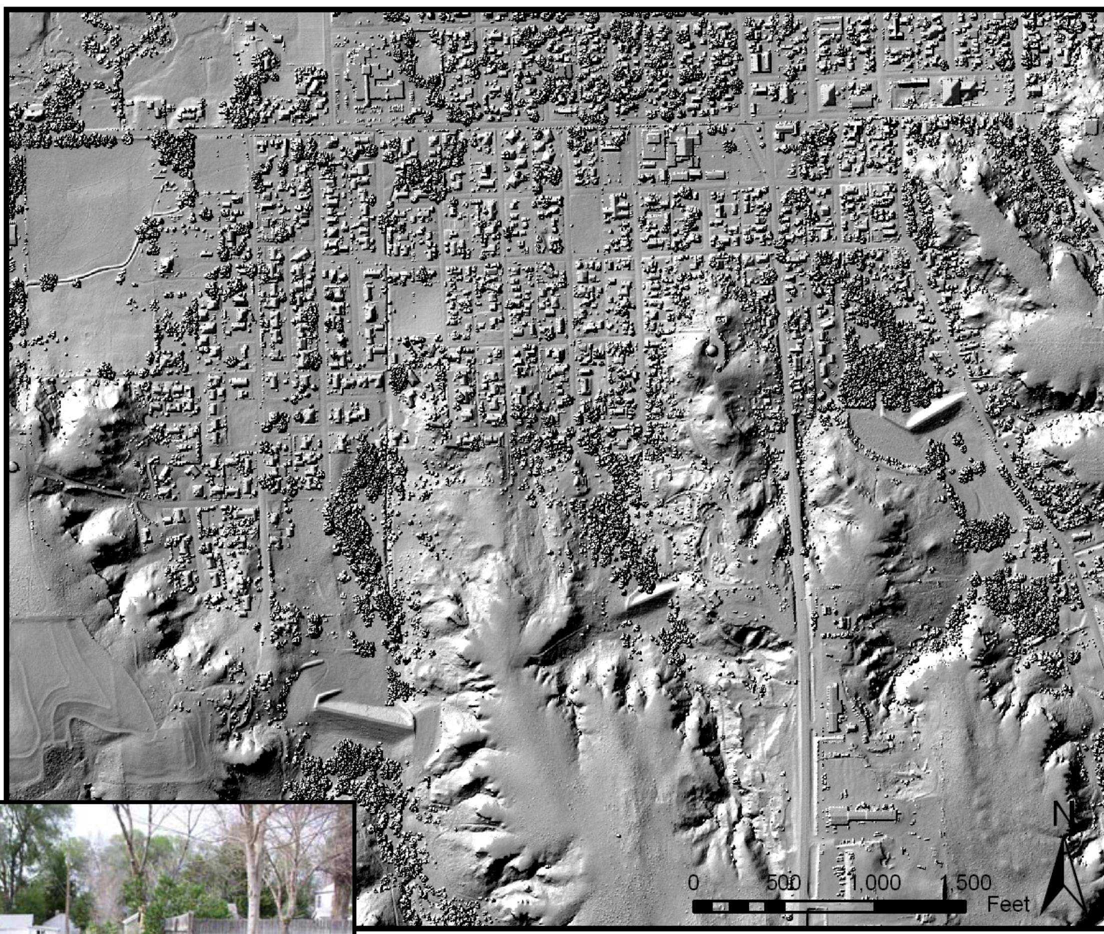
# Hydrologic Study of Wray Floodwater Detention Structures

Yuma County, Colorado
October 2006





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NRCS

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**NATURAL RESOURCES CONSERVATION SERVICE**
**ROCKY MOUNTAIN ENGINEERING TEAM**

**Lakewood, Colorado**

**October 23, 2006**

# Hydrologic Study of Wray Floodwater Detention Structures

**Job Number:** CO-0603.
**Short Job Description:** flood study of existing flood detention structures.
**Location:** Wray; Yuma County, Colorado.

**Summary:** Predictions have been made of the expected hydrologic response of the watersheds above, from and immediately below six floodwater detention structures. Both 6-hour and 24-hour storms were modeled for the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year events. The probable maximum precipitation event was also modeled. Extensive details were provided on how the modeling was performed. Results are provided in the HYDROLOGIC MODELING RESULTS and HYDRAULIC MODELING RESULTS sections.

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# INTRODUCTION

In the late 1950's the U.S. Department of Agriculture's Small Watershed Program financed the design and construction of six flood retention structures in Wray, Colorado to attenuate flood flows produced in the bluffs south of town. These structures are at or near their initial design life. Floodways downstream of these structures have seen substantial encroachment and alteration. A reevaluation of the effectiveness and risks of these structures to the lives and property of Wray was needed.

To assist in this hydrologic and hydraulic assessment, a survey of each of the six flood-retention structures, their flood pools, and downstream floodways was required. Due to poorly-defined flow paths for large events, substantial aerial extents, and the high resolution of the required survey, remotely-sensed collection of the elevation data was deemed the most appropriate approach. LiDAR (Light Detection and Ranging) from an aerial platform was implemented to collect these elevation data. LiDAR uses high-accuracy aerial GPS linked with onboard laser ranging to collect elevation data. Spectrum Mapping was retained to perform this service, providing break-line enhanced bare earth and first return data on a 1-meter grid for an 18.4 square mile area of interest. Additionally, 6-inch resolution color aerial imagery was also collected for the development of breaklines, for use in the hydrologic analyses, and for presenting results.

This report details the methods and results of a hydrologic and hydraulic study of six floodwater detention structures to the immediate south of the town of Wray. Storms of both 6-hour and 24-hour lengths were modeled. Storm frequencies of 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year, plus the Probable Maximum Precipitation (PMP) event, were analyzed. This report is intended for use by the Natural Resources Conservation Service (NRCS), the Yuma County Conservation District, and the town of Wray.

This analysis is limited to evaluating the flow runoff to be expected from the 5 drainages that have flood detention structures. Hydrologic modeling was performed for each of the six structures' watersheds, as well as a number of small watersheds downstream of the structures. Hydraulic modeling was performed only to the points where the drainages flow into an urban stormwater management system. This project's scope does not include the evaluation of Wray's stormwater management system. Additionally, the potential flooding of Wray from the Republican River has also not been evaluated.

Extensive details have been provided regarding the analysis methodology. This information is intended for other hydrologic professionals, so that this analysis can be defensively implemented in later studies. **For analysis results, please see the HYDROLOGIC MODELING RESULTS and HYDRAULIC MODELING RESULTS sections.** For details on the structures, see the FLOOD RETENTION STRUCTURES section on the next page.

**FLOOD DETENTION STRUCTURES**

The six floodwater retention structures are illustrated in Figure 1. Two of the structures, specifically structures 3 and 4, are in series. The remaining structures are in parallel. All of the structures' outflows pass through Wray. Table 1 provides the basic statistics for each of the structures. Tables 2 through 7 provide stage-storage and stage-discharge relationships of each of the structures. The stage-storage relationships were developed from the LiDAR data. The stage-discharge relationships were developed through a combination of the LiDAR data, the "as-built" drawings, and measurements taken in the field.

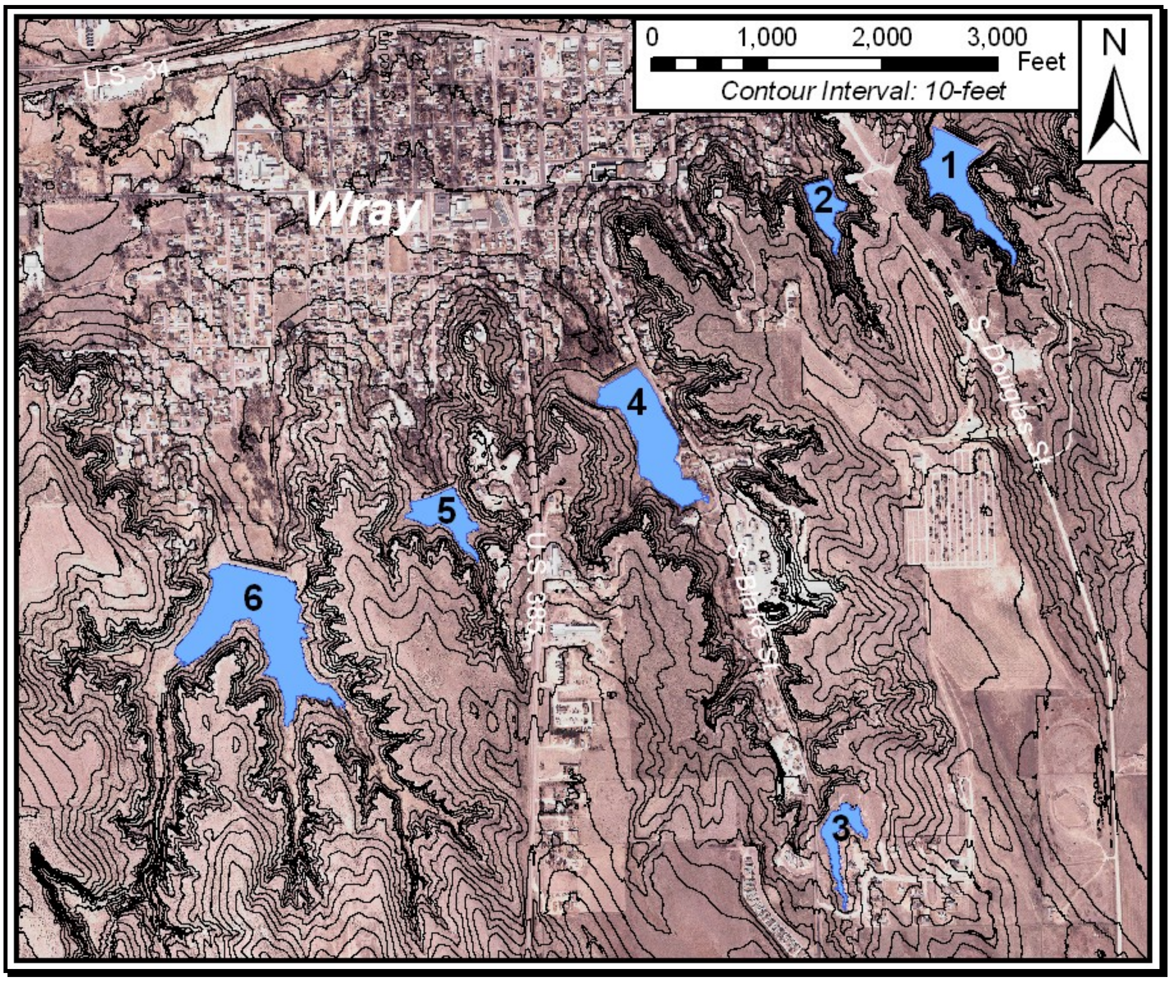



**Figure 1:** Wray flood retention structures, with pool extents at crest of the emergency spillway.

**Table 1:** Elevation and volume data, Wray flood retention structures.

| Structure Number: | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| Elevation, top of dam (feet): | 3590.0 | 3590.9 | 3675.5 | 3598.8 | 3624.7 | 3626.6 |
| Elevation, emergency spillway crest (feet): | 3583.5 | 3586.5 | 3670.6 | 3592.8 | 3619.4 | 3620.4 |
| Elevation, principal spillway crest (feet): | 3567.1 | 3576.3 | 3652.1 | 3584.2 | 3608.7 | 3605.1 |
| Volume, top of dam (acre-feet): | 118.6 | 28.4 | 45.2 | 141.5 | 49.4 | 320.5 |
| Volume, emergency spillway crest (acre-feet): | 74.1 | 16.1 | 29.8 | 67.5 | 28.3 | 196.4 |
| Volume, principal spillway crest (acre-feet): | 10.4 | 0.25 | 2.8 | 10.2 | 4.3 | 25.5 |

For all structures, flow thought the principal spillway was computed by comparing the potential flow for each the riser entrance, the culvert entrance and the culvert barrel capacity. The minimum of the each of these for each reservoir stage was used as the principal spillway capacity.

### Structure 1

Structure 1 is an embankment dam with one principal spillway, a drawdown pipe, and two emergency spillways. The principal spillway inlet is a single 2.0 feet by 3.5 feet horizontal orifice in a concrete riser that is drained by an 18-inch reinforced concrete pipe. The drawdown pipe is an 8 inch asphalt-coated welded steel pipe that drains into the concrete principal spillway riser. The principal spillway outfall is through a corrugated metal pipe. The orifice entrances were modeled with an orifice coefficient of 0.7. For depths less than 1.0 feet, the principal entrance was modeled as a weir, with a weir coefficient of 2.8. The eastern (right) emergency spillway has a bottom width of 80 feet and an average width of approximately 90 feet. The western (left) emergency spillway has a bottom width of 60 feet and an average width of approximately 70 feet. It was found that the drawdown entrance orifice is the control at stages up to the principal spillway inlet. Above this elevation, the culvert barrel is the control.

The embankment and reservoir pools at the principal spillway crest, emergency spillway crest and the embankment crest are shown in Figure 2. Currently, no structures exist within any of the pool extents.

**Table 2:** Stage-storage and stage-discharge, structure 1.

| elevation (feet) | Volume (acre-feet) | Spillway Outflow (cfs) Principal & Drawdown | Emergency East | Emergency West | Total Outflow (cfs) |
|---|---|---|---|---|---|
| 3560.0 | 0 | 0 | 0 | 0 | 0.0 |
| 3562.0 | 1.4 | 2.5 | 0 | 0 | 2.5 |
| 3564.0 | 4.3 | 3.7 | 0 | 0 | 3.7 |
| 3566.0 | 8.0 | 4.6 | 0 | 0 | 4.6 |
| 3567.1 | 10.4 | 5.1 | 0 | 0 | 5.1 |
| 3568.0 | 12.4 | 24.1 | 0 | 0 | 24.1 |
| 3570.0 | 17.4 | 25.9 | 0 | 0 | 25.9 |
| 3572.0 | 23.2 | 27.6 | 0 | 0 | 27.6 |
| 3574.0 | 29.8 | 29.2 | 0 | 0 | 29.2 |
| 3576.0 | 37.3 | 30.7 | 0 | 0 | 30.7 |
| 3578.0 | 45.8 | 32.1 | 0 | 0 | 32.1 |
| 3580.0 | 55.1 | 33.5 | 0 | 0 | 33.5 |
| 3582.0 | 65.5 | 34.8 | 0 | 0 | 34.8 |
| 3583.5 | 74.1 | 35.8 | 0 | 0 | 35.8 |
| 3583.7 | 75.3 | 35.9 | 23 | 0 | 58.4 |
| 3584.0 | 77.0 | 36.1 | 89 | 32 | 157 |
| 3585.0 | 83.5 | 36.7 | 463 | 291 | 790 |
| 3586.0 | 89.9 | 37.3 | 996 | 684 | 1717 |
| 3587.0 | 96.8 | 37.9 | 1650 | 1175 | 2863 |
| 3588.0 | 103.7 | 38.5 | 2406 | 1748 | 4192 |
| 3589.0 | 111.2 | 39.1 | 3250 | 2391 | 5681 |
| 3590.0 | 118.6 | 39.6 | 4176 | 3099 | 7315 |

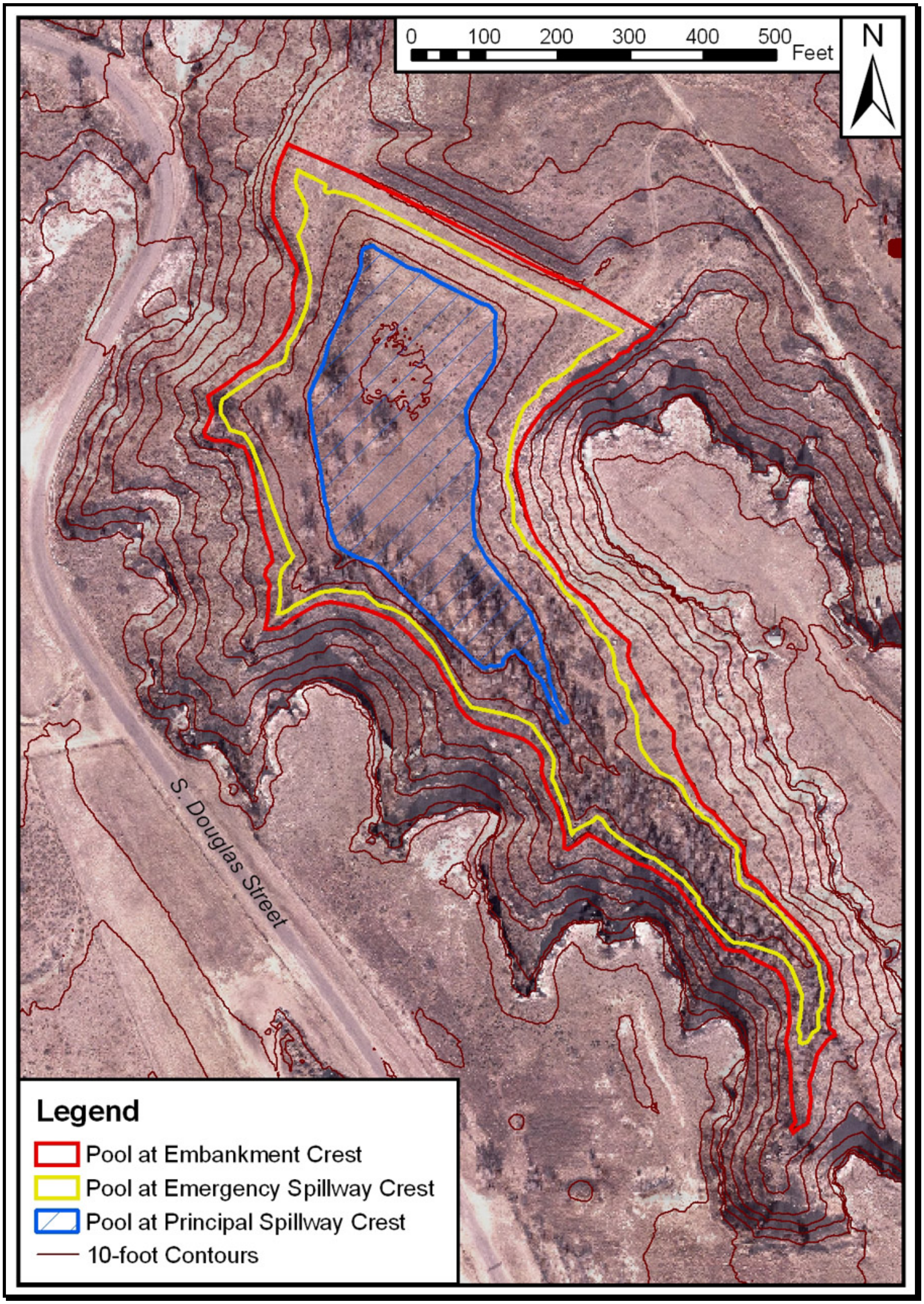



**Figure 2:** Reservoir pools, structure 1.

## Structure 2

Structure 2 is an embankment dam with one principal spillway and one emergency spillway. There is not an additional drawdown pipe. The principal spillway inlet is a single 0.7 feet by 1.25 feet vertical orifice in a concrete riser that is drained by an 18-inch reinforced concrete pipe. The drawdown pipe is an 8 inch asphalt-coated welded steel pipe that drains into the concrete principal spillway riser. The principal spillway outfall is through a corrugated metal pipe. The orifice entrance was modeled with an orifice coefficient of 0.7. The emergency spillway has a bottom width of 40 feet and an average width of approximately 46 feet. The entrance orifice is the control at all computed flow stages.

The embankment and reservoir pools at the principal spillway crest, emergency spillway crest and the embankment crest are shown in Figure 3. Currently, no structures exist within any of the pool extents.

**Table 3:** Stage-storage and stage-discharge, structure 2.

| elevation (feet) | Volume (acre-feet) | Spillway Outflow (cfs) Principal | Emergency | Total Outflow (cfs) |
|---|---|---|---|---|
| 3576.0 | 0 | 1.5 | 0 | 1.5 |
| 3576.3 | 0.3 | 1.6 | 0 | 1.6 |
| 3578.0 | 1.7 | 8.5 | 0 | 8.5 |
| 3580.0 | 4.0 | 12.1 | 0 | 12.1 |
| 3582.0 | 7.0 | 14.8 | 0 | 14.8 |
| 3584.0 | 10.5 | 17.1 | 0 | 17.1 |
| 3586.0 | 14.8 | 19.1 | 0 | 19.1 |
| 3586.5 | 16.1 | 19.6 | 0 | 19.6 |
| 3587.0 | 18.0 | 20.0 | 45.5 | 65.6 |
| 3588.0 | 19.9 | 20.9 | 236.6 | 258 |
| 3589.0 | 22.8 | 21.7 | 509.1 | 531 |
| 3590.0 | 25.6 | 22.6 | 843.4 | 866 |
| 3590.9 | 28.4 | 23.3 | 1188.8 | 1212 |

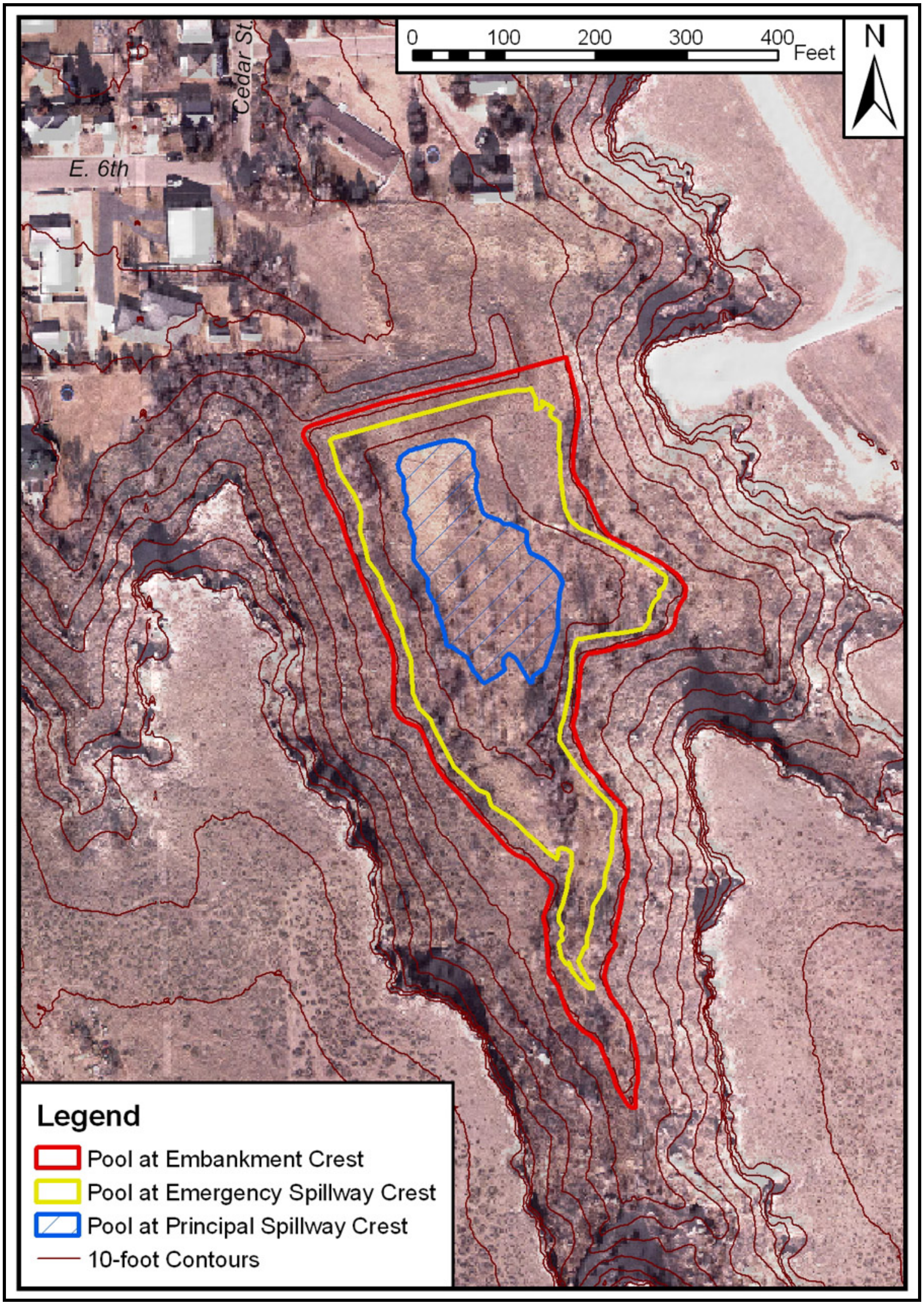



**Figure 3:** Reservoir pools, structure 2.

### Structure 3

Structure 3 is an embankment dam with one principal spillway, a drawdown pipe, and one emergency spillway. The principal spillway inlet is a single 0.7 feet by 1.5 feet vertical orifice in a concrete riser that is drained by an 18-inch reinforced concrete pipe. The drawdown pipe is an 8 inch asphalt-coated welded steel pipe that drains into the concrete principal spillway riser. The principal spillway outfall is through a corrugated metal pipe. The orifice entrances were modeled with a coefficient of 0.7. The emergency spillway has a bottom width of 180 feet and an average width of approximately 190 feet. The drawdown pipe and principal spillway entrance orifices are the control at lower reservoir stages while the outlet pipe barrel is the control at higher reservoir stages.

The embankment and reservoir pools at the principal spillway crest, emergency spillway crest and the embankment crest are shown in Figure 4. Currently, no structures exist within any of the pool extents, though two residences are very close to the embankment crest pool extent.

**Table 4:** Stage-storage and stage-discharge, structure 3.

| elevation (feet) | Volume (acre-feet) | Spillway Outflow (cfs) Principal & Drawdown | Spillway Outflow (cfs) Emergency | Total Outflow (cfs) |
|---|---|---|---|---|
| 3646.0 | 0 | 2.8 | 0 | 2.8 |
| 3648.0 | 0.5 | 3.9 | 0 | 3.9 |
| 3650.0 | 1.5 | 4.8 | 0 | 4.8 |
| 3652.0 | 2.8 | 5.5 | 0 | 5.5 |
| 3652.1 | 2.9 | 5.6 | 0 | 5.6 |
| 3654.0 | 4.4 | 14.3 | 0 | 14.3 |
| 3656.0 | 6.4 | 18.4 | 0 | 18.4 |
| 3658.0 | 8.6 | 21.7 | 0 | 21.7 |
| 3660.0 | 11.2 | 24.4 | 0 | 24.4 |
| 3662.0 | 14.0 | 26.9 | 0 | 26.9 |
| 3666.0 | 20.3 | 30.6 | 0 | 30.6 |
| 3670.0 | 28.3 | 32.8 | 0 | 32.8 |
| 3670.6 | 29.8 | 33.1 | 0 | 33.1 |
| 3671.0 | 30.8 | 33.3 | 135 | 168 |
| 3672.0 | 33.4 | 33.9 | 881 | 915 |
| 3673.0 | 36.5 | 34.4 | 1978 | 2012 |
| 3674.0 | 39.5 | 34.9 | 3335 | 3370 |
| 3675.0 | 43.2 | 35.4 | 4910 | 4945 |
| 3675.5 | 45.1 | 35.6 | 5770 | 5806 |

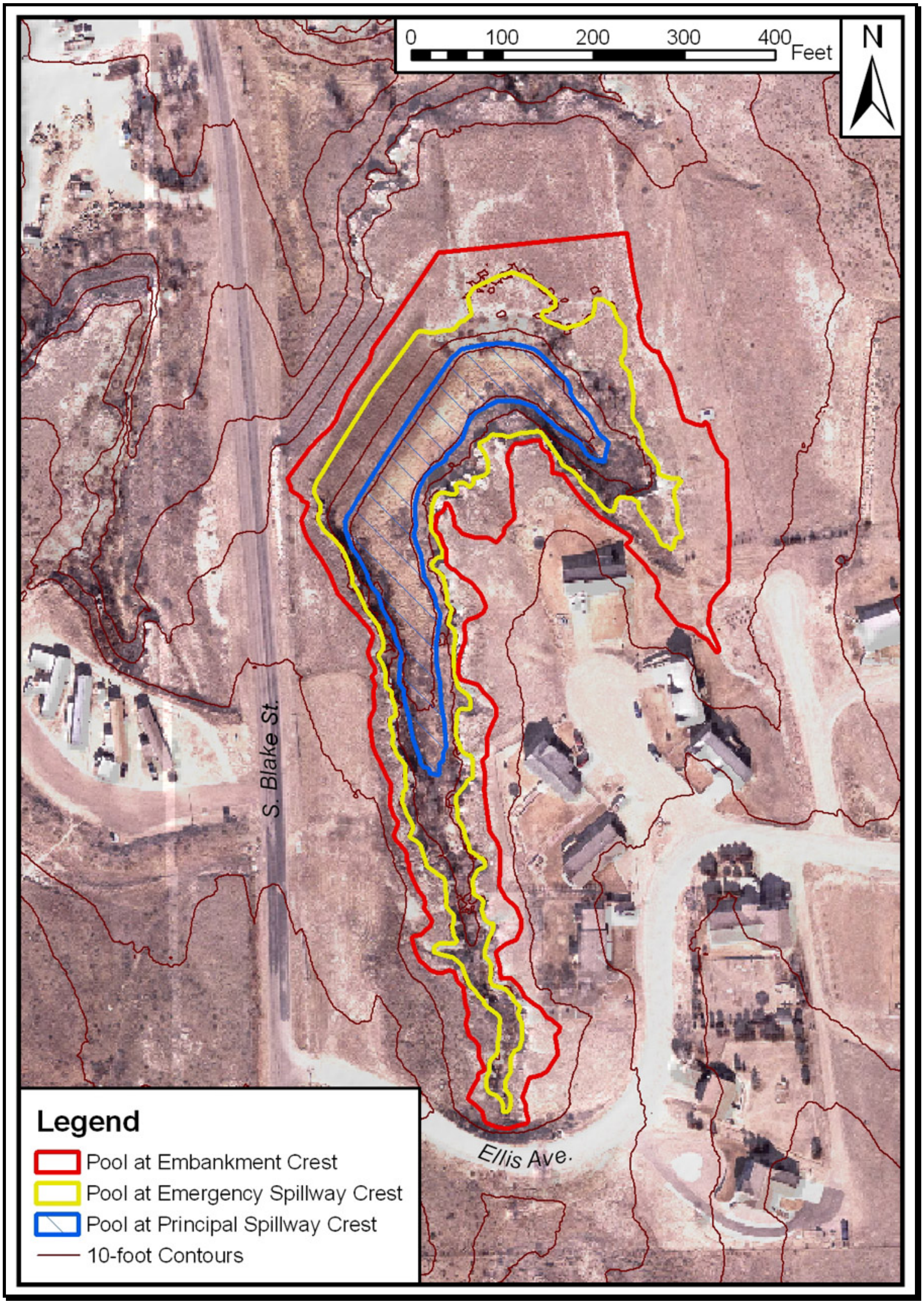



**Figure 4:** Reservoir pools, structure 3.

### Structure 4

Structure 4 is an embankment dam with one principal spillway, a drawdown pipe, and one emergency spillway. The principal spillway inlet is a complex, anti-vortex, multiple inlet structure with two 0.68 x 1.25 ft vertical orifices and two additional 1.05 x 3.5 ft vertical orifices. The drawdown pipe is a 12-inch asphalt-coated welded steel pipe that drains into the concrete principal spillway riser. The concrete riser is drained by an 18-inch reinforced concrete pipe. The principal spillway outfall is through a corrugated metal pipe. The orifice entrances were modeled with a coefficient of 0.7. The emergency spillway has a bottom width of 180 feet and an average width of approximately 186 feet. The drawdown pipe entrance and lower riser orifices are the control at lower reservoir stages. At higher stages the outlet pipe barrel is the control.

The embankment and reservoir pool at the principal spillway crest, emergency spillway crest and the embankment crest are shown in Figure 5. A number of structures are located within the emergency spillway crest pool and the embankment crest pool.

**Table 5:** Stage-storage and stage-discharge, structure 4.

| elevation (feet) | Volume (acre-feet) | Spillway Outflow (cfs) Principal & Drawdown | Spillway Outflow (cfs) Emergency | Total Outflow (cfs) |
|---|---|---|---|---|
| 3578.0 | ---- | 0 | 0 | 3.4 |
| 3580.0 | 1.0 | 7.1 | 0 | 7.1 |
| 3582.0 | 3.7 | 9.5 | 0 | 9.5 |
| 3584.0 | 8.8 | 11.3 | 0 | 11.3 |
| 3586.0 | 17.1 | 26.4 | 0 | 26.4 |
| 3588.0 | 28.3 | 26.9 | 0 | 26.9 |
| 3590.0 | 42.2 | 28.9 | 0 | 28.9 |
| 3592.0 | 59.4 | 30.7 | 0 | 30.7 |
| 3592.8 | 67.6 | 31.4 | 0 | 31.4 |
| 3593.0 | 69.6 | 31.6 | 47 | 78.2 |
| 3594.0 | 79.8 | 32.5 | 685 | 717 |
| 3595.0 | 94.5 | 33.3 | 1699 | 1733 |
| 3596.0 | 103.1 | 34.1 | 2981 | 3015 |
| 3597.0 | 116.4 | 34.9 | 4483 | 4518 |
| 3598.0 | 129.7 | 35.7 | 6176 | 6211 |
| 3598.8 | 141.5 | 36.3 | 7654 | 7690 |

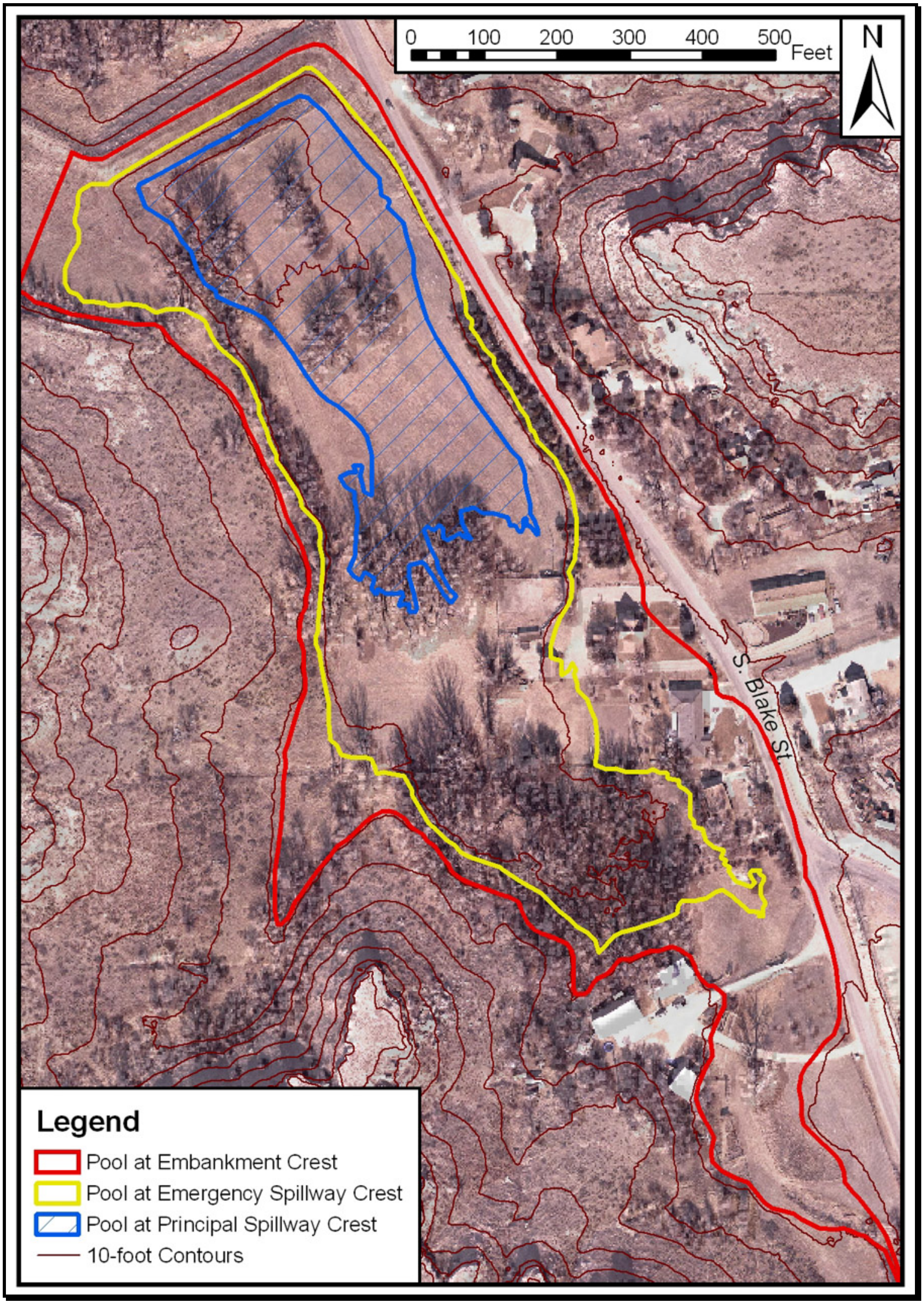



**Figure 5:** Reservoir pools, structure 4.

**Structure 5**

Structure 5 is an embankment dam with one principal spillway, a drawdown pipe, and one emergency spillway. The principal spillway inlet is a single 0.93 feet by 1.9 feet vertical orifice in a concrete riser that is drained by an 18-inch reinforced concrete pipe. The drawdown pipe is an 8 inch asphalt-coated welded steel pipe that drains into the concrete principal spillway riser. The principal spillway outfall is through a corrugated metal pipe. The orifice entrances were modeled with a coefficient of 0.7. The emergency spillway has a bottom width of 120 feet and an average width of approximately 135 feet. The drawdown pipe entrance and lower riser orifices are the control at lower reservoir stages. At higher stages the outlet pipe barrel is the control.

The embankment and reservoir pool at the principal spillway crest, emergency spillway crest and the embankment crest are shown in Figure 6. Currently, no structures exist within any of the pool extents.

**Table 6:** Stage-storage and stage-discharge, structure 5.

| | | Spillway Outflow (cfs) | | Total |
|---|---|---|---|---|
| elevation (feet) | Volume (acre-feet) | Principal & Drawdown | Emergency | Outflow (cfs) |
| 3604.0 | 0.0 | 0 | 0 | 0.0 |
| 3606.0 | 1.3 | 3.4 | 0 | 3.4 |
| 3608.0 | 3.4 | 4.4 | 0 | 4.4 |
| 3608.7 | 4.3 | 4.7 | 0 | 4.7 |
| 3610.0 | 5.9 | 5.2 | 0 | 5.2 |
| 3612.0 | 9.2 | 23.9 | 0 | 23.9 |
| 3614.0 | 13.2 | 29.4 | 0 | 29.4 |
| 3616.0 | 18.0 | 31.3 | 0 | 31.3 |
| 3618.0 | 23.7 | 33.0 | 0 | 33.0 |
| 3619.4 | 28.3 | 34.2 | 0 | 34.2 |
| 3620.0 | 30.2 | 34.6 | 176 | 210 |
| 3621.0 | 34.0 | 35.4 | 765 | 800 |
| 3622.0 | 37.7 | 36.2 | 1585 | 1621 |
| 3623.0 | 41.9 | 36.9 | 2582 | 2619 |
| 3624.0 | 46.1 | 37.7 | 3729 | 3767 |
| 3624.7 | 49.4 | 38.2 | 4612 | 4650 |

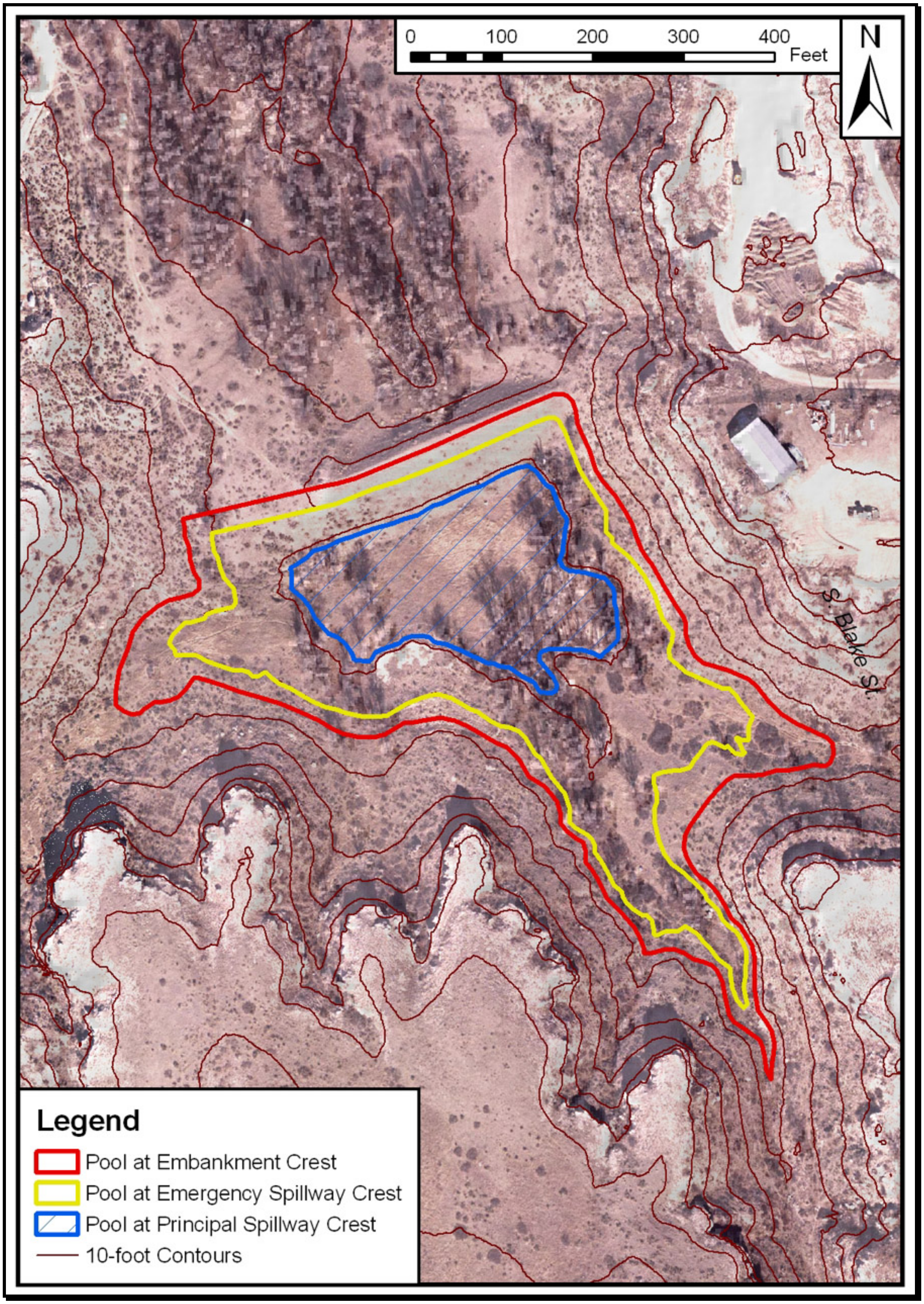



**Figure 6:** Reservoir pools, structure 5.

## Structure 6

Structure 6 is an embankment dam with one principal spillway, a drawdown pipe, and two emergency spillways. The principal spillway inlet is a complex, anti-vortex, multiple inlet structure with two 0.74 x 1.95 ft vertical orifices and two additional 1.25 x 3.5 ft vertical orifices. The drawdown pipe is a 12-inch asphalt-coated welded steel pipe that drains into the concrete principal spillway riser. The concrete riser is drained by a 24-inch reinforced concrete pipe. The principal spillway outfall is through a corrugated metal pipe. The orifice entrances were modeled with a coefficient of 0.7. The (east) emergency spillway has a bottom width of 140 feet and an average width of approximately 145 feet. The (west) emergency spillway has a bottom width of 160 feet and an average width of approximately 165 feet. The drawdown pipe entrance and lower riser orifices are the control at lower reservoir stages. At higher stages the outlet pipe barrel is the control.

The embankment and reservoir pool at the principal spillway crest, emergency spillway crest and the embankment crest are shown in Figure 7. Currently, no structures exist within any of the pool extents.

**Table 7:** Stage-storage and stage-discharge, structure 6.

| | | Spillway Outflow (cfs) | | | Total |
|---|---|---|---|---|---|
| elevation (feet) | Volume (acre-feet) | Principal & Drawdown | Emergency East | Emergency West | Outflow (cfs) |
| 3596.0 | 0.0 | 5.6 | 0 | 0 | 5.6 |
| 3598.0 | 0.8 | 8.4 | 0 | 0 | 8.4 |
| 3600.0 | 3.7 | 10.4 | 0 | 0 | 10.4 |
| 3602.0 | 9.6 | 12.2 | 0 | 0 | 12.2 |
| 3604.0 | 18.9 | 13.7 | 0 | 0 | 13.7 |
| 3605.1 | 25.6 | 14.4 | 0 | 0 | 14.4 |
| 3606.0 | 31.0 | 30.4 | 0 | 0 | 30.4 |
| 3608.0 | 45.6 | 43.8 | 0 | 0 | 43.8 |
| 3610.0 | 62.8 | 53.8 | 0 | 0 | 53.8 |
| 3612.0 | 82.9 | 56.7 | 0 | 0 | 56.7 |
| 3614.0 | 105.9 | 59.4 | 0 | 0 | 59.4 |
| 3616.0 | 131.1 | 62.1 | 0 | 0 | 62.1 |
| 3618.0 | 158.7 | 64.6 | 0 | 0 | 64.6 |
| 3620.0 | 189.5 | 67.0 | 0 | 0 | 67.0 |
| 3620.4 | 196.4 | 67.5 | 0 | 0 | 67.5 |
| 3621.0 | 206.8 | 68.2 | 189 | 0 | 257 |
| 3621.4 | 213.7 | 68.7 | 406 | 0 | 475 |
| 3622.0 | 224.1 | 69.4 | 822 | 215 | 1106 |
| 3623.0 | 243.6 | 70.5 | 1702 | 935 | 2708 |
| 3624.0 | 263.1 | 71.6 | 2773 | 1937 | 4782 |
| 3625.0 | 284.8 | 72.8 | 4006 | 3156 | 7234 |
| 3626.0 | 306.4 | 73.8 | 5380 | 4558 | 10,012 |
| 3626.6 | 320.5 | 74.5 | 6268 | 5478 | 11,821 |

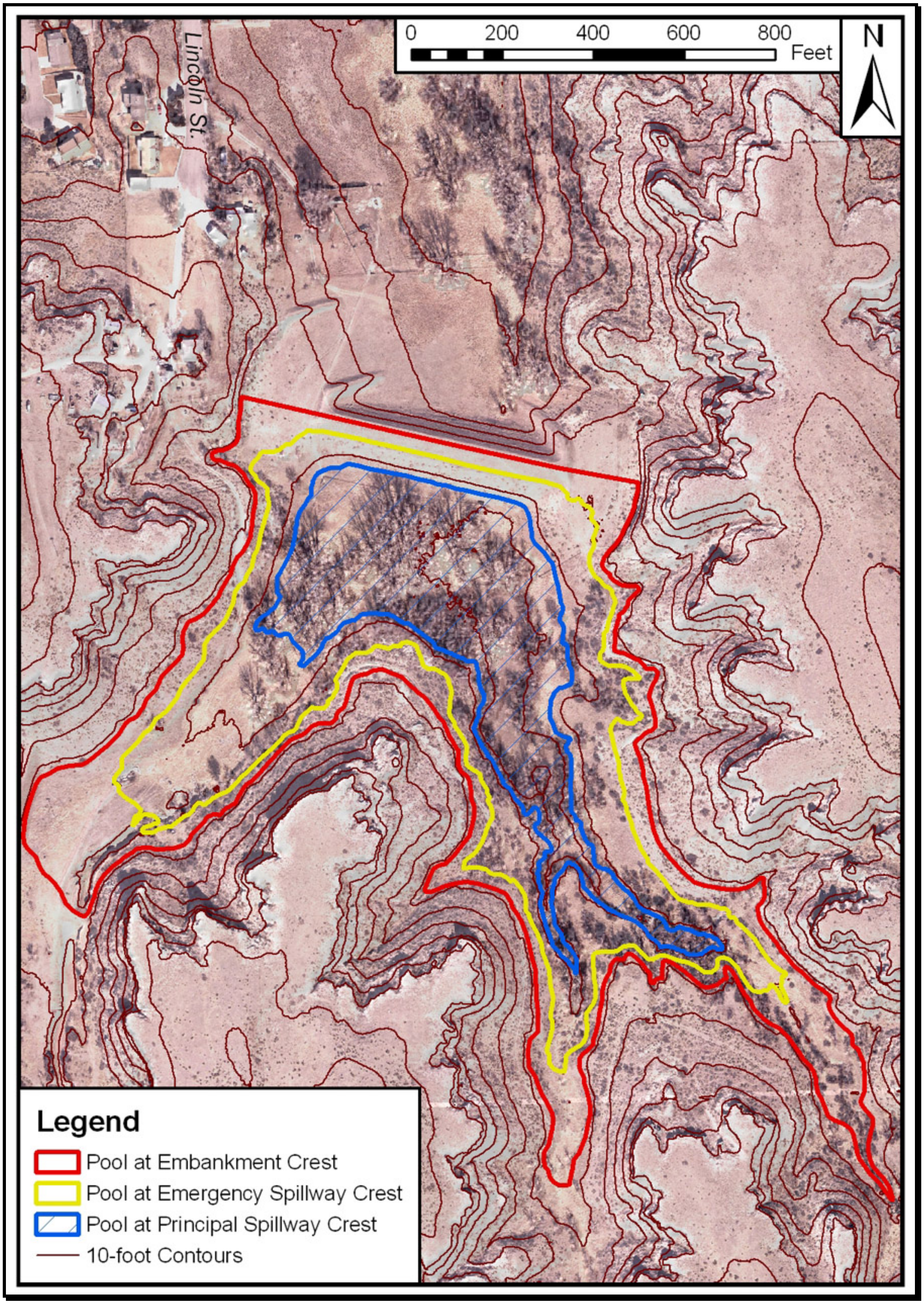



**Figure 7:** Reservoir pools, structure 6.

## PRECIPITATION

The performance of the reservoir structures for a wide range of precipitation events is needed. Accordingly, hydrologic analyses have been performed for 2- 5-, 10-, 25-, 50-, 100-, 200- and 500-year events, as well as the Probable Maximum Precipitation (PMP) event. Both the 6-hour and 24-hour events have been simulated. Additionally, the precipitation return-period of each structure at the crest of the emergency spillway has also been determined. The precipitation was developed from NOAA Atlas II (Miller et. al. 1973). PMP estimates where provided by Schreiner and Riedel (1978). Precipitation depths are provided in Table 8 and Figure 8.

**Table 8:** Precipitation depths, Wray, Colorado.

| Precipitation Frequency | Depth (inches) 6-hour | 24-hour |
|---|---|---|
| 2-year | 2.1 | 2.5 |
| 5-year | 2.7 | 3.1 |
| 10-year | 3.1 | 3.6 |
| 25-year | 3.7 | 4.3 |
| 50-year | 4.2 | 4.9 |
| 100-year | 4.7 | 5.5 |
| 200-year* | 5.2 | 6.2 |
| 500-year* | 6.0 | 6.9 |
| PMP | 24.3 | 30.5 |

* extraploated

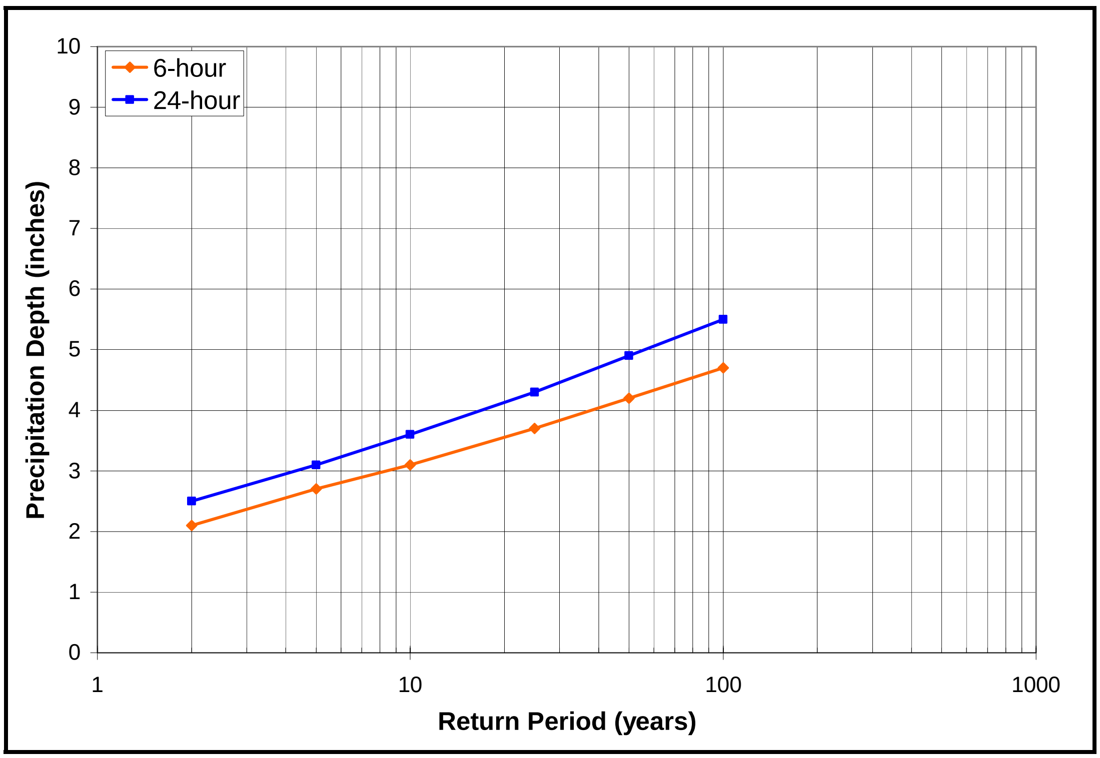



**Figure 8:** Precipitation frequency, 6-hour and 24-hour.

The drainage areas of each of the structures are (well) below 10 square miles – no aerial reduction factors for rainfall have been used.

The NRCS Type II synthetic rainfall distribution (NRCS 1986) has been used for all rainfall events less than the PMP. For the PMP storm, the TR-60 dimensionless design storm distribution (NRCS 2005a) has been used. The storm distribution for the 100-year event is illustrated in Figure 9. Precipitation intensity for the 100-year event, at 15-minute increments, is shown in Figure 10.

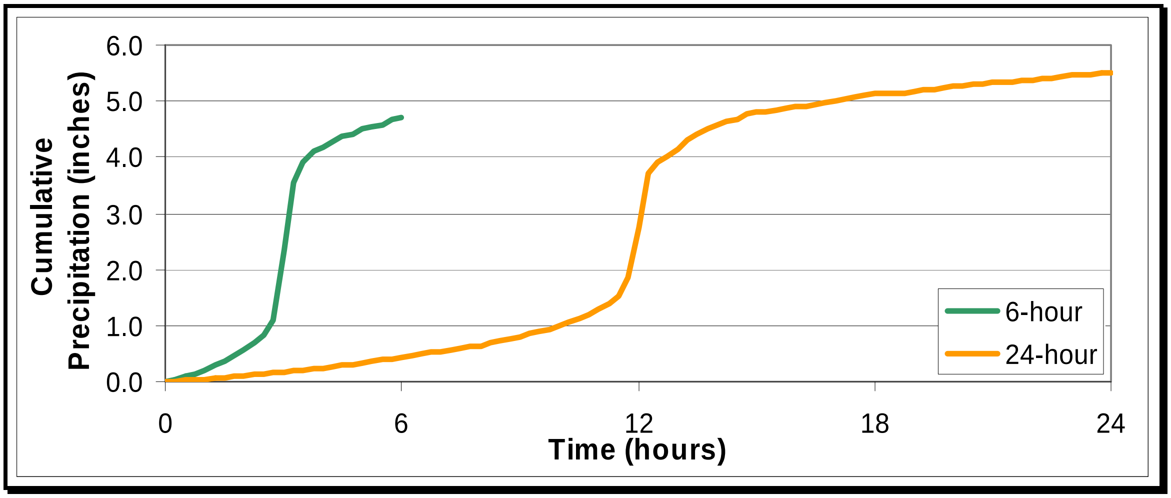



**Figure 9:** 100-year Type II rainfall distribution.

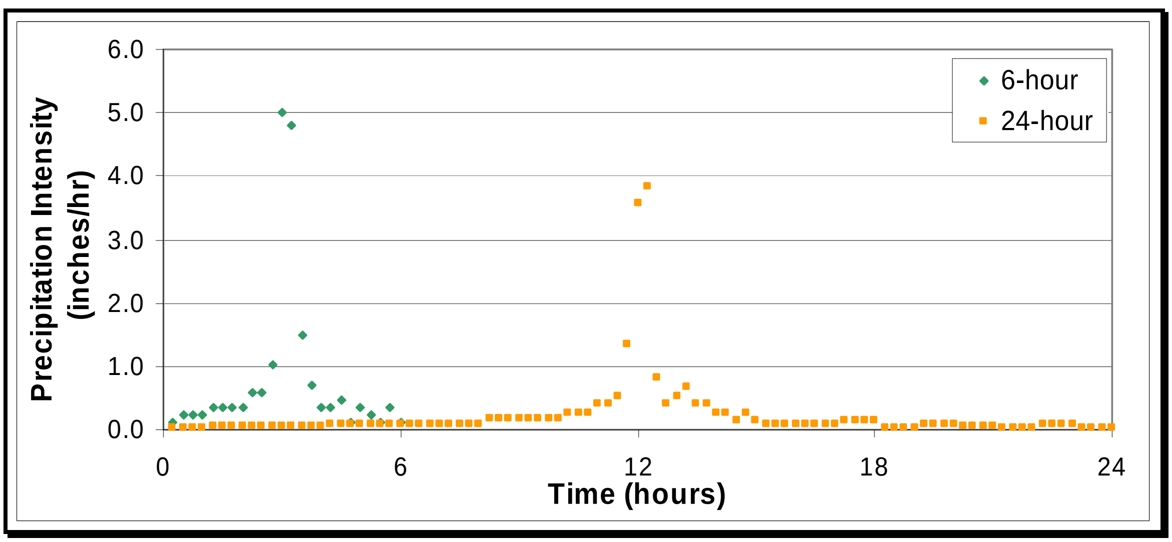



**Figure 10:** 100-year Type II distribution rainfall intensity.

## HYDROLOGIC MODELING

Hydrologic modeling was performed using the program HEC-HMS (version 3.0.1), a model developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center. The NRCS curve number (CN) technique for estimating direct runoff from rain events in ungaged watersheds was used in this analysis.

### Watersheds

Watersheds have been manually delineated using 2-foot contours created from the 1-meter LiDAR elevation grid. The reservoir watershed limits differed at times from the original project delineation – the watersheds were found to be slightly different than the original watershed delineations used in the design. This is due to the high resolution elevation data allowing more accurate estimates. The reservoir watersheds are illustrated in Figure 11 while all modeled watersheds are illustrated in Figure 12.

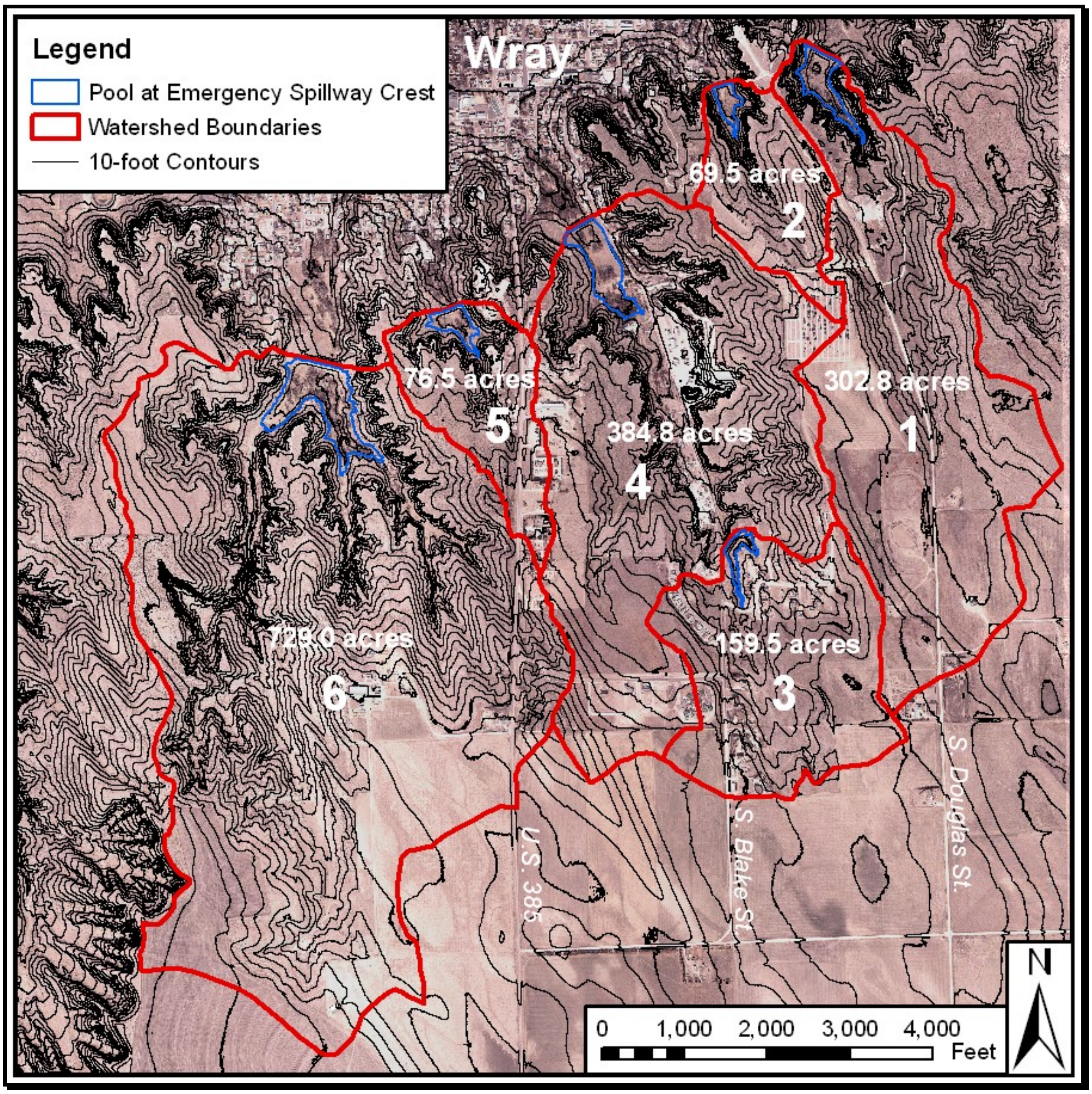



**Figure 11:** Reservoir watershed boundaries.

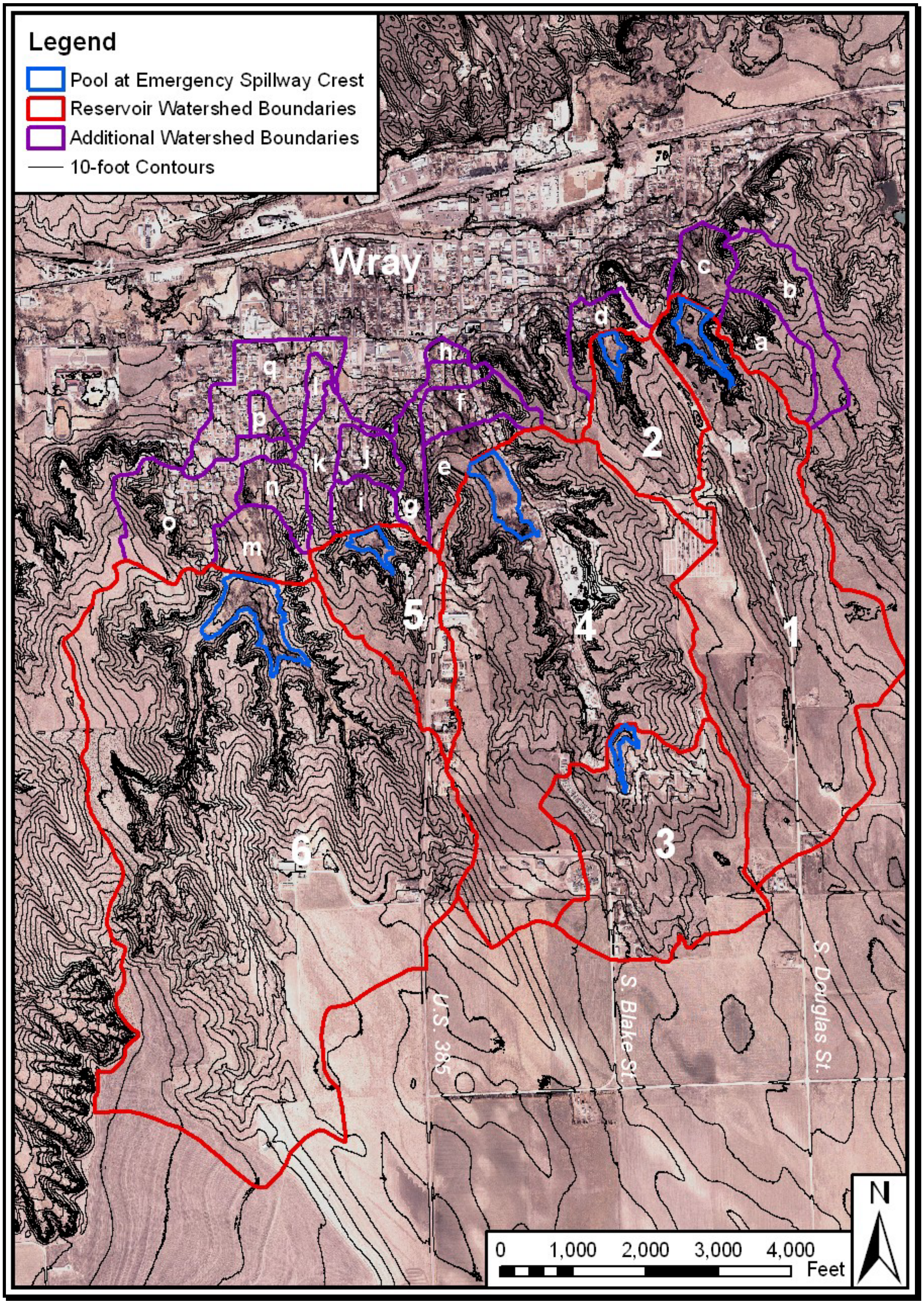



**Figure 12:** All watershed boundaries.

There are a number of closed depressions where surface water has no exit and pools. An example of this is illustrated in Figure 13. These closed depressions were excluded as contributing flow to the watersheds in question in the rainfall-runoff modeling.

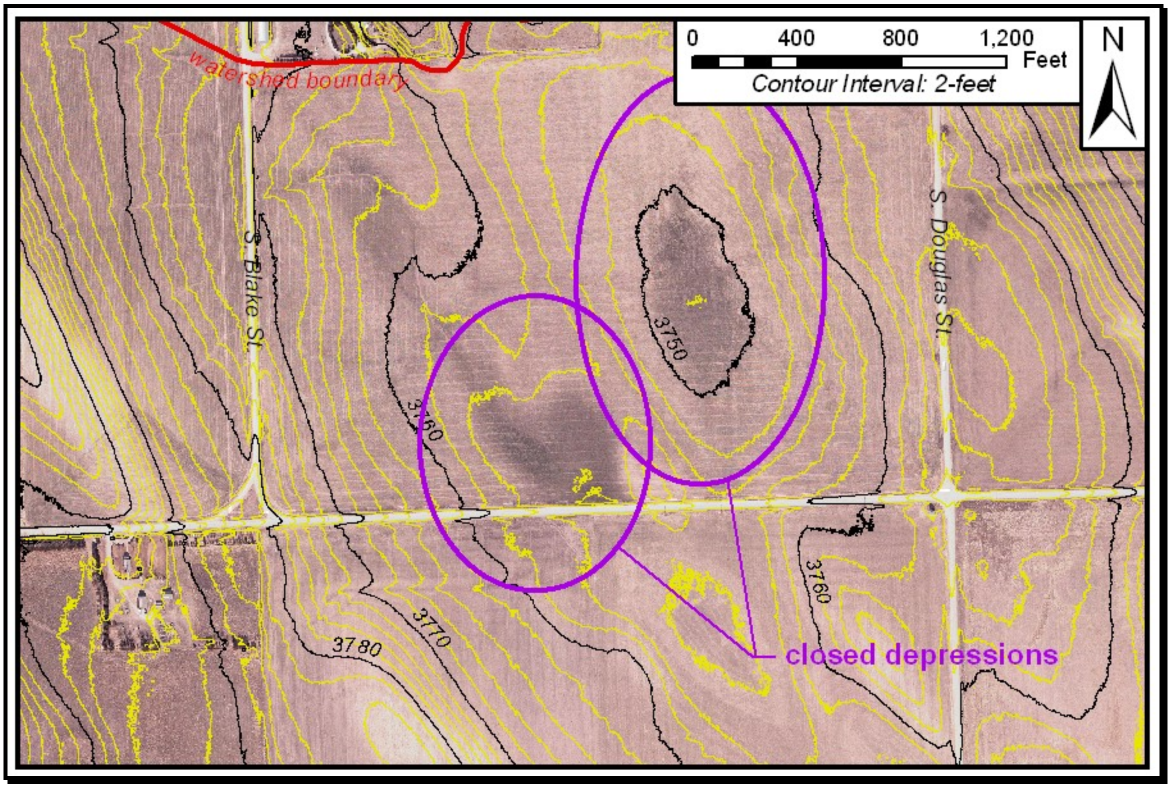



**Figure 13:** Closed depressions just south of the watershed boundaries.

**Model Form**

As documented in NRCS (2004b), the NRCS method for estimating direct runoff from individual storm rainfall events follows the following form:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{if } P > I_a$$

$$Q = 0 \qquad \text{if } P \leq I_a$$

Where Q is the depth of runoff (inches), P is the depth of rainfall (inches), $I_a$ is the initial abstraction (inches), and S is the maximum potential retention (inches). The derivation of this equation is not physically based but does respect conservation of mass (NRCS 2004b).

The Curve Number (CN) is defined as:

$$CN = \frac{1000}{10 + S}$$

The initial abstraction was initially described and has traditionally been used as:

$$I_a = 0.2S$$

This relationship is fairly poor, as Figure 10-1 in NRCS (2004b) illustrates.

## CN Development

The CN method is a simple and widely used technique for estimating a stream hydrograph at the outlet of a watershed. According to NRCS (2004a), the "combination of a hydrologic soil group (soil) and a land use and treatment class (cover) is a hydrologic soil-cover complex". Through catchment-scale empirical studies, each with one complex of hydraulic soil group and cover, runoff curve numbers have been assigned to complexes (Mochus, 1964). Documentation is provided on the method in the NRCS National Engineering Handbook, Section 4, Hydrology, Chapters 9 and 10 (NRCS 2004a, NRCS 2004b), in Rallison (1980), as well as numerous other publications. However, little quantitative information has been published of the data base on which it was developed (Maidment 1992) and many of the curves used in the development have been misplaced (Woodward 2005). The method was developed for rural non-mountainous watersheds in various parts of the United States, within 24 states; was developed for single storms, not continuous or partial storm simulation; and was not intended to recreate a specific response from an actual storm (Rallison, 1980). This latter point is disconcerting but understandable considering that typical condition CNs are being applied to the real-world variability of soil moisture, spatial precipitation variability, variation in precipitation intensity, and interception. Most fundamentally, the conceptual foundation of the CN technique is disconnected with actual streamflow generating processes during more-frequent small to moderate rain events. The CN is a simple watershed-scale method that gives simplified results at a watershed outlet for larger events. This hydrologic study falls within the applicability of the CN method.

### *Land Use*

With the use of the aerial imagery and the assistance of Gary Campfield, District Conservationist, a land use map was developed for the area of analysis. This landuse mapping is provided in Figure 14. The twenty landuse types, with the codes used in this analysis, are provided below:

- 1 - wheat, fallow, low residue
- 2 - wheat, fallow, medium residue
- 3 - range/pasture
- 4 - farmstead
- 5 - wheat, corn, fallow, high residue
- 6 - wheat, corn, fallow, high residue
- 7 - steep, rocky rangeland
- 8 - paved road
- 9 - pasture, terraced
- 10 - unpaved road
- 11 - trailer park
- 12 - residential
- 13 - commercial/industrial
- 14 - cemetery
- 15 - urban
- 16 - trees, brush
- 17 - wooded

- 18 - water
- 19 - railroad
- 20 - quarry

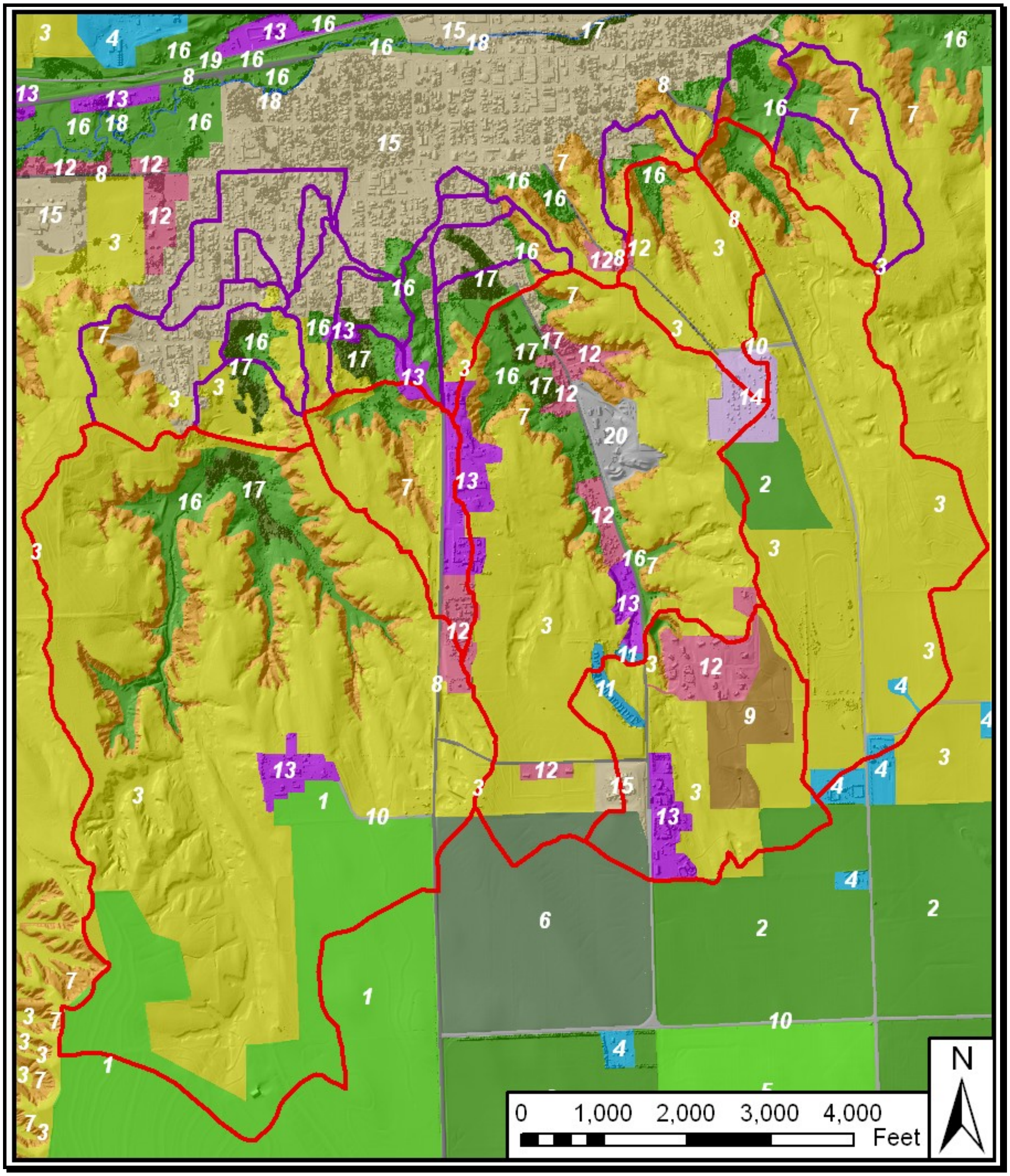



**Figure 14:** Land use of the Wray area.

### *Hydrologic Soil Group Classification*

As described by the NRCS, hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are placed into four groups A, B, C, and D, and three dual classes, A/D, B/D, and C/D. Definitions of the classes are as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only soils that are rated D in their natural condition are assigned to dual classes.

Hydrologic groups for the soils of the reservoir watersheds, as provided by the Yuma county NRCS soil survey, are provided in Figure 15.

### *CN Assignments*

Table 9 provides a list of landuse types and their associated estimated CN values for hydrologic groups B and D. CN estimates have developed through the use of NRCS (2004a).

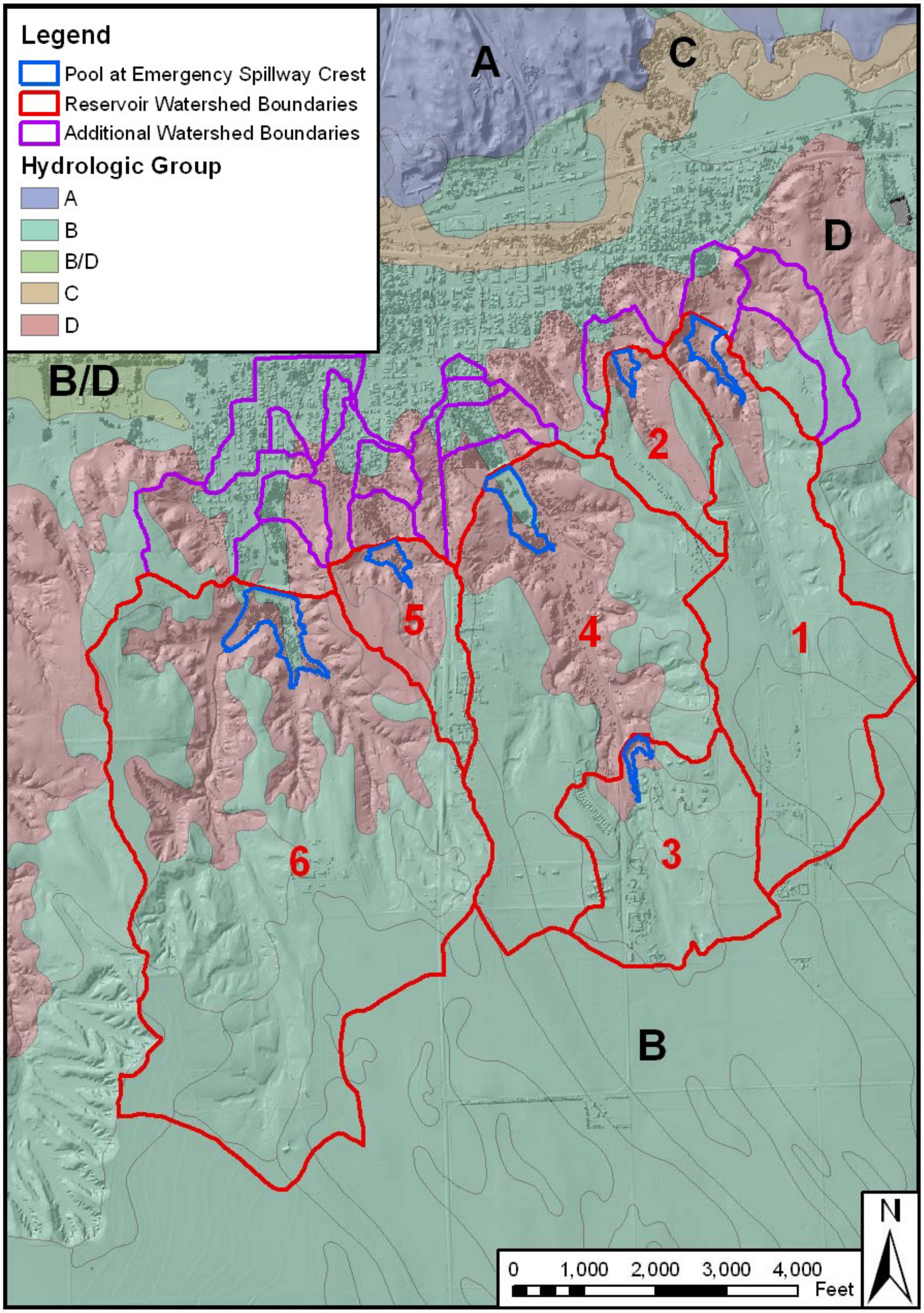



**Figure 15:** Hydrologic soil group classification.

**Table 9:** CN assignments for hydrologic soil groups B and D.

| Landuse Description | ID | B | D |
|---|---|---|---|
| wheat, fallow, low residue | 1 | 75 | 86 |
| wheat, fallow, medium residue | 2 | 72 | 84 |
| range/pasture | 3 | 69 | 84 |
| farmstead | 4 | 74 | 86 |
| wheat, corn, fallow, high residue | 5 | 80 | 90 |
| wheat, fallow, sorghum, medium residue | 6 | 72 | 84 |
| steep, rocky range land | 7 | 85 | 92 |
| paved road | 8 | 98 | 98 |
| pasture, terraced | 9 | 61 | 80 |
| unpaved road | 10 | 82 | 89 |
| trailer park | 11 | 75 | 87 |
| residential | 12 | 68 | 84 |
| commercial/industrial | 13 | 88 | 93 |
| cemetary | 14 | 69 | 84 |
| urban | 15 | 85 | 92 |
| trees, brush | 16 | 66 | 83 |
| wooded | 17 | 60 | 79 |
| water | 18 | 98 | 98 |
| railroad | 19 | 85 | 91 |
| quarry | 20 | 86 | 94 |

### *Catchment Composite CNs*

The soils, landuse and watersheds shapefiles were merged. The Table 9 CN assignments were applied to this merged file to develop a CN for each of the 375 resulting polygons. Table 10 list the composite CNs, as well as other characteristics, for the 23 watersheds illustrated in Figure 12.

## Initial Abstraction

Recently, it has been suggested that the use of an initial abstraction, $I_a$, of 0.2S, where S is the maximum potential retention after runoff begins, is too high. Instead, it has been found that the use of 0.05S is more appropriate (NRCS 2005b). To make use of the most-recently available information, it would have been preferred to use an $I_a$ of 0.05S. However, since changing the $I_a$ assumption would change the CNs listed in NRCS (2004a), an $I_a$ of 0.2S was used in this analysis. Initial abstraction estimates for each watershed is provided in Table 10.

## Lag-Time Estimates

Using the physically-simplified CN methodology, precipitation that is not initially abstracted or infiltrated becomes excess precipitation that flows down-gradient to the sub-basin outlet, which is modeled using a transform method. HEC-HMS allows the use of transform methods to route excess flow to the mouth of each sub-basin, but this method is not preferred since the CN technique is typically used with a time-of-concentration estimate. This latter method was used in this analysis.

The methods documented in SCS 1972, NEH Section 4, Chapter 15, were used to compute lag estimates for each sub basin. Results are provided in Table 10. The method applied to this analysis consisted of the use of the following equation (SCS 1972), created from research watersheds:

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}$$

where

$$S = \frac{1000}{CN'} - 10$$

and where $L$ is lag time in hours, $l$ is the hydraulic length, $Y$ is the average watershed slope in percent, and *CN'* is a measure of the runoff retardance. *CN'* has been approximated by the CN. The average watershed slope was computed using the 1-meter LiDAR data in GIS.

**Table 10:** Composite CNs and other characteristics for the Wray watersheds illustrated in Figure 12.

| Sub-Basin ID | Area (acres) | Composite CN | Average Slope | Initial Abstraction (inches) | Lag Time (hours) |
|---|---|---|---|---|---|
| 1 | 302.8 | 71.7 | 9.3 | 0.79 | 0.73 |
| 2 | 69.5 | 77.0 | 16.0 | 0.60 | 0.22 |
| 3 | 159.5 | 71.4 | 8.4 | 0.80 | 0.40 |
| 4 | 384.8 | 76.5 | 12.7 | 0.61 | 0.51 |
| 5 | 76.5 | 84.2 | 15.2 | 0.38 | 0.19 |
| 6 | 729.0 | 75.3 | 14.0 | 0.66 | 0.60 |
| a | 29.0 | 76.6 | 18.0 | 0.61 | 0.16 |
| b | 34.8 | 80.3 | 14.7 | 0.49 | 0.20 |
| c | 18.7 | 82.0 | 20.1 | 0.44 | 0.09 |
| d | 16.4 | 85.3 | 21.2 | 0.34 | 0.08 |
| e | 22.3 | 83.5 | 18.0 | 0.40 | 0.12 |
| f | 20.0 | 82.9 | 14.9 | 0.41 | 0.06 |
| g | 22.2 | 86.2 | 16.9 | 0.32 | 0.13 |
| h | 3.3 | 85.0 | 9.1 | 0.35 | 0.05 |
| i | 13.6 | 83.6 | 18.1 | 0.39 | 0.08 |
| j | 13.4 | 84.8 | 16.5 | 0.36 | 0.09 |
| k | 20.6 | 86.2 | 10.9 | 0.32 | 0.16 |
| l | 5.9 | 85.4 | 5.2 | 0.34 | 0.15 |
| m | 24.0 | 75.5 | 16.8 | 0.65 | 0.13 |
| n | 17.1 | 79.3 | 14.2 | 0.52 | 0.13 |
| o | 54.0 | 82.9 | 15.1 | 0.41 | 0.16 |
| p | 5.2 | 86.1 | 7.7 | 0.32 | 0.07 |
| q | 41.8 | 85.3 | 5.2 | 0.34 | 0.25 |

## Reservoir Routing

Reservoir stage-storage and stage-discharge tables were developed from flood pool geometry and the principal and emergency spillway characteristics of each structure, as described in the FLOOD RETENTION STRUCTURES section. These tables were inputted into HEC-HMS for the reservoir routing computations.

## HYDROLOGIC MODELING RESULTS

Hydrologic analyses have been performed for 2- 5-, 10-, 25-, 50-, 100-, 200- and 500-year events, as well as the Probable Maximum Precipitation (PMP) event. For all frequencies, both the 6-hour and 24-hour events have been simulated for the reservoirs. For the other watersheds, the 6-hour event was modeled, since this event was found to create higher peak flows for unregulated watersheds. A 2-minute time step was used. Simulations were performed for 5 days, which envelope the hydrologic response for all structures and rainfall frequencies. The results for each structure are provided in the following tables.

For the location of each of the six structures, see Figure 1.

**Structure 1** was found to be capable of passing an event greater than the 500-year event through the principal spillway without use of the emergency spillways. The 500-year, 24-hour event is estimated to fill the reservoir to within 1.6 feet of the emergency spillway crest, while passing a peak flow of 34.7 cfs. The 100-year, 24-hour event is estimated to fill the reservoir to within 6.4 feet of the emergency spillway crest, while passing a peak flow of 31.5 cfs. The 6-hour probable maximum precipitation event is estimated to fill the reservoir to within 3.0 feet of the crest of the embankment, while passing a peak flow of 3850 cfs. Substantial attenuation will occur for the 2-year through 500-year events, with the reservoir peak outflow ranging from 7 to 21 percent of the peak inflow. A summary of the analysis results is provided in Table 11.

**Table 11:** Hydrologic modeling results summary, structure 1. Emergency spillway crest elevation = 3583.5 feet.

| Storm Description | Peak Inflow (cfs) | Peak Outflow (cfs) | Peak Storage (acre-feet) | Peak Elevation (feet) |
|---|---|---|---|---|
| **2-year, 6-hour** | 58.2 | 4.4 | 7.0 | 3565.5 |
| **2-year, 24-hour** | 57.4 | 4.7 | 8.6 | 3566.3 |
| **5-year, 6-hour** | 117 | 21.4 | 12.1 | 3567.9 |
| **5-year, 24-hour** | 104 | 22.0 | 12.2 | 3567.9 |
| **10-year, 6-hour** | 164 | 25.4 | 15.9 | 3569.4 |
| **10-year, 24-hour** | 147 | 25.5 | 16.3 | 3569.5 |
| **25-year, 6-hour** | 242 | 27.8 | 24.2 | 3572.3 |
| **25-year, 24-hour** | 215 | 28.0 | 24.8 | 3572.5 |
| **50-year, 6-hour** | 312 | 29.6 | 32.0 | 3574.6 |
| **50-year, 24-hour** | 282 | 29.9 | 33.1 | 3574.9 |
| **100-year, 6-hour** | 388 | 31.2 | 40.5 | 3576.8 |
| **100-year, 24-hour** | 346 | 31.5 | 42.0 | 3577.1 |
| **200-year*, 6-hour** | 466 | 32.7 | 49.5 | 3578.8 |
| **200-year*, 24-hour** | 426 | 33.2 | 53.1 | 3579.6 |
| **500-year*, 6-hour** | 598 | 34.7 | 64.6 | 3581.8 |
| **500-year*, 24-hour** | 509 | 34.7 | 64.9 | 3581.9 |
| **PMP, 6-hour** | 3846 | 2811 | 96.5 | 3587.0 |
| **PMP, 24-hour** | 1538 | 1531 | 88.6 | 3585.8 |

* extrapolated precipitation value

**Structure 2** was found to be capable of passing an event greater than the 500-year event through the principal spillway without the use of the emergency spillway. The 500-year, 6-hour event is estimated to fill the reservoir to within 0.3 feet of the emergency spillway crest, while passing a peak flow of 19.3 cfs. The 100-year, 6-hour event is estimated to fill the reservoir to within 2.8 feet of the emergency spillway crest, while passing a peak flow of 16.8 cfs. The 6-hour probable maximum precipitation event is estimated to fill the reservoir to within 0.5 feet of the crest of the embankment, while passing a peak flow of 1010 cfs. Substantial attenuation will occur for the 2-year through 500-year events, with the reservoir peak outflow ranging from 7 to 20 percent of the peak inflow. A summary of the analysis results is provided in Table 12.

**Table 12:** Hydrologic modeling results summary, structure 2. Emergency spillway crest elevation = 3586.5 feet.

| Storm Description | Peak Inflow (cfs) | Peak Outflow (cfs) | Peak Storage (acre-feet) | Peak Elevation (feet) |
|---|---|---|---|---|
| **2-year, 6-hour** | 41.2 | 8.2 | 1.6 | 3577.9 |
| **2-year, 24-hour** | 40.4 | 8.2 | 1.6 | 3577.9 |
| **5-year, 6-hour** | 73.2 | 10.7 | 3.1 | 3579.2 |
| **5-year, 24-hour** | 62.7 | 10.4 | 2.9 | 3579.0 |
| **10-year, 6-hour** | 96.2 | 12.3 | 4.2 | 3580.2 |
| **10-year, 24-hour** | 83.2 | 12.2 | 4.1 | 3580.1 |
| **25-year, 6-hour** | 132 | 14.1 | 6.2 | 3581.5 |
| **25-year, 24-hour** | 114 | 14.0 | 6.1 | 3581.4 |
| **50-year, 6-hour** | 164 | 15.5 | 8.1 | 3582.6 |
| **50-year, 24-hour** | 141 | 15.4 | 7.9 | 3582.5 |
| **100-year, 6-hour** | 198 | 16.8 | 10.0 | 3583.7 |
| **100-year, 24-hour** | 168 | 16.7 | 9.9 | 3583.6 |
| **200-year*, 6-hour** | 231 | 17.8 | 12.0 | 3584.7 |
| **200-year*, 24-hour** | 200 | 17.9 | 12.3 | 3584.8 |
| **500-year*, 6-hour** | 287 | 19.3 | 15.4 | 3586.2 |
| **500-year*, 24-hour** | 234 | 19.1 | 14.7 | 3586.0 |
| **PMP, 6-hour** | 1076 | 1008 | 26.8 | 3590.4 |
| **PMP, 24-hour** | 396 | 394 | 21.3 | 3588.5 |

* extrapolated precipitation value

**Structure 3** was found to be capable of passing an event approximately equal to the 500-year event through the principal spillway without the use of the emergency spillway. The 500-year, 6-hour event is estimated to fill the reservoir to crest of the emergency spillway. The 100-year, 24-hour event is estimated to fill the reservoir to within 5.5 feet of the emergency spillway crest, while passing a peak flow of 29.8 cfs. The 6-hour probable maximum precipitation event is estimated to fill the reservoir to within 2.5 feet of the crest of the embankment, while passing a peak flow of 1960 cfs. Substantial attenuation will occur for the 2-year through 500-year events, with the reservoir peak outflow ranging from 9 to 20 percent of the peak inflow. A summary of the analysis results is provided in Table 13.

**Table 13:** Hydrologic modeling results summary, structure 3. Emergency spillway crest elevation = 3670.6 feet.

| Storm Description | Peak Inflow (cfs) | Peak Outflow (cfs) | Peak Storage (acre-feet) | Peak Elevation (feet) |
|---|---|---|---|---|
| **2-year, 6-hour** | 42.7 | 5.6 | 2.9 | 3652.1 |
| **2-year, 24-hour** | 43.5 | 7.0 | 3.2 | 3652.4 |
| **5-year, 6-hour** | 87.0 | 15.6 | 5.0 | 3654.6 |
| **5-year, 24-hour** | 78.3 | 15.5 | 5.0 | 3654.6 |
| **10-year, 6-hour** | 122.2 | 19.5 | 7.1 | 3656.7 |
| **10-year, 24-hour** | 110.6 | 19.5 | 7.1 | 3656.7 |
| **25-year, 6-hour** | 180.0 | 24.1 | 10.9 | 3659.8 |
| **25-year, 24-hour** | 162.0 | 24.2 | 11.0 | 3659.9 |
| **50-year, 6-hour** | 232.6 | 27.2 | 14.5 | 3662.3 |
| **50-year, 24-hour** | 209.1 | 27.3 | 14.8 | 3662.5 |
| **100-year, 6-hour** | 288.9 | 29.5 | 18.5 | 3664.9 |
| **100-year, 24-hour** | 255.8 | 29.8 | 18.9 | 3665.1 |
| **200-year*, 6-hour** | 346.4 | 31.3 | 22.8 | 3667.3 |
| **200-year*, 24-hour** | 313.9 | 31.6 | 24.1 | 3667.9 |
| **500-year*, 6-hour** | 443.4 | 45.1 | 29.9 | 3670.6 |
| **500-year*, 24-hour** | 374.6 | 33.1 | 29.7 | 3670.6 |
| **PMP, 6-hour** | 1967.9 | 1957.9 | 36.4 | 3673.0 |
| **PMP, 24-hour** | 870.2 | 869.0 | 33.2 | 3671.9 |

* extrapolated precipitation value

**Structure 4** was found to not be capable of the passing the 100-year event through the principal spillway without the use of the emergency spillway. For the 100-year event, the depth of flow through the emergency spillway is estimated at 0.2 feet. Flow is also expected through the emergency spillway for longer duration events, with the 50-year, 24-hour storm producing 0.1 foot flow depth on the emergency spillway. The 50-year, 6-hour storm is estimated to pass through the principal spillway without the use of the emergency spillway. The 6-hour probable maximum precipitation event is estimated to fill the reservoir to within 0.8 feet of the crest of the embankment, while passing a peak flow of 6190 cfs. Substantial attenuation will occur for the 2-year through 500-year events, with the reservoir peak outflow ranging from 5 to 36 percent of the peak inflow. A summary of the analysis results is provided in Table 14.

**Structure 5** was found to be capable of passing an event greater than the 500-year event through the principal spillway without the use of the emergency spillway. The 500-year, 6-hour event is estimated to fill the reservoir to within 2.6 feet of the emergency spillway crest, while passing a peak flow of 32.0 cfs. The 100-year, 6-hour event is estimated to fill the reservoir to within 5.1 feet of the emergency spillway crest, while passing a peak flow of 29.7 cfs. The 6-hour probable maximum precipitation event is estimated to fill the reservoir to within 3.2 feet of the crest of the embankment, while passing a peak flow of 1240 cfs. Substantial attenuation will occur for the 2-year through 500-year events, with the reservoir peak outflow ranging from 6 to 14 percent of the peak inflow. A summary of the analysis results is provided in Table 15.

**Table 14:** Hydrologic modeling results summary, structure 4. Emergency spillway crest elevation = 3592.8 feet.

| Storm Description | Peak Inflow (cfs) | Peak Outflow (cfs) | Peak Storage (acre-feet) | Peak Elevation (feet) |
|---|---|---|---|---|
| 2-year, 6-hour | 145.1 | 18.2 | 12.6 | 3584.9 |
| 2-year, 24-hour | 143.0 | 20.4 | 13.8 | 3585.2 |
| 5-year, 6-hour | 261.3 | 26.7 | 23.5 | 3587.1 |
| 5-year, 24-hour | 231.6 | 26.7 | 23.7 | 3587.2 |
| 10-year, 6-hour | 350.3 | 27.5 | 32.6 | 3588.6 |
| 10-year, 24-hour | 312.0 | 27.9 | 35.2 | 3589.0 |
| 25-year, 6-hour | 493.0 | 29.4 | 47.1 | 3590.6 |
| 25-year, 24-hour | 434.4 | 30.1 | 53.3 | 3591.3 |
| 50-year, 6-hour | 616.8 | 30.7 | 59.8 | 3592.0 |
| 50-year, 24-hour | 543.7 | 44.9 | 68.2 | 3592.9 |
| 100-year, 6-hour | 747.8 | 83.4 | 69.7 | 3593.0 |
| 100-year, 24-hour | 650.9 | 94.8 | 69.9 | 3593.0 |
| 200-year*, 6-hour | 881.0 | 168.9 | 71.1 | 3593.1 |
| 200-year*, 24-hour | 788.3 | 211.2 | 71.7 | 3593.2 |
| 500-year*, 6-hour | 1104.3 | 391.6 | 74.6 | 3593.5 |
| 500-year*, 24-hour | 926.9 | 324.3 | 73.5 | 3593.4 |
| PMP, 6-hour | 6393.0 | 6193.4 | 129.6 | 3598.0 |
| PMP, 24-hour | 2959.8 | 2947.9 | 102.7 | 3596.0 |

* extrapolated precipitation value

**Table 15:** Hydrologic modeling results summary, structure 5. Emergency spillway crest elevation = 3619.4 feet.

| Storm Description | Peak Inflow (cfs) | Peak Outflow (cfs) | Peak Storage (acre-feet) | Peak Elevation (feet) |
|---|---|---|---|---|
| 2-year, 6-hour | 80.0 | 4.7 | 4.1 | 3608.6 |
| 2-year, 24-hour | 72.7 | 4.7 | 4.4 | 3608.8 |
| 5-year, 6-hour | 124 | 8.4 | 6.5 | 3610.3 |
| 5-year, 24-hour | 102 | 8.2 | 6.4 | 3610.3 |
| 10-year, 6-hour | 154 | 14.8 | 7.6 | 3611.0 |
| 10-year, 24-hour | 129 | 14.6 | 7.6 | 3611.0 |
| 25-year, 6-hour | 201 | 24.5 | 9.6 | 3612.2 |
| 25-year, 24-hour | 166 | 24.0 | 9.2 | 3612.0 |
| 50-year, 6-hour | 241 | 27.3 | 11.7 | 3613.3 |
| 50-year, 24-hour | 199 | 26.7 | 11.2 | 3613.0 |
| 100-year, 6-hour | 281 | 29.7 | 13.9 | 3614.3 |
| 100-year, 24-hour | 230 | 29.5 | 13.3 | 3614.1 |
| 200-year*, 6-hour | 320 | 30.6 | 16.2 | 3615.3 |
| 200-year*, 24-hour | 268 | 30.6 | 16.1 | 3615.2 |
| 500-year*, 6-hour | 386 | 32.0 | 20.2 | 3616.8 |
| 500-year*, 24-hour | 307 | 31.6 | 19.1 | 3616.4 |
| PMP, 6-hour | 1269 | 1236 | 36.0 | 3621.5 |
| PMP, 24-hour | 443 | 442 | 31.7 | 3620.4 |

* extrapolated precipitation value

**Structure 6** was found to be capable of passing an event greater than the 500-year event through the principal spillway without the use of the emergency spillways. The 500-year, 24-hour event is estimated to fill the reservoir to within 0.7 feet of the emergency spillway crest, while passing a peak flow of 66.7 cfs. The 100-year, 24-hour event is estimated to fill the reservoir to within 5.0 feet of the emergency spillway crest, while passing a peak flow of 61.2 cfs. The 6-hour probable maximum precipitation event is estimated to fill the reservoir to within 1.5 feet of the crest of the embankment, while passing a peak flow of 7490 cfs. Substantial attenuation will occur for the 2-year through 500-year events, with the reservoir peak outflow ranging from 4 to 10 percent of the peak inflow. A summary of the analysis results is provided in Table 16.

**Table 16:** Hydrologic modeling results summary, structure 6. Emergency spillway crest elevation = 3620.4 feet.

| Storm Description | Peak Inflow (cfs) | Peak Outflow (cfs) | Peak Storage (acre-feet) | Peak Elevation (feet) |
|---|---|---|---|---|
| **2-year, 6-hour** | 224 | 14.1 | 22.6 | 3604.6 |
| **2-year, 24-hour** | 220 | 16.8 | 26.4 | 3605.2 |
| **5-year, 6-hour** | 414 | 37.6 | 38.8 | 3607.1 |
| **5-year, 24-hour** | 367 | 37.7 | 8.9 | 3607.1 |
| **10-year, 6-hour** | 560 | 47.4 | 51.8 | 3608.7 |
| **10-year, 24-hour** | 499 | 47.9 | 52.6 | 3608.8 |
| **25-year, 6-hour** | 789 | 55.5 | 47.4 | 3611.2 |
| **25-year, 24-hour** | 705 | 55.7 | 76.0 | 3611.3 |
| **50-year, 6-hour** | 1007 | 58.2 | 95.7 | 3613.1 |
| **50-year, 24-hour** | 892 | 58.5 | 98.1 | 3613.3 |
| **100-year, 6-hour** | 1228 | 60.7 | 118.3 | 3615.0 |
| **100-year, 24-hour** | 1077 | 61.2 | 123.1 | 3615.4 |
| **200-year*, 6-hour** | 1456 | 63.1 | 141.9 | 3616.8 |
| **200-year*, 24-hour** | 1303 | 64.1 | 153.5 | 3617.6 |
| **500-year*, 6-hour** | 1833 | 66.4 | 181.1 | 3619.5 |
| **500-year*, 24-hour** | 1535 | 66.7 | 185.3 | 3619.7 |
| **PMP, 6-hour** | 7792 | 7492 | 286.8 | 3625.1 |
| **PMP, 24-hour** | 3884 | 3852 | 254.4 | 3623.6 |

* extrapolated precipitation value

The peak flows for the hydrologic modeling for the other, downstream watersheds are provided in Table 17. Results are provided for the 2-year through 500-year events, plus the PMP. Hydrographs for all hydrologic modeling, with 2-minute time steps, can be provided upon request to the author.

**Table 17:** Hydrologic modeling results summary for 6-hour storms, other watersheds.

| ID | Peak Flow (cfs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 2-yr | 5-yr | 10-yr | 25-yr | 50-yr | 100-yr | 200-yr | 500-yr | PMP |
| a | 18.7 | 32.7 | 42.9 | 59.0 | 73.1 | 87.7 | 102 | 126 | 474 |
| b | 27.2 | 45.2 | 57.7 | 77.3 | 94.3 | 112 | 128 | 157 | 553 |
| c | 19.5 | 30.7 | 38.2 | 49.6 | 59.6 | 69.6 | 79.3 | 95.8 | 331 |
| d | 21.2 | 31.7 | 38.6 | 49.3 | 58.4 | 67.6 | 76.3 | 91.1 | 301 |
| e | 24.6 | 38.3 | 47.5 | 61.5 | 73.6 | 85.7 | 97.3 | 117 | 393 |
| f | 22.5 | 34.9 | 43.1 | 55.6 | 66.5 | 77.4 | 87.7 | 105 | 359 |
| g | 28.1 | 42.2 | 51.6 | 65.8 | 77.9 | 90.1 | 102 | 121 | 391 |
| h | 4.3 | 6.5 | 7.9 | 10.0 | 11.8 | 13.7 | 15.4 | 18.3 | 60.7 |
| i | 15.7 | 24.0 | 29.5 | 38.0 | 45.3 | 52.8 | 59.7 | 71.7 | 242 |
| j | 16.5 | 25.0 | 30.5 | 39.0 | 46.3 | 53.7 | 60.7 | 72.7 | 241 |
| k | 24.8 | 37.4 | 45.8 | 58.5 | 69.3 | 80.2 | 90.9 | 109 | 348 |
| l | 7.0 | 10.6 | 13.0 | 16.7 | 19.8 | 23.0 | 26.1 | 31.2 | 101 |
| m | 15.3 | 27.3 | 35.9 | 49.3 | 61.2 | 73.7 | 85.8 | 107 | 409 |
| n | 14.4 | 23.9 | 30.4 | 40.6 | 49.7 | 58.8 | 67.7 | 82.7 | 296 |
| o | 53.3 | 84.5 | 106 | 138 | 166 | 194 | 222 | 268 | 902 |
| p | 6.9 | 10.3 | 12.5 | 15.8 | 18.6 | 21.5 | 24.2 | 28.8 | 94.2 |
| q | 42.6 | 65.6 | 81.3 | 105 | 125 | 146 | 166 | 200 | 646 |

It is interesting to note the hydrologic modeling indicates just how much higher peak flow is expected out of the much smaller, uncontrolled watersheds than the much larger, controlled watersheds. Figure 16 provides an example of this for structure 1 and two smaller watersheds (*a* and *b*) that discharge flow just below the structure. Watershed 1 has an area of 303 acres, while watersheds *a* and *b* have areas of 29 and 35 acres, respectively. The uncontrolled flow above structure 1 is greatly attenuated so that its peak flow, through the 500-year event, is substantially less than the peak flows predicted from watersheds a and b, even though these watersheds are both about 1/10$^{th}$ the size. Structure 1 is highly effective at attenuating flood flow, as are the other structures. Only structure 4 will pass significant peak flows for events less than the 500-year event.

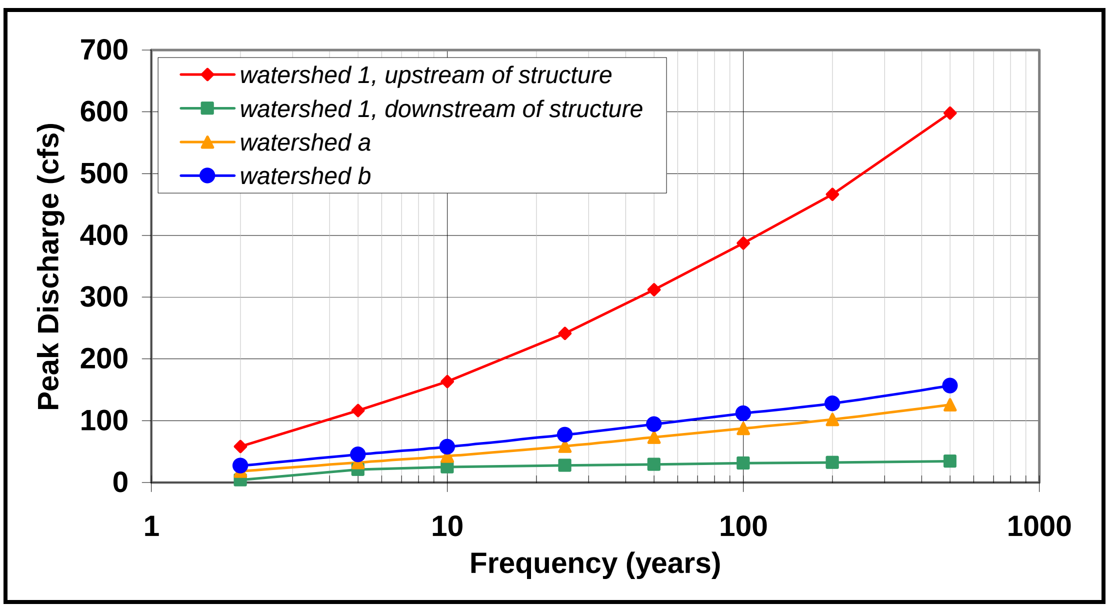



**Figure 16:** Flow-frequency comparison, structure *1* and watersheds *a* and *b*.

## HYDRAULIC MODELING

Hydraulic modeling was performed using HEC-RAS 3.1.3, in combination with ArcGIS 9.0 and HEC-GeoRAS 4.1. Six models were developed, one for each of the six structures. Modeling was performed for the 10-, 25-, 50- and 100-year events. For each structure, hydrographs are presented for the bottom limit of each analysis. Additionally, an inundation map for the 100-year event was also produced. Results are provided in the HYDRAULIC MODELING RESULTS section. Peak flow characteristics at selected cross sections are provided in Appendix A. These sections are illustrated with the inundation mapping.

Cross-section development was performed using the LiDAR data and the aerial imagery. Sections were developed with a spacing of about 60 to 100 feet. Since these models are relatively-steeply sloped, additional sections were needed to produce a stable model. Interpolated sections were added, with spacing from 20 to 50 feet.

The 1-meter LiDAR data, provided by Spectrum Mapping, was the source of all elevation data. Vertical accuracy was reported to have a RMSE (Root Mean Square Error) of 0.1051 m (Spectrum Mapping 2006).

Manning's *n* selection for the hydraulic modeling was performed using standard visual inspection techniques. Photographs were taken in the field and, with the assistance of the 0.5 foot resolution aerial imagery, *n* values were assigned for the stream and flood plain using GIS and HEC-RAS's horizontal variation in *n* option. The guides provide in Chow (1959), Arcement & Schneider (1989), and Brunner & Goodell (2002) were implemented to assign the *n* values. The Manning's *n* values used in this analysis are: 0.05 for channels with vegetation; 0.05 for floodplains with grass/brush; 0.08 for brush; 0.10 for dense brush; 0.12 for dense trees; 0.015 for paved surfaces and 0.08 for residential areas with trees and brush.

Downstream boundary conditions were assumed to be normal depth, computed from adjacent 2-foot contours along the drainage paths.

### Computation Methodology

To support the basis of the modeling used in this dam breach analysis and to discourage a "black box" mentality, the basic equations used in these computations are briefly presented.

The physical laws that govern unsteady flow modeling, as presented in the HEC-RAS Hydraulic Reference Manual (Brunner and Goodwell, 2002), are conservation of mass (the continuity equation) and conservation of momentum. The general continuity equation (not separately written for both the channel and floodplain) is:

$$\frac{\partial A}{\partial t} + \frac{\partial S}{\partial t} + \frac{\partial Q}{\partial x} - q_1 = 0$$

Where: $\partial$ = partial differential.
A = cross-sectional area.
t = time.
S = storage from non conveying portions of cross section.
Q = flow.

x = distance along the channel.
$q_l$ = lateral inflow per unit distance.

The momentum equation can be stated as "the net rate of momentum entering the volume (momentum flux) plus the sum of all external forces acting on the volume be equal to the rate of accumulation of momentum" (Brunner and Goodwell, 2002). In differential form, it is:

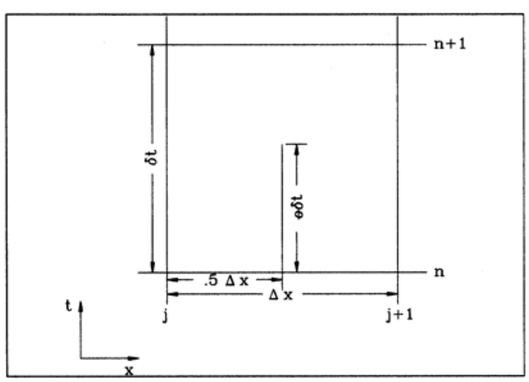
$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0$$

$$S_f = \frac{Q|Q|n^2}{2.208R^{4/3}A^2}$$

Where: V = velocity
g = acceleration due to gravity.
$\frac{\partial z}{\partial x}$ = water surface slope.
$S_f$ = friction slope.
n = Manning's roughness estimate.
R = hydraulic radius = area/wetted perimeter.

The most successful and accepted procedure for approximating solutions to the non-linear unsteady flow equations is with a four-point implicit solution scheme, also known as a box scheme (Brunner and Goodwell, 2002). The HEC-RAS Hydraulic Reference Manual describes this as follows:

> *Under this scheme, space derivatives and function values are evaluated at an interior point, $(n+\theta)\Delta t$. Thus values at $(n+1)\Delta t$ enter into all terms in the equations. For a reach of a river, a system of simultaneous equations results. The simultaneous solution is an important aspect of this scheme because it allows information from the entire reach to influence the solution at any one point Consequently, the time step can be significantly larger than with explicit numerical schemes.*




[Typical finite difference cell used in HEC-RAS computations (from Brunner and Goodwell, 2002).]

The general implicit finite difference forms are as follows:

The time derivative is approximated as: $\frac{\partial f}{\partial t} \approx \frac{\Delta f}{\Delta t} = \frac{0.5(\Delta f_{j+1} + \Delta f_j)}{\Delta t}$

The space derivative is approximated as: $\frac{\partial f}{\partial x} \approx \frac{\Delta f}{\Delta x} = \frac{(f_{j+1} - f_j) + \theta(\Delta f_{j+1} - \Delta f_j)}{\Delta x}$

The function value is: $f \approx \overline{f} = 0.5(f_j + f_{j+1}) + 0.5\theta(\Delta f_j + \Delta f_{j+1})$

Where: $\Delta$ = difference or change in.

Using this methodology, the finite difference form of the continuity equation used by HEC-RAS (which separates channel and floodplain flow) is:

$$\Delta Q + \frac{\Delta A_c}{\Delta t}\Delta x_c + \frac{\Delta A_f}{\Delta t}\Delta x_f + \frac{\Delta S}{\Delta t}\Delta x_f - \overline{Q_l} = 0$$

Where: c = channel.
f = floodplain.
$\overline{Q_l}$ = average lateral inflow.

Assuming a horizontal water surface across the cross section and perpendicular flow to the plane of the cross section, the finite difference form of the momentum equation is:

$$\frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t \Delta x_e} + \frac{\Delta(\beta VQ)}{\Delta x_e} + g\overline{A}\left(\frac{\Delta z}{\Delta x_e} + \overline{S_f} + \overline{S_h}\right) = \xi \frac{Q_l V_l}{\Delta x_e}$$

Where: $\Delta x_e$ = equivalent flow path
$\Delta(\beta VQ) = \Delta(V_c Q_c) + \Delta(V_f Q_f)$
$S_f$ = frictional slope for the entire cross section.
$S_h$ = local frictional slope, from bridge piers, navigation dams, cofferdams, etc.
$Q_l$ = lateral inflow.
$V_l$ = average velocity of lateral inflow.
$\xi$ = fraction of momentum entering a receiving stream.

If the implicit finite difference solution scheme is applied directly to these non-linear equations, a series of non-linear algebraic equations result. To avoid the resulting slow and unstable iteration solution schemes, these equations are linearized for their use in HEC-RAS (Brunner and Goodwell, 2002).

For a more comprehensive presentation of the solution equations and techniques used in HEC-RAS, please see the HEC-RAS Hydraulic Reference Manual.

## HYDRAULIC MODELING RESULTS

Hydraulic modeling was performed to route the storm flows to the entrance of Wray's stormwater system. The modeling was terminated at this point due to the lack of available information for the culverts and due to stormwater system modeling being outside the scope of this project.

Hydrographs at the culvert inlets are provided, with electronic files of these hydrographs available upon request to the report's author. For each hydraulic model segment, an inundation map for the 100-year event is also provided.

Rough capacities and associated return periods for the culvert entrances are also provided. These values need to be considered approximate, due to the possibility that unknown downstream stormsewer culvert geometry could provide the flow control instead of the entrance. Additionally, culvert inverts and overtopping elevations were approximated using the LiDAR data.

### Structure 1

Downstream of structure 1 there is approximately 1200 feet of surface flow before the effluent from the structure is drained into a 6-foot diameter stormwater culvert (Figure 17). At the exit this 6-foot culvert is narrowed to a 2.5 foot diameter culvert (Figure 18), substantially deceasing the capacity of the structure and increasing the frequency of overtopping and flooding of US-34. Two relatively substantial uncontrolled watersheds drain into this drainage between structure 1 and US-34, increasing the needed conveyance capacity under the highway.

**Figure 17:** US-34 culvert, 6' inlet.

**Figure18:** US-34 culvert, 2.5' outlet.

The 10-, 25-, 50- and 100-year flows have been routed through the US-34 culvert from structure 1 and the intervening watershed. The hydraulic model used 26 sections. Peak flows from all storm events can be found in Table 11 for the structure and Table 17 for the downstream drainage area. Hydrographs just below the culvert are provided in Figure 20. The approximate inundation extent for the 100-year event for existing conditions is provided in Figure 19. At selected cross sections, approximate peak flow characteristics for the 10-, 25-, 50- and 100-year storms are provided in Table A-1.

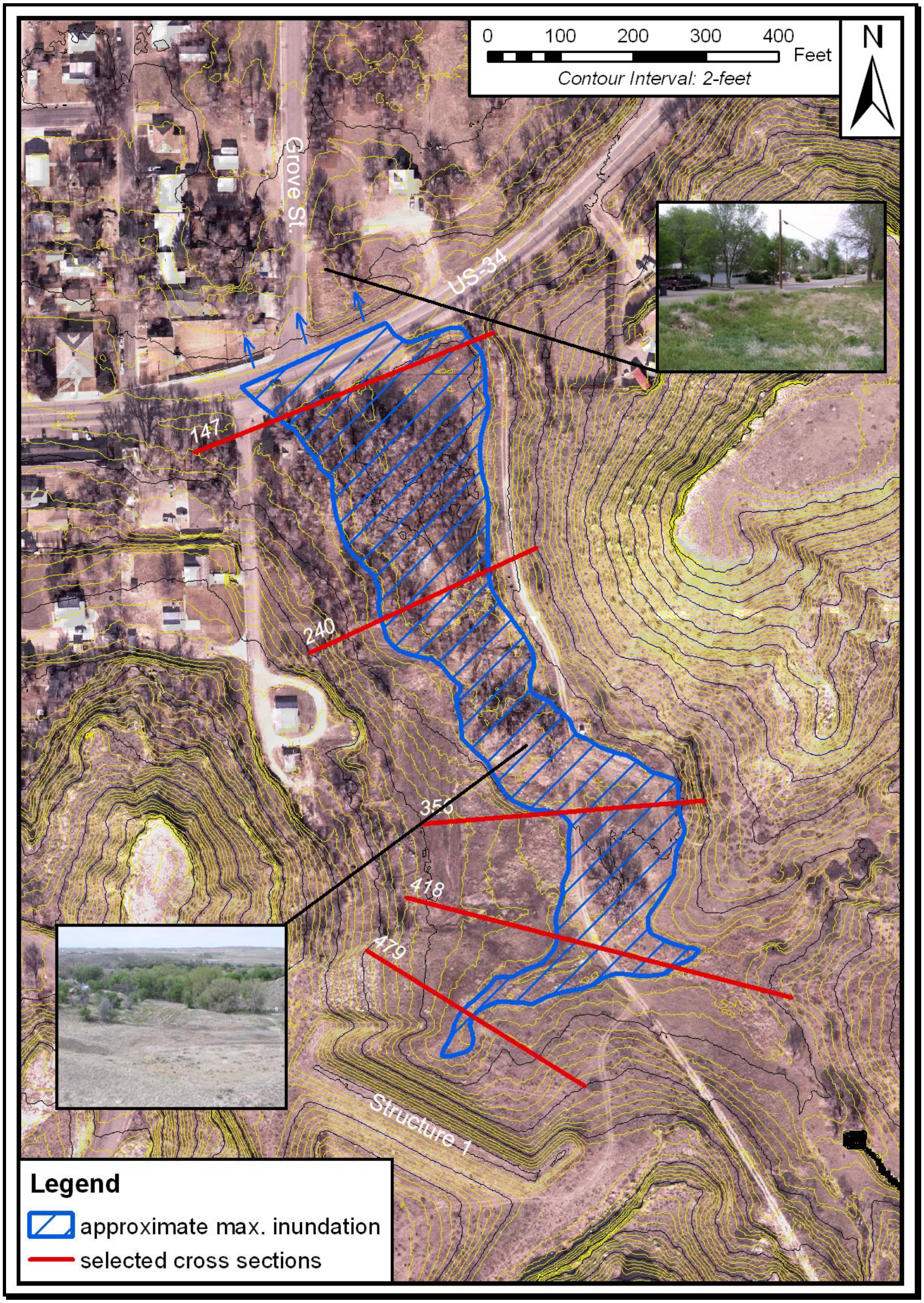



**Figure 19:** Approximate inundation extents for the 100-year peak flow event, from structure 1 to US-34.

The 2.5’ diameter pipe that has been added onto the end of the 6-foot diameter culvert, substantially decreases the flow capacity of this culvert. Using a simple entrance control approximation, the existing 2.5-foot diameter culvert has a capacity of approximately 64 cfs before the highway embankment is overtopped. The 6-foot diameter culvert, if it was not constrained by the 2.5-foot culvert section, would pass a flow of approximately 320 cfs before US-34 would be overtopped.

The 10-year storm has been modeled to pass through this restriction without overtopping the embankment, with attenuation that has been simulated to occur just upstream of the US-34 embankment. During the 25-year storm, approximately 40 cfs of flow will pass over the roadway and through the properties immediately adjacent to the roadway. During the 50- and 100-year storms, approximately 110 cfs and 150 cfs, respectively, will pass over US-34. The restriction decreases the culvert capacity so that the flow will now overtop the US-34 between once every 10 to 25 years. Without this restriction, the 6’ culvert under US-34 will pass the 100-year event.

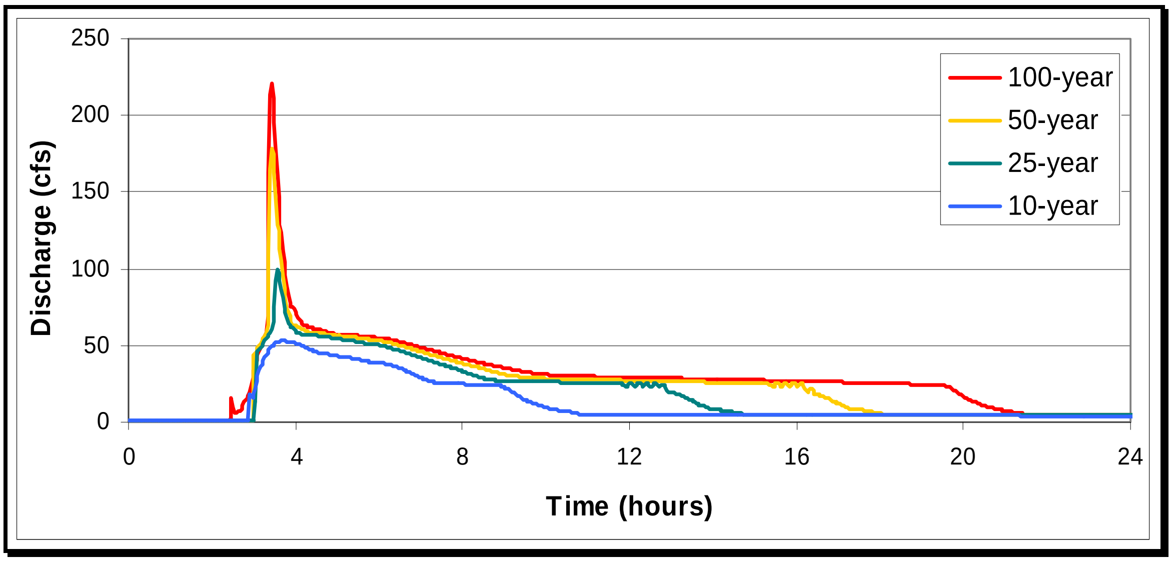



**Figure 20:** Hydrographs from 6-hour storms, just downstream of the US-34 culvert downstream of structure 1.

### Structure 2

Downstream of structure 2 there is approximately 570 feet of surface flow before the effluent from the structure is drained into a 3-foot diameter stormwater culvert (Figure 22). The hydraulic model used 9 sections to model this reach. The 10-, 25-, 50- and 100-year flows have been routed to the stormwater culvert from structure 2 and the intervening watershed. The peak flows from all storm events can be found in Table 12 for the structure and Table 17 for the downstream drainage area. Hydrographs at the culvert entrance are provided in Figure 21. The approximate inundation extent for the 100-year event is provided in Figure 22. At selected cross sections, approximate peak flow characteristics for the 10-, 25-, 50- and 100-year storms are provided in Table A-2.

Using a simple entrance control computation, the 3-foot diameter culvert was computed to have an approximate flow capacity of 47 cfs. As shown below, the 10-year event has an approximate peak flow of 45 cfs at this culvert. The 10-year peak for flow exiting structure 2 is 12 cfs, with the downstream drainage contributing a peak of 38 cfs slightly earlier. The culvert has the potential to have its capacity exceeded approximately once every 10 years, on average, with most of this flow peak being yielded from the steep or urbanized uncontrolled drainage between structure 2 and the stormwater culvert.

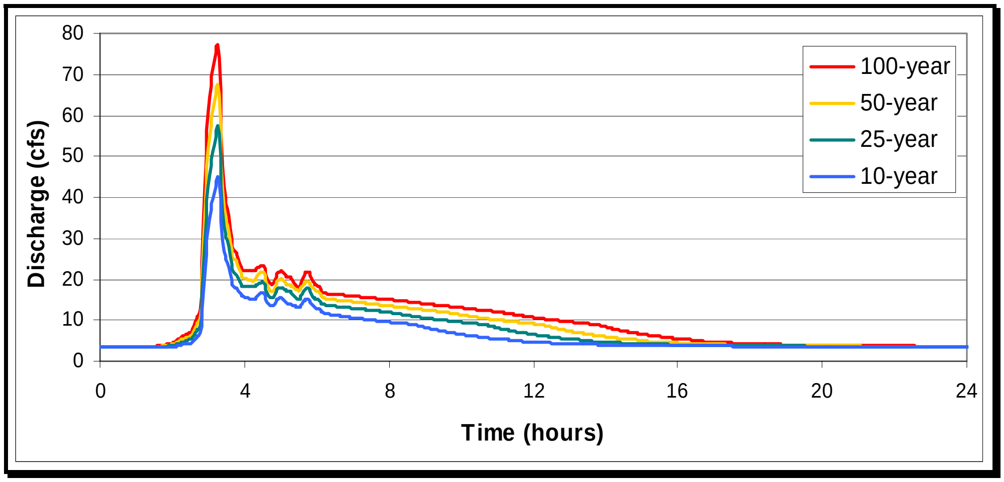



**Figure 21:** Hydrographs from 6-hour storms, at first culvert entrance downstream of structure 2.

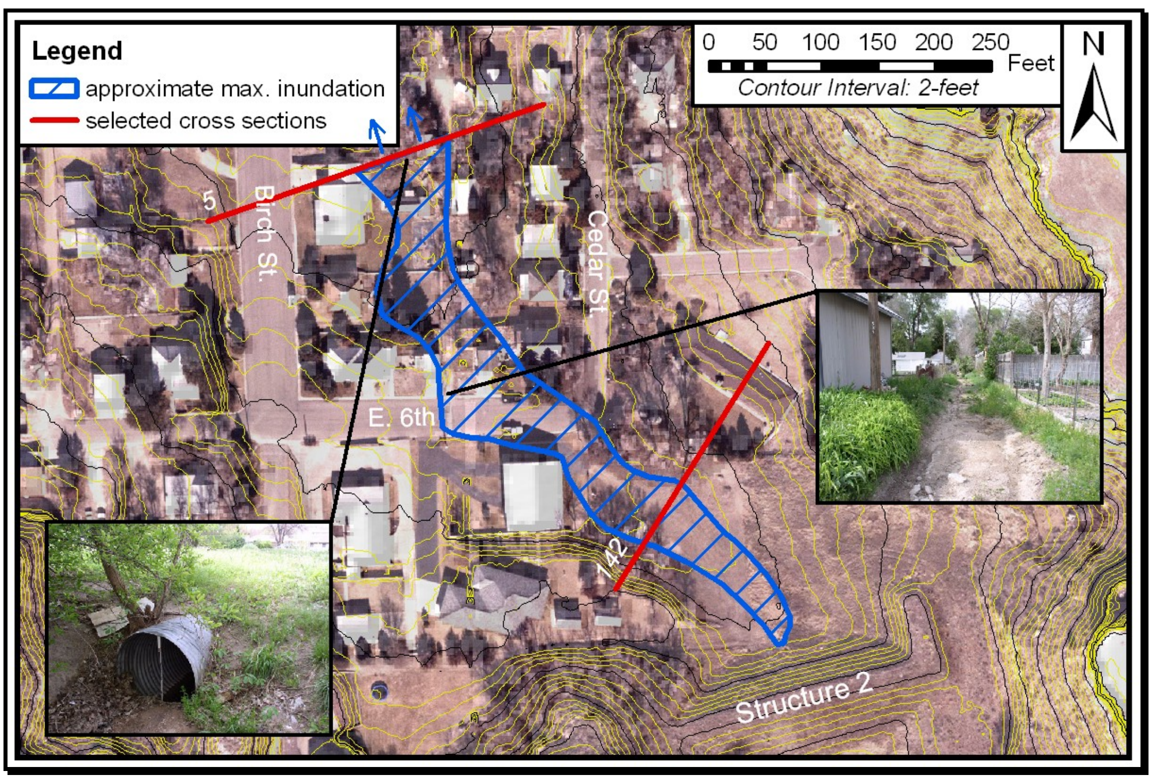



**Figure 22:** Approximate inundation extents for the 100-year peak flow event, from structure 2 to the storm water culvert entrance.

**Structure 3**

Structures 3 and 4 are two dams in series. Downstream of structure 3 there is approximately 4300 feet of surface flow before the inlet to structure 4. The hydraulic model used 29 sections to model this reach. The 10-, 25-, 50- and 100-year flows modeled to flow out of structure 3, plus the flows modeled to be yielded from the contributing watershed between structures 3 and 4, have been routed to structure 4. The peak flows from all storm events can be found in Table 13 for structure 3 and Table 17 for the intervening drainage area. Hydrographs at the culvert entrance are provided in Figure 23. The approximate inundation extent for the 100-year event is provided in Figures 24 and 25. At selected cross sections, approximate peak flow characteristics for the 10-, 25-, 50- and 100-year storms are provided in Table A-3.

The peak flow modeled to exit structure 3 for the 100-year, 6-hour event is 29.5 cfs. This flow is almost insignificant compared to the flow modeled to be produced from the intervening watershed – a peak of 750cfs is expected from this watershed for the 100-year event.

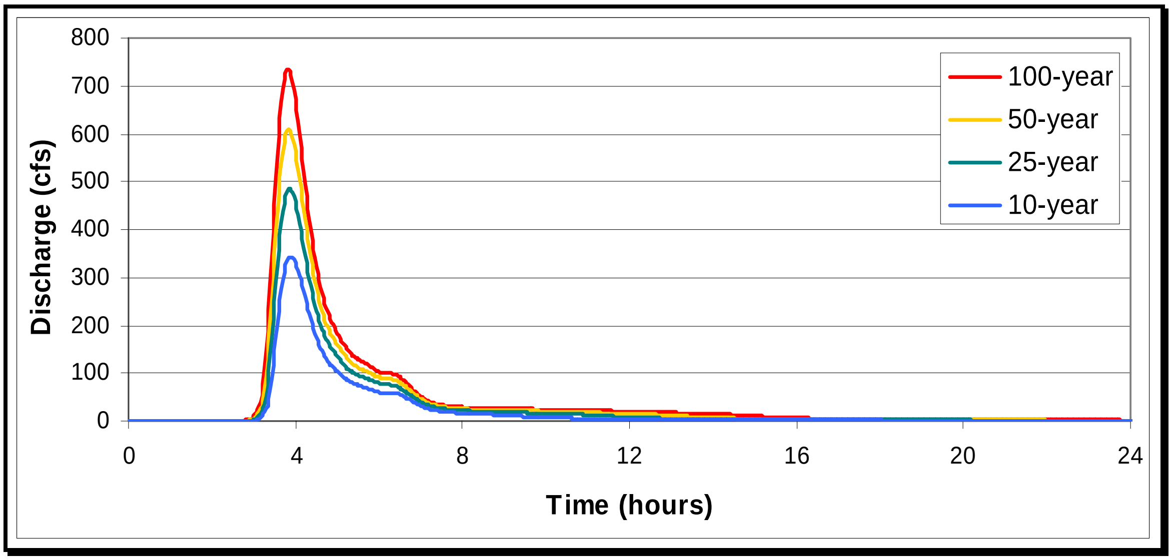



**Figure 23:** Hydrographs from 6-hour storms, downstream of structure 3 just above structure 4.

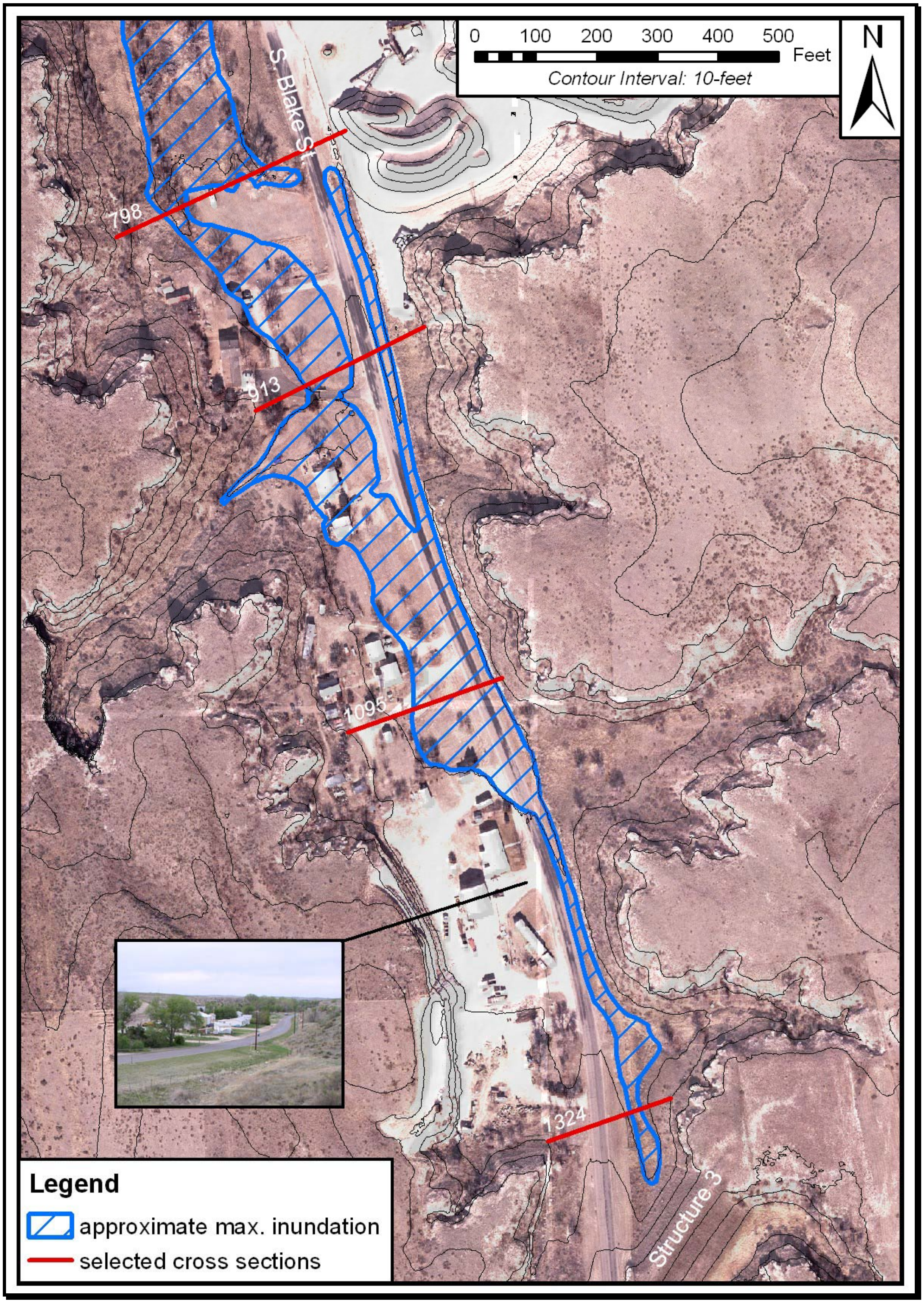



**Figure 24:** Approximate southern (upper) inundation extents for the 100-year peak flow event, from structure 3 to structure 4.

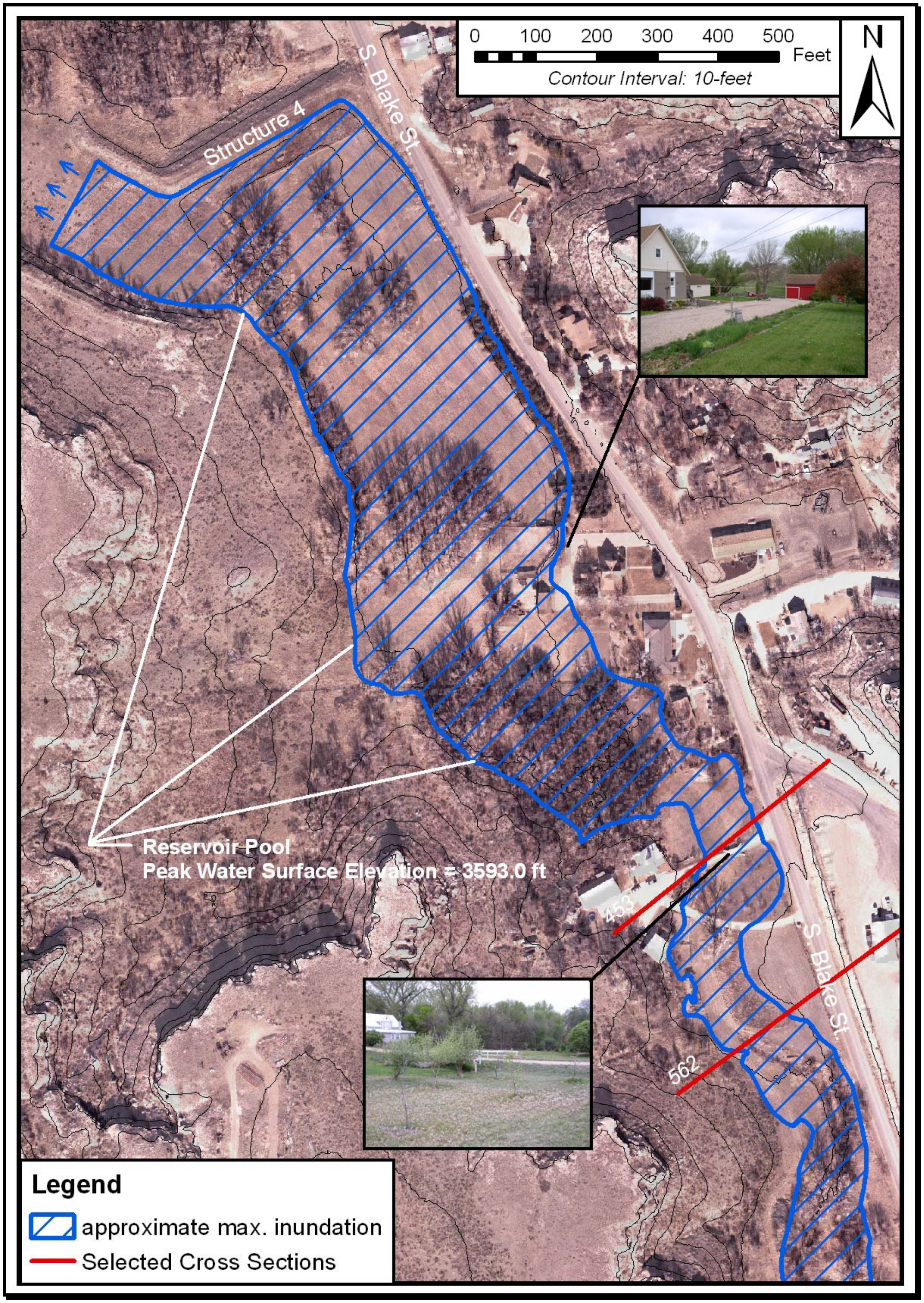



**Figure 25:** Approximate northern (lower) inundation extents for the 100-year peak flow event, from structure 3 to structure 4.

## Structure 4

Downstream of structure 4 there is approximately 1050 feet of surface flow before the effluent from the structure is drained into a 4.6 foot by 8.5 foot box culvert (Figure 27). The hydraulic model used 14 sections to model this reach. The 10-, 25-, 50- and 100-year flows have been routed to the stormwater culvert from structure 4 and the intervening watersheds. The peak flows from all storm events can be found in Table 14 for the structure and Table 17 for the downstream drainage area. Hydrographs at the culvert entrance are provided in Figure 26. The approximate inundation extent for the 100-year event is provided in Figure 27. At selected cross sections, approximate peak flow characteristics for the 10-, 25-, 50- and 100-year storms are provided in Table A-4.

The hydrographs of Figure 26 show the hydrologic responses to the storm events from both the structure and the downstream drainage area, to the culvert entrance. The first 100-year peak (in red) is the estimated flow yielded from the watershed downstream of the structure. The second peak is flow from the emergency spillway of structure 4, which will flow for events greater than or equal to the 50-year storm.

Using a simple entrance control computation, the box culvert was computed to have an approximate flow capacity of about 340 cfs. This capacity is assuming that there are no restrictions downstream that could potentially cause decreased flow capacities through this box culvert. Additionally, this capacity estimate also assumes that debris will not be lodged in the culvert entrance, potentially decreasing flow capacities.

As shown below, the 100-year event produces an approximate peak flow of 170 cfs at the culvert entrance. The second peak of 83 cfs represents the small amount of emergency spillway flow that is predicted to occur during the 100-year event. It appears that the box culvert that takes the drainage from structure 4 has a capacity that exceeds the 100-year storm.

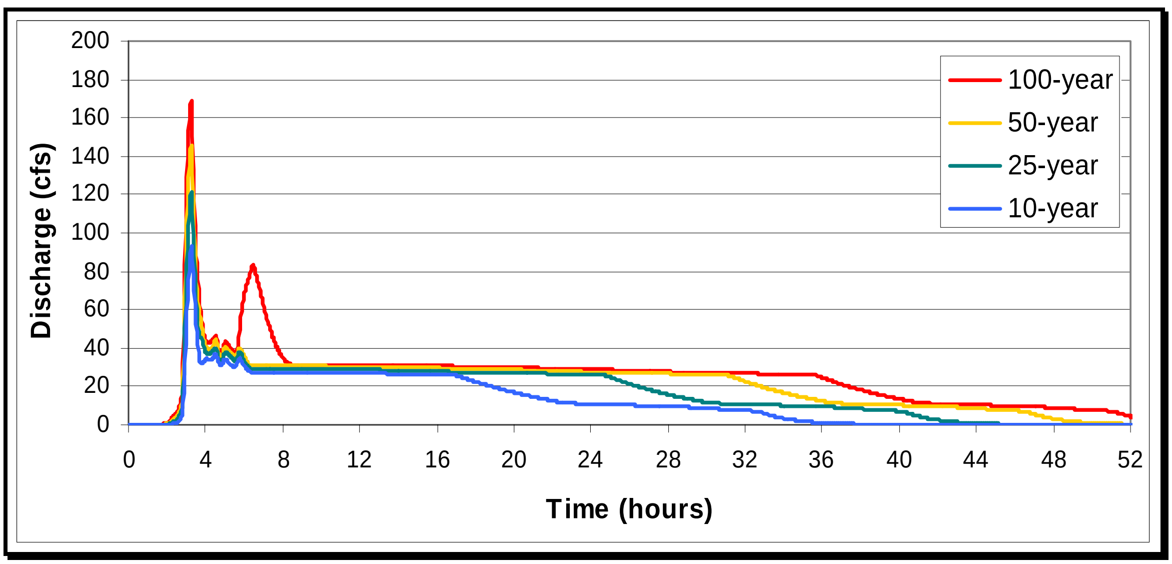



**Figure 26:** Hydrographs from 6-hour storms, at first culvert entrance downstream of structure 4.

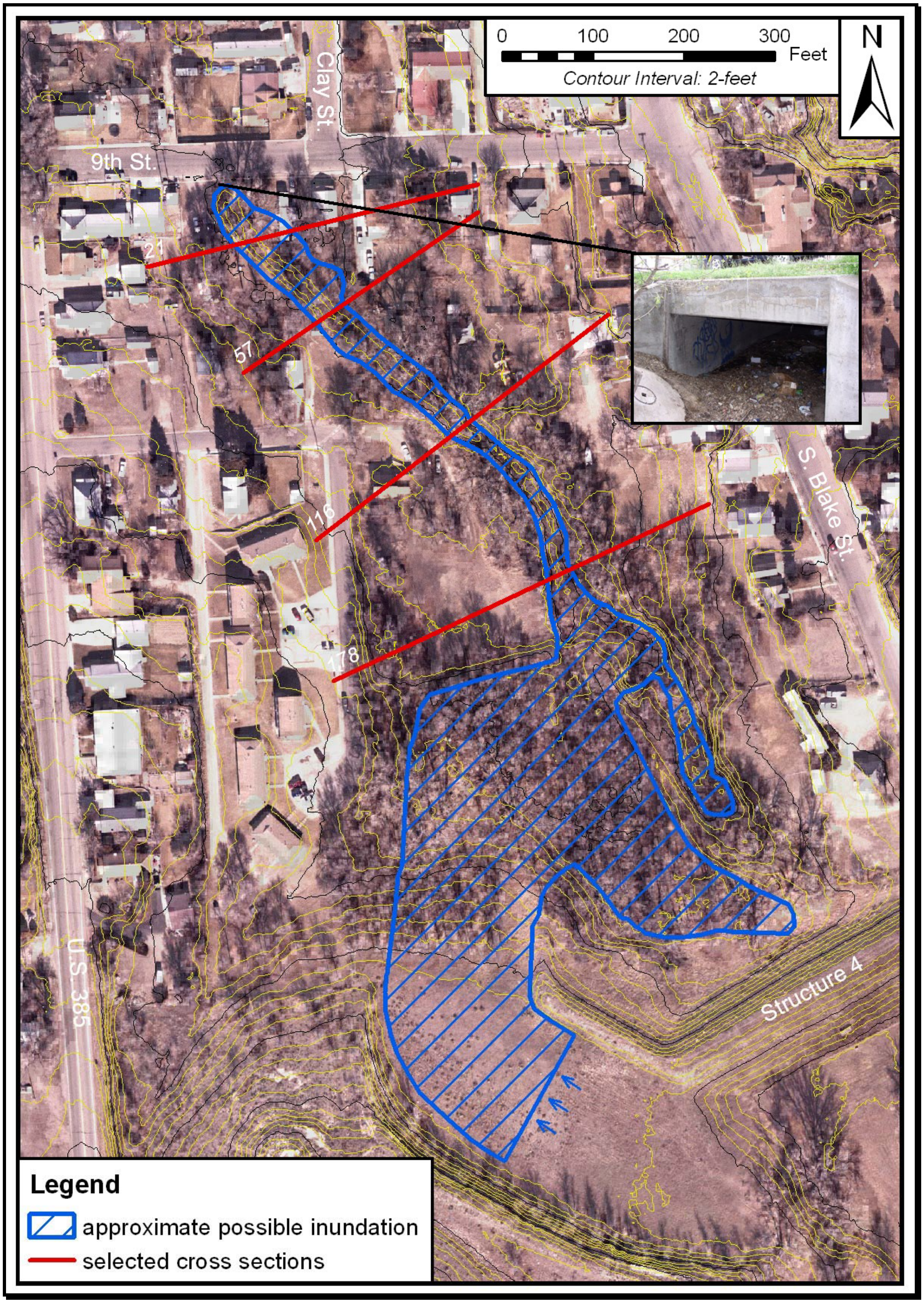



**Figure 27:** Approximate inundation extents for the 100-year peak flow event, from structure 4 to the first culvert.

### Structure 5

Downstream of structure 5 there is approximately 1440 feet of surface flow, with one 3-foot roadway culvert, before the effluent from the structure is drained into a 3-foot diameter stormwater culvert at W. 10th Street (Figure 29). The hydraulic model used 17 sections to model this reach. The 10-, 25-, 50- and 100-year flows have been routed to this stormwater culvert from structure 5 and the intervening watershed. The peak flows from all storm events can be found in Table 15 for the structure and Table 17 for the downstream drainage area. Hydrographs at the culvert entrance are provided in Figure 28. The approximate inundation extent for the 100-year event is provided in Figure 29. At selected cross sections, approximate peak flow characteristics for the 10-, 25-, 50- and 100-year storms are provided in Table A-5.

The culvert under W. 11th Street has been modeled to pass all storm events up to the 100-year storm. Limited attenuation has been modeled to occur just upstream of this culvert.

Using a simple entrance control computation, the 3-foot diameter culvert was computed to have an approximate flow capacity of 57 cfs. As shown below, the 10-year event has an approximate peak flow of 59 cfs just above this culvert. The 100-year storm has an estimated peak flow at this point of 100 cfs. Despite the low peak flow simulated to exit structure 5 (Table 15), substantial flow is estimated to flow off of the watershed immediately above 10th street, which causes this peak to exceed the flow capacity of this stormwater inlet. The culvert has the potential to have its capacity exceeded approximately once every 10 years, on average, with most of this flow peak being yielded from the uncontrolled drainage between structure 5 and 10th street.

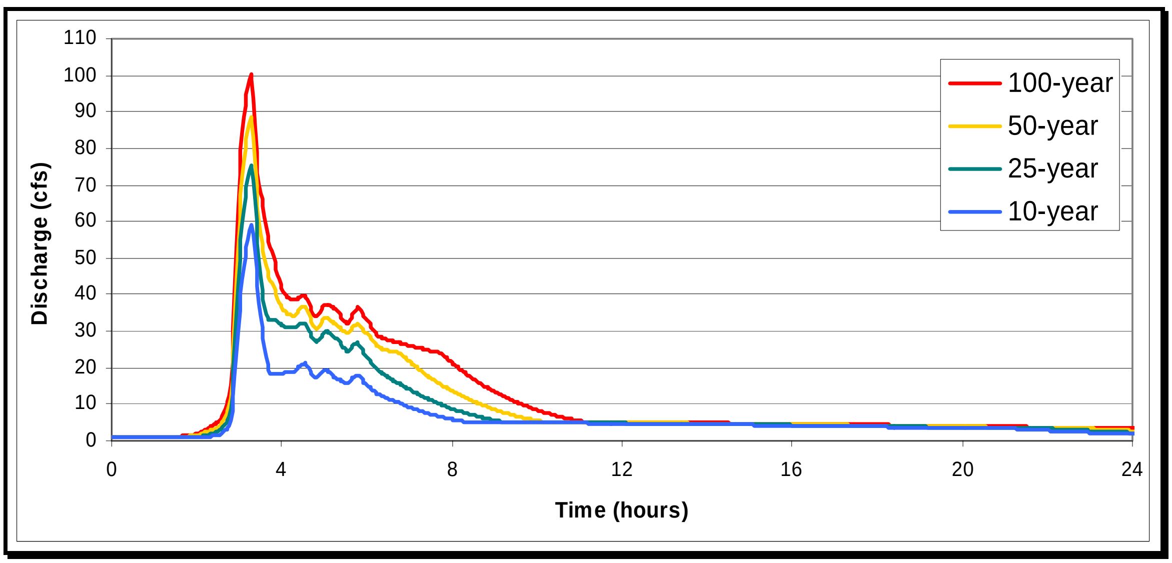



**Figure 28:** Hydrographs from 6-hour storms, at entrance to stormwater culvert at W. 10th Street downstream of structure 5.

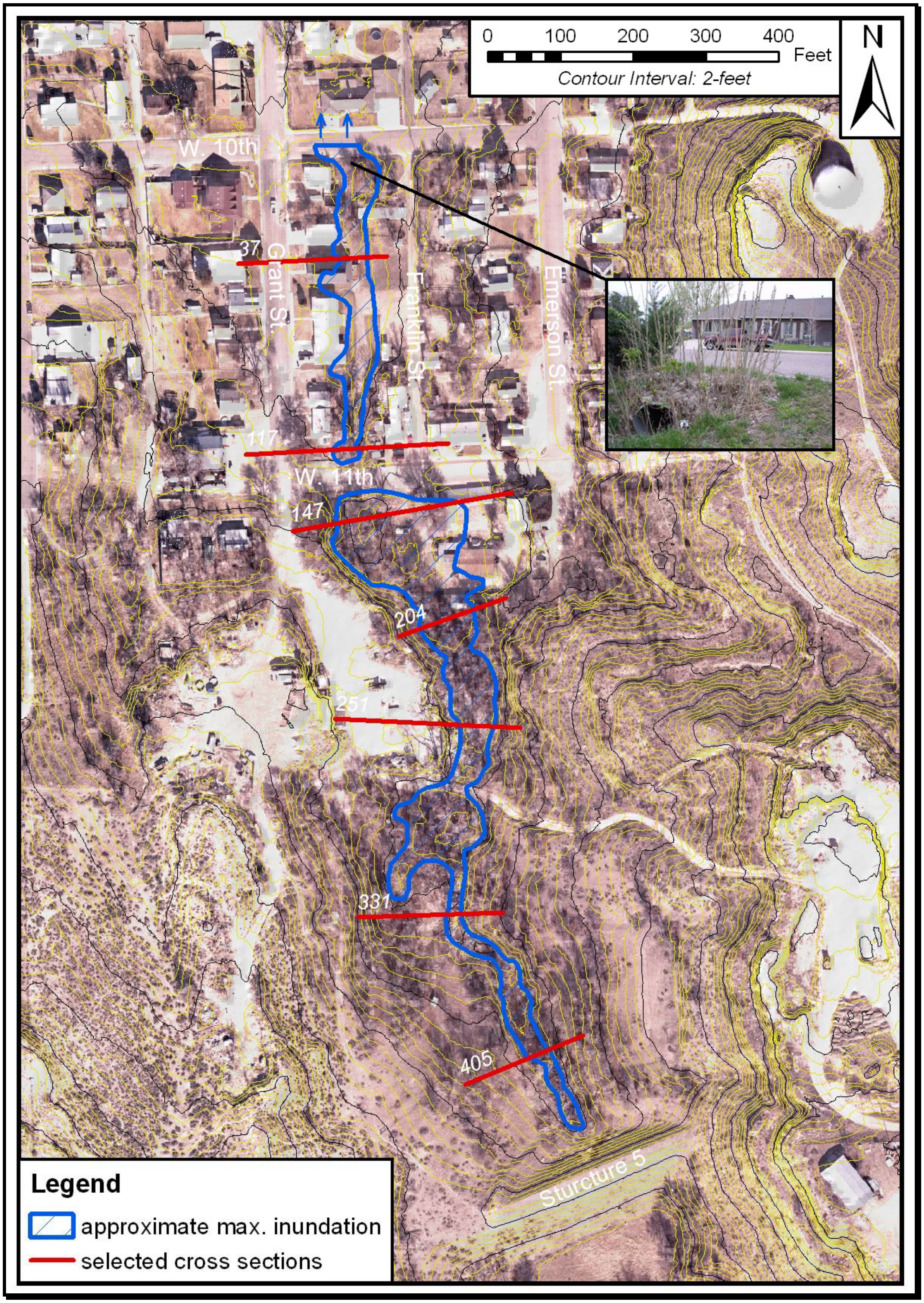



**Figure 29:** Approximate inundation extents for the 100-year peak flow event, from structure 5 to stormwater culvert entrance at 10th Street.

**Structure 6**

Downstream of structure 6 there is approximately 3400 feet of surface flow, with substantial additional uncontrolled urbanized drainage area as well as the drainage from structure 5, before the effluent from structure 6 is drained into a 6-foot diameter stormwater culvert just after a much larger bridge opening under W. 7th Street (Figure 30). The culvert replaced an open channel drainage that was immediately adjacent to a hospital (Figure 31). The hydraulic model used 23 sections to model this reach. The 10-, 25-, 50- and 100-year flows have been routed to this stormwater culvert from structure 6 and the additional watersheds. The peak flows from all storm events can be found in Table 16 for the structure and Table 17 for the downstream drainage areas. Hydrographs at the culvert entrance are provided in Figure 32. Approximate inundation extents for the 100-year event is provided in Figures 33 and 34. At selected cross sections, approximate peak flow characteristics for the 10-, 25-, 50- and 100-year storms are provided in Table A-6.

**Figure 30:** Culvert downstream of 7th St.

**Figure 31:** Drainage adjacent to hospital.

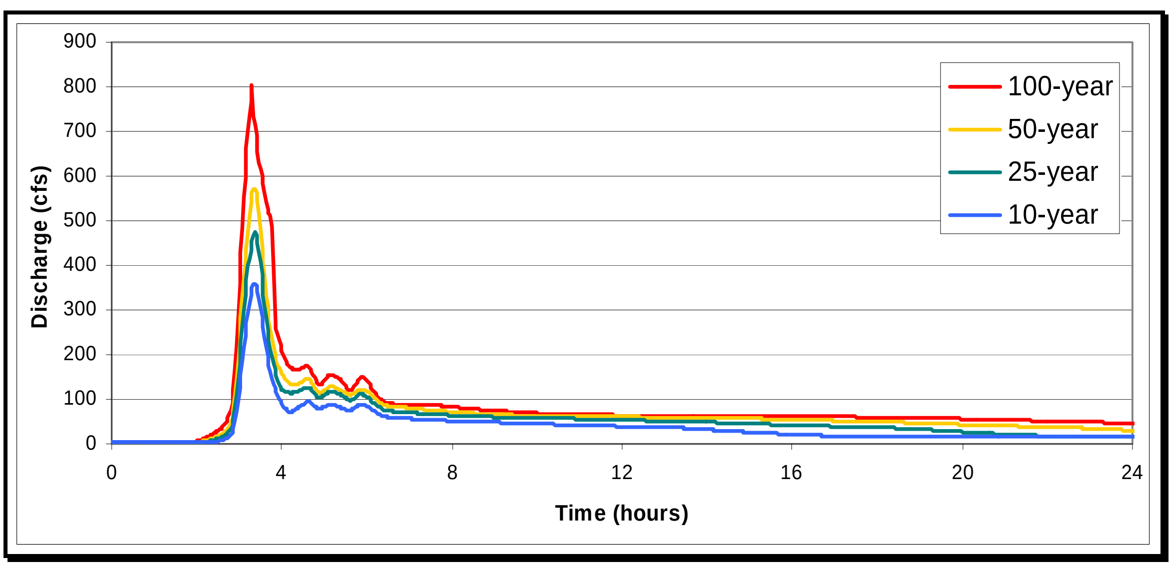



**Figure 32:** Hydrographs from 6-hour storms, downstream of structure 6 at 7th street.

**Figure 33:** Channel upstream of 7th St. **Figure 34:** Drainage at 5th St.

Using a simple entrance control computation, the 6-foot diameter culvert was computed to have an approximate flow capacity of 320 cfs. This culvert just downstream of the bridge under W. 7th Street has been analyzed to have the capacity to convey close to the peak flow modeled to be produced from a 10-year storm from this relatively substantial drainage area. The channel upstream of W. 7th street (Figure 33) has been modeled to have the capacity to convey storms in excess of the 100-year event. If this channel still existed downstream of the 7th street, and if the downstream street crossing at 5th Street (Figure 34) had sufficient conveyance capacity, much more substantial flows could be transported past the hospital without flooding. The installed culvert immediately adjacent to the hospital has greatly reduced the flow capacity of this drainage and increased the risk of flooding to the hospital.

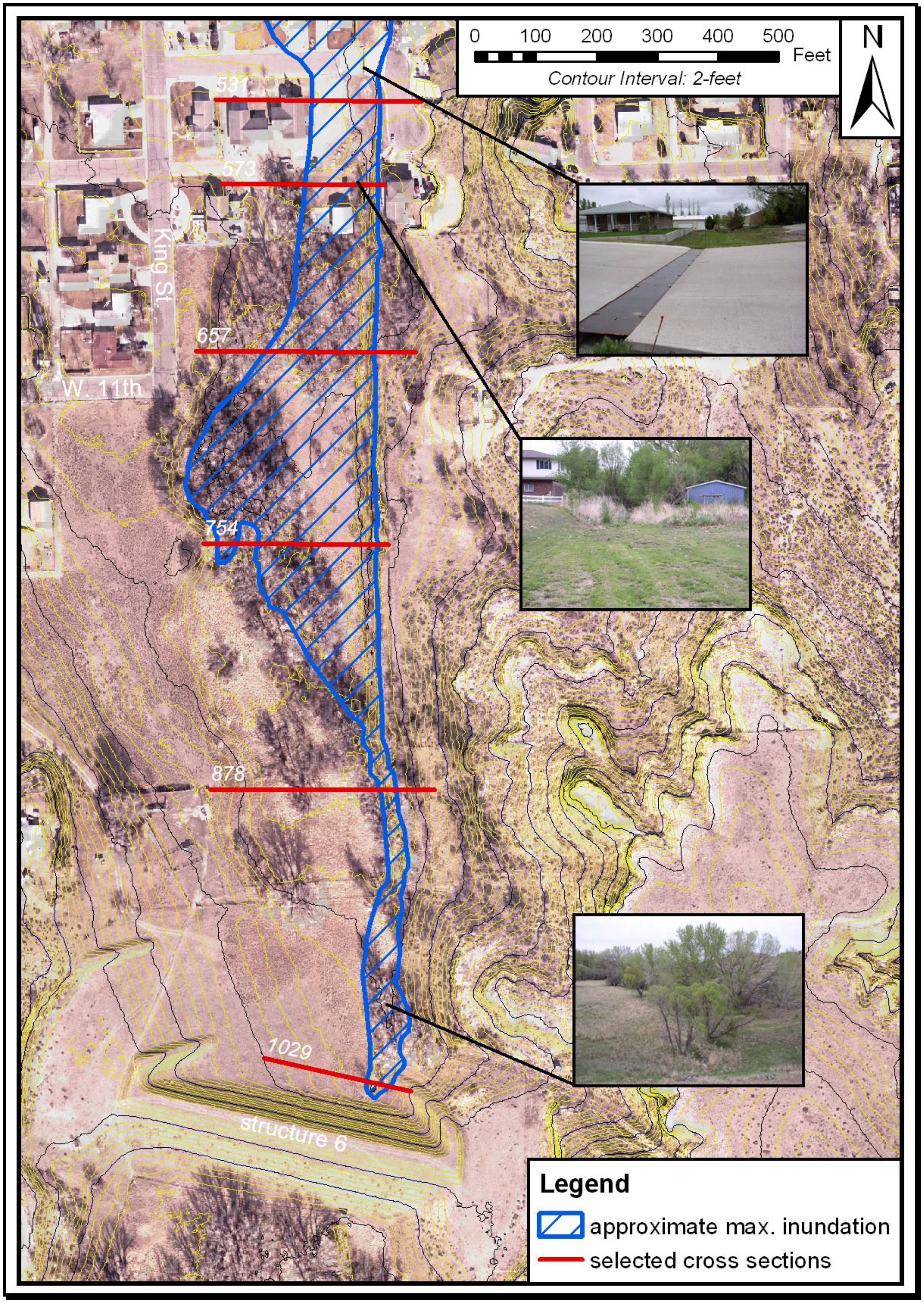



**Figure 35:** Southern (upstream) extent of the approximate inundation extents for the 100-year peak flow event, from structure 6 to 7th Street.

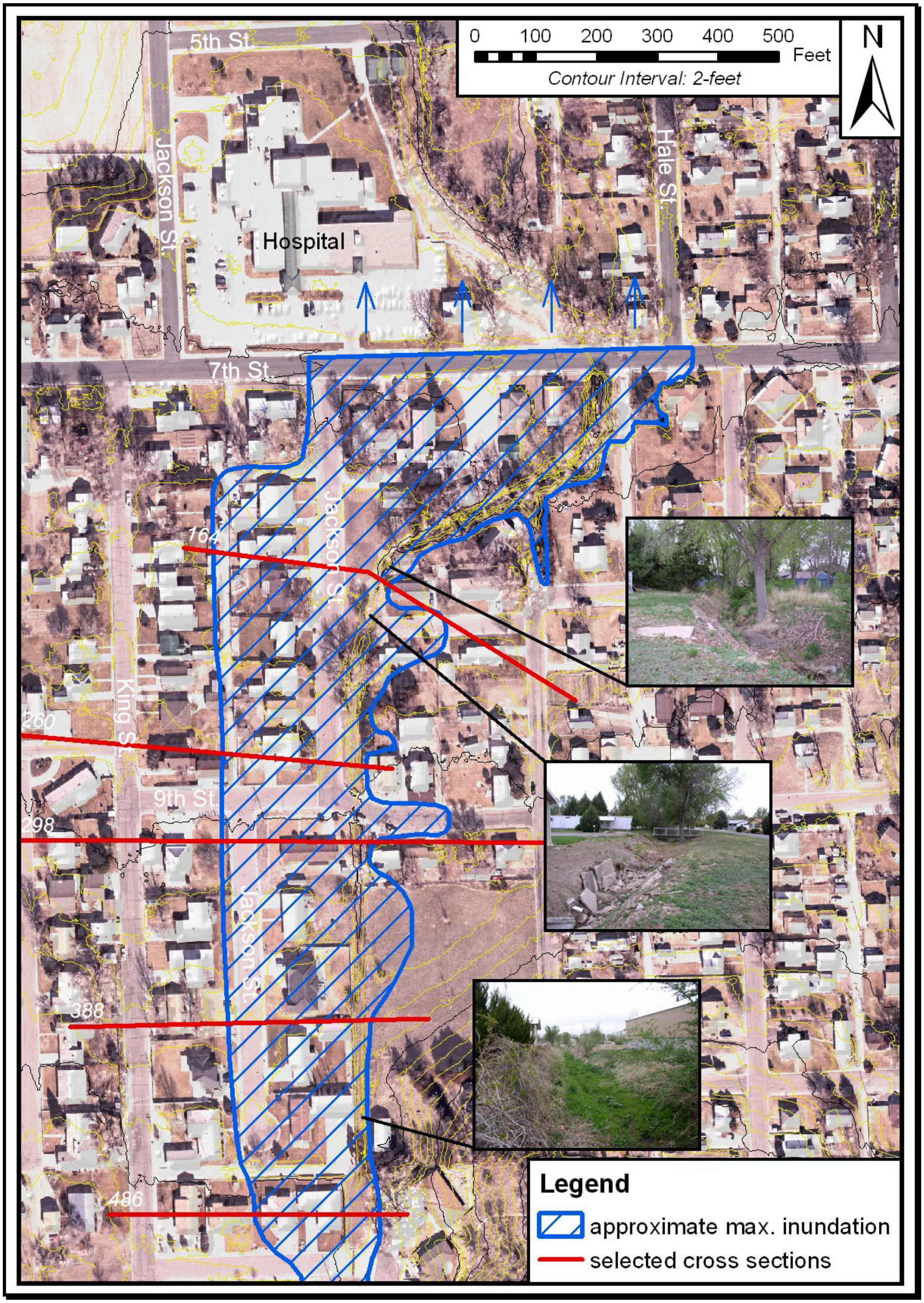



**Figure 36:** Northern (downstream) extent of the approximate inundation extents for the 100-year peak flow event, from structure 6 to 7th Street.

## CONCLUSIONS

Hydrologic analyses have been performed for the 6 flood detention structures immediately to the South and upstream of Wray, Colorado. The six structures have drainage areas ranging from 70 to 303 acres. Storage capacity, to the crest of the emergency spillway, ranges from 16 to 196 acre-feet. Conveyance capacities of the emergency and principal spillways were found to range from 1,200 to 11,800 cfs. The inundation pool extents are provided as figures in the FLOOD RETENTION STRUCTURES section. Only structure 4 was found to have a flood pool that will submerge existing structures.

All structures except number 4 were computed to be capable of conveying flows at or in excess of the 500-year storm event. Structure 4 will begin to use its emergency spillway during storms greater than or equal to approximately the 50-year event.

All six of the structures were found to be capable of conveying the probable maximum precipitation (PMP) event through their principal and emergency spillways without overtopping their embankments. For all structures, the six hour storm was found to cause higher pool water surface elevations than the 24-hour storm. Modeling of structures 1 and 2 indicate that the PMP will fill the reservoirs to within 3.0 and 0.5 feet, respectively, of the top of embankment. Modeling of structures 3 and 4 indicate that the PMP will fill the reservoirs to within 2.5 and 0.8 feet, respectively, of the top of embankment. Modeling of structures 5 and 6 indicate that that the PMP will fill the reservoirs to within 3.2 and 1.5 feet, respectively, of the top of embankment.

The uncontrolled flow above each detention structure is greatly attenuated so that reservoir outlet flows, through the 500-year event, are substantially less than the flows entering each structure. Flow peak are attenuated between 64 and 96 percent, depending upon the structure and the storm magnitude. These structures are highly effective at reducing flooding in the town of Wray.

The hydraulic analysis is limited to evaluating the flow runoff to be expected from the 5 drainages that have flood detention structures. Hydraulic modeling was performed only to the point where the drainages flow into an urban stormwater management system. This project’s scope does not include the evaluation of Wray’s stormwater management system – it is recommended that such an analysis be performed to assess the capabilities of the entire system. Additionally, the potential flooding of Wray from the Republican River has also not been evaluated.

Inundation mapping for the 100-year event for each of the floodways below each structure have been developed and are provided in the previous section.

There is currently a 2.5 foot diameter section of culvert (Figure 18) restricting the exit of the 6-foot diameter pipe (Figure 17) that passes flow from structure 1 and adjacent uncontrolled drainages under US-34. The restriction decreases that capacity so that the flow will now overtop the US-34 once every 10 to 25 years. Without this restriction, the 6’ culvert under US-34 will likely pass the 100-year event. To reduce the hazard for vehicles traveling on US-34, it is recommended that the restriction of the 2.5’ diameter portion of the culvert under US-34 be removed and that the full culvert capacity be reestablished.

The 3-foot diameter culverts that serve as entrances to the urban stormwater system downstream of structures 2 and 4 can both convey approximately the 10-year storm event. The town may want to consider enlarging these culverts, or reestablishing surface drainage, to convey higher flow capacities and reduce flood hazards to the adjacent homes. Additionally, the drainage channel for structure 2 has been eliminated in places while the drainage for structure 5 has been reduced by infilling and encroachment (Figures 22 and 29). These channels should be recreated or enlarged and a maintenance program initiated to maintain the conveyance capacity of these channels.

Between structures 3 and 4 several structures are located within the 100-year floodplain and will be inundated in the case of such an event. This is in addition to potential flooding of several structures in the 100-year flood pool for structure 4.

Downstream of structure 4 there is an adequate drainage channel (Figure 25) up to the box culvert entrance to the urban stormwater system. This box culvert (Figure 25) has the capability of conveying flow in excess of the 100-year storm. If there are no downstream restrictions that limit flow conveyance, this drainage appears to be adequate. The capability of the downstream stormwater system should be verified.

Downstream of structures 5 and 6, the 6 foot diameter culvert just downstream of the bridge at W. 7$^{th}$ Street has been analyzed to be capable of conveying a bit less than the 10-year peak flow. The channel upstream of W. 7$^{th}$ street (Figure 31) has been modeled to have the capacity to convey storms in excess of the 100-year event. The installed culvert immediately adjacent to the hospital has greatly reduced the flow capacity of this drainage and increased the risk of flooding in the vicinity of the hospital. To reduce flood hazard to the hospital and adjacent homes, the removal of this culvert, installation of sufficient flow capacity at 5$^{th}$ St. and the reestablishment of an open channel should be considered. Such a channel would be capable of transporting substantially larger flows.

Additionally, enlarging the channel where it is less substantial between structure 6 and 8$^{th}$ street (Figures 35 and 36) would convey higher flow peaks and reduce flood hazard to adjacent houses. The existing insufficient channel capacity endangers homes in the proximity of this channel. Enlarging this channel should be considered.

Fundamental to the accuracy of a hydrologic study is the selection of an appropriate rainfall distribution. In this analysis a NRCS Type II distribution was used. This distribution is conservative, with high precipitation intensities that result in higher peak flows. Additional analysis may indicate that a different rainfall distribution may be more appropriate for these watersheds.

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## Appendix A: Approximate peak flow characteristics at selected model sections.

Table A-1: Approximate peak flow characteristics downstream of structure 1.

| Section ID | Storm Frequency | Peak Discharge (cfs) | Peak W.S. Elev (ft) | Maximum Depth (ft) | Velocity Channel (ft/s) | Velocity Fldpln Left (ft/s) | Velocity Fldpln Right (ft/s) | Channel Froude # |
|---|---|---|---|---|---|---|---|---|
| 479 | 10-year | 25.3 | 3554.0 | 1.0 | 2.5 | 1.5 | 0.2 | 0.57 |
| 479 | 25-year | 27.8 | 3553.9 | 0.9 | 2.8 | 1.7 | 0.2 | 0.64 |
| 479 | 50-year | 29.6 | 3554.0 | 1.0 | 2.9 | 1.7 | 0.3 | 0.64 |
| 479 | 100-year | 31.2 | 3554.0 | 1.0 | 2.7 | 1.6 | 0.5 | 0.57 |
| 418 | 10-year | ---- | 3552.4 | 1.1 | ---- | ---- | ---- | ---- |
| 418 | 25-year | 27.6 | 3552.3 | 1.0 | 2.0 | 0.7 | 0.8 | 0.42 |
| 418 | 50-year | 27.6 | 3552.3 | 1.0 | 1.9 | 0.7 | 0.8 | 0.39 |
| 418 | 100-year | ---- | 3552.7 | 1.4 | ---- | ---- | ---- | ---- |
| 356 | 10-year | 100.8 | 3549.0 | 1.5 | 4.9 | 1.5 | 1.8 | 0.89 |
| 356 | 25-year | 136.2 | 3549.0 | 1.5 | 6.3 | 1.9 | 2.4 | 1.15 |
| 356 | 50-year | 168.3 | 3549.0 | 1.5 | 6.3 | 2.1 | 2.6 | 1.11 |
| 356 | 100-year | 200.7 | 3549.2 | 1.7 | 4.4 | 1.7 | 2.3 | 0.71 |
| 240 | 10-year | 55.1 | 3542.5 | 2.0 | 1.0 | 0.4 | 0.1 | 0.14 |
| 240 | 25-year | 101.1 | 3543.8 | 3.3 | 0.7 | 0.3 | 0.1 | 0.07 |
| 240 | 50-year | 180.1 | 3543.9 | 3.4 | 1.1 | 0.5 | 0.2 | 0.12 |
| 240 | 100-year | 223.8 | 3544.0 | 3.5 | 1.4 | 0.6 | 0.3 | 0.14 |
| 147 | 10-year | 52.9 | 3542.5 | 6.3 | 0.6 |  | 0.2 | 0.05 |
| 147 | 25-year | 99.3 | 3543.8 | 7.6 | 0.2 | 0.0 | 0.1 | 0.02 |
| 147 | 50-year | 178.1 | 3543.9 | 7.7 | 0.4 | 0.1 | 0.2 | 0.03 |
| 147 | 100-year | 220.9 | 3544.0 | 7.8 | 0.5 | 0.1 | 0.2 | 0.03 |

Table A-2: Approximate peak flow characteristics downstream of structure 2.

| Section ID | Storm Frequency | Peak Discharge (cfs) | Peak W.S. Elev (ft) | Maximum Depth (ft) | Velocity Channel (ft/s) | Velocity Fldpln Left (ft/s) | Velocity Fldpln Right (ft/s) | Channel Froude # |
|---|---|---|---|---|---|---|---|---|
| 142 | 10-year | 12.3 | 3566.2 | 0.6 | 2.6 | ---- | ---- | 0.84 |
| 142 | 25-year | 14.1 | 3566.3 | 0.7 | 2.3 | 0.1 | ---- | 0.67 |
| 142 | 50-year | 15.5 | 3566.4 | 0.8 | 2.1 | 0.2 | 0.5 | 0.56 |
| 142 | 100-year | 16.7 | 3566.4 | 0.8 | 2.1 | 0.2 | 0.6 | 0.55 |
| 5 | 10-year | 45.0 | 3555.7 | 1.0 | 3.7 | 1.3 | 0.6 | 0.79 |
| 5 | 25-year | 57.3 | 3555.8 | 1.1 | 4.1 | 1.4 | 0.8 | 0.81 |
| 5 | 50-year | 67.5 | 3555.9 | 1.2 | 4.3 | 1.5 | 0.9 | 0.81 |
| 5 | 100-year | 77.4 | 3556.0 | 1.3 | 4.5 | 1.6 | 1.1 | 0.82 |

Table A-3: Approximate peak flow characteristics downstream of structure 3.

| Section ID | Storm Frequency | Peak Discharge (cfs) | Peak W.S. Elev (ft) | Maximum Depth (ft) | Velocity Channel (ft/s) | Velocity Fldpln Left (ft/s) | Velocity Fldpln Right (ft/s) | Channel Froude # |
|---|---|---|---|---|---|---|---|---|
| 1324 | 10-year | 19.5 | 3637.7 | 0.8 | 3.2 | 1.2 | 0.4 | 0.74 |
| 1324 | 25-year | 24.1 | 3637.7 | 0.9 | 3.5 | 1.4 | 0.6 | 0.77 |
| 1324 | 50-year | 27.2 | 3637.8 | 0.9 | 3.7 | 1.4 | 0.7 | 0.78 |
| 1324 | 100-year | 29.5 | 3637.8 | 1.0 | 3.8 | 1.5 | 0.7 | 0.79 |
| 1095 | 10-year | 80.5 | 3626.6 | 0.9 | 1.5 | 2.2 | 0.3 | 0.31 |
| 1095 | 25-year | 114.0 | 3626.7 | 1.1 | 1.7 | 2.3 | 0.4 | 0.32 |
| 1095 | 50-year | 141.7 | 3626.8 | 1.2 | 1.8 | 2.4 | 0.5 | 0.32 |
| 1095 | 100-year | 170.2 | 3626.9 | 1.2 | 1.9 | 2.5 | 0.6 | 0.32 |
| 913 | 10-year | 145.6 | 3618.9 | 2.4 | 3.1 | 1.0 | 1.8 | 0.38 |
| 913 | 25-year | 207.4 | 3619.2 | 2.7 | 3.4 | 1.1 | 1.9 | 0.39 |
| 913 | 50-year | 259.1 | 3619.5 | 3.0 | 3.6 | 1.2 | 2.0 | 0.39 |
| 913 | 100-year | 311.4 | 3619.7 | 3.2 | 3.8 | 1.3 | 2.0 | 0.39 |
| 798 | 10-year | 214.4 | 3610.9 | 2.8 | 4.9 | 1.5 | 2.0 | 0.61 |
| 798 | 25-year | 304.0 | 3611.3 | 3.2 | 5.5 | 1.8 | 2.2 | 0.62 |
| 798 | 50-year | 380.3 | 3611.5 | 3.5 | 5.9 | 2.0 | 2.3 | 0.63 |
| 798 | 100-year | 457.6 | 3611.7 | 3.7 | 6.5 | 2.2 | 2.6 | 0.68 |
| 562 | 10-year | 281.3 | 3598.1 | 2.4 | 4.8 | 2.7 | 2.6 | 0.6 |
| 562 | 25-year | 397.7 | 3598.4 | 2.7 | 5.2 | 2.9 | 3.1 | 0.62 |
| 562 | 50-year | 498.0 | 3598.5 | 2.8 | 5.5 | 3.1 | 3.4 | 0.62 |
| 562 | 100-year | 599.7 | 3598.7 | 3.0 | 5.7 | 3.3 | 3.7 | 0.63 |
| 453 | 10-year | 281.0 | 3594.4 | 2.1 | 4.5 | ---- | 2.3 | 0.61 |
| 453 | 25-year | 397.2 | 3594.7 | 2.4 | 5.3 | 0.8 | 2.8 | 0.65 |
| 453 | 50-year | 497.5 | 3594.9 | 2.6 | 5.8 | 1.2 | 3.2 | 0.69 |
| 453 | 100-year | 599.2 | 3595.1 | 2.8 | 6.4 | 1.6 | 3.5 | 0.72 |

Table A-4: Approximate peak flow characteristics downstream of structure 4.

| Section ID | Storm Frequency | Peak Discharge (cfs) | Peak W.S. Elev (ft) | Maximum Depth (ft) | Velocity Channel (ft/s) | Velocity Fldpln Left (ft/s) | Velocity Fldpln Right (ft/s) | Channel Froude # |
|---|---|---|---|---|---|---|---|---|
| 178 | 10-year | 54.3 | 3560.4 | 2.6 | 1.5 | ---- | ---- | 0.2 |
| 178 | 25-year | 69.2 | 3560.6 | 2.8 | 1.7 | ---- | ---- | 0.22 |
| 178 | 50-year | 82.2 | 3560.8 | 3.0 | 1.9 | ---- | ---- | 0.23 |
| 178 | 100-year | 95.0 | 3561.0 | 3.2 | 2.0 | ---- | ---- | 0.24 |
| 116 | 10-year | 54.2 | 3559.2 | 1.5 | 2.5 | ---- | ---- | 0.42 |
| 116 | 25-year | 69.0 | 3559.4 | 1.7 | 2.6 | ---- | ---- | 0.41 |
| 116 | 50-year | 82.0 | 3559.6 | 1.9 | 2.7 | ---- | ---- | 0.41 |
| 116 | 100-year | 94.8 | 3559.7 | 2.1 | 2.8 | ---- | ---- | 0.41 |
| 57 | 10-year | 52.5 | 3558.6 | 2.9 | 1.4 | ---- | ---- | 0.2 |
| 57 | 25-year | 67.6 | 3558.9 | 3.2 | 1.4 | ---- | ---- | 0.2 |
| 57 | 50-year | 80.8 | 3559.1 | 3.4 | 1.5 | ---- | ---- | 0.21 |
| 57 | 100-year | 93.9 | 3559.3 | 3.6 | 1.6 | ---- | ---- | 0.21 |
| 21 | 10-year | 93.4 | 3558.3 | 3.1 | 2.5 | 0.2 | ---- | 0.35 |
| 21 | 25-year | 121.8 | 3558.5 | 3.3 | 2.8 | 0.3 | 0.0 | 0.38 |
| 21 | 50-year | 145.5 | 3558.7 | 3.5 | 3.1 | 0.4 | 0.2 | 0.39 |
| 21 | 100-year | 169.3 | 3558.9 | 3.6 | 3.3 | 0.4 | 0.1 | 0.41 |

Table A-5: Approximate peak flow characteristics downstream of structure 5.

| Section ID | Storm Frequency | Peak Discharge (cfs) | Peak W.S. Elev (ft) | Maximum Depth (ft) | Velocity Channel (ft/s) | Velocity Fldpln Left (ft/s) | Velocity Fldpln Right (ft/s) | Channel Froude # |
|---|---|---|---|---|---|---|---|---|
| 405 | 10-year | 14.8 | 3595.6 | 0.9 | 2.6 | ---- | 0.4 | 0.65 |
| 405 | 25-year | 24.5 | 3595.8 | 1.1 | 3.1 | ---- | 0.6 | 0.68 |
| 405 | 50-year | 27.3 | 3595.8 | 1.1 | 3.2 | ---- | 0.7 | 0.69 |
| 405 | 100-year | 29.6 | 3595.8 | 1.1 | 3.3 | ---- | 0.7 | 0.69 |
| 331 | 10-year | 14.8 | 3589.2 | 1.0 | 2.8 | ---- | 0.9 | 0.65 |
| 331 | 25-year | 24.5 | 3589.3 | 1.1 | 3.4 | 0.3 | 1.2 | 0.69 |
| 331 | 50-year | 27.3 | 3589.4 | 1.2 | 3.5 | 0.3 | 1.2 | 0.70 |
| 331 | 100-year | 29.6 | 3589.4 | 1.2 | 3.6 | 0.4 | 1.3 | 0.71 |
| 251 | 10-year | 3.2 | 3586.0 | 1.2 | 0.3 | 0.1 | 0.1 | 0.05 |
| 251 | 25-year | 3.9 | 3586.1 | 1.3 | 0.3 | 0.1 | 0.1 | 0.05 |
| 251 | 50-year | 4.2 | 3586.2 | 1.4 | 0.3 | 0.1 | 0.1 | 0.04 |
| 251 | 100-year | 4.9 | 3586.3 | 1.5 | 0.3 | 0.1 | 0.1 | 0.04 |
| 204 | 10-year | 32.4 | 3583.6 | 0.8 | 3.2 | 0.7 | 0.6 | 0.73 |
| 204 | 25-year | 41.6 | 3583.7 | 0.9 | 3.5 | 0.8 | 0.7 | 0.76 |
| 204 | 50-year | 49.4 | 3583.7 | 0.9 | 3.7 | 0.8 | 0.8 | 0.77 |
| 204 | 100-year | 56.9 | 3583.8 | 1.0 | 3.9 | 0.9 | 0.8 | 0.76 |
| 147 | 10-year | 30.8 | 3580.7 | 3.2 | 0.5 | 0.2 | 0.1 | 0.05 |
| 147 | 25-year | 38.7 | 3581.3 | 3.8 | 0.5 | 0.2 | 0.1 | 0.05 |
| 147 | 50-year | 44.3 | 3581.9 | 4.4 | 0.5 | 0.2 | 0.1 | 0.04 |
| 147 | 100-year | 48.9 | 3582.4 | 4.9 | 0.4 | 0.1 | 0.1 | 0.04 |
| 117 | 10-year | 30.8 | 3578.9 | 1.4 | 3.8 | ---- | 0.2 | 0.80 |
| 117 | 25-year | 38.7 | 3579.0 | 1.5 | 4.0 | 0.2 | 0.7 | 0.79 |
| 117 | 50-year | 44.3 | 3579.1 | 1.6 | 4.1 | 0.4 | 1.0 | 0.77 |
| 117 | 100-year | 48.9 | 3579.1 | 1.6 | 4.2 | 0.5 | 1.1 | 0.77 |
| 37 | 10-year | 29.3 | 3574.9 | 0.8 | 1.9 | 0.7 | 0.5 | 0.39 |
| 37 | 25-year | 36.9 | 3575.0 | 0.9 | 2.0 | 0.8 | 0.6 | 0.38 |
| 37 | 50-year | 42.4 | 3575.1 | 1.0 | 2.0 | 0.8 | 0.6 | 0.38 |
| 37 | 100-year | 46.9 | 3575.1 | 1.1 | 2.1 | 0.8 | 0.6 | 0.37 |

Table A-6: Approximate peak flow characteristics downstream of structure 6.

| Section ID | Storm Frequency | Peak Discharge (cfs) | Peak W.S. Elev (ft) | Maximum Depth (ft) | Velocity Channel (ft/s) | Velocity Fldpln Left (ft/s) | Velocity Fldpln Right (ft/s) | Channel Froude # |
|---|---|---|---|---|---|---|---|---|
| 1029 | 10-year | 47 | 3592.2 | 2.4 | 2.9 | ---- | ---- | 0.47 |
| 1029 | 25-year | 55 | 3592.3 | 2.5 | 3.1 | ---- | ---- | 0.49 |
| 1029 | 50-year | 58 | 3592.3 | 2.5 | 3.2 | ---- | ---- | 0.50 |
| 1029 | 100-year | 61 | 3592.3 | 2.5 | 3.2 | ---- | ---- | 0.51 |
| 878 | 10-year | 47 | 3584.3 | 2.3 | 2.7 | ---- | ---- | 0.45 |
| 878 | 25-year | 55 | 3584.4 | 2.5 | 2.8 | ---- | ---- | 0.45 |
| 878 | 50-year | 58 | 3584.5 | 2.5 | 2.9 | ---- | ---- | 0.45 |
| 878 | 100-year | 61 | 3584.5 | 2.5 | 2.9 | ---- | ---- | 0.46 |
| 754 | 10-year | 50 | 3580.6 | 2.0 | 1.9 | 0.3 | 0.4 | 0.24 |
| 754 | 25-year | 59 | 3580.7 | 2.1 | 2.0 | 0.4 | 0.5 | 0.25 |
| 754 | 50-year | 71 | 3580.8 | 2.2 | 2.1 | 0.4 | 0.5 | 0.26 |
| 754 | 100-year | 84 | 3580.9 | 2.4 | 2.2 | 0.5 | 0.5 | 0.26 |
| 657 | 10-year | 50 | 3576.1 | 2.0 | 3.4 | 0.2 | ---- | 0.57 |
| 657 | 25-year | 59 | 3576.3 | 2.2 | 3.6 | 0.3 | ---- | 0.58 |
| 657 | 50-year | 70 | 3576.4 | 2.3 | 3.9 | 0.4 | 0.2 | 0.59 |
| 657 | 100-year | 83 | 3576.5 | 2.4 | 4.1 | 0.3 | 0.3 | 0.60 |
| 573 | 10-year | 70 | 3571.7 | 1.7 | 4.7 | 0.8 | 1.4 | 0.71 |
| 573 | 25-year | 95 | 3572.0 | 1.9 | 5.4 | 0.9 | 1.7 | 0.75 |
| 573 | 50-year | 117 | 3572.1 | 2.1 | 5.9 | 1.0 | 1.9 | 0.79 |
| 573 | 100-year | 138 | 3572.3 | 2.2 | 6.4 | 1.1 | 2.1 | 0.82 |
| 531 | 10-year | 70 | 3569.7 | 1.0 | 3.2 | 0.8 | 1.7 | 0.60 |
| 531 | 25-year | 95 | 3569.8 | 1.1 | 3.5 | 1.0 | 1.9 | 0.60 |
| 531 | 50-year | 117 | 3570.0 | 1.3 | 3.7 | 1.1 | 2.0 | 0.59 |
| 531 | 100-year | 138 | 3570.1 | 1.4 | 3.8 | 1.2 | 2.1 | 0.58 |
| 486 | 10-year | 70 | 3568.9 | 2.5 | 1.3 | 0.6 | 0.5 | 0.15 |
| 486 | 25-year | 95 | 3569.2 | 2.8 | 1.5 | 0.7 | 0.6 | 0.16 |
| 486 | 50-year | 115 | 3569.4 | 3.0 | 1.6 | 0.8 | 0.6 | 0.17 |
| 486 | 100-year | 137 | 3569.6 | 3.2 | 1.8 | 0.8 | 0.7 | 0.18 |
| 388 | 10-year | 169 | 3565.3 | 2.7 | 5.5 | ---- | ---- | 0.82 |
| 388 | 25-year | 227 | 3565.7 | 3.1 | 5.8 | ---- | ---- | 0.82 |
| 388 | 50-year | 278 | 3566.0 | 3.4 | 6.0 | ---- | ---- | 0.81 |
| 388 | 100-year | 282 | 3566.4 | 3.8 | 4.8 | 0.3 | 0.5 | 0.61 |
| 298 | 10-year | 175 | 3560.6 | 3.3 | 4.9 | ---- | ---- | 0.75 |
| 298 | 25-year | 236 | 3560.9 | 3.7 | 4.9 | ---- | ---- | 0.78 |
| 298 | 50-year | 290 | 3561.2 | 3.9 | 5.0 | ---- | ---- | 0.79 |
| 298 | 100-year | 523 | 3561.6 | 4.4 | 1.0 | 0.9 | 0.1 | 0.15 |
| 260 | 10-year | 175 | 3558.0 | 3.2 | 5.3 | ---- | ---- | 0.75 |
| 260 | 25-year | 236 | 3558.4 | 3.6 | 5.7 | ---- | ---- | 0.76 |
| 260 | 50-year | 290 | 3558.7 | 3.9 | 5.9 | ---- | ---- | 0.77 |
| 260 | 100-year | ---- | 3559.2 | 4.4 | 4.6 | 2.8 | 0.5 | 0.54 |
| 164 | 10-year | 251 | 3553.1 | 3.4 | 6.8 | 0.6 | ---- | 0.93 |
| 164 | 25-year | 339 | 3553.6 | 3.9 | 3.8 | 2.8 | 0.1 | 0.47 |
| 164 | 50-year | 374 | 3553.7 | 4.1 | 3.3 | 2.7 | 0.3 | 0.40 |
| 164 | 100-year | 624 | 3553.9 | 4.3 | 4.4 | 3.9 | 0.5 | 0.51 |