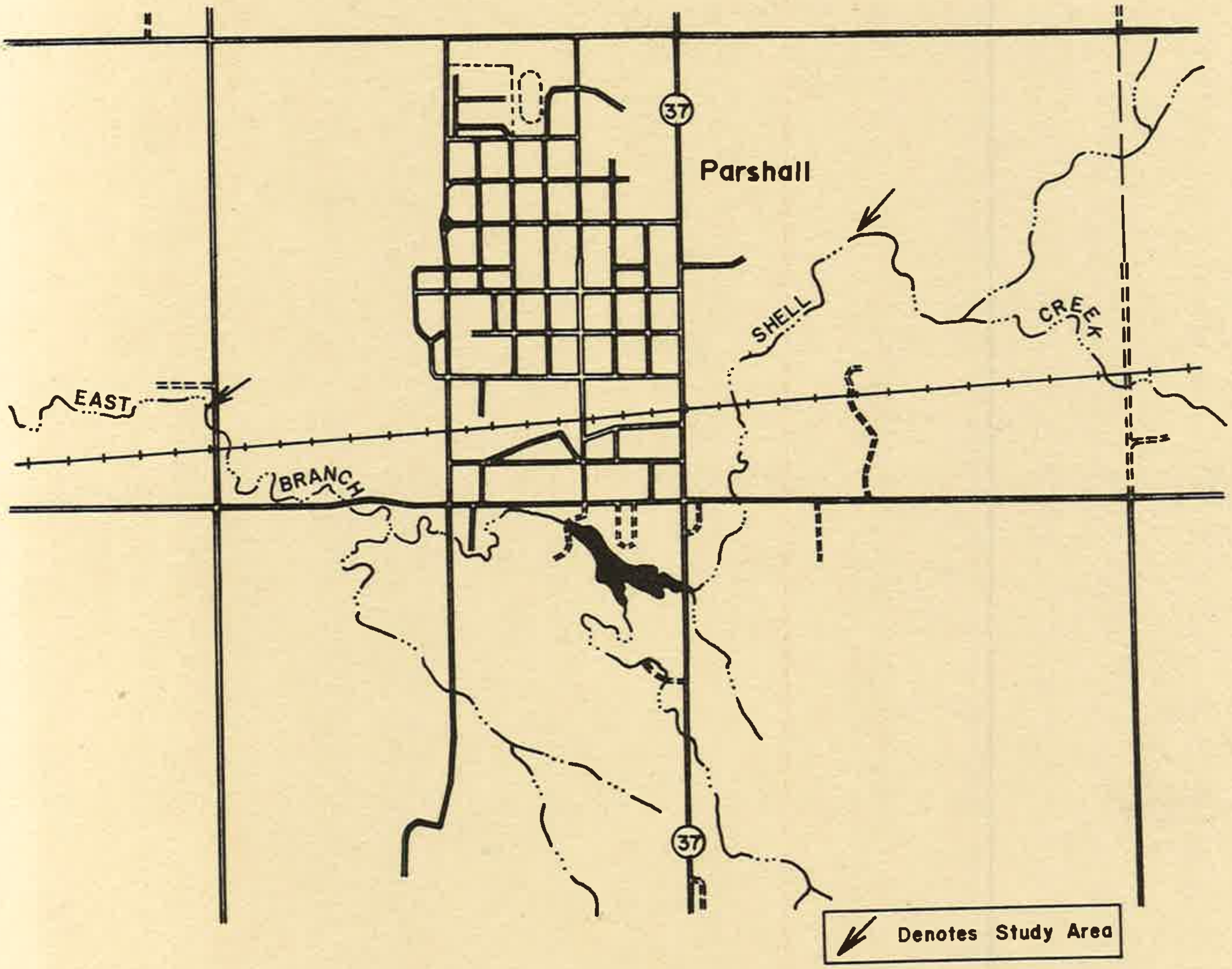
PRELIMINARY ENGINEERING REPORT

EAST BRANCH OF SHELL CREEK FLOOD CONTROL THROUGH PARSHALL

S.W.C. PROJECT NO. 1656



NORTH DAKOTA

STATE WATER COMMISSION

SEPTEMBER 1980

PRELIMINARY ENGINEERING REPORT

East Branch of Shell Creek Flood
Control Through Parshall

North Dakota State Water Commission
State Office Building
900 East Boulevard
Bismarck, North Dakota 58505

Prepared By:

Paul D. Urban
Paul D. Urban
Investigation Engineer

Submitted By:

David A. Sprynczynatyk
David A. Sprynczynatyk
Director of Engineering

Approved By:

Vernon Fahy
Vernon Fahy
State Engineer

TABLE OF CONTENTS

TABLES

TABLE OF CONTENTS (cont'd)

FIGURES

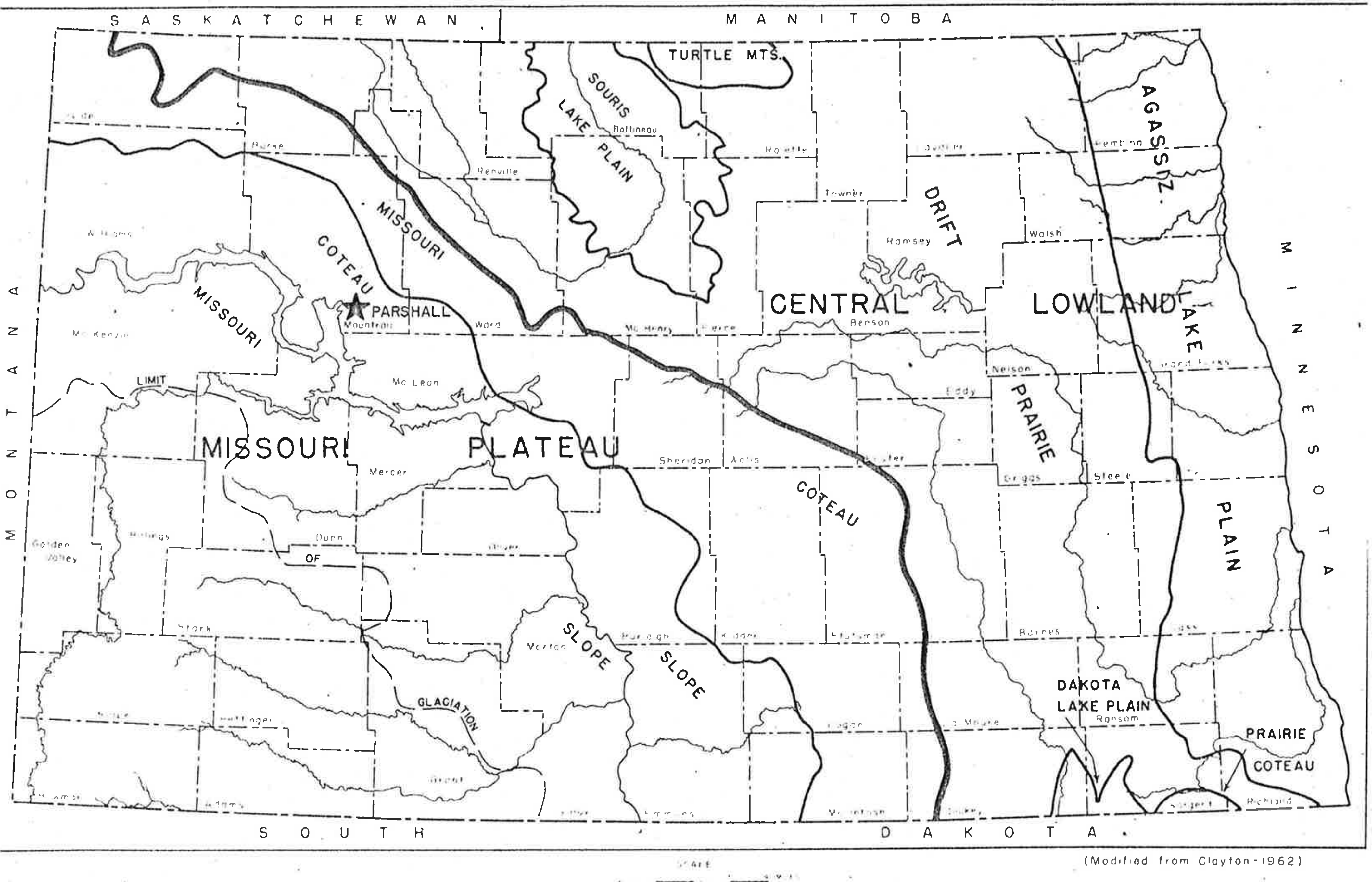
TABLE OF CONTENTS (cont'd)

Page

1. INTRODUCTION

This report on the flooding problems of the East Branch of Shell Creek near Parshall contains the results of a study conducted by the State Water Commission in cooperation with the Mountrail County Water Management Board. Figure 1 shows the general location of the study area. The study's major objective was to analyze the flow of the creek during flood conditions. From this analysis, locations affecting the free flow of the water were identified. Recommendations as to what could be done to improve the flood flow were presented along with an estimated cost for their implementation.

The engineering analysis of the East Branch of Shell Creek included a hydrologic study. Peak discharges for various locations were calculated. Water surface elevations at various control sections and crossings were calculated. This was done for both the existing and improved conditions at the crossings. From this information, water surface profiles were drawn for the existing and improved conditions. These show the improvement that can be realized from various alternatives that reduce water levels behind the crossings.



NORTH DAKOTA
LOCATION OF STUDY AREA
FIGURE 1

II. DESCRIPTION OF STUDY AREA

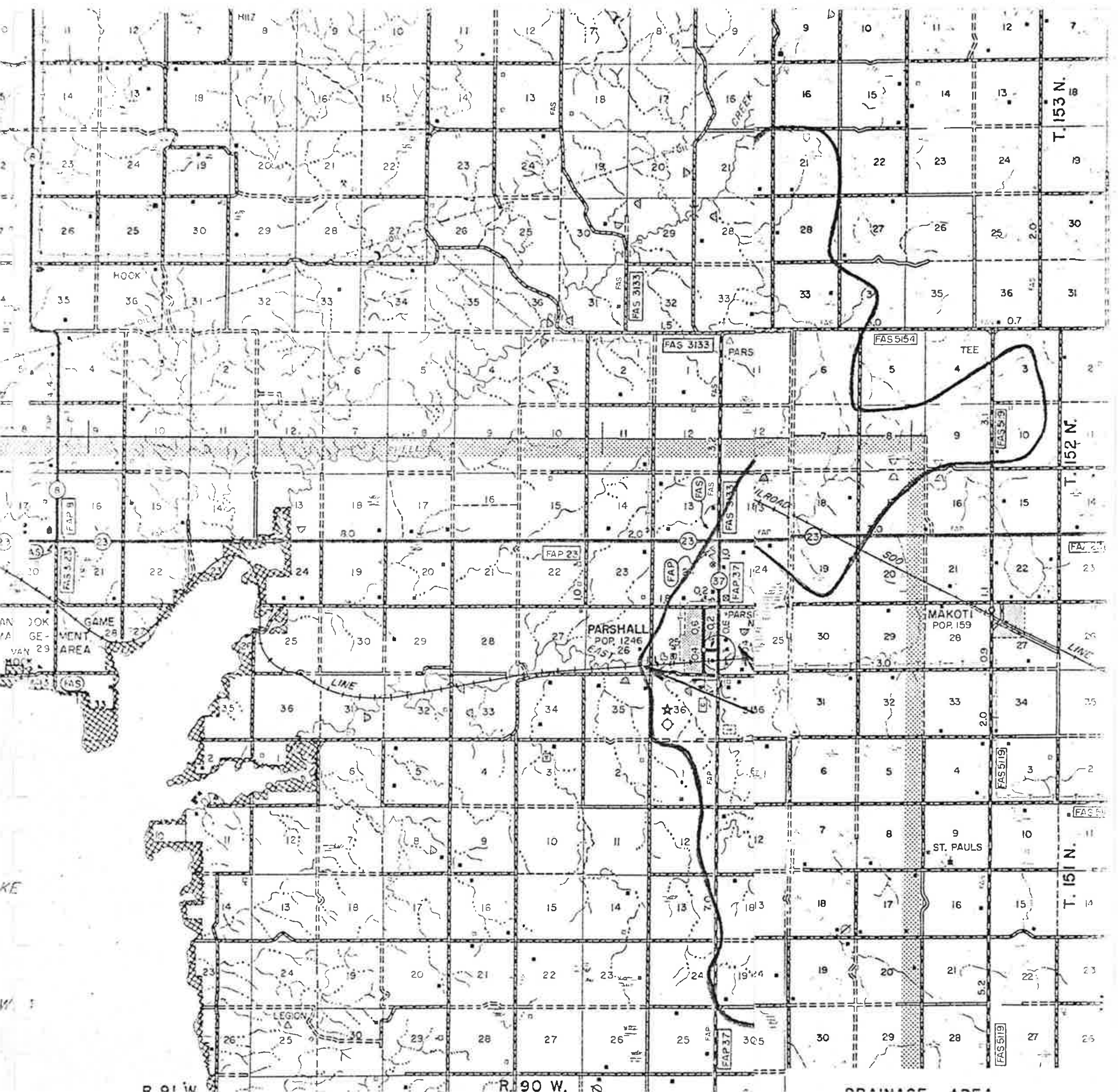
The portion of the East Branch of Shell Creek investigated by the State Water Commission consists of approximately 2.5 miles of river channel. Starting at the crossing located between Sections 25 and 26, Township 152 North, Range 90 West, the study area extends eastward to a point where the channel crosses the line common to the northwest and southwest quarters of Section 30, Township 152 North, Range 89 West (see Figure 2). This segment of the creek is part of the main channel which starts approximately 6.5 miles northwest of Plaza and outlets into Lake Sakakawea.

The East Branch of Shell Creek flows through a well defined valley. This valley was formed by runoff of glacial meltwaters. These melt-waters were of much larger quantities than the flows which can be expected from precipitation. Therefore, the channel occupies only a small portion of the valley. Through Parshall this valley widens out and has a much flatter flood plain than what exists a few miles up or downstream.

On the average the channel slopes at a rate of about 6.3 feet per mile. However, in the wider flood plain area just east of the Highway 37 crossing there are some relatively flat areas. One thousand feet down-stream from the highway crossing is small low head dam. This dam has a height of about six feet with a three foot high concrete weir spillway. The channel depth averages about five feet, however, in the flat area above the dam the channel averages only about 2.5 feet deep. Capacity for an average cross section along the creek is about 550 cfs. In the area having the shallow channel the capacity is only 200 cfs.

III. HISTORICAL BACKGROUND

The flooding problem along the East Branch of Shell Creek in the Parshall area was first brought to the attention of the State Water Commission in October, 1972. At this time the Mountrail County Water Management Board requested that a study be made of the creek. The proposed study was to extend from Lake Sakakawea to about five miles



Steve Hoetzer and C.P. Nelson of the State Water Commission met with the Mountrail County Water Management Board and about 20 landowners on February 17, 1976. At that time C.P. Nelson suggested that a general idea of the creek problem area might be attained by using railroad plan and profile. This information was obtained in March, 1976 and was drawn up using scaled distances along the creek bed for stationing.

From this information, C.P. Nelson made some recommendations on the scope of the proposed investigation. He mentioned that the dam and pump station at the refuge road be given a low priority since it could not hold back a large enough proportion of the flood waters to reduce the flow. The upstream improvements were also recommended to be given a lesser priority. Because of the many railroad crossings, no flood retention dam could be built on the main channel. Possible relief due to construction of detention reservoirs on branch coulees could be investigated. However, C.P. Nelson mentioned that without U.S. Geological Survey topographic maps this would be difficult and costly. These dams would probably not reduce the cost of needed downstream improvements enough to justify them. It was recommended that the first phase of the East Branch of Shell Creek investigation consist of a survey from Lake Sakakawea to the junction of the lateral coulees south and northwest of Parshall.

On June 30, 1976, an investigation agreement between the State Water Commission and the Mountrail County Water Management Board was signed. The investigation was to survey the creek from Lake Sakakawea to a point two miles east of Parshall. Also included was a feasibility study for the correction of inadequate crossings, channel capacities, and existing channel changes. Improvements were to provide adequate flood control for east Parshall and adjacent farmland. They were also to provide pasture drainage both upstream anddownstream from Parshall.

A meeting was held at Parshall in December of 1978 to determine the areas in which the study of the East Branch of Shell Creek should be directed. The flooding problem was attributed to:

1. Inadequate capacity of the bridges on Highway 37.
2. Inadequate capacity of the Parshall Dam spillway.
3. Questionable capacity of the culverts through the east-west road just east of Parshall and east of Highway 37.
4. Runoff through the town from north of Parshall.
5. Creek meanders.
6. Creek obstructions and siltation.

The city wished to retain the dam unless it was absolutely necessary that it be removed.

In the spring of 1979, Parshall experienced severe flooding on its east and south sides. Dan Asplund of the State Water Commission went up to inspect the area. It appeared that if the structures around Parshall allowed the runoff waters to get downstream faster, the flooding would be less severe. He mentioned that some flooding has always occurred on the east and south sides of the city. Therefore, it was suggested that the project be designed to lessen the severity rather than alleviate all flooding. It was noted that the land to the west of Parshall is of less value than that land immediately around the city. Allowing the water to reach the area west of Parshall faster would probably cause more flooding to the west but would reduce the flood's severity around Parshall.

Taking this into account, it was decided to study the effects of channel improvement to allow flood waters to pass through Parshall faster.

The highway bridge across the East Branch of Shell Creek was washed out by the 1979 flood. It was replaced with a 65 foot long reinforced concrete bridge having a trapezoidal flow area and a 30 foot bottom width. The south bridge was replaced with culverts. Excess flow from this south tributary now flows north along the highway ditch to the new bridge.

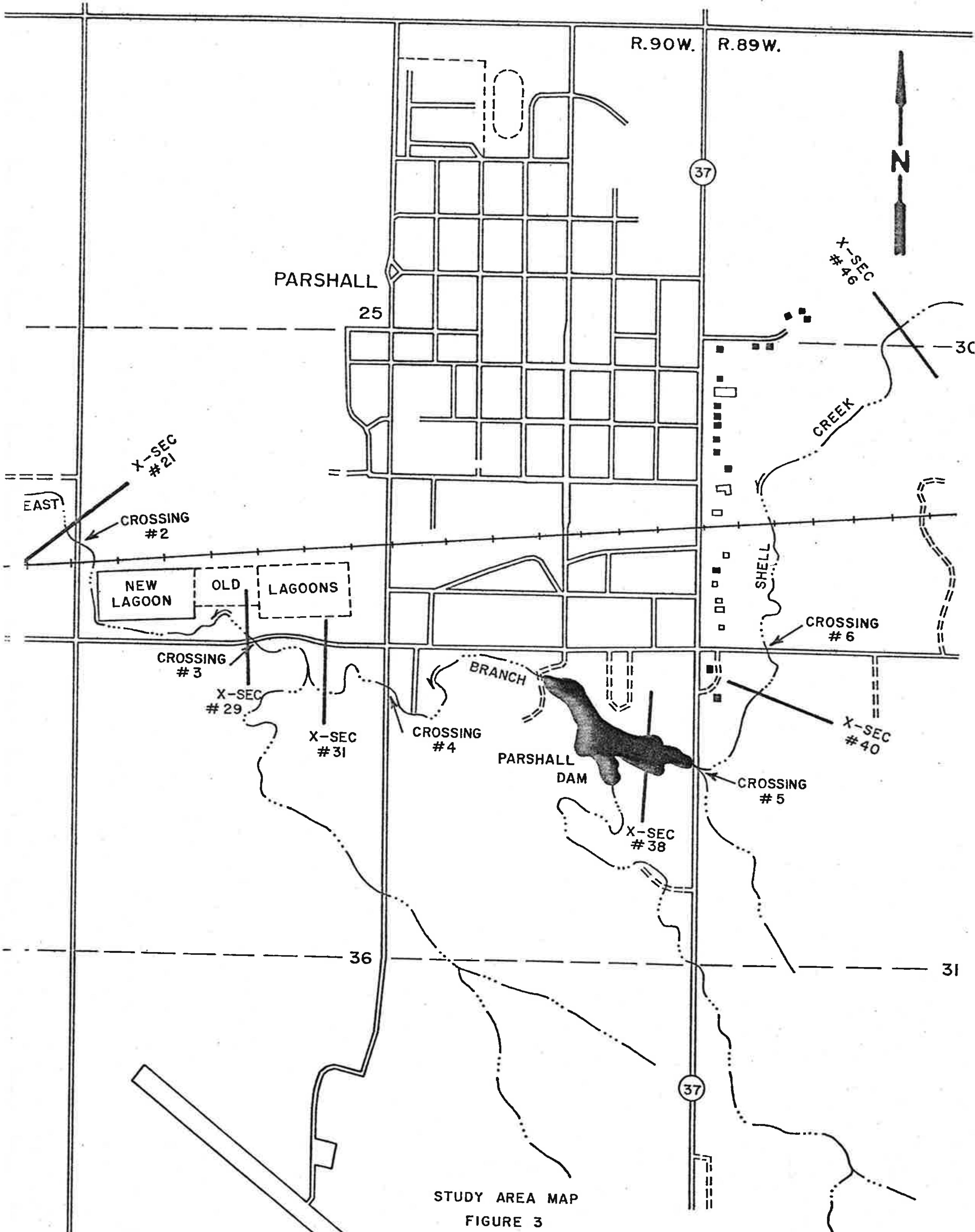
IV. ENGINEERING ANALYSIS

Existing Conditions

The East Branch of Shell Creek is a small stream that flows through a glacial meltwater floodplain. Above and below the City of Parshall a few miles, the creek is fairly well contained in the central portion of this well defined valley. However, as the creek comes into the Parshall area from the east, the valley widens. In this area there exists a wide and very flat floodplain. This condition exists from about 1.5 miles straight east of Parshall to about 2.5 miles straight west of the city. Most of the flooding problems, as far as damage potential is concerned, are in the area along the east and south sides of Parshall. This is due to the existence of developed areas in the fringe of this floodplain. Flooding upstream and downstream affect mostly pastureland.

On the east side of Parshall there are houses on the edge of the floodplain. In 1979 floodwaters threatened them. South of the railroad there is a house and numerous grain bins. These grain bins were flooded in 1979. Water also backed up along the railroad tracks and flooded a vacant lot and a grain elevator.

Figure 3 shows the limits of the study area. On this map the roadway crossings and locations of various cross sections used in the analysis are identified. Seven crossings are in the 2.5 mile study area. Two of these are railroad crossings. Roadway crossing number one is downstream from the study area and is not shown on the map. The railroad crossings along the entire length of the creek are all timber trestles, except for some in the upper reaches of the creek. These are generally



STUDY AREA MAP
FIGURE 3

large enough to have little or no effect on the floodwaters. The same is not true for the roadway crossings. Most of these are corrugated metal pipe culverts. These are inadequate and can handle no more than a three or four year event. Crossings number two and six used to be timber bridges but were replaced with culverts. Culverts are adequate in the creek's upper reaches. However, from a point about six miles straight east of Parshall and extending downstream to Lake Sakakawea, the drainage area is large enough to require bridges on all crossings. As a result, these crossings are frequently overtopped. Table 1 lists what exists in the roadway crossings found in the study area.

Most of the culverts are still in good condition. Some of the culverts at crossings number two and four have bent inlets and outlets. However, this is not too serious. The culverts at crossing number three are deformed and have been squashed in the middle. Crossings two, three, and four have debris in the culvert or debris built up a few feet away from the outlet. This is no doubt a result of the 1979 flood. Many of the culverts have been placed in the crossings haphazardly. Some slope the wrong way. These all tend to reduce the expected efficiency of the crossings, especially at lower flows. At higher flows these culverts will back up water, develop pressure flow and eventually overtop the roadway.

Table 2 is a list of the roadway crossings in the study area and their estimated capacities.

TABLE 1

Inventory of the Existing Roadway Crossings

Crossing Number	Type of Crossing	In Elevation	Out Elevation
2	6' Corr. Metal Pipe	1908.7	1908.6
	8' Corr. Metal Pipe	1906.9	1906.9
	8' Corr. Metal Pipe	1906.7	1906.7
3	6' Corr. Metal Pipe	1911.7	1912.0*
	6' Corr. Metal Pipe	1911.1	1911.1
	6' Corr. Metal Pipe	1911.7	1911.4
4	6' Corr. Metal Pipe	1913.2	1913.8*
	7' Corr. Metal Pipe	1912.7	1912.0
	7' Corr. Metal Pipe	1912.5	1913.0*
5	65' Bridge with 30' bottom width Trapezoidal Area = 425 S.F.	1920.0	1920.0
6	6' Corr. Metal Pipe	1920.0	1920.0
	5.5'x7.5' Corr. Metal Eliptical Pipe	1920.9	1920.1
	5.5'x7.5' Corr. Metal Eliptical Pipe	1921.0	1921.6*
	5.5'x7.5' Corr. Metal Eliptical Pipe	1922.1	1922.1

* Culverts sloping the wrong way

TABLE 2

Crossing Capacities

Crossing Number	Capacity	Head
2	1200 cfs	2.2 ft.
3	900 cfs	3.5 ft.
4	300 cfs	2.0 ft.
5	2200 cfs	6.7 ft.
6	900 cfs	2.3 ft.

The Highway 37 bridge was an old concrete bridge with an eleven foot by twenty foot opening. This washed out in 1979 and was replaced by a 65 foot bridge having a 30 foot bottom width and a free flow area of 425 square feet. The channel bottom of the old bridge was 5.5 feet deeper than that of the existing bridge. When the new bridge was constructed, its deck elevation was higher and the old sag was filled in. A new sag in the highway profile, slightly higher, was put in south of the bridge.

As is true with most small streams in North Dakota, the channel of the East Branch of Shell Creek has the capacity to handle only about a two or three year event. On the average, the channel in the study area is about five to six feet deep having a capacity of about 550 cfs. There is a small dam located west of the Highway 37 crossing. The area above this dam and to the east of Parshall is in a wide flat floodplain, as mentioned before. Here the channel is only 2.5 feet deep and has a capacity of only about 200 cfs. In this area the channel gradient has many flat spots causing the channel to fill with stagnant water. Therefore, the bottom of this floodplain experiences flooding in all but the driest years.

If flows increase above those listed in Table 2, the channel downstream from a crossing backs up water either from natural channel capacity limits or capacity limits of a downstream crossing. As these tailwaters rise, the available head at the crossing decreases. This causes the flow through the culverts to lessen causing the water to back up behind the crossing and eventually overtop the road. The pool of water formed behind the crossing may increase the tailwater at the next upstream crossing if it is close enough. Then the same cycle of events happens at this crossing. Therefore, when there are many crossings next to each other, as in the study area, they tend to reduce the capacity of each other.

Many of the roadways are overtopped by the ten year flood. This is due not only to the limited channel and crossing capacities but also by the small elevation difference between some of the roads and their ditches or the surrounding land. In some cases the road is flooded by backwater from a downstream constriction.

The new city waste water treatment lagoon, built in 1979, was constructed in an oxbow of the East Branch of Shell Creek. This new lagoon is located west of the existing cells between crossings number two and three. To accommodate flows from the creek, a very small diversion ditch was built to divert them around the new lagoon. This ditch is approximately 10 feet wide at the bottom, and five feet deep and has a capacity of 400 cfs. The side slopes are very steep. The north side of the channel is the toe slope for a portion of the south dike of the new lagoon. Even though it is riprapped, the inadequacy of the diversion ditch threatens to erode this dike during flows over about 400 cfs.

This is about a two or three year event. The limited capacity of this channel will back up water even during lower flows. This diversion channel has a capacity only slightly smaller than the rest of the creek in the area, however, its location between the road and the lagoon dikes causes a constriction. Whereas in other places along the creek the water can spread out, here it is more confined.

HYDROLOGIC INVESTIGATION

The TR-20 computer program developed by the U.S. Soil Conservation Service was used to determine peak discharges and corresponding flow volumes for various frequency storms. The program formulates a mathematical model of the watershed based on the following input data: the rainfall distribution, type of soil, soil moisture conditions, land use, time of concentration, hydraulic characteristics of the channels and the size of the drainage area. An analysis of the drainage basin above the study area was made considering both rainfall and snowmelt. Rainfall is given as the number of inches of runoff that would occur over a 10 day period.

The drainage basin above the study area has 140 square miles that contribute runoff to the East Branch of Shell Creek. Above the study area there are 20 miles of river channel. In this distance there is an elevation difference of 133 feet. Land use in the drainage area was estimated and broken down as follows:

Small Grains	60.0%
Fallow	30.0%
Rangeland	6.2%
Roads & Municipal	3.6%
Farmsteads	0.2%
	100.0%

The TR-20 computer model generated hydrographs for both rainfall and snowmelt events with recurrance intervals of 10, 25 and 100 years. These hydrographs were generated at three locations in the study area.

Each roadway crossing was studied to determine how it would handle the flows from the 10, 25 and 100 year floods. To do this, peak flows were taken from the generated hydrographs and used to estimate tailwater and backwater elevations at each crossing. Table 3 shows the peak flow at each roadway crossing for various frequency floods. The greater of the rainfall or snowmelt event was chosen at this peak flow.

WATER SURFACE PROFILE ANALYSIS

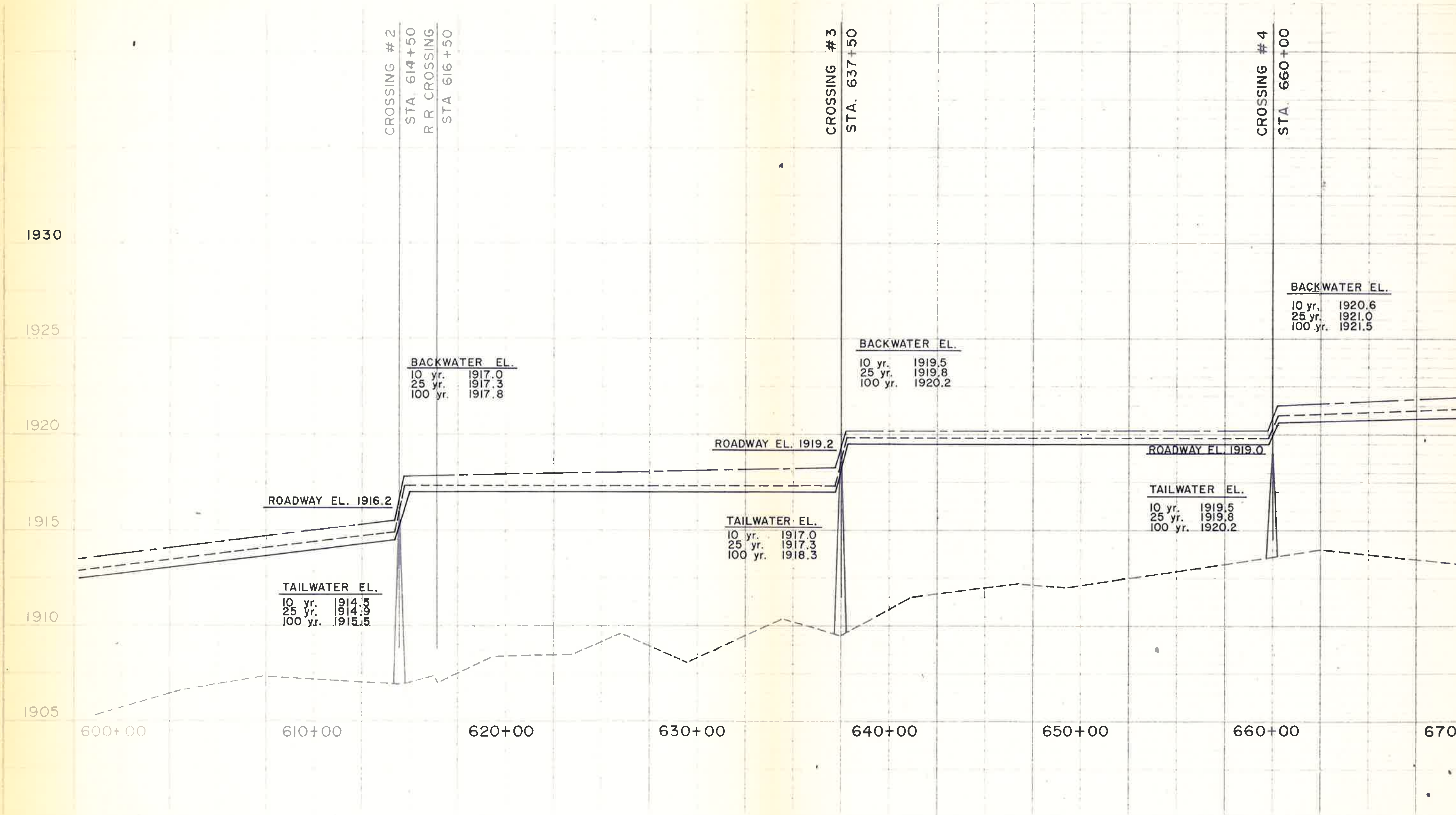
In order to evaluate the existing conditions of the East Branch of Shell Creek, as well as the conditions after making certain improvements, water surface elevations at various cross sections and crossings were calculated. These are shown on Figure 4. This process started by choosing a cross section just downstream from crossing number two, the first one on the downstream end of the study area. Since there are no crossings a short distance downstream from here to back up flows, calculating the water level needed to pass various flows through the cross section resulted in a tailwater elevation for crossing number two. Using pipe flow equations, the amount of flow through the culverts was calculated.

Flow through the pipes varies with the difference in elevation between the water levels on the upstream and downstream sides of the crossing. If the upstream level is at the top of the road and the pipes are unable to pass the expected flow, the water will back up more and

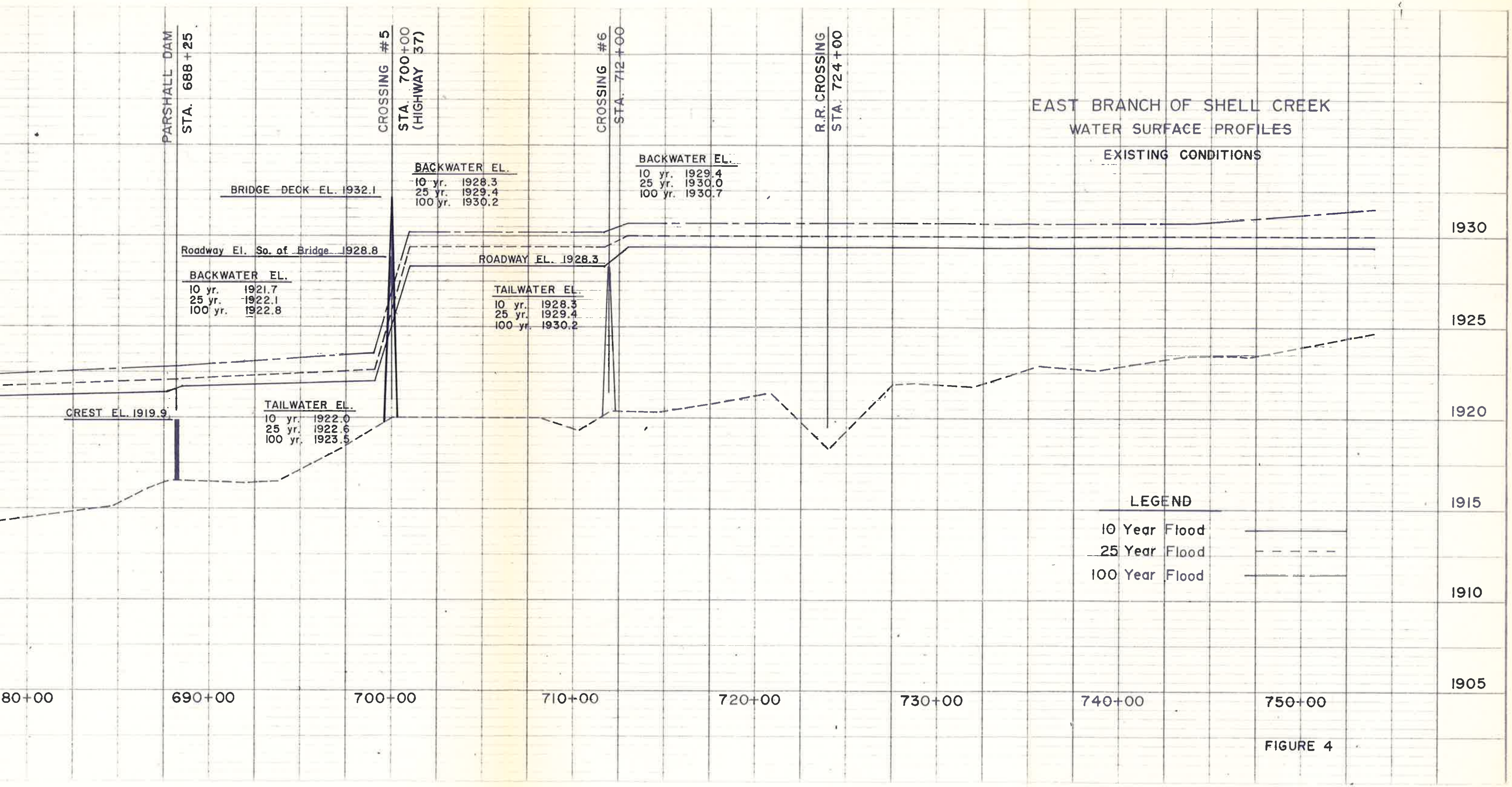
TABLE 3

Peak Flows

Crossing Number	Peak Flow	Causing Event
2	10 yr. 1942 cfs	snowmelt
	25 yr. 2726 cfs	snowmelt
	100 yr. 4292 cfs	rainfall
3	10 yr. 1942 cfs	snowmelt
	25 yr. 2726 cfs	snwomelt
	100 yr. 4292 cfs	rainfall
4	10 yr. 1938 cfs	snowmelt
	25 yr. 2720 cfs	snowmelt
	100 yr. 4260 cfs	rainfall
Parshall Dam	10 yr. 1935 cfs	snowmelt
	25 yr. 2715 cfs	snowmelt
	100 yr. 4234 cfs	rainfall
5	10 yr. 1935 cfs	snowmelt
	25 yr. 2715 cfs	snowmelt
	100 yr. 4234 cfs	rainfall
6	10 yr. 1480 cfs	snowmelt
	25 yr. 2093 cfs	snowmelt
	100 yr. 3220 cfs	snowmelt



CROSSING #2
STA 614+50
R R CROSSING
STA 616+50
CROSSING #3
STA. 637+50
CROSSING #4
STA 660+00
1930
1925
1920
1915
1910
1905
BACKWATER EL.
10 yr. 1917.0
25 yr. 1917.3
100 yr. 1917.8
ROADWAY EL. 1916.2
TAILWATER EL.
10 yr. 1914.5
25 yr. 1914.9
100 yr. 1915.5
ROADWAY EL. 1919.2
TAILWATER EL.
10 yr. 1917.0
25 yr. 1917.3
100 yr. 1918.3
BACKWATER EL.
10 yr. 1919.5
25 yr. 1919.8
100 yr. 1920.2
ROADWAY EL. 1919.0
TAILWATER EL.
10 yr. 1919.5
25 yr. 1919.8
100 yr. 1920.2
BACKWATER EL.
10 yr. 1920.6
25 yr. 1921.0
100 yr. 1921.5
600+00
610+00
620+00
630+00
640+00
650+00
660+00
670



EAST BRANCH OF SHELL CREEK
WATER SURFACE PROFILES
EXISTING CONDITIONS

FIGURE 4

run over the road. Then the flow at the crossing is a combination of pipe flow through the culverts and weir flow over the road. From this the backwater elevation built up behind the crossing was calculated.

Another cross section was chosen just downstream of the next upstream road crossing, number three. Again the water level needed to pass various flows through the cross section was calculated. This water surface elevation was compared to the backwater elevation built up behind the downstream crossing, number two. The higher of these was used as the tailwater elevation for crossing number three. When the backwater elevation is higher than the water surface elevation needed to pass the flow through the cross section, the pool formed by the downstream crossing controls the tailwater elevation for the upstream crossing. The backwater elevation at crossing number three was calculated. Then the same process was followed for the remaining upstream crossings in the study area.

From this series of calculations the tailwater and backwater elevations were found for all the roadway crossings in the study area. These were plotted on a drawing of the streambed profile (see Figure 4). This drawing shows how high the water will back up behind the various crossings and how much water, if any, will flow over the road.

To find the water level needed to pass the various flows at the cross sections, Mannings equation was used. The equation is usually seen in the following form:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

V = Velocity
n = Roughness coefficient
R = Hydraulic radius
S = Slope

Multiplying the velocity by the area of water in the cross section will give the flow rate. The hydraulic radius is the cross sectional area of water divided by the wetted perimeter. To take into account the irregularities and other conditions that restrict flow, the roughness coefficient is used.

Flow through pipes happens in one of three ways. When the water surface is below the top of the pipe there is open channel flow. This is estimated by using Mannings equation. Two things can happen when the water surface is above the top of the pipe. The flow may be controlled either by the pipe itself or by the opening at the entrance, orifice control. Flow through the pipe is figured for both conditions and the smallest value is used. These relationships for pipe flow are as follows:

For pipe control:

$$Q = A \cdot \sqrt{\frac{64.4 \cdot h_L}{K_e + 1 + \frac{29 \cdot n^2 \cdot L}{R^{4/3}}}}$$

Q = Flow
A = Area
K_e = Entrance Coefficient
h_L = Headless through pipe
n = Roughness coefficient
R = Hydraulic radius
L = Pipe length

For orifice control:

$$Q = C \cdot A \cdot \sqrt{64.4 \cdot h}$$

c = orifice coefficient
h = head on orifice

Looking at the water surface profiles for the existing conditions along the East Branch of Shell Creek shows that all the crossings in the study area, except the highway bridge at crossing number five, are overtopped by the ten year flood. Table 2 shows the crossing capacities. As mentioned before, higher flows will result in water backing up causing the available head of the crossings to be reduced. This, of course, reduces the flow through the culverts. The water surface profiles clearly show how the crossings are causing this back up.

Crossing number four is flooded by the backwater pool behind crossing number three. Most of the flow must overtop the crossing with the road acting as a submerged weir. This weir action provides at least a little head to enable some flow through the culverts.

The Parshall Dam seems to back up little water during flows above the 10 year flood. High flows will bypass the dam. There is only a half foot difference between the crest of the spillway and the land to the south of the dam. This means that the spillway will handle a flow of about 60 cfs before the water will flow around the south end of the dam. Figure 10 is a cross sectional view of the dam. It shows how the flows will easily bypass the dam by flowing around the south end and then flow generally westward until they enter the creek channel again just upstream from crossing number four (see Figure 3).

Crossing number five, the Highway 37 bridge, by far causes the largest back up of water. Being as this water affects the east side of Parshall, this is an important crossing. Backwater from it inundates crossing number six. Therefore, as with crossing number four, the culverts can handle very little flow. The water must flow over the road at crossing number 6 with the roadway acting as a submerged weir. The water backed up behind crossing number six causes the flooding problems on Parshall's east side. It can be reduced the most by reducing the backwater elevation behind crossing number five. The increase in water surface elevation across number six is small.

V. ALTERNATIVES

This section will discuss different ideas on things that can be done to help reduce the effects of flooding in the study area. The merits of each alternative will be looked at. Some of the alternatives concern changes throughout the entire study area while others deal with specific problem areas. They are presented with the idea that they each achieve certain results. No one alternative will alleviate the total problem. The summary will discuss what the engineering staff of the State Water Commission believes to be the most feasible alternative or group of alternatives. Alternatives discussed are the addition of new culverts, the construction of bridges, removing Parshall Dam, diverting the south tributary, dikes, removal of structures, and floodplain management.

CULVERTS

As mentioned earlier, the corrugated metal pipe crossings are inadequate. They were analyzed to determine how they affect various flows. The ten year flood was used as the design flow. From this the size and number of pipes required to pass the design flow without overtopping the road was calculated. Table 4 lists the recommended improvements to be installed at each crossing for this alternative. It is estimated that this alternative will cost about $224,000. A breakdown of these costs is listed in Table 5. The effects of the new crossings on various flows was also estimated. Figure 5 shows the water surface profile after installing these new culverts. Table 6 lists the tailwater and backwater elevations for both the existing and improved conditions.

TABLE 4

Culvert Alternative Proposed Improvements

CROSSING	IMPROVEMENTS
2	Remove Existing 6' C.M.P. Relay the two existing 8' C.M.P. Add 4-8' C.M.P. Widen approaches to the crossing
3	Remove the two existing 6' C.M.P. Remove the existing 5.5' C.M.P. Add 8-6' C.M.P. Widen approaches to the crossing Straighten jog down from the outlet.
4	Relay the two existing 7' C.M.P. Relay the existing 6' C.M.P. Add 2-6' C.M.P.
5	Leave the existing bridge as it is.
6	Existing 6' C.M.P. to remain The two 5.5' x 7.5' H.E.C.M.P. to remain. Relay the two 5.5' x 7.5' H.E.C.M.P. sloping the wrong way. Add 5 - 5.5' x 7.5' H.E.C.M.P.

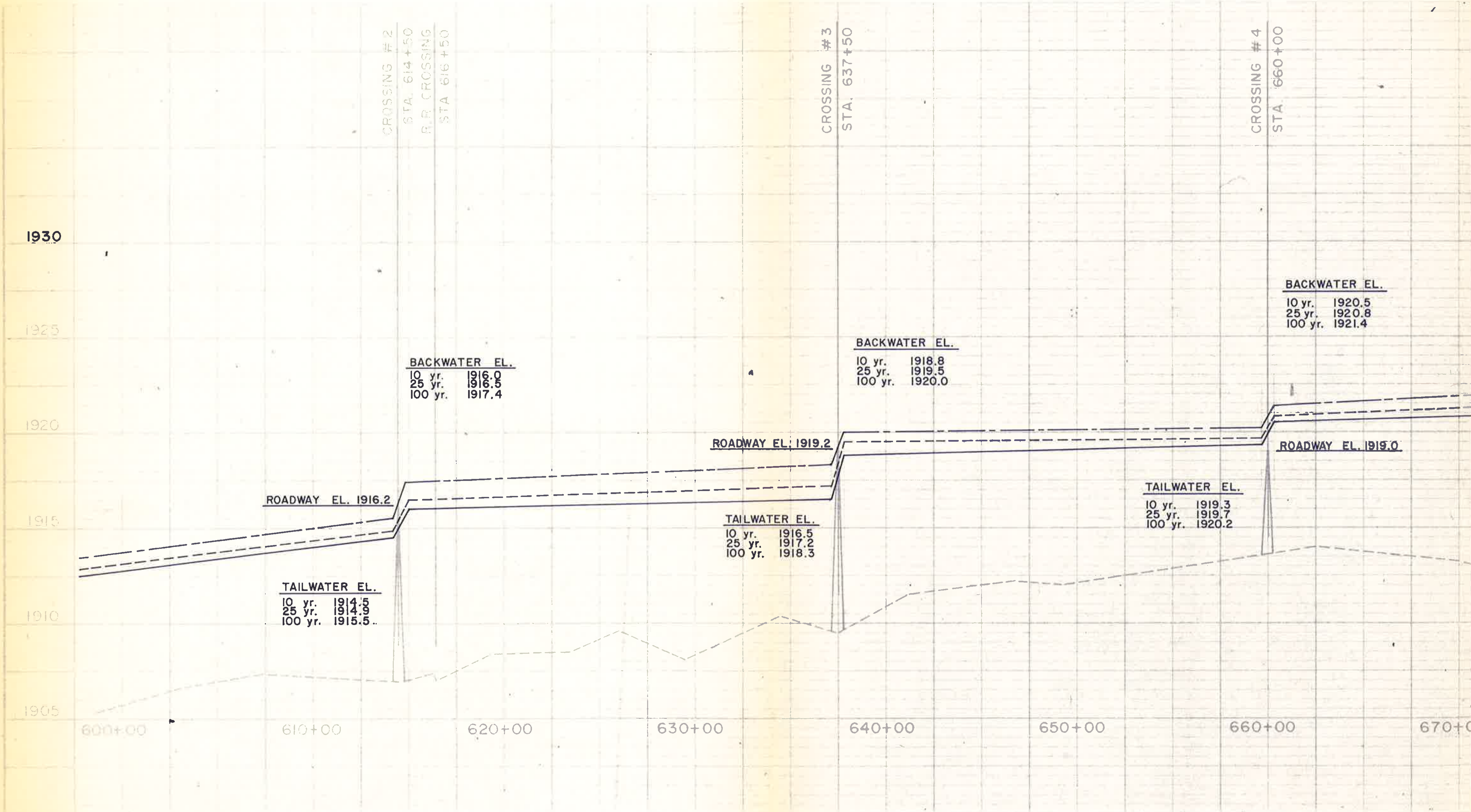
C.M.P. - Corrugated Metal Pipe
H.E.C.M.P. - Horizontal Eliptical Corrugated Metal Pipe
The pipe should be made of aluminized steel.

TABLE 5

Cost Estimate
Culvert Alternative

ITEM	UNIT	QUANTITY	UNIT COST	COST
Remove 6' C.M.P.	L.F.	150	7	1,050
Remove 5.5' x 7.5' H.E.C.M.P.	L.F.	65	7	455
Relay 6' C.M.P.	Ea.	50	45	2,250
Relay 7' C.M.P.	Ea.	100	48	4,800
Relay 8' C.M.P.	Ea.	100	50	5,000
Relay 5.5' x 7.5' H.E.C.M.P.	Ea.	130	48	6,240
6' Dia. C.M.P. (Aluminized Steel)	L.F.	500	114	57,000
8' dia. C.M.P. (Aluminized Steel)	L.F.	200	134	26,800
5.5' x 7.5' H.E.C.M.P.	L.F.	325	122	39,650
Excavation	C.Y.	29,000	100	29,000
Seeding	Acre	1	200	200

SUBTOTAL	$172,445
Engineering	17,185
Contract Administration	17,185
Contingencies	17,185
TOTAL COST	$224,000


CROSSING #2
STA. 614+50
R.R. CROSSING
STA 616+50
CROSSING #3
STA. 637+50
CROSSING #4
STA 660+00
1930
1925
1920
1915
1910
1905
BACKWATER EL.
10 yr. 1916.0
25 yr. 1916.5
100 yr. 1917.4
ROADWAY EL. 1916.2
TAILWATER EL.
10 yr. 1914.5
25 yr. 1914.9
100 yr. 1915.5
BACKWATER EL.
10 yr. 1918.8
25 yr. 1919.5
100 yr. 1920.0
ROADWAY EL. 1919.2
TAILWATER EL.
10 yr. 1916.5
25 yr. 1917.2
100 yr. 1918.3
BACKWATER EL.
10 yr. 1920.5
25 yr. 1920.8
100 yr. 1921.4
ROADWAY EL. 1919.0
TAILWATER EL.
10 yr. 1919.3
25 yr. 1919.7
100 yr. 1920.2
600+00
610+00
620+00
630+00
640+00
650+00
660+00
670+00

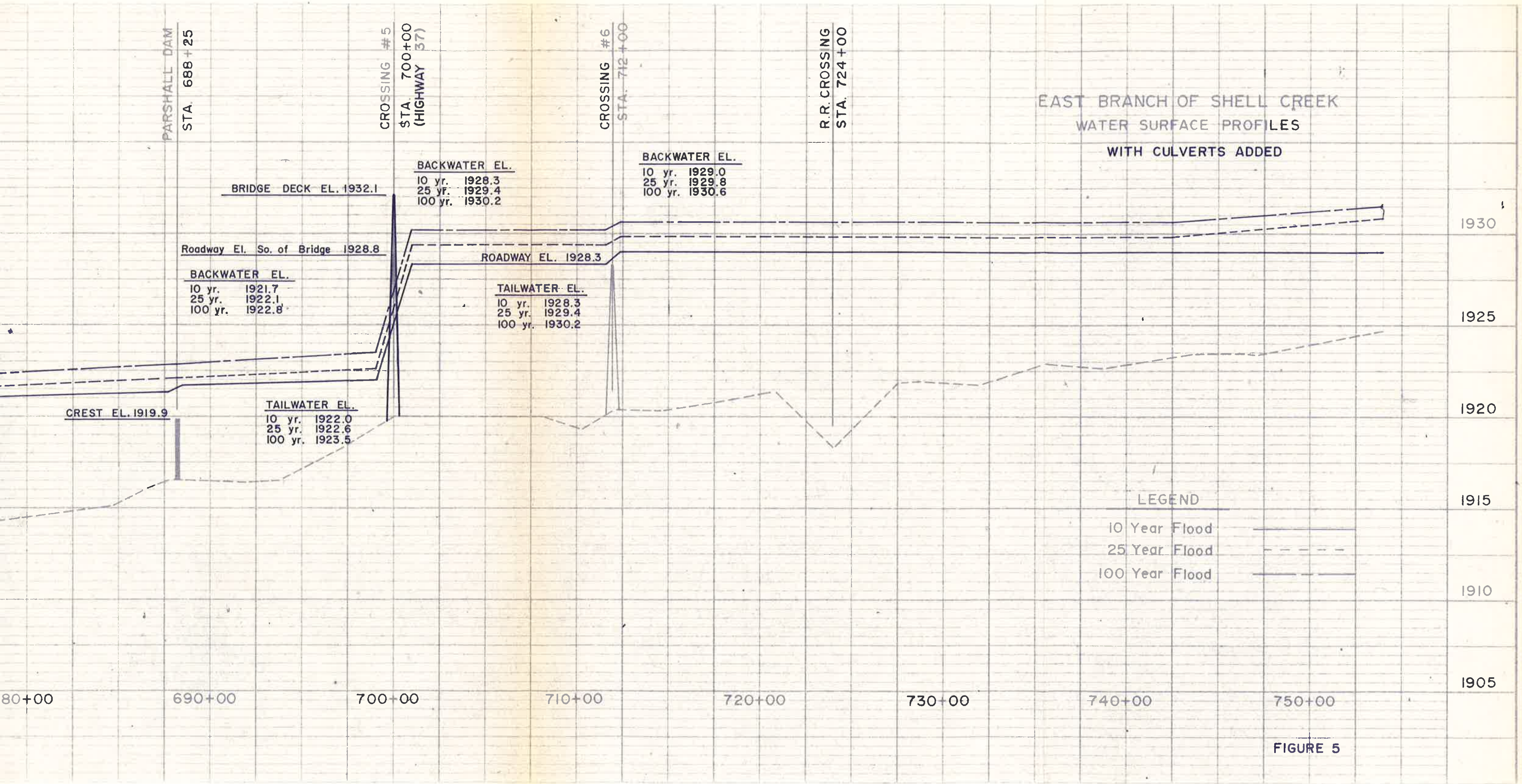

EAST BRANCH OF SHELL CREEK
WATER SURFACE PROFILES
WITH CULVERTS ADDED
PARSHALL DAM
STA. 688+25
CROSSING #5
STA. 700+00
(HIGHWAY 37)
CROSSING #6
STA. 712+00
R.R. CROSSING
STA. 724+00
BRIDGE DECK EL. 1932.1
BACKWATER EL.
10 yr. 1928.3
25 yr. 1929.4
100 yr. 1930.2
BACKWATER EL.
10 yr. 1929.0
25 yr. 1929.8
100 yr. 1930.6
Roadway El. So. of Bridge 1928.8
ROADWAY EL. 1928.3
BACKWATER EL.
10 yr. 1921.7
25 yr. 1922.1
100 yr. 1922.8
TAILWATER EL.
10 yr. 1928.3
25 yr. 1929.4
100 yr. 1930.2
CREST EL. 1919.9
TAILWATER EL.
10 yr. 1922.0
25 yr. 1922.6
100 yr. 1923.5
LEGEND
10 Year Flood
25 Year Flood
100 Year Flood
1930
1925
1920
1915
1910
1905
80+00
690+00
700+00
710+00
720+00
730+00
740+00
750+00
FIGURE 5

TABLE 6

Water Surface Elevations for Culvert Alternative

10 Year Flood

LOCATION	TAILWATER		BACKWATER	
	Existing	Improved	Existing	Improved
Crossing #2	1914.5	1914.5	1917.0	1916.0
Crossing #3	1917.0	1916.5	1919.5	1918.8
Crossing #4	1919.5	1919.3	1920.6	1920.5
Parshall Dam	---	---	1921.7	1921.7
Crossing #5	1922.0	1922.0	1928.3	1928.3
Crossing #6	1928.3	1928.3	1929.4	1929.0

25 Year Flood

LOCATION	TAILWATER Existing	TAILWATER Improved	BACKWATER Existing	BACKWATER Improved
Crossing #2	1914.9	1914.9	1917.3	1916.5
Crossing #3	1917.3	1917.2	1919.8	1919.5
Crossing #4	1919.8	1919.7	1921.0	1920.8
Parshall Dam	---	---	1922.1	1922.1
Crossing #5	1922.6	1922.6	1929.4	1929.4
Crossing #6	1929.4	1929.4	1930.0	1929.8

100 Year Flood

LOCATION	TAILWATER Existing	TAILWATER Improved	BACKWATER Existing	BACKWATER Improved
Crossing #2	1915.5	1915.5	1917.8	1917.4
Crossing #3	1918.3	1918.3	1920.2	1920.0
Crossing #4	1920.2	1920.2	1921.5	1921.4
Parshall Dam	---	---	1922.8	1922.8
Crossing #5	1923.5	1923.5	1930.2	1930.2
Crossing #6	1930.2	1930.2	1930.7	1930.6

Crossings two and three can handle the design flow with the addition of four, eight foot diameter corrugated metal pipe (C.M.P.) at crossing two and replacing the existing pipe at crossing three with eight, six foot diameter C.M.P. Due to the bad condition of the pipe in crossing three, they should all be replaced. At design flow the water level behind the crossing will be slightly below the top of the lowest spot in the road near the crossing. The amount of pipe in each case will require a crossing width of about seventy-two feet. Therefore, the channel will have to be widened at the crossing and tapered to fit with the existing channel. There is a jog in the channel on the outlet side of crossing three. This should be straightened. Also the eight foot pipe at crossing two should be relayed to slope properly, and the six foot pipe removed.

A new wastewater treatment lagoon was built between crossings two and three. The southwest corner was built over an oxbow in the channel of the East Branch of Shell Creek. A ditch with a bottom width of approximately ten feet and 2 to 1 (horizontal to vertical) side slopes was built to divert flows along the south side of the lagoon. The capacity of this channel is slightly less than the rest of the creek in the area. To improve this situation, the ditch should be widened to a thirty foot bottom width. This widening would be on the south side of the existing ditch since the north side is the dike for the lagoon. The new south side should slope at a rate of 3 to 1 (horizontal to vertical). Approximately 29,000 cubic yards of material will be excavated. The new ditch would have a depth of about five feet.

Since crossing number four is inundated by the backwater from crossing three, any pipes installed here will have very little effect on the backwater elevation. To keep the road from being overtopped, it would have to be raised and eight, six foot culverts installed. This would increase the backwater behind the crossing since water would no longer be flowing over the road. The intersection of this road and the main east-west road to the north is lower than the road at the crossing. This area floods already due to the backwater from crossing number three and should be raised. If the road over crossing four is raised, the backwaters will be higher and the east-west road will have to be raised higher.

Since the road over crossing number four only provides access to a small landing field, it would not be worthwhile to do all the necessary roadway raising and install the necessary pipes. Therefore, it is recommended that two, six foot diameter C.M.P. be installed. This was determined on the basis of room available in the channel at the crossing. Because the tailwater floods the crossing during high flows most of the water will flow over the road. However, the two additional six foot pipes will pass more flow before the water backs up and allow the backwater to drain away faster after the tailwater recedes.

Under this alternative crossing number five will remain the same. This causes a high tailwater at crossing number six that just starts to flood the road there during the design flow. Again it would take a grade raise on the road and added culverts to keep the road passable during the design flow. The area behind this crossing is already having flood problems and raising the road would increase them. It would take an unreasonable number of culverts to keep the water level behind the crossing the same with the grade raise as without it when the water flows over the road. It is recommended that the roadway elevation remain unchanged and that the existing six foot C.M.P. remain. Also the two 5.5' x 7.5' horizontal eliptical corrugated metal pipe (H.E.C.M.P.) that are sloped the wrong way should be relayed. The two that slope correctly can remain. Five additional 5.5' x 7.5' pipe should be installed. (This was calculated assuming changes done at crossing five which will be discussed later.) Even though these new pipe would not greatly reduce the backwater elevation, they will drain the backwaters away faster when the tailwater starts to recede.

BRIDGES

The drainage area above the study area is about 140 square miles. This is large enough to require bridges on all the crossings. The alternate proposes that bridges be built on crossings two, three, and six. They are designed to handle a twenty-five year flood. Figure 6 shows the water surface profiles that can be expected with the added bridges. These bridges will reduce the water levels needed to pass the twenty-five year flood. A comparison of the tailwater and backwater elevations for the existing and improved conditions is shown in Table 7.

Crossing number two requires a bridge having a flow area of 612 square feet. The proposal calls for a trapezoidal flow area with a bottom width of 50 feet, 2 to 1 side slopes and a depth of nine feet from the bottom of the beams to channel bottom. This results in a bridge with a length of 86 feet. The roadway width will be 30 feet.

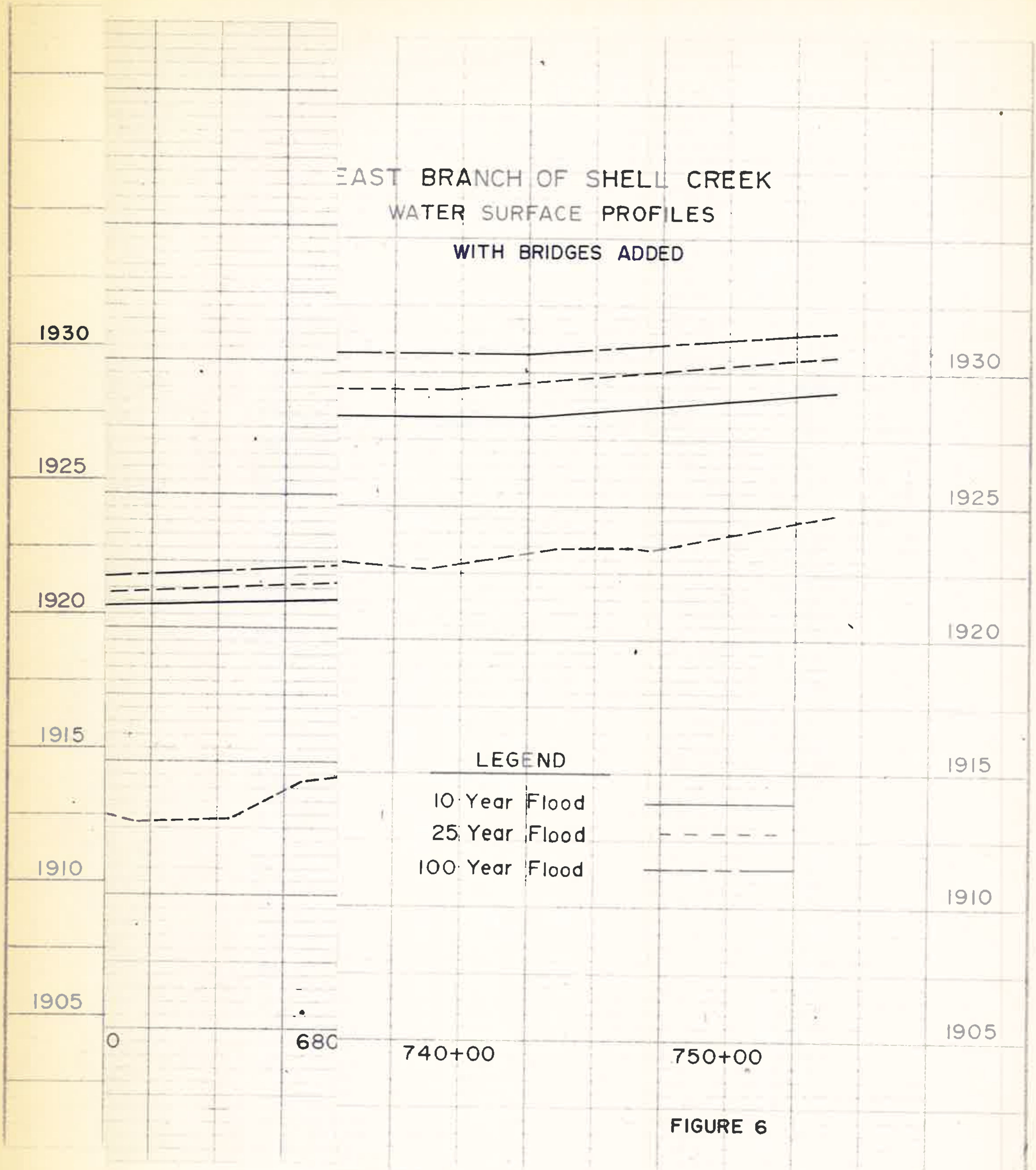

EAST BRANCH OF SHELL CREEK
WATER SURFACE PROFILES
WITH BRIDGES ADDED
1930
1925
1920
1915
1910
1905
0
680
740+00
750+00
LEGEND
10 Year Flood
25 Year Flood
100 Year Flood

FIGURE 6

TABLE 7

Water Surface Elevations for Bridge Alternative

10 Year Flood

LOCATION	TAILWATER		BACKWATER	
	Existing	Improved	Existing	Improved
Crossing #2	1914.5	1914.5	1917.0	1914.5
Crossing #3	1917.0	1916.5	1919.5	1916.5
Crossing #4	1919.5	1919.3	1920.6	1920.5
Parshall Dam	---	---	1921.7	1921.7
Crossing #5	1922.0	1922.0	1928.3	1928.3
Crossing #6	1928.3	1928.3	1929.4	1928.3

25 Year Flood

Crossing #2	1914.9	1914.9	1917.3	1915.2
Crossing #3	1917.3	1917.2	1919.8	1917.3
Crossing #4	1919.8	1919.7	1921.0	1920.8
Parshall Dam	---	---	1922.1	1922.1
Crossing #5	1922.6	1922.6	1929.4	1929.4
Crossing #6	1929.4	1929.4	1930.0	1929.4

100 Year Flood

Crossing #2	1915.5	1915.5	1917.8	1916.6
Crossing #3	1918.3	1918.3	1920.2	1919.4
Crossing #4	1920.2	1920.2	1921.5	1921.4
Parshall Dam	---	---	1922.8	1922.8
Crossing #5	1923.5	1923.5	1930.2	1930.2
Crossing #6	1930.2	1930.2	1930.7	1930.7

The installation of two additional six foot diameter C.M.P. made of aluminized steel is proposed for crossing number four. This causes a greater jump in the water surface profile across this crossing than the others. Tailwaters from the downstream channel flood the road over crossing four from a short distance north of the crossing to the intersection with the east-west road north of the crossing (see Figure 3). These tailwaters also flood the east-west road for a distance. Any improvements to the crossing will not prevent the roads from being flooded. In order to keep the roads dry, they will have to be raised about 2.5 to 3 feet at the intersection. This will put the road above the estimated twenty-five year flood level.

Since the road over crossing number three must be raised to accommodate the proposed bridge, the east-west road should be raised from this bridge to about 600 feet east of the intersection.

Building a bridge at crossing four, similar to the one at crossing three, would reduce the backwater elevations about a foot. This would reduce the required roadway grade rise needed to keep the roads above the 25 year flood by about a foot. However, only pastureland is flooded by these backwaters and they do not greatly affect the backwater elevations behind the Parshall Dam needed for flow over it. This is because the tailwater below the dam is controlled by channel characteristics and will remain the same. The road over crossing number four goes to a small airport. Water flowing over this road for a time would not cause any great inconvenience. Therefore, money spent on a bridge here is not warranted.

Crossing number five is to remain the same. Again one can see that the existing bridge here causes the largest jump in the water surface profile. This causes the tailwaters at crossing number six to be high enough to flood the road. Existing water surface profile increases are small across here due to the fact that most of the flow goes over the road.

This alternate includes a bridge at crossing six. A flow area of 459 square feet is required. It is proposed that the shape of the flow area be trapezoidal with 2 to 1 side slopes, 30 foot bottom and a 9.4 foot clearance between the channel bottom and the beams. This results in a bridge 66 feet long. The roadway width will be 30 feet. To get the beams above the 1929.4 backwater elevation the bridge deck will have to be raised to about elevation 1933.0. It is proposed to raise the road to the bridge deck elevation of 1933.0 from Highway 37 east to just past the bridge. To allow higher flows to cross the road with a minimal increase in backwater elevation, a flat section about 150 feet long at elevation 1929.5 should be constructed. For the water surface calculations, two percent slopes were assumed for the transition to the low overflow area.

At elevation 1929.4 the backwater is almost flowing through the overflow section. The water surface profiles were calculated assuming the overflow area of this elevation. However, if the roadway in this section was raised 0.5 feet to 1930.0 the road would be more passable during the twenty-five year flood. This would raise only the 100-year backwater elevation, resulting in a level closer to elevation 1931.0.

In addition to the added bridges at crossings two, three, and six, and the two added six foot diameter culverts at crossing four, there are two existing culverts at crossing four that should be relayed to slope the right direction. Also, the channel diverting the East Branch of Shell Creek around the new wastewater treatment lagoons should be widened to a 30 foot bottom width, as discussed in the culvert alternate. The jog in the channel alignment just downstream from crossing three is to be straightened out. Since the bridges are designed to handle the twenty-five year flow without being overtopped, the east-west road between crossing three and six is proposed to be raised above the twenty-five year water levels. A bridge at crossing four, as mentioned before, is not proposed but if it is put in it would lower the backwater elevations for the twenty-five year event by about a foot. If a bridge is built, the road over crossing four should be raised also. This would extend from the hill immediately to the south of the crossing to where it intersects the east-west road. If this is done, a low spot should be maintained to minimize the increase in the 100-year water levels due to raising the road. Only the land between crossing four and the dam would receive a slight benefit.

The cost for the alternate as proposed is $565,000. Table 8 shows a breakdown of these costs. If it is decided to construct a bridge on crossing four, it would increase the cost by $175,000.

LOWER CROSSING FIVE

Crossing number five causes the greatest increase in water surface elevations. This crossing used to have a small 11' x 20' bridge which washed out in 1979. The bottom of this old bridge was 5.5 feet lower than that of the existing bridge. This lower bottom elevation, together with the smallness of the bridge opening, allowed pressure flow to

TABLE 8

Cost Estimate - Bridge Alternate

Item	Unit	Quantity	Unit Cost	Cost
Crossing #2, Bridge	L.S.	1	129,000.00	$129,000
Crossing #3, Bridge	L.S.	1	133,500.00	133,500
Crossing #6, Bridge	L.S.	1	99,000.00	99,000
6' Dia. C.M.P. (Aluminized Steel)	L.F.	100	114.00	11,400
Excavation	C.Y.	28,000	1.00	28,000
Borrow	C.Y.	6,000	1.50	9,000
Gravel	Ton	1,450	4.00	5,800
Hot Bit. Pavement	Ton	715	15.00	10,725
85-100 Asphalt Cement	Ton	43	120.00	5,160
Fog Coat	Gal.	530	1.00	530
Seeding	Acre	2.2	200.00	440
	Subtotal			$432,555
	Engineering			44,145
	Contract Administration			44,145
	Contingencies			44,145
	TOTAL COST			$565,000

develop. The existing bridge was designed for open channel flow which depends on stream gradient to provide energy for flow. This bridge is 65 feet long and has a trapezoidal flow area with a bottom width of 30 feet (see Figure 7).

If the bottom of the channel below the bridge is lowered by two feet (see Figure 7), the backwater elevations can be reduced as shown in Table 9.

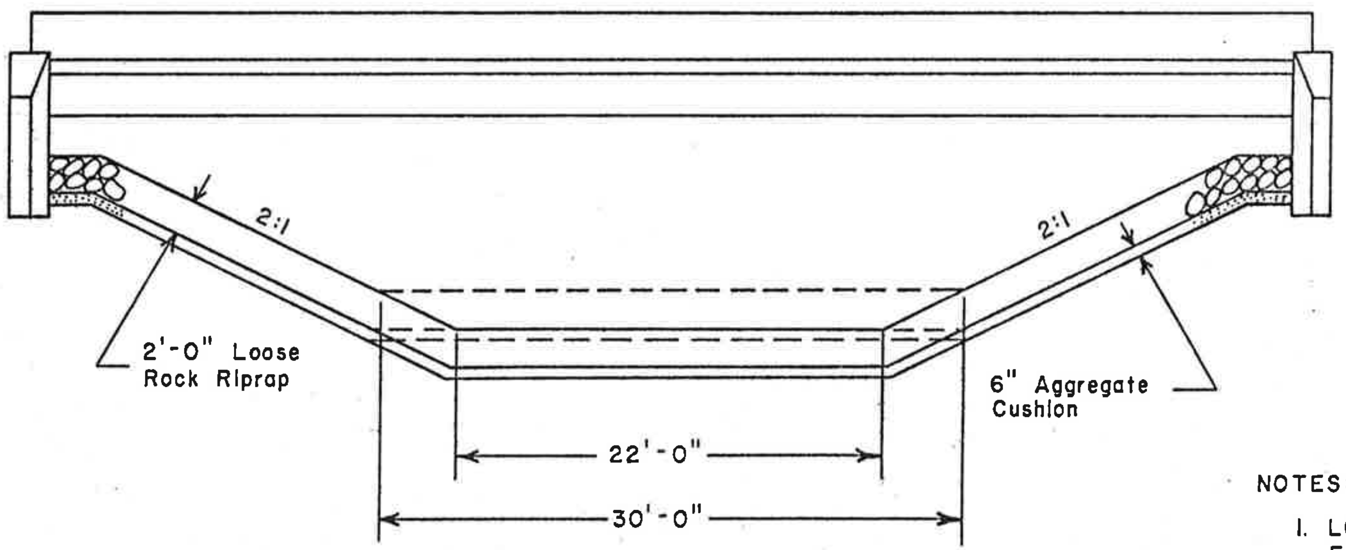
TABLE 9
Backwater Elevation at Lowered Crossing Five

Frequency	Flow	Backwater Elevations Existing	Backwater Elevations Lowered Crossing
10 yr.	1935 cfs	1928.3	1927.3
25 yr.	2715 cfs	1929.4	1929.0
100 yr.	4234 cfs	1930.2	1930.0

This channel lowering will start just below crossing number six. The channel gradient will be changed so the section below crossing five is two feet lower than existing. This new gradient will be continued until it daylights into the reservoir. The channel will have a 30 foot bottom width 4 to 1 (horizontal to vertical) side slopes. This alternative will cost $12,000.

The amount of water level reduction achieved during the 10 year flood will enable the road over crossing six to remain above water. This will allow head to build up so flow through the culverts can be increased. Adding five 5.5' x 7.5' H.E.C.M.P. culverts will pass the 10 year flow with the backwater at the elevation of the low spot in the road. Figure 8 shows the water surface profile if crossing five is lowered and the five culverts are installed. Table 10 compares the tailwater and backwater elevations of the existing and improved conditions.

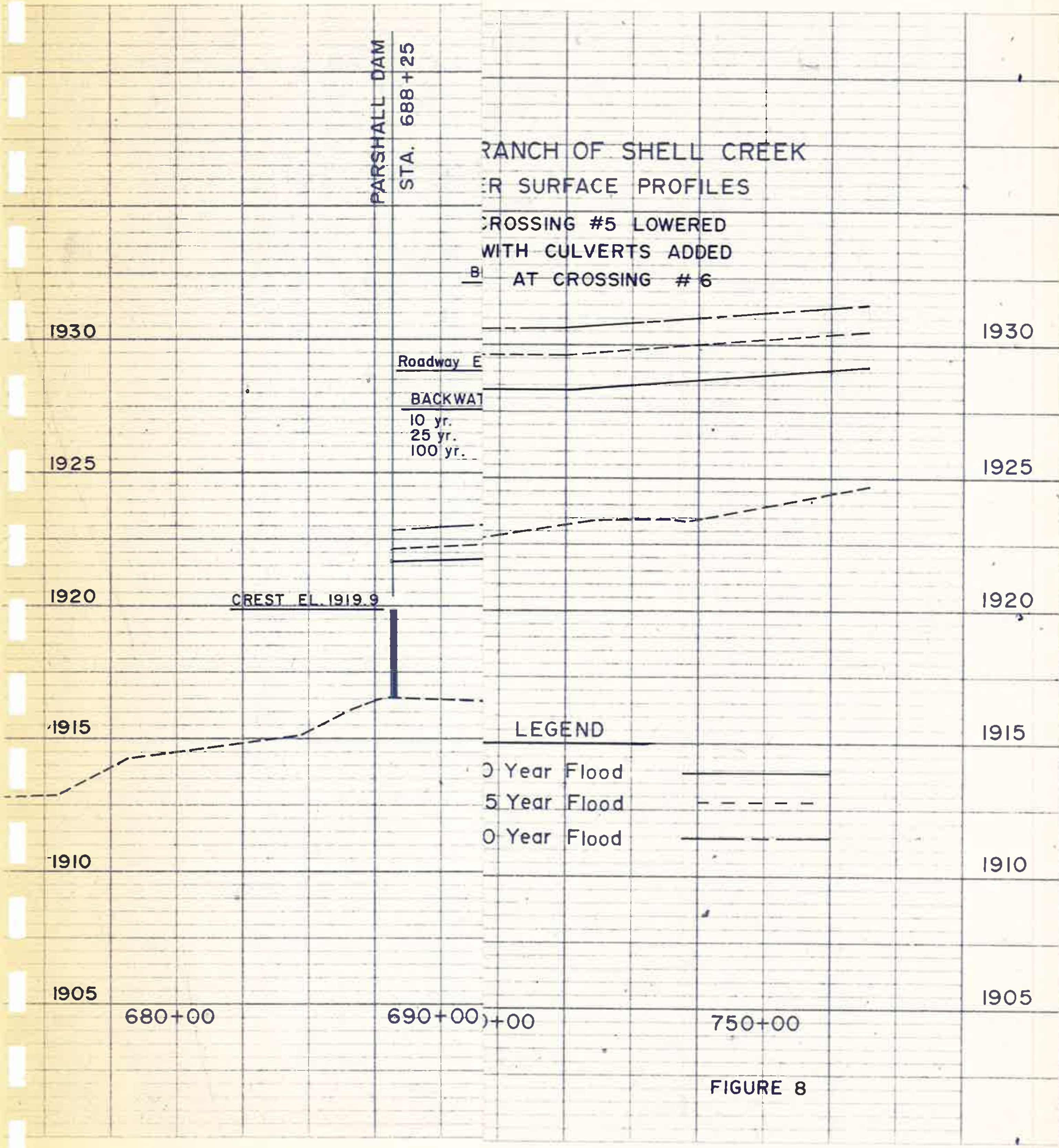
EAST BRANCH OF SHELL CREEK
CHANNEL LOWERING AT CROSSING #5



NOTES:

1. LOWER CHANNEL FROM EL. 1920.0 TO EL. 1918.0.
2. REMOVE & REPLACE ROCK RIPRAP.
3. EXISTING CHANNEL — — —
4. LOWERED CHANNEL ———

FIGURE 7



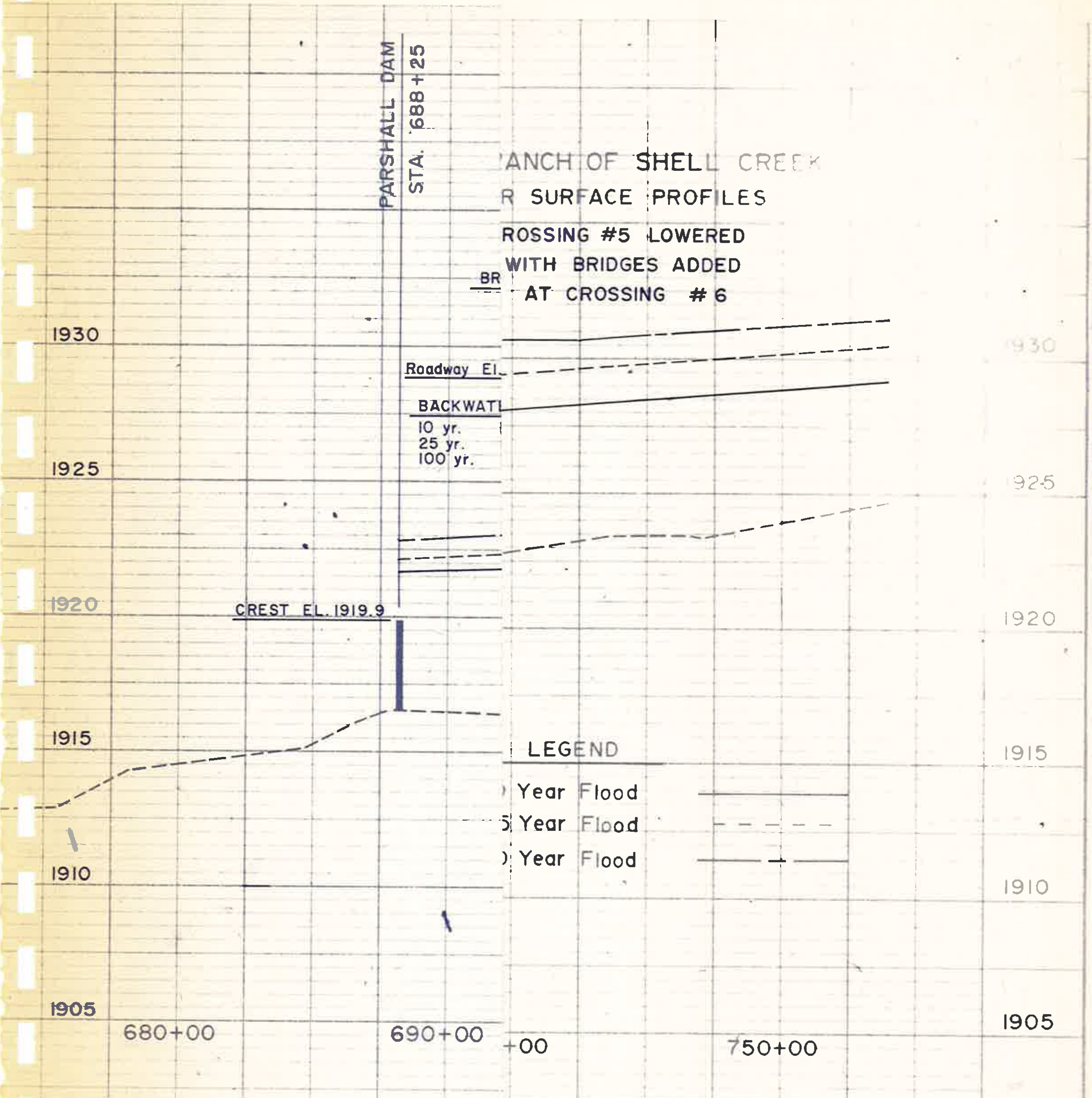
RANCH OF SHELL CREEK
ER SURFACE PROFILES
CROSSING #5 LOWERED
WITH CULVERTS ADDED
AT CROSSING # 6
PARSHALL DAM
STA. 688+25
Roadway E
BACKWAT
10 yr.
25 yr.
100 yr.
CREST EL. 1919.9
LEGEND
0 Year Flood
5 Year Flood
0 Year Flood
1930
1925
1920
1915
1910
1905
680+00
690+00
0+00
750+00

FIGURE 8

Figure 9 shows the water surface profile if crossing five is lowered and a bridge, as described in the bridge alternate, is constructed. A comparison of existing and improved conditions is given in Table 11. Adding the bridge will allow the road to be above water during a twenty-five year event. The roadway from the highway east to just past the new bridge should be raised to elevation 1933. To allow higher flows to pass the crossing without greatly increasing the 100 year backwater elevation, an overflow section 150 feet long at elevation 1929.5 should be constructed. In the analysis, two percent grades were assumed to transition the road profile into and out of this overflow section. This section was assumed to be at elevation 1929.5 which is just above the twenty-five year backwater elevation of 1929.4. To assure that the road be above water for this flood, the overflow section should be constructed at elevation 1930.0. This will raise the 100 year backwater closer to elevation 1931.0.

PARSHALL DAM

It has been mentioned at past meetings with the engineering staff of the State Water Commission that removing the small dam located about one thousand feet west of the highway would reduce flood levels. There is only about 0.5 feet of difference between the crest of the spillway weir and the existing ground a few hundred feet to the south (see Figure 10). Flows will easily go around the south side of the dam. Table 12 shows the backwater elevations needed behind the existing dam to pass various flows and the estimated elevations if the dam were removed.



PARSHALL DAM
STA. 688+25
'ANCH OF SHELL CREEK
R SURFACE PROFILES
ROSSING #5 LOWERED
BR
WITH BRIDGES ADDED
AT CROSSING # 6
Roadway El.
BACKWATI
10 yr.
25 yr.
100 yr.
CREST EL. 1919.9
LEGEND
Year Flood
Year Flood
Year Flood
1930
1925
1920
1915
1910
1905
680+00
690+00
+00
750+00

FIGURE 9

TABLE 10

Water Surface Elevations for Lowered Crossing #5 and Culverts at Crossing #6

10 Year Flood

LOCATION	TAILWATER		BACKWATER	
	Existing	Improved	Existing	Improved
Parshall Dam	---	---	1921.7	1921.7
Crossing #5	1922.0	1922.0	1928.3	1927.3
Crossing #6	1928.3	1927.3	1929.4	1928.3

25 Year Flood

LOCATION	TAILWATER Existing	TAILWATER Improved	BACKWATER Existing	BACKWATER Improved
Parshall Dam	---	---	1922.1	1922.1
Crossing #5	1922.6	1922.6	1929.4	1929.0
Crossing #6	1929.4	1929.0	1930.0	1929.6

100 Year Flood

LOCATION	TAILWATER Existing	TAILWATER Improved	BACKWATER Existing	BACKWATER Improved
Parshall Dam	---	---	1922.8	1922.8
Crossing #5	1923.5	1923.5	1930.2	1930.0
Crossing #6	1930.2	1930.0	1930.7	1930.4

TABLE 11

Water Surface Elevations for Lowered Crossing #5 and Bridge at Crossing #6

10 Year Flood

LOCATION	TAILWATER		BACKWATER	
	Existing	Improved	Existing	Improved
Parshall Dam	---	---	1921.7	1921.7
Crossing #5	1922.0	1922.0	1928.3	1927.3
Crossing #6	1928.3	1927.3	1929.4	1927.5

25 Year Flood

LOCATION	TAILWATER Existing	TAILWATER Improved	BACKWATER Existing	BACKWATER Improved
Parshall Dam	---	---	1922.1	1922.1
Crossing #5	1922.6	1922.6	1929.4	1929.0
Crossing #6	1929.4	1929.0	1930.0	1929.0

100 Year Flood

LOCATION	TAILWATER Existing	TAILWATER Improved	BACKWATER Existing	BACKWATER Improved
Parshall Dam	---	---	1922.8	1922.8
Crossing #5	1923.5	1923.5	1930.2	1930.0
Crossing #6	1930.2	1930.0	1930.7	1930.7

EAST BRANCH OF SHELL CREEK
PARSHALL DAM PROFILE

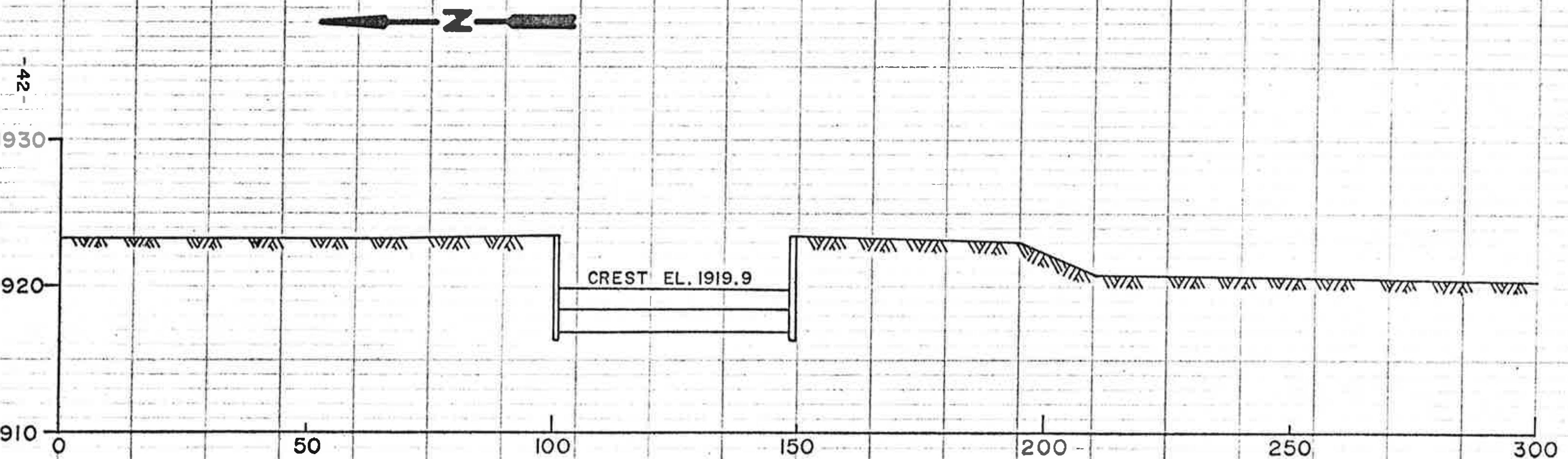


FIGURE 10

TABLE 12

Backwater Elevations at Parshall Dam

Frequency	With the Dam	Without the Dam
10 yr.	1921.7	1921.3
25 yr.	1922.1	1921.5
100 yr.	1922.8	1921.9

Water backed up from the dam does not seem to cause any major flooding problems. It floods the low areas of the golf course and an old ball park facility. The backwater elevations do not greatly affect the flow through crossing five. Because of the constricted flow condition, the free flowing water surface elevation at the bridge outlet is higher than the backwater elevations. Therefore, there is no urgent need to remove the dam. Its removal will only reduce water levels around the reservoir.

SOUTH TRIBUTARY DIVERSION

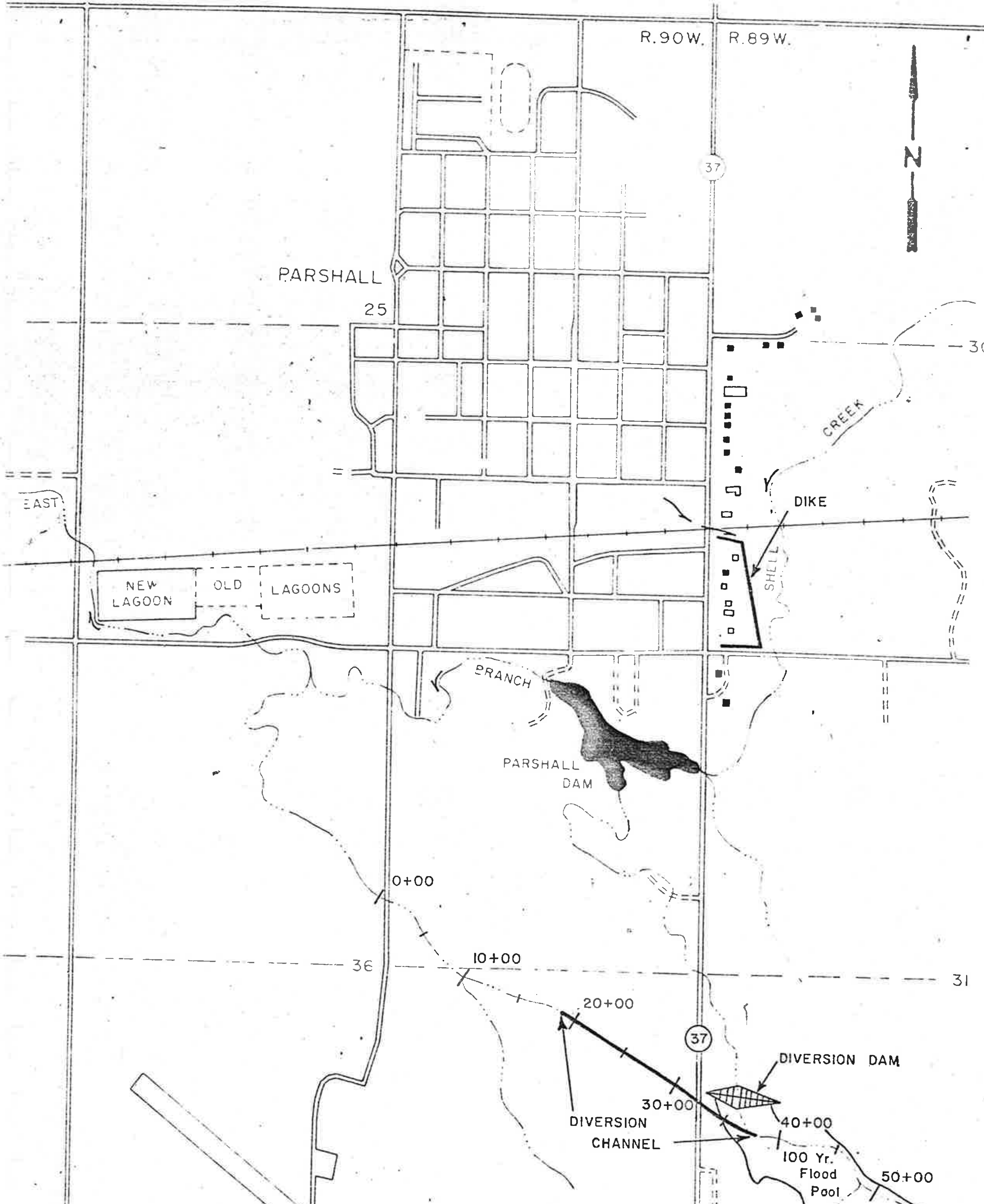
It is possible to divert the tributary coming into the reservoir behind the Parshall Dam from the south. This stream will be referred to as the south tributary. The location of this alternate is shown on Figure 11. To divert this south tributary, a dam will have to be built. along with a channel to carry the diverted flows. The channel would have a trapezoidal section with 3 to 1 (horizontal to vertical) side slopes and a 15 foot bottom width. It will be 1800 feet long and have an average cut of 10 feet. The maximum cut would be 26 feet at the Highway 37 crossing. A 10' x 8' (span-rise) reinforced concrete box culvert 142 feet long is needed to take the flows under the highway. From here the flow will follow the diversion channel and outfall into a coulee. This coulee crosses the road to the airport. At this crossing, two six foot diameter corrugated metal pipe culverts are needed to pass the 10 year diverted flow. Figure 12 shows a profile of the channel and the proposed dam. Tabe 13 lists the expected flows in the south tributary that are to be diverted and the existing flows in the coulee that will receive these flows. These come from a drainage area of 18 square miles.

TABLE 13

Diverted & Existing Flows

South Tributary Flows	Existing Flows in Receiving Coulee
Q_{10} = 480 cfs	Q_{10} = 45 cfs
Q_{25} = 650 cfs	Q_{25} = 70 cfs
Q_{100} = 1030 cfs	Q_{100} = 170 cfs

The ten year peak flows through crossing number five, the Parshall Dam and crossing number four will be reduced from about 1935 cfs to 1500 cfs. This would lower the backwater elevation at crossing five from 1928.3 to 1927.3 which would reduce the tailwater at crossing six. With this reduction, the road over crossing six would not be flooded by these tailwaters and installing either five 5.5' x 7.5' horizontal eliptical corrugated metal pipe or a bridge, as proposed in the bridge alternate, would keep the road dry. Backwater elevations would be 1928.3 with the added culverts, 1927.5 with a bridge, and 1928.9 with the existing crossing. The lowest roadway elevation near the crossing is 1928.3. Without the diversion, the backwater elevation behind crossing six is 1929.4.



MAP OF DIVERSION & DIKE ALTERNATIVE
FIGURE 11

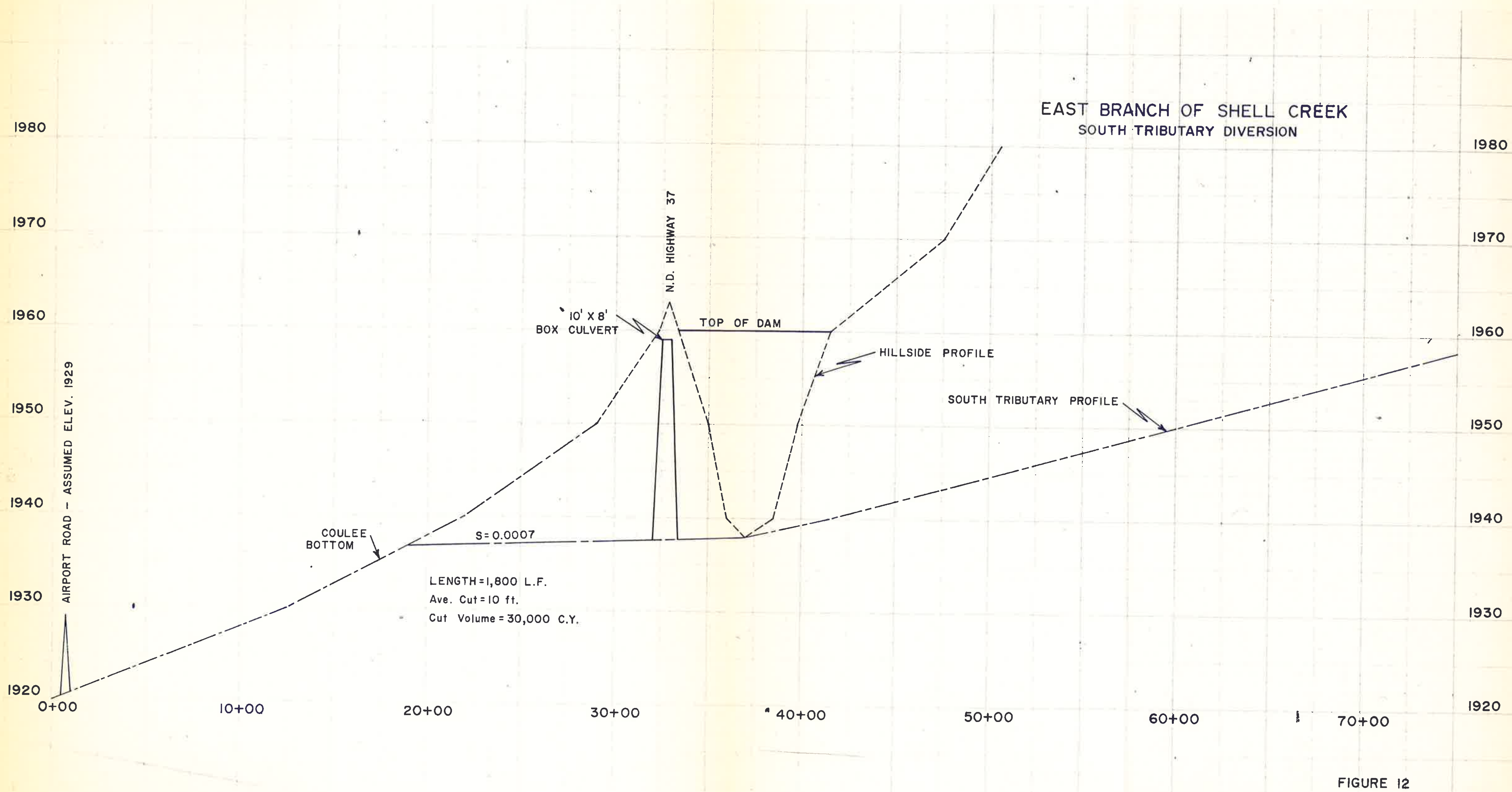

EAST BRANCH OF SHELL CREEK
SOUTH TRIBUTARY DIVERSION
1980
1970
1960
1950
1940
1930
1920
0+00
10+00
20+00
30+00
40+00
50+00
60+00
70+00
AIRPORT ROAD - ASSUMED ELEV. 1929
N.D. HIGHWAY 37
10' X 8'
BOX CULVERT
TOP OF DAM
HILLSIDE PROFILE
SOUTH TRIBUTARY PROFILE
COULEE BOTTOM
S=0.0007
LENGTH=1,800 L.F.
Ave. Cut=10 ft.
Cut Volume=30,000 C.Y.

FIGURE 12

For a twenty-five year flood, flows at the same crossings would be reduced from 2715 cfs to 2100 cfs. The tailwater elevation at crossing six would be lowered from 1929.4 to 1928.6. Backwater elevations would be lowered 0.6 feet to elevation 1929.2 with the added culverts. They would be lowered 0.4 feet to elevation 1929.0 with a bridge installed and lowered 0.2 feet to elevation 1929.8 with the existing crossing. The backwater elevation behind crossing six is 1930.0 without the diversion.

The 100 year flood flows between crossing five and four would be reduced from 4234 cfs to 3220 cfs. This will lower the backwater elevation at crossing five from 1930.2 to 1929.7. Due to this, the backwater with the added culverts would be lowered 0.2 feet to elevation 1930.4. If this diversion is done along with the bridge alternate, the backwater would be lowered 0.4 feet to elevation 1930.3. For the existing conditions, the backwater would be lowered 0.1 feet to elevation 1930.6.

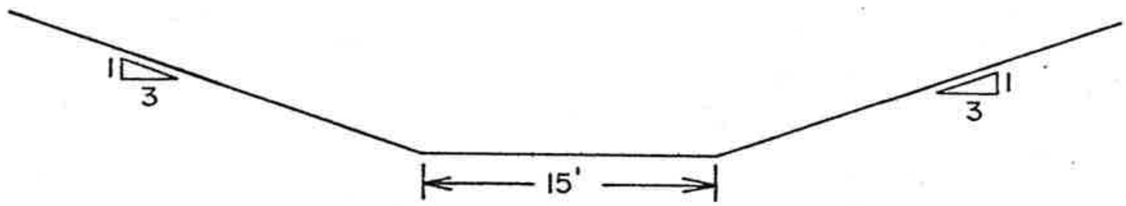
There would be small reductions in water surface elevations at the dam. However, the levels reached without the diversion under existing conditions do not seem to cause much problems around the reservoir. Most of any flooding occurs on nearby areas of the golf course and an old ball park facility. The rest of any flooded area is undeveloped. Water may get close to some buildings along the east-west road to the north. Because of the constricted flow conditions at crossing number five, a reduction in the reservoir's surface elevation will not greatly affect flow through there. This is because the expected tailwater at crossing five, with and without this diversion, would be lower than the elevation of the free flowing water surface at the outlet of the bridge.

Diverted flows from the south tributary would reunite with the East Branch of Shell Creek just below crossing number four. Because of this, the tailwater at crossing four will not be changed. The roads in the area will still flood due to the tailwater at crossing four. Therefore it is expected that there will be only a slight change in the flow characteristics at this crossing. The water levels will be near those calculated for the existing conditions, the culvert alternate and the bridge alternate.

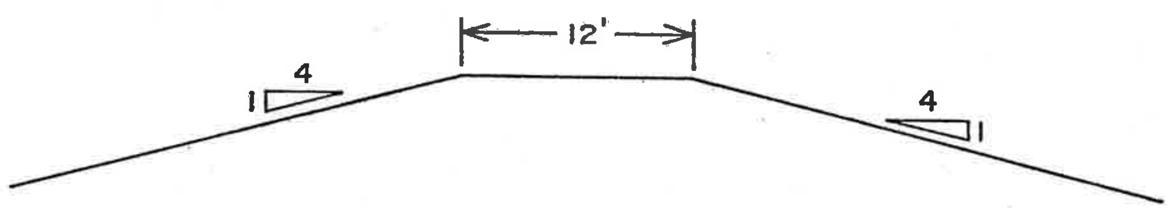
Material excavated during the construction of the diversion channel can be used to construct the dam. This amounts to 30,000 cubic yards. The proposed dam will have a 12 foot top width with 4 to 1 (horizontal to vertical) side slopes. Figure 13 shows a typical cross section of both the diversion channel and the dam. For design of the box culvert it was assumed that 12 feet of water would back up in order to pass the 100 year flow through it. Allowing five feet of freeboard gave a top elevation of 1955.0. To use all 30,000 cubic yards of material from the channel the side slope would have to be about 6.5 to 1 (horizontal to vertical). This would have a 233 foot base width. The proposed top elevation of 1960.0, as shown on Figure 13 with 4 to 1 (horizontal to vertical) side slopes would use the 30,000 cubic yards of material and have a base width of 188 feet. It would also provide more protection from overtopping.

This alternate requires daming up a stream, excavating a fairly deep channel and releasing into a coulee ten times the existing flows it now experiences. During the 100 year flood, 12 feet of water will be backed up behind the highway 37 crossing. This will flood a 2400 foot

EAST BRANCH OF SHELL CREEK
SOUTH TRIBUTARY DIVERSION



CHANNEL X- SEC.



DAM X-SEC.

FIGURE 13

length of the triburary covering 35 acres. Figure 11 shows the area flooded by the 100 year event. Easements will have to be attained for the 35 acres below elevation 1950.0 that will be flooded. The cost of this alternate is about $180,000 not including the cost of any easements or land acquisition for the dam and the channel.

Increased flows will cause erosion problems along the receiving coulee which the Mountrail County Water Management Board may be held responsible for. This could result in the expenditure of more money for erosion control. Due to the increased flows in the coulee, easements will have to be attained for the areas flooded during high flows. Therefore, this alternate is not recommended, due to its cost and the damaging effects it will have on the receiving coulee.

DIKES

Most of the damage due to flooding of the East Branch of Shell Creek occurs along the east side of Parshall. The 1979 flood was near the levels estimated for the 100 year flood. Therefore areas flooded at that time give a good indication of the areas expected to flood during a 100 year event. This alternate proposes that dikes be constructed to protect against flooding.

Figure 11 is a map of the study area. North of the railroad tracks are two barns and a house that had water get close to them in 1979. On the north side of the road over crossing six, at its intersection with highway 37, is a complex of grain bins including a few larger buildings and quansets. This area was flooded in 1979. South of this road are two houses surrounded by some smaller buildings. The houses were above the 1979 water levels. North of the grain bin complex is a small farmstead. Two low lying grain bins near the railroad tracks were flooded and water was up to the side of his small barn. The elevator just west of the highway was surrounded by water backing up into a drainage ditch leading into the East Branch of Shell Creek from the city.

The barns and the house north of the tracks are on the edge of the 100 year floodplain. They are probably high enough to escape damage from the 100 year flood. Any problem they may have should be easily taken care of by sandbagging or piling up dirt to make a temporary dike. The same is true of the two houses south of the grain bin complex. Some of the small sheds around these houses may flood. If needed, they can be protected with sandbags or a temporary dike. Temporary dikes would also protect the elevator. Standby pumps may be needed to pump water that may seep into the elevator's scale pits or the basements of any affected houses.

A dike should be constructed along and following the ridge delineating the creek's floodplain extending south from the railroad tracks to the road over crossing number six. To protect against a 100 year flood, the top of the dike should be at elevation 1931.5. This allows 0.8 foot of freeboard over the water surface elevation of 1930.7 which is the estimated backwater elevation for the existing crossing six. Looking at the water surface profiles at this crossing, it appears that any improvements implemented will lower the ten and twenty-five year flood levels. The 100 year flood levels range from 1930.4 to 1930.7 depending on the improvement made. Therefore, it appears that even with improvements to crossing six, and in one case crossing five also, dikes may be needed in some areas to protect them from the 100 year flood.

Figure 11 shows the location of the proposed dikes. They will have a ten foot top width and 2 to 1 (horizontal to vertical) side slopes. There will be 1400 feet of dike south of the tracks. Since the top of the ridge above the floodplain is near elevation 1930, the dike will be about two feet above the existing high ground. At the grain bin location, the dike height will be about four feet. There is little room for the dike on top of the ridge. Therefore the landside edge of the dike top will be located at the edge of the ridge. From here the dike will extend out into the low area below the ridge. The two grain bins on the side of the ditch just south of the tracks will have to be moved.

The dike construction will require about 9,700 cubic yards of material that will have to be hauled in. This material will have to be well compacted. Pipes with flap gates will be installed at various places to allow drainage from the area inside the dikes. The cost of this alternate will be about $27,000.

STRUCTURE REMOVAL

Rather than diking areas that flood, in some cases it may be more advantageous to remove or relocate any structures that frequently are damaged by flooding. This would apply to the low lying grain bins and small sheds surrounding the small farmstead just south of the railroad tracks. It would also apply to the grain bin complex just south of this farmstead where Highway 37 intersects the road over crossing six.

The problem involved with this alternate is finding higher ground nearby to relocate to. There should be room to move the two grain bins located just south of the tracks in the ditch side slopes. It appears that there is little or no adjacent high ground available to relocate all the buildings and grain bins in the grain bin complex to. In this case either a new location will have to be found, the structures could be permanently removed, or they could remain and be allowed to flood during high flows.

To move the bins and buildings located in the grain bin complex along with the two low lying bins of the farmstead north of it will cost about $33,000. This cost assumes that there is an adjacent or nearby location to relocate the structures. If they have to be moved over a half mile the costs would be significantly higher.

FLOOD PLAIN MANAGEMENT

The city and county can minimize future flood losses by planning for the protection, wise use and orderly development of the flood plain area. The overall plans of the community for industrial, commercial and residential areas, for streets, utilities, parks and schools must be coordinated with the need to temporarily store (if possible) and convey floodwaters.

A planning procedure such as this is a vital part of the comprehensive flood plain management program. Effective flood plain management involves the full range of public policy and action needed for the wise use and development of the flood plain. It includes a range of measures from collection and dissemination of flood control information to acquisition of flood plain lands, construction of control structures, and enactment of ordinances and statutes regarding flood plain land use and development.

A sound local flood plain management program is comprised of numerous elements. Some of these are: structural flood control works to protect existing development; regulations to guide new development; flood insurance to protect existing and new buildings and individual protection measures, such as flood proofing.

Flood plain regulations are designed to permit realistic use of flood plain areas without materially increasing the flood damage potential. Among the various elements used to accomplish this are zoning ordinances, subdivision regulation, building codes and sanitary and utility regulations. For a guide, see "A Perspective on Flood Plain Regulations for Flood Plain Management," Corps of Engineers Manual EP 1165-2-3-4, 1 June 1976.

Under the National Flood Insurance Act of 1968 (P.L. 90-448), the Federal Emergency Management Agency (FEMA), Division of Federal Insurance and Mitigation (FIM), is authorized to carry out a National Flood Insurance Program (NFIP), which makes flood insurance coverage available to all walled and roofed structures used for residential, business, religious and agricultural purposes, buildings occupied by nonprofit organizations, and those owned by state or local governments or their agencies. Coverage is also available for the contents. The city of Parshall is participating in FIM's Emergency Program. In those communities participating in the FIM's program, owners and occupiers of all buildings and mobile homes in the entire community are eligible to obtain flood insurance coverage; and it is recommended that buildings and mobil homes within or adjacent to the delineated flood hazard areas carry flood insurance on the structure and contents.

Further inquiries about the flood insurance program should be directed to the North Dakota State Water Commission, the official state coordinating agency for flood insurance.

Land use and other regulatory controls, including zoning, subdivision regulation and building codes, play an important role in flood plain management. However, in order for these measures to be effective, it is important that the community take action to implement other programs and measures to supplement these controls. A few possible measures are: (1) open space land acquisition programs, (2) urban renewal programs, (3) preferential tax assessment, (4) flood proofing of existing structures and (5) public policy governing the construction of utilities and public facilities such as bridges and streets in a manner to control development in flood prone areas.

The North Dakota State Water Commission, upon request, will provide assistance in flood proofing techniques, the implementation of a flood warning system, and establishment of a local flood data collection program.

Encroachment of flood plains, including filling, reduces the flood carrying capacity and increases flood heights, thus increasing flood hazards in areas beyond the encroachment itself. One aspect of flood plain management involves balancing the economic gain from flood plain development against the resulting increase in flood hazard. For purposes of the National Flood Insurance Program, the concept of the floodway is used as a tool to assist local communities in this aspect of flood plain management. Under this concept the area of the 100 year flood is divided into a floodway and a floodway fringe (see Figure 14). The floodway is the channel of the stream, plus adjacent flood plain areas, that must be kept free of encroachment in order that the 100 year flood be carried without substantial increases in flood heights. Criteria adopted by the Division of Federal Insurance and Mitigation limit such increases in flood heights to 1.0 foot, provided that hazardous velocities are not produced.

The area between the floodway and the boundary of the 100 year flood is termed the floodway fringe. The floodway fringe thus encompasses the portion of the flood plain that could be completely obstructed without increasing the water-surface elevation of the 100 year flood more than 1.0 foot at any point. Typical relationships between the floodway and floodway fringe and their significance to flood plain development are shown in Figure 14.

The basic purpose of flood plain regulations is to regulate development on the flood plain consistent with nature's needs for the conveyance of flood flows and the community's land use and development objectives, in order to reduce future flood losses.

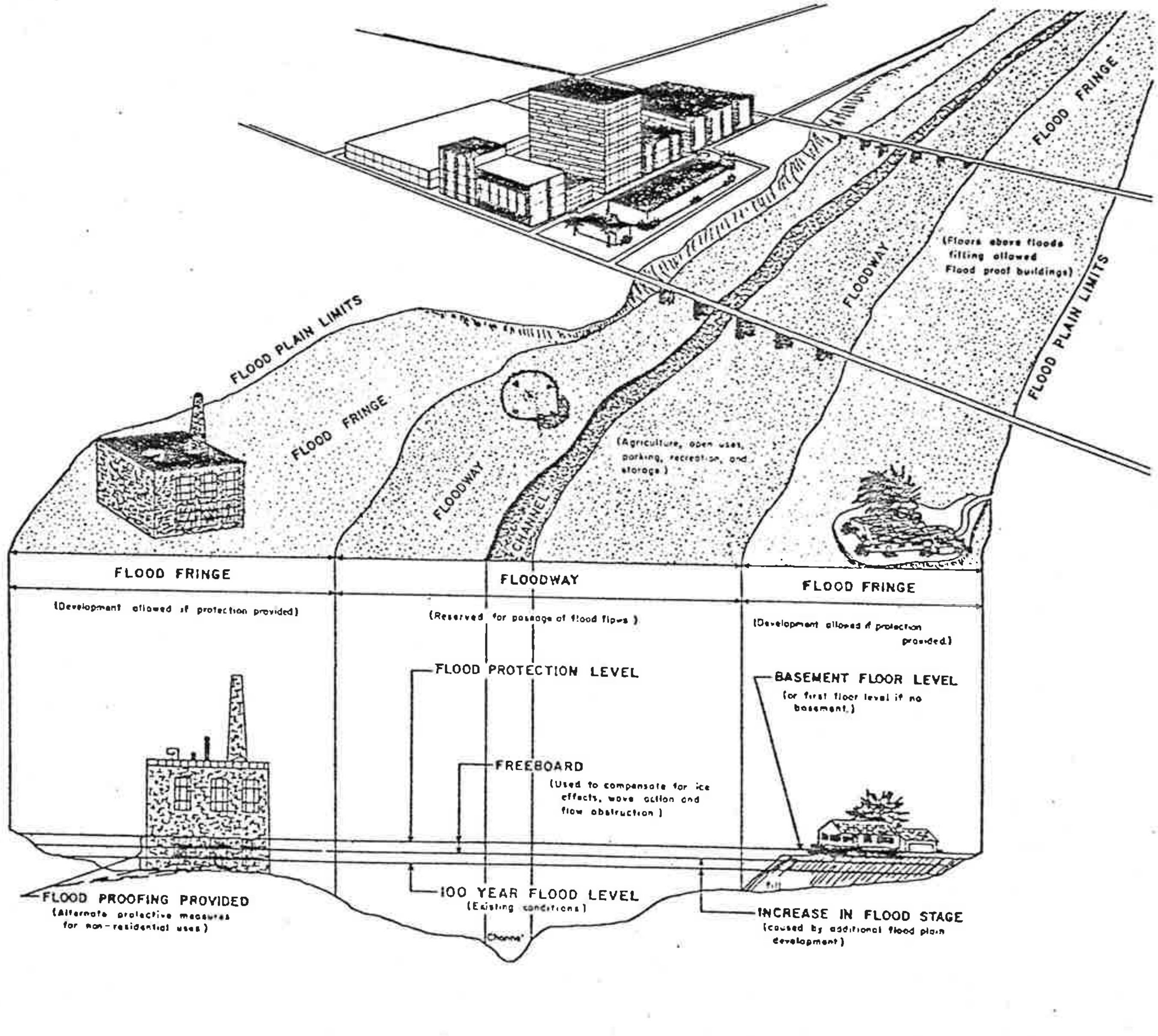


FIGURE 14 Perspective and cross sectional view of the structure of a typical regulatory flood plain.

VI. SUMMARY

This report has presented an analysis of the existing conditions along the East Branch of Shell Creek. From this study the water surface elevations at the roadway crossings were estimated. These were plotted on a profile of the existing channel bottom to come up with a water surface profile for the existing conditions. The existing water surface profile shows that all of the crossings, except Highway 37, are overtopped by a ten year flood. In fact the flood waters tend to easily spread out onto the adjacent land. This is due to the small channel that exists and the wide flat floodplain the water can spillout onto.

The main flooding problem in the study area occurs on the east side of the City of Parshall. This problem area is generally located between the creek and Highway 37. Looking at the existing water surface profile shows that the highway bridge causes a large backup of water. This causes the tailwater at crossing number six to be so high that the roadway is starting to flood during ten year event. Because of this, the capacity of these culverts is greatly reduced and most of the flow must go over the roadway. These backwaters cause the flooding problem. Since the water surface profiles show only a small jump across crossing six, the major constriction to flow is caused by crossing five, the highway bridge.

Since most of the crossings flood during the ten year flood, the various alternates also looked at possible improvements at these locations attempting to lower the water surface elevation and keep the roads above water during a flood. Again the water surface elevations at the crossings were estimated and plotted to get a water surface profile for the improvements made by each alternate. The alternatives that were looked at were as follows:

1. Add culverts to the existing crossing plus some channel work.

2. Install bridges at crossings number two, three and six. To keep the roads passable during the 25 year flood would involve raising some of the roadways. There would also be some channel improvements between crossings two and three. If the board felt it necessary, a bridge could be installed at crossing four along with some grade raising to keep that road from flooding.

3. Lower crossing five, the highway bridge, to lower the tailwater at crossing six which will allow added culverts to handle the estimated ten year flows. This would include deepening the channel along a new grade line between the reservoir and crossing six.

4. Diverting the south tributary coming into the reservoir behind Parshall Dam through the golf course. This would lessen the flows through the highway bridge which would also lower the tailwater at crossing six enabling it to pass the ten year flows with added culverts.

5. Diking the affected properties.

6. Removal of structures commonly subjected to flooding.

The cost of the culvert alternate would be about $224,000. Constructing bridges at crossing two, three and six along with some other incidental items would cost $565,000. A bridge at crossing four would cost an additional $175,000. Lowering crossing five will cost $12,000. It would cost about $180,000, not including easements, to divert flows from the south tributary. Diking the affected areas between Highway 37 and the East Branch of Shell Creek will cost about $27,000 while removing the structures subject to flooding will cost about $33,000. This would increase if they had to be relocated a distance of more than about a half mile or so.

Construction of dikes around the areas subject to flooding would be the most feasible alternate as far as flood protection is concerned. In areas where only a few small structures are commonly flooded it would be better to relocate them to higher ground or remove them completely. It is recommended that the dike alternative be constructed. Along with this, the City of Parshall should implement the floodplain management principles discussed earlier. The city has adopted ordinances following the guidelines set forth in section 1910.3(b) of the National Flood Insurance Program. These ordinances should be enforced. This, however, does not solve the problem of water flowing over the roadways and making them impassable. If it is desired to improve the various crossings to allow either a ten or twenty-five year flood to pass without flooding the road, then either the culvert or bridge alternate should be chosen, respectively. Crossing five should also be lowered if one of these alternates is implemented. It will have the greatest initial effect on the water levels behind crossing six even though additional culverts or a bridge is needed to protect the crossing completely from a ten or twenty-five year flood respectively. The diversion of the south tributary does not seem to be a feasible alternate due to its cost and the erosion damage that may occur.