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FLOODPLAIN MANAGEMENT STUDIES OF THE SHINGLE CREEK BASIN

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The purpose of this report is to provide a technical summary of the studies carried out by the District from 1974 to 1979 in developing a management plan for the Shingle Creek Basin. The studies were conducted by the authors within the Water Resources Division, Resource Planning Department. In addition, supportive analyses were carried out during 1976-77 by Charles Tai, Technical Review Division, Resource Control Department.

This report was compiled and edited by Susan McCormick, Director, Land Resources Division, Resource Planning Department.

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DEVELOPMENT OF THE PLAN

The initial study in developing the proposed Management Plan for the Shingle Creek Basin was a report prepared by Reynolds, Smith and Hills[1] in 1974. This report of existing flooding conditions established the basic methodology for analyzing the hydrology and hydraulics of the basin.

As part of the Reynolds, Smith and Hills report, the 100-year flood hazard area along the main channel of Shingle Creek, from its outlet in Lake Tohopekaliga on the south to the Old Winter Garden Road in the City of Orlando on the north, was defined and recommendations for further research and interim management regulations proposed. The 100-year design flood was computed by means of the hydrometeorological approach, the computation of design hydrographs from rainfall for 54 sub-basins in the basin, and the flood routing of these hydrographs through the natural floodplain under existing and committed land use conditions as of September, 1974. Committed land use was defined as "that land for which there is a valid development contract between a government entity and a developer. A development contract shall be construed to include land for which there is an approved PUD final development plan, a subdivision plan, or a commercial development plan. Land zoned, but for which no development plans have been approved, will not be considered committed land."[2] Detailed documentation on the selection of the 100-year design storm, the development of design hydrographs for the 54 sub-basins from the unit hydrograph principle, the flood routing procedure, the backwater

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[1]Reynolds, Smith and Hills, Inventory of Existing Flood Conditions, Shingle Creek, a Preliminary Engineering Report prepared for Orange County, Osceola County, Central and Southern Florida Flood Control District, and Division of State Planning as a part of the Project to Prevent the Eutrophication of Lake Okeechobee, Sept., 1974.

[2]Ibid., p. A-6.

computation to determine the 100-year flood profile, and the delineation of the flood hazard area are all included in the report.

Utilizing the Reynolds, Smith and Hills report as the starting point, subsequent analyses were carried out by District staff in the following phases:

Phase I:

1. Determination of a new 100-year flood profile and flood hazard area for the Shingle Creek Basin, using the methodology established in the RS&H report and a revised 100-year flood stage on Lake Tohopekaliga.

2. Comparison of the 100-year flood hazard area and flood profile with the results presented in the RS&H report.

3. Delineation of the encroachment line on the floodplain area by allowing 0.5 ft. rise of water surface above the 100-year flood profile.

Phase II:

1. Development of the design water surface profile assuming improvements to the existing channel north of the Florida Turnpike.

2. Determination of the effect of improved channelization north of the Florida Turnpike on the entire Shingle Creek flood hazard area.

3. Delineation of the encroachment line on the floodplain area assuming 0.5 ft. rise above 100-year flood profile.

4. Determination of flood profiles and encroachment limits under proposed improvements, including bridges.

Phase III:

1. Development of a refined flood profile for the lower reach of Shingle Creek.

2. Evaluation of alternatives for detaining flood flows in the large cypress marsh north of the Orange-Osceola County line.

3. Development and evaluation of alternatives for lowering the flood profile in the reach below the SCL Railroad bridge.

PHASE I STUDY

Methodology

The information used in Phase I was basically available from RS&H. Cross-sections along the creek and the bridge sections were obtained from RS&H, with the roughness coefficient shown on each cross-section. The runoff distribution along each reach at various inflow points was available from the RS&H report. Therefore, Phase I primarily involved setting up an input data system in machine-readable form which met the requirements of the HEC-2 program. The HEC-2 program computes backwater profiles for river channels of any cross-section for either subcritical or supercritical flow conditions. The effects of various hydraulic structures such as bridges, culverts, weirs, embankments and dams may be used for various frequency floods for both natural and modified conditions. In setting up the input job stream for this study, the energy losses due to pier shape, friction, exit, contraction and expansion, etc. were considered by using special or normal bridge routine and roughness coefficient cards. A transition at every 50 ft. above or below a highway bridge was provided. The selection of a special or normal bridge routine was based on bridge cross-sectional information. If the bridge geometry could be approximated by a regular shape such as trapezoidal or rectangular, then a special bridge routine was used. Otherwise, a normal bridge routine was used for proper computation of energy losses through the bridge.

The delineation of the encroachment line on the floodplain areas was determined by using method 4 of the HEC-2 program. This method is based on an assumption that an equal loss of conveyance occurs on each side of the channel due to a 0.5 ft. rise of the backwater profile. Normally, if half of the loss cannot be obtained on one overbank, the difference will be made up, if possible, by the other overbank. No encroachment is allowed to fall within the main channel. The cross-sections were plotted on USGS Quad Sheets using a 5 ft. contour interval map, with the encroachment station for each cross-section available on the computer output. Using a 5 ft. contour interval map, the encroachment line was then linked together. The elevation shown on the cross-sectional map was also used to help judge the reliability of the contour map.

Assumptions

The following assumptions were made during the process of this study:

1. The bridges and waterways were not obstructed by trees, brush or other debris during the flood period.

2. All bridges over Shingle Creek were of sufficient strength to resist such a major flood.

3. A transition at 50 ft. above or below every bridge was either provided or based on the extension of the last cross-section available. A deep canal section below the channel bottom in the vicinity of the B-2, B-3, Bee Line connector, and Conway Road bridges was assumed to be maintained in its existing condition. However, such an assumption was not necessary under the Phase II study due to the channel improvements proposed for this portion of the creek.

4. Outflow from Turkey Lake, Clear Lake, and Lake Mann would not occur until sometime after 30 hours due to high tailwater conditions in Shingle Creek. Therefore, it was assumed that the outflow from these lakes would not contribute to the peak flow in Shingle Creek.

Results

1. Flood Profiles. As mentioned previously, the source of data

for the HEC-2 program was the same for both the RS&H and District studies except for some minor adjustment of cross-sections that did not extend completely across the floodplain and the transitional sections upstream and downstream of the highway bridges. In general, the results from both computations are very close except at the locations of highway bridge crossings. The special bridge and normal bridge routines that were used in this study depend on the available existing geometry of the bridge cross-sectional information. The special bridge routine was used on all bridges by RS&H. However, the differences between the two are within 0.5 ft. except for the State Road 530 bridge near Section 7. A transitional cross-section at a distance of 50 ft. from that bridge was assumed by using the same cross-section as Section 6 in this study, instead of the Section 7 used by RS&H, with the result that the net area under the bridge was 2,277 sq. ft. instead of the 1,462 sq. ft. used by RS&H. Due to these differences in bridge sections, a greater difference in the backwater computation resulted. However, the backwater profiles became close again for the following reaches, with a computed stage at the upper end of 97.90 ft. msl as compared to the 97.50 ft. computed by RS&H.

The backwater computation indicates that a number of bridges will be submerged during a 100-year flood. These bridges are at Taft Vineland Road, Sand Lake Road, Americana Boulevard, and McLeod Road (State Road 446). A number of bridges will be partially submerged. They are the Old Tampa Highway, SCL Railroad, Powerline Road, Road "E" Bridge (B-2), Road "D" Bridge (B-3), Florida Turnpike, Oak Ridge Road and Abilene Trail. In other words, twelve of the nineteen existing bridges crossing Shingle Creek will be either completely or partially submerged under this 100-year flood. Those bridges with timber piles, such as State Road 531, Old Tampa Highway, and the SCL Railroad, may not be able to resist such a large flood since the flow velocity under this bridges is much greater than a permissible velocity of 2.5 ft/sec. The mean velocity in the floodway areas would not generally exceed 2.5 ft/sec.; however, the mean velocity in the main channel slightly exceeded 2.5 ft/sec. in the reach near Oak Ridge Bridge (i.e., Stations 1207+20 through 1231+20).

2. Flood Hazard Area and Encroachment Line. The outline station of the flood stage along Shingle Creek and the encroachment station on every cross-section are available from the computer output. They were also plotted on the USGS Quadrangle sheets along with the location of each cross-section. With the aid of contours and elevations from the cross-sections these points were linked together. However, a field trip was taken to assist in the process of delineation of the encroachment line at the following locations (where information wasn't available on either the contour maps or the cross-sections): (a) the area near Lake Tohopekaliga, (b) the swampy area north of the SCL Railroad and west of the Kissimmee Airport, (c) the swampy area south of Taft Vineland Road, and (d) the urban area along the existing channel from Lake Clear to Shingle Creek. (The Reedy Creek Swamp area was excluded in this study.)

The outline limit of the floodplain area is approximately the same as the flood hazard area shown by RS&H except in the following locations: (a) the area near the Oak Ridge bridge where a portion of the currently developed area would be within the floodplain, particularly the area east of Shingle Creek, (b) the area east of Interstate 4 bridge crossing, and (c) the area near the WLOF Radio Towers. (This may be caused by different assumptions in the extension of cross-sections.)

Generally, the encroachment line falls within the 100-year flood hazard area identified by RS&H. That portion between the encroachment lines along both sides of the main channel is called the designated floodway. This is the channel of the water course where the greatest velocities and depths of water occur. That portion of the adjoining floodplain between the designated floodway and the natural outline of the selected flood is referred to as the floodway fringe. This portion of land can be considered for development either by filling to a required elevation or by applying other flood proofing measures. As a result of delineation of the encroachment line a substantial floodway fringe is available. The residential areas which were developed prior to 1970 and which are currently inside the 100-year flood hazard area are primarily in this floodway fringe or outside the designated floodway limits.

PHASE II STUDY

Methodology

Phase II was approached in a slightly different fashion from Phase I. Multiple surface profiles for various discharges, in order to establish stage-discharge relationships and stage-storage relationships at every reach, were computed and used to develop storage-discharge relationships for the proposed channel improvements north of the Turnpike. The following steps were taken to establish such relationships:

1. The extent of channelization north of the Turnpike was based on a 30% Standard Project Flood design section which approximates a 10-year frequency flood for the area.

2. The design sections under the highway bridges were established by assuming two ft. of excavation at the center portion of the bridge section, with a 2 horizontal to 1 vertical side slope to the embankment of the bridge. It was felt that this improvement for the bridge section would not cause any damage to the substructures of the existing bridges.

3. The storage-discharge relationship was developed by running

the HEC-2 program to obtain multiple surface profiles for Q = 500, 1000, 1500, 2000, 3000, ..., 9000 cfs at different initial stages between 54.0 and 57.5 ft. msl. The amount of discharge at the middle and upper reach was varied to a combination of 3000, 2000, 1000, 500 and 300 cfs. The purpose of this was to estimate the variable backwater effects, channel storage, and return of overbank flow on these reaches.

4. A flood routing computation was performed for all 21 reaches using the storage-discharge relationship described above. The routing program developed was based on the modified Puls method for a particular river reach.

$$\frac{(I_1 + I_2)}{2} \Delta t - \frac{(O_1 + O_2)}{2} \Delta t = S_2 - S_1 = \Delta S \qquad (1)$$

where,

Δt = time interval $t_2 - t_1$

I_1 = inflow at time 1 (rate)

I_2 = inflow at time 2

O_1 = outflow at time 1

O_2 = outflow at time 2

S_1 = storage at time 1 (volume)

S_2 = storage at time 2

ΔS = change in volume of storage for the time interval

This equation can be further rewritten in the following manner:

$$\frac{I_1 + I_2}{2} + \frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_2}{\Delta t} + \frac{O_2}{2} \qquad (2)$$

Knowing the relationship between discharge and storage for each reach, the developed routing program based on Equation (2) was applied to route the flood, combined with the developed design flood hydrograph from each subdrainage basin given in the RS&H report. These local design flood hydrographs were added as local inflow to the reach before flood routing, or to the outflow from the reach after flood routing.

5. The flows used to determine the maximum water surface elevation along the creek were determined by using the peak discharge from the inflow-outflow routed design flood hydrograph discussed in Item 4.

6. Discharge figures close to the previous routed peak discharge at each reach along the creek were used to run multiple surface profiles. Then the procedures discussed in Item 3 were used to refine the outflow-storage relationship for each reach. A new routing process was done in the same manner as described in Item 4 to compute a new set of design flood distributions along the creek.

7. The HEC-2 program, utilizing this new set of design flood distributions at each reach, was then used to compute the backwater profiles. A new 100-year flood profile and delineation of flood hazard areas were determined in the same fashion as described in Phase 1.

Results

1. Storage-Discharge Relationship. This relationship was developed from a multiple run of surface profiles through the HEC-2 program. The discharges used were 500, 1000, 1500, 2000, 3000, up to 9000 cfs with different initial stages such as 54.0, 55.0, 56.0, and 57.5 ft. msl at Lake Tohopekaliga. It was discovered later that the backwater stages for different initial stages of the same discharge were about the same. However, the various discharges were tested to estimate the storages due to variable backwater effects, channel storage, local inflows, etc. A runoff distribution was first estimated, then a refined storage-discharge relationship was developed through the process described in the general procedures. The storage of the swamp area in Reach #10B was added to the storage-stage relationship obtained from the computer output since no survey information was available. Thus, the computation done by RS&H was used to adjust the

storage-stage relationship and storage-discharge relationship.

2. Design Discharge Distribution. The above storage-discharge relationship for each reach was used in the developed flood routing program which was also combined with the developed design flood hydrograph from each subdrainage basin provided by RS&H. The peak discharge was selected as the design discharge distribution for the backwater computation. The results of peak discharge and time of peaks are not significantly different from the results obtained by RS&H for the lower reaches of the creek (Reaches #7 through #16). However, a slightly higher discharge of about 300 to 500 cfs resulted from the reaches with the channel improvement (i.e. the channelized portion north of the Turnpike) as compared to the RS&H results. There is no significant difference in time to peak; with few exceptions, they generally agree within approximately an hour.

3. Flood Profiles. The maximum water surface elevation was determined by computing water surface curves using the computed peak discharge in each reach. This provided a more conservative backwater surface profile. The peak discharges used in this study are comparably higher than those used by RS&H. However, the resulting flood profile for the lower reaches is about the same.

The profile for the design channel portion is comparably much lower than the natural profile (Phase I Study), particularly in the reach between Oak Ridge Road and Interstate 4. The profile for the reach between Orlando Vineland Road and the upper end of the creek is about 1.9 ft. lower than the natural profile. However, there is a two ft. drop of stage through the bridge crossing at the Orlando Vineland Road. This backwater resulted from the restriction of an inadequate bridge opening at this location. Therefore, the assumption of two ft. of excavation under the existing bridge section may not be a good assumption at this bridge.

The mean velocity in the main channel is also shown at each station. The velocity near most bridge sections slightly exceeds 2.5 ft/sec. except at Station 1231+20 which is 3.69 ft/sec. as compared to 5.7 ft/sec. with the natural channel. Since the design section was based on a one in ten year frequency, the velocity under the 100-year frequency exceeded the permissible velocity for the channel. The velocities in the overbank areas are much less than 2.5 ft/sec.

4. Delineation of Flood Areas. The outline limit of the flood area was determined in the same way as described previously. The flooded areas for the lower reaches are approximately the same as in the Phase I Study. Generally, for the reaches north of the Florida Turnpike, the outline limit falls within the encroachment line that resulted from the Phase I Study and a substantial portion of the floodplain will be outside the floodway limit. For the reach between Oak Ridge Road and Orlando Vineland Road, the floodway will be generally confined to within 100 ft. of either side of the main channel. But the lower land areas of the reach between Orlando Vineland Road and Section 39 (Station 1445+00) will be flooded due to the backwater effect that results from the restriction of the Orlando Vineland Road Bridge. For the rest of the two upper reaches which are already urbanized, the flood stage will be confined to within the design channel. However, the depth of flooding is considerably less than that which resulted under natural channel conditions. The flooded area would be reduced even further by increasing the flow cross-section under the Orlando Vineland Bridge.

DISCUSSION OF PHASE I AND II STUDIES

Generally, there are two approaches in floodplain management; one is through the enactment of floodplain regulations; the other is through the provision of flood control works. The studies discussed herein were directed toward both these approaches in an effort to provide in-depth information on the flood stage and flood hazard areas within the Shingle Creek Basin and, also, to evaluate the feasibility of flood control works by considering improvements to the existing Shingle Creek channel north of the Florida Turnpike. As defined previously, the flood hazard area is the floodplain along the main channel of Shingle Creek which would be inundated by the 100-year frequency design storm, assuming existing and committed land use. This 100-year flood is approximately equal to the Corps of Engineers Intermediate Regional Flood.

The results of the District studies can be summarized as follows:

1. The flood profiles for the 100-year storm under natural channel conditions generally agree with the results computed by RS&H. The outline limits of the flood hazard area also agree very closely with the RS&H results except for the three locations identified earlier. Taking a conservative approach, the maximum outer limits of both studies were used to establish the outline limit of the 100-year flood hazard area.

2. Application of the designated floodway results in a substantial reduction of the flooded area. The developed areas inside the flood hazard zone prior to 1970 are mostly located outside of the designated floodway. However, portions of presently urbanized areas are located within the encroachment line.

3. The flood hazard area for the Reedy Creek Basin and the area adjacent to the existing canal from Lake Clear to Shingle Creek can be delineated by using the flood stage computed in this study and the discharge developed by RS&H for the appropriate sub-basin; however, additional cross-sections will be required to define the flooded area.

4. The 100-year flood stage in Lake Tohopekaliga may be slightly

higher than 57.5 ft. It is believed that the effect of the increased stage would be felt only downstream of the SCL Railroad due to the large amount of storage available northwest of the SCL Railroad.

5. The designated floodway is very critical. The only types of land use that can be permitted within the floodway are those which do not require filling or impede or obstruct flows in any way. Failure to protect this zone will cause massive flooding upstream and downstream.

6. The delineation of the encroachment line on the floodplain area by allowing 0.5 ft. rise above 100-year flood profile under the channelized condition is not included in this report. However, the information is available in computer output form.

7. Several areas, such as Lake Mann, Turkey Lake, Westside Manor, etc., did not contribute to the peak flood stages due to a time lag. Therefore, the flood hazard area has not been delineated. More field work will be required before the flood hazard area for these areas can be delineated. Any future improvements to the outfall system of these lakes may reduce the time lag and cause these areas to contribute to the flood peak.

8. The results of the improved channel north of Florida Turnpike can be briefly detailed in the following paragraphs:

a. The peak discharge for the lower reaches (south of the Florida Turnpike) would not be significantly increased with the improved flow conditions in the upper reach. This is probably due to the large amount of storage available in the cypress marsh area immediately downstream of Taft Vineland Road.

b. The flood stages and flood hazard areas in the lower reaches would not be changed as a result of improvement in flow conditions north of the Florida Turnpike.

c. The flood stages and flooded areas in the channelized reach would be greatly reduced in depth and area.

d. The flooded areas in the channelized reach would fall within the encroachment line that results from allowing a 0.5 ft. rise of water surface elevation above the 100-year natural flood profile.

e. The assumption of 2 ft. of excavation for a design channel under the Orlando Vineland Bridge is not adequate, since approximately two ft. of backwater would result north of the bridge crossing due to restriction of flow by that bridge opening. Therefore, the design channel section used in this study can be improved, and the flood stages north of Orlando Vineland Road can be lowered, if the design section under the existing bridge is improved.

f. In addition to the Orlando Vineland Bridge, improvements to the following bridges should be investigated in the formation of a flood-plain management plan:

(1) The present section of State Road 600 is inadequate for the flow generated by the one in ten year storm proposed in the Corps of Engineers' "Survey-Review Report on Central and Southern Florida Project - Shingle Creek Basin". The bridge has adequate length but the substructure needs investigation.

(2) State Road 531 is an old concrete bridge with a restricted opening. It may possibly fail if there is a large build-up of debris during the flood.

(3) The Old Tampa Highway bridge will probably fail as it did in 1960.

(4) The Conroy Road bridge is a new concrete bridge. The high velocities can be controlled by use of rip-rap.

(5) There are other bridges that will be submerged under the 100-year flood. It will be necessary to investigate their structural

stability.

As a result of the discussion of these conclusions with the local governments concerned, the following suggestions were considered for inclusion in the management plan:

1. Improvements to the bridges at State Road 600 and State Road 531.

2. Removal or replacement of the Old Tampa Bridge.

3. Excavation to the design bottom elevation of the proposed channel section improvements at the bridges north of the Florida Turnpike.

4. Construction of a water control structure at approximately Station 1207+00 in order to prevent over-drainage and control erosion. A control stage upstream at 86 ft. msl was suggested with the proposed structure designed to pass the 100-year storm with a 0.5 ft. head loss above the 100-year flood profile. Since the 100-year flood profile allows a 0.5 ft. increase of water surface elevation above the natural flood, the result is a one ft. rise above the natural flood profile.

5. Redesign of the design bottom elevation from 85 ft. msl to 83 ft. msl in the reach north of Station 1427+00 as an erosion control measure.

Incorporating the improvements suggested above, a new 100-year flood profile, flood hazard area, and designated floodway, allowing for a 0.5 ft. increase in flood stage, were computed using the same methods outlined previously. Generally, the two profiles agreed closely except in the following reaches:

1. Between State Road 600 and State Road 531 there was no significant difference in flood stage; however, the velocity was reduced to less than $\pm$ 2.5 ft/sec. instead of the 5.2 and 10.1 ft/sec. velocities that had existed before the improvements.

2. Between the SCL Railroad and Station 454+00 the new profile varied from 1 ft. below the old profile at the SCL Railroad to zero

reduction at Station 454+00. The reduction in the width of the floodplain varied from 300 ft. at the SCL Railroad to zero at Station 454+00 while the encroachment line showed no significant reduction. To be on the conservative side, it was suggested that the slight reduction in the floodplain be ignored in the water management plan.

3. Between Station 1207+00 and Station 1285+10 (Interstate 4) the profile for the improved condition was up to 1.0 ft. higher than the previous profile. This was due to the head loss through the proposed water control structure. However, the floodplain area in this reach did not increase due to the fact that this reach has been developed and the ground elevation has been filled to above flood stage. The flood was mostly confined to the channel except for the reach between Oak Ridge Road and the proposed water control structure.

4. Between Station 1285+00 (Interstate 4) and Station 1373+85 (McLeod Road), the profile for the improved condition is much lower (over 2 ft.) than the previous profile. Thus, the flood flow is confined to the main channel and a substantial portion of the existing floodplain would be available for development.

5. Between Station 1373+85 and the northern end of Shingle Creek, the flood stage ranged from 94.50 ft. msl at McLeod Road to 95.61 ft. msl at the northern end of the creek. The lands in the reach from Station 1374+60 to Station 1427+00 are low lying, with existing elevation mostly below 95.00 ft. msl; therefore, some inundation in this reach can be expected. However, the existing developed area will be excluded from the floodplain. For the developed area north of Station 1445+00, the flood will be contained in the main channel.

PHASE III STUDY

The analyses conducted through early 1977 indicated that flood stages and flood hazard areas in the lower reaches would not be increased as a result of the proposed improvements north of the Florida Turnpike. Although the routings carried out up to that time indicated very limited flooding problems to existing development, continued local concerns that future development in Orange County might increase flooding in the downstream reaches of Shingle Creek in Osceola County necessitated a reexamination of the potential problem. The basic data and assumptions upon which the prior analyses were based were reviewed and field reconnaissance investigations, together with topographic surveys, were carried out. For these purposes, it was assumed that future development of uncommitted lands would occur in a land use pattern similar to that of existing and committed land uses.

From the revised routings, which incorporated more accurate and up-to-date tributary basin areas and conveyance cross-section areas, it was apparent that a significant flooding problem for existing development existed in the area downstream of the SCL Railroad bridge.

Initial proposals to eliminate this problem centered on attenuating upstream flows from Orange County. To this end, a tie-back levee and weir were envisioned to detail flood flows in the large cypress marsh immediately north of the Orange-Osceola County line. A proposed low head weir (0.5 ft.) was located at Station 747+00, immediately north of the county line. This weir created only very marginal storage increases over and above the excellent existing detention features of the natural marsh. A higher head weir (1.0 to 1.3 ft.) was later evaluated in terms of a length of both 250 ft. and 180 ft. and a crest elevation of 74 ft. msl. Figures 1 and 2 show the stage-storage curves for each of the proposed weir lengths, based on the discharge equation for a free fall rectangular weir where,

$$Q_1 = CLH^{3/2} \qquad (1)$$

and for the submerged flow

$$Q = Q_1 \left(1 - \frac{H_o}{H}^{3/2}\right)^{0.385} \qquad (2)$$

in which H_o is the total head at the tailwater.

Under these conditions the 100-year storm created a peak discharge of 5440 cfs and flood stages of 78.1 ft. msl and 78.4 ft. msl for the two weir lengths, respectively. This flood stage is 1.0 ft. to 1.3 ft. above the 100-year natural flood stage at the location within the proposed weir.

In evaluating the impacts of this proposed weir, flooding conditions resulting from the 100, 25 and 10 year frequency storms were determined for the following cases:

Case 1. Future full development without an on-stream retention structure and without local runoff restrictions.

Case 2. Future full development with an on-stream retention structure.

Case 3. Future full development with local runoff restrictions for Orange County and without an on-stream retention structure.

Local runoff restrictions refer to a detention storage area, controlled by structure measures such as a weir, to store the first inch of storm runoff. The subsequent runoff is allowed to overflow a free fall rectangular weir designed so that the flow at a 3 ft. head is equivalent to the peak discharge resulting from a 25-year frequency storm under existing and committed land use as of September 1974. This assumption enabled the District to incorporate the local runoff restriction into our flood routing model developed for the basin.

The peak flow rate at the head and foot of each channel reach along with the time to peak are tabulated in Table 1. Table 2 shows the 100-year flood discharge along Shingle Creek for all reaches. It can be seen that there was not a significant difference in discharge between any of the cases, with the greatest difference being approximately 500 cfs in the lower reaches. The change in time to peak was also insignificant. Therefore, it is apparent that the effect of the proposed weir (with 1.0 ft. to 1.3 ft. head above natural profile) on discharge is negligible, and that the existing detention features of the natural cypress marsh are excellent. This can be further illustrated by the inflow hydrograph and outflow hydrographs from the marsh area under the 100, 25, and 10 year storm frequencies and without the on-stream retention structure (see Figures 3, 4 and 5). Comparing Cases 1 and 2 to existing conditions, the increased peak discharge is generally much greater for the reaches above the marsh area immediately north of the county line, while little difference is apparent for the reaches below the marsh. Comparing Case 3 to the existing condition, the peak discharge is slightly less than the existing condition. Therefore, the substantial effect of local runoff restrictions in reducing peak discharge is obvious.

The index of flood duration is defined as the duration of flood flow within 90% of the peak discharge. Flood duration is an important factor in evaluating flood damage. Table 3 shows the 100-year flood duration index at the foot of each channel reach along Shingle Creek for the three cases as well as for existing conditions. In general, future development in the potentially developable area increases not only the peak discharge but also the inundation period. The local runoff restrictions not only reduce the peak rate of flood but also result in no significant increase in flood duration except in reaches 12 through 14. Those reaches above the marsh did not show any significant difference.

The flood stage along Shingle Creek is listed in Table 4 and Figures 6 and 7. It was developed using the Corps of Engineers HEC-2 program and the same cross-sectional data along Shingle Creek discussed in Phases I and II. The channelization plan north of the Florida Turnpike and some bridge improvements were also considered in this study. For those reaches below the cypress marsh, there were no significant differences in flood stages for Cases 1 and 2 except for the reach immediately below the on-stream retention structure. In the upper reaches of Shingle Creek, however, there were some significant increases in flood stage, such as the reach from Orlando Vineland Road north to the end of the basin, where increases ranged from 0.5 ft. up to 1.80 ft. Thus, for the 100-year storm, some areas would be inundated under future full development conditions.

The small difference between Case 3 and existing conditions indicates that the effect of local runoff restrictions on the flood stage along Shingle Creek is very significant, particularly in those reaches north of the cypress marsh. Flood stages, assuming local runoff restrictions, would be either the same as existing conditions or slightly lower. Backwater profiles for the 25 and 10 year frequency storm under assumed future development are shown in Figures 8, 9 and 10.

Therefore, flooding conditions under future full development in the upper reaches of Shingle Creek would be relieved were local runoff restrictions implemented. In addition, due to the excellent existing detention features of the natural marsh, runoff from Orange County would not increase the flood hazard in the lower reaches of the Shingle Creek basin.

Although the previous study had illustrated the effectiveness of the natural marsh versus the on-stream retention structure, the District made a more complete topographic survey of the large cypress marsh north of the Osceola County line during September, 1978 in order to better define its water storage capabilities. With this information, the effects of a higher head weir (3 ft.) with a tie-back levee to detain flood flows in the large cypress marsh was also investigated. The crest elevation for this higher head weir was 77.0 ft. msl and the proposed length was 200 ft. Stage-storage curves for this weir are shown in Figures 11 and 12.

The 100-year flood was then routed through this proposed facility together with the enlarged drainage area for the Reedy Creek swamp. Figure 13 shows the entire Shingle Creek basin with the Reedy Creek swamp included. It became apparent that the largest component of the flood peak passing through the concerned area was a result of the Reedy Creek swamp runoff rather than the Orange County runoff. Examination of flood hydrographs from USGS gaging stations at the U.S. 192 and Old Tampa Highway bridges indicated that a relatively quick runoff hydrograph (time of concentration less than 10 hours) is generated between the two stations before the Orange County flows arrive. This can be seen on the hydrographs shown on Figure 14 which are the result of hydrologic routings for reaches 14 and 15. The high weir resulted in a reduction of peak flows at State Road 531 from approximately 8000 to 6800 cfs which was not enough to prevent most of the damages. In fact, total retainage of the flow from Orange County will not reduce this flow peak further. Therefore, it can be concluded that:

1. The existing marsh behaves as an efficient buffer between the counties,

2. Although Orange County runoff will extend the period of inundation of flooding in Osceola County, the peak flow and stage is

largely generated locally.

To reduce the flood stage in the area below the SCL Railroad to below damage levels, there are two alternatives which may be considered. The first would be to reduce the amount of flow while the second would be to increase the floodway conveyance. To accomplish the first would require detention of runoff in the Reedy Creek Swamp drainage area, which is now primarily pasture with a developed secondary drainage system. If a significant portion of this were detained, it would then have to be released at a later time where it would combine with upstream flows resulting in the same flooding problem which currently exists, but at a later time. Therefore, in order to resolve the current problem, it appears that the remaining alternative of increased conveyance must be considered.

The reach considered for this conveyance improvement is between U.S. 17-92 and the Old Tampa Highway. Two separate schemes have been considered. First, the conventional approach, which would widen the existing natural creek. Second, because of obvious esthetic and potential environmental problems with the first approach, a floodway bypass was considered which would only carry water during periods of flood conditions, leaving the natural creek essentially unchanged.

As noted previously in the discussion of the Phase I and II studies, the 100-year stage on Lake Tohopekaliga could be higher than 57.5 ft. msl. Therefore, an initial stage of 58.5 ft. msl was used, and several backwater profiles were computed by using HEC-2 program. The results are shown on Tables 5 through 9 and Figure 15. The backwater profiles for the initial stages of 57.5 and 58.5 ft. in Lake Tohopekaliga did not show a significant difference for the reach above State Road 531 and were about 0.15 ft. higher for the reach between U.S. 17-92 and State Road 531.

However, the 100-year flood profile elevation in the concerned reach is over 63.0 ft. msl with the inclusion of inflow from the Reedy Creek swamp area. To evaluate actual damage levels, a District survey crew was sent to the area in December, 1975 and finished floor elevations of numerous apartments and homes in the area were determined. In the Aldersgate community, the floor elevation for 198 apartments ranged from 61.9 ft. to 62.2 ft. msl. Therefore, it is necessary to lower the profile to about 61.0 ft. msl elevation in this area in order to prevent flooding during the 100-year storm.

For the conventional approach the creek would need to be widened to a bottom width of 150 ft. which would yield a total top width of about 235 ft. This assumes that the natural bottom elevation of 45 ft. msl would be maintained. As the creek varies from about 40 to 70 ft. in width, this widening would require about 175 ft. of bank removal. The existing State Road 531 bridge is a source of a large energy loss and would thus require rebuilding to an accomodating width of 300 ft. This alternative would eliminate at least half of the existing shoreline vegetation, notably large overhanging oak and cypress trees. It is estimated to cost $1,317,000 at a minimum.

The second approach would be to clear a floodway bypass around to the north of the creek through existing pasture lands (see Figure 16). Though this would be 500 ft. wide, the depth would be shallow with a bottom elevation of 55.5 ft. msl. Very gradual (10 horizontal to one vertical) side slopes to existing land would insure continued utility of the land. A bridge crossing State Road 531 would also have to be built for this scheme. With a bottom elevation of 55.5 ft. (Lake Tohopekaliga is regulated to a maximum 55.5 ft.), flow in the bypass could only begin to occur when flow in the natural creek exceeded about 500 cfs. It would, therefore, be dry except during brief storm periods of less than annual expectation. At the 100-year flood peak, the flow would be divided with approximately 6000 cfs discharging through the bypass and 2000 cfs remaining in the existing channel. These flows would join again on the upstream side of the U.S. 17-92 crossing. The results of this approach are shown on Figure 17. Estimated costs for the floodway bypass are $1,062,500.

Based on a comparative analysis of the preliminary costs and the environmental impacts associated with each of the alternatives, the floodway bypass was recommended.

DISCUSSION OF PHASE III STUDIES

Due to the long and narrow shape of the drainage basin, the flood flow in the main stream is compounded by local runoff inflows. Whether the peak flow responds to local runoff or to the flow in the main stream is generally determined by the relative size of the immediate sub-basins. Although there is a great difference in local hydrographs resulting from various conditions, they are generally attenuated by the Shingle Creek floodplain.

The results of the Phase III studies can be briefly stated as follows:

1. Assuming no local runoff restrictions, future development in potentially developable areas will increase the flood stage along the main channel to a certain degree depending upon the amount of urbanization. For reaches near the northern end of Shingle Creek, with the proposed channelization plan, the flood stage may increase up to 1.8 ft. above 100-year flood profile.

2. The creation of local runoff restrictions in potentially

developable areas can reduce the flood stage which would result from such development.

3. The on-stream retention structure proposed for the outlet to the large cypress marsh north of Orange-Osceola County line is ineffective in reducing flood stages downstream.

4. The existing large cypress marsh behaves as an efficient buffer between the two counties as far as flood flows from Orange County are concerned.

5. Although Orange County runoff will extend the period of inundation of flooding in Osceola County, the peak flow and stage is largely generated locally (along the reaches in Osceola County).

6. The 100-year flood stage in Lake Tohopekaliga has a very limited effect on the backwater except at the lowest reach near the lake.

7. The creation of a bypass floodway at the reach of Shingle Creek between U.S. 17-92 and Old Tampa Highway will be the most effective way to relieve the existing flooding problem at Aldersgate community due to existing development in the flood hazard area of Shingle Creek itself.

REFERENCES

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5. Miller, C. R. and Veissman, W. Jr., "Runoff Volumes for Small Urban Watersheds", Water Resources Research, Vol. 2 No. 2, April 1972.

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TABLE 1 PEAK DISCHARGE UNDER FUTURE FULL DEVELOPMENT

Reach Number	With On Stream Retention: Peak Discharge @ Head (cfs)	Peak Discharge @ Foot (cfs)	Time to Peak @ Head (Hours)	Time to Peak @ Foot (Hours)	Without On Stream Retention: Peak Discharge @ Head (cfs)	Peak Discharge @ Foot (cfs)	Time to Peak @ Head (Hours)	Time to Peak @ Foot (Hours)
1	780	510	10.0	11.5	780	510	10.0	11.5
2A	4370	2440	10.0	12.0	4370	2440	10.0	12.0
2B	5710	3480	10.0	13.5	5710	3480	10.0	13.5
3	4690	3610	10.5	15.0	4690	3610	10.5	15.0
4	3730	3730	14.5	14.0	3730	3730	14.5	14.0
5A	4740	4710	10.5	11.0	4740	4710	10.5	12.0
5B	4920	4710	10.5	12.0	4920	4710	10.5	12.0
6A	6350	5930	11.0	12.5	6350	5930	11.0	12.5
6B	8180	5960	11.0	15.0	8180	5960	11.0	15.0
7	6610	5700	14.0	17.5	6610	5700	14.0	17.5
8A	5910	5890	16.5	17.5	5910	5890	16.5	17.5
8B	6280	6110	15.5	19.0	6280	6110	15.5	19.0
9	7900	7030	11.5	15.0	7900	7030	11.5	15.0
10A	7930	7870	13.5	14.5	7930	7870	13.5	14.5
10B	10220	5540	12.5	27.0	10220	5540	12.5	27.0
11A	5660	5440	25.5	30.5				
11B	5440	5430	30.5	31.5	5660	5520	25.5	29.0
12	5870	5630	17.5	24.0	6520	6020	15.5	23.0
13	5730	5680	24.0	26.5	6140	6080	22.0	24.5
14	5730	5570	25.5	32.5	6140	5890	24.5	29.5
15	7760	7720	23.5	25.0	8220	8170	23.5	25.0
16	7830	7760	24.5	26.0	8280	8170	24.5	26.5

TABLE 2 100 YEAR FLOOD PEAK DISCHARGE ALONG SHINGLE CREEK (AT FOOT OF THE REACH)

Reach No.	Existing Condition[1]	Case 1	Case 2	Case 3
1	450	510	510	350
2A	2,230	2,440	2,440	2,260
2B	1,910	3,480	3,480	2,360
3	2,340	3,610	3,610	2,650
4	2,550	3,730	3,730	2,760
5A	3,710	4,710	4,710	3,430
5B	3,720	4,710	4,710	3,530
6A	5,200	5,930	5,930	4,710
6B	5,340	5,960	5,960	4,880
7	5,490	5,700	5,700	4,930
8A	6,180	5,900	5,900	5,330
8B	6,390	6,110	6,110	5,570
9	7,630	7,030	7,030	6,390
10A	8,700	7,910	7,910	6,870
10B	5,950	5,540	5,540	4,980
11A	5,590	5,520	5,440	
11B	5,590	5,520	5,430 (5360)[2]	4,870
12	5,630	6,020	5,630 (5420)	4,880
13	5,580	6,080	5,690 (5380)	4,880
14	5,420	5,890	5,570 (5350)	4,880
15	7,420	8,170	7,720 (7580)	7,080
16	7,530	8,170	7,760 (7640)	7,140

Note: [1]Refers to existing and committed land use as of Sept. 1974.

[2]Peak discharge with 180 ft. long rectangular weir.

Case 1: Future full development without on-stream retention structure.
Case 2: Future full development with on-stream retention structure (weir length = 250 ft.).
Case 3: Future full development with local runoff restrictions for Orange County and without on-stream retention structure.

TABLE 3 100 YEAR FLOOD INDEX FLOODING DURATION* ALONG SHINGLE CREEK (AT FOOT OF THE REACH)

Reach No.	Existing Condition	Case 1	Case 2	Case 3
1	2.5	2.5	2.5	3.5
2A	4.0	4.0	4.0	4.0
2B	5.5	6.5	6.0	6.0
3	8.5	8.0	8.0	8.5
4	6.0	9.0	9.0	10.0
5A	3.0	3.0	3.0	5.5
5B	3.0	4.5	4.5	5.5
6A	3.5	4.0	4.0	4.0
6B	4.0	5.0	5.0	5.5
7	5.5	7.5	7.0	6.5
8A	5.5	7.5	7.5	6.5
8B	6.0	9.0	9.0	7.0
9	6.5	10.0	9.5	7.0
10A	6.5	6.0	6.0	7.5
10B	8.5	15.5	15.5	15.0
11A			17.0	
11B	10.0	16.5	17.0	15.5
12	11.5	18.5	22.5	27.0
13	12.0	17.0	22.0	26.5
14	13.5	19.0	22.5	27.0
15	9.0	11.0	12.5	13.0
16	13.5	12.5	14.0	14.5

Case 1: Future Full Development Without On-Stream Retention Structure

Case 2: Future Full Development With On-Stream Retention Structure

Case 3: Future Full Development With Local Runoff Restrictions for Orange County and Without On-Stream Retention Structure.

* Index of flooding duration is defined as the duration of flood flow within 90% of the peak discharge in hours.

TABLE 4 SHINGLE CREEK 100 YEAR FLOOD STAGE
STAGE, FT. ABOVE MSL

Station	Identification	Existing[1] Condition	Case 1	Case 2	Case 3
293+00	Lake Toho.	57.50	57.50	57.50	57.50
310+50	U.S. 17, S.R. 600	59.42	60.02	59.75	59.35
345+20	S.R. 531	59.89	60.16	60.02	59.85
388+00	Old Tampa Hwy.	62.75	63.32	63.15	62.92
389+30	SCL R.R.	64.28	65.09	64.74	64.32
537+17	U.S. 192, S.R. 530	71.04	71.23	71.08	70.76
817+70	Power Line Road	79.09	79.11	79.32	78.82
903+15	Rd. "E" Bridge B-2	82.57	81.95	81.93	81.60
956+19	Taft Vine Lane Rd.	83.02	82.47	82.46	82.20
975+58	Rd. "D", Bridge B-3	83.13	82.56	82.55	82.25
999+29	Beeline Connector	83.27	82.69	82.68	82.41
1113+55	Sand Lake Road	87.81	88.12	88.12	87.76
1159+35	Florida's Turnpike	88.02	88.35	88.35	87.94
1214+30	Oak Ridge Road	90.18	90.62	90.62	89.95
1269+00	Tropical Drive	91.45	92.49	92.49	91.26
1285+80	I-4, S.R. 400	92.30	93.75	93.75	92.22
1293+50	Orlando, Vineland Rd.	92.33	94.11	94.11	92.33
1326+25	Conroy Road	92.65	94.36	94.36	92.62
1373+85	Mcleod Dr., SR. 446	93.50	96.22	96.22	94.00
1395+30	Abilene Trail	94.60	96.43	96.43	94.00
1485+30	Northern End	95.61	96.90	96.90	94.97

Note: [1]Refers to existing and committed land use as of September 1974.

Case 1: Future Development Without On-Stream Retention Structure.

Case 2: Future Full Development With On-Stream Retention Structure.

Case 3: Future Full Development With Local Runoff-Restrictions for Orange County and Without On-Stream Retention Structure.

TABLE 5 100 YEAR FLOOD PROFILE WITH EXISTING LAND USE - NO IMPROVEMENT IN BRIDGE SECTION - WITH ON-STREAM RETENTION (HIGHER HEAD WEIR) IN SHINGLE CREEK - INITIAL STAGE - 58.50 FT. MSL.

Station ft.	Q cfs	Water Elev. ft. msl.
293+00	7060	58.50
309+84	7060	58.59
310+50	7060	59.02
310+51	7060	59.28
310+65	7060	59.39
310+80	7060	59.79
320+30	7060	59.83
341+00	7060	61.09
344+58	7060	61.22
345+08	7060	61.23
345+20	7060	61.24
345+72	7060	61.25
361+00	6760	61.92
388+00	6760	62.77
388+50	6760	62.63
388+51	6760	62.36
388+70	6760	63.10
389+00	6760	63.96
389+30	6760	64.08
389+90	6760	64.19
390+40	6760	64.31
402+70	2890	64.46
454+00	2890	64.71
507+00	3120	69.06
536+67	3120	69.83
537+50	3120	69.84
538+00	3120	69.84
539+50	3120	70.04
580+00	3120	72.89
603+00	3580	73.06
675+50	3580	73.83
708+50	1975	74.41
747+00	1975	74.95

TABLE 6 100 YEAR FLOOD PROFILE WITH EXISTING LAND USE AND NO IMPROVEMENT IN BRIDGE SECTIONS. REEDY CREEK SWAMP INFLOW INCLUDED WITH INITIAL STAGE OF 57.50 FT. MSL AT LAKE TOHOPEKALIGA.

Station ft.	Q cfs	Water Elev. ft. msl.
293+00	8070	57.50
309+84	8070	57.86
310+50	8070	59.02
310+65	8070	59.29
310+80	8070	59.92
320+30	8070	59.96
341+00	8070	61.37
344+58	8070	61.50
345+08	8070	61.52
345+20	8070	62.47
345+72	8070	62.48
361+00	8030	62.89
388+00	8030	63.56
388+50	8030	63.38
388+70	8030	64.39
389+00	8030	65.18
389+30	8030	65.18
389+90	8030	65.23
390+40	6010	65.37
402+70	6010	65.47
454+00	6010	66.00
507+00	6110	70.40
536+67	6110	71.21
537+50	6110	71.23
538+00	6110	71.23
539+50	6110	71.47
580+00	6110	74.44
603+00	6270	74.66
675+50	6270	75.44
708+50	5590	76.12
747+00	5590	77.11
781+00	5590	78.75
817+70	7880	99.20
831+05	7930	79.34
858+40	7930	79.57
885+52	7930	80.10
888+00	7930	80.42
902+65	7930	81.58
903+15	7460	81.58
903+55	7460	81.59
904+10	7460	81.59
912+60	7460	81.68
918+60	7460	81.87
955+70	7460	82.75
956+20	7460	82.75

TABLE 6 (Continued)

956+50	7460	82.80
956+80	6180	82.80
975+09	6180	82.83
975+59	6180	82.83
975+70	6180	82.85
976+00	6180	82.86
998+80	6180	82.96
999+30	6180	82.93
1000+70	6390	82.94
1001+70	6390	83.08
1028+00	5490	84.39
1056+45	5340	85.71
1063+00	5340	86.32
1081+40	5340	87.35
1113+00	5340	87.66
1113+55	5340	87.73
1113+90	5340	87.79
1115+50	5340	87.79
1157+85	5340	87.91
1158+35	5340	87.83
1159+35	5340	87.92
1159+85	5340	88.12

TABLE 7 100 YEAR FLOOD PROFILE WITH EXISTING LAND USE AND IMPROVEMENT IN BRIDGE SECTIONS. REEDY CREEK SWAMP INFLOW INCLUDED WITH INITIAL STAGE OF 57.50 FT. MSL AT LAKE TOHOPEKALIGA.

Station ft.	Q cfs	Water Elev. ft. msl.
293+00	8070	57.50
309+84	8070	57.86
310+50	8070	59.02
310+65	8070	59.30
310+80	8070	59.84
320+30	8070	59.89
341+00	8070	61.34
344+58	8070	61.47
345+08	8070	61.49
345+20	8070	62.46
345+72	8070	62.47
361+00	8030	62.88
388+00	8030	63.56
388+50	8030	63.37
388+70	8030	64.39
389+00	8030	65.17
389+30	8030	65.18
389+90	8030	65.22
390+40	6010	65.34
402+70	6010	65.47
454+00	6010	66.00
507+00	6110	70.40
536+67	6110	71.21
537+50	6110	71.23
538+00	6110	71.23
539+50	6110	71.47
580+00	6110	74.44
603+00	6270	74.66
675+50	6270	75.44
708+50	5590	76.12
747+00	5590	77.11
781+00	5590	78.75
817+70	7880	79.20
831+05	7930	79.34
858+40	7930	79.57
885+52	7930	80.10
888+00	7930	80.42
902+65	7930	81.58
903+15	7460	81.58
903+55	7460	81.59
904+10	7460	81.59
912+60	7460	81.68
918+60	7460	81.87
955+70	7460	82.75
956+20	7460	82.75

TABLE 7 (Continued)

956+50	7460	82.80
956+80	6180	82.80
975+09	6180	82.83
975+59	6180	82.83
975+70	6180	82.85
976+00	6180	82.86
998+80	6180	82.96
999+30	6180	82.93
1000+70	6390	82.94
1001+70	6390	83.08
1028+00	5490	84.39
1056+45	5340	85.71
1063+00	5340	86.32
1081+40	5340	87.35
1113+00	5340	87.66
1113+55	5340	87.73
1113+90	5340	87.79
1115+50	5340	87.79
1157+85	5340	87.91
1158+35	5340	87.83
1159+35	5340	87.92
1159+85	5340	88.12

TABLE 8 100 YEAR FLOOD PROFILE WITH EXISTING LAND USE AND NO IMPROVEMENT IN BRIDGE SECTION - REEDY CREEK SWAMP INFLOW INCLUDED WITH INITIAL STAGE OF 58.50 FT. MSL AT LAKE TOHOPEKALIGA.

Station ft.	Q cfs	Water Elev. ft. msl.
293+00	8070	58.50
309+84	8070	58.62
310+50	8070	59.02
310+65	8070	59.74
310+80	8070	60.30
320+30	8070	60.32
341+00	8070	61.53
344+58	8070	61.65
345+08	8070	61.66
345+20	8070	62.52
345+72	8070	62.53
361+00	8030	62.93
388+00	8030	63.58
388+50	8030	63.40
388+70	8030	64.41
389+00	8030	65.19
389+30	8030	65.20
389+90	8030	65.24
390+40	6010	65.39
402+70	6010	65.48
454+00	6010	66.01
507+00	6110	70.40
536+67	6110	71.21
537+50	6110	71.23
538+00	6110	71.23
539+50	6110	71.47
580+00	6110	74.44
603+00	6270	74.66
675+50	6270	75.44
708+50	5590	76.12
747+00	5590	77.11
781+00	5590	78.75
817+70	7880	79.20
831+05	7930	79.34
858+40	7930	79.57
885+52	7930	80.10
888+00	7930	80.42
902+65	7930	81.58
903+15	7460	81.58
903+55	7460	81.59
904+10	7460	81.59
912+60	7460	81.68
918+60	7460	81.87
955+70	7460	82.75
956+20	7460	82.75
945+50	7460	82.80
956+80	6180	82.80
975+09	6180	82.80
975+59	6180	82.83

TABLE 9 100 YEAR FLOOD PROFILE WITH BRIDGE IMPROVEMENT AND BY-PASS CHANNEL - INITIAL STAGE OF 58.50 FT. MSL.

Station ft.	Q cfs	Water Elev. ft. msl.
293+00	8070	58.50
309+84	8070	58.62
310+50	8070	59.02
310+51	8070	59.99
310+65	8070	59.99
310+80	8070	59.99
320+30	8070	60.45
341+00	8070	60.96
344+58	8070	60.99
345+08	8070	61.00
345+20	8070	61.00
345+72	8070	61.03
361+00	8030	61.31
388+50	8030	60.99
388+70	8030	62.51
389+00	8030	63.90
389+30	8030	64.07
389+90	8030	64.23
390+40	6010	64.41
402+70	6010	64.59

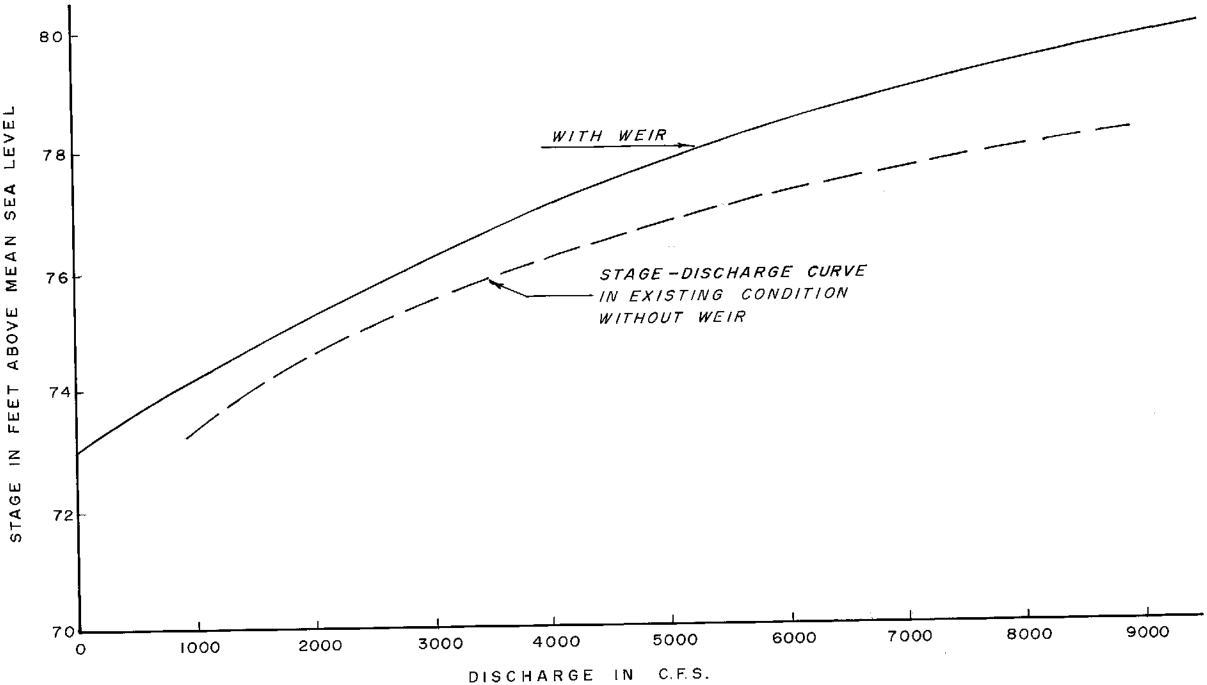


Figure I RATING CURVE AT STA. 747+00 WITH AND WITHOUT WEIR

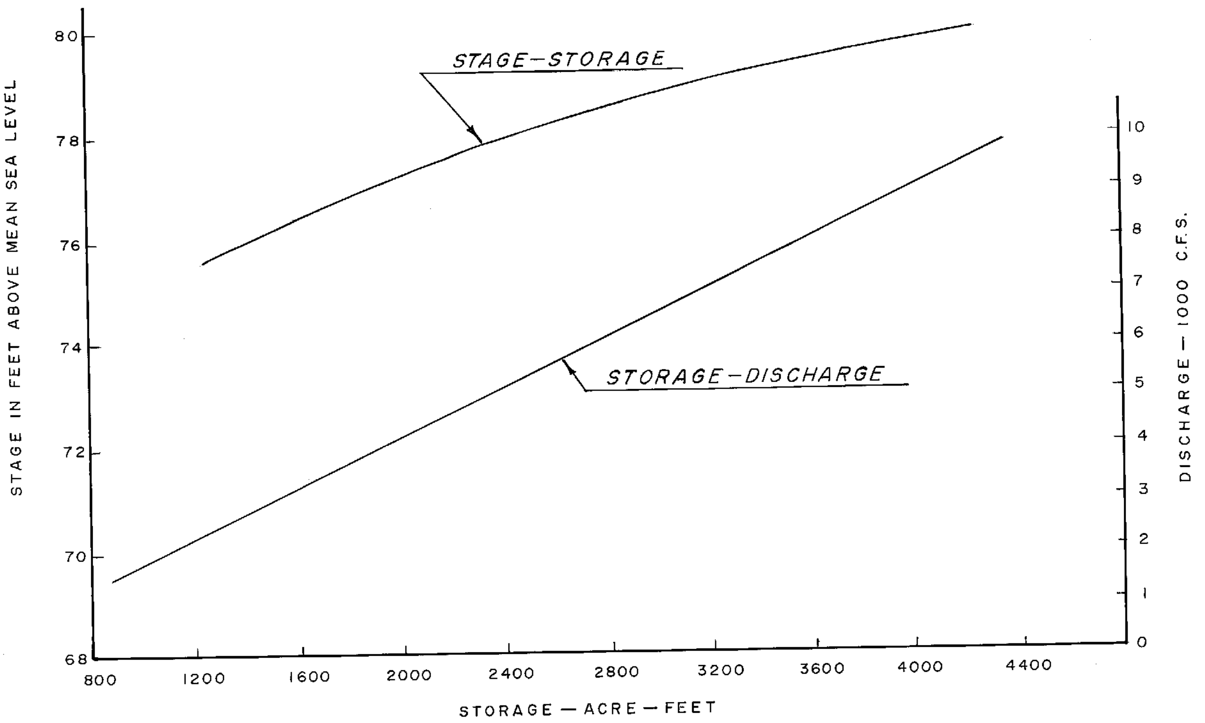


Figure 2 STAGE, STORAGE, AND DISCHARGE RELATION FOR REACH No. 11A WITH ON STREAM STRUCTURE

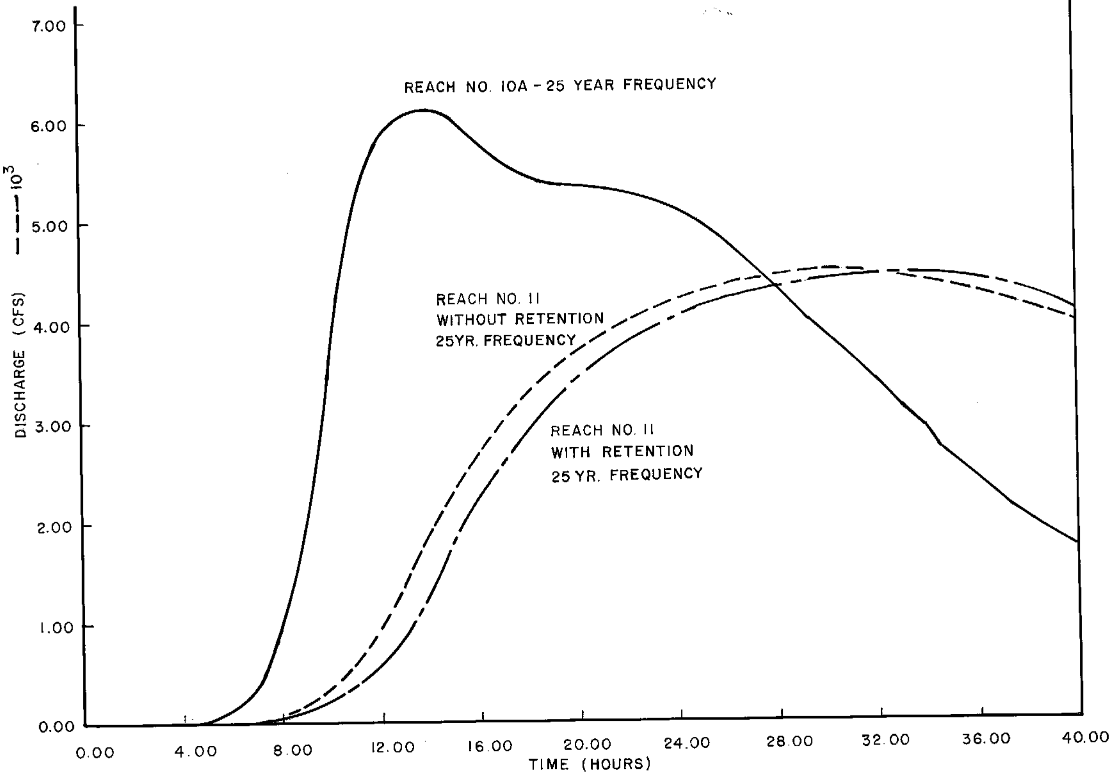


Figure 3 COMPARISONS OF INFLOW AND OUTFLOW HYDROGRAPHS FROM LARGE CYPRESS MARSH UNDER 25 YEAR FLOOD WITH & WITHOUT ON-STREAM RETENTION.

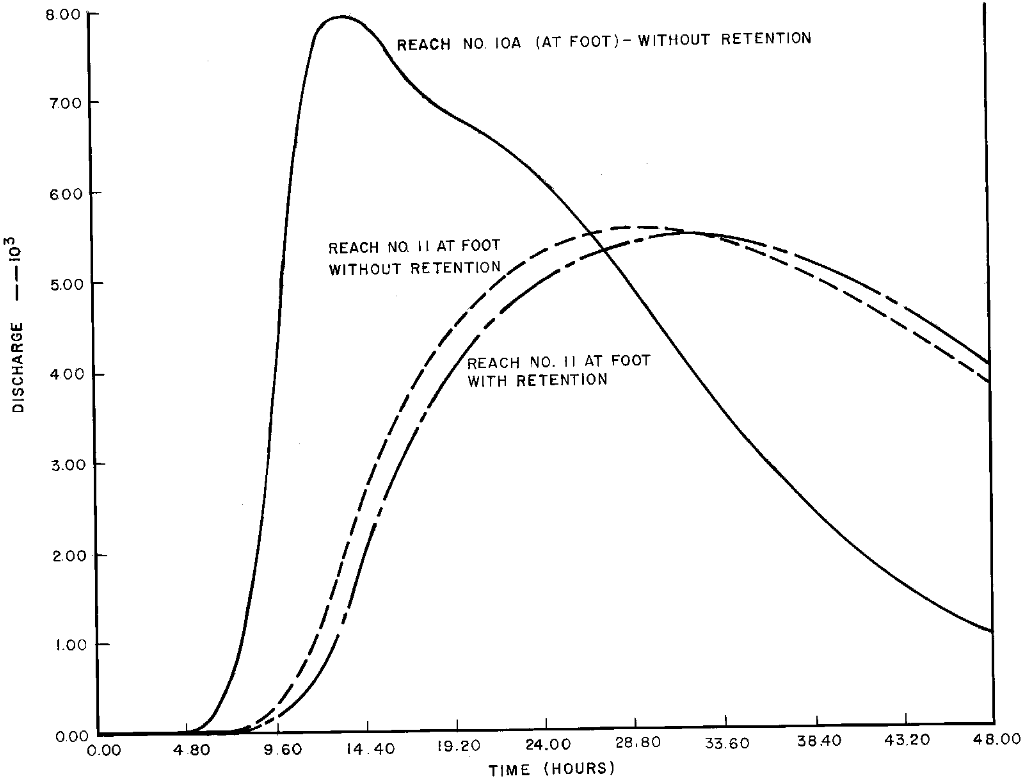


Figure 4 COMPARISON OF INFLOW AND OUTFLOW HYDROGRAPHS FROM LARGE CYPRESS MARSH UNDER 100 YEAR FLOOD WITH & WITHOUT ON-STREAM RETENTION.

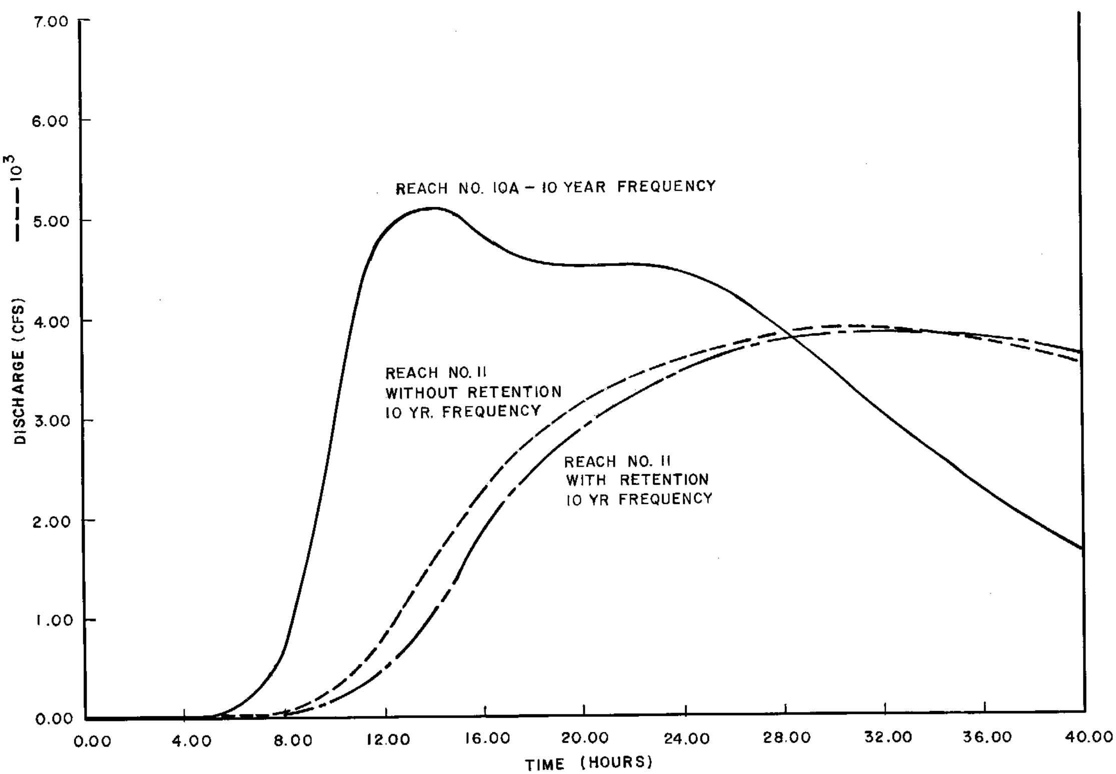
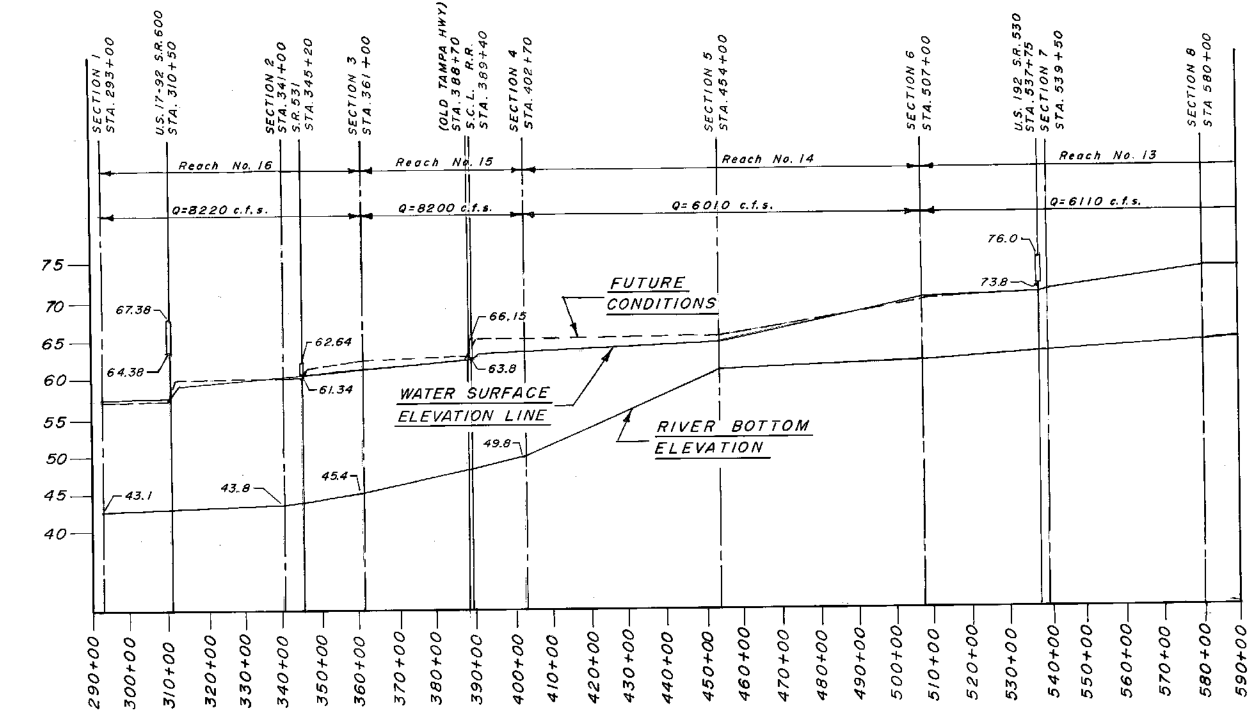


Figure 5 COMPARISON OF INFLOW AND OUTFLOW HYDROGRAPHS FROM LARGE CYPRESS MARSH UNDER 10 YEAR FLOOD WITH & WITHOUT ON-STREAM RETENTION.



PROFILE

——— WATER SURFACE WITH BRIDGE IMPROVED AND EXISTING CONDITION.

– – – FUTURE CONDITIONS W/O ON SITE & ON-STREAM RETENTION.

Figure 6a WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 290 TO STA. 590

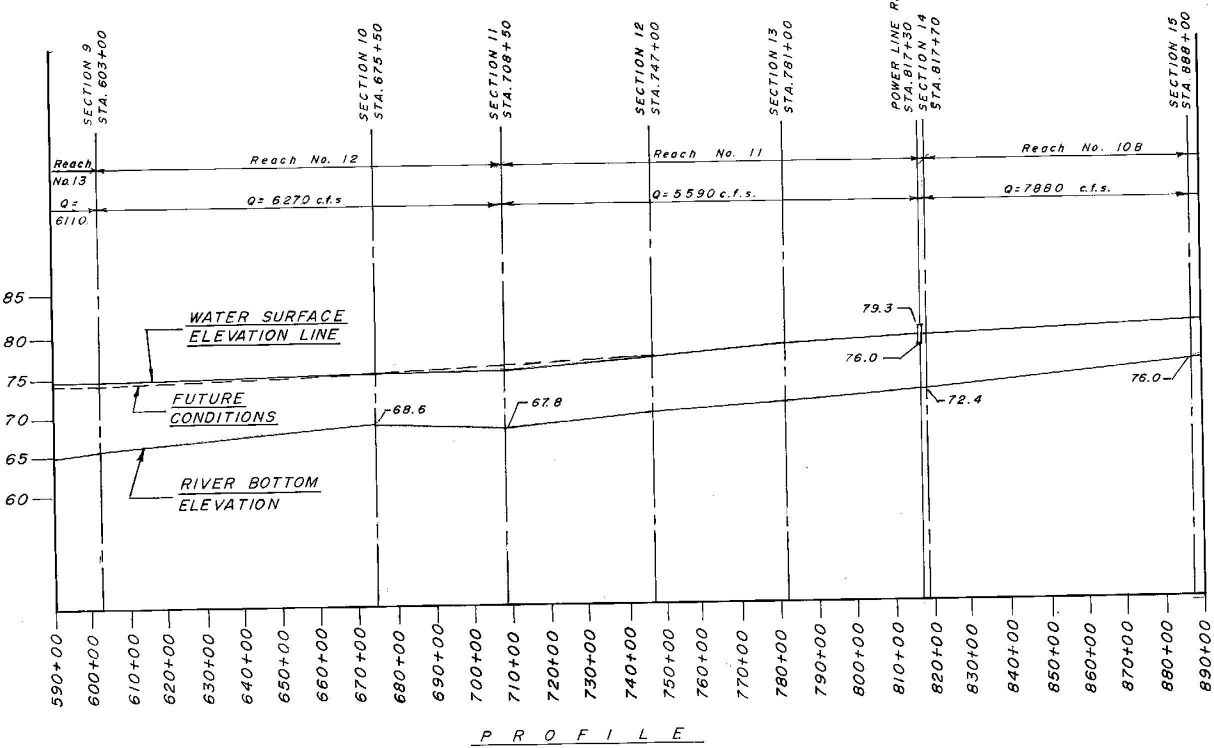


Figure 6b WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 590 TO STA. 890

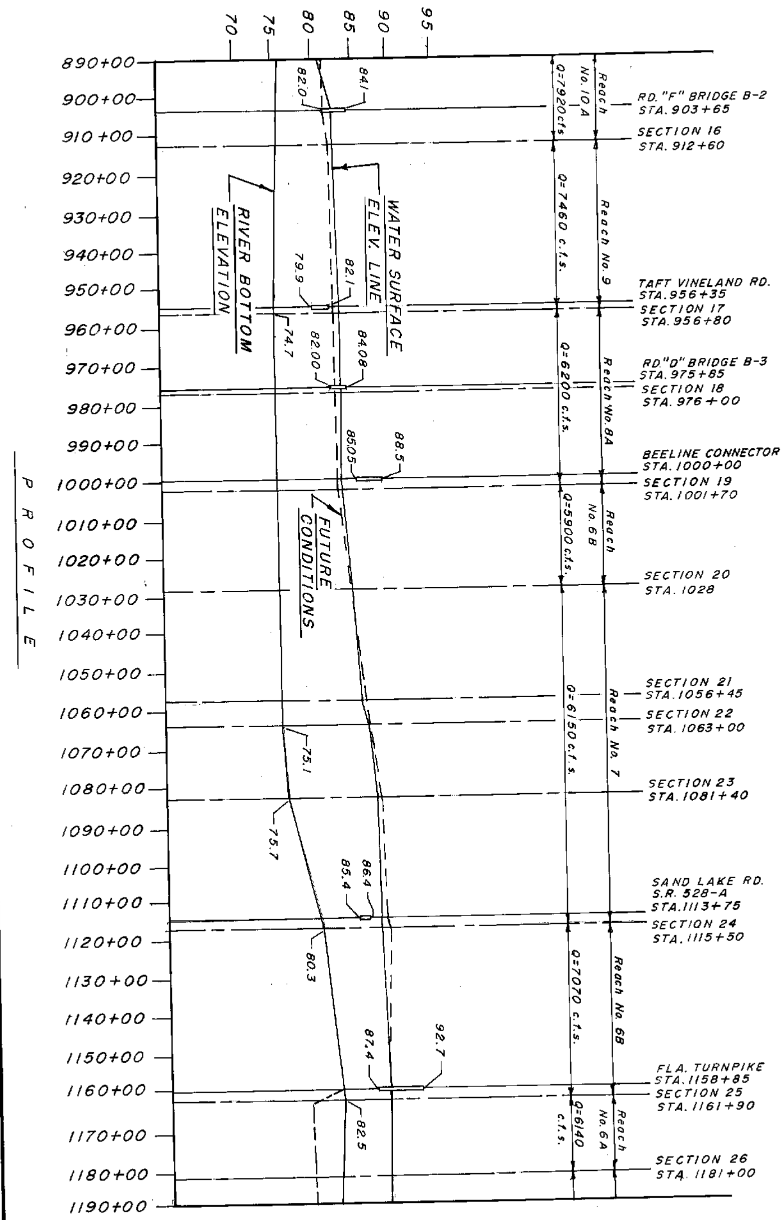


Figure 6c WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 890 TO STA. 1190

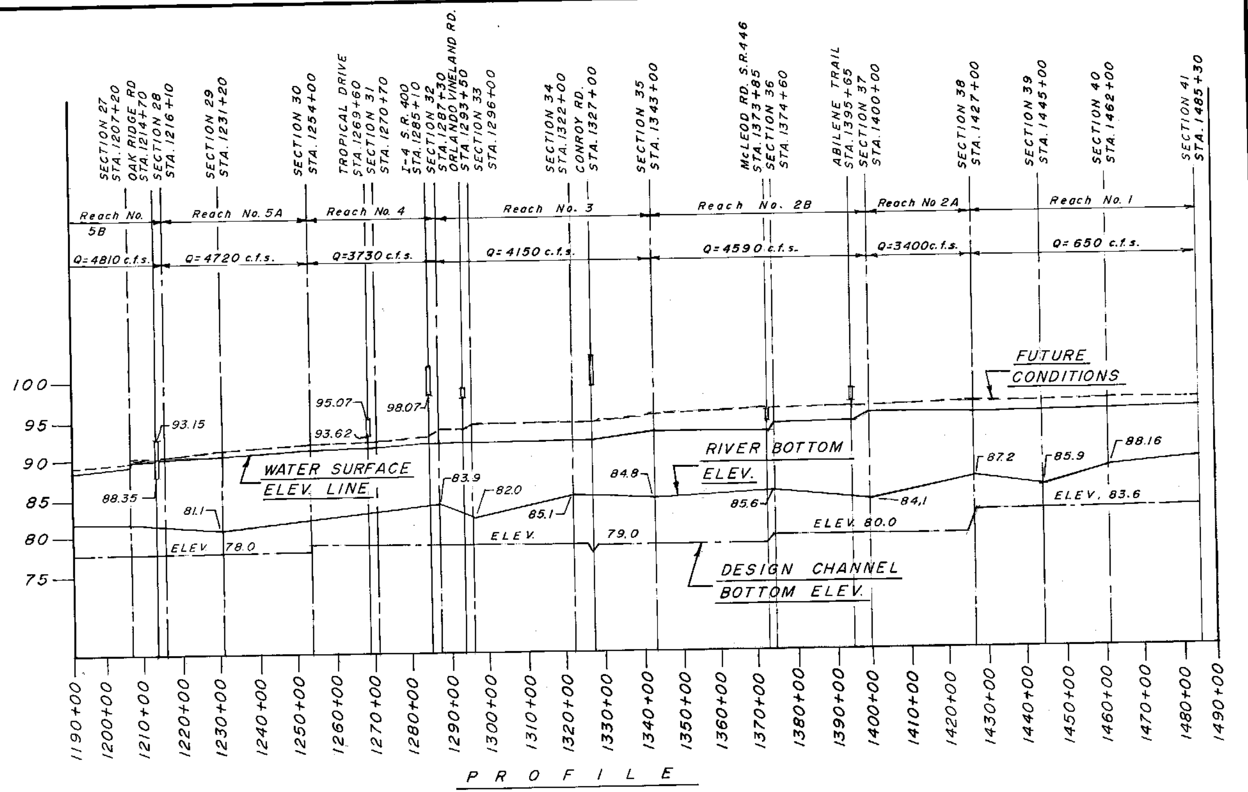


Figure 6d WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 1190 TO STA. 1490

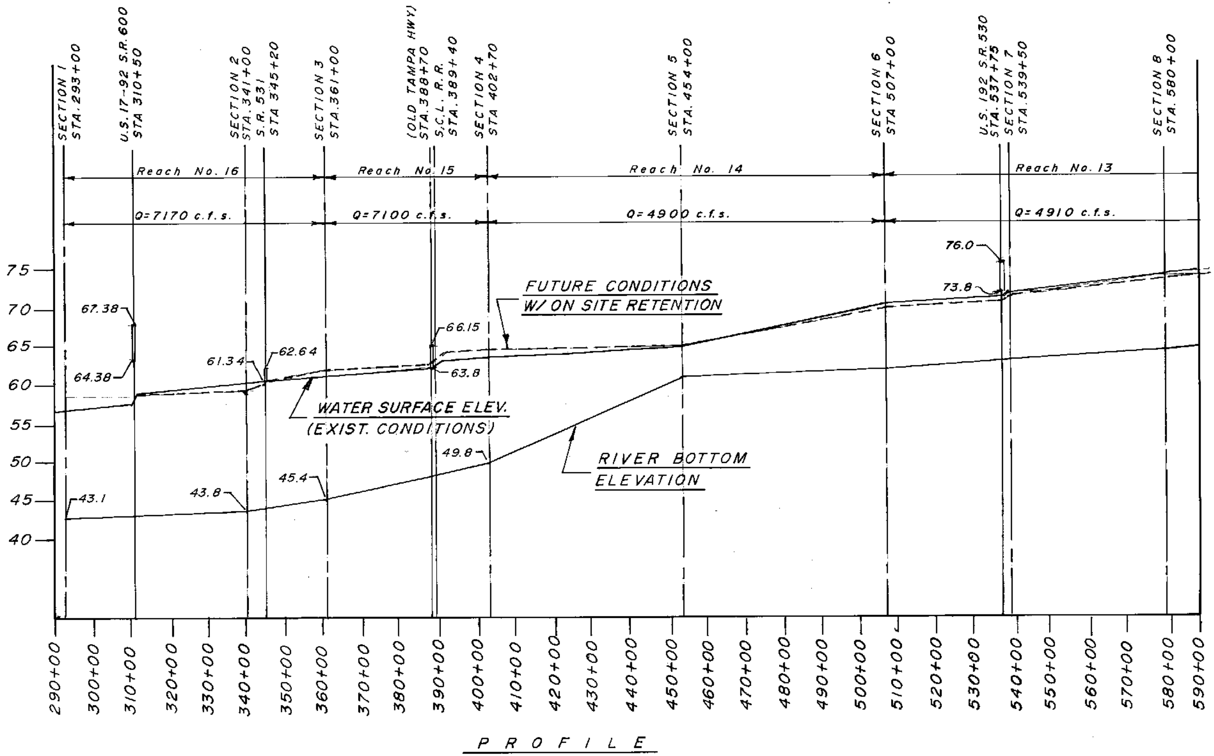


Figure 7a WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 290 TO STA. 590

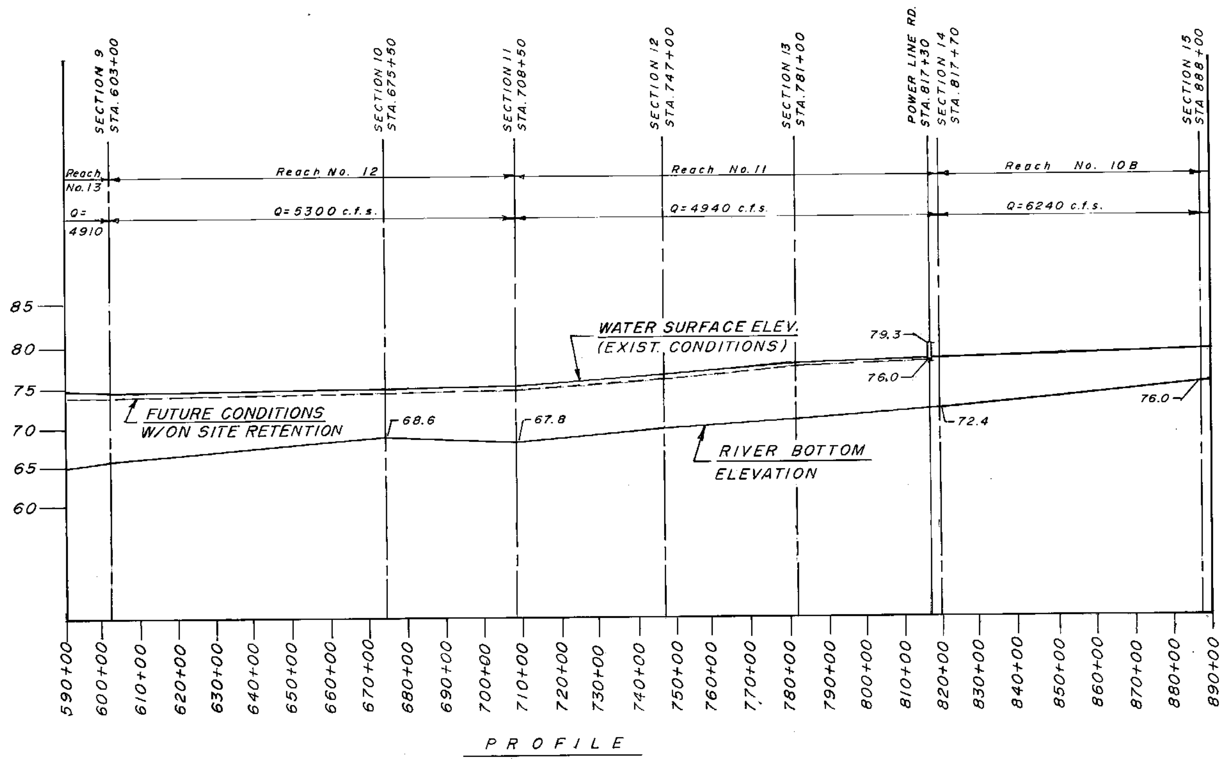


Figure 7b WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 590 TO STA. 890

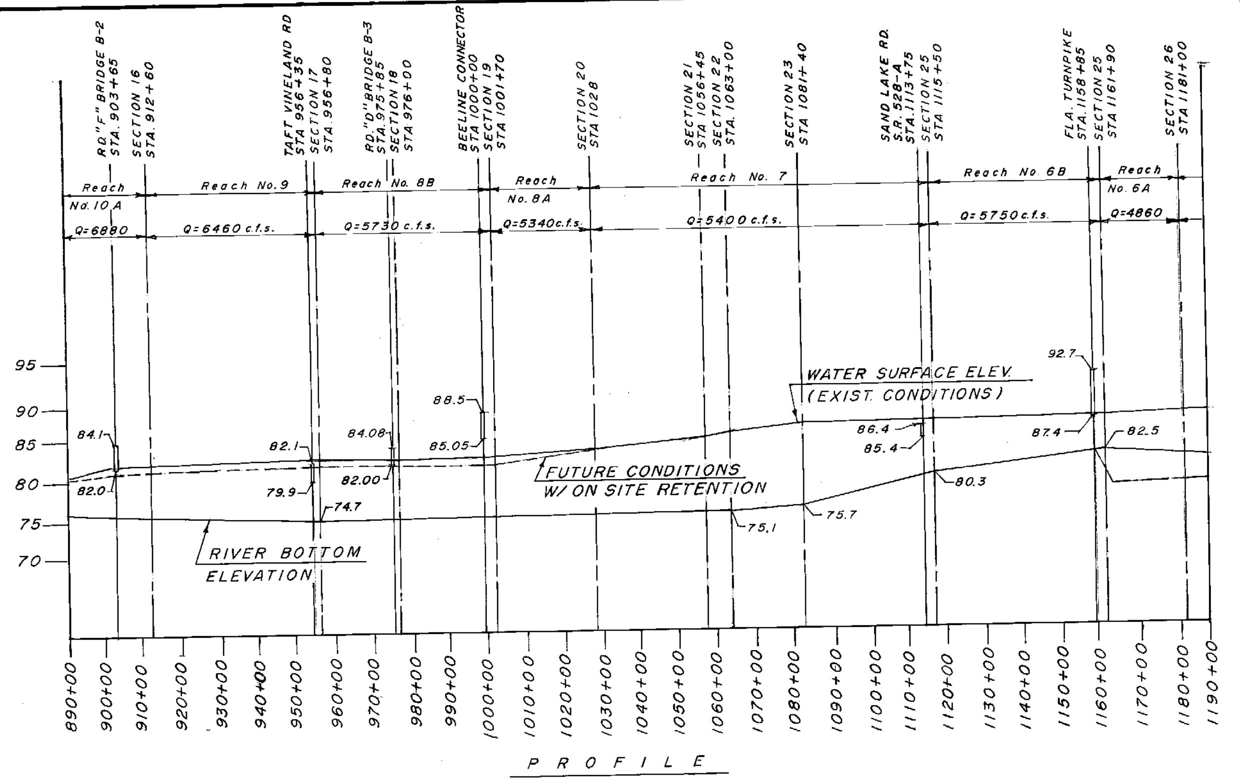


Figure 7c WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 890 TO STA. 1190

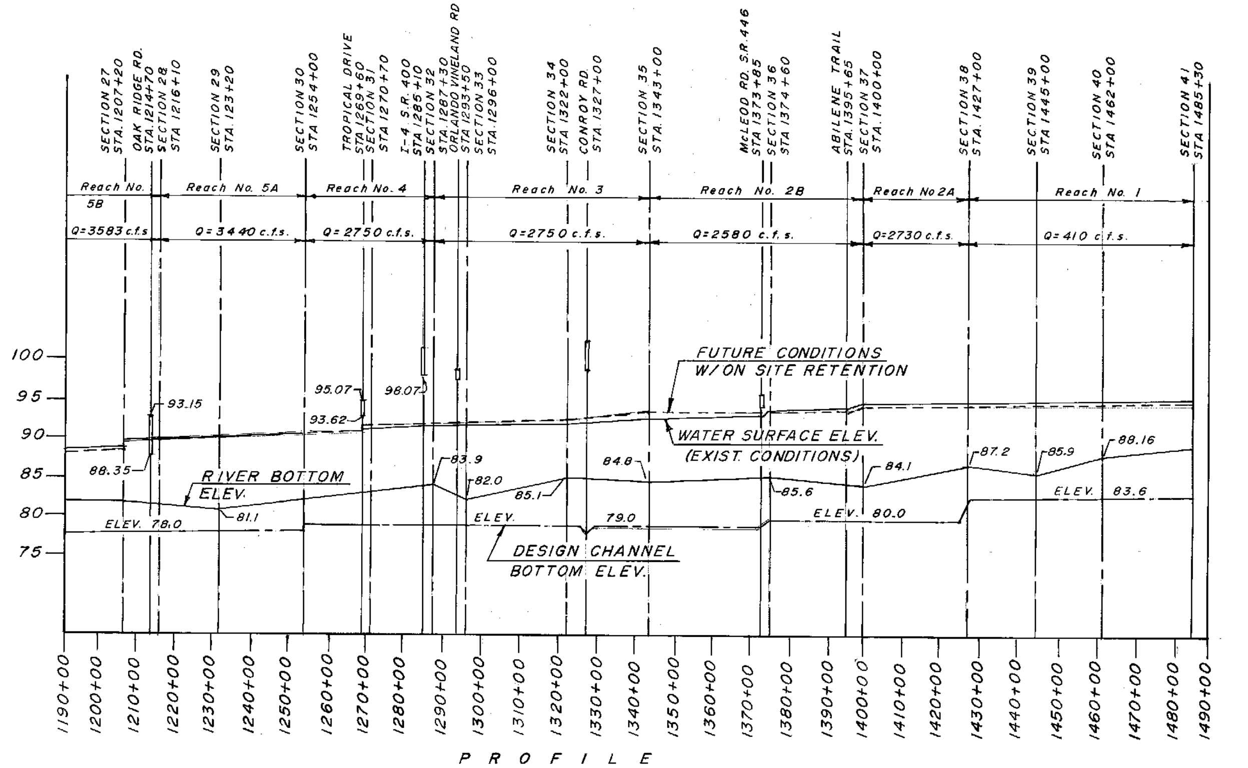


Figure 7d WATER SURFACE ELEVATION LINE, 100 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 1190 TO STA. 1490

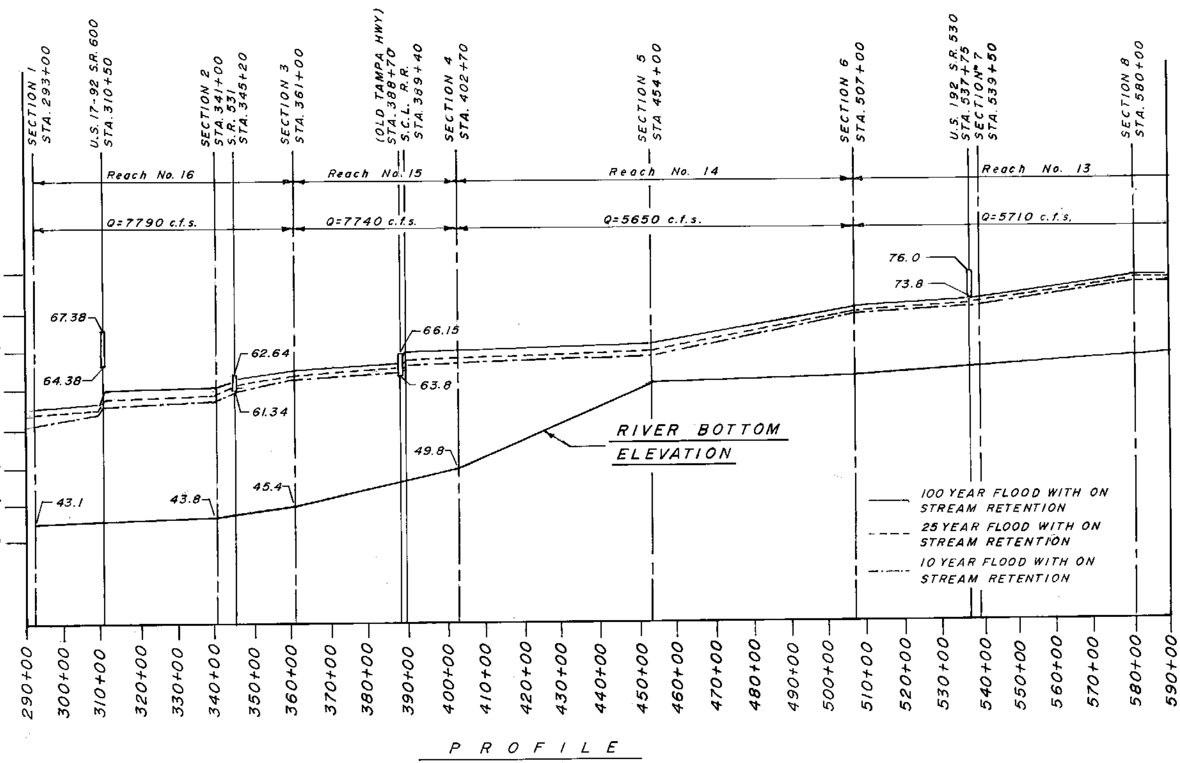


Figure 8a WATER SURFACE ELEVATION LINE, 10, 25, AND 100 YEAR FLOODS, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 290 TO STA. 590

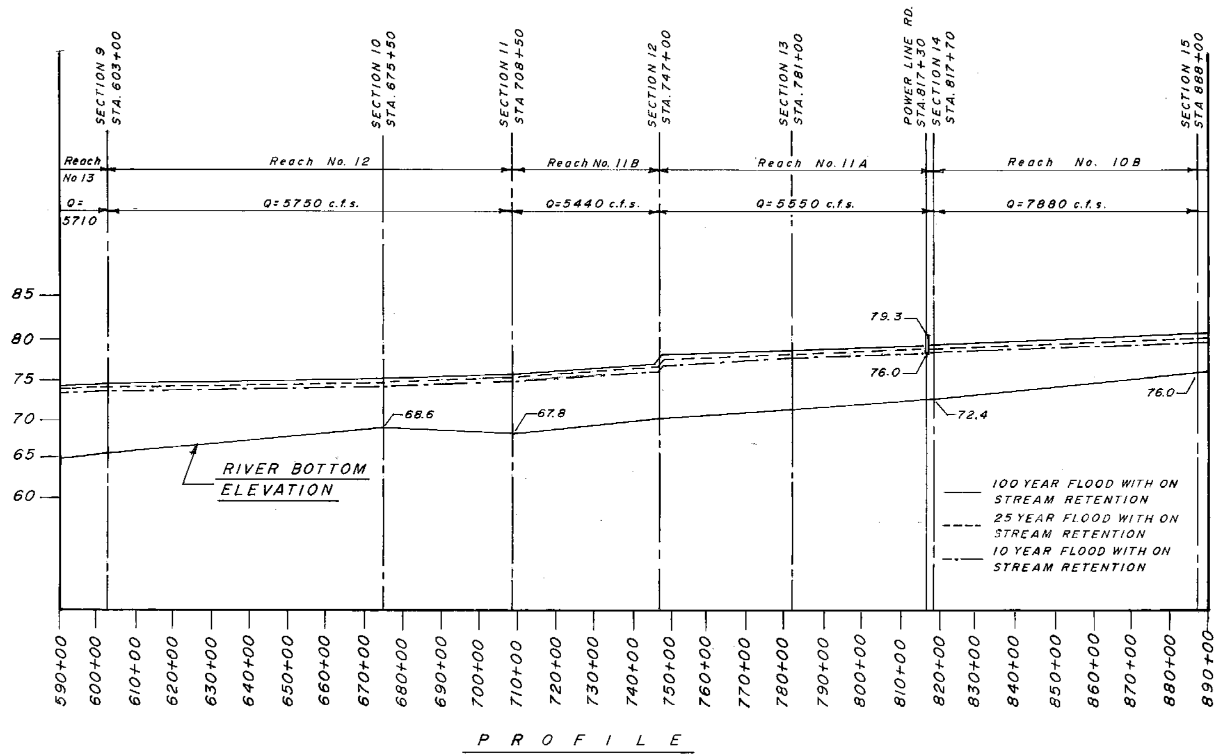


Figure 8b WATER SURFACE ELEVATION LINE, 10,25, AND 100 YEAR FLOODS,(WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 590 TO STA. 890

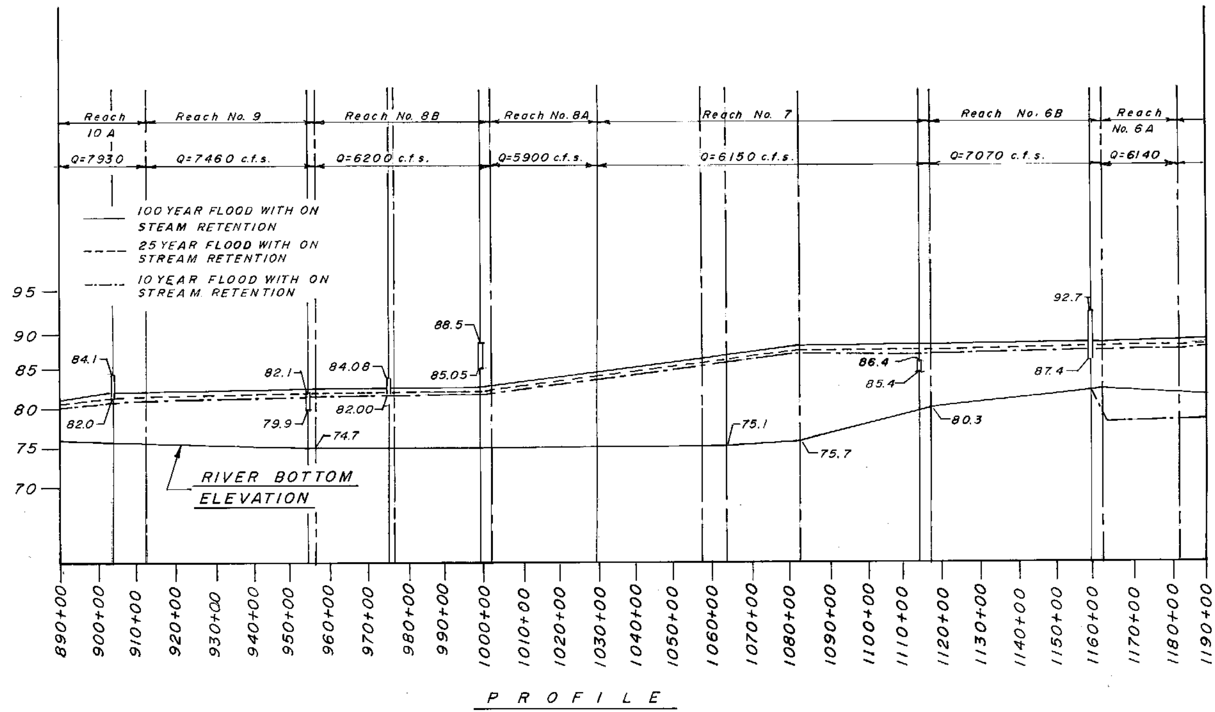


Figure 8c WATER SURFACE ELEVATION LINE, 10, 25, AND 100 YEAR FLOODS, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 890 TO STA. 1190

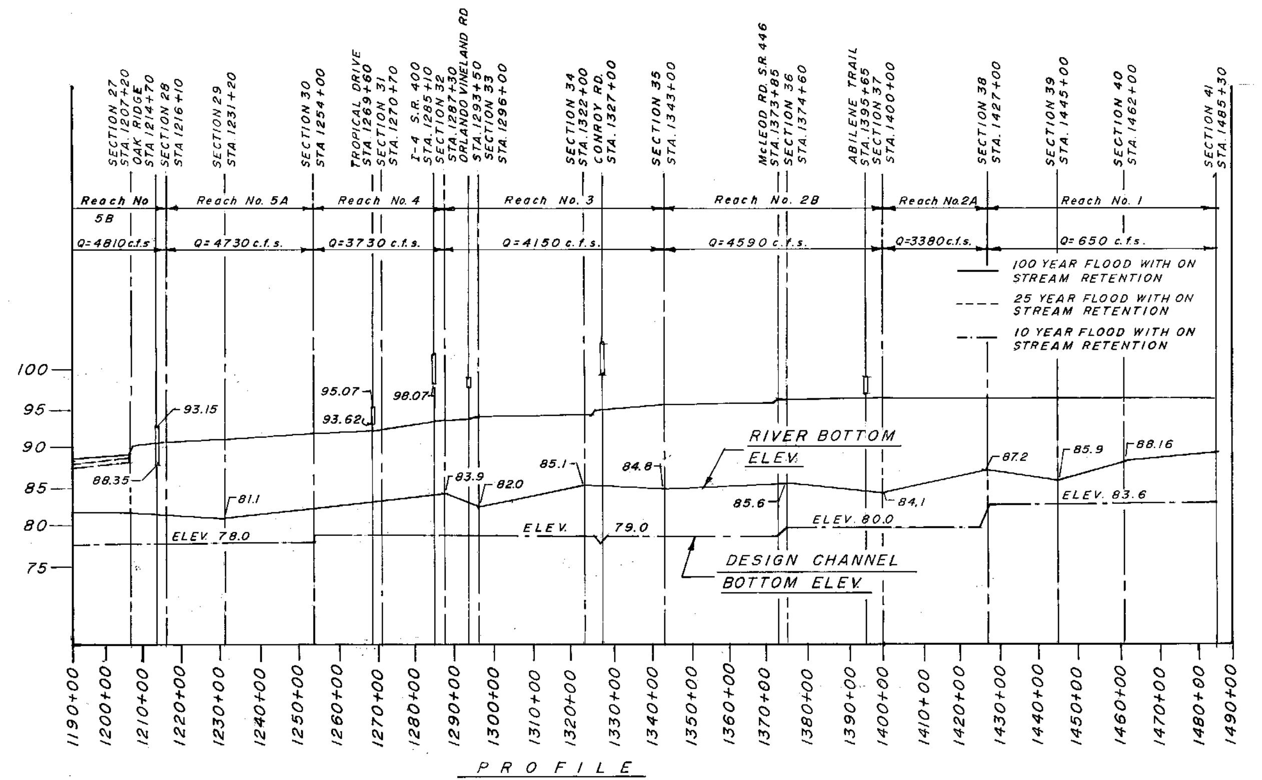


Figure 8d WATER SURFACE ELEVATION LINE, 10, 25, AND 100 YEAR FLOODS, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 1190 TO STA. 1490

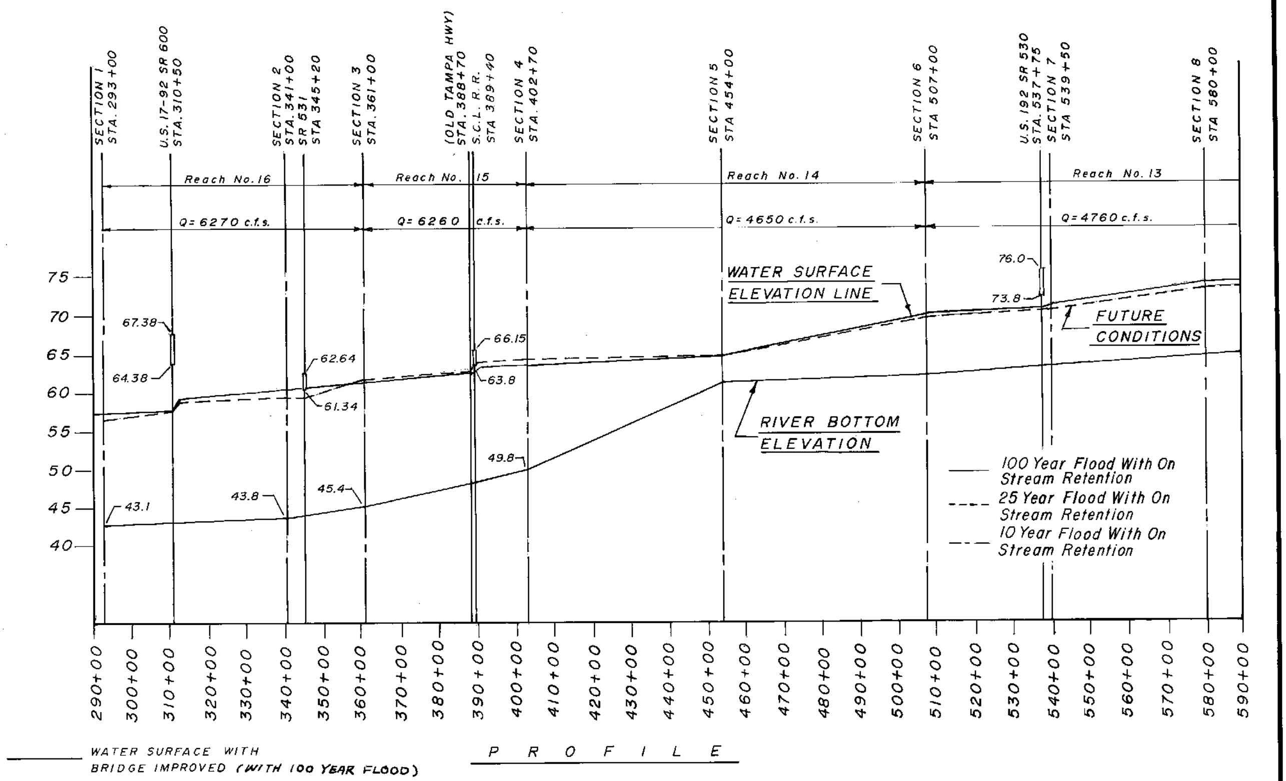


Figure 9a WATER SURFACE ELEVATION LINE, 25 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 290 TO STA. 590

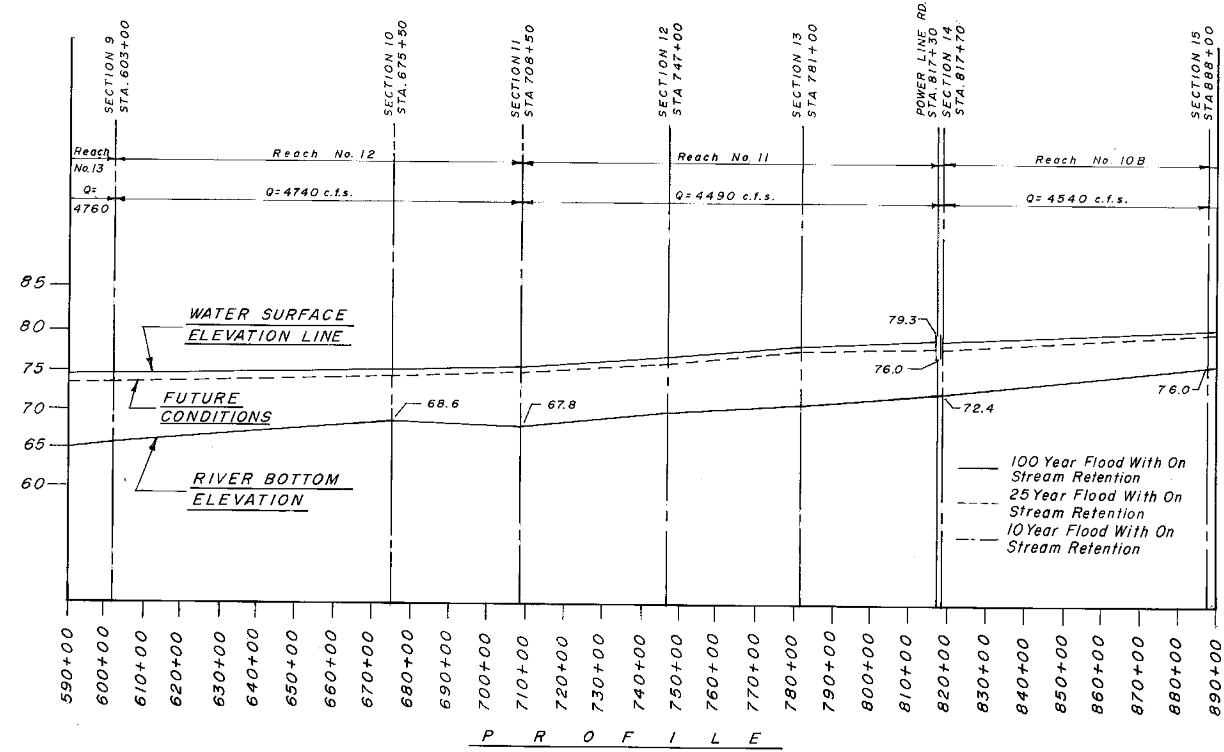


Figure 9b WATER SURFACE ELEVATION LINE, 25 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 590 TO STA. 890

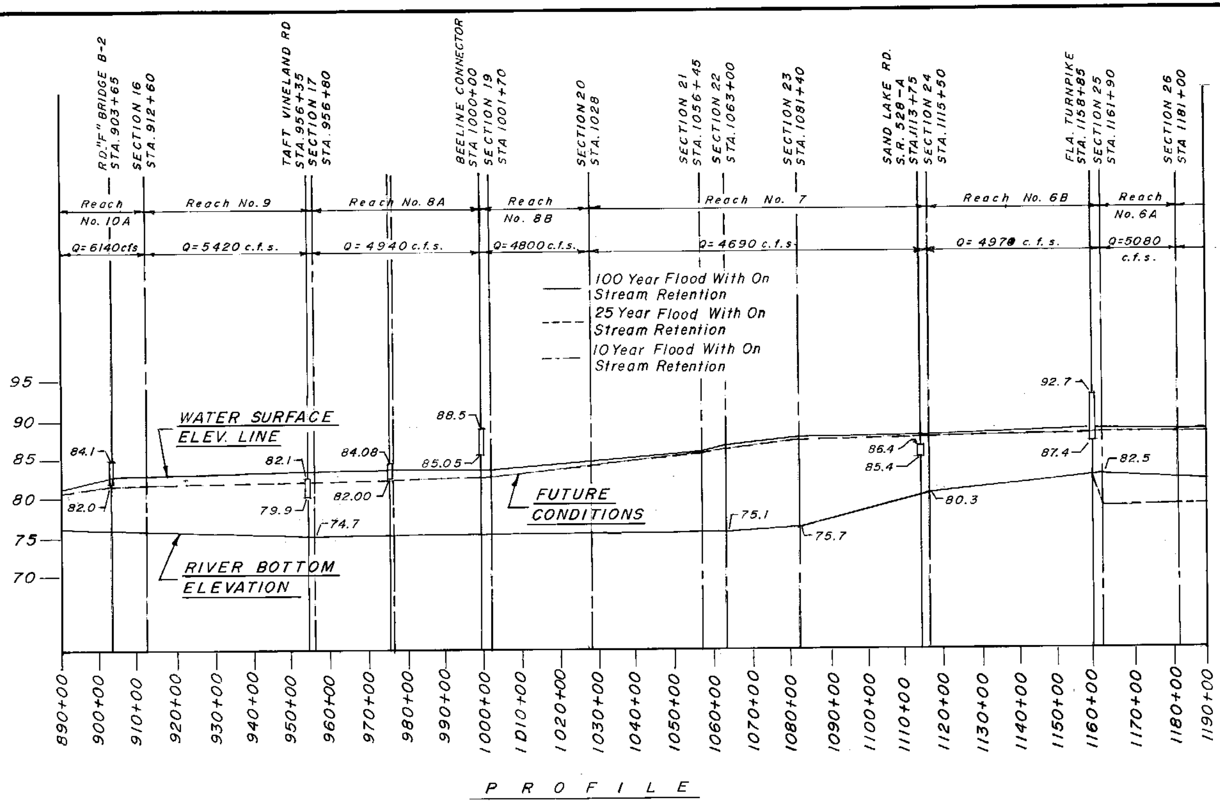


Figure 9c WATER SURFACE ELEVATION LINE, 25 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 890 TO STA. 1190

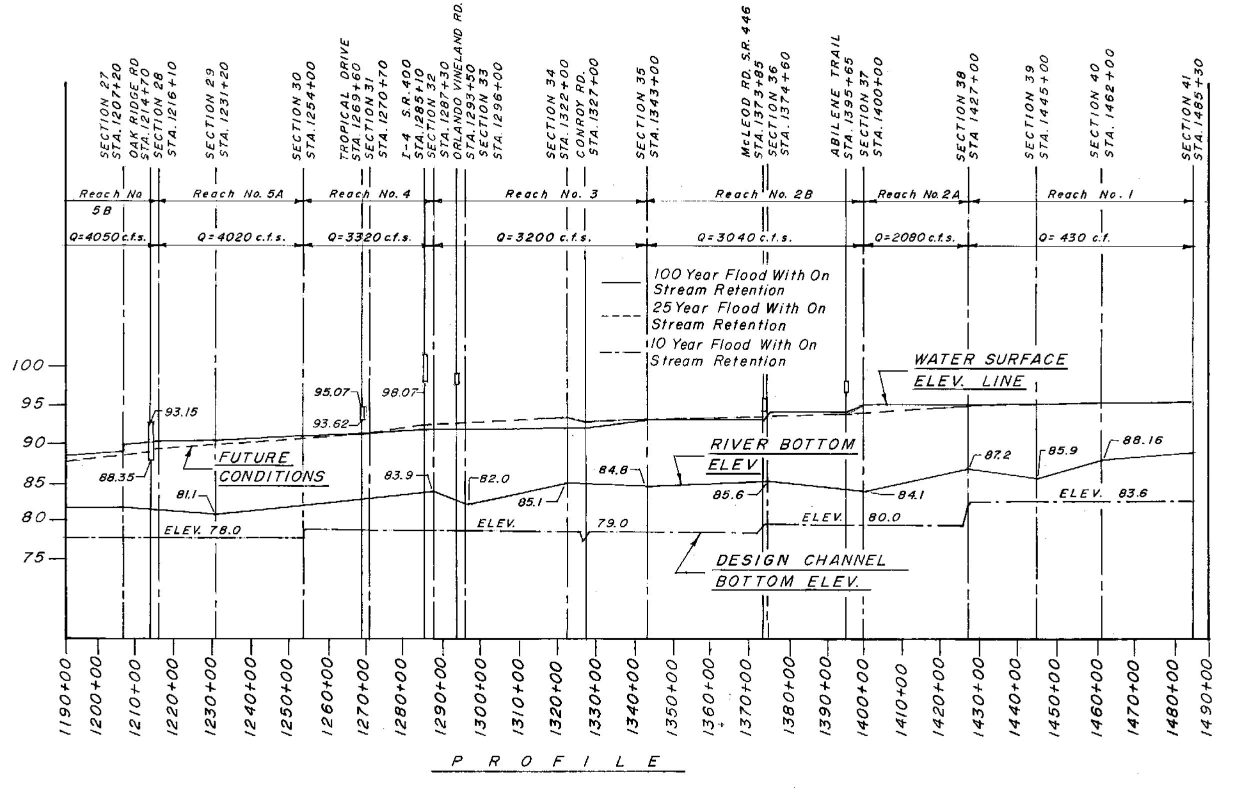


Figure 9d WATER SURFACE ELEVATION LINE, 25 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 1190 TO STA. 1490

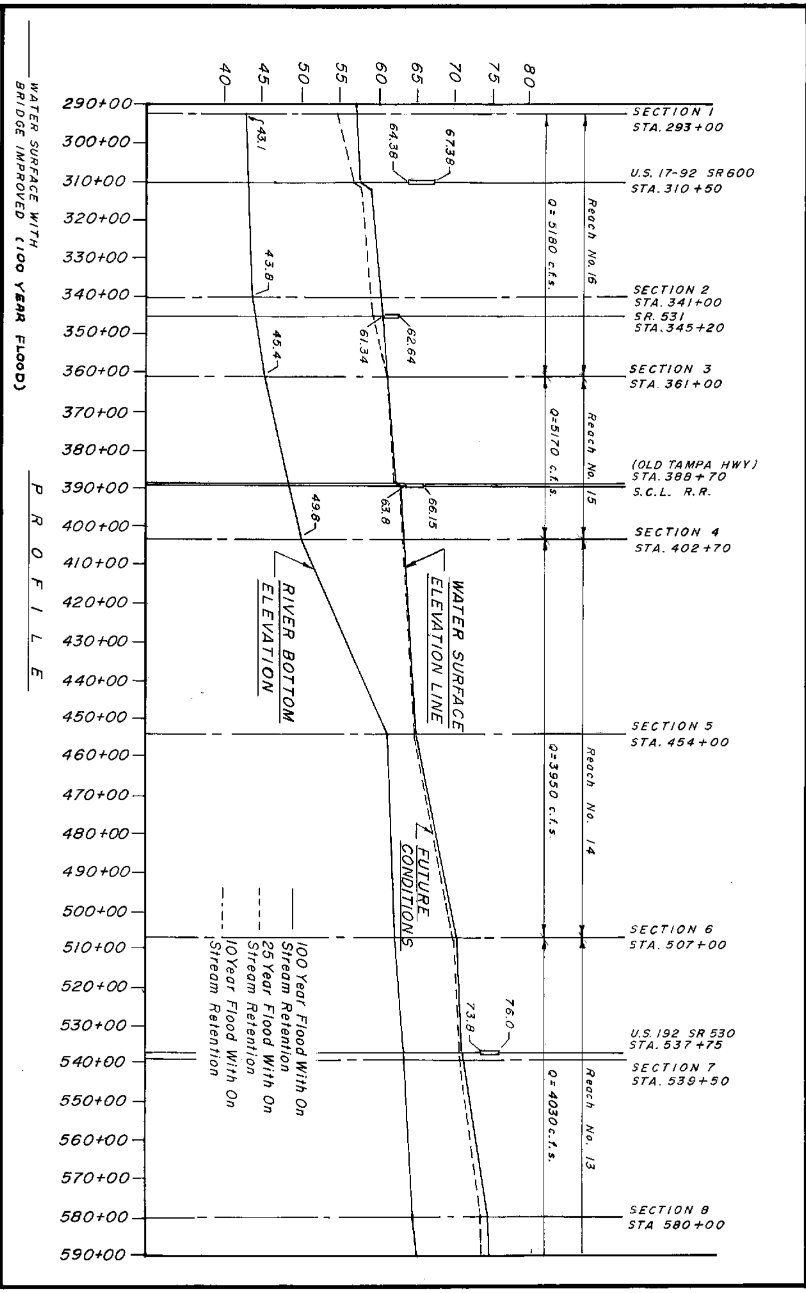


Figure 10a WATER SURFACE ELEVATION LINE, 10 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 290 TO STA. 590

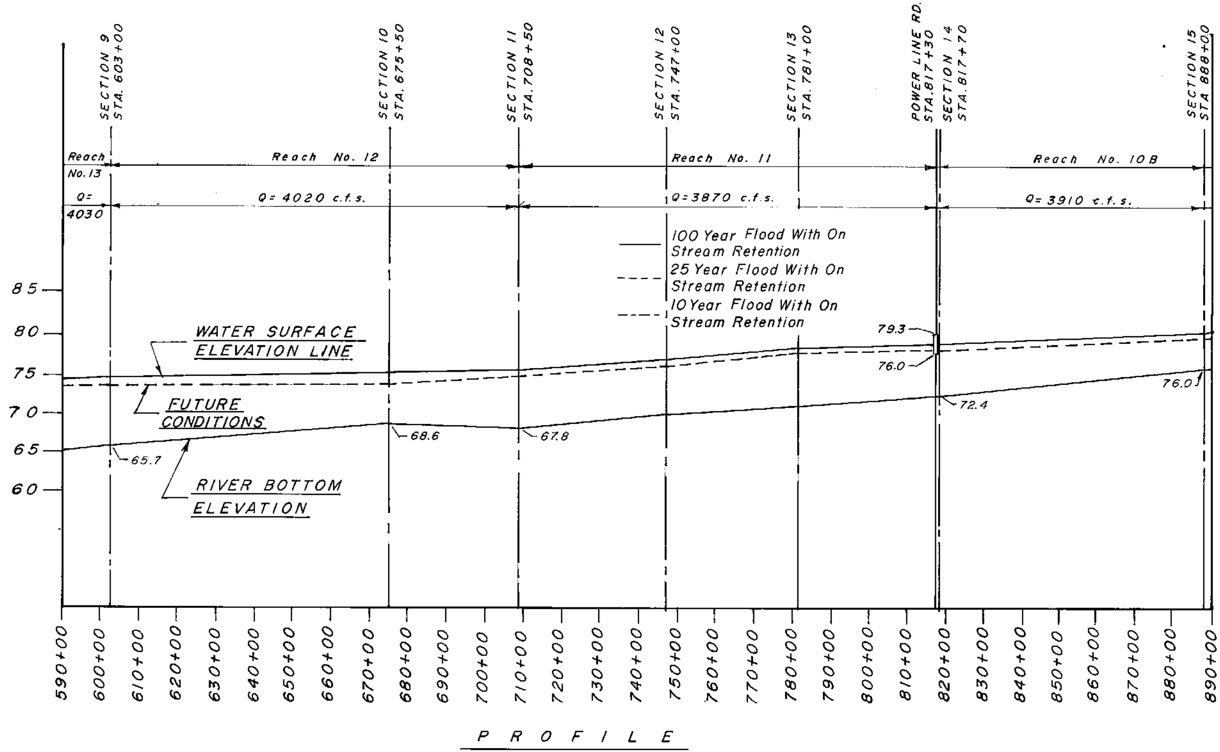


Figure 10b WATER SURFACE ELEVATION LINE, 10 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 590 TO STA. 890

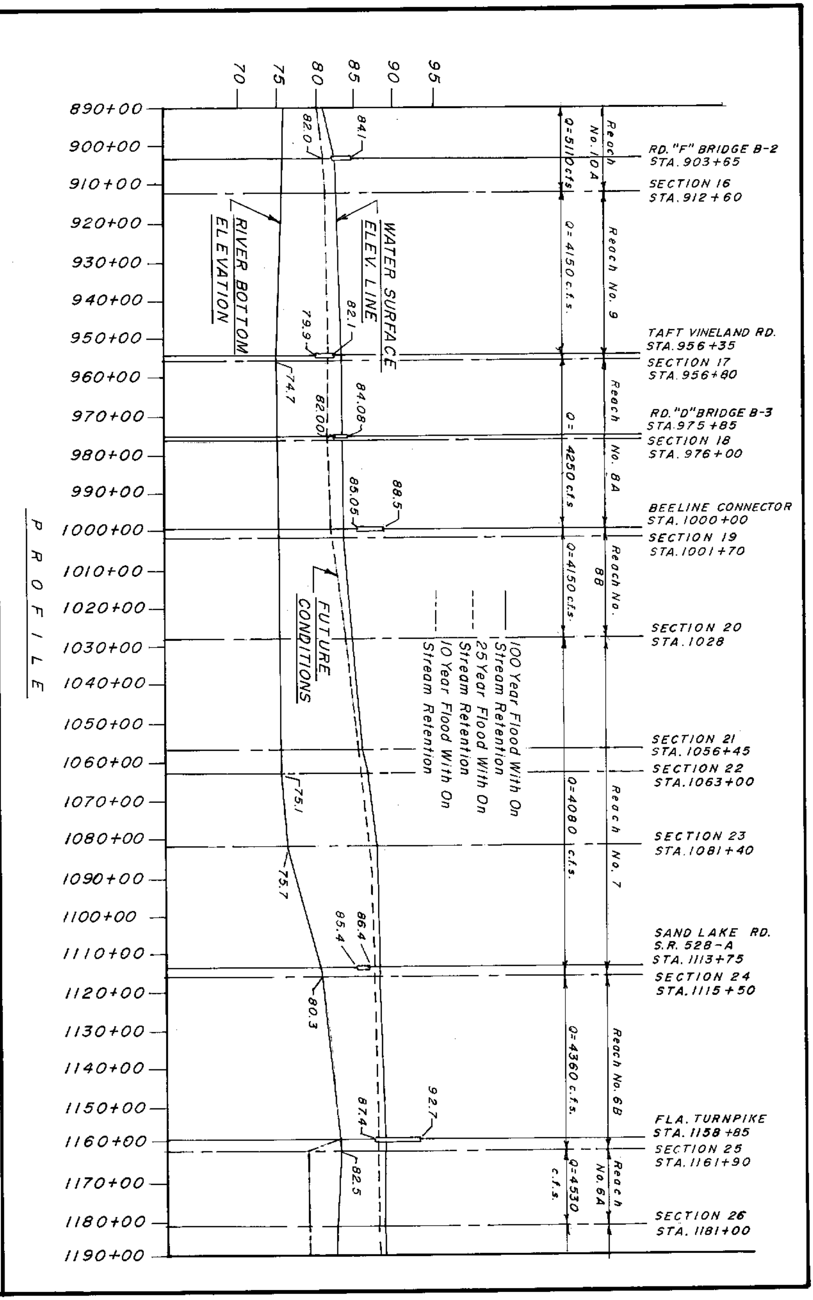


Figure 10c WATER SURFACE ELEVATION LINE, 10 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 890 TO STA. 1190

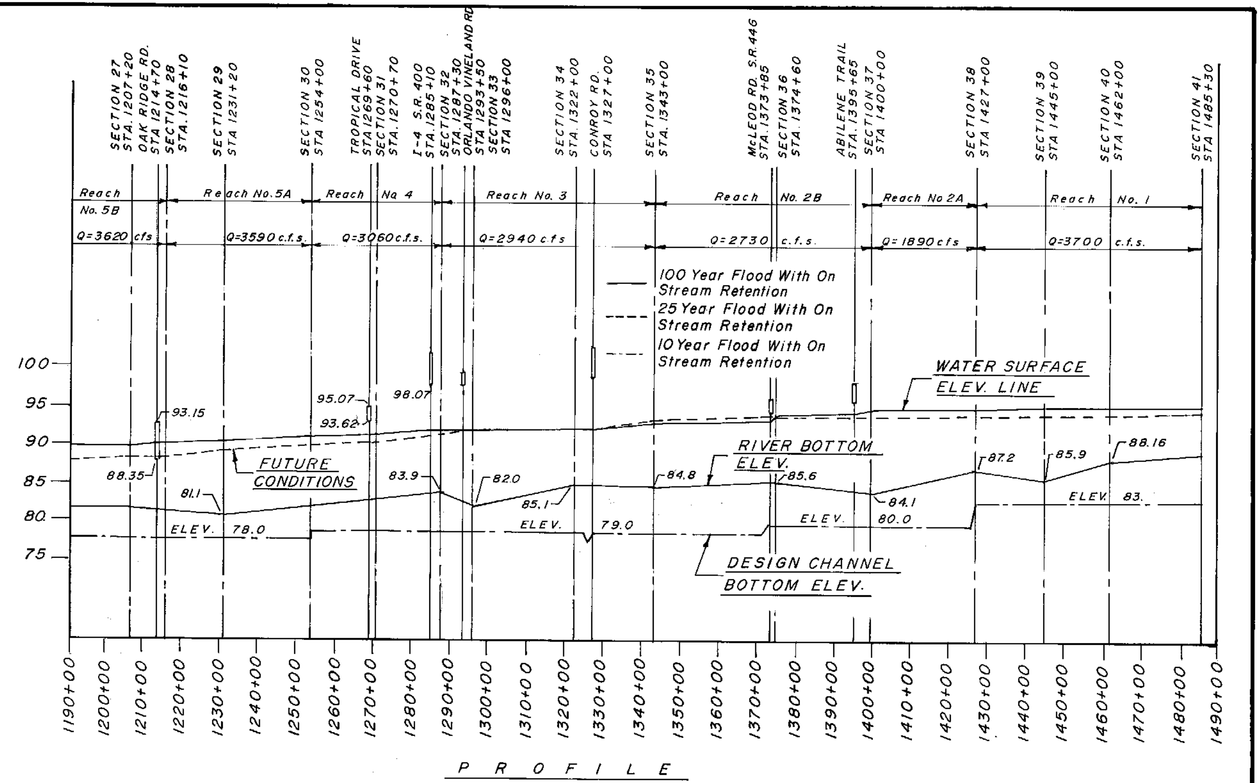


Figure 10d WATER SURFACE ELEVATION LINE, 10 YEAR FLOOD, (WITH CHANNELIZATION NORTH OF TURNPIKE) STA. 1190 TO STA. 1490

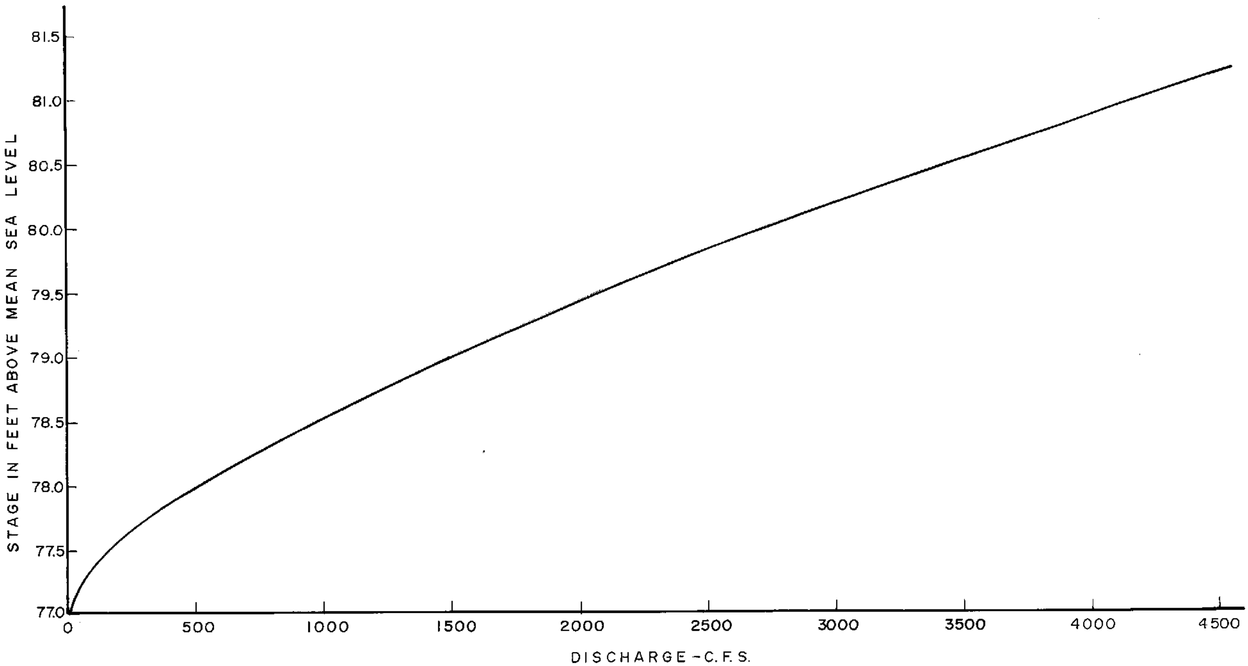


Figure II STAGE-DISCHARGE FOR 200 FOOT WEIR AT STATION 747+00

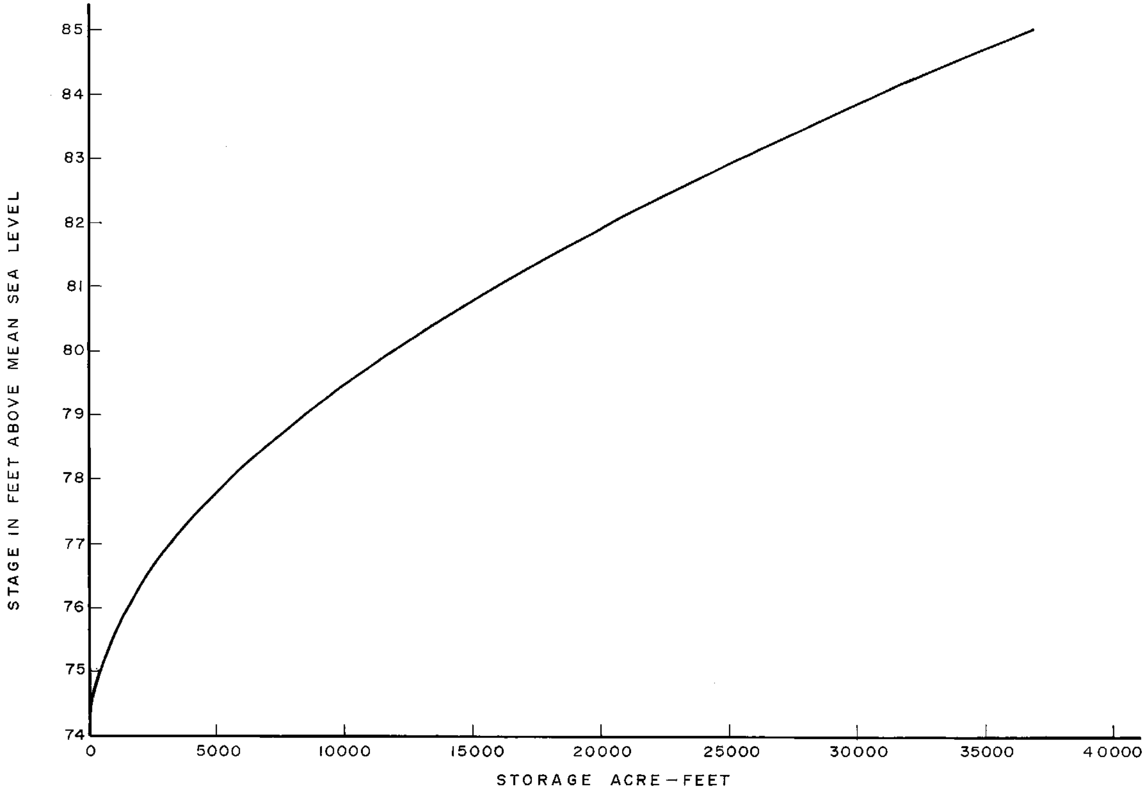


Figure 12 STAGE STORAGE SHINGLE CREEK SWAMP NORTH OF OSCEOLA CO. LINE BASED ON RECENT (1978) TOPOGRAPHY

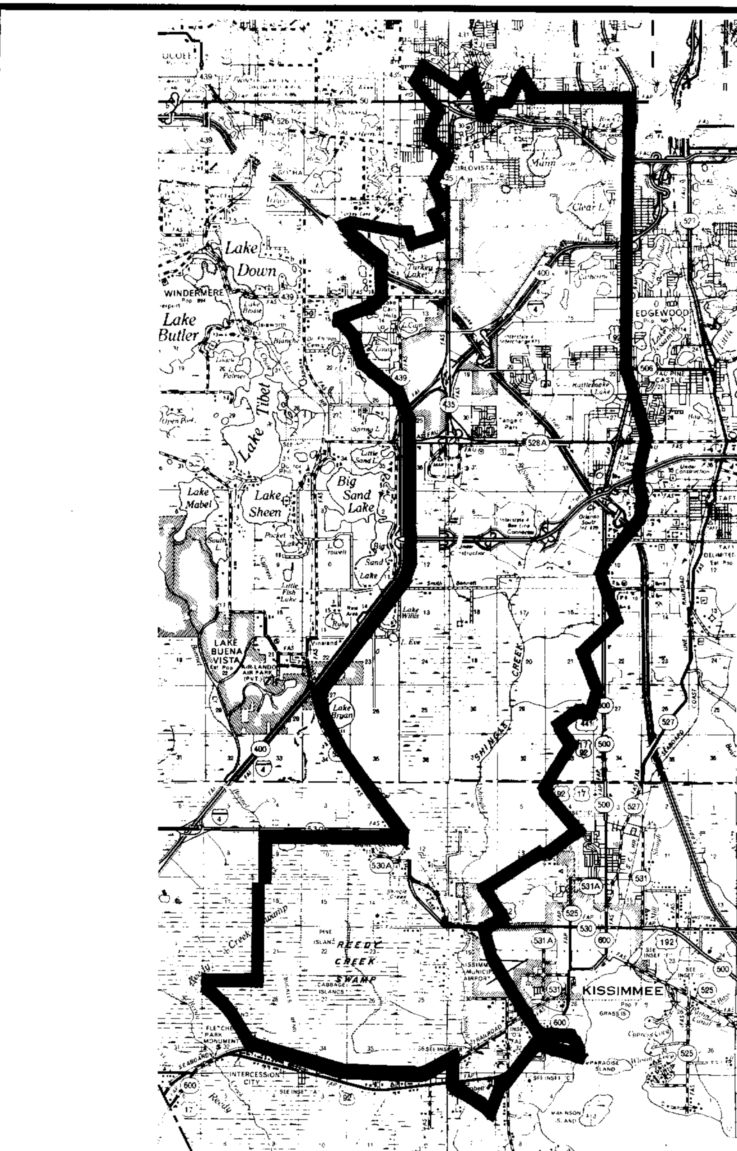


Figure 13 SHINGLE CREEK BASIN

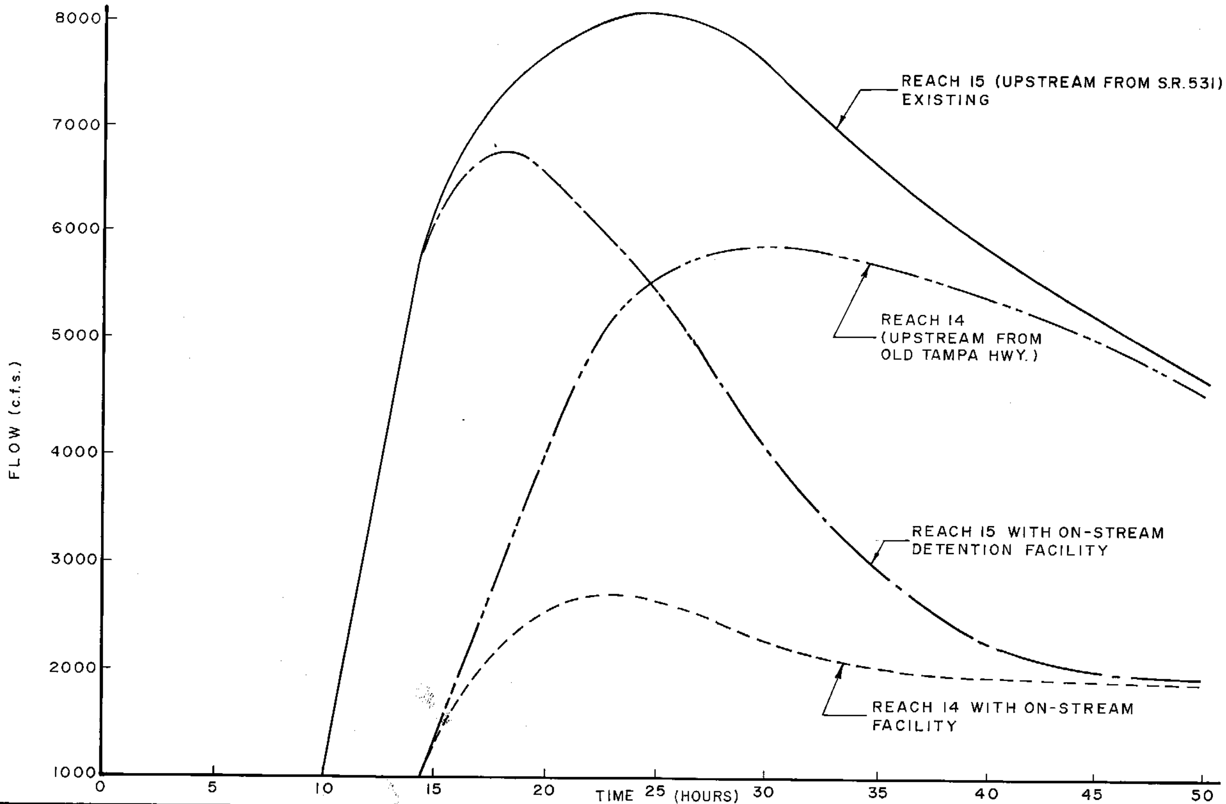


Figure 14 RUNOFF HYDROGRAPHS AT END REACH OF 14 & 15 WITH & WITHOUT ON-STREAM RETENTION LOCATED AT LARGE CYPRESS MARSH JUST NORTH OF COUNTY LINE

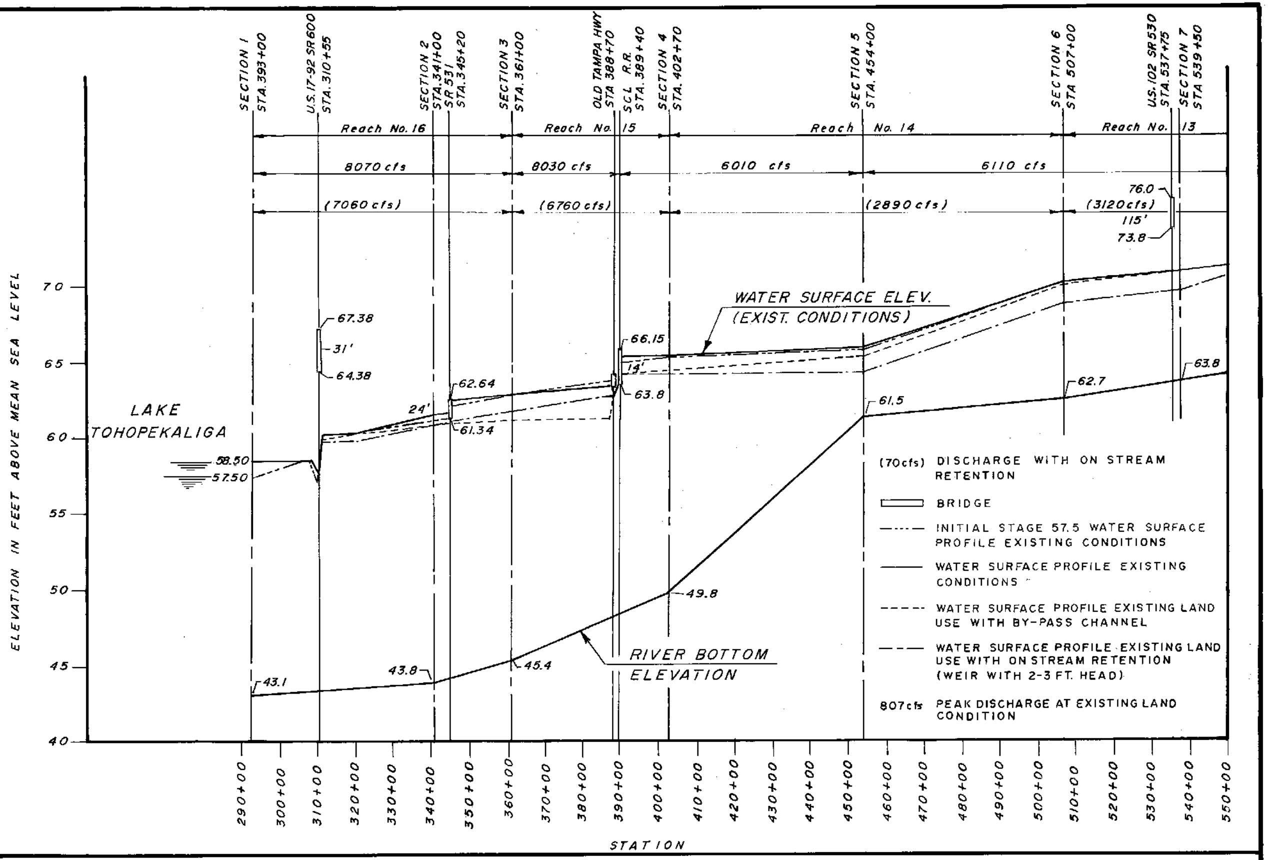
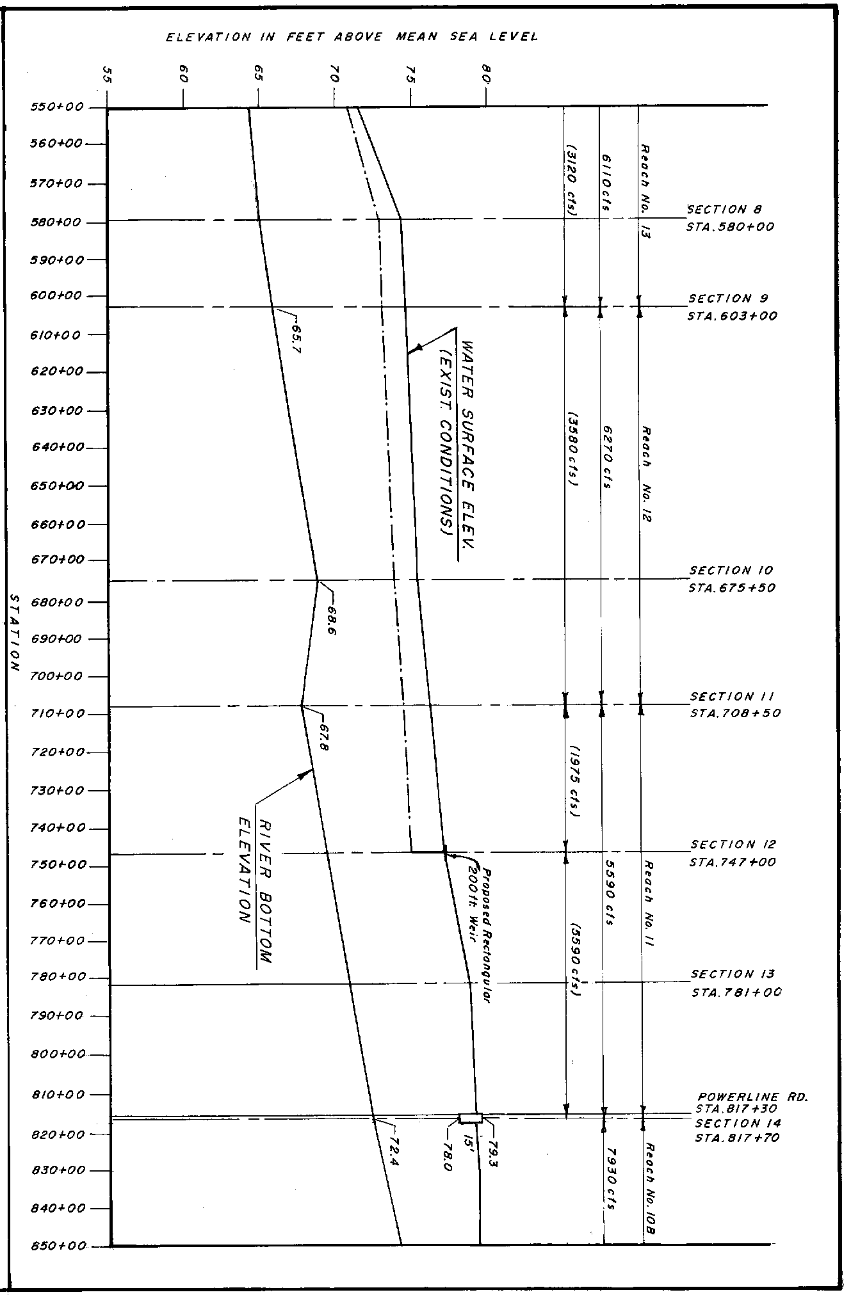


Figure 15a SHINGLE CREEK 100 YEAR FLOOD PROFILES

Figure 15b SHINGLE CREEK 100 YEAR FLOOD PROFILES



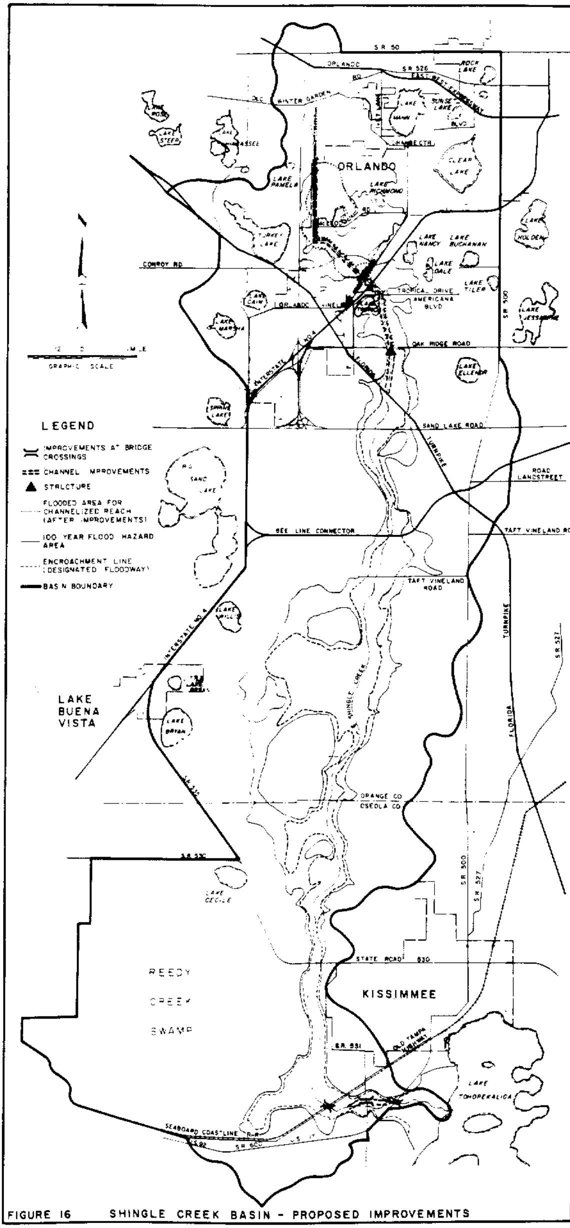


FIGURE 16 SHINGLE CREEK BASIN - PROPOSED IMPROVEMENTS

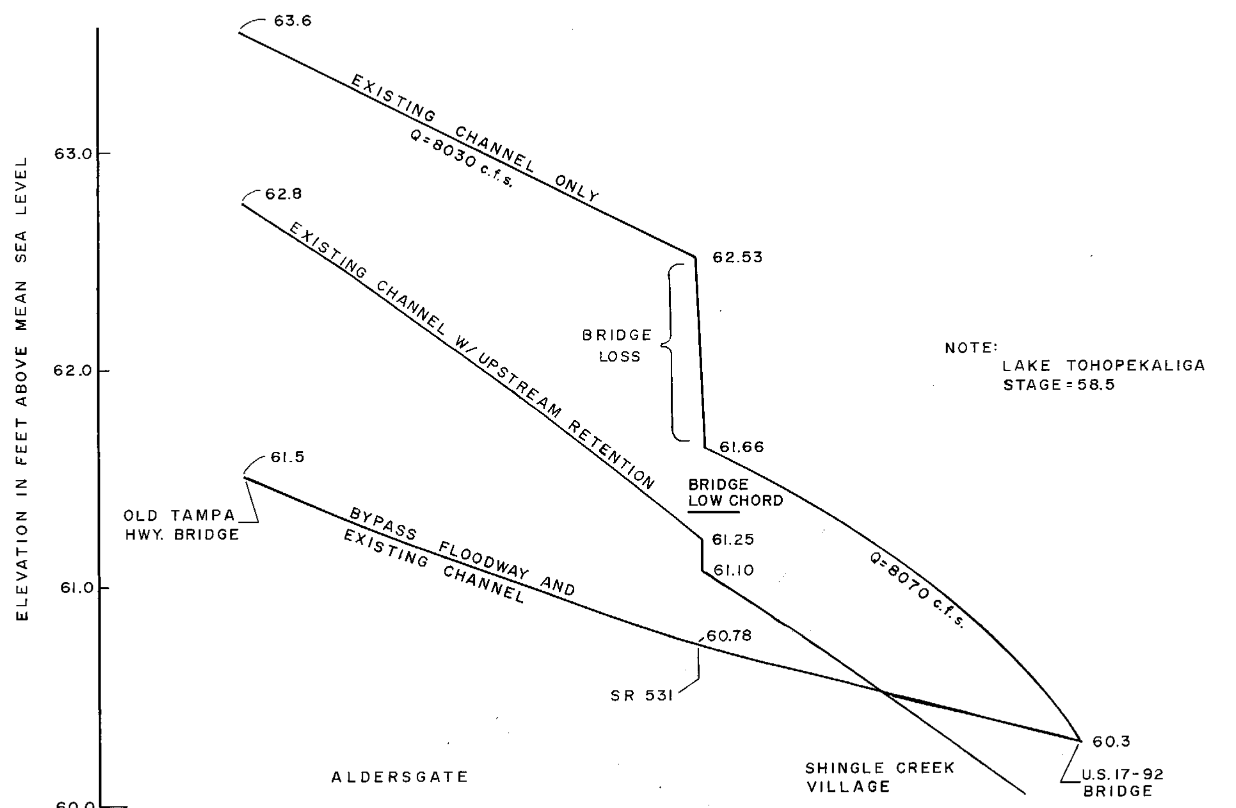


Figure 17 BACKWATER STAGES AT THE REACH BETWEEN ALDERSGATE AND SHINGLE CREEK VILLAGE RESULTING FROM A 100 YEAR FLOOD