, would be safe for 231 hours of continuous flow at maximum design capacity in a wash having a slope of 0.013, or 83 hours in a wash having a slope of 0.025, or 23 hours in a wash having a slope of 0.058, etc. If at any time after successive floods it is found that the scour is approaching the bottom of the original chute, it may be extended to some greater depth without impairing its efficiency, since the baffles will dissipate the increase in energy developed by the increase in drop. It will probably be many years before such an extension to the chutes will be necessary, since ample length has been provided initially and because the washes are intermittent streams which will carry the maximum design discharge on only rare occasions.

CONCLUSIONS

Summary. As a result of the model investigations of overchutes applicable to the wash crossings of the Coachella Canal, the following conclusions have been drawn:

(a) The overchute inlets were satisfactory as originally designed according to figures 2 and 9.

(b) The outlets of the overchutes as originally designed were inadequate for flows of 35 second-feet per lineal foot of barrel curb in washes where retrogression is a predominating factor, as shown by figures 3, 10, 19, 20, and 27.

(c) Failure of the sheet piling and the subsequent undermining of the structures as originally designed was the result of degradation due to retrogression of the channel downstream, as shown by figures 19 and 26, the latter being a slight variation from the original design.

(d) Riprap and rock-fill at the sheet piling and upstream between the wing walls offer little protection against degradation (figures 10 and 26).

(e) Increasing the vertical length of the sheet piling and the size and extent of the rock-fill did not prevent failure of the structures, as shown by figure 26.

(f) A stilling pool in the outlet at the siphon is adequate protection when local scour only is involved, but not when retrogression is present (figures 15 and 20).

(g) A bucket-type stilling pool is not applicable to the overchutes because of the small drop available and because the tailwater was inadequate to form a roller. Retrogression would eventually drop the channel below the lip of the bucket.

(h) The cascade arrangement as shown on figure 21, section C-C, was satisfactory in the model; however, the results obtained with some dams designed with this feature showed that at small discharges the depth of scour was reduced but at greater discharges silt carried over the structure deposited on the cascades transforming the jagged surface into a comparatively smooth one, which conducts the flow at a steep angle onto the unprotected bed.

(1) The chute and baffle arrangement of the final design was shown to be satisfactory for the wash crossings of the Coachella Canal. Although this design, as shown by figures 8 and 22 to 25 did not prevent retrogression, it did furnish protection to the structures. As degradation continues the chute can be extended.

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3. Samuel Shulits, The Schoklitsch Bed-Load Formula, Engineering, June 21, 1935, page 644.

4. F. T. Mavis, Te-Yun, and Edward Soucek, The Transportation of Detritus by Flowing Water - II, University of Iowa Studies, New Series No. 341, Sept. 1, 1937, University of Iowa, Iowa City, Iowa.

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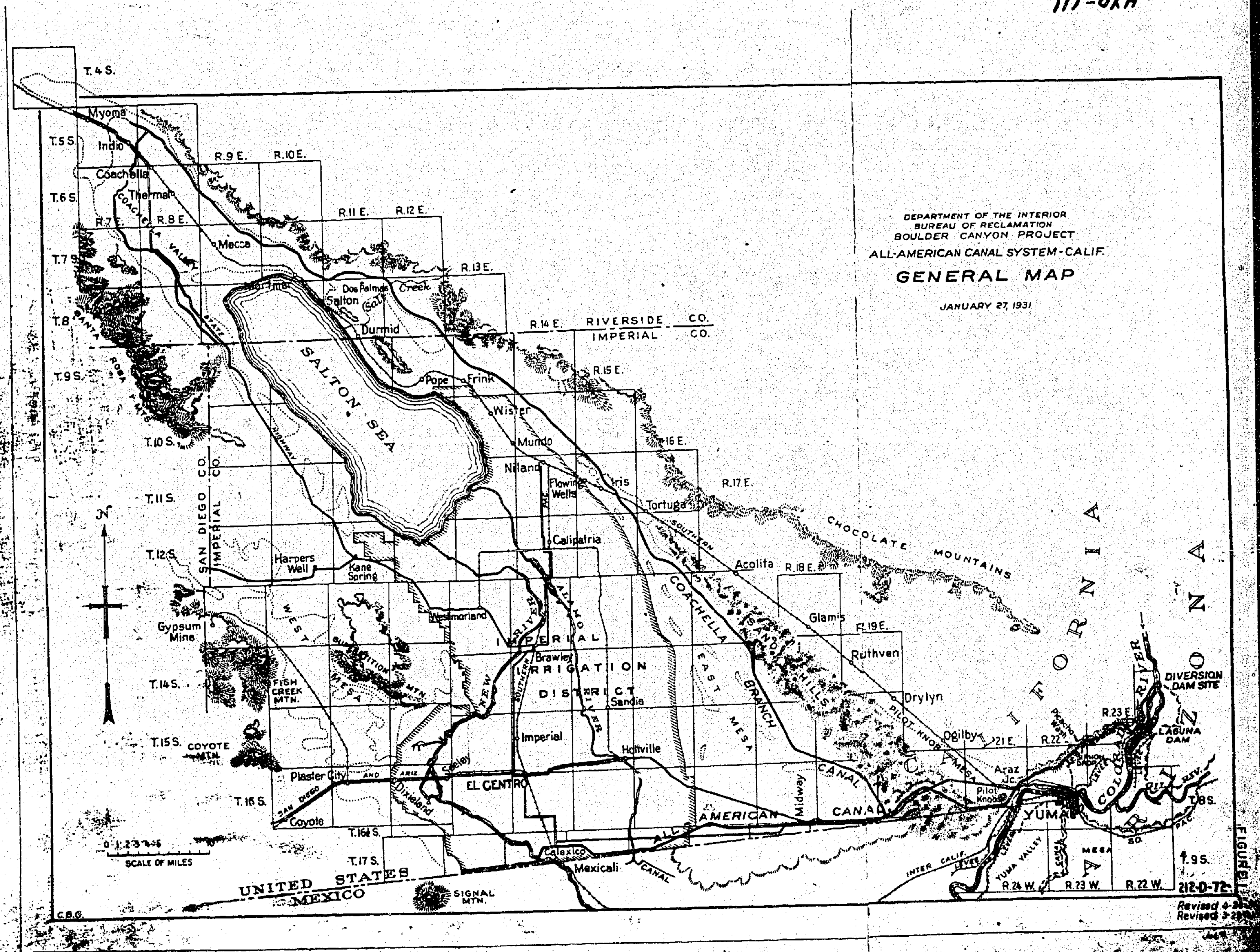
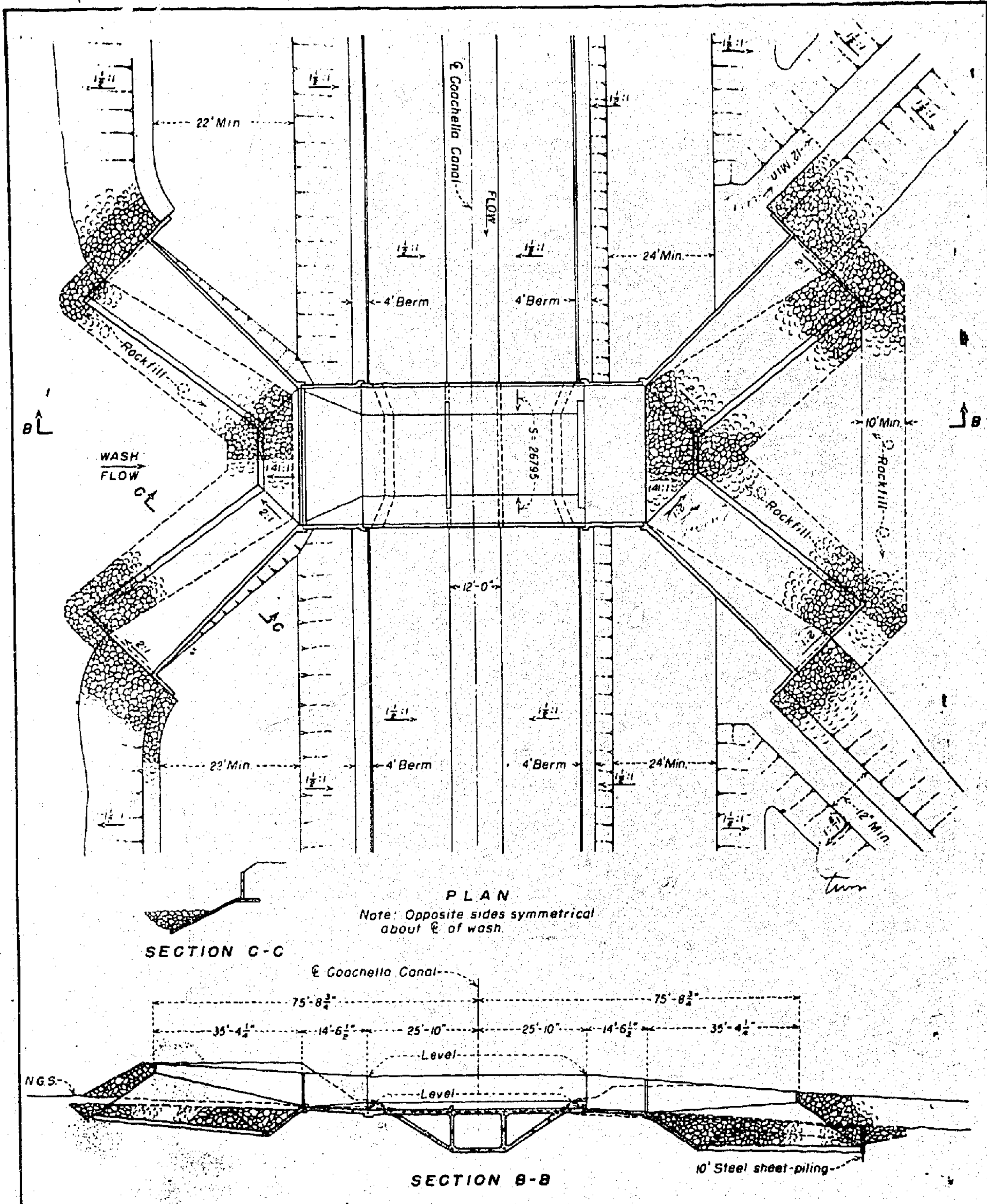
7. C. H. MacDougal, Bed Sediment Transportation in Open Channels, Transactions American Geophysical Union, 1933, page 491.


DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
ALL-AMERICAN CANAL SYSTEM-CALIF.
GENERAL MAP
JANUARY 27, 1931
SALTON SEA
CHOCOLATE MOUNTAINS
RIVERSIDE CO.
IMPERIAL CO.
SAN DIEGO CO.
IMPERIAL CO.
IMPERIAL IRRIGATION DISTRICT
ALL AMERICAN CANAL
COACHELLA BRANCH
EAST MESA
WEST MESA
UNITED STATES
MEXICO
CALIFORNIA
ARIZONA
COLORADO RIVER
DIVERSION DAM SITE
LAGUNA DAM
YUMA
EL CENTRO
Indio
Coachella
Thermal
Mecca
Myoma
Salton
Durmid
Niland
Calipatria
Brawley
Imperial
Holtville
Calexico
Mexicali
Seeley
Dixieland
Plaster City
Coyote
Westmorland
Kane Spring
Harpers Well
Gypsum Mine
Glamis
Ruthven
Drylyn
Ogilby
Araz Jc.
Pilot Knob
Iris
Tortuga
Acolita
Flowing Wells
Mundo
Wister
Frink
Pope
Midway
SIGNAL MTN.
COYOTE MTN.
FISH CREEK MTN.
SCALE OF MILES
FIGURE 1
212-D-72

HYD-411

FIGURE 2



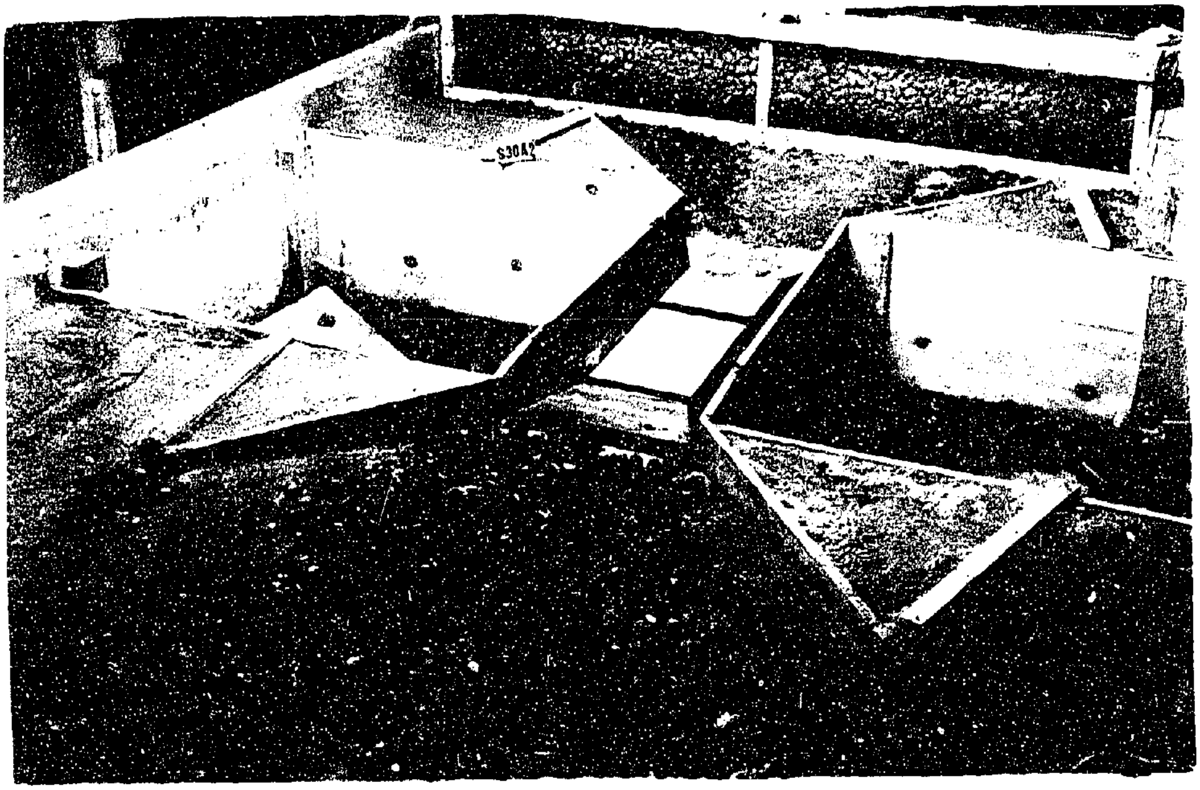
ALL-AMERICAN CANAL SYSTEM - CALIF.

COACHELLA CANAL

TYPICAL PLAN AND SECTIONS
OVERCHUTES WITH TRAPEZOIDAL SIPHONS
ORIGINAL DESIGN

... ORIGINAL DESIGN OF SMALL COACHELLA OVERCHUTE

FIGURE 3

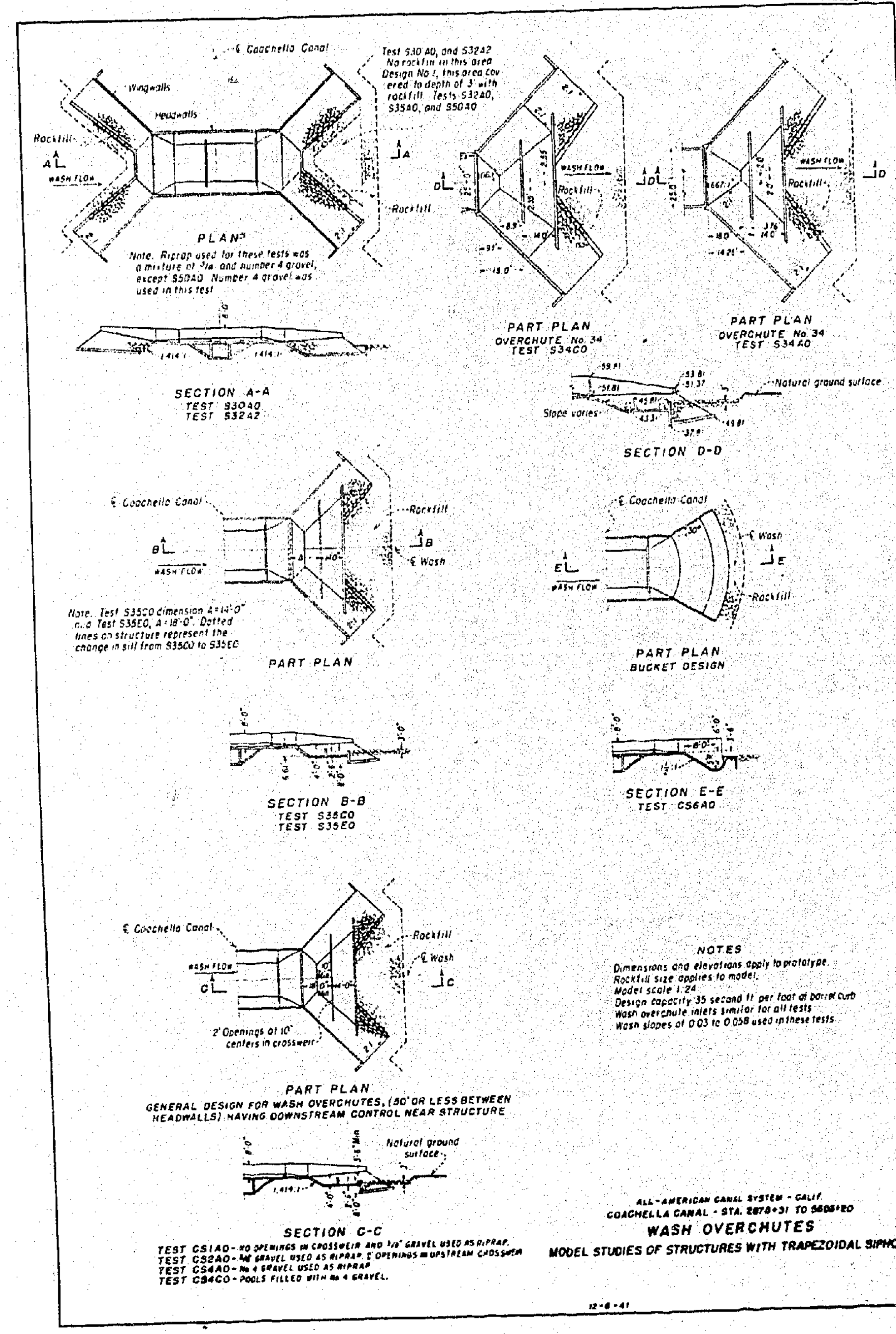


A. SCOUR AFTER 30-MINUTE FLOW AT DESIGNED CAPACITY - WASH SLOPE 0.031

B. SCOUR AT SHEET PILING AFTER 30-MINUTE FLOW AT DESIGNED CAPACITY - WASH SLOPE [illegible]

SCOUR IN ORIGINAL DESIGN.

FIGURE 4



℄ Coachella Canal
Wingwalls
Headwalls
Rockfill
WASH FLOW
PLAN
Note. Riprap used for these tests was a mixture of 3/16 and number 4 gravel, except S50A0. Number 4 gravel was used in this test.
Test S30A0, and S3242. No rockfill in this area. Design No 1, this area covered to depth of 3' with rockfill. Tests S32A0, S35A0, and S50A0
SECTION A-A
TEST S30A0
TEST S3242
PART PLAN
OVERCHUTE No. 34
TEST S34C0
PART PLAN
OVERCHUTE No. 34
TEST S34A0
Natural ground surface
Slope varies
SECTION D-D
℄ Wash
Note. Test S35C0 dimension A=14'-0" and Test S35E0, A=18'-0". Dotted lines on structure represent the change in sill from S35C0 to S35E0
PART PLAN
PART PLAN
BUCKET DESIGN
SECTION B-B
TEST S35C0
TEST S35E0
SECTION E-E
TEST CS6A0
2' Openings at 10' centers in crosswell
PART PLAN
GENERAL DESIGN FOR WASH OVERCHUTES, (50' OR LESS BETWEEN HEADWALLS) HAVING DOWNSTREAM CONTROL NEAR STRUCTURE
NOTES
Dimensions and elevations apply to prototype.
Rockfill size applies to model.
Model scale 1:24
Design capacity 35 second ft per foot of barrel curb
Wash overchute inlets similar for all tests
Wash slopes of 0.03 to 0.058 used in these tests.
Natural ground surface
SECTION C-C
TEST CS1A0 - NO OPENINGS IN CROSSWEIR AND 3/8" GRAVEL USED AS RIPRAP.
TEST CS2A0 - 3/8" GRAVEL USED AS RIPRAP, 2' OPENINGS IN UPSTREAM CROSSWEIR
TEST CS4A0 - No 4 GRAVEL USED AS RIPRAP
TEST CS4C0 - POOLS FILLED WITH No 4 GRAVEL.
ALL-AMERICAN CANAL SYSTEM - CALIF.
COACHELLA CANAL - STA. 2878+31 TO 5606+20
WASH OVERCHUTES
MODEL STUDIES OF STRUCTURES WITH TRAPEZOIDAL SIPHONS
12-6-41

A. MODEL BEFORE TEST.

B. FLOW CONDITIONS.

C. SCOUR AFTER 20 MINUTES.

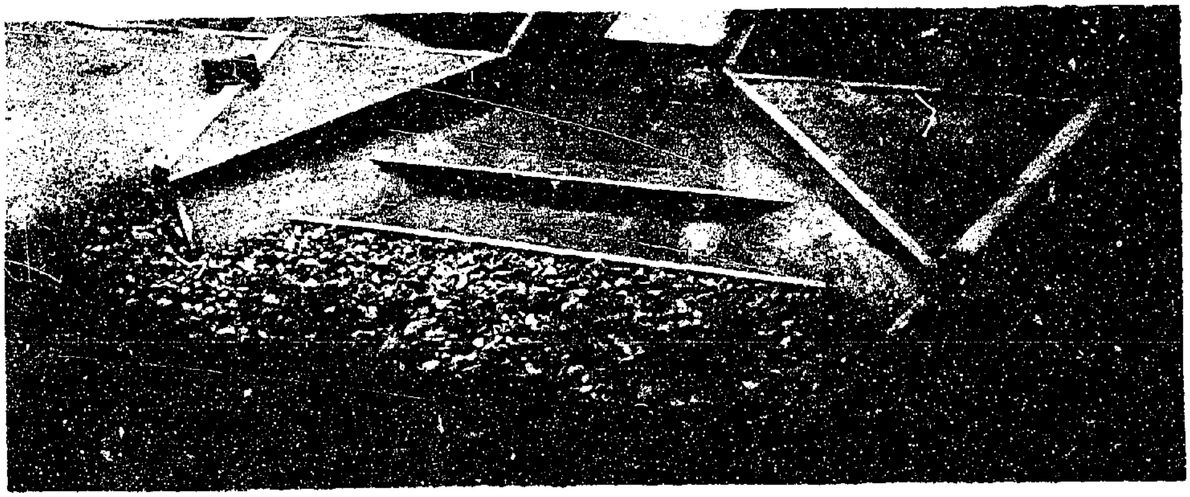
SCOUR IN ORIGINAL DESIGN WITH SLOPE OF 0.034 AND DESIGNED FLOW.

FIGURE 6

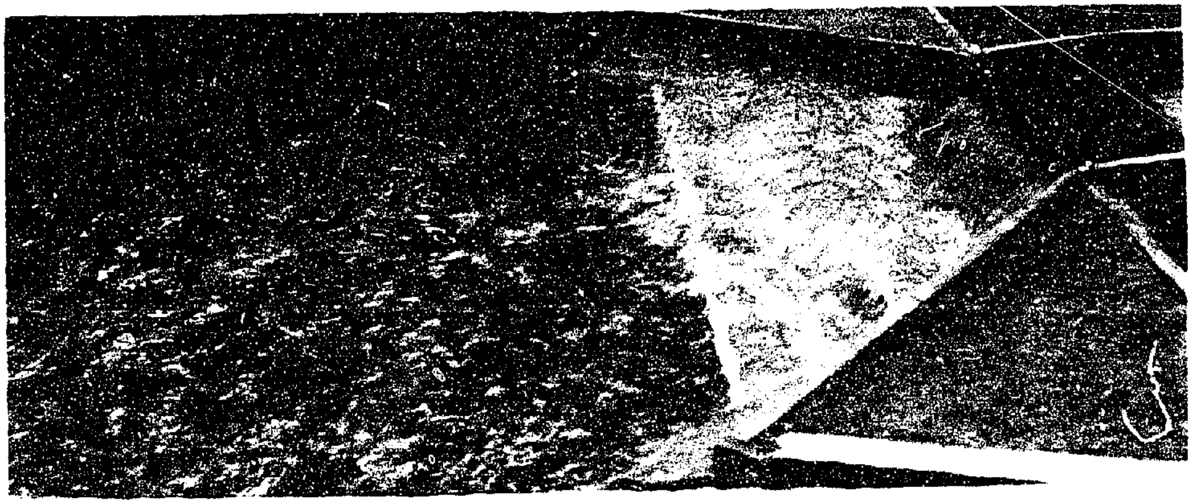
A. FLOW CONDITIONS.

B. SCOUR AFTER 10 MINUTES.

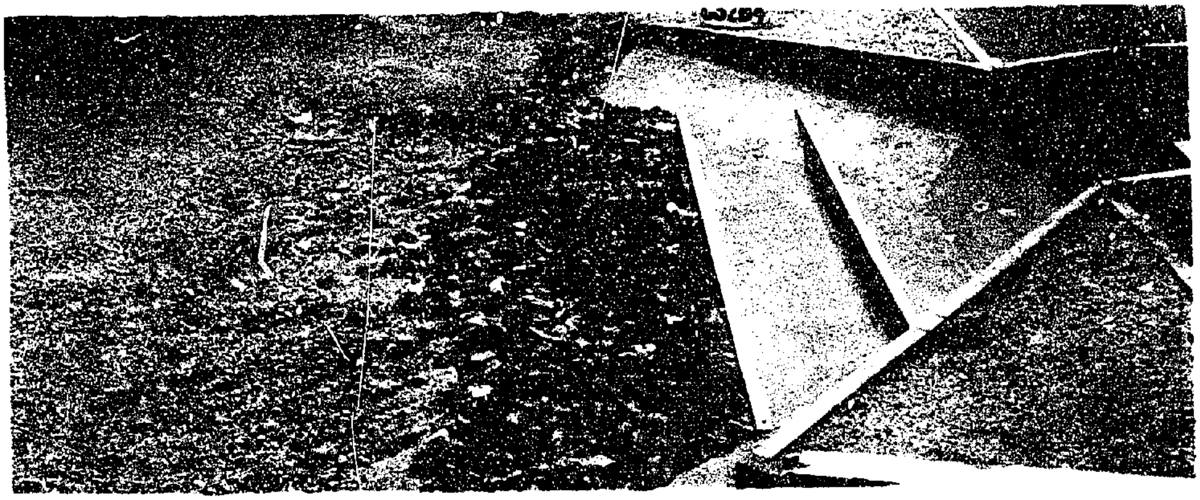
SCOUR IN ORIGINAL DESIGN WITH SLOPE OF 0.034 AND TWICE DESIGNED FLOW.



A. MODEL ARRANGEMENT FOR NARROW OVERCHUTE - WASH SLOPE 0.042



B. FLOW CONDITIONS - 200% DESIGN CAPACITY.



C. SCOUR AFTER 2-HOUR FLOW AT 200% DESIGN CAPACITY.

DOUBLE POOL DESIGN - NARROW OVERCHUTES.

HYD-111

FIGURE 8
DRAWING No. 100

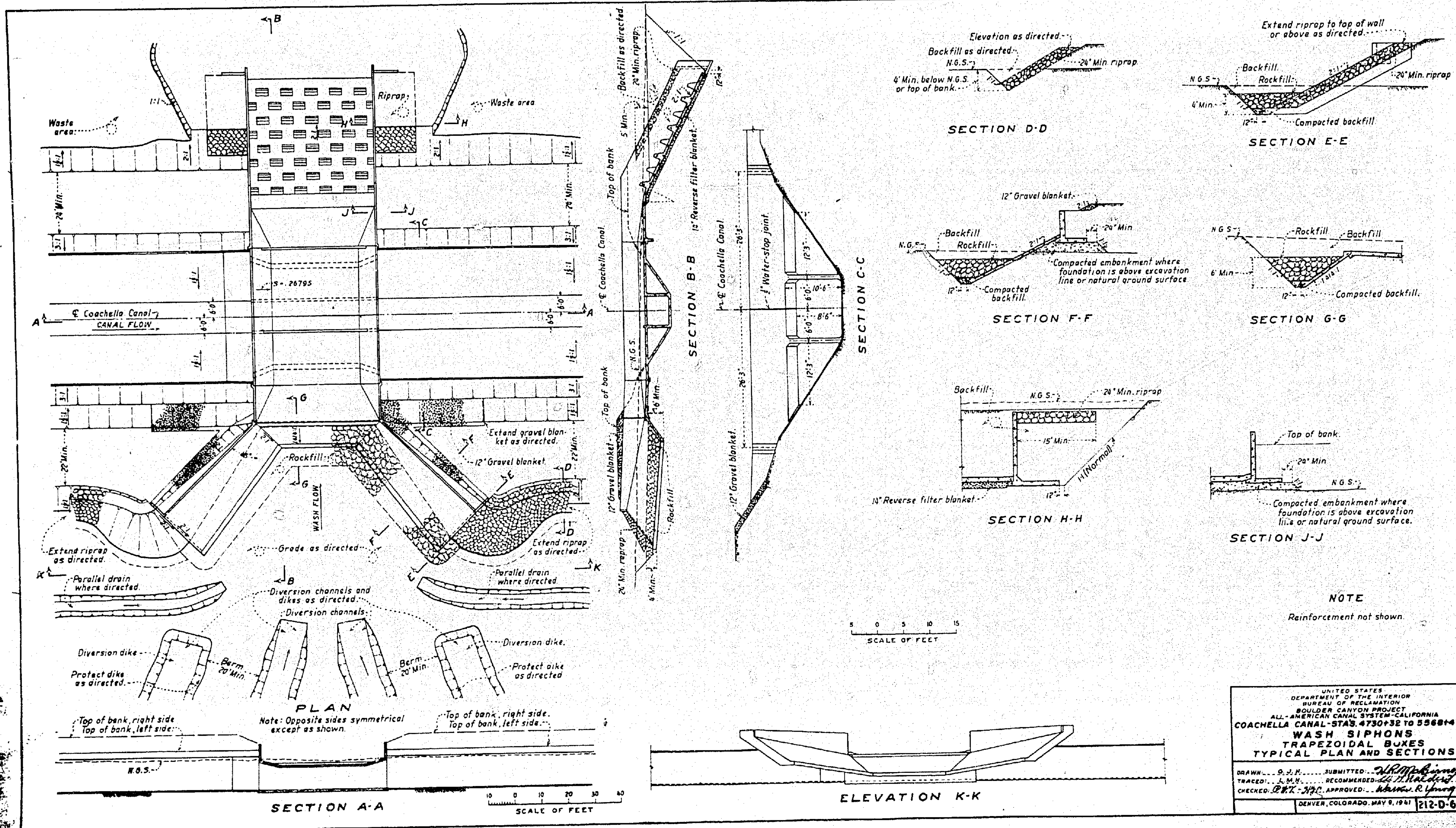
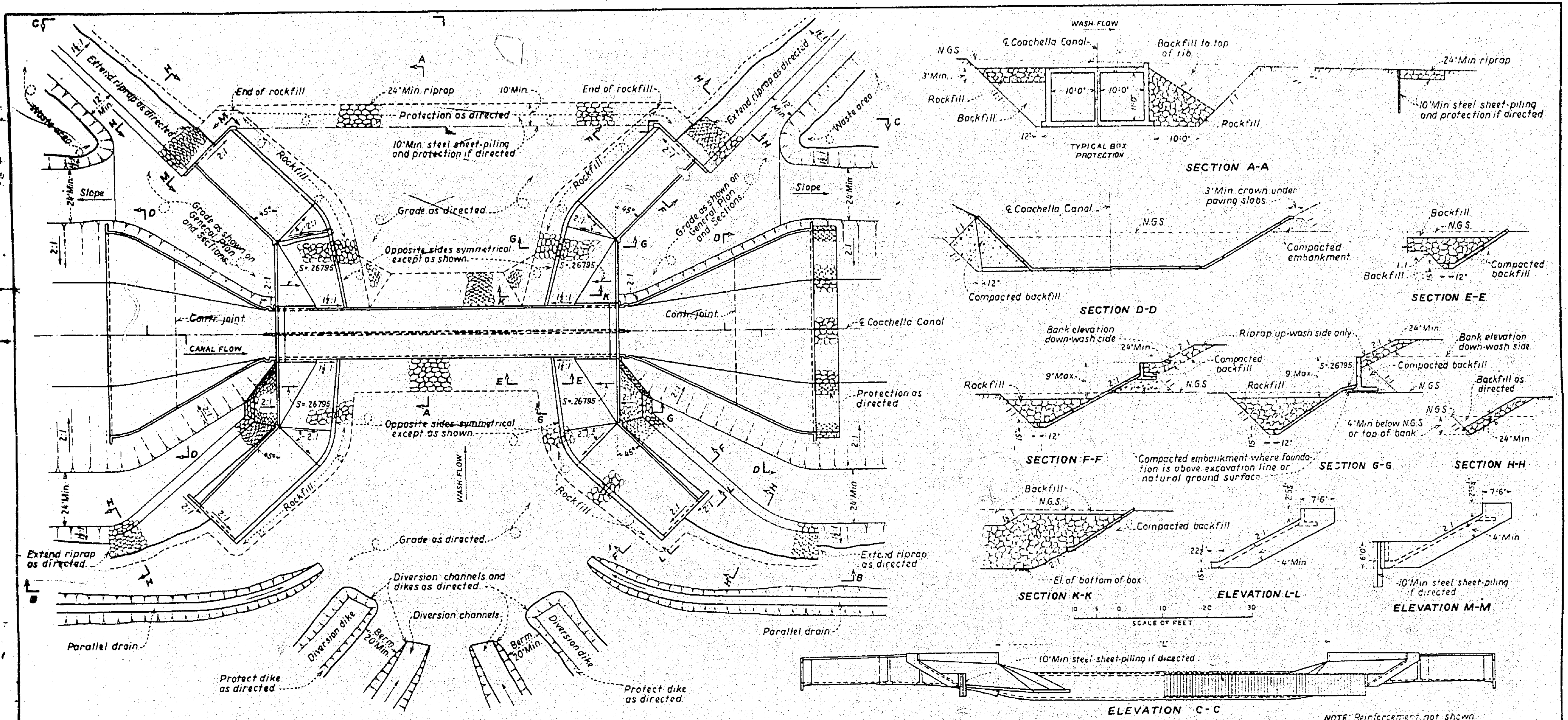
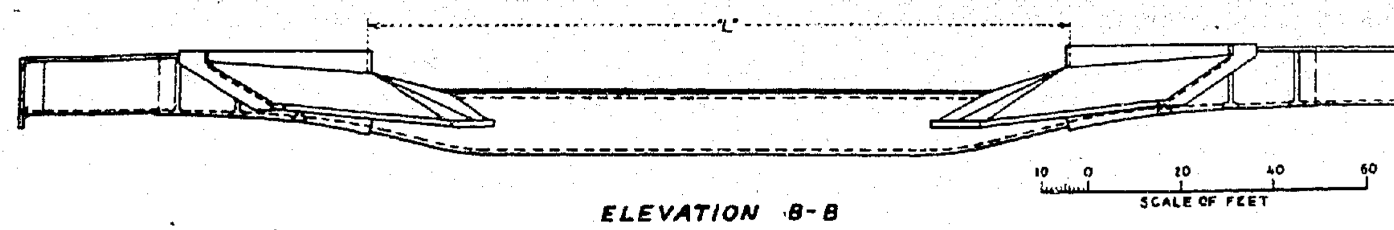


FIGURE 9





ELEVATION B-B

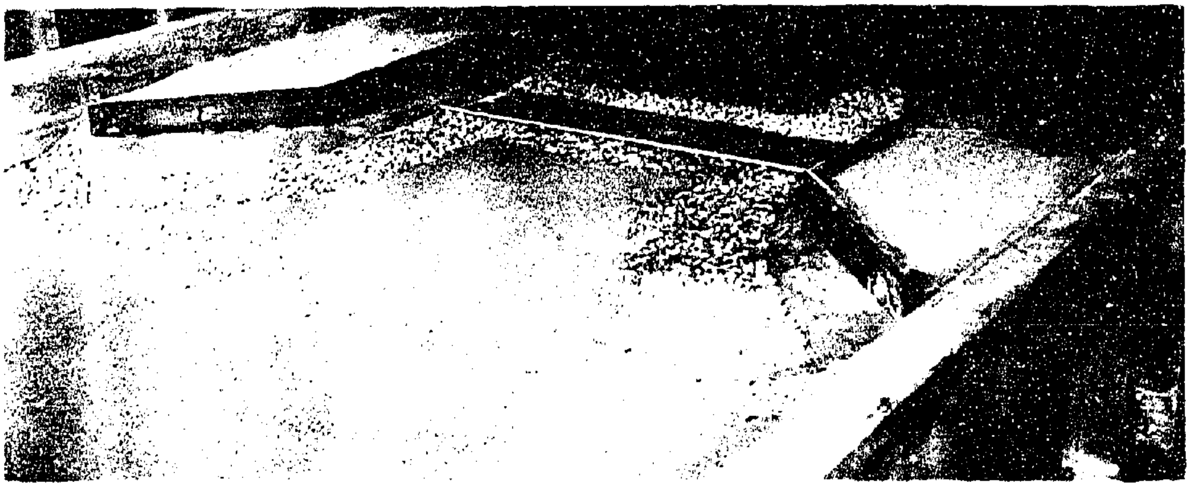
NOTE: Reinforcement not shown.
THIS DRAWING SUPERSEDES DRAWING 212-D-5072

TABLE OF BARREL LENGTHS

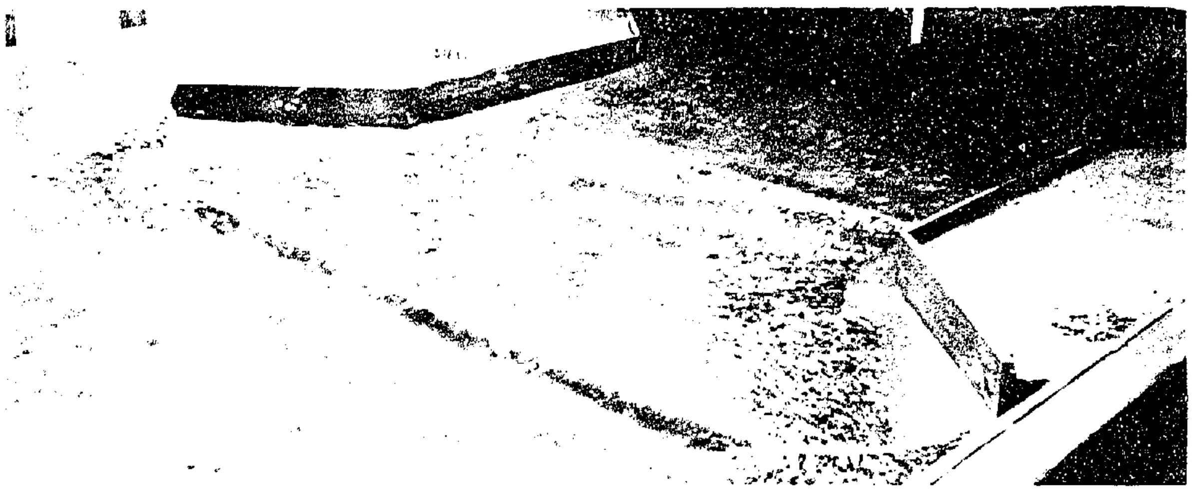
SIPHON No.	LENGTH "L" SPEC. No. 846	LENGTH "L" REVISED	SIPHON No.	LENGTH "L" SPEC. No. 846	LENGTH "L" REVISED	SIPHON No.	LENGTH "L" SPEC. No. 848	LENGTH "L" REVISED
1	195	265	12	277	277	23	387	548
2	204	300	13	133	181	24	434	614
3	101	137	14	150	198	25	67	93
4	97	133	15	112	157	26	116	144
5	120	152	16	113	153	28	97	125
6	175	245	17	134	176	29	157	213
7	185	255	18	49	65	30	47	53
8	102	142	19	61	70	31	114	156
9	129	177	20	65	76	32	32	40
10	126	172	21	58	68			
11	127	166	22	164	223			

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
ALL-AMERICAN CANAL SYSTEM-CALIFORNIA
COACHELLA CANAL-STA'S. 2078+31 TO 4563+10
WASH SIPHONS
DOUBLE 10'x 11' BOXES
TYPICAL PLAN AND SECTIONS

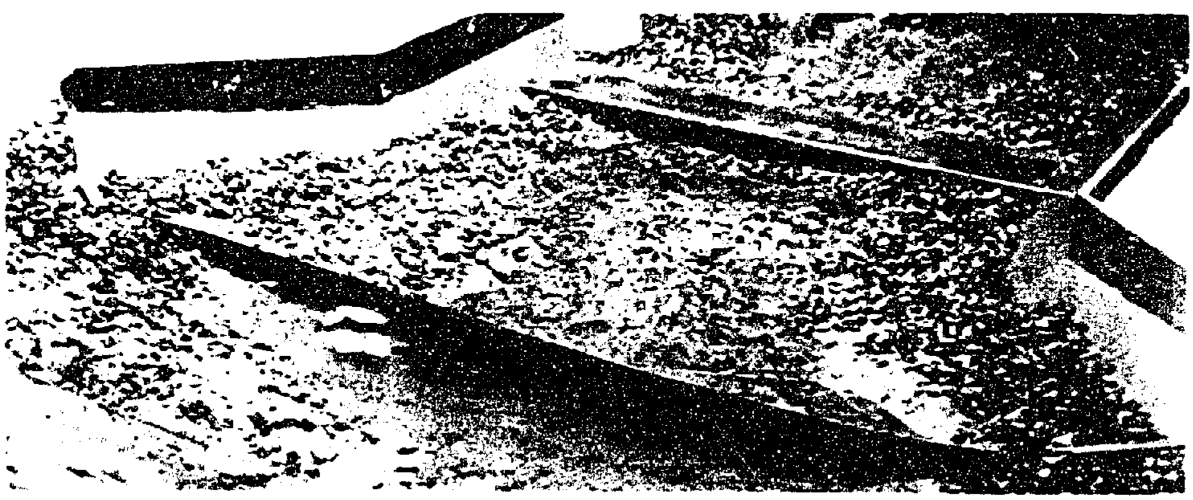
DRAWN E.R.H. SUBMITTED
TRACED K.A.R. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO DEC. 22, 1939 212-D-5866



A. MODEL ARRANGEMENT OF ORIGINAL DESIGN - WASH SLOPE 0.0[illegible]8



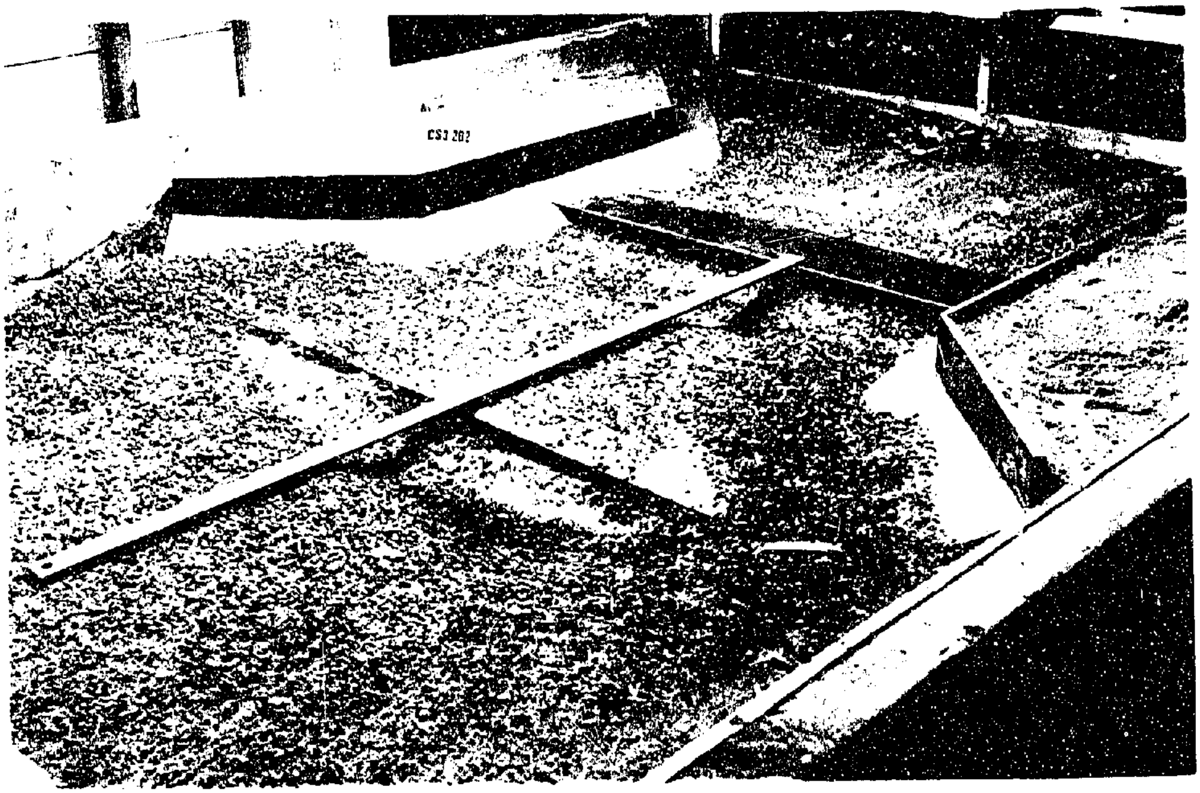
B. FLOW CONDITIONS AT DESIGNED CAPACITY.



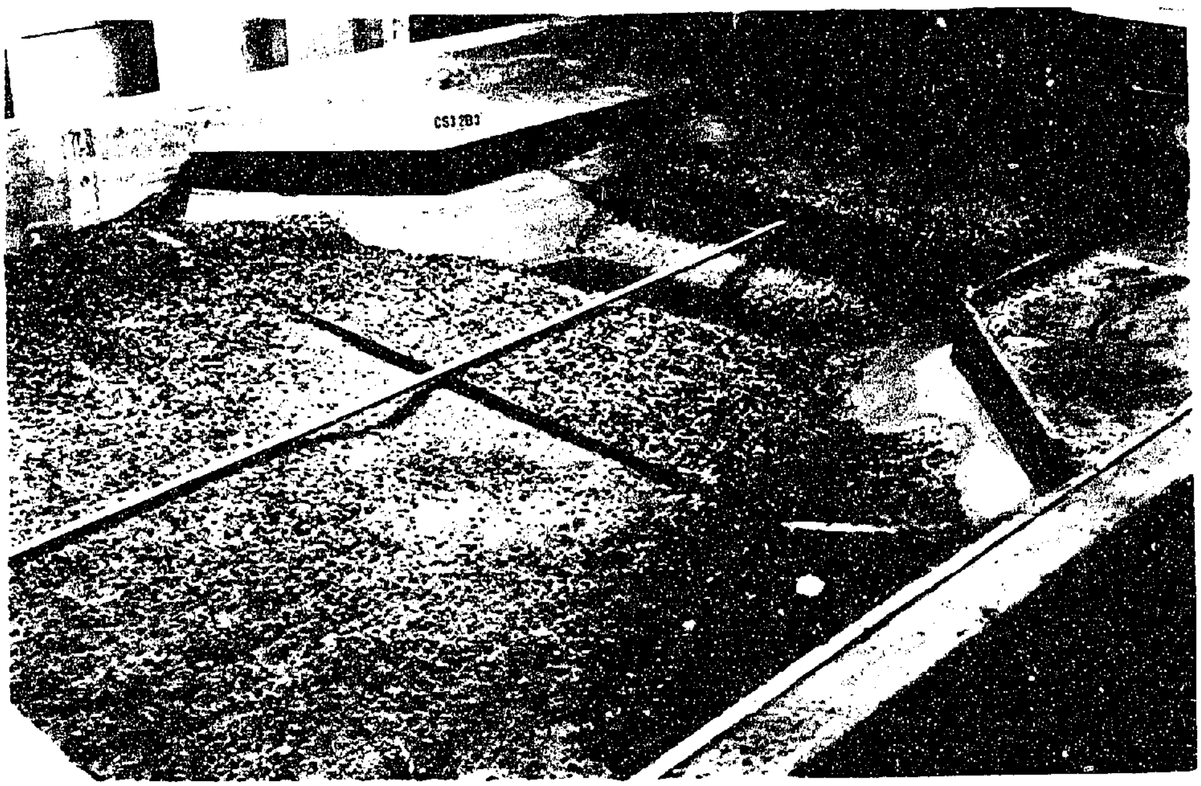
C. SCOUR AFTER 20 MINUTE FLOW.

SCOUR IN ORIGINAL DESIGN OF WIDE OVERCHUTE

FIGURE 11



A. SCOUR AFTER FLOWS OF [illegible] MINUTES EACH AT [illegible] AND 100% DESIGN CAPACITY.
[illegible] SLOPE [illegible]

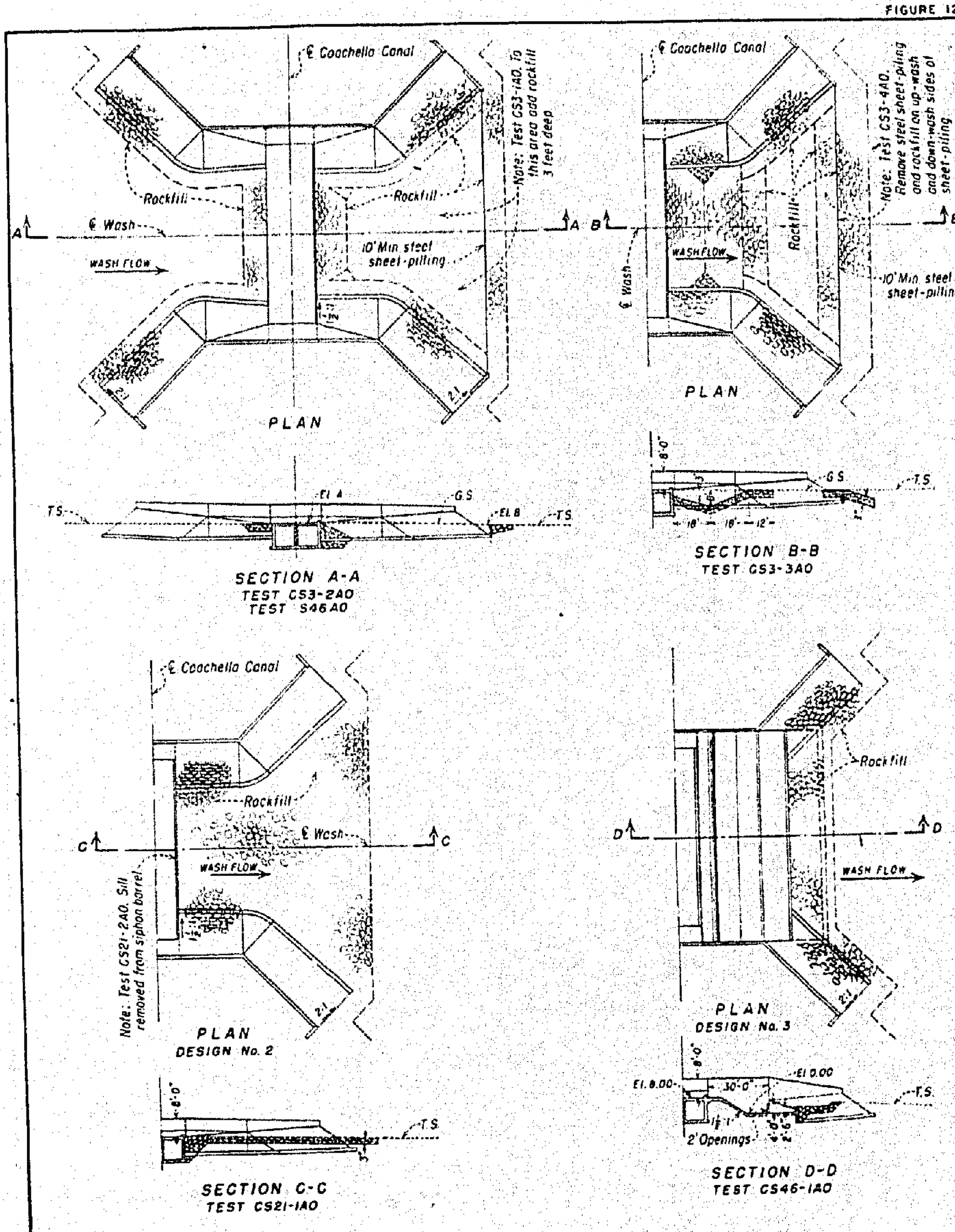


B. SCOUR AFTER FLOWS OF [illegible] MINUTES EACH AT [illegible], [illegible] AND [illegible]% DESIGN CAPACITY.
[illegible] SLOPE [illegible]

SCOUR WITH GRAVEL BLANKET IN OUTLET

HYD-111

FIGURE 12



TEST No.	EL. A	EL. B	* T.S.
CS3-2AO	0.83'	0.00	1.30
S46AO	1.93'	0.00	5.80

* T.S. (Transv. Slope) is drop per 100'.

NOTES

Model scale 1:24.
No. 4 gravel used for riprap (model scale).
Design capacity 35 second feet per foot at barrel curb.
Wash overchute inlets similar for all tests.
Dimensions and elevations apply to prototype.

ALL-AMERICAN CANAL SYSTEM - CALIF.
COACHELLA CANAL - STA. 2078+31 TO 5696+20

WASH OVERCHUTES

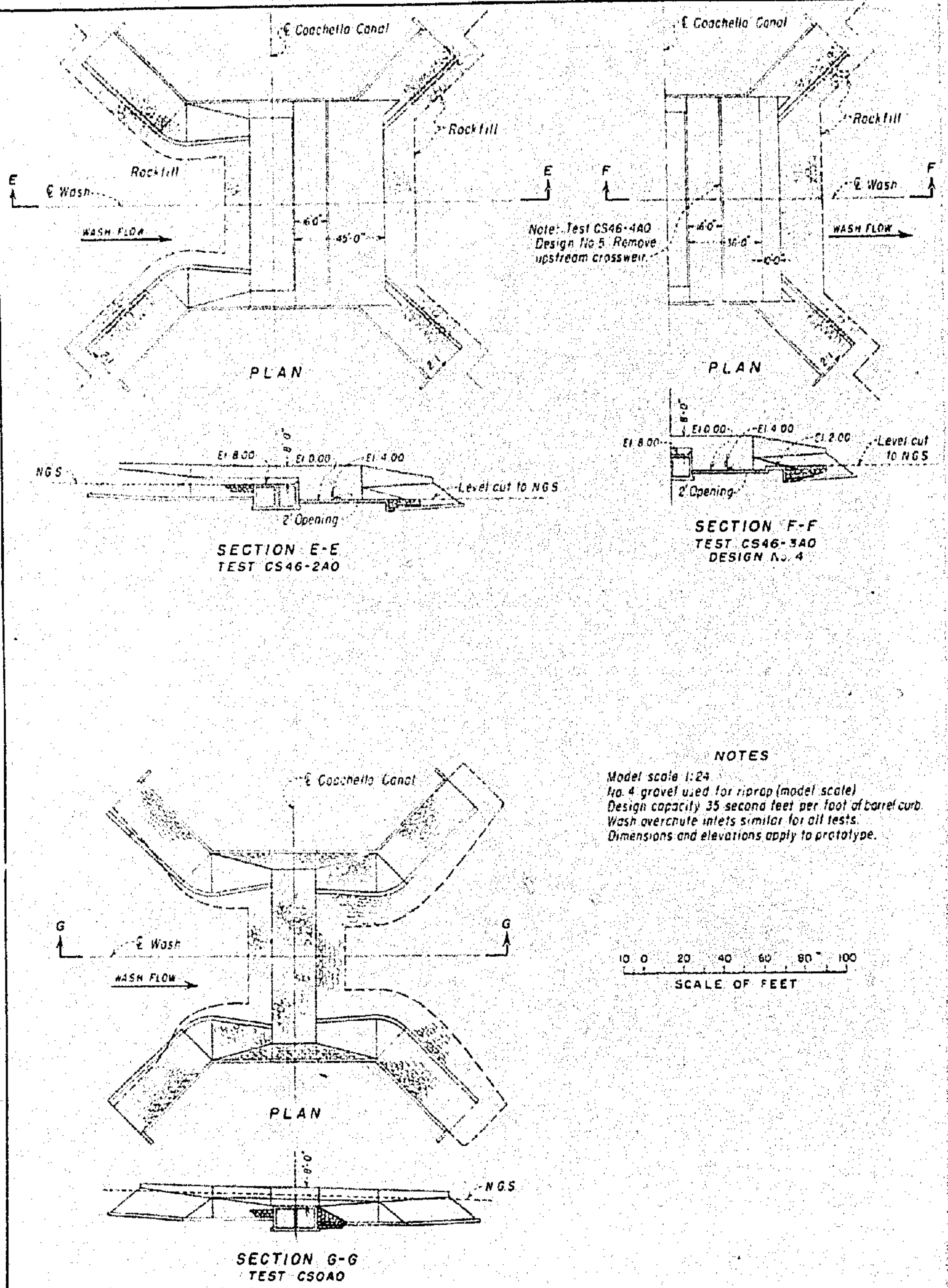
MODEL STUDIES OF STRUCTURES WITH RECTANGULAR SIPHONS

SHEET 1 OF 2

12-8-41

HYD-111

FIGURE 13



ALL-AMERICAN CANAL SYSTEM - CALIF.

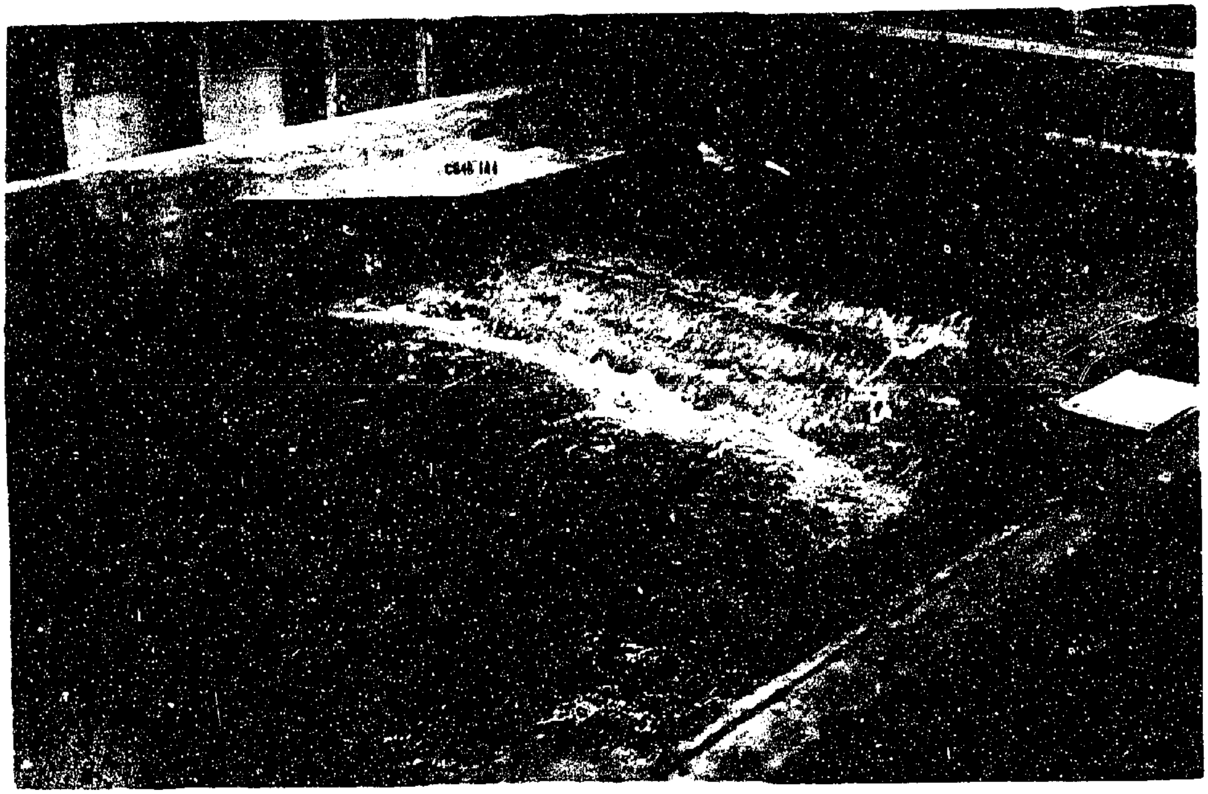
COACHELLA CANAL - STA. 2078+31 TO 5696+20

WASH OVERCHUTES

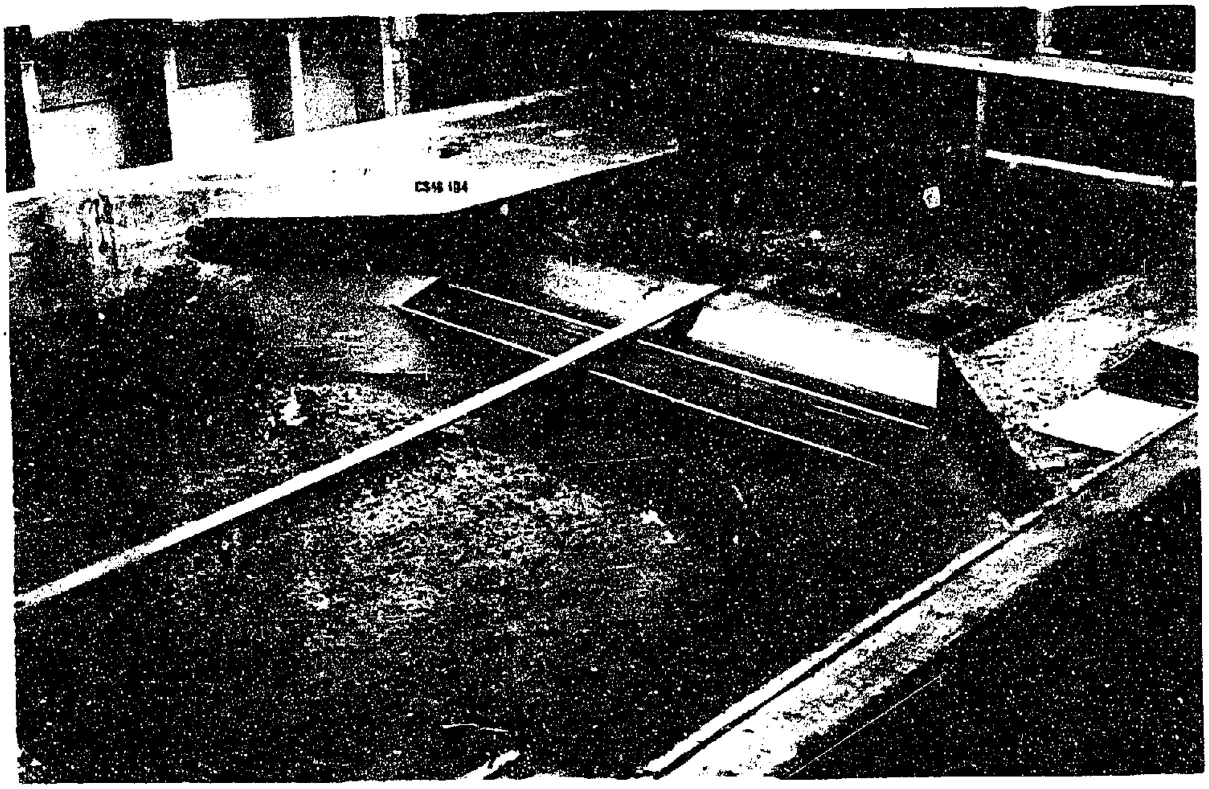
MODEL STUDIES OF STRUCTURES WITH RECTANGULAR SIPHONS

SHEET 2 OF 2

12-8-41



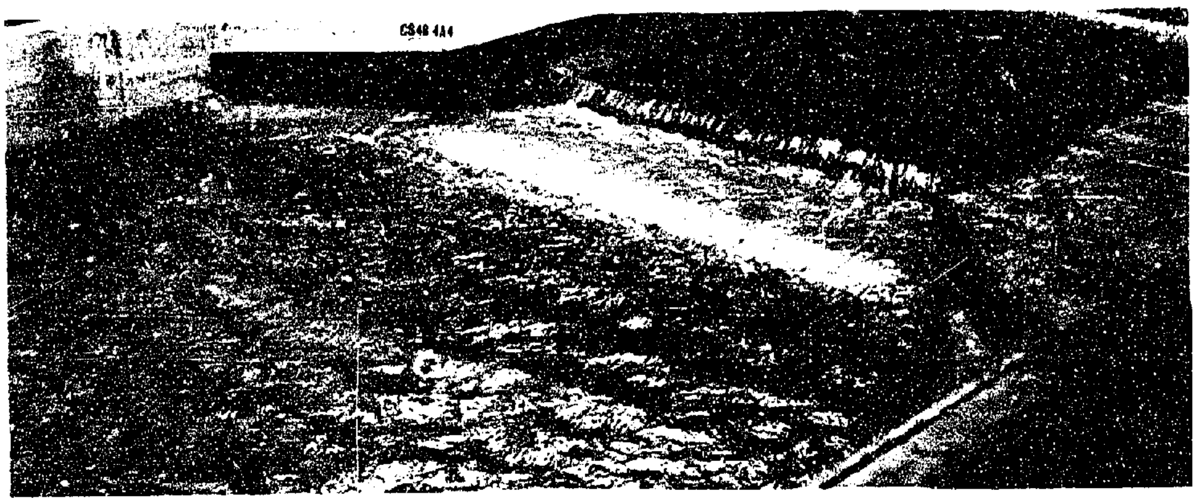
A. FLOW CONDITIONS AT DESIGNED CAPACITY. WASH SLOPE 0.058.



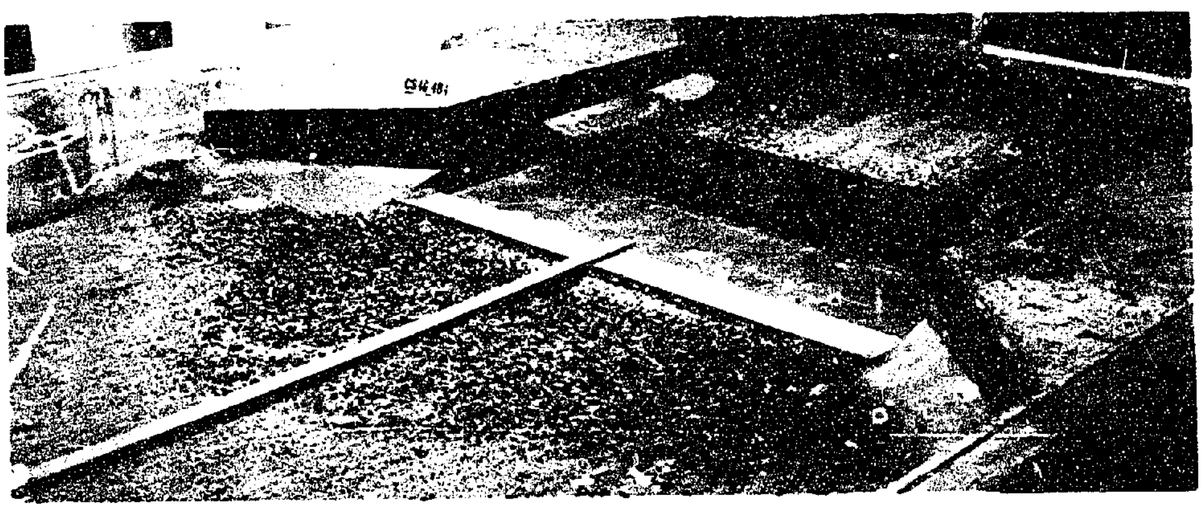
B. SCOUR AFTER 1-HOUR FLOW.

DOUBLE POOL DESIGN

A. MODEL BEFORE TEST.



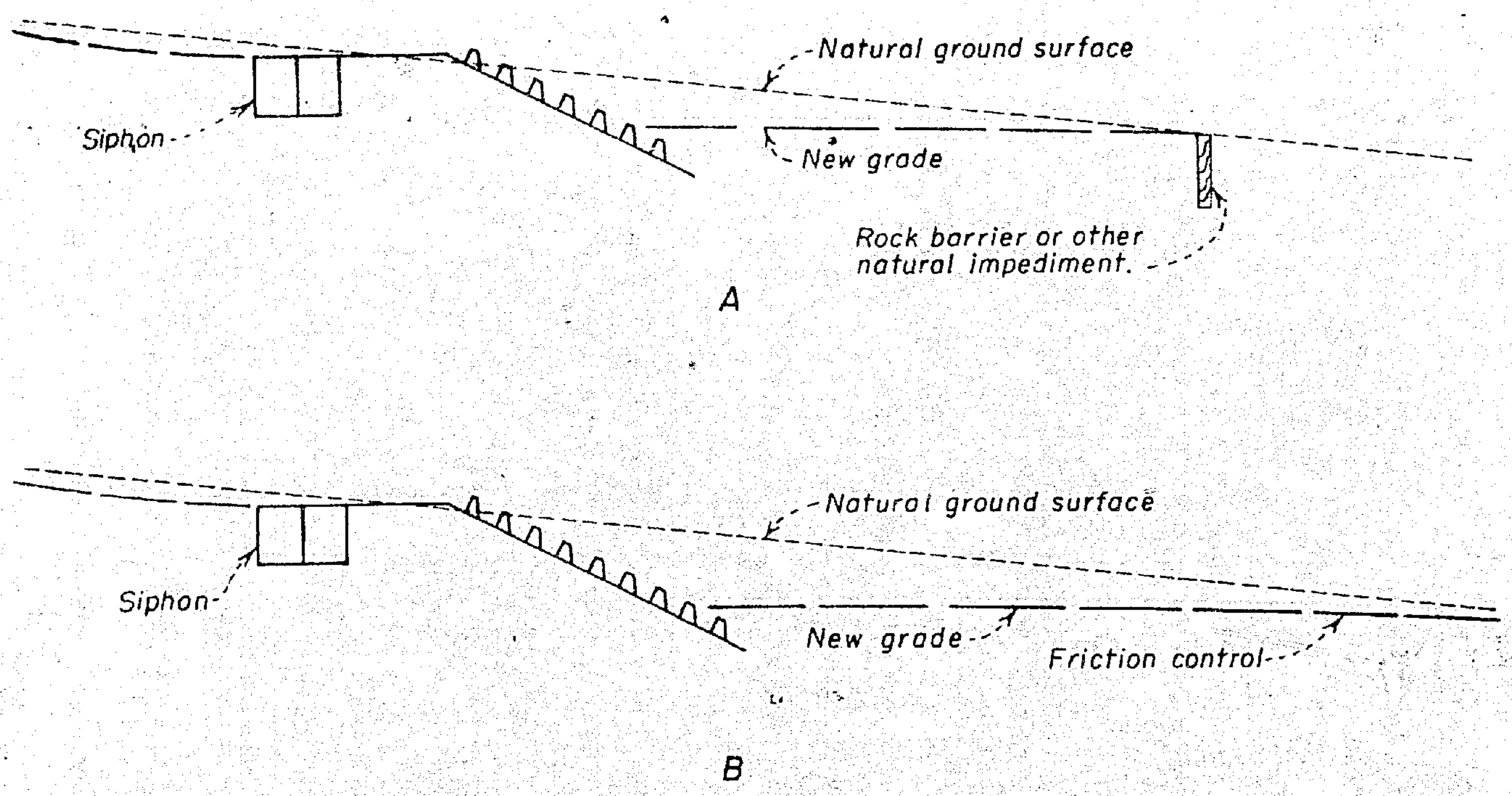
B. FLOW CONDITIONS AT DESIGN CAPACITY.



C. SCOUR AFTER 4[illegible]-MINUTE FLOW.

SINGLE POOL DESIGN

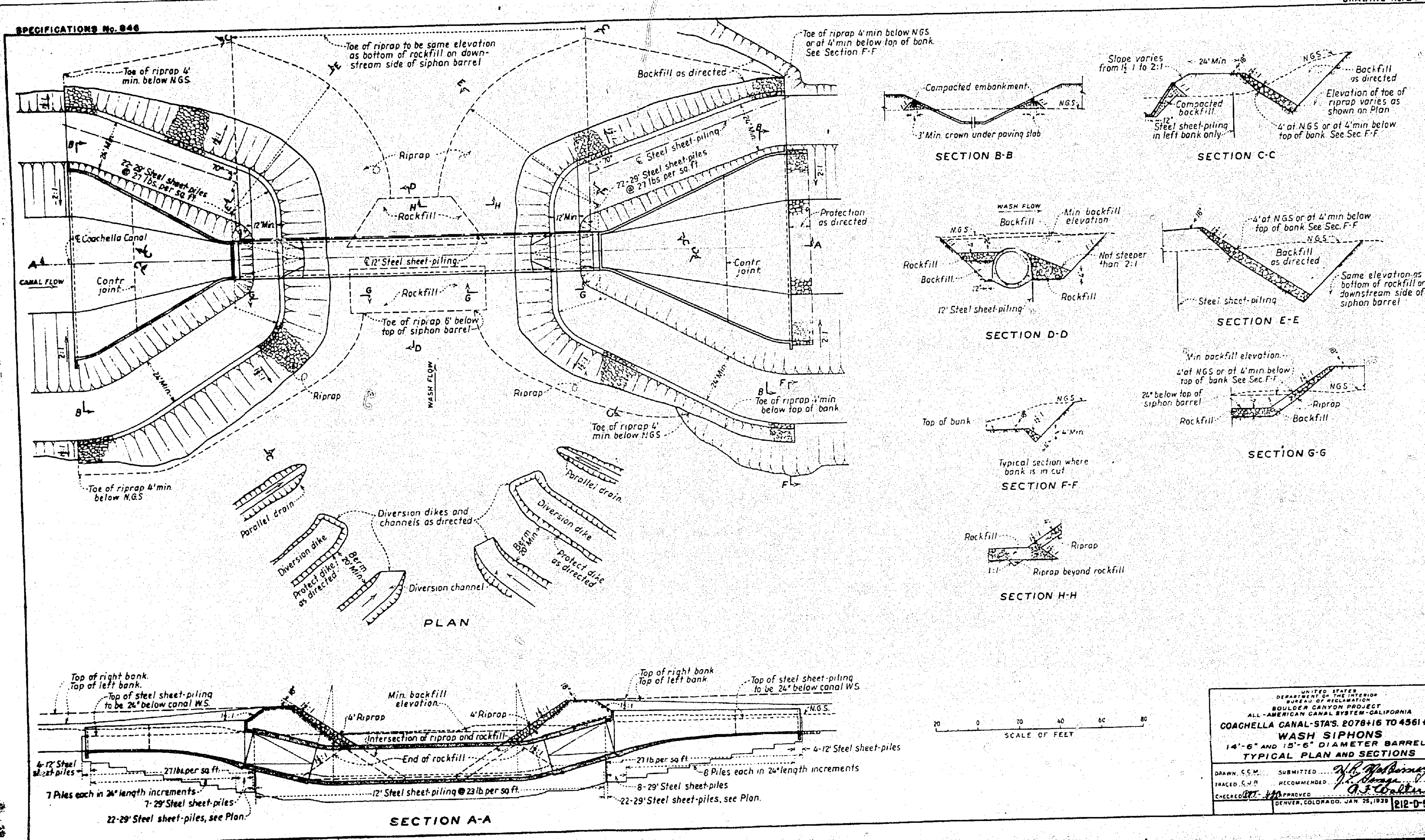
FIGURE 16

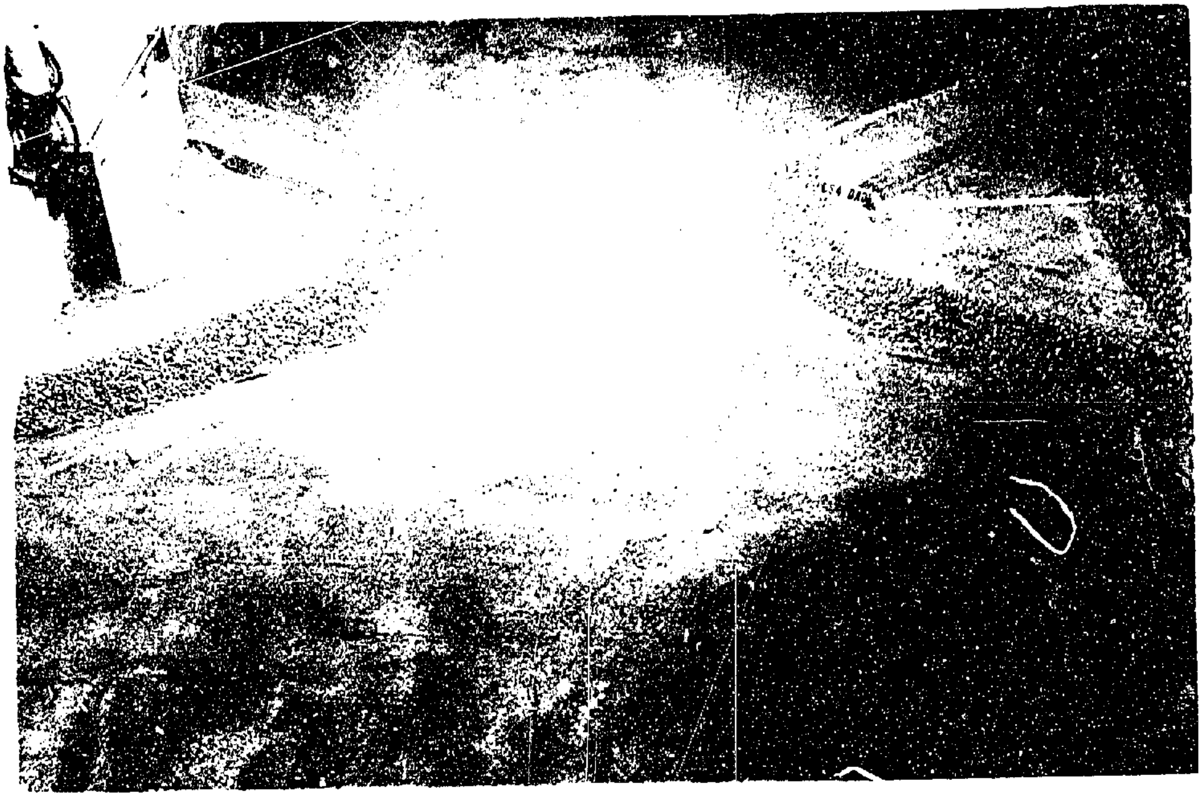


RELATION OF RETROGRESSION TO ORIGINAL SLOPE

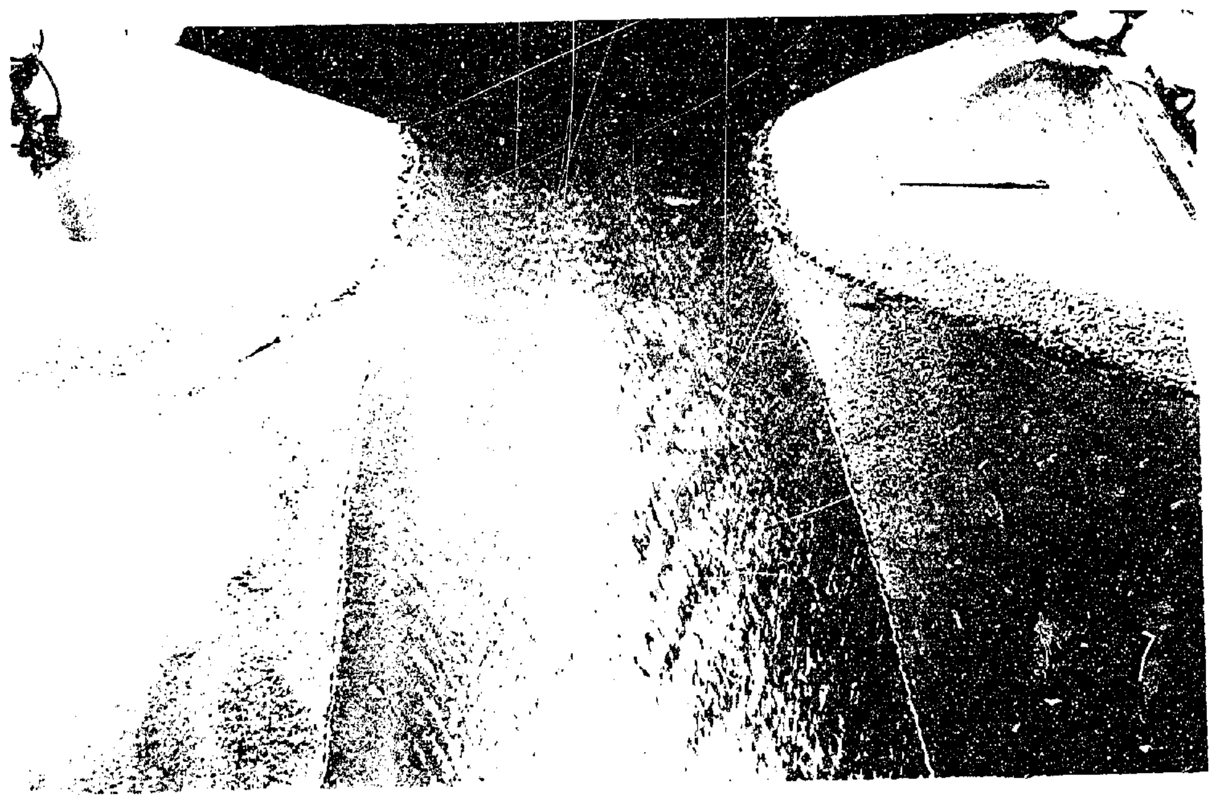
1-6-42

FIGURE 17
DRAWING No. 24



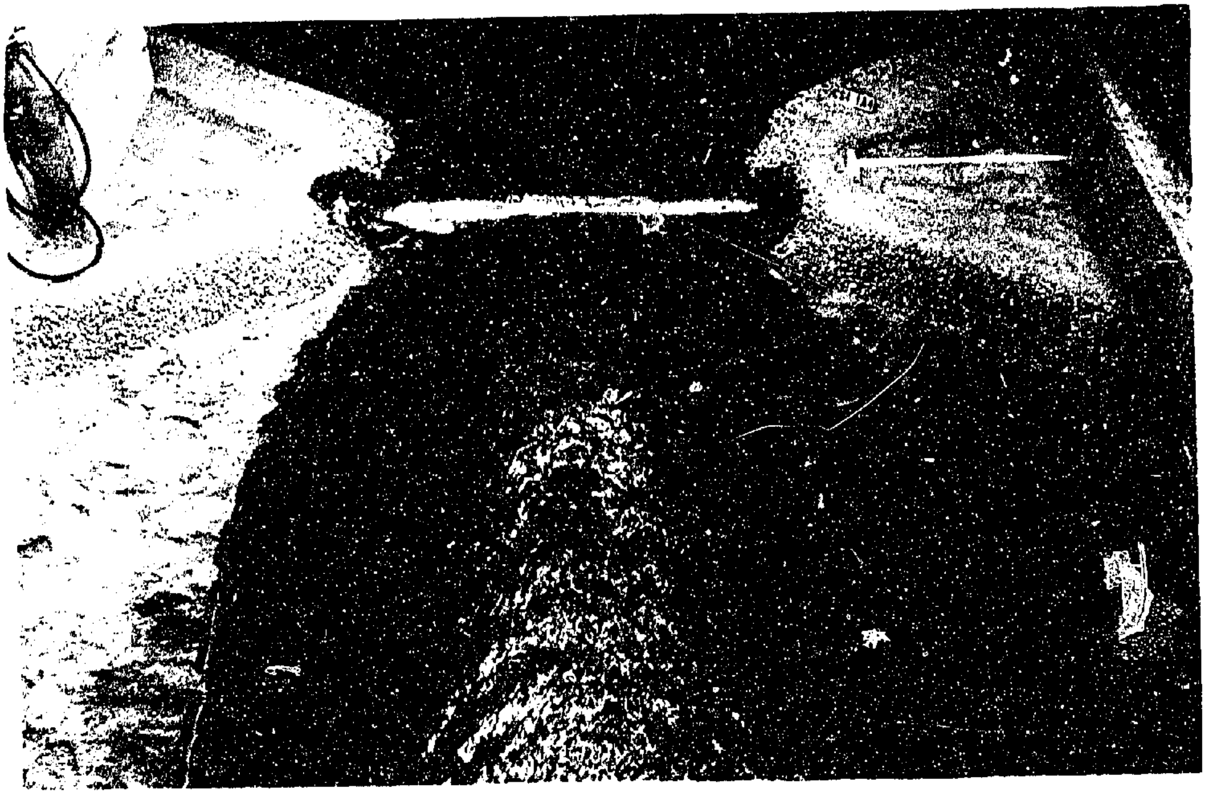


A. MODEL BEFORE TEST. WASH SLOPE 0.028



B. FLOW CONDITION AT DESIGN CAPACITY - SLOPE 0.028.

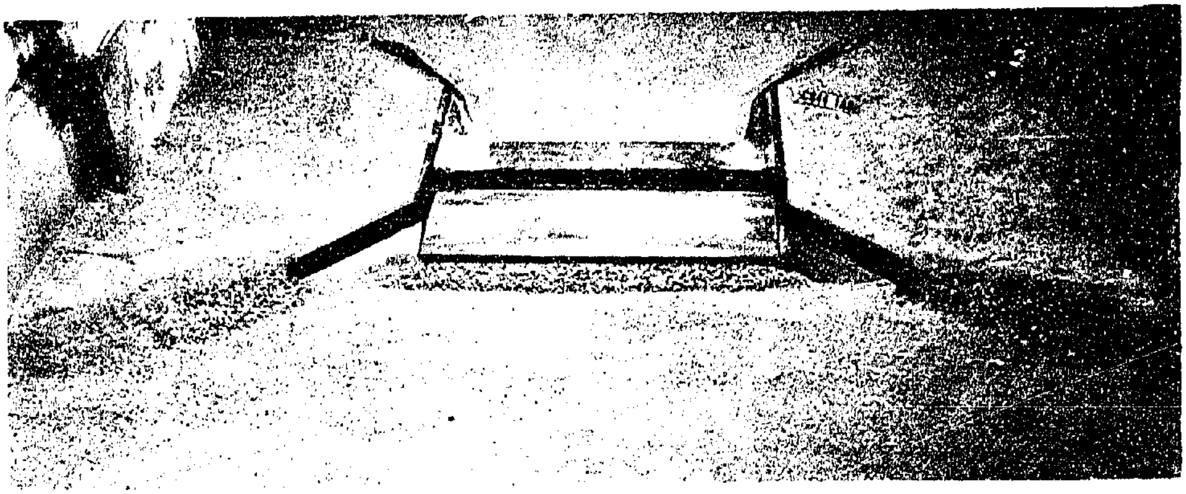
PROGRESS OF SCOUR BELOW CIRCULAR SIPHON DUE TO RETROGRESSION - SHEET 1 OF 2



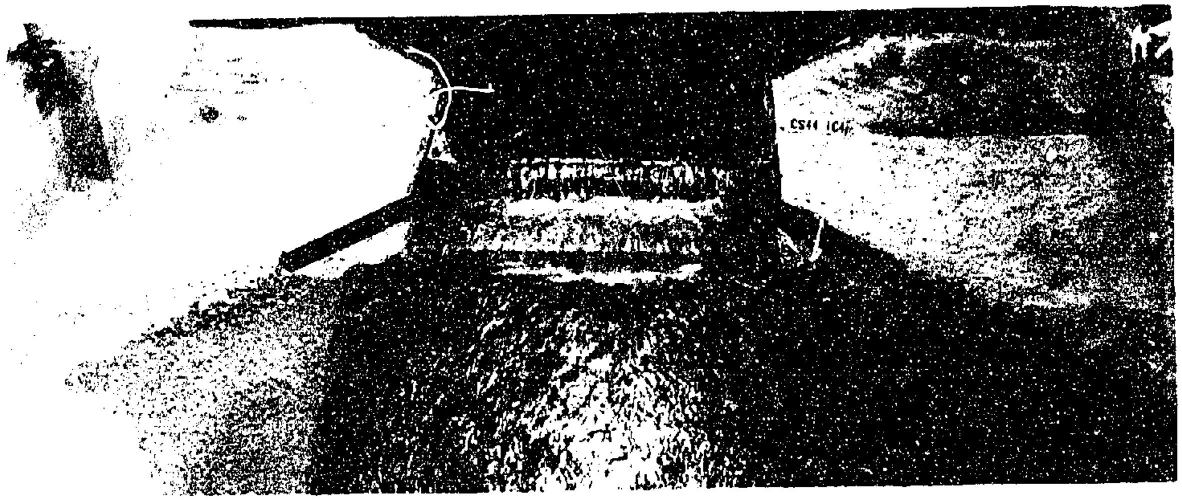
A. FLOW CONDITIONS AT DESIGNED CAPACITY - CONTROL LOWERED 1 INCH.

B. SCOUR WITH CONTROL LOWERED 1 INCH.

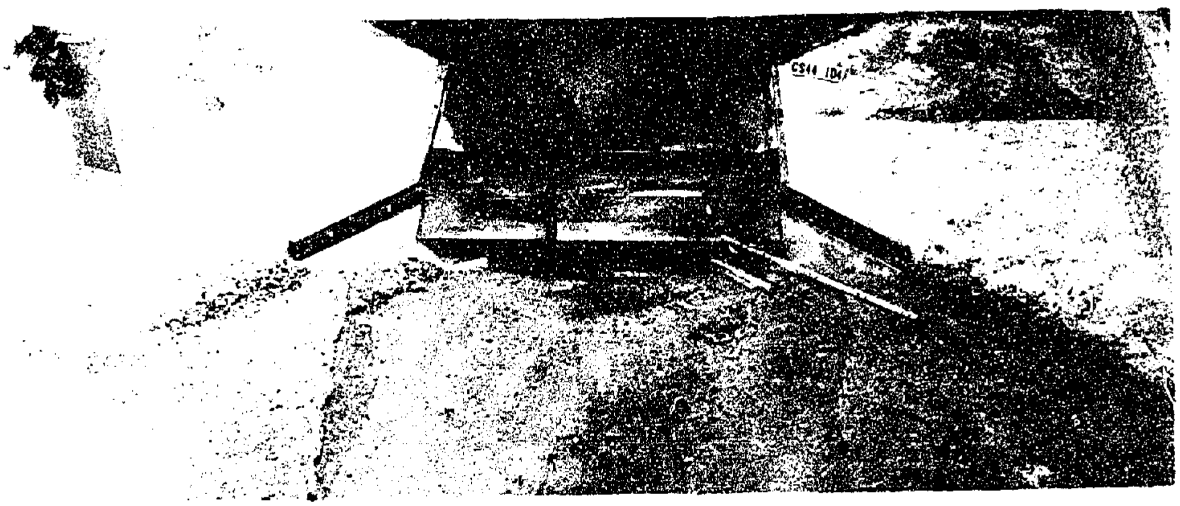
PROGRESS OF SCOUR BELOW CIRCULAR SIPHON DUE TO RETROGRESSION - SHEET 2 OF 2



A. MODEL BEFORE TEST - WASH SLOPE 0.028



B. FLOW CONDITIONS AT DESIGNED CAPACITY

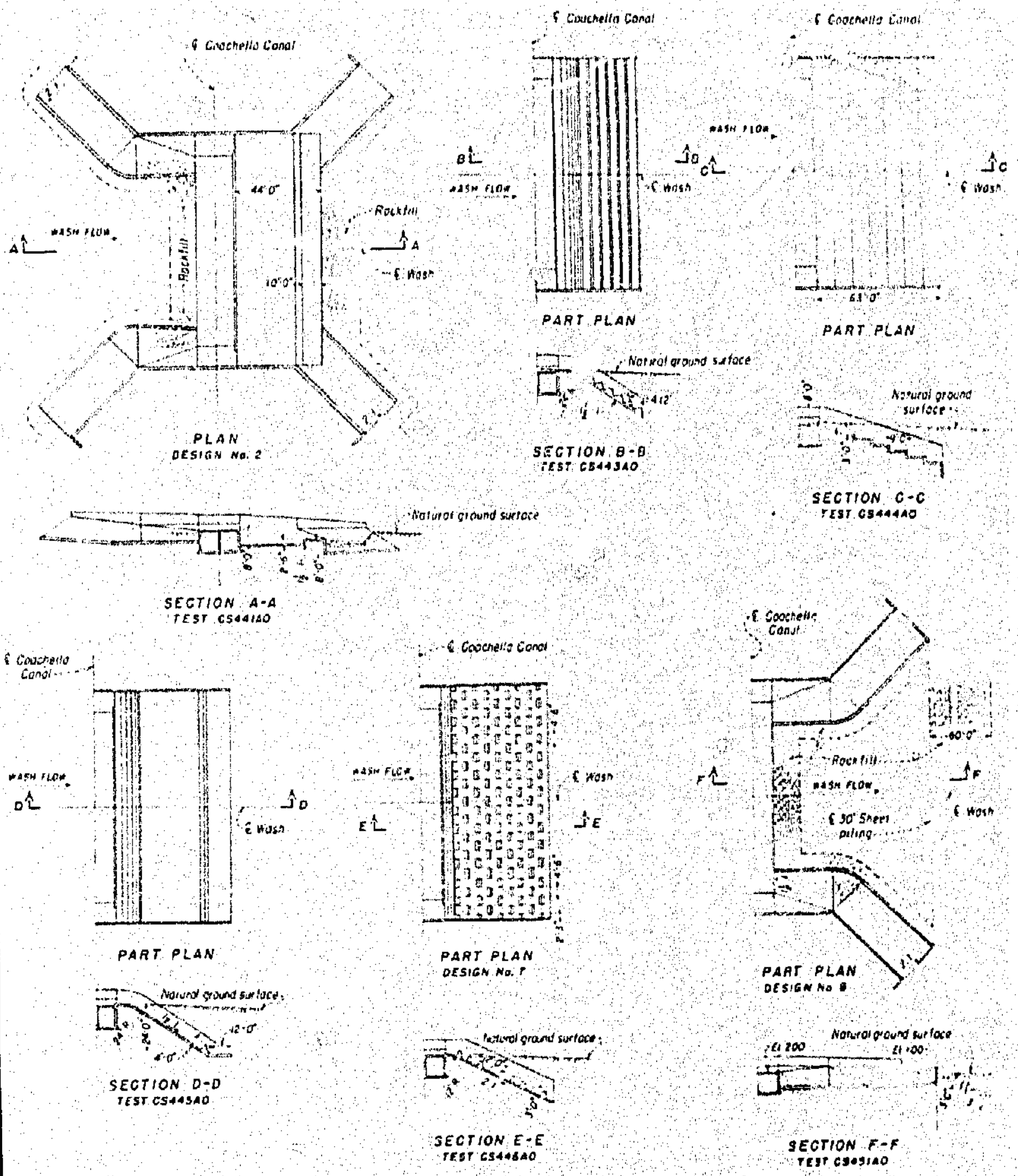


C. SCOUR AFTER 1-HOUR FLOW AT DESIGNED CAPACITY

RETROGRESSION EFFECT AT SINGLE POOL

HYD-111

FIGURE 21

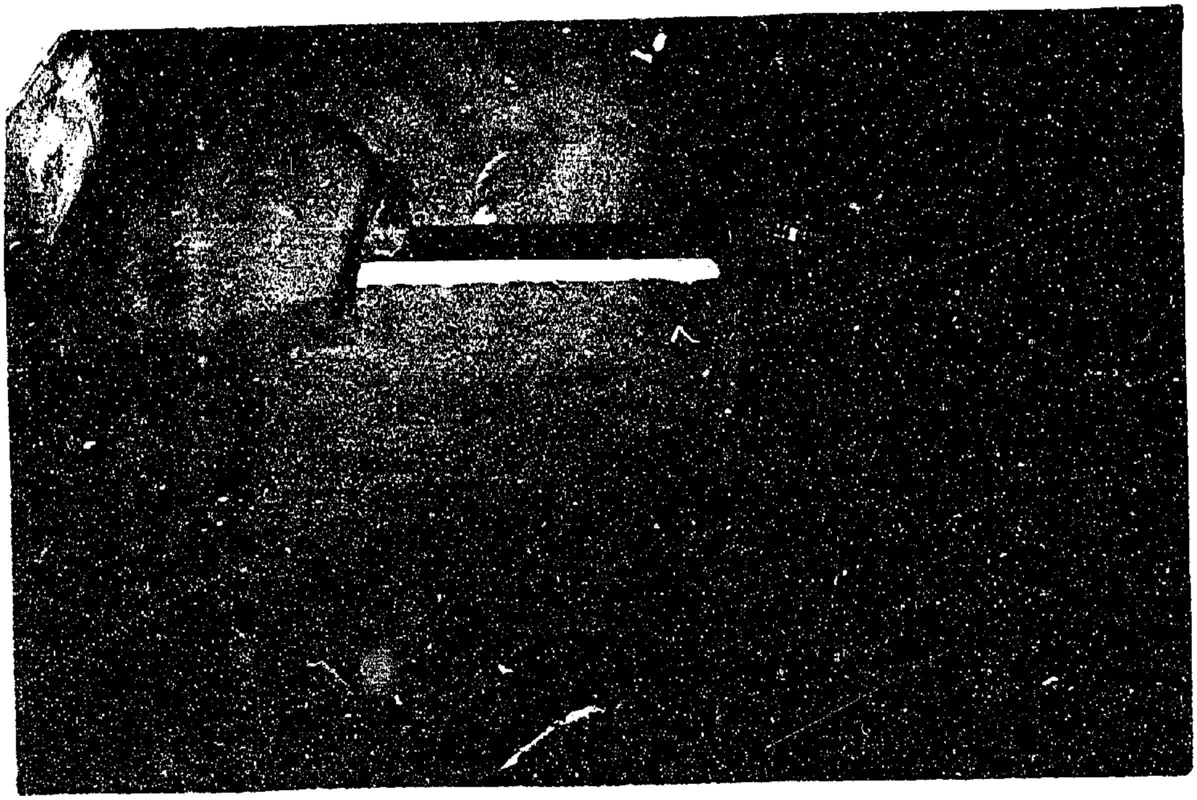


NOTES

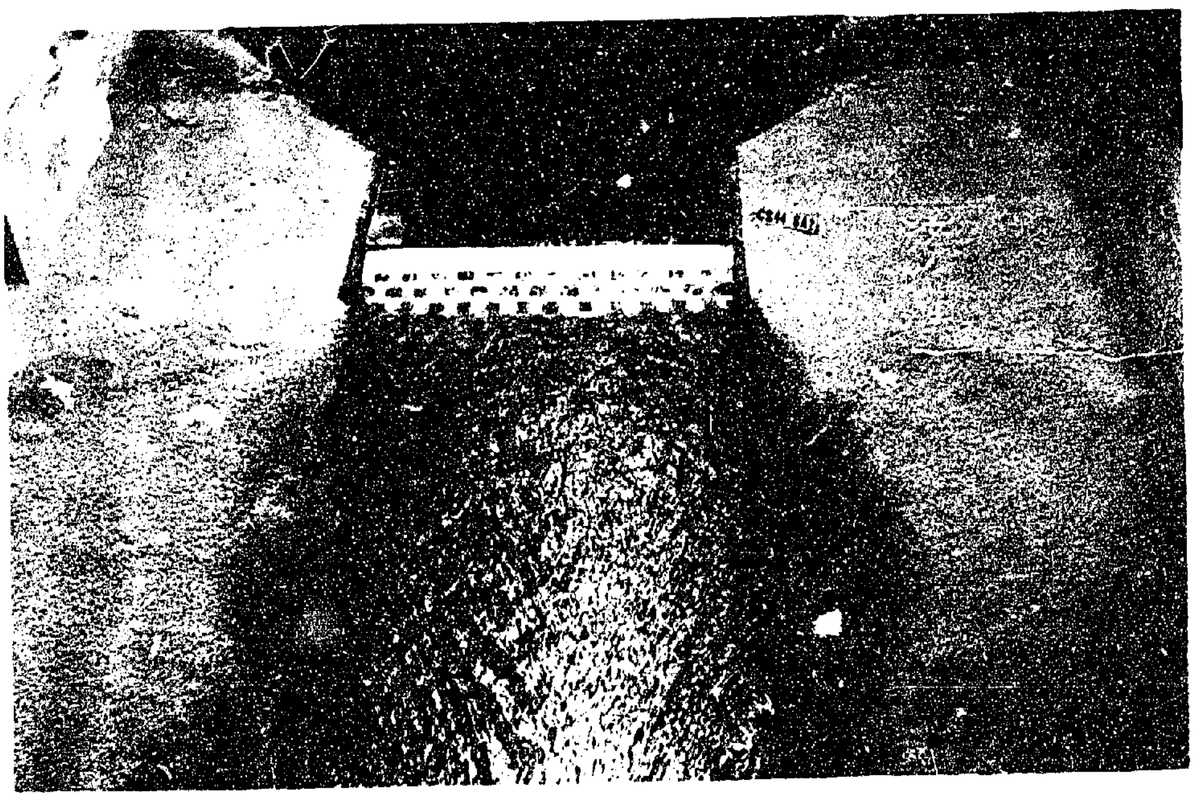
Dimensions and elevations apply to prototype
Model scale = 1:36
Wash overchute inlets similar for all tests
No 4 gravel used as rockfill, except in test CS451AO, where a mixture of No 8 to 3/4" gravel was used (Model size)
Design capacity 35 sec ft per foot of barrel curb

ALL-AMERICAN CANAL SYSTEM - CALIF
COACHELLA CANAL - STA 4580+50 TO 5896+20
WASH OVERCHUTES
STRUCTURES 35 TO 52 INCL
STUDIES OF DIFFERENT METHODS OF ENERGY DISSIPATION

12-8-41



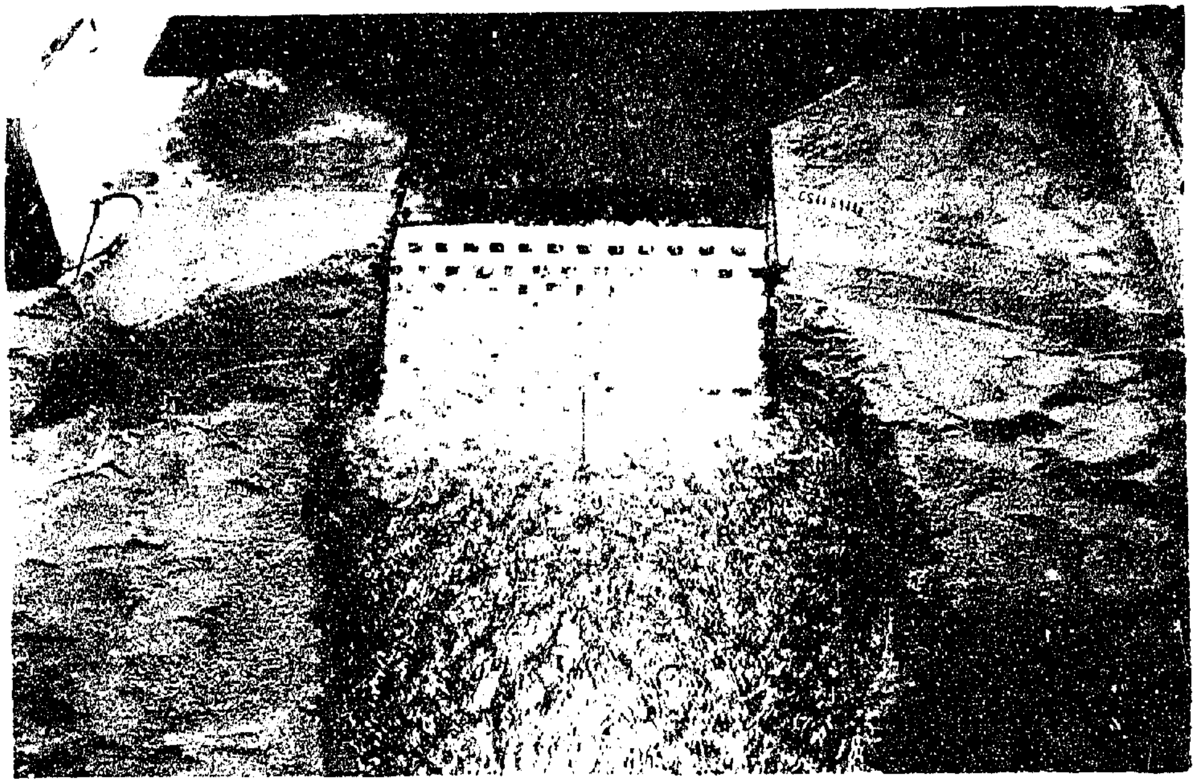
A. MODEL BEFORE TEST - WASH SLOPE 0.028



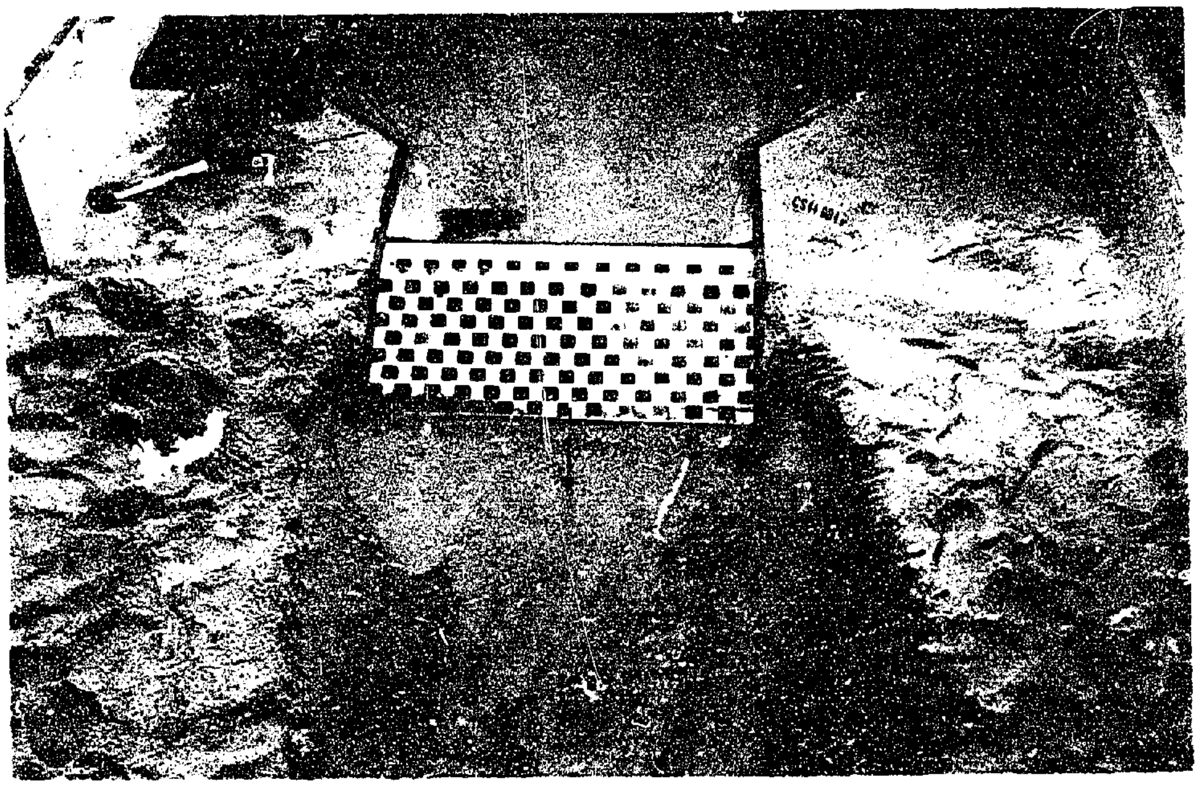
B. FLOW CONDITIONS AT 50% DESIGNED CAPACITY

PROGRESS OF RETROGRESSION WITH THE FINAL DESIGN - SHEET 1 OF 2

FIGURE 23



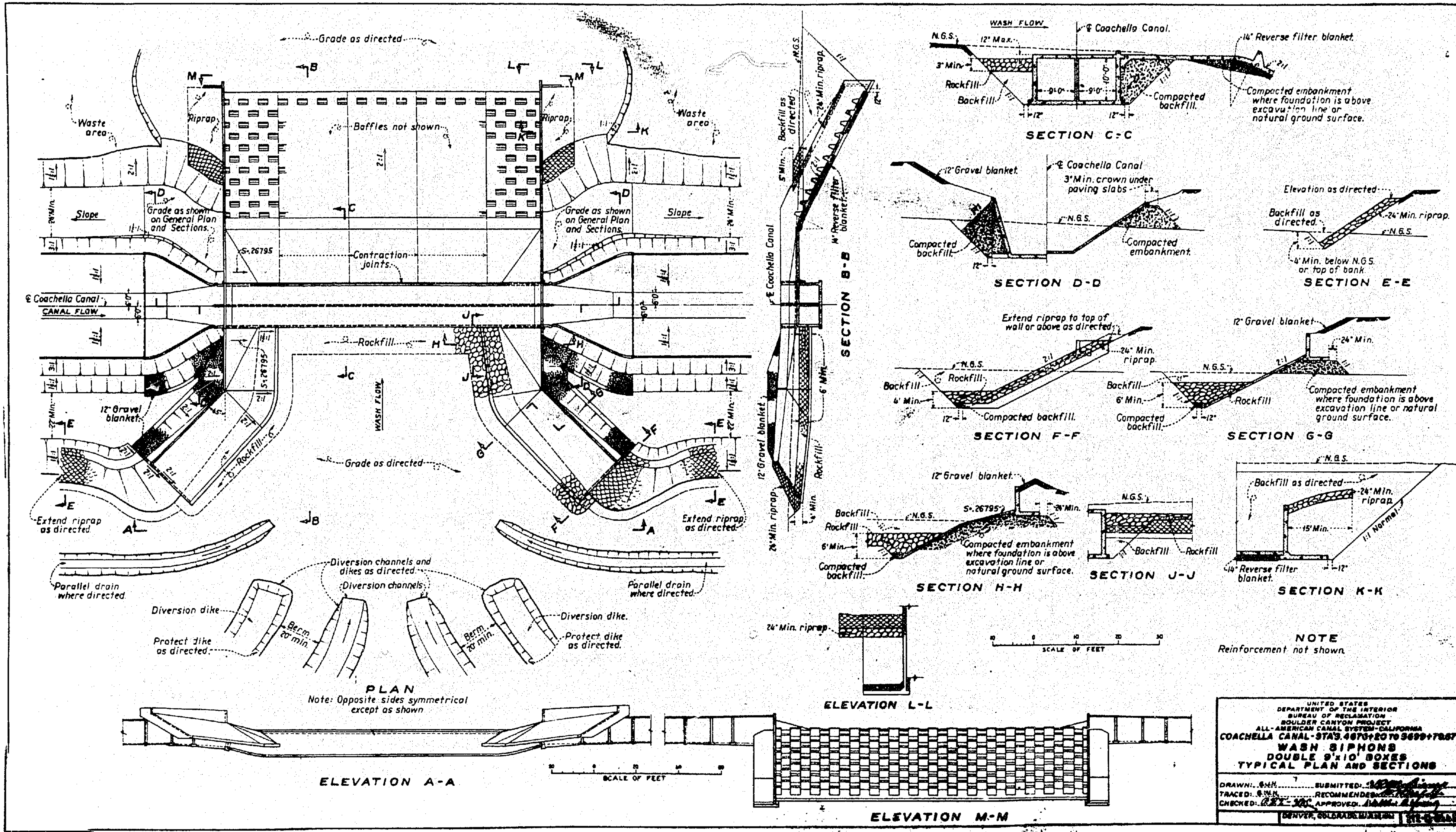
A. FLOW CONDITIONS AT DESIGNED CAPACITY



B. SCOUR AFTER 8½-HOUR FLOW AT DESIGNED CAPACITY

PROGRESS OF RETROGRESSION WITH THE FINAL DESIGN - SHEET 2 OF 2

FIGURE 24



PLAN
Note: Opposite sides symmetrical except as shown
Grade as directed
Baffles not shown
Waste area
Riprap
Slope
Grade as shown on General Plan and Sections.
Contraction joints.
S=.26795
℄ Coachella Canal
CANAL FLOW
Rockfill
WASH FLOW
12" Gravel blanket.
Extend riprap as directed
Parallel drain where directed.
Diversion channels and dikes as directed.
Diversion channels.
Diversion dike.
Protect dike as directed.
Berm 20' min.
ELEVATION A-A
SCALE OF FEET
SECTION B-B
Backfill as directed
24" Min. riprap.
14" Reverse filter blanket.
12" Gravel blanket.
WASH FLOW
℄ Coachella Canal.
N.G.S.
12" Max.
3" Min.
Rockfill
Backfill
Compacted backfill.
Compacted embankment where foundation is above excavation line or natural ground surface.
SECTION C-C
℄ Coachella Canal
3" Min. crown under paving slabs
Compacted embankment.
SECTION D-D
Elevation as directed
Backfill as directed.
24" Min. riprap.
4' Min. below N.G.S. or top of bank.
SECTION E-E
Extend riprap to top of wall or above as directed.
24" Min. riprap.
Compacted backfill.
SECTION F-F
Compacted embankment where foundation is above excavation line or natural ground surface.
SECTION G-G
SECTION H-H
SECTION J-J
Backfill as directed
15' Min.
1:1 Normal
14" Reverse filter blanket.
SECTION K-K
24" Min. riprap
ELEVATION L-L
SCALE OF FEET
NOTE
Reinforcement not shown.
ELEVATION M-M
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
ALL-AMERICAN CANAL SYSTEM-CALIFORNIA
COACHELLA CANAL-STA'S. 4673+20 TO 3699+78.67
WASH SIPHONS
DOUBLE 9'x10' BOXES
TYPICAL PLAN AND SECTIONS
DRAWN:
TRACED:
CHECKED:
SUBMITTED:
RECOMMENDED:
APPROVED:
DENVER, COLORADO

A. FLOW CONDITIONS AT 25% DESIGNED CAPACITY

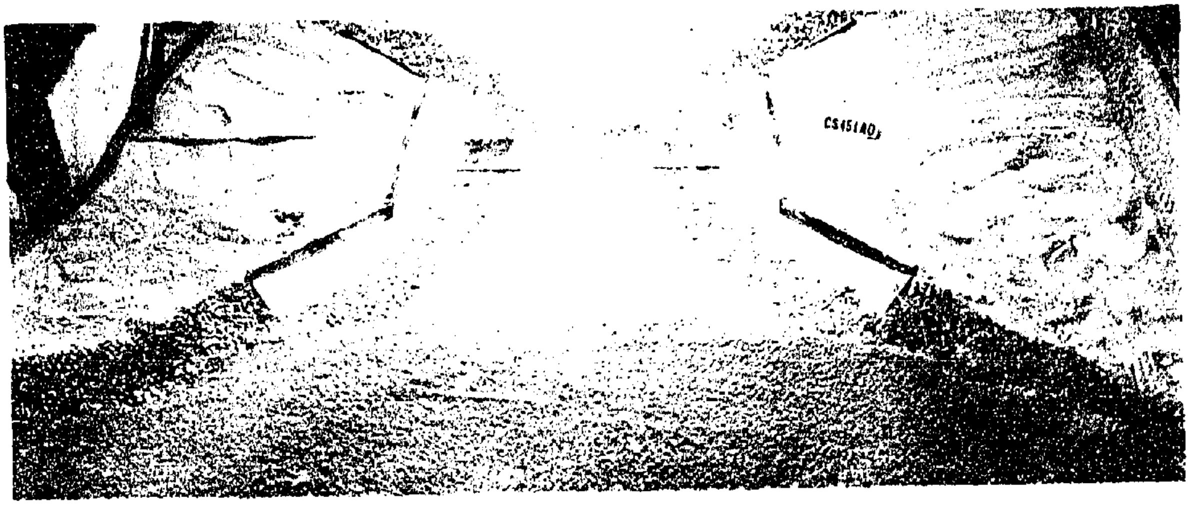
B. FLOW CONDITIONS AT 50% DESIGNED CAPACITY.

C. FLOW CONDITIONS AT 75% DESIGNED CAPACITY.

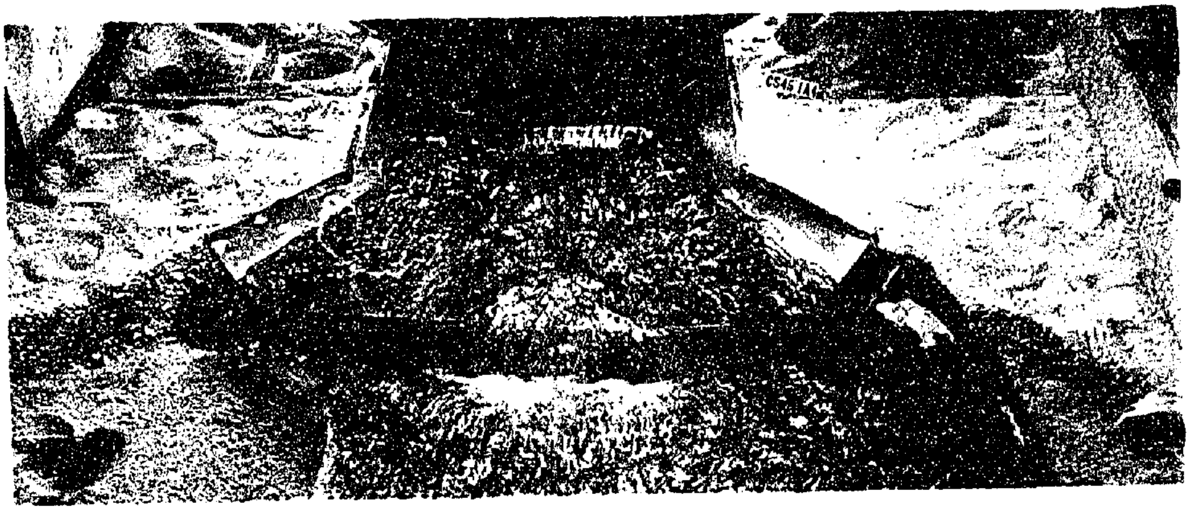
D. FLOW CONDITIONS AT 100% DESIGNED CAPACITY.

SECTIONAL MODEL OF FINAL DESIGN

FIGURE 26



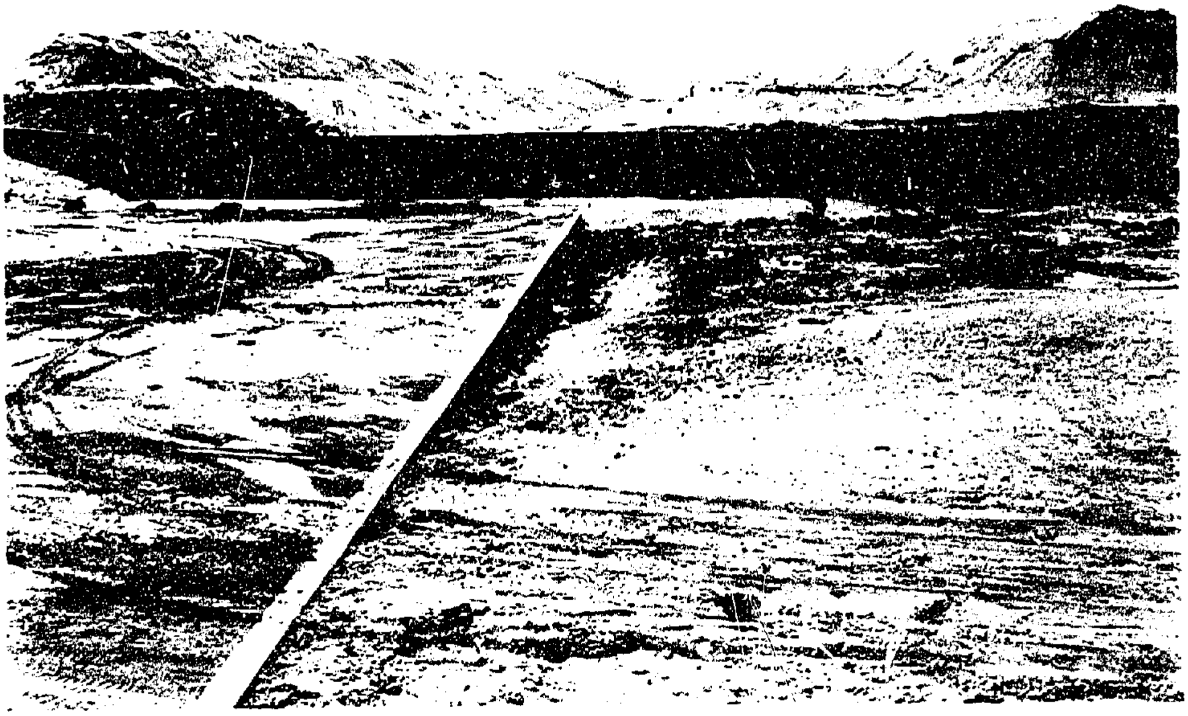
A. MODEL ARRANGEMENT BEFORE TEST - WASH SLOPE 0.040.



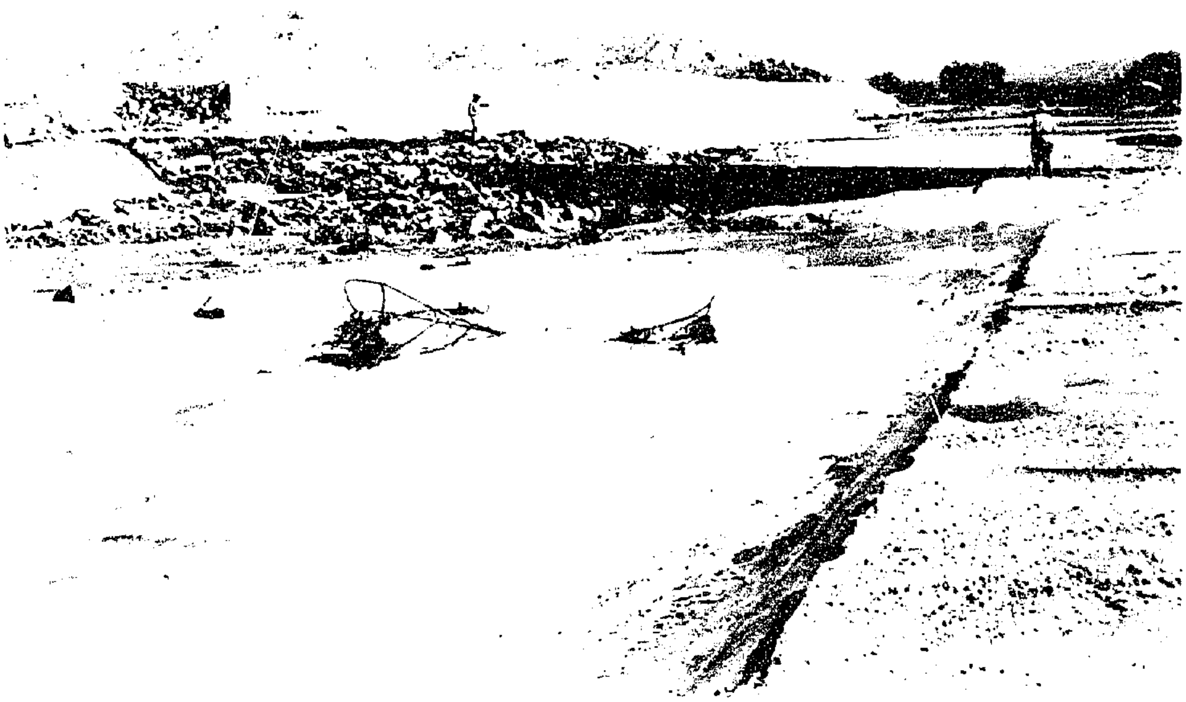
B. FLOW CONDITIONS AT DESIGNED CAPACITY

C. SCOUR AFTER 15-MINUTE FLOW AT DESIGNED CAPACITY.

SCOUR WITH GRAVEL BLANKET AND LONGER SHEET-PILES



A. SCOUR AT SIPHON FOR FLOW OF APPROXIMATELY [illegible] DESIGN CAPACITY - WASH SLOPE 0.013
WASH OVERCHUTE NO. [illegible]



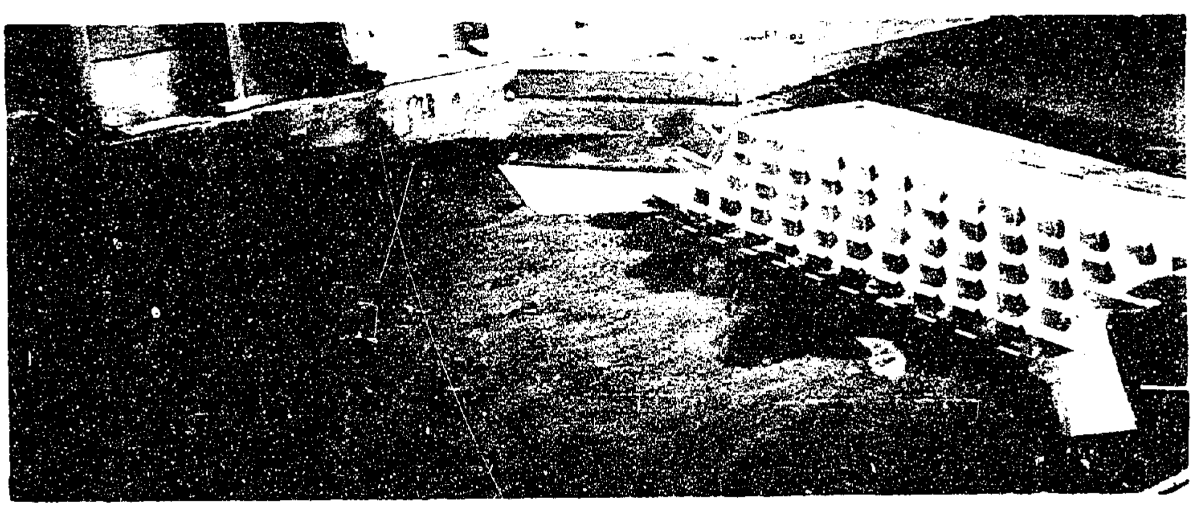
B. SCOUR AT SHEET PILING AFTER FLOW OF APPROXIMATELY 25% DESIGN CAPACITY.
WASH SLOPE 0.012. WASH OVERCHUTE NO. [illegible]

SCOUR AT PROTOTYPE OVERCHUTES

FIGURE 28

A. MODEL BEFORE TEST.

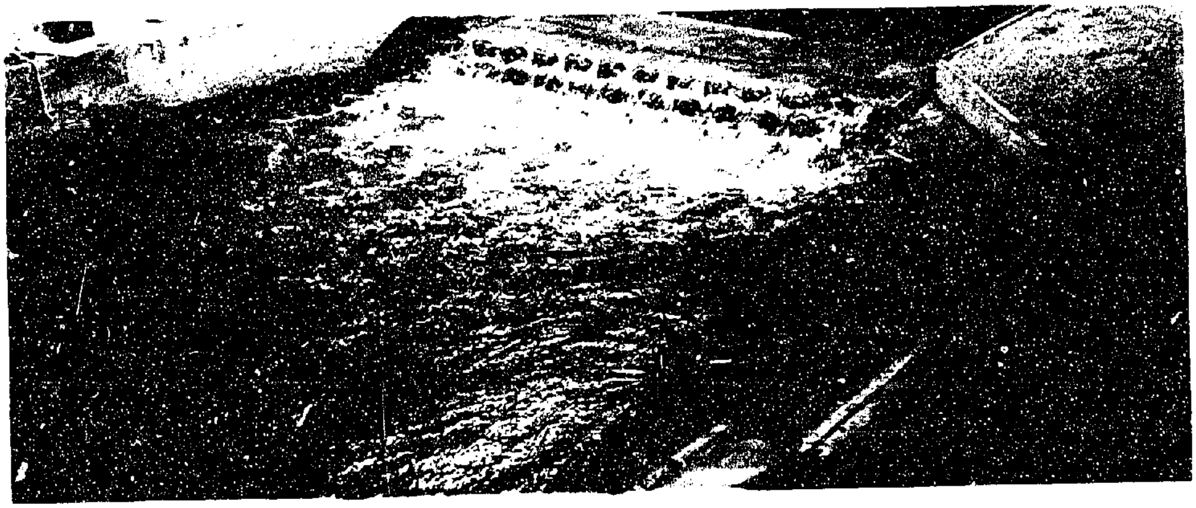
B. FLOW CONDITIONS AT DESIGNED CAPACITY.



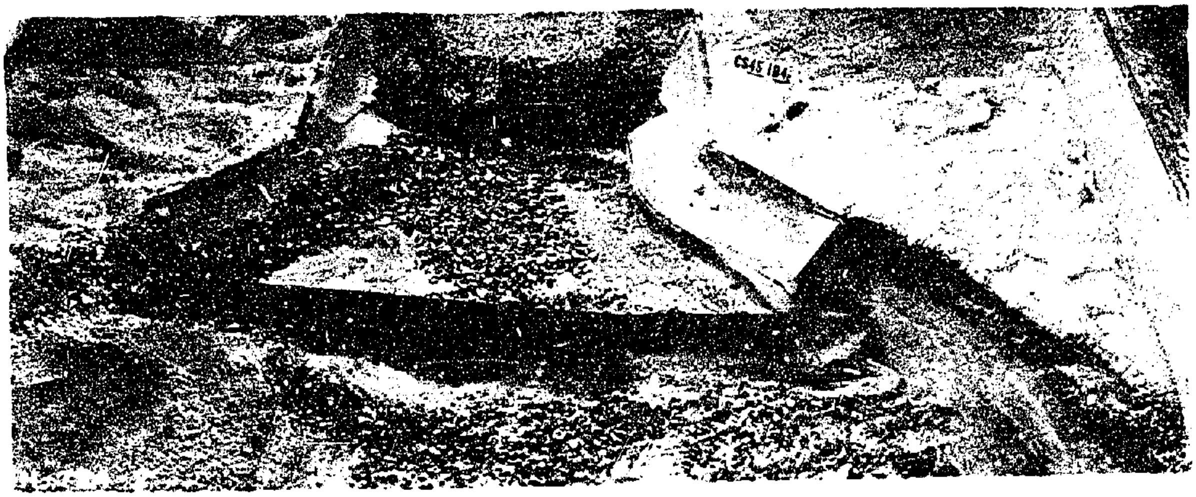
C. SCOUR AFTER 1-HOUR FLOW AT DESIGNED CAPACITY.

INITIAL REVISION TO COMPLETED OVERCHUTES

A. MODEL ARRANGEMENT BEFORE TEST.



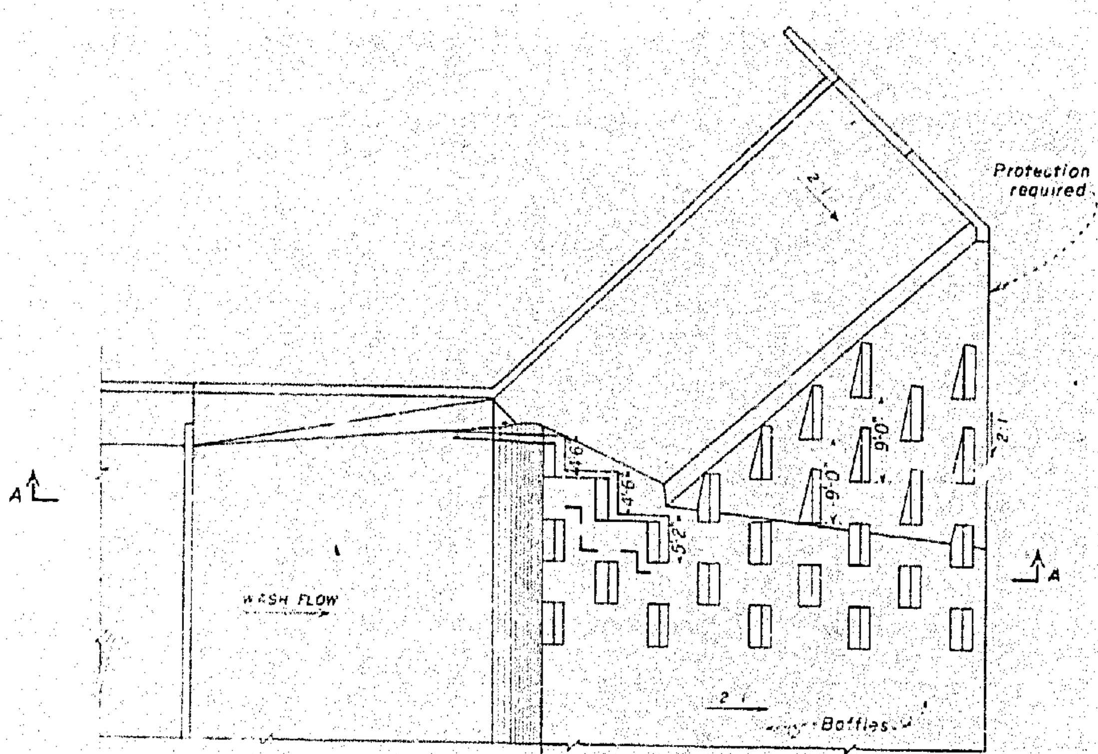
B. FLOW CONDITIONS AT DESIGNED CAPACITY.



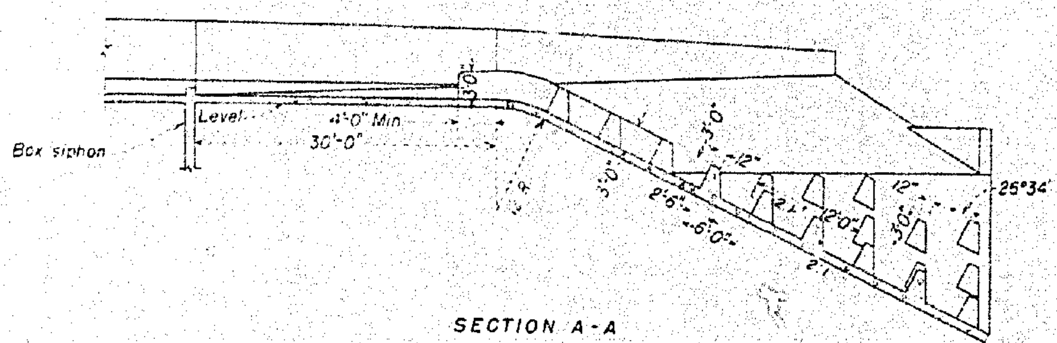
C. RECOMMENDED REVISION AT RIGHT - SCOUR AFTER 1-HOUR FLOW AT DESIGNED CAPACITY.

RECOMMENDED REVISION FOR COMPLETED OVERCHUTES SHOWING FLOW AND EROSION

FIGURE 30



PART PLAN



SECTION A-A

ALL-AMERICAN CANAL SYSTEM - CALIF.

COACHELLA CANAL - STA. 2078+31 TO 4561+57

WASH OVERCHUTES

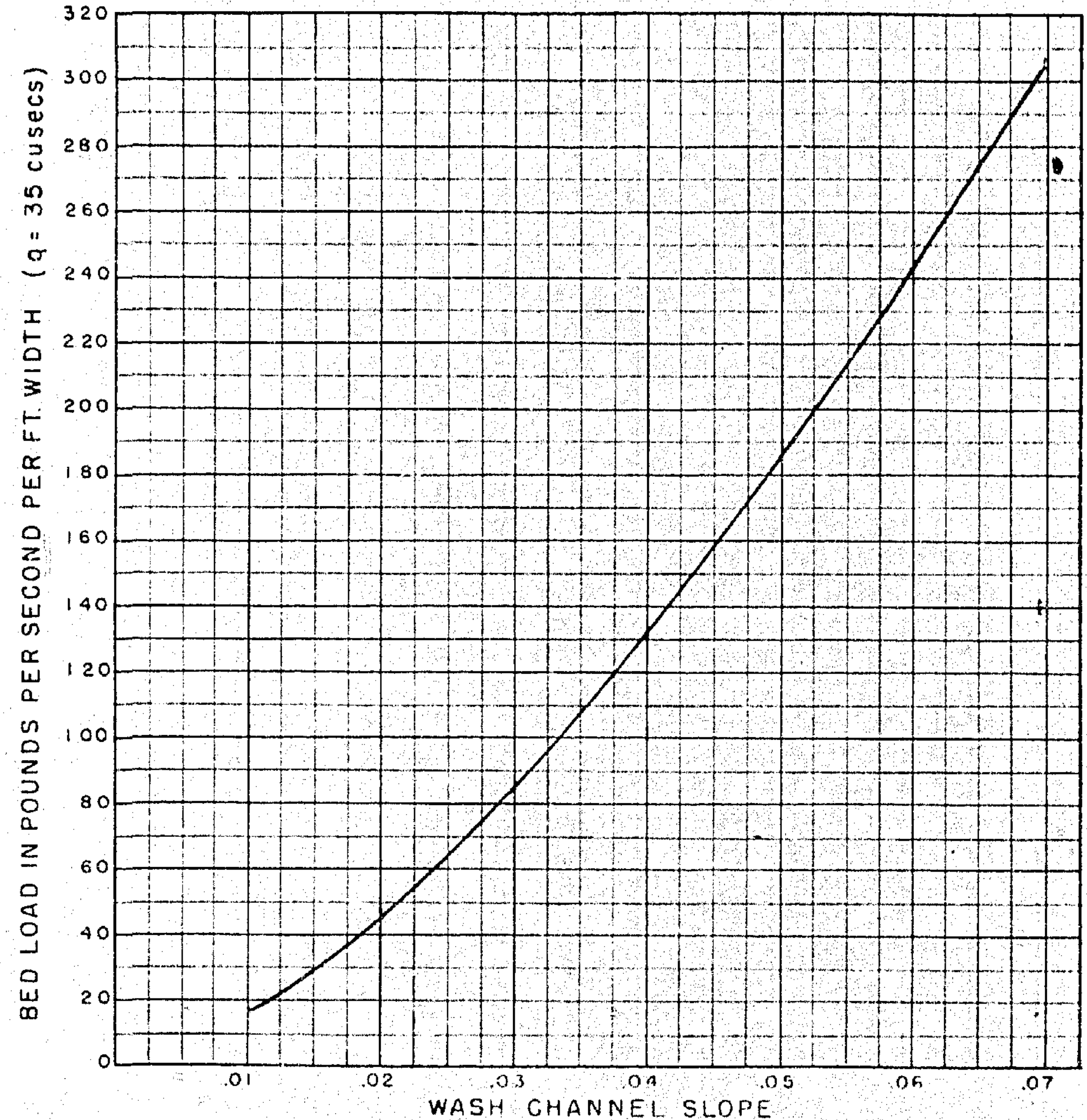
No. 1 TO 32 INCL.

DETAILS OF HYDRAULIC MODEL - REDESIGN No. 2

RECOMMENDED REVISION FOR COMPLETED OVERCHUTES

12-8-41

FIGURE 31



RELATION OF WASH CHANNEL SLOPE TO BED LOAD

F.L. 10-21-41