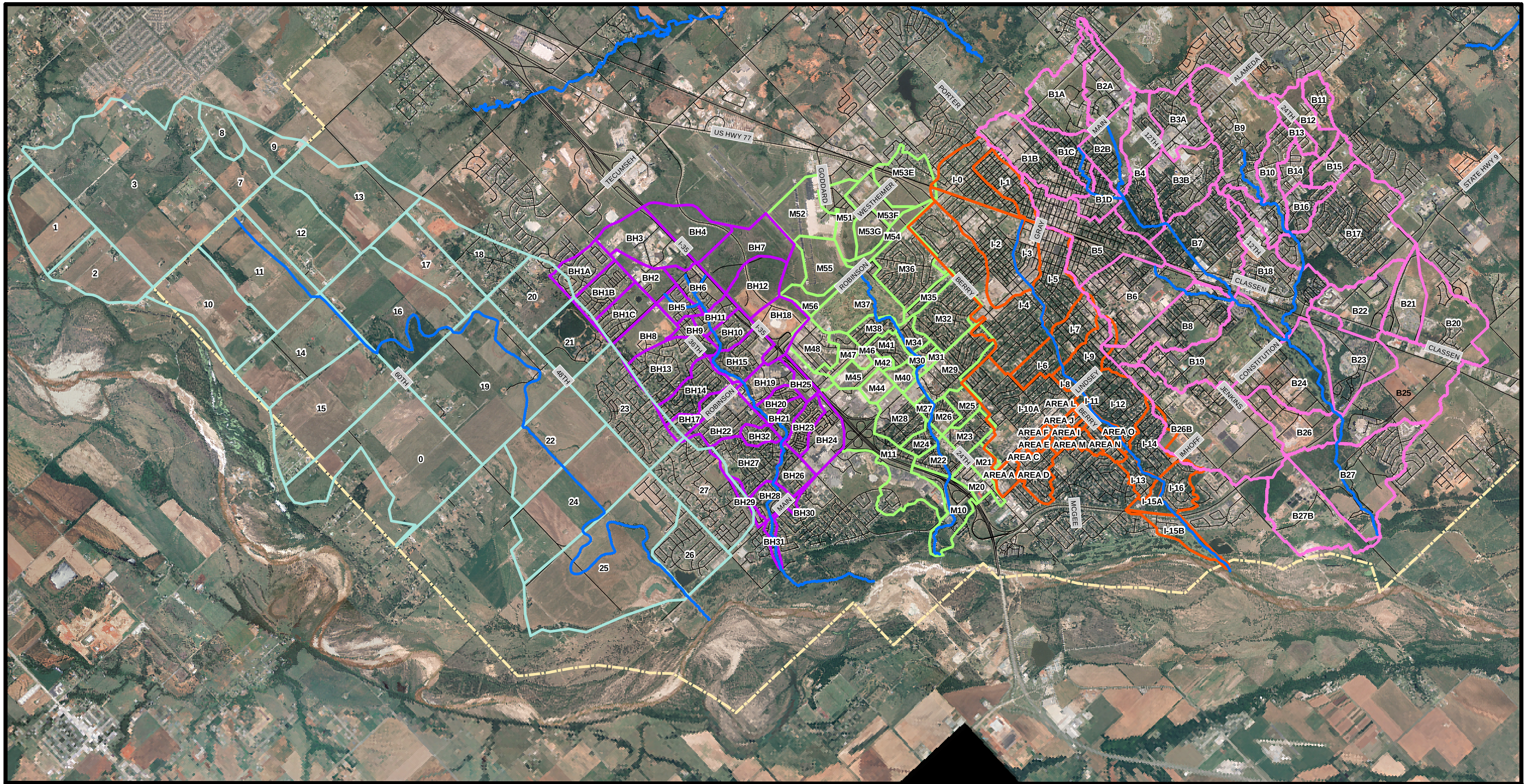


Legend
- Bishop Creek
- Brookhaven Creek
- Imhoff Creek
- Merkle Creek
- Ten Mile Flat
- City Boundary

0 2,500 5,000 10,000 Feet

Aerial Photography: 2006
Coordinate System: Oklahoma State Plane, South Zone
Horizontal Datum: NAD 1983
Vertical Datum: NAVD 1988

PBSJ

Storm Water Master Plan

Figure 4-3

Urban Watersheds

Job No.: 044194100	Date: 12-19-08	1 inch equals 4,500 feet
File: W:/WR/proj/441941_Norman/Report/Figures/DrainageAreaFigures/		

turf grass that would be typical in such developed areas. In areas projected for low density development (cres, vlres, open, park and fplain), the 300-ft sheet flow length was retained.

The shallow concentrated flow path was also reviewed to determine whether it was predominantly within one or more of the low-density categories. If it was determined to be in such an area, the shallow concentrated flow path was considered to remain unpaved. For other, more densely developed land uses, the shallow concentrated flow path was considered paved.

Since much of the area in the Level 1 watershed area is projected for low density development, many of the future condition lag times change very little compared to the existing conditions. The assumption of a higher sheet flow n-value (based on dense grass, which corresponds to yards or other maintained/manicured spaces) for developed areas also tends to reduce the potential change in the lag time due to development. However, the lag times do generally tend to decrease for most subbasins when comparing existing and future lag times. The existing and future lag times along with other HEC-HMS parameters are shown in Appendix F.

4.1.1.1.6 Loss Rate Parameters

The NRCS (formerly SCS) Curve Number methodology was used to develop the loss rate parameters for all detailed hydrologic modeling. The curve numbers used for the study were derived from the curve number values provided in the NRCS TR-55 document with the assumption of antecedent moisture condition II.

Existing Condition Curve Numbers

The existing condition NRCS curve numbers for Level 1 study areas were developed from a combination of a base curve number combined with a percentage of impervious cover. The curve number and impervious percentage were then input into HEC-HMS and the model was allowed to calculate the composite curve number. This approach was selected over the alternative of selecting pre-weighted curve numbers that are available from TR-55 and a variety of other sources because of the availability of detailed impervious cover data for the City of Norman. The availability of this data allowed for a more detailed accounting of impervious cover that would be possible from average values from a table. For the studied portions of the Little River watershed north of the City of Norman and outside of the area with available impervious cover data, the percentage of impervious cover was estimated based on 2006 aerial photography.

The Level 2 studies, which directly adapted existing hydrologic models provided by the City, retained the existing condition curve numbers included in those models. Revised, future condition curve numbers were calculated for these watersheds. The development of future condition curve number for the Level 1 and 2 studies is described in a subsequent section.

The base curve number used for the existing condition determinations were derived from the TR-55 values for “Pasture, grassland, or range – continuous forage for grazing” and “Woods.” The pasture category is equivalent to the “Open space (lawns, parks, golf courses, cemeteries, etc.” and is appropriate for both open spaces in developed areas and non-wooded, undeveloped areas. The curve numbers for these classifications and the four hydrologic soil groups are shown in Table 4-5. Good hydrologic conditions were assumed for both classifications.

Table 4-5
Base Curve Numbers for Existing Conditions

Cover Type	Hydrologic Condition	Curve Number of Hydrologic Soul Group			
		A	B	C	D
Pasture, grassland, or range – continuous forage for grazing	Good	39	61	74	80
Woods	Good	30	55	70	77

The base, existing condition curve numbers were developed in GIS by combining the Cleveland County SSURGO soils data (hydrologic soil groups), City of Norman land cover (identifies wooded areas), National Land Cover Dataset (NLCD – identify wooded areas outside of the City of Norman) and subbasin polygons developed for the Level 1 study areas. The weighted curve number for each subbasin was calculated as a weighted average from the intersection of the subbasin polygons, woods land cover (all areas not covered by woods are assumed to be pasture/open space), and hydrologic soil groups.

The impervious cover percentage was developed from the impervious layers (roads, buildings and paved areas) provided by the City of Norman. These layers were intersected with the final subbasin boundaries to determine the impervious area within each subbasin. The impervious percentage was then calculated from this area. All of the impervious cover indicated in the City’s data layers was assumed to be directly connected for the purposes of the hydrologic modeling. This assumption will tend to produce slightly conservative flows. The loss rate and unit hydrograph parameters for the Level 1 watershed subbasins are shown in Appendix F.

Future Condition Curve Numbers

The future condition NRCS curve numbers were calculated with a somewhat different approach compared to the existing condition curve numbers. This is due to the nature of the data available for the determination of future conditions. The primary dataset used to define the future conditions was the City of Norman 2025 land use projections. This polygon dataset extends beyond the city limits of Norman and provides the projected land use for all areas within the master plan study area. In addition to the future land use layer, the process used to develop the future curve numbers also incorporated the final subbasin boundaries and the hydrologic soil groups.

Since detailed estimates of impervious cover are not available for the 2025 projections, the land use dataset was used as a proxy for this information. Each 2025 land use type was associated with a corresponding TR-55 cover with its accompanying set of curve numbers. These curve numbers already incorporate an estimate of the impervious percentage based on typical values for such land uses. The cover types and curve numbers associated with the 2025 land use are shown in Table 4-6.

Table 4-6
Future (2025) Condition Curve Number Table

2025 Land Use Value	Description	Corresponding Classification (Norman Drainage Criteria – Table 5005.2)	Corresponding SCS Classification (TR-55)	A	B	C	D
open	Open	Park, Cemeteries	Open Space (Fair)	49	69	79	84
comm	Commercial	Business – Commercial Areas	Urban District (Commercial & Business)	89	92	94	95
cres	Country Residential (1D/10ac)		Open Space (Good)	39	61	74	80
flplain	Floodplain	Park, Cemeteries	Open Space (Good)	39	61	74	80
hres	High-Density Residential	Residential – Multi-unit (attached)	Residential (1/8 acre)	77	85	90	92
ind	Industrial	Industrial – Heavy uses	Urban District (Industrial)	81	88	91	93
inst	Institutional	Business – Neighborhood Areas	Urban District (Commercial & Business)	89	92	94	95
lake	Lake	Water	Water	99	99	99	99
lres	Low-Density Residential (4D/ac)		Residential (1/4 acre)	61	75	83	87
mres	Medium-Density Residential (8-10D/ac)		Residential (1/8 acre)	77	85	90	92
mu	Mixed Use	Business – Neighborhood Areas	Urban District (Commercial & Business)	89	92	94	95
nloop	North Loop	Streets – Paved, Unpaved Area	Streets & Roads (Paved & Storm Sewers)	98	98	98	98
office	Office	Business – Commercial Areas	Urban District (Commercial & Business)	89	92	94	95
park	Park	Park, Cemeteries	Open space (Good)	39	61	74	80
row	Right-of-way	Streets – Paved, Unpaved Areas	Streets & Roads (Gravel)	76	85	89	91
trans	Transportation	Streets – Paved	Streets & Roads (Paved & Open ditches)	83	89	92	93
vlres	Very Low-Density Residential (1D/2ac)		Residential (2 acre)	46	65	77	82

The future condition curve numbers were calculated based on the intersection of the 2025 land use layer, the hydrologic soil group layer and the final subbasin delineations. These curve numbers were calculated for both the Level 1 and Level 2 study areas. The calculated future condition curve numbers were then compared to the existing condition curve numbers to ensure that they either increase or were equal to the existing condition curve numbers. This comparison required the computation of impervious cover weighted curve numbers for the existing condition dataset. Due to the two methods used to develop the existing and future curve numbers, it is possible for this to occur in limited cases. If the calculated future condition curve number was lower than the existing condition value, the existing condition curve number was retained. The future condition curve numbers are shown in Appendix F.

4.1.1.1.7 Hydrologic Routing

The hydrologic routing of flows between combination points in the HEC-HMS model can have a significant impact on the magnitude and timing of the peak flows in a watershed. Routing typically causes some attenuation of the peak flow, although the attenuation is not always significant. The type of routing selected and the parameters used for that routing can have a significant impact on the level of attenuation produced by the hydrologic routing. The models used for this master plan included a variety of routing methodologies. The Level 1 models used Modified Puls and Muskingum-Cunge routing exclusively. The Level 2 models primarily used Muskingum Routing.

The Modified Puls routing approach was used in the Level 1 models for all stream reaches for which HEC-RAS modeling was available. The Modified Puls method provides the most direct accounting of the available storage within the floodplain of any of the methods available in HEC-HMS. The HEC-RAS models developed within the watershed were used to develop the storage-discharge curves required for the method. In order to generate these curves, a set of routing flows bounding the full range of anticipated flows was developed, the cross sections bounding the various routing reaches were identified and coded into the Storage Outflow option of the DSS export from HEC-RAS and the results were saved to a HEC-DSS file for use with HEC-HMS. The storage-discharge curves generated by HEC-RAS were check to ensure that there were no significant discontinuities or abrupt changes in the curve. Any such changes were smoothed out to provide a more stable routing curve. In addition to the routing curves, the average channel velocities in HEC-RAS models were used to develop the number of routing steps to be used for each routing reach.

Muskingum-Cunge routing was used for routing reaches in Level 1 watersheds that were not covered by HEC-RAS models. The 8-point cross section version of this routing method was used based on representative cross sections derived from the new 2007 topographic data.

A variety of routing methodologies were used in the various Level 2 hydrologic models. For all Level 2 watersheds, the routing used in the available models was retained for the purposes of the master planning effort. The most commonly used routing method for these models was the Muskingum method. This method was used exclusively for the Bishop Creek, Imhoff Creek and Merkle Creek watersheds and for a majority of the routing reaches in the Brookhaven Creek model. The Brookhaven Creek model also used the Kinematic Wave and Modified Puls routing methods to a limited extent. The Ten Mile Flat Creek model used the Muskingum-Cunge method exclusively. The Muskingum routing method tends to produce very little attenuation of the peak flow through a routing reach. It is quite possible that the hydrologic models for the watersheds that predominantly use this method could be under-predicting the capacity of the channel and associated floodplain to attenuate peak flows.

4.1.1.2 Summary of Hydrologic Modeling for Level 1 Watersheds

The Level 1 watersheds were modeled with the HEC-HMS model as described in the methodology sections above. These watersheds are illustrated in Exhibit 4-1 and Figures 4-1 and 4-2. The models for these watersheds were developed from scratch based on the new, 2007 topographic data for the City of Norman with parameters developed as described in the preceding sections. Unique aspects of the hydrologic modeling for each Level 1 watershed are discussed in detail in the following subsections. Both existing and future or ultimate buildout (baseline) conditions were developed for each watershed.

4.1.1.2.1 Dave Blue Creek

The Dave Blue watershed is located on the developing eastern edge of the urbanized portion of the City of Norman. The watershed is characterized by considerably steeper slopes than those of the core urban area. The 10.1-square-mile portion of the Dave Blue Creek watershed upstream of 60th Avenue was modeled in detail with HEC-HMS. The watershed modeling for Dave Blue Creek followed the methodology outlined above and did not include any significant complications.

4.1.1.2.2 Dave Blue Creek – Tributaries

The Dave Blue Creek Tributaries watershed is located just to the north of the main Dave Blue Creek watershed described above. The watershed drains to Tributary 1 to Dave Blue Creek, which ultimately flows into the main stem just downstream of 72nd Avenue. The hydrologic modeling performed for this watershed a part of the master plan encompassed 0.5 square mile and extended to a point approximately 2,400 ft downstream of 48th Avenue. The watershed modeling for Dave Blue Creek Tributaries followed the methodology outlined above and did not include any significant complications.

4.1.1.2.3 Little River

The Little River watershed is by far the largest watershed modeled as part of the master plan. The Little River model includes the Woodcrest Creek and Tributary G to Little River watersheds and encompasses a total drainage area of approximately 54.5 square miles upstream of 48th Avenue East (downstream limit of detailed study). The westernmost portion of the watershed along the IH 35 corridor has relatively flat slopes while the eastern portions of the watershed, except for the wide floodplain of Little River, is similar in character to the Rock Creek watershed. The Tributary G watershed is located predominantly in the flatter, western portion of the overall watershed. The Woodcrest watershed is located in the transitional zone between the flatter westerns and the steeper portions of the overall watershed.

The primary difference between the hydrologic modeling for the Little River watershed and the other Level 1 watersheds was the need to account for areal reduction of the design rainfall due to the size of the watershed. Areal reduction was applied to combination points along the main stem of Little River as described in subsection 4.1.1.1.2. Cumulative areas in the model with less than 10 square miles did not have areal reduction applied to develop the design flows. Such areas included Tributary G to Little River and Woodcrest Creek.

4.1.1.2.4 Rock Creek

The Rock Creek watershed, located to the north east of the currently urbanized portion of the City of Norman, is similar in characteristics to the Dave Blue Creek watershed. Like the Dave Blue watershed it has relatively steep slopes over most of the drainage area. The headwater reaches that border the Bishop Creek watershed are more developed (primarily residential) than similar areas in Dave Blue Creek. The modeled watershed encompassed 6.7 square miles and extended to a point on the main stem of Rock Creek approximately 900 ft downstream of 48th Avenue East. The watershed modeling for Rock Creek followed the methodology outlined above and did not include any significant complications. The existing small ponds in the vicinity of Robinson Street and 24th and 36th avenues were not directly modeled in the HEC-HMS model. However, they were accounted for in consideration of the time of concentration developed for the corresponding subbasins.

4.1.1.3 Summary of Hydrologic Modeling for Level 2 Watersheds

As described above, the Level 2 hydrologic models were adapted directly from existing watershed models provided by the City of Norman. The origins of these models were described in Table 4-1. The Bishop Creek, Imhoff Creek and Merkle Creek models were provided by the City in HEC-1 format while HEC-HMS models were provided for Brookhaven Creek and Ten Mile Flat Creek. HEC-HMS version 3.1.0 was used to develop the final flows for these two models. Some of the models were modified slightly so that they could more easily be used to evaluation potential solutions, to correct minor issues found in the models and to extend the models into previously unstudied areas. Specific details related to the modeling of each watershed are described in the subsections below.

The most significant modification made to the Level 2 hydrologic models was the creation of a full build-out version to represent the anticipated level of development of the watersheds as presented in the Norman 2025 plan. The models for full build-out (baseline) conditions were developed as described in the preceding methodology sections. For the Level 2 watersheds, only the curve number was modified in order to represent the increased levels of impervious cover anticipated. A majority of the area encompassed by these watersheds is either already developed, or in the case of much of the area in the Ten Mile Flat Creek watershed, marginally developable. As a result, the lag times for the subbasins in these models were not expected to change significantly. The Imhoff Creek watershed is the most heavily developed of the watersheds in the City with only minimal area available for additional development. Existing conditions in this watershed were assumed to be equivalent to the full build-out condition.

4.1.1.3.1 Bishop Creek

The Bishop Creek HEC-1 model used for the master plan was based on a 1996 model developed by Mansur-Daubert-Strella Engineers. The model uses the NRCS (SCS) unit hydrograph methodology. This version replaced a 1995 HEC-1 model, also by Mansur-Daubert-Strella Engineers, that used the Snyder unit hydrograph methodology. The report and associated documentation for the Mansur-Daubert-Strella Engineers study was not available for review during the preparation of the master plan.

The HEC-1 model provided for the Bishop Creek watershed consisted of 32 subbasins and covered approximately 8.64 square miles. This watershed area of 8.64 square miles reflects the watershed area modeled, based on the HEC-1 model obtained from the City as the starting point for the master plan analyses. The supporting report and watershed map associated with this model were not available to the project team during the development of the master plan. An approximate subbasin delineation based on the new topographic data for the City, with minor modifications made by hand (delineation shown in Figure 4-3), produced a somewhat larger area. The area shown as subwatersheds B26B and B27B in Figure 4-3 (essentially the area south of Timbrell and west of Jenkins) does not appear to be included in the HEC-1 model. This area only contributes flow at the downstream end of the hydraulically modeled stream so it was not used in the hydraulic modeling. This point is essentially at the edge of the Canadian River floodplain. In other locations in the report, such as in Section 5, a larger drainage area is given for Bishop Creek (9.87 square miles), which reflects the area downstream of where subwatersheds B26B and B27B join the main branch. This area included the drainage to the main stem as well as Tributaries A, B and C to Bishop Creek. Since existing drainage area maps were not available for the Bishop Creek watershed, the subbasins were delineated using automated routines and the topography for the area. These subbasins were then modified slightly to better conform to the areas in the model and used as a reference for the placement of flows and development of solutions. These subbasins were not intended to match the model exactly and should not be assumed to accurately reflect the delineations made for the original model. Five existing detention ponds are modeled in the Bishop Creek HEC-1 model.

4.1.1.3.2 Brookhaven Creek

The Brookhaven Creek model used for the master plan was based on a 2007 HEC-HMS model provided by C.H. Guernsey. This HEC-HMS model was developed by C.H. Guernsey based on a 1993 Letter of Map Revision HEC-1 model by Clour Engineers. The Guernsey model was used for the design of the 36th Avenue NW bridge. The 2007 model added the additional area between Robinson Street and Willow Grove Drive to the extent of the 1993 model.

The HEC-HMS model for the Brookhaven Creek watershed consisted of 33 subbasins and covered approximately 3.5 square miles. This area included the drainage to Tributaries A and B in addition to the main stem of Brookhaven Creek. The Brookhaven model includes a single detention pond.

4.1.1.3.3 Imhoff Creek

The Imhoff Creek model used for the master plan was based on the HEC-1 model from the 2001 LOMR by Baldischwiler. This LOMR incorporates refinements to the subbasin delineations and connectivity in the Lindsey and McGee area based on the Phase A improvements constructed as documented in the 1997 Baldischwiler study. The Phase A improvements provide additional drainage capacity to the south of Lindsey.

The 2001 LOMR HEC-1 model for Imhoff Creek consisted of 33 subbasins and covered approximately 3.4 square miles. This includes the area surrounding the Lindsey and McGee intersection, which has a long history of flooding issues. The subbasins in this portion of the model are quite small since they were used in the sizing of the three phase improvements proposed for the area in the 1997 Baldischwiler report. During the initial review of this model for use in the master plan, small discrepancies were found in these subbasin areas. These discrepancies were corrected with the additional of approximately 27.7 acres to subarea I-10A and the inclusion of Subbarea I-11 (5.4) acres that was missing in the model. In addition to these corrections, subbasin I-2 was split into two pieces in order to facilitate the input of flows in to the Imhoff HEC-RAS model and to facilitate the hydrologic modeling of proposed detention in the upper portion of the Imhoff Creek watershed. The final HEC-1 model used in the master plan included 34 subbasins.

4.1.1.3.4 Merkle Creek

The Merkle Creek model used for the master plan was based on the 1995 LOMR HEC-1 model developed by JWB for Clour Engineering. The 1995 LOMR model replaced the 1994 LOMR model developed by Clour Engineering and included the modeling of two detention ponds (I and II) upstream of Robinson. A subsequent LOMR in 1996 did not produce any additional changes in the HEC-1 model.

The 1995 HEC-1 model for Merkle Creek consisted of 36 subbasins and covered approximately 3.2 square miles. The 1995 model stopped at IH 35. As part of this master plan, the model was extended downstream to the confluence with the Canadian River floodplain. This extension of the model included the addition of two subbasins (M-10 and M-11). Subbasin M-10 incorporates the drainage directly to the main stem of Merkle Creek downstream of IH 35. Subbasin M-11 includes the drainage along the IH 35 corridor from the north. The contributing area of subbasin M-20 was also modified slightly to incorporate additional contributing area to the south. These changes resulted in a increase in the overall watershed area of approximately 0.56 square miles for a total area of 3.76 square miles. An additional Muskingum routing reach was also added to the model to route flows through subbasin M-10.

The Merkle Creek model includes four detention pond structures (actually five, Ponds I and II are modeled together) and four reaches of storage routing to account for the impact of backwater upstream of Robinson Street. A larger pond has recently been constructed upstream of Robinson Street. This pond will replace Pond III and will be considerably larger. The modeling for this detention facility is discussed in detail under the solutions modeling section.

4.1.1.3.5 Ten Mile Flat Creek

The Ten Mile Flat Creek model used for the master plan was based on the recently completed MacArthur Engineering CLOMR model. The HEC-HMS model for this study was completed in 2005. However the CLOMR was not ultimately approved until 2007. The Ten Mile Flat model is the only model used in the master plan that employs the Snyder unit hydrograph methodology. The unit hydrograph parameters used in the model were developed based on the USACE Tulsa District methodology. The MacArthur model for Ten Mile Flat also exclusively used the Muskingum-Cunge routing method.

The 2005 Ten Mile Flat HEC-HMS model consisted of 24 subbasins and covered approximately 11.7 square miles. The Ten Mile Flat watershed is located at the far western end of the City of Norman and is considerably different in character from the other watersheds in the City. The terrain in the watershed is very flat and much of the total area is effectively located in either the 100-year or 500-year floodplain of the Canadian River.

Much of the flow pattern within the Ten Mile Flat watershed is determined by the orientation and elevation of the existing roads. The model includes four detention ponds that are effectively formed by the backwater created by Franklin Road (ponds 2, 3 and 4) and Indian Hill Road (pond 1). Overflows along 60th Avenue NW and Tecumseh Road also have a significant impact on the hydraulic modeling for the watershed.

4.1.2 Hydrologic Modeling for Level 3 and 4 Streams

Level 3 and 4 streams, which included a majority of the streams in the undeveloped northern and western portions of the City of Norman, were analyzed with the goal of producing planning level floodplains or "Stream Planning Corridors." The hydrologic analysis used to develop flows for these streams was based on the U.S. Geological Survey regional regression equations for the State of Oklahoma. The USGS equations were used with a series of GIS tools to produce a grid of flow values. This grid was then used with the Rapid Floodplain Delineation (RFD) tool to produce basic hydraulic models and delineate floodplains for the streams. The details of this approach are described in the following subsections.

4.1.2.1 Methodology – Rapid Floodplain Delineation (RFD) Tool

The Rapid Floodplain Delineation (RFD) tool is software that automates many aspects of floodplain modeling and delineation. The program can automatically generate cross-sections, perform a backwater calculation, and delineate a floodplain in a single step. The primary goal of the program is to perform its calculations quickly and with minimum input required by the user. For example, once the stream centerline and topography have been created, a typical reach of 10 miles with cross sections spaced at 250 ft takes about 10 seconds to model and delineate. Shorter reaches can be done in 2 to 4 seconds.

The calculation method used by RFD is similar to the approach used in HEC-RAS, although much more simplified. A backwater calculation is performed that considers Manning's roughness coefficients (using one Manning's value per cross-section) and expansion and contraction losses. The version of the program used for the master plan work allowed for the input of an energy loss at stream crossings. The program currently does not include the capability to model bridges or structures in detail.

RFD also has a number of options to further facilitate rapid modeling. It can automatically generate cross-sections, and it has numerous configurable options to adjust the orientation, spacing, and width of the cross-sections. An important feature is that RFD can generate floodplains even when the cross-sections intersect, regardless of whether the intersection occurs in the floodplain or not. Since cross-section intersection is common with automatically generated sections, this is an important feature which allows a floodplain to be generated quickly without modification to the cross-sections.

Compared to a detailed hydraulic model such as HEC-RAS, RFD has some simplifying assumptions. For example, a single n-value is assigned for each cross-section, a single reach length is assigned between any two cross sections, and some other assumptions are made to speed the computation. Despite these simplifications, it is conceptually and computationally superior to any estimates of water surface elevations using normal depth approximations.

4.1.2.1.1 Preparation of Topography

The topography must be in raster (grid) form, using the gridfloat format. Gridfloat is a simple format that requires two files, one with a .hdr extension, and the other with a .flt extension. The .hdr file is a short text file which contains information about the grid cell size, size of the grid, and coordinate location of the grid. The .flt file is a binary file containing the elevation of each grid cell as single precision floating-point value. ArcMap rasters can be converted to gridfloat format using ArcMap or ArcInfo.

If different streams in a large region are being modeled, it may not be practical to mosaic all the topography available for the region into one large grid or to create numerous version of the topography for the various streams. If the user has a "checkerboard" of topography, then RFD can select the correct topography, and if needed, mosaic topography on-the-fly.

The 2007 topographic data was used with the RFD tool to develop the floodplains for the Level 3 and 4 streams. A tiled set of grids (10-ft spacing) was generated from the topographic dataset. The tiling allowed the RFD tool to use only the portions of the topographic data required for a particular stream and facilitated more rapid development of the models and floodplains.

4.1.2.1.2 Preparation of the Stream Centerline

A stream centerline or hydraulic baseline must be developed for each stream upon which the RFD tool is used. The stream centerline must be:

1) A shapefile with only one single-part line.
2) Drawn in an upstream to downstream fashion.
3) Projected (i.e., have a .prj) file, and the coordinate system must be in feet.

If the stream centerline file has more than one line, only the first line will be used by RFD. If the first line is a multi-part line, only the first part will be used by RFD as the streamline.

Traditionally, the streamline follows the thalweg — or low-flow channel — along the stream. It is also possible to use NHD (National Hydrography Dataset) centerlines or other pre-existing streamlines as the source. However, be sure the line goes from upstream to downstream — for example the NHD lines go from downstream to upstream.

The hydraulic baselines used in the RFD modeling for the City of Norman were developed directly from the 2007 topographic data. Arc Hydro tools were used to develop flow accumulation grids which were then converted into streamline grids based on an upper threshold of 40 acres. The resultant streamline grid was converted to a set of lines and minor refinements made to produce the final set of stream lines for the modeling.

4.1.2.1.3 Reading Discharges from a Grid

RFD can read discharges from a raster and assign these discharges automatically to the cross-sections. The `Q-grid` must be in gridfloat format (same format as the topography). *The discharge grid must be in geographic coordinates, NAD83*, regardless of the projection of the other files.

The RFD tool includes several options to facilitate the use of the `Q-grid`. The `qmin` option specifies the minimum flow to be used. If the value read from the grid falls below the value, the `qmin` value is used instead. If no minimum is desired then specify `qmin = 0`.

The `qdsignore` option tells RFD for how many feet at the downstream end of the reach to ignore the discharges from the grid. This option appears because many times at the downstream end of a reach, there are `q` values that are from a larger river nearby, and RFD may grab these unintended larger discharges. When this option is used, the first cross-section upstream from the point that is the `qdsignore` from the downstream limit of the streamline will be used to assign discharges to all cross-sections downstream. For example, say cross section 520 is the first cross-section more than 500 ft from the downstream limit of the centerline. The discharge at this cross-section is read from the nearest non-null cell on the `Q-grid` and is 1,760 cfs. This 1760 cfs will be assigned to all cross-sections downstream (lower numbered) of the cross-section 520.

The discharge or `Q-grid` itself must be a raster that is in the same coordinate system and datum as the stream centerline shapefile. RFD locates the grid cell where the streamline and the cross-section intersect, and checks if there is a discharge specified at that cell. If there is, that discharge is assigned to the cross-section. If not (e.g., the cell is a null cell), then RFD looks at neighboring cells and searches in larger neighbors (e.g., 1 cell away, 2 cells away) until a discharge is found. If more than one discharge is found during the search of a “neighborhood” then the highest discharge is selected.

A sample flow raster is shown in Figure 4-4. The black cells are discharge values, and the white cells are null values. In any discharge raster, the vast majority of the cells should have a null value; only those cells associated with streamlines should have discharge values.

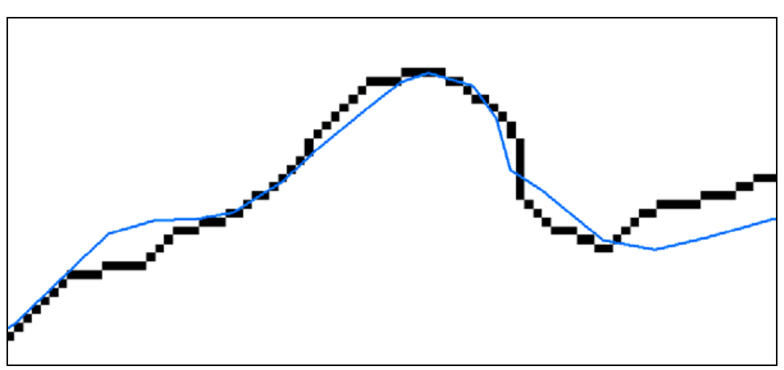


Figure 4-4. River centerline overlaid on sample flow raster.

If RFD reads a lower discharge in the downstream direction that was read upstream, RFD will assume this lower discharge is in error and will use the higher upstream discharge — thus RFD will not allow flows to decrease when going in the downstream direction.

4.1.2.2 USGS Regression Equations

USGS regression equations were used to develop full build-out condition flows for the delineation of Stream Planning Corridors on Level 3 and 4 streams. The regression equations were adapted from the Water Resources Investigation Report 97-4202, “Techniques for Estimating Peak-Streamflow Frequency for Unregulated Streams and Streams Regulated by Small Floodwater Retarding Structures in Oklahoma” (Tortorelli, 1997). This report describes the derivation of regional regression equations based on statistical analysis of historical records at gages and the characteristics of the watersheds draining to those gages within the State of Oklahoma. No significant regionalization effects were observed in the data, so a single set of equations was developed for the state.

The 100-year discharge for rural areal is defined as follows:

$$\mathbf{Q_{100(r)}} = 35.6\ A^{0.614}\ S^{0.202}\ P^{0.907}$$

Where:

A = Drainage area – the contributing drainage are of the basin, in square miles.

S = Main-channel slope – the slope measured at the points that are 10 percent and 85 percent of the main-channel length between the study site and the drainage divide, in feet per mile.

P = Mean-annual precipitation – the point mean-annual precipitation at the study site, from the period 1961–1990, in inches.

The WRI report suggests that the equations not be used outside of the range of predictor parameters used in the derivation of the equations. These ranges are defined in Table 4-7.

Table 4-7
Recommended Parameter Ranges for the USGS Regression Equations

Parameter	Lower Limit	Upper Limit
A	Equal to or greater than 0.144 mi^2	Less than or equal to 2,510 mi^2
S	Equal to or greater than 1.89 ft/mi	Less than or equal to 288 ft/mi
P	Equal to or greater than 15.0 in	Less than or equal to 55.2 in

The recommended lower limit for the area parameter is 92 acres. The lower limit of drainage areas used in the derivation of the Stream Planning Corridors was 40 acres. Even though this area threshold falls below the suggested lower limit, the extrapolation was considered reasonable given the purpose of the analysis (provide preliminary future condition 100-year floodplains) and the need to develop planning corridors for hundreds of miles of streams. Further, the Stream Planning Corridors developed with these flows matched well in overlapping areas that also received a Level 1 analysis.

The report also provides a methodology to adjust the regression-based flows to account for the level of development within a watershed. This method requires estimates of the percentage of impervious cover in the basin and the percentage of the basin served by storm sewers. The Norman 2025 land use layer was used to identify the anticipated land use types in the areas to be mapped with Stream Planning Corridors. Each 2025 land use type was related to a classification in Table 5005.2 of the City's drainage criteria. The percentage of impervious cover for each land use classifications was established as the average of the upper and lower limits listed in Table 5005.2. The percentage of the area served by storm sewers was estimated based on a review of existing storm sewers in the City of Norman and similar experience from other master planning efforts.

The 100-year discharge adjusted for urbanization is defined as follows:

$$\mathbf{Q_{100(u)}} = 2.27\ (R_L\text{-}1)\ Q_{2(r)} + 0.0167\ (7\text{-}R_L)\ Q_{100(r)}$$

Where:

$\mathbf{R_L}$ = urban adjustment factor – defined by a figure in WRIR 97-4202. The values determined for impervious percentage and percentage of area served by storm sewer are used to enter the figure and determine a value of $\mathbf{R_L}$ from a series of curves.

$\mathbf{Q_{x(r)}}$ = the regression estimate of peak discharge for ungaged sites on natural unregulated streams, for recurrence interval x, in ft^3/s.

$$\mathbf{Q_{2(r)}} = 0.075\ A^{0.615}\ S^{0.159}\ P^{2.103}$$

4.1.2.3 Development of Discharge Grid (Q-Grid) for RFD Tool

As described above, a gridded representation of flows along the study streams is used as the hydrologic input for the RFD tool. This grid is hereafter referred to as the Q-Grid. A set of spatial processing tools was used to automate the process of deriving flow values at each grid point along the study streams. This process was based on an analysis of a gridded version of the topographic data for the area. The USGS National Elevation Dataset (NED) 30-meter DEM data was used as the basis for this analysis. This data was sufficiently accurate for the derivation of the Q-Grid, especially since the areas analyzed were typically in undeveloped areas with steeper terrain, and could be processed much more efficiently.

The multi-stage process included the development of a flow direction grid based on the elevation grid, followed by the development of a flow accumulation grid and finally a stream grid based on flow accumulations above the threshold of 40 acres of contributing area. The flow accumulation and stream grids were used to calculate the contributing area **(A)** and slope **(S)** at all points along the stream grid. A grid of the mean annual precipitation was developed for the City of Norman Area. The stream grid was then intersected with the mean annual precipitation grid in order to assign a value of mean annual precipitation **(P)** to each grid cell. These steps provided the variables required to calculate the rural regression flows based on the USGS regression equations. The value of the 2-year and 100-year rural flow was calculated for each stream cell.

The urban adjustment factor **(R_L)** was required in order to complete the calculations for the urbanized regression flows. This required that the drainage accumulation grid be intersected with a grid of land use values based on the Norman 2025 data layer. This intersection was used to compute the percentage of impervious cover and the percentage of area served by storm drains at each point along the stream grid. This was then used with a discretized version of the $\mathbf{R_L}$ table from WRIR 97-4202 to determine the urban adjustment factor at each stream cell. The grid of $\mathbf{R_L}$ factors was then used with the grids of 2- and 100-year rural flows to calculate the urbanized, 100-year regression flow Q-Grid.

4.1.3 Hydrology for Local Drainage Issues

In several cases, it was necessary to develop flows for localized drainage issues. In most cases, these areas were either not covered by a detailed hydrologic model or the model in the particular area was too coarse for the specific drainage issue. In such cases, the Rational Method, as outlined in the drainage criteria for the City of Norma, was used to develop flow values.

4.1.4 Hydrologic Modeling Results

The hydrologic analyses for the master plan produced flows that were generally consistent with previous studies. Flows at selected locations in the various study watersheds are shown in Table 4-8. Flows from the recent countywide Flood Insurance Study (FIS) at comparable locations are shown in Table 4-9. As would be expected given the sources of the Level 2 models, the flows for the master plan are almost identical to the FEMA flows in most cases. Figure 4-5 shows a comparison of the unit discharges for taken from the 2008 effective FIS report and the master plan hydrologic models. The values for Level 1 and 2 watersheds are shown with separate symbols in order to better compare the results.

As the figure shows, the results for the Level 1 streams are generally consistent with those from the FIS for the same streams. The Level 2 results are also generally consistent with the exception of the significant outliers highlighted on the figure. Each of these outliers has an exceptionally high unit discharge. The two Bishop Creek Tributary outliers are simply conservative repetitions of the full basin flow at the upstream end of the studied stream. The Imhoff Creek outlier was corrected through the modifications made to the Imhoff HEC-1 model (refer to the preceding discussion of

Table 4-8
Summary of Flows at Selected Locations for Level 1 and 2 Watersheds

				Existing Condition Flows (cfs)				Full Buildout Condition Flows (cfs)			
Flow Change Location	HEC-RAS Station	HEC-HMS ID	Drainage Area (mi^2)	10-yr	50-yr	100-yr	500-yr	10-yr	50-yr	100-yr	500-yr
Bishop Creek Main Stem											
Just Downstream of Main Street	36300	2	0.45	353	508	588	748	392	549	629	788
Approximately 200 feet downstream of Apache Street	33300	9	0.85	761	1072	1231	1550	804	1116	1274	1592
Just Downstream of Confluence with Tributary B	32300	10	1.26	1178	1654	1905	2428	1280	1753	2015	2536
Approximately 60 feet upstream of Boyd	29710	20	1.95	1779	2533	2917	3734	1955	2710	3097	3917
Just Downstream of Confluence with Tributary C	25000	40	3.45	2843	4134	4795	6106	3255	4560	5223	6513
Just Downstream of Confluence with Tributary A	22120	50	6.06	4839	7128	8313	10680	5553	7909	9105	11452
Approximately 200 feet upstream of State Hwy 9	17120	60	7.50	5373	7929	9256	12183	6247	8909	10304	13257
Approximately 2700 feet upstream of State Hwy 9	14120	70	8.36	5563	8273	9682	12738	6596	9407	10908	13984
Bishop Creek Trib A											
Approximately 600 feet upstream of Sinclair Street	11000	B-9	0.47	560	811	939	1197	618	869	997	1252
Approximately 600 feet upstream of Concord	5800	48	1.21	1246	1808	2096	2675	1403	1969	2255	2829
Approximately 1200 feet upstream of 12th Ave	3760	49	1.89	1706	2602	3026	3881	1981	2816	3240	4091
Bishop Trib B											
Just downstream of Main Street	3840	DP-1	0.17	225	297	349	468	258	329	393	545
Approximately 2000 feet upstream of Alameda	2400	TRIBB	0.41	478	671	779	1012	532	707	814	1050
Bishop Trib C											
Approximately 850 feet upstream of Brooks	4600	B-5	0.40	389	566	658	843	463	643	733	915
Approximately 100 feet downstream of Brooks	3660	31	0.72	649	963	1125	1452	796	1115	1277	1600
Approximately 850 feet upstream of Lindsey	1440	TRIBC	1.06	933	1398	1638	2116	1082	1527	1752	2153
Brookhaven Main Stem											
Just Upstream of Rock Creek Rd	21230	PT 4A	0.42	357	512	591	821	342	507	571	815
Just Downstream of Confluence with Brookhaven Trib B	20140	PT 3A	0.66	446	648	750	1046	452	670	755	1079
Just Downstream of Confluence with Brookhaven Trib A	19580	PT 3	1.34	858	1295	1517	2228	916	1371	1557	2311
Just Upstream of Robinson Road	15150	PT 12	2.26	1676	2457	2860	3993	1705	2654	2993	4761
Approximately 60 feet downstream of Robinson Road	15020	PT22	2.65	2180	3150	3590	4700	2024	3205	3648	5566
Approximately 100 feet upstream of 36th NW Ave	11650	PT23-24	3.06	2430	3650	4200	5600	2470	3766	4310	6340
Just Upstream of Main Street	7650	PT27+26	3.33	2800	4120	4730	6300	2748	4157	4742	7028
Approximately 150 feet upstream of Willow Grove	5065	PT31	3.47	2970	4330	4970	6600	2829	4311	4898	7266
Brookhaven Trib A											
Just Upstream of Rock Creek Rd	31688	PT 2	0.48	333	512	610	946	326	518	597	932
Brookhaven Trib B											
Just Downstream of I-35	41060	BH7	0.14	85	125	146	207	83	126	143	207

Table 4-8, cont'd

				Existing Condition Flows (cfs)				Full Buildout Condition Flows (cfs)			
Flow Change Location	HEC-RAS Station	HEC-HMS ID	Drainage Area (mi^2)	10-yr	50-yr	100-yr	500-yr	10-yr	50-yr	100-yr	500-yr
Dave Blue											
Approximately 350 feet downstream of Post Oak Rd	15776	USPostOak	0.51	300	602	748	1152	883	1309	1500	1994
Approximately 1100 feet upstream of Cedar Lane	10847	J691A	1.02	542	1083	1359	2057	1273	1963	2291	3309
Approximately 350 feet upstream of 48th Ave	8716	J676	5.67	2513	4923	6117	9185	4236	7232	8541	12110
Approximately 600 feet upstream of confluence with Trib A to Dave Blue Creek	2382	J696	9.00	3686	7208	9166	13965	5912	10500	12539	18435
Just Downstream of Confluence with Trib A to Dave Blue Creek	1505	J699	9.99	3817	7564	9735	15109	6109	10907	13412	19748
Just Upstream of 60th SE Ave	744	DBC_Outlet	10.08	3782	7550	9672	15140	6039	10874	13424	19742
Trib A To Dave Blue											
Approximately 400 feet upstream of State Hwy 9	5740	J99	0.11	165	265	310	432	215	330	382	521
Approximately 1400 feet upstream of confluence with Dave Blue Creek	1420	J699B	0.60	481	840	1049	1493	608	1108	1324	1759
Tributary 1 to Dave Blue Creek											
Approximately 2700 feet upstream of 48th SE Ave	5102	J119	0.11	137	233	278	396	202	322	377	522
Just Upstream of 48th SE Ave	2401	J_48th	0.44	472	834	1014	1467	680	1142	1347	1886
Imhoff Creek (Existing Conditions = Full Build-Out)											
Upstream of railroad	19798	NODE 1	0.32	549	776	892	1227	549	776	892	1227
Upstream of Park Footbridge	19209	NOD 2A	0.44	714	1013	1165	1605	714	1013	1165	1605
Between University and Park	18382	NOD 2B	0.53	844	1201	1382	1908	844	1201	1382	1908
Upstream of Tonhawa	17571	NODE 2	0.75	1245	1767	2033	2803	1245	1767	2033	2803
Approximately 700 ft upstream of Symmes	15927	NOD 3A	0.97	1536	2201	2540	3524	1536	2201	2540	3524
Upstream of McNamee	14551	NODE 3	1.50	2181	3127	3609	5010	2181	3127	3609	5010
Downstream of Boyd	13758	NODE 4	1.71	2344	3387	3920	5470	2344	3387	3920	5470
Between Lindsey and Brooks	11840	NODE 5	1.88	2451	3563	4132	5789	2451	3563	4132	5789
Downstream of Lindsey	10928	NOD 5A	2.37	2955	4350	5067	7162	2955	4350	5067	7162
Outfall of Lindsey-McGee Phase I - 125 ft downstream from start of articulated block	9700	NODE Z	2.57	3114	4608	5378	7629	3114	4608	5378	7629
200 ft upstream from end of articulated block	7300	NODE 6	2.90	3304	4925	5761	8209	3304	4925	5761	8209
Upstream of Imhoff	5320	NODE 7	3.13	3489	5160	6021	8612	3489	5160	6021	8612
Upstream of SH 9	3194	NODE 8	3.29	3622	5392	6306	8978	3622	5392	6306	8978
Downstream of SH9	2944	NODE 9	3.39	3572	5318	6219	8856	3572	5318	6219	8856
Little River											
Just Downstream of 36th Avenue	72792	J584	3.39	1870	3016	3550	4979	2428	3641	4193	5638
Approximately 1200 feet Upstream of BNSF RR	68683	J4200	8.91	3802	6566	7877	11388	4901	7933	9309	12796
Approximately 650 feet downstream of 24th	63381	J4190	10.06	4108	6984	8392	12181	5237	8449	10020	13403
Approximately 1000 feet Upstream of Franklin Road	52401	J4233	11.73	4286	7262	8734	12662	5448	8798	10453	13923
Approximately 300 feet Upstream of 12th NW Ave	46416	J4264	17.83	5633	10237	12576	17540	8063	12895	15108	19398

Table 4-8, cont'd

Flow Change Location	HEC-RAS Station	HEC-HMS ID	Drainage Area (mi^2)	Existing Condition Flows (cfs) 10-yr	50-yr	100-yr	500-yr	Full Buildout Condition Flows (cfs) 10-yr	50-yr	100-yr	500-yr
Approximately 550 feet Upstream of Porter	37539	J4239	20.47	6066	10874	13330	18577	8744	13717	16012	20887
Approximately 550 feet Upstream of 12th NE Ave	29068	J4181	41.36	10035	17294	21634	31899	13881	23683	27896	39425
Approximately 1350 feet Upstream of 24th NE Ave	22197	J376	43.83	10163	17506	21806	32349	14015	23897	28217	39982
Just Upstream of 24th NE Ave	20857	J_24th	44.46	10185	17542	21835	32425	14029	23934	28271	40062
Just Upstream of Franklin Road	14100	J_Franklin_Rd	46.03	10257	17645	21940	32634	14087	24035	28423	40313
Just Upstream of 36th Avenue	11322	J_36th	48.02	10422	17908	22277	33305	14080	24250	28699	40801
Just Upstream of 48th Avenue	2481	Outlet_LR_48th	54.03	10630	18328	22778	34292	14399	24804	29372	41970
Trib G Little River											
Approximately 2300 ft upstream of Franklin Road	12645	J4206	1.91	1146	1953	2325	3331	1785	2755	3203	4339
Approximately 200 ft upstream of I-35	9825	J4203	2.17	1239	2106	2589	3642	1881	3035	3473	4758
Just Upstream of I-35	9695	J4212	2.67	1507	2534	3080	4353	2279	3639	4154	5691
Approx.600 ft upstream of Hwy 77	8529	J4267	3.38	1933	3217	3883	5439	2854	4465	5114	6542
Approx. 1300 ft downstream of RR	4017	TribG_to_LittleRiver_Outlet	4.03	1987	3056	3402	4479	2719	3706	4152	5101
Woodcrest											
Approximately 2700 feet upstream of Rock Creek Rd	15786	J4225	0.49	466	740	866	1193	530	818	948	1279
Approximately 2700 feet upstream of Rock Creek Rd	13759	J777	0.89	696	1145	1352	1949	860	1334	1548	2193
Approximately 1600 feet upstream of Nantucket	10003	J4236	2.53	1595	2649	3144	4511	2168	3373	3921	5386
Approximately 2200 feet upstream of conflence with Little River	2235	Woodcrest_outlet	3.01	1602	2714	3241	4780	2197	3497	4087	5766
Merkle Creek											
Just Downstream of Robinson Street	16217	R 1E-1	0.99	473	825	1060	1685	529	943	1165	1783
Approximately 550 feet upstream of Iowa Street	12912	R 3-3D	1.63	1039	1483	1753	2812	1115	1567	1931	2961
Approximately 50 feet downstream of Iowa Street	12252	PT 5A	1.72	1084	1544	1792	2869	1159	1628	1970	3023
Approximately 50 feet upstream of Crestmont	11152	PT 5	1.92	1282	1816	2098	3156	1353	1899	2180	3317
Approximately 250 feet upstream of Main Street	9877	PT 6	2.41	1683	2348	2708	3754	1769	2451	2809	3922
Just Downstream of Main Street	9500	PT 7	2.69	1946	2713	3117	4313	2036	2811	3219	4418
Approximately 1000 feet downstream of 24th SW Ave	6680	PT 8	3.01	2252	3168	3633	5019	2362	3276	3742	5134
Approximately 50 feet downstream of Brooks Street	4850	PT 9	3.18	2398	3393	3897	5374	2514	3508	4012	5490
Approximately 400 feet downstream of I-35	3897	PT10	3.74	2835	4061	4684	6493	3017	4244	4866	6668
Rock Creek											
Approximately 1500 feet Upstream of confluence with TribA to Rock Creek	19965	J855	1.10	909	1492	1766	2511	1445	2200	2533	3404
Just Downstream of Confluence with Trib A to Rock Creek	18467	J863	1.80	1750	2874	3300	4903	2546	3944	4486	5886
Just Downstream of Confluence with Trib B to Rock Creek	13726	J845	2.26	1709	3256	3706	5242	2868	3888	4595	6564
Just Downstream of Confluence with Trib C to Rock Creek	11837	J858	3.02	1991	4287	4880	6687	3178	4918	5512	7804
Just Upstream of Robinson Street	10762	J866	3.23	1913	4119	4942	6889	2810	4942	5616	7973

Table 4-8, cont'd

				Existing Condition Flows (cfs)				Full Buildout Condition Flows (cfs)			
Flow Change Location	HEC-RAS Station	HEC-HMS ID	Drainage Area (mi^2)	10-yr	50-yr	100-yr	500-yr	10-yr	50-yr	100-yr	500-yr
Just Downstream of Confluence with Trib D to Rock Creek	5481	J881	5.92	3110	6425	8210	11882	4333	8281	10395	13305
Approximately 2800 feet Upstream of 48th NE Ave	3785	J878	6.32	3156	6460	8319	12119	4384	8380	10106	13479
Approximately 250 feet downstream of 48th NE Ave	738	Rock_Creek_Outlet	6.70	3199	6473	8311	12192	4431	8380	10005	13400
Trib A to Rock Creek											
Just Downstream of Robinson Street	1782	J98	0.53	733	1163	1360	1878	957	1403	1603	2116
Approximately 750 feet Upstream of confluence with Rock Creek	755	TribA_Outlet	0.58	773	1237	1453	2020	995	1474	1686	2236
Trib B to Rock Creek											
Approximately 100 feet Downstream of Silverao Street	2180	J88	0.10	120	208	249	360	166	270	318	446
Approximately 680 feet Upstream of confluence with Rock Creek	680	TribB_Outlet	0.20	217	379	464	671	289	497	586	823
Trib C to Rock Creek											
Just Downstream of Alameda RD	6075	J82	0.07	112	180	211	294	172	251	287	381
Approximately 1000 feet Downstream of Alameda Rd	5022	J103	0.30	315	545	653	944	513	807	940	1304
Approximately 400 feet Downstream of Akerman Rd	1077	J850	0.67	522	1071	1279	1682	902	1403	1583	2258
Approximately 300 feet Upstream of confluence with Rock Creek	314	TribC_Outlet	0.70	536	1108	1320	1748	925	1440	1626	2307
Trib D to Rock Creek											
Just Downstream of Rock Creek Rd	4442	J108	0.32	386	628	738	1025	513	801	931	1272
Approximately 1250 feet Upstream of confluence with Rock Creek	1250	TribD_Outlet	0.54	413	749	909	1316	527	905	1076	1519

Table 4-9. Comparison of Master Plan and FEMA Flows at Comparable Locations

				Existing Conditions Flows (cfs)				Full Buildout Condition Flows (cfs)					FEMA Effective Flows			
Flow Change Location	HEC-RAS Station	HEC-HMS ID	Drainage Area (mi^2)	10-yr	50-yr	100-yr	500-yr	10-yr	50-yr	100-yr	500-yr	FIS Drainage Area (mi^2)	10-yr	50-yr	100-yr	500-yr
Bishop Creek Main Stem																
Approximately 200 feet downstream of Apache Street	33300	9	0.85	761	1072	1231	1550	804	1116	1274	1592	0.91	2400	3200	3600	4400
Approximately 60 feet upstream of Boyd	29710	20	1.95	1779	2533	2917	3734	1955	2710	3097	3917	2.13	2330	3140	3540	4350
Just Downstream of Confluence with Tributary C	25000	40	3.45	2843	4134	4795	6106	3255	4560	5223	6513	3.43	5900	8000	9000	11200
Approximately 2700 feet upstream of State Hwy 9	14120	70	8.36	5563	8273	9682	12738	6596	9407	10908	13984	8	7300	10000	11400	14300
Bishop Creek Trib A																
Approximately 600 feet upstream of Concord	5800	48	1.21	1246	1808	2096	2675	1403	1969	2255	2829	1.27	840	1130	1270	1600
Bishop Trib B																
Approximately 2000 feet upstream of Alameda	2400	TRIBB	0.41	478	671	779	1012	532	707	814	1050	0.43	550	750	830	1020
Bishop Trib C																
Approximately 850 feet upstream of Brooks	4600	B-5	0.40	389	566	658	843	463	643	733	915	0.41	1340	1780	2010	2500
Brookhaven Main Stem																
Just Upstream of Rock Creek Rd	21230	PT 4A	0.42	357	512	591	821	342	507	571	815	0.42	360	515	600	820
Just Downstream of Confluence with Brookhaven Trib A	19580	PT 3	1.34	858	1295	1517	2228	916	1371	1557	2311	1.34	860	1300	1520	2230
Just Upstream of Robinson Road	15150	PT 12	2.26	1676	2457	2860	3993	1705	2654	2993	4761	2.26	1680	2460	2860	4000
Brookhaven Trib A																
Just Upstream of Rock Creek Rd	31688	PT 2	0.48	333	512	610	946	326	518	597	932	0.48	335	515	610	950
Brookhaven Trib B																
Just Downstream of I-35	41060	BH7	0.14	85	125	146	207	83	126	143	207	0.14	85	125	150	210
Dave Blue																
Approximately 1100 feet upstream of Cedar Lane	10847	J691A	1.02	542	1083	1359	2057	1273	1963	2291	3309	1.03	867	1395	1630	2119
Imhoff Creek (Existing Conditions = Full Build-Out)																
Downstream of Boyd	13758	NODE 4	1.71	2344	3387	3920	5470	2344	3387	3920	5470	1.74	2430	3500	4050	5640
200 ft upstream from end of articulated block	7300	NODE 6	2.90	3304	4925	5761	8209	3304	4925	5761	8209	2.94	3300	4480	5630	7940
Little River																
Approximately 1200 feet Upstream of BNSF RR	68683	J4200	8.91	3802	6566	7877	11388	4901	7933	9309	12796	8.7	2590	4355	5320	7680
Woodcrest																
Approximately 2700 feet upstream of Rock Creek Rd	15786	J4225	0.49	466	740	866	1193	530	818	948	1279	0.5	790	1060	1190	1500
Approximately 2700 feet upstream of Rock Creek Rd	13759	J777	0.89	696	1145	1352	1949	860	1334	1548	2193	0.88	960	1290	1450	1850
Approximately 2200 feet upstream of conflence with Little River	2235	Woodcrest_outlet	3.01	1602	2714	3241	4780	2197	3497	4087	5766	3	2310	2290	3730	4800
Merkle Creek																
Just Downstream of Robinson Street	16217	R 1E-1	0.99	473	825	1060	1685	529	943	1165	1783	0.99	470	830	1060	1690
Approximately 250 feet upstream of Main Street	9877	PT 6	2.41	1683	2348	2708	3754	1769	2451	2809	3922	2.41	1950	2710	3120	4310
Approximately 50 feet downstream of Brooks Street	4850	PT 9	3.18	2398	3393	3897	5374	2514	3508	4012	5490	3.18	2380	3370	3870	5330

Figure 4-5: Comparison of Unit Discharges for Level 1 and Level 2 Watersheds

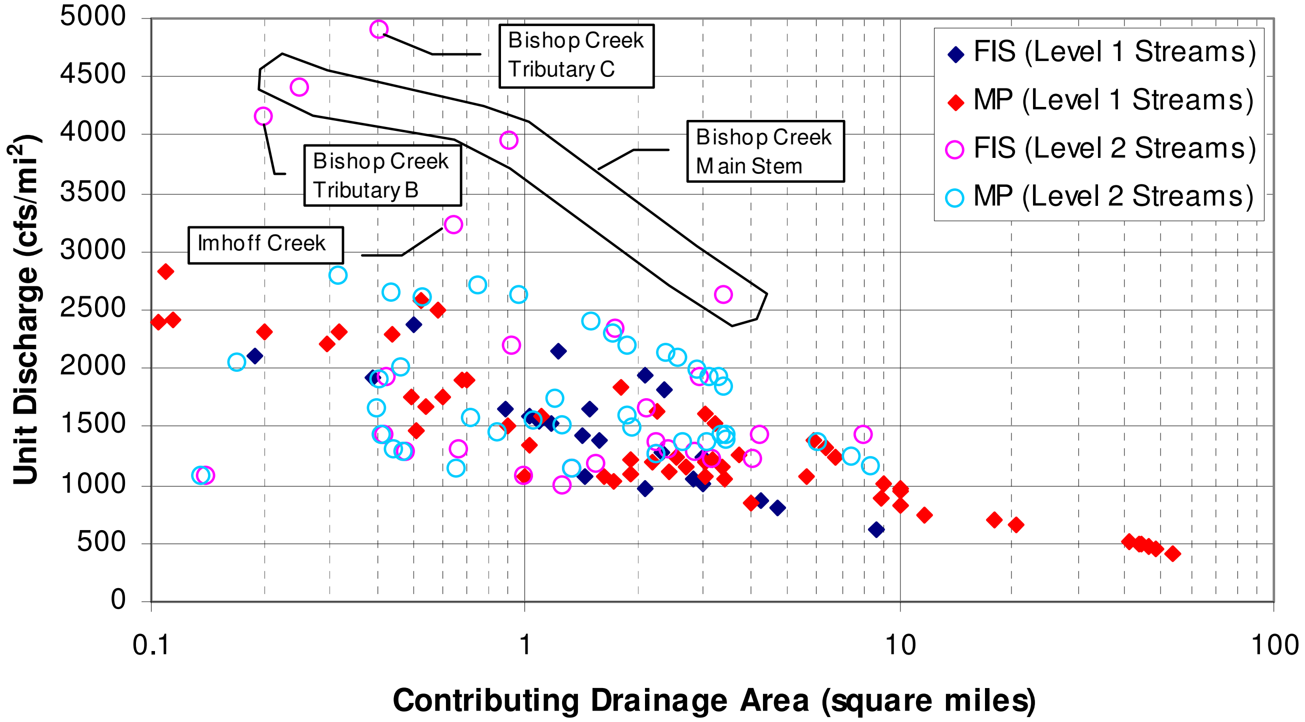
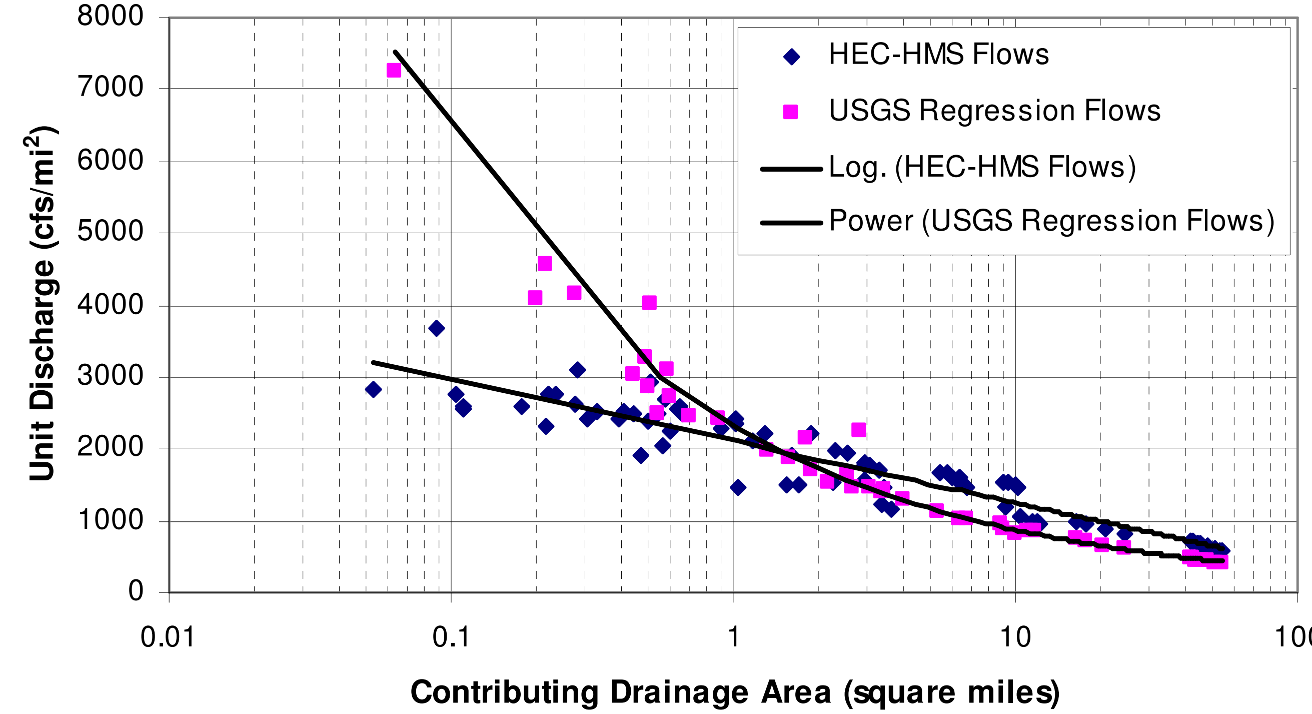


Figure 4-6: Comparison of Unit Discharges between Level 1 Models and USGS Regression Equations



the Imhoff Creek hydrology modeling in this section and the memorandum included in Appendix F). The Bishop Creek outliers are exceptionally high when compared to the bulk of the modeling results. The Bishop Creek model used in the master plan produced significantly lower unit discharges for comparable areas. The discrepancy may be the result of a difference between the models used for the master plan and FIS (the flows reported in the 2008 FIS report match those in the 1999 FIS report). The full documentation of the Bishop Creek model was not available for consideration as part of this master plan. However, the version of the Bishop Creek model used for the master plan produced flows that were more reasonable in comparison to similar watersheds in the urbanized portions of the City of Norman.

The flows generated with the detailed hydrologic models also were compared to the Q-grid results derived based on the USGS regression equations. The USGS regression equations for Oklahoma tend to produce higher flows (considerably higher than HEC-HMS) for smaller areas and lower flows for larger areas. For small areas (<0.5 square mile) the USGS flows tend to be conservatively high. Simplifying the comparison to two curves (unit discharge versus area), the USGS curve rises much more quickly than the HMS curve and produces much higher flows for small areas. The USGS curve then tends to flatten out more quickly than the HMS curve. These unit discharge curves for the HEC-HMS models and USGS regression equations are shown in Figure 4-6. As shown in the figure, the two curves tend to cross in the 1- to 3-square-mile range.

4.2 HYDRAULIC ANALYSIS

Hydraulic modeling of the study streams provided the primary basis for the identification or confirmation (areas previously identified by the City) of flooding issues, for the development of flood and erosion control solutions and the identification of floodplain planning corridors. The U.S. Army Corps of Engineers HEC-RAS version 3.1.3 modeling system and the PBS&J RFD were the primary tools used in this analysis. The following sections provide details of the approach and methodologies used in the hydraulic analyses produced for the Level 1, 2, 3 and 4 streams in the study.

4.2.1 Detailed Hydraulic Modeling for Level 1 and 2 Streams

Detailed hydraulic models were developed or adapted from existing models for all Level 1 and 2 streams studied as part of the master plan. New HEC-RAS (version 3.1.3) models were built for the Level 1 watersheds while existing models were updated to HEC-RAS version 3.1.3 and modified as necessary to reflect new information for the Level 2 watersheds. Table 4-10 provides a summary of the hydraulic models used for the master plan and a brief description

Table 4-10
Summary of Hydraulic Models for Levels 1 and 2 Watersheds

Detailed Streams	Study Level	Hard Copy of Model	Hydraulic Model	Program	Year	Company	Purpose	Source	Comments
Ten Mile Flat Creek	2	Y	Y	HEC-RAS	2005	MacArthur	CLOMR	CoN	MacArthur Associated Consultants CLOMR
Bishop Creek	2	N	Y	HEC-RAS	1997			CoN	According to City Staff, the 1997 version of the model is the latest version. This version was used as the base model for the master plan.
Trib A to Bishop Creek	2	N	Y	HEC-RAS	2003			CoN	2003 LOMR (upper) and 2004 LOMR (lower)
Trib B to Bishop Creek	2	N	Y	HEC-RAS	1997			CoN	According to City Staff, the 1997 version of the model is the latest version. This version was used as the base model for the master plan.
Trib C to Bishop Creek	2	N	Y	HEC-RAS	1997			CoN	According to City Staff, the 1997 version of the model is the latest version. This version was used as the base model for the master plan.
Brookhaven Creek	1/2	Y	Y	HEC-2	1993	Clour (1993) Guernsey (2007)	LOMR (1993) Bridge design (2007)	Guernsey	Clour (1993 HEC-2) model based on 1979 FIS with incorporation of LOMCs and correction of stream lengths. Guernsey HEC-RAS model incorporated Clour HEC-2 north of Robinson and extended to Willow Grove.
Trib A to Brookhaven Creek	2	N	N	HEC-RAS					Converted from HEC-2 (Clour) to HEC-RAS (Guernsey), probably without modification. Junctions modeled improperly.
Trib B to Brookhaven Creek	2	N	N	HEC-RAS					Converted from HEC-2 (Clour) to HEC-RAS (Guernsey), probably without modification. Junctions modeled imporperly.
Imhoff Creek	2	Y	Y	HEC-RAS	2000	Baldischwiler	LOMR	CoN	Combined 1997 LOMR (full stream) with 2001 LOMR (Whispering Pines to Lindsey) to produce model. Refer to memo for major updates.
Merkle Creek	1/2	Y	Y	HEC-2	1996	Clour (1994) JWB for Clour (1995) Baldischwiler (1996)	LOMR	CoN	Original 1994 LOMR modified by 1995 LOMR (improvements at 24th Ave SW, Robinson St, CHIMP from 24th to Main, updated topography). 1996 LOMR - Revised 1995 LOMR (CHIMP Main to ~450-ft upstream of Crestmont, new culvert at Crestmont, correction of low chord at Main).
Little River	1			HEC-RAS	2008	PBS&J	Master Plan	New	New modeling based on new topographic data and survey.
Woodcrest Creek	1			HEC-RAS	2008	PBS&J	Master Plan	New	New modeling based on new topographic data and survey.
Tributary G	1			HEC-RAS	2008	PBS&J	Master Plan	New	New modeling based on new topographic data and survey.
Rock Creek	1			HEC-RAS	2008	PBS&J	Master Plan	New	New modeling based on new topographic data and survey.
Dave Blue Creek	1			HEC-RAS	2008	PBS&J	Master Plan	New	New modeling based on new topographic data and survey.
Tributary to Dave Blue Creek	1			HEC-RAS	2008	PBS&J	Master Plan	New	New modeling based on new topographic data and survey.

of their origins and subsequent modifications. The models for these watersheds are discussed in more detail under the individual sections for each watershed.

4.2.1.1 Field Reconnaissance

Field reconnaissance was performed for each of the Level 1 and 2 study streams. This reconnaissance included walking of almost the entire lengths of the urban streams; limited creek walks and visits to key locations in the more rural areas; and photographs of structures, typical channels (for n-value determinations and erosion assessment) and other key features. The notes and photographs from this effort were used to facilitate the modeling of structures and the assignment of n-values in the hydraulic models.

4.2.1.2 Field Survey

Detailed field survey was performed at a number of stream crossings and other key structures for the Level 1 streams. This includes the small segments of Level 1 study at the downstream ends of Merkle Creek, Brookhaven Creek and on Brookhaven Creek Tributary A. Table 4-11 provides a summary of the stream crossings surveyed for each Level 1 study reach.

Table 4-11
Detailed Survey for Level 1 Streams

Level 1 Stream	Number of Surveyed Crossings
Brookhaven Creek (Downstream End)	2
Brookhaven Creek Tributary A	1
Dave Blue Creek	2
Dave Blue Creek Tributary 1	1
Dave Blue Creek Tributary 2	1
Dave Blue Creek Tributary 3	1
Little River	12
Little River Tributary G	5
Merkle Creek (Downstream End)	2
Rock Creek	4
Rock Creek Tributary A	1
Rock Creek Tributary B	1
Rock Creek Tributary C	2
Woodcrest Creek	4

4.2.1.3 Datum Adjustment

The vertical datum used for the elevation information in the models was a key consideration in the study. The vertical datum used in the hydraulic modeling for the City of Norman prior to this master plan and the recent countywide FIS study was the National Geodetic Vertical Datum (NGVD) of 1929. The floodplains defined for the countywide study were adjusted to the North American Vertical Datum (NAVD) of 1988. All new survey data and modeling for the master plan was developed on the NAVD88 datum. In order to ensure consistency between all models, the hydraulic models provided by the City and used as the basis for modeling of the Level 2 streams was adjusted to the NAVD88 datum. This adjustment is relatively easy to make directly in the HEC-RAS model. A conversion factor of 0.369 ft (NGVD29 to NAVD88) was added to all elements in the Level 2 hydraulic models. This correction is consistent with the adjustment made in the countywide FIS study.

4.2.1.4 Determination of Flow Change Locations

The key interaction point between the hydrologic and hydraulic models for a watershed is at the flow change locations selected for the HEC-RAS model. It is at these points that the flows generated by the hydrologic model are input into the hydraulic model. For the existing Level 2 models, these flow changes were checked and general not modified. The flow change locations for Imhoff Creek are the one exception to this. They were found to be overly conservative and were modified to produce a more reasonable representation of the flows in the upper half of the hydraulic model. For the Level 1 models, the flow change locations were determined based on an overlay of the hydraulic model cross sections on the subbasin and stream network delineations for the hydrologic model. In the case of tributary confluences at the mouths of subbasins, the corresponding flow was input at the next downstream cross section (occasionally a section or two upstream if the main stem and tributary near the confluence was modeled with a single, wide cross section). For flow changes that resulted from the inflows of subbasins contributing directly to the modeled stream, the flow change was generally located between one third and one half of the distance along the modeled stream within the subbasin. This location varied depending on the location of the majority of the inflow within the subbasin.

4.2.1.5 Level 1 Streams

Hydraulic models for Level 1 streams were initially developed with the HEC-GeoRAS application and then modified to incorporate structures, ineffective flow areas, blocked obstructions, expansion and contraction coefficients, final roughness coefficients, flow change locations and boundary conditions. The 2007 aerial topographic data for the City of Norman was used to develop the basic geometry of the model cross sections. This information was augmented by survey data at structures and other key locations. The extent of hydraulic modeling for Level 1 Streams is shown in Exhibit 4-1. The existing and future/full buildout condition profiles for the Level 1 hydraulic models are shown in Appendix J.

4.2.1.5.1 Brookhaven Creek (Downstream End)

The segment of the main stem of Brookhaven Creek between Main Street and Willow Grove Drive was restudied as part of the master plan. This section of the existing model (refer to section 4.2.1.6.2 for a discussion of the existing Brookhaven model) did not adequately represent the flooding issues along Brookhaven Creek in this reach. The existing cross sections were relatively narrow and did not properly represent the overflows that are predicted to occur in the larger design events. These cross sections were replaced with new, extended sections based on the 2007 topographic data collected for the City. These new cross sections were extended much farther on both the right and left overbanks (looking downstream) so that overflows from the main channel could be properly represented.

Based on the revised modeling, flooding in this reach was found to occur primarily in the right overbank. Flows begin to escape the channel at Main Street and flow toward the wide, relatively flat area in the right overbank. The left bank at Main Street and for a few hundred feet downstream of Main Street is considerably higher than the right bank, which prevents overflows along the left overbank immediately downstream of Main Street. The bulk of the overflows occur in this area just downstream of Main Street. This water flows to the west, into the lower-lying flat area and then south along a smaller ditch until it intersects the channel that flows along the northern limit of the Canadian floodplain and confluences with Brookhaven Creek just upstream of Willow Grove Drive. The floodplains in this area can be seen on Exhibits 4-2 through 4-4. The spillage into the right overbank reduces the flow and water surface in the main channel sufficiently that the left overbank along the reach is not overtopped.

4.2.1.5.2 Dave Blue Creek

Two streams were modeled in the Dave Blue Creek watershed. The main stem of Dave Blue Creek was modeled from 60th Avenue SE to just downstream of Post Oak Road. The model includes the crossings at 60th Avenue SE, 48th Avenue SE, and Cedar Lane. The crossing at 48th Avenue East was modeled as a multiple opening structure with the flows from the tributary immediately to the north contributing at the multiple opening. Tributary A to Dave Blue Creek was modeled from the confluence with Dave Blue Creek (approximately 1,000 ft upstream of 60th Avenue) to approximately 500 ft upstream of State Highway 9 (SH 9). The tributary model included a single culvert crossing at SH 9.

4.2.1.5.3 Dave Blue Creek – Tributaries

Tributary 1 to Dave Blue Creek, which flows to the east just south of Lindsey Street, was included in the HEC-RAS project for the overall Dave Blue Creek watershed. Tributary 1 was modeled from a point approximately 2,400 ft downstream of 48th Avenue (at the confluence with another tributary that flows west to east on the north side of Lindsey) to a point approximately 3,300 ft upstream of Cedar Lane. The model included a single culvert crossing at 48th Avenue. The model was not extended to the confluence with the main stem and is not directly connected to the main stem and tributary network in the HEC-RAS geometry for the watershed.

4.2.1.5.4 Little River

Little River and its tributaries effectively dominates the northern portion of the City of Norman from the boundary of the Ten Mile Flat Creek to the west to approximately 96th Avenue in the east. The main stem of Little River was modeled in detail from a point approximately 2,400 ft downstream of 48th Avenue NE to approximately 1,900 ft upstream of IH 35. The model included 12 stream crossings, 10 of which were modeled as bridges. Survey data was used to develop the information required to model these structures. The 13.8 mile length of Little River included in the study was modeled with 103 cross sections with an average spacing of just over 700 ft.

4.2.1.5.5 Tributary G to Little River

Tributary G flows from west to east into Little River approximately 2,700 ft upstream of 12th Avenue NW. The Tributary G watershed includes the developing areas along and to the west of the IH 35 corridor. The modeled portion of the stream extends from the confluence with Little River to a point just downstream of 36th Avenue NW. The model included 5 culvert crossings. The BNSF Railroad culvert crossing, which was the downstream-most modeled crossing, exerts a significant backwater impact for a considerable distance upstream. This is discussed in greater detail in Section 5.

4.2.1.5.6 Woodcrest Creek

Woodcrest Creek flows from south to north into the Little River approximately 2,100 ft downstream of Porter Avenue. The modeled portion of the stream extends from the confluence with Little River to a point approximately 2,700 ft upstream of Rock Creek Road. The Woodcrest model included four culvert crossings. The downstream-most of these crossings at an unnamed dirt road is a small, low-water crossing.

4.2.1.5.7 Merkle Creek (Downstream End)

The downstream end of the existing Merkle Creek model was extended from IH 35 to a point approximately 1,700 ft downstream of Lindsey Street. The extension included the culvert crossings at IH 35 and Lindsey Street. Cross sections for this reach were added based on the 2007 topographic data while the culverts were added based on survey data. The full Merkle Creek model is described in greater detail in Section 4.2.1.6.4.

4.2.1.5.8 Rock Creek

The main stem of Rock Creek and four tributary streams were modeling in the Rock Creek watershed. The main stem and tributaries were modeled in a single, networked HEC-RAS geometry file. The main stem model, which consists of five reaches, extended from approximately 1,000 ft downstream of 48th Avenue NE to a point approximately 2,000 ft upstream of the confluence with Tributary A. The upstream limit of study on the main stem is just downstream from a small dam. The main stem model includes three culvert and one bridge crossing.

The four modeled tributaries were spaced out along the length of the main stem. The most upstream tributary, Tributary A, flows into the main stem approximately 1,800 ft downstream of its crossing of Robinson Street. The Tributary A model reach extends from the confluence to just downstream of Robinson Street. There were no stream crossings modeled on Tributary A. The Tributary B model reach extends from the confluence with the main stem approximately 400 ft upstream of 36th Avenue NE to Silverado Way just downstream of a small dam. The reach does not include any stream crossings. The Tributary C model reach extends from the confluence with the main stem approximately 1,100 ft upstream of Robinson Street to the downstream face of Alameda Street (just downstream for a subdivision detention facility). The reach includes a bridge crossing at Ackerman Road and a culvert crossing at 36th Avenue NE. The Tributary D model reach, downstream-most of the four tributaries, extends from the confluence with the main stem approximately between the Robinson Street and 48th Avenue NE crossings on the main stem to the downstream face of Rock Creek Road. The reach includes no modeled stream crossings.

4.2.1.6 Level 2 Streams

The hydraulic models for the Level 2 streams were adapted from existing hydraulic models for the watersheds. A majority of these models, as indicated in Table 4-10, were HEC-RAS models. However, the Brookhaven Creek and Merkle Creek models were HEC-2 models. These HEC-2 models were converted to HEC-RAS with the modeling of structures updated as necessary in order to make the models compatible with and accurate in HEC-RAS. All final Level 2 hydraulic models for the master plan were updated and run with HEC-RAS version 3.1.3. The extent of hydraulic modeling for Level 2 Streams is shown in Exhibit 4-1. The existing and future/full buildout condition profiles for the Level 2 hydraulic models are shown in Appendix J.

4.2.1.6.1 Bishop Creek and Tributaries A, B, and C

The Bishop Creek HEC-RAS model used for the master plan was based on a 1997 HEC-RAS model provided by City Staff. The HEC-RAS models for Tributaries B and C were also derived from the 1997 study. Presumably, these models were developed as part of the Mansur-Daubert-Strella Engineers study that produced the 1996 HEC-1 model for the watershed. However, documentation was not available to confirm this. The report and associated documentation for the Mansur-Daubert-Strella Engineers study was not available for review during the preparation of the master plan. The HEC-RAS model for Tributary A to Bishop Creek was derived from a pair of LOMRs for the stream. A LOMR in 2003 updated the lower portion of the stream, while a 2004 LOMR updated the upper portion of the stream.

The HEC-RAS model for the main stem of Bishop Creek extends from a point approximately 5,700 ft downstream of SH 9 (approximately at the edge of the Canadian River floodplain) to approximately 600 ft upstream of Cockrell Street. The model includes a total of 14 stream crossings, one of which (Constitution) is a multiple opening structure.

The model for Tributary A to Bishop Creek model extends from a point approximately 550 ft downstream of the BNSF railroad crossing to approximately 260 ft upstream of Sinclair Street. The actual confluence with the main stem of Bishop Creek is just downstream of Constitution. However, the Constitution crossing of Tributary A is modeled as part of the multiple-opening structure in the main stem model. The Tributary B model extends from the confluence with the main stem approximately 380 ft downstream of Alameda Street to just downstream of Main Street. The model includes a single stream crossing at Alameda Street. The Tributary C model extends from the confluence with the main stem approximately to just downstream of the BNSF Railroad. The model includes four stream crossings, one of which crosses the series of ponds on the east side of the University of Oklahoma campus just upstream of Lindsey Street.

4.2.1.6.2 Brookhaven Creek

The Brookhaven Creek HEC-RAS model used for the master plan was based on the 2007 HEC-RAS model developed by C.H. Guernsey for the design of the 36th Avenue NW bridge. The Guernsey model was based on the 1993 LOMR model developed by Clour, which was in turn based on the 1979 FIS study with the incorporation of LOMCs and the correction of stream lengths. The Clour model provided information for the portion of Brookhaven Creek upstream of Robinson Street. The Guernsey study extended the model from Robinson Street downstream to Willow Grove Drive. The portion of the Brookhaven Creek main stem downstream of Main Street was restudied as part of the master plan. This segment of the HEC-RAS model was replaced with new cross sections cut from the 2007 topographic data for the City along with new survey data for the two crossings in this reach. Details of this update and the complex nature of the flooding in this area are provided in the Level 1 section above.

The Guernsey and Clour models directly incorporated Tributaries A and B into the main stem model for Brookhaven Creek. The contributing drainage areas for the tributaries and main stem at their respective confluences are comparable, so the assumption of coincident peaking required by their inclusion was not unreasonable. However, the tributaries were incorporated by Guernsey exactly as they were represented in the Clour HEC-2 model. As a result, the HEC-RAS model included a repeated main channel section for each of the tributaries. This resulted in reach lengths and lengths across the two junctions that are not completely accurate and geometry at the downstream end of each tributary that is not fully representative of the tributary stream. This issue should not have a significant impact on the water surface elevations in the stream and, as a result of the desire to directly incorporate the existing models, was not corrected.

The HEC-RAS model for the main stem of Brookhaven Creek extends from just downstream of Willow Grove Drive (effectively to the Canadian River floodplain) to its upstream limit just upstream of Rock Creek Road. The modeled reach for Tributary A to Brookhaven Creek extends from the confluence with the main stem (approximately 460 ft downstream of Pendleton Drive on the tributary) to just upstream of Rock Creek Road. The modeled reach for Tributary B extends from the confluence with the main stem (approximately 940 ft downstream of Rock Creek Road along the main stem) to the downstream face of the south-bound Interstate 35 frontage road. The model includes a total of 7 stream crossings on the main stem, two on Tributary A and none on Tributary B.

4.2.1.6.3 Imhoff Creek

The Imhoff Creek HEC-RAS model used for the master plan was based on a combination of two LOMR models. The 1997 Baldischwiler Engineering Consultants LOMR model, which included the full modeled length of the stream was combined with the 2001 Baldischwiler LOMR model for the portion of the creek between Whispering Pines and Lindsey Street. This truncated model represented the improvements associated with the trapezoidal articulated block channel constructed in this reach.

Once combined, the model was reviewed based on site visit photographs, the new 2007 topographic data and the general modeling procedures used. A number of issues were identified and corrected as a result of this review. A summary of the identified issues is provided below. These issues are more fully documented in the memorandum in Appendix F.

- The downstream boundary condition was switched from a known water surface to normal depth.
- The overbank Manning's roughness coefficients were generally too low in the overbanks and in the natural portions of the main channel and were increased.
- The HEC-1 flow input locations in the HEC-RAS model were overly conservative and were revised.
- The distances and cross section geometries in the vicinity of the school footbridge downstream of Main Street, along with the length of the Main Street culverts and immediately adjacent alley were corrected.
- Forced water surface elevations at cross section 11840 and unnecessary ineffective area settings upstream of Lindsey Street were removed.
- The culvert models were modified so that the model no longer forced the selection of inlet control and the roadway weir coefficients for culverts and bridges was changed from 1.0 to 2.6.

These and other minor changes resulted in a general increase in the water surface elevation along the majority of the modeled length of Imhoff Creek.

In addition to the changes described above, the portion of the model downstream of Imhoff Road was replaced based on the new 2007 topographic data. The SH 9 culvert crossing in this reach was adapted from the original model. The occurrence of flooding during large design events in the subdivision on the left bank (looking downstream) between Imhoff Road and SH 9 necessitated the re-visitation of this portion of the model. The original cross section location and geometries were not adequate to clearly determine the nature and extent of the flooding in this area. These flooding issues and the proposed solutions to address them are discussed in Sections 5 and 6.

4.2.1.6.4 Merkle Creek

The Merkle Creek HEC-RAS model used for the master plan was based on a series of LOMR models, the latest of which was the 1996 LOMR by Baldischwiler Engineering Company. This LOMR model was based on a 1995 LOMR model prepared by JWB engineers for Clour. The JWB model was based in-turn on thee 1994 LOMR model prepared by Clour. The 1995 LOMR modified the original Clour model to include improvements at 24th Avenue SW and Robinson Street, channel improvements in the reach between 24th Avenue SW and Main Street and updated topographic data. The 1996 LOMR included an additional 450 ft of channel modifications upstream of Main Street, a new culvert at Crestmont and correction of the low chord at Main Street. The 1996 LOMR HEC-2 model was converted to HEC-RAS version 3.1.3 for us in the master plan.

The converted model was reviewed based on site visit photos, aerials and the new 2007 topographic data. As a result of this review a couple of minor modifications were made to the model, primarily to facilitate the evaluation of solutions. The modifications included additional cross sections downstream of 24th Avenue SW to better define the shape and extent of the concrete lined channel and upstream of Crestmont Street where the 2007 topography indicated that a hump in the channel represented in the model was not actually present. In addition to these minor modifications, the downstream end of the Merkle Creek model was extended from IH 35 to a point approximately 1,700 ft downstream of Lindsey Street. The downstream extension of the model is described in more detail in Section 4.2.1.5.7. The extended Merkle Creek model included six culvert and 1 bridge crossing.

4.2.1.6.5 Ten Mile Flat Creek

The Ten Mile Flat Creek hydraulic model used for the master plan was adapted from the HEC-RAS model developed for the 2005 MacArthur Associated Consultants study of Ten Mile Flat Creek in support of a CLOMR for the watershed. This model extends from approximately 500 ft downstream of the Main Street crossing to a point approximately 4,900 ft upstream of the Franklin Road crossing. The Ten Mile Flat Creek floodplain is quite wide and flat as its name implies. Much of the area is dominated by the Canadian River floodplain. However, there are wide portions of the Ten Mile Flat Creek floodplain proper and fairly complex overflow situations that occur at various places along the length of the stream. Significant reaches of the stream have been straightened and channelized.

The Ten Mile Flat Creek model includes a main channel with six stream crossings and two overflow/bypass channels with one stream crossing each. The main stem crossings at 60th Avenue NW, Franklin Road, and Tecumseh Road were modeled as multiple openings. The northeast overflow channel roughly parallels the main stem for several hundred feet upstream and downstream of 60th Avenue NW. Prior to the reconstruction and elevation of 60th Avenue NW, this area was modeled with a lateral weir (60th Avenue NW) to pass overflows into the parallel channel. The improvements to 60th Avenue NW eliminate these overflows. The second overflow area occurs at 60th Avenue NW and Tecumseh Road. Flows that do not pass through the structure at 60th Avenue NW continue to flow south and overtop Tecumseh, which was modeled as a lateral weir. These overflows continue south along the west side of 60th Avenue NW until some of the flow overtops 60th and returns to the main channel between Rock Creek Road and Robinson Street. The remainder of the overflow enters the Canadian River floodplain.

4.2.2 Hydraulic Modeling for Level 3 and 4 Streams

The hydraulic modeling for Level 3 and 4 streams was performed with the Rapid Floodplain Delineation (RFD) tool that was initially described in Section 4.1.2.1. The RFD tool provided the ability to rapidly generate floodplains for the over 300 miles of Level 3 and 4 streams with a minimal amount of initial input data.

4.2.2.1 RFD Inputs and Outputs

The RFD tool required the following inputs:

1. A short configure file specifying input parameters.

2. The ground surface as gridfloat raster which was generated from the 2007 topographic data as previously described.
3. A shapefile representing the stream centerline (e.g., hydraulic baseline).
4. A shapefile representing a set of cross-sections, attributed with Manning's *n* values and discharges. For the Level 3 and 4 streams the RFD option to automatically generate the cross-section locations based on the centerline and some parameters specified by the user was employed. The Q-grid also described above was used with the RFD tool to automate the attribution of flows to the generated cross sections.
5. (optional) A shapefile which contains flow limits. These represent obstructions or ineffective flow areas. It is not required that any particular section have flow limits assigned to it.

Given these inputs, the RFD tool performs the following functions:

1. Creates a cross-section shapefile using the hydraulic baseline (if requested in the configuration file).
2. Projects (cuts) the cross-sections onto the topography and creates a hydraulic model.
3. Calculates water surface elevations by using a backwater analysis.
4. Creates a shapefile of the cross-sections with the calculated water surface elevations and other hydraulic parameters in the attribute table.
5. Creates elevation plots of the cross-sections as pages of a PDF.
6. Creates profile plots of the cross-sections as pages of a PDF.
7. Delineates a floodplain polygon as a shapefile which can be viewed using GIS software.

All these steps are performed sequentially in batch mode, without user intervention.

4.2.2.2 RFD Processing

For each generated cross-section, RFD extracts the raster elevations along the surface and generates station – elevation points for each cross-section. The downstream distance in feet to the next cross-section is obtained by calculating the difference in station between each cross-section and the next lower section.

In addition to cross sections, RFD allowed for the identification of stream crossings based on the intersection of the hydraulic base line and a road layer. An energy grade line drop of 1 ft was specified between bounding cross sections at each of these crossings in order to represent the impacts of structures on the water surface profile.

Once RFD has developed the required cross section information and associated the flows from the Q-grid, it performs a backwater calculation to determine the water surface elevations. RFD uses the 1-D energy equation to compute the water surface elevations. This is done using an iterative procedure, where the upstream water surface elevation is assumed, and the error in the energy equation is the calculated, then the water surface is refined until the error in the energy is reduced below a certain tolerance.

RFD uses the cross sections and associated water surface elevations in concert with the underlying topographic grids to calculate a depth grid for the stream. All values that have a positive depth are assigned the value 1 (inundated) while all other cells are assigned the value 0. This results in a "pixilated" grid representing a crude floodplain. RFD at this point interpolates a floodplain boundary between each pair of cells along the gridded floodplain boundary. This is essentially similar to a contouring algorithm.

The results have been compared to HEC-RAS + HEC-GeoRAS results both in terms of delineated floodplain and water surface elevation. The water surface elevations are generally within 0.1 ft of the HEC-RAS water surface elevations.

4.2.2.3 RFD Application for Level 3 and 4 Streams

The RDF tool was applied for each of the Level 3 and 4 streams identified in the City of Norman. A stream centerline or hydraulic baseline was established for each and the basic parameters for application of the RFD tool were set. The RFD tool was then run and the steps described above followed for each stream. The resultant cross sections and floodplain polygons were reviewed for reasonableness and then the floodplains were combined to form a single stream planning corridor layer for the City. The stream planning corridor floodplains are shown on Exhibit 4-4.

4.2.3 Hydraulics for Local Drainage Issues

Several local drainage issues identified by City staff of citizen complaint were investigated as part of the master plan. In cases where a detailed hydraulic model was not available for such an area, alternative methodologies were used for the evaluation and recommendation of solutions. Undersized roadway crossings in these areas were evaluated and resized with the Haestad Culvert Master application. Flows for these analyses were derived from either the detailed hydrologic model when available or from the Q-grid developed for the RFD when detailed hydrologic models were not available.

In the case of issues related to closed systems or requiring such systems in order to address the identified problem, a full flow analysis of the system capacity was used. An example of such an analysis is the sizing of the diversion system to carry ponded flood water from the Lindsey and McGee area to the ditch along the IH 35 right-of-way. Flows for such analyses were derived from detailed hydrologic models where available and from the Rational method when otherwise necessary.

4.3 HYDROLOGIC AND HYDRAULIC MODELING FOR SOLUTIONS

The hydrologic and hydraulic models developed or adapted for the study watersheds as described in the preceding sections were the primary tools used for the evaluation of structural solutions for identified flooding issues. They were also used to define the parameters and constraints used in the development of solutions for erosion control and channel restoration. The specific problem areas identified and the solutions evaluated to address them are described in detail in Sections 5 and 6. The following subsections describe the general approach and methodologies used in the design and evaluation of the proposed solutions and the additional or specialized analyses required for specific solutions.

The proposed solutions were evaluated in a two-step process. A proposed solution would first be evaluated in isolation when possible. For some watersheds and study streams such as Imhoff Creek, it was not practical to evaluate all potential solutions by themselves due to the density of flooding issues. Once a potential solution was developed for the isolated issues along a study stream, they were combined so that their interactions could be evaluated. In some cases, such interactions necessitated revisions to the evaluated solutions. In many cases, a downstream improvement was first required in order to achieve the full desired benefit for an upstream solution impacted by backwater from the downstream issue. In the case of detention, it was necessary to evaluate the impacts of the proposed pond on other downstream solutions. The locations of the proposed solutions and the associated floodplain modifications for each study stream are shown on Exhibits 6-1 through 6-19. These exhibits also include profile views to show the extent and impact of the improvements.

4.3.1 Hydrologic Modeling General Approach

The hydrologic modeling associated with the development of solutions focused on the evaluation of detention facilities and consideration of flow diversions. The hydrologic evaluation of potential detention solutions ranged from the relatively straight-forward adaptation of existing detention plans for the pond on Merkle Creek upstream of Robinson Street to a complex analysis of interconnected pond in the area of Andrews Park in the upper Imhoff Creek watershed. The general approach to such detention analysis proceeded as outlined below:

1. Flooding issues were identified within the subject watershed.
2. Properties with sufficient open space for detention facilities were identified.
3. The identified properties were evaluated to determine whether they could realistically be considered for purchase.
4. Potential detention areas were maximized for properties identified for further consideration.
5. Reasonable assumptions for the layout, depth, side slopes, and inflow and outflow structure locations and types.
6. Elevation-storage curves were developed for the evaluated facilities based on the assumptions made for the layout of the pond.
7. Inflow rating curves were developed in the case of facilities not directly in-line with their contributing storm drains.
8. Parameters for the ponds were entered into HEC-HMS and initial outlet structures were assumed.
9. Outflow structures (typically an orifice for low flow and weir for high flows) were optimized to maximize the potential flood control benefits.
10. The revised flows based on the presence of the detention facility were entered into the associated hydraulic model to evaluate the potential downstream benefits.

A majority of the detention facilities evaluated for the master plan were located in or near the headwaters of the study stream or portion of the watershed related to a specific local problem area. Such facilities were typically in-line ponds that directly accepted flow from a contributing drainage channel rather than off-line facilities connected to the drainage channel via a side weir along the bank of the channel. The detention facilities considered for the Imhoff Creek watershed are the most notable exception to the general procedure outlined above. The analyses for these facilities will be discussed in greater detail in the watershed-specific discussions below.

The consideration of diversion channels or closed systems was a more straight-forward process than that used for evaluation of detention options. The diversion systems were sized to either carry the maximum possible flow given the constraints on the potential system or a target flow based on the conditions in the channel from which the flow was to be diverted. A diversion rating curve was then developed based on the characteristics of the preliminary conceptual design for the diversion. The size of the diversion system and the characteristics of the diversion rating curve were then optimized through HEC-HMS runs to achieve the target flow or the maximum potential benefit.

4.3.2 Hydraulic Modeling General Approach

Solutions evaluated with the hydraulic models developed or adapted for the master plan were generally of three types. The most common was the evaluation and sizing of enlarged stream crossings. The second was the evaluation of enlarged channel sections to provide additional flow capacity. The third and simplest was the evaluation of the impacts of the reduced flows provided by the various detention options.

As discussed subsequently in Section 5, each study watershed has at least one existing undersized stream crossing. These inundated crossings were first identified with the hydraulic models and then resized to accommodate the requisite design flows per the City's drainage criteria (culvert to pass the 50-year flow and bridges to pass the 100-year flow with 1 ft of freeboard). In some cases it was not possible or practical to achieve the full criteria requirements. In these cases, culvert designs were typically reduced to the 10-year event and bridge designs reduced to the 50-year event. This is especially true for Imhoff Creek where the density of stream crossings and limited width available for improvements limits the target design event. Where possible, crossings were first resized in isolation from other improvements and then integrated to evaluate the interactions of improvements and to optimize the designs. In many cases, downstream improvements were necessary in order to be able to achieve the design goals for an upstream structure.

A majority of the structures for which solutions were proposed were culverts. When practical, the proposed solution preserved the existing barrels and added one or more parallel barrels. In many cases this was not possible and the entire structure was proposed to be replaced. The initial culvert sizing for proposed solutions was based on the required capacity to pass the design flow. The design was then optimized to allow for downstream backwater or other conditions and to achieve the target design criteria. In some cases, bridges were proposed to replace culverts. These proposed bridges and other pure bridge enlargements were evaluated with an approach similar to that applied for culverts.

Channel modifications were proposed in a number of areas to both reduce flooding directly and to improve downstream backwater conditions so that reasonable designs for stream crossings could be achieved. The proposed channel modifications generally used a typical section that provided a more natural channel appearance than existing channel modifications. In some cases, such as along Imhoff Creek, these natural channel sections would be more difficult to achieve but remain an alternative to consider during preliminary design engineering for such improvements. However, retaining the WPA channel type appears to be the preferred design choice given the space limitations and the historic nature of this design type. The design considerations for such channel modifications are discussed in greater detail in Section 6.

Channel modifications were optimized to the extent practical in order to minimize the required improvement footprint. The modifications were typically developed in HEC-RAS with the channel modification option. An initial improvement layout was determined based on the availability of right-of-way and an estimate of the required capacity. Iterations of the improvement size were then made until the design criteria were achieved. In the case of limited channel modifications required downstream of structures, the modifications to the channel geometry were made directly for the impacted cross sections.

4.3.3 Specific Modeling Considerations for Study Watersheds

Alternative analyses beyond the general methodologies outlined above were required in order to evaluate certain solutions. There were also special considerations involved in the standard approach for specific solutions. Such exceptions and special considerations are described in the following subsections. The discussions are organized by major study watershed.

4.3.3.1 Imhoff Creek

The solutions developed for the Imhoff Creek watershed required the most extensive analyses of any of the study watersheds. The density of stream crossings and associated flooding issues effectively required the proposed improvements for a majority of the stream to be evaluated as a single, interconnected solution. This solution integrated detention in and near Andrews Park, extensive channel modifications from downstream of Lindsey Street to James Garner Avenue and diversion of flows from the vicinity of the Lindsey and McGee intersection in the west central portion of the Imhoff Creek watershed. Once the comprehensive solution was developed, it was divided into logical, smaller increments as described in Section 6.

4.3.3.1.1 Detention Analysis

The most complex analysis was associated with the conceptual design for detention at Andrews Park. The final configuration of the Andrews Park detention and the alternative detention upstream of Acres Street was modeled in HEC-1. However, the configuration was optimized based on a Surface Water Management Model (SWMM) developed specifically for the design of the detention facility and associated flow diversions. The EPA SWMM 5 model was used to perform this analysis. The SWMM model was constructed with three storage areas for the base solution. These consisted of detention in Andrews Park proper (2 storage areas) and in the triangular area bounded by Webster Avenue, Park Avenue and Imhoff Creek. A fourth storage area was added for the simulation of the proposed detention upstream of Acres Street.

The use of the SWMM model allowed for direct representations of the inflow diversions, connections between ponds and outflow structures. The primary inflow to the facility was a diversion from the Imhoff Creek channel near the intersection of Beal Road and Jones Avenue. Three reinforced concrete pipes carried flow from the Imhoff channel, under the railroad and James Garner Avenue to flow into in the Andrews Park detention facility. This inflow occurred at the location of the existing concrete water tank, which would be removed as part of the proposed improvement. Inflows also entered the facility from the drainage area and existing channel to the north of Andrews Park. The portion of the park to the east of the existing drainage channel (including the removed concrete tank) was simulated a one storage area with a weir connecting this storage area to the primary portion of the Andrews Park detention to the west of the existing channel. Low flows into the first storage area were allowed to pass through an outlet structure along the alignment of the existing channel. High flow passed over the weir into the primary detention area. From this area, flows passed through a reinforced concrete pipe outfall into the triangle area detention component. From the triangle, flow discharge directly to the main channel through either a reinforced concrete pipe or via an overflow weir.

Once the geometry for the detention facility was established, the inlet and outlet structures were optimized to provide significant flow reductions while not overtopping the facility. The flows used to drive the SWMM model were adapted from the hydrographs produced by the HEC-1 model for the watershed. The output from the SWMM model was used to develop diversion and outflow rating curves for the facility that could be used in the HEC-1 model. The final geometry and rating curves were then entered into the HEC-1 model for the generation of the reduced flows for the solution. These reduced flows were then considered in the designs for the downstream channel and stream crossing improvements.

4.3.3.1.2 Channel and Stream Crossing Improvements

Much of the Imhoff Creek channel, especially in the Work Projects Administration (WPA)-constructed reaches, is considerably undersized. The rectangular channel either constructed by the WPA or constructed to roughly match the WPA channel begins just downstream of Boyd Street and extends to the upstream limit of the study at the BNSF Railroad crossing. The existing concrete v-shaped channel between the upstream end of the articulated block lining (approximately 1,250 ft downstream of Lindsey Street) and the beginning of the WPA-style channel is undersized to a lesser degree. The flooding issues at the lower end of this reach (downstream of Lindsey) are in large part caused by the constricted overbanks in the area. The stream crossings in both reaches are correspondingly undersized for the required design events. There are a total of 21 stream crossing of Imhoff Creek. Enlarged solution designs were developed for 16 of these crossings. The channel and crossing solutions were developed under the assumption of reduced flows as a result of the proposed Andrews Park detention (without the portion above Acres Street). The solutions were then checked against unreduced flows. In order to fully accommodate the unreduced flows and still achieve the design targets, additional enlargement and optimization would be required for some of the crossings and stream reaches. The various flooding issues and solutions related to the channel and crossings are described in greater detail in sections 5 and 6.

An iterative approach was used to develop the flood control solutions for the channel reaches and associated crossings described above. The target design solution, as agreed upon with the City, focused primarily on the 10-year event for a majority of the channel and crossings. Exceptions to this were the solutions for the Lindsey Street and Main Street crossings, which were sized to accommodate the 100-year event, and the Boyd Street crossing, which was sized to accommodate the 50-year event. The first step in the modeling of the proposed solutions was to establish an upper limit for the sizes of the stream crossings. These were determined based on the assumption of full flow through a corresponding set of box culverts. Culvert and bridge crossings are typically more efficient than a simple assumption of full flow in a culvert, so the final solutions tended to be considerably smaller than these maximums. The maximum opening sizes also provided an estimate of the maximum channel width that could be required from a hydraulic perspective. The available land along the length of the channel to be improved and the desire to minimize the required width resulted in channels that were considerably smaller than the maximum sizes.

Once the maximum potential sizes were determined, an initial approximation of the required width of the channel modifications and the width (bridge) or number of barrels (culverts) was made. The width assumed for the channel and bridge modifications was increased with each significant addition of flow from the hydrologic model. The cut widths and the variables required to properly align the channel cuts were developed in a spreadsheet and then transferred to the HEC-RAS channel modification table in order to develop a revised set of channel geometry. Culverts and bridges were then sized to match the modified channel. This process was repeated several times until a channel modification solution that met the 10-year design target was generally achieved. The culverts and bridges and associated segments of channel were then optimized to meet higher design goals specified for Lindsey, Boyd, and Main streets.

4.3.3.1.3 West Central Imhoff Creek Watershed Improvements (Lindsey and McGee Diversion)

The flooding issues in and around the Lindsey and McGee intersection and the area between this intersection along Lindsey Street to Imhoff Creek are well known and have been documented and evaluated through a number of studies. The solutions developed for the master plan considered some new approaches that either built on previous studies or introduced new solution concepts. The evaluated solutions included detention and associated flow diversions in selected locations and a diversion directly to the Canadian River. The details of the issues and proposed solutions are discussed in sections 5 and 6.

The primary solution proposed in this master plan for the flooding in the area of the Lindsey and McGee intersection is a large diversion system and associated storm drainage improvements that would carry the bulk of the flow at the intersection directly to the Canadian River. The closed-system diversion would run west along Lindsey Street to Murphy, south along Murphy to Briggs, west along Briggs to the IH 35 right-of-way and outfall to the IH 35 drainage ditch near the junction of the SH 9 ramp with the north-bound IH 35 main lanes. The solutions also included enlargements of the roadside ditch and an additional culvert under SH 9. In addition to the diversion, a modified version of the Phase C storm drainage system described by Baldischwiler (1997) was proposed to carry the flows from north of Lindsey Street and east of McGee Drive to Imhoff Creek.

In order to model the solutions, it was necessary to modify the HEC-1 model so that flows to the various portions of the proposed system could be directly considered. Subbasin I-10A in the original HEC-1 model was subdivided into four smaller subbasins. Three of these subbasins (roughly the portion of I-10A west of Wylie Road) drained to the diversion system while the fourth drained to the proposed Wylie/Lindsey system improvements that drain directly to Imhoff Creek at Lindsey Street. The subdivision of subbasin I-10A also was configured to allow for consideration of detention at Whittier Middle School. Detention at the school was modeled in the HEC-1 model based on a rough determination of the available volume. However, this option was not recommended in the final set of solutions. The proposed diversion was modeled in the HEC-1 model and the resultant decreases in the Imhoff Creek main channel flows were evaluated in the development of solutions for the channel.

The proposed sizes for the diversion system were based on the consideration of full flow in the proposed box culverts for a 10-year flood event. The allowable slopes for the various segments of the proposed system were effectively set by the elevation at the outfall of the system, which allowed for a maximum 0.3% composite slope for the total length of the system. The sizes for the proposed systems along McGee Drive north of Lindsey Street (draining to the diversion) and along Wylie Road and Lindsey Street (draining to Imhoff Creek) were initially based on the sizes proposed in the Baldischwiler (1997) report. The sizes were then checked based on the HEC-1 flows and modified as necessary.

4.3.3.2 Merkle Creek

The hydrologic and hydraulic modeling of the solutions for Merkle Creek included the modeling of a large detention facility currently under construction upstream of Robinson and a set of interdependent channel and crossing modifications. A large detention facility has recently been constructed in the area between the airport and Robinson Street in the upper portion of the Merkle Creek watershed. Since the pond was not complete at the time of the master plan, it was modeled under the solutions rather than as part of the existing conditions. This pond significantly enlarges the existing pond at the site and takes in considerable additional area adjacent to Robinson Street. The two small ponds adjacent to the airport were not modified. The existing pond and two associated routing reaches in the HEC-1 model were replaced with the new enlarged pond. The storage-area-elevation curves and outlet structure parameters for the pond were obtained from the PondPack modeling used in the design of the facility (SMC Consulting Engineers, 2006). The impact of the pond is discussed in greater detail in Section 6.

The other complication to the solution model for Merkle Creek was the interdependency of the flood control solutions for the Main Street, Crestmont Street, and Iowa Street crossings. Both the Crestmont Street and Iowa Street culvert crossings were heavily impacted by the backwater conditions caused by Main Street. Without the proposed Main Street improvements, the Crestmont and Iowa improvements were found to be cost prohibitive if not completely unfeasible. In addition to the Main Street improvements, channel modifications between Main Street and Crestmont

Street were also required in order to develop a reasonable solution at Crestmont. These improvements were modeled both individually and together with the HEC-RAS model in order to develop the final solution recommendations.

4.4 FLOODPLAIN MAPPING

Floodplain mapping was an essential component for the identification of flooding issues and the quantification of the benefits provided by the various solutions proposed in this master plan. For Level 1 and 2 streams the 100-year and 500-year floodplains were delineated for existing conditions while the 10-year and 100-year floodplains were delineated for the future or full-buildout conditions. These floodplains are shown in Exhibits 4-2 and 4-3. The stream planning corridor floodplains developed with the RFD tool for Level 3 and 4 streams are shown in Exhibit 4-4. The floodplain modifications produced by the various proposed solutions are shown on Exhibits 6-1 through 6-19. The following sections describe the procedures used to map the floodplains for the various study streams.

4.4.1 Level 1 Streams

HEC-GeoRAS was the primary tool used to delineate the floodplains for the Level 1 and 2 streams. HEC-GeoRAS allowed for the direct import of the modeling results from the fully geo-referenced Level 1 streams and automated the subsequent generation of floodplains based on the modeled water surface elevations. The floodplains generated by HEC-GeoRAS were smoothed to eliminate the stair-stepped floodplain boundary created by the use of a grid-based elevation dataset. The floodplains were then revised manually to reduce small “islands” inside (dry) and outside (inundated) of the primary floodplain boundary. Any areas cut-off by the bounding polygon generated by HEC-GeoRAS from the cross section extents were fixed and any extraneous artifact “appendages” to the floodplain were removed.

4.4.2 Level 2 Streams

A more manual process was required to generate the floodplains for the Level 2 streams. Geo-referenced cross sections were not readily available for the existing models for these streams. Work maps showing the cross section locations were available for the Ten Mile Flat Creek watershed, but not for any of the other Level 2 watersheds. The lettered cross sections from the FEMA data layers were the only know cross section locations for these streams. These cross sections, augmented by additional cross sections added upstream and downstream of structures and at other key, identifiable locations were used as the base layer for delineation of the Level 2 floodplains.

Once the cross section layer was established for a Level 2 stream, the floodplain delineation process was essentially a manual version of the process employed by HEC-GeoRAS. The simulated water surface elevations were added to the cross section layer as attribute fields. A script linking a series of ArcGIS tools was then used to develop a water surface TIN, convert the TIN to a grid, intersect the water surface grid with the topography grid, determine inundated grid cells, generate floodplain polygons and smooth the resultant floodplain polygons. Once the raw floodplain polygon was generated, the same procedure employed for the Level 1 streams was used to clean the floodplains.

4.4.3 Level 3 and 4 Streams

The floodplain mapping for Level 3 and 4 streams was performed with the RFD tool as described in Section 4.2.2.

5.0 STORM WATER PROBLEMS

A key component in completing the SWMP was the identification of storm water related problems within the City. Similar to municipalities throughout the country, Norman is experiencing a variety of challenges and problems associated with storm water that is generated within it jurisdictional limits in addition to storm water it receives from neighboring cities and unincorporated areas. For this City-wide undertaking, these problems are generally grouped into stream flooding, stream erosion, water quality, and local drainage to assist in understanding and evaluating their respective nature. A few of the problems or problem areas have more than one characterization type. For instance, there are some problem areas that have flooding as well as stream erosion issues. The identification of problems was primarily accomplished by a variety of means including reviewing and evaluating items such as: the City's GIS data, past water quality studies, past hydrologic and hydraulic modeling, as well as other information and data collected (Section 2); watershed assessments including field reconnaissance trips (Section 3); hydrologic, hydraulic, and floodplain mapping efforts (Section 4); the City staff knowledge of past problems; input obtained from the various committees and the SWMP Task Force; and input received from the general public as provided through the City staff.

A watershed-specific approach in identifying problems was followed as the nature of storm water problems relate directly to the characteristics and activities occurring, or expected to occur, in the watersheds in which the problems are located. As discussed subsequently in Section 6, solutions were developed considering that the potential exists to positively or negatively affect other locations within that respective watershed. In order to focus on the more critical areas and respect budget limitations, the level of study and analysis varied throughout the City as discussed in Section 2. To recap previous discussions herein, storm water analyses were analyzed in more detail for Level 1 and Level 2 stream reaches in comparison to Level 3 and Level 4 reaches. Further, differing study levels within watersheds focused efforts and study detail on those areas experiencing, or expected to experience, the worst problems.

The identification and evaluation of problems were performed for existing as well as future watershed conditions. Although existing conditions were reviewed and considered, the identification and evaluation of flooding along major streams primarily focused on future watershed conditions that reflect the City's 2025 Plan. The identification of stream erosion problems was primarily based on existing conditions consistent with the watershed assessments.

Due to their "non-point source" nature, water quality problems were evaluated on a citywide scale similar to what has been done in many similar studies conducted throughout the country. The extent of water quality problems focused on urbanized areas with some distinction being made between the areas that drain directly into the Canadian River versus those that drain into Lake Thunderbird, the City's drinking water source.

5.1 SUMMARY OF PROBLEMS

Fifty-nine flood-related and stream erosion problems were identified within the City from the many investigations and evaluations performed. The problem locations are spread over a large part of the City but all are located along, or west of, 48th Avenue East. Each problem (and matching solution), also referred to as a "project" at times, has been given an identification number such as "IC-1," which is a specific problem in the Imhoff Creek watershed. Again, the identification numbers, location and nature of these problems coincide with the matching solutions presented in various watershed/stream-specific exhibits in Section 6. As discussed above, water quality problems are dispersed throughout the City, including the urban core area as well as the area that drains into Lake Thunderbird. Due to the nature of the water quality problems, as defined by federal and state regulations, individual problems or problem locations were not identified other than the City as a whole with a focus on urbanized areas.

Of the 59 problems or problem areas identified, 34 (58%) have an element related to stream flooding (structures and/or roadway crossings) along Level 1 and 2 streams, 14 (24%) involve stream erosion along Level 1 and 2 streams, and 12 (20%) are local drainage problems. One of the problems (BHC-1) has a flood related as well as a stream erosion aspect. Of the 34 flood related problems on Level 1 and 2 streams, 26 involve structure or building flooding, and 28 include road crossings that are flooded (overtopped by flood waters). Most problems occur on property with insufficient or no drainage easements or rights-of-way. Some of the problem areas cover an extended length of stream while others affect a relatively short stream reach.

Table 5-1 provides a citywide overview of problem types and locations by watershed. As anticipated, this information documents that almost 84% of the problems occur in the urbanized watersheds that include Bishop Creek, Brookhaven Creek, Imhoff Creek, Merkle Creek, and Woodcrest Creek. In addition to the discussions in this section, the flood prone nature of the Level 1 and 2 study reaches and the City in general is presented with the 100- and 500-year existing condition floodplains provided in Exhibit 4-2, the 10- and 100-year baseline (future conditions) floodplains presented in Exhibits 4-3 and 4-4, and the various plan view and stream flood profile exhibits in Section 6.

Table 5-1
Number of Watershed-Specific Problem Locations
Experiencing Respective Problem Types*

Watershed	Structures Flooded	Road Crossings Overtopped	Stream Erosion	Local Drainage	Totals
Bishop Creek	6	4	6	5	21
Brookhaven Creek	1	4	4	3	12
Canadian River Area	0	0	0	1	1
Clear Creek	0	0	0	1	1
Dave Blue Creek	0	2	0	0	2
Imhoff Creek	9	9	2	1	21
Little River Mainstem	1	0	1	0	2
Little River – Trib G	0	1	0	0	1
Little River – Woodcrest Creek	3	2	1	0	6
Merkle Creek	4	3	0	0	7
Rock Creek	2	3	0	0	5
Ten Mile Flat	0	0	0	1	1
Total Problem Types	26	28	14	12	80

*Several problem locations have multiple problem types.

Table 5-2 provides key watershed-specific information related to the respective problems such as their type and description, and as applicable, basic information related to flooding and/or stream erosion such as the number of structures in the baseline (future conditions) 100-year floodplain, the number of road crossings that are overtopped with floodwaters, and the length of stream erosion problems identified. By concurrently reviewing Table 5-2 and the exhibits in Section 6 that locate associated improvements, the baseline 100-year floodplain, and/or show flood profiles relative to road crossing elevations, an understanding of each problem can be gained. The watershed plan view and/or stream profiles exhibits in Section 6 show the location and extent of the problems as the problems mirror the recommended solutions shown therein. Discussion beyond that provided in Table 5-2 is provided below for some of the more significant problems throughout the City organized by the watersheds in which the problems exist. *Again, the stream flooding and stream erosion problem areas identified are only for the Level 1 and Level 2 stream reaches studied. Localized problems are problems identified throughout the watersheds beyond the Level 1 and 2 reaches as identified by the City.*

Bishop Creek

As shown in Table 5-2 and Exhibits 6-1a, 6-1b, 6-2a, and 6-2b (Bishop flood profile), Bishop Creek has a greater number of individual problem areas than any other watershed in the City with 17 that represent all of the various problem types. One reason is that the Bishop Creek watershed, at 9.87 square miles, is the largest of the urban core watersheds and much of the watershed has been developed for a relatively long time. There are all types of problems with many being relatively small and scattered throughout the watershed.

Overall in the watershed, there are 69 buildings/structures in the baseline floodplain, five flood prone road crossing structures (some may also be a localized problem), and 1,350 ft of eroding stream length. Of the 17 problems identified, six have flooded structures, five have one or more flooded roadways, six result from stream erosion, and five are localized drainage problems. Only four of the 17 problems occur along the mainstem of Bishop Creek with the others being located in Tributaries A and C as well as in various localized areas. The most significant problem along the mainstem is a stream flooding problem, BC-4, in which 49 homes are located in the baseline (100-year) floodplain but these homes also flood from more frequent events such as the 10-year event. In this upper reach, Bishop Creek consists of a small mortared rock channel built during the WPA program about 70 years ago. The capacity of this WPA channel is woefully inadequate which results in the flooding problem.

Tributary A has six problems with the most prominent one being the BC-10 problem where seven homes are in the baseline floodplain upstream of the road crossings at Sinclair Drive and Beaumont Drive. Many of these homes will flood during more frequent events as the capacity of Tributary A is significantly undersized and homes have been built near the creek. Additionally, the culverts beneath the Sinclair Road and Beaumont creek crossings are significantly undersized and are flood prone. A significant problem in Tributary C is the BC-12 problem where the undersized Brooks Street culvert system causes several apartments buildings to be in the baseline (100-year) floodplain upstream of the roadway.

WPA Channel downstream of Carter Avenue – Bishop Creek

Stream erosion caused by the increased flow volumes attributable to urbanization is also occurring in individual short reaches of the mainstem such as described for BC-1, BC-2, BC-5, BC-7, BC-9, and BC-11. Until stabilized, these stream erosion problems collectively totaling 1,350 ft will very likely worsen until the stream reaches stabilize themselves.

Eroding stream upstream of SH 9 – Bishop Creek

Table 5-2
Summary of Storm Water Problems

Project ID	Watershed	Stream	Problem Type*	Problem	100-Year Floodplain Structures In	Stream Length Eroded (ft)
BC-1	Bishop Creek	Bishop Creek	SE	400 LF of bank erosion located approximately 400 LF upstream of SH 9. 300 LF of the bank erosion is on the left bank of the creek and gets close to an existing parking lot. 100 LF of the bank erosion is on the right bank.	---	400
BC-2	Bishop Creek	Bishop Creek	SE	200 LF of severe bank erosion downstream of the confluence of Tributary C and the mainstem. The bank erosion occurs on the left side of the stream.	---	200
BC-3	Bishop Creek	Bishop Creek	FR/FS	50-year and 100-year future flows are overtopping the existing three 8-x-4-ft RCB system at Alameda Street. Structures upstream of Alameda Street are in the future 100-year floodplain.	2	---
BC-4	Bishop Creek	Bishop Creek	FS	Structures are flooded by the 10-year and 100-year future flows between Symmes Street and Main Street.	49	---
BC-5	Bishop Creek	Trib A to Bishop Creek	SE	300 LF of bank erosion located downstream of Constitution Road. There is severe bed and bank erosion located along the left bank downstream of Constitution. The bank erosion along the right bank occurs approximately 150 LF downstream of Constitution Road.	---	300
BC-6	Bishop Creek	Trib A to Bishop Creek	FS	Structures located approximately 450 LF northwest of the intersection of Classen Street and 12th SE Street are in the future 100-year floodplain.	4	---
BC-7	Bishop Creek	Trib A to Bishop Creek	SE	Outfall located along the right bank approximately 175 LF upstream of 12th SE Street has failed due to bank erosion around the headwall.	---	50
BC-8	Bishop Creek	Trib A to Bishop Creek	FR/FS	10-year, 50-year, and 100-year future flows are overtopping the existing two 72-inch CMP structure at Lindsey Street.	1	---
BC-9	Bishop Creek	Trib A to Bishop Creek	SE	200 LF of bank erosion along the right bank located approximately 400 LF upstream of Lindsey Street.	---	200
BC-10	Bishop Creek	Trib A to Bishop Creek	FR/FS	50-year and 100-year future flows are overtopping the existing 10-x-6-ft RCB system at Sinclair Drive and the 8-x-5-ft RCB system at Beaumont Drive. Structures upstream and downstream of Sinclair Drive are in the future 100-year floodplain.	7	---
BC-11	Bishop Creek	Trib C to Bishop Creek	SE	200 LF of severe bank erosion and steep bed slope along the right bank located approximately 75 LF upstream of the confluence between Tributary C and the mainstem. The top of the right bank is close to the maintenance building for a local apartment complex.	---	200
BC-12	Bishop Creek	Trib C to Bishop Creek	FR/FS	10-year, 50-year, and 100-year future flows are overtopping the existing 10-x-4.5-ft RCB system at Brooks Street. Structures located upstream of Brooks Street are located in the future 100-year floodplain.	6	---
BC-13	Bishop Creek	Local	LD	The existing detention pond southeast of 12th Ave SE and Alameda Street intersection is not large enough to detain the existing runoff.	---	---
BC-14	Bishop Creek	Local	LD	Two existing ditches located northwest of Tahoe Street and 24th SE Street currently do not contain the existing flows.	---	---
BC-15	Bishop Creek	Local	LD	The existing ditch between Stinson Road and Fleetwood Road floods frequently.	---	---
BC-16	Bishop Creek	Local	LD	The existing storm sewer system between College Street and Tributary C to Bishop Creek along Lindsey Street is not adequate to handle the 10-year storm event.	---	---
BC-17	Bishop Creek	Local	LD	The existing two 8-x-4-ft RCB structure at Mockingbird Lane is frequently overtopped during rain events.	---	---

Table 5-2, cont'd

Project ID	Watershed	Stream	Problem Type*	Problem	100-Year Floodplain Structures In	Stream Length Eroded (ft)
BHC-1	Brookhaven Creek	Brookhaven Creek	FR/FS/ SE/SC	10-year, 50-year, and 100-year future flows are overtopping the existing two 9.5-x-6.4-ft arch pipes at Main Street. Structures located downstream of Main Street are in the future 100-year floodplain. The existing channel for approximately 2,000 LF downstream of Main Street lacks capacity to contain the future 100-year flows.	276	2,000
BHC-2	Brookhaven Creek	Brookhaven Creek	SE	125 LF of bank erosion on both banks of the channel located approximately 265 LF upstream of Main Street.	---	125
BHC-3	Brookhaven Creek	Brookhaven Creek	SE	225 LF of severe bank erosion along the right bank located approximately 400 LF upstream of Willow Branch Road. Properties located along the right bank are close to the top of bank.	---	225
BHC-4	Brookhaven Creek	Brookhaven Creek	SE	800 LF of channel bank erosion located along both banks just downstream of 36th Avenue NW. Approximately 275 LF downstream of 36th Avenue NW the bank erosion gets close to an existing parking lot.	---	800
BHC-5	Brookhaven Creek	Brookhaven Creek	LD	Channel underneath Robinson Road is constricted due to concrete riprap rubble.	---	---
BHC-6	Brookhaven Creek	Brookhaven Creek	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 60-inch RCP structure at Rock Creek Road.	0	---
BHC-7	Brookhaven Creek	Trib A to Brookhaven Creek	FR	50-year and 100-year future flows are overtopping the existing 10-x-7-ft RCB structure at Pendleton Road.	0	---
BHC-8	Brookhaven Creek	Trib A to Brookhaven Creek	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 72-inch RCP structure at Rock Creek Road.	0	---
BHC-9	Brookhaven Creek	Local	LD	The existing storm sewer system near the Rambling Oaks and Tall Oaks intersection is not adequate.	---	---
BHC-10	Brookhaven Creek	Local	LD	The existing storm sewer system near the Rambling Oaks and Havenbrook intersection is not adequate.	---	---
CC-1	Clear Creek	Local	LD	The existing four 36-inch CMP structure at 120th SE Avenue is frequently overtopped during rain events.	---	---
CR-1	Canadian River	Local	LD	The intersection at Westbrooke Terrace Road and Hollywood Street has deep water after heavy rains.	---	---
DBC-1	Dave Blue Creek	Dave Blue Creek	FR	10-year, 50-year, and 100-year future flows are overtopping the existing two 10-ft CMPs on the mainstem and the 10-ft CMP on the tributary at 48th Ave SE.	0	---
DBC-2	Dave Blue Creek	Trib 1 to Dave Blue Creek	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 54-inch CMP at 48th Ave SE.	0	---
IC-1	Imhoff Creek	Imhoff Creek	SE	800 LF of bank erosion on both banks downstream of SH 9. The erosion along the banks have caused trees to fall into the creek.	---	800
IC-2	Imhoff Creek	Imhoff Creek	SE	4,200 LF of severe bank erosion along both banks beginning at the upstream face of SH 9 to approximately 2,000 LF upstream of Imhoff Rd. The erosion along the banks have caused property fences and trees to fall into the creek.	---	4,200
IC-3A	Imhoff Creek	Imhoff Creek	FR/FS/SC	Reach from Elmwood Drive dead end to Madison Street dead end. The 10-year storm event and larger events are overtopping the existing three 8-x-6-ft RCB culvert system at Lindsey Street. Structures are located within the future 100-year floodplain due to lack of channel capacity.	14	---
IC-3B	Imhoff Creek	Imhoff Creek	FR/FS/SC	Reach from Madison Street dead end to a location approximately 150 LF downstream of W. Boyd Street. Storm events larger than the 10-year event are overtopping the existing 30-x-8.5-ft concrete lined slab bridge at Brooks Street. Structures are located within the future 100-year floodplain due to lack of channel capacity.	32	---

Table 5-2, cont'd

Project ID	Watershed	Stream	Problem Type*	Problem	100-Year Floodplain Structures In	Stream Length Eroded (ft)
IC-3C	Imhoff Creek	Imhoff Creek	FR/FS/SC	Reach from 150 LF downstream of W. Boyd Street to just below McNamee Street. The 10-year storm event and larger events are overtopping the existing 12-x-6-ft slab bridge at Boyd Street and the 12-x-5-ft slab bridge at Pickard Street. Structures are located within the future 100-year floodplain due to lack of channel capacity.	13	---
IC-3D	Imhoff Creek	Imhoff Creek	FR/FS/SC	Reach from just downstream of McNamee Street to just upstream of Symmes Street. The 10-year storm event and larger events are overtopping the existing McNamee Street (12-x-5-ft slab bridge), Flood Avenue (15-x-5-ft slab bridge), and Symmes Street (15-x-5-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	29	---
IC-3E	Imhoff Creek	Imhoff Creek	FR/FS/SC	Reach from just upstream of Symmes Street to just downstream of Main Street. The 10-year storm event and larger events are overtopping the existing school footbridge (10-x-6-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	25	---
IC-3F	Imhoff Creek	Imhoff Creek	FR/SC	Reach from just downstream of Main Street to just upstream of Main Street. The 10-year storm event and larger events are overtopping the existing 12-x-5.5-ft slab bridge at Main Street. Structures located upstream of this reach are within the future 100-year floodplain due to lack of channel capacity.	0	---
IC-3G	Imhoff Creek	Imhoff Creek	FR/FS/SC	Reach from just upstream of Main Street to just upstream of W. Tonhawa Street. The 10-year storm event and larger events are overtopping the existing W. Gray Street (10-x-5-ft slab bridge), N. Lahoma Street (10-x-5.1-ft slab bridge), and W. Tonhawa Street (10-x-5-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	22	---
IC-3H	Imhoff Creek	Imhoff Creek	FR/FS/SC	Reach from just upstream of W. Tonhawa Street to just upstream of N. Webster Avenue. The 10-year storm event and larger events are overtopping the existing W. Daws Street (10-x-4-ft slab bridge), N. University Boulevard (10-x-4-ft slab bridge), N. Park Avenue (10-x-3.5-ft slab bridge), and N. Webster Avenue (10-x-3-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	64	---
IC-4	Imhoff Creek	Imhoff Creek	FR/FS/SC	There are flooded buildings and road structures along the Imhoff Creek stream corridor due to increasing development over the years and lack of channel capacity to contain the flows.	360	---
IC-4A	Imhoff Creek	Imhoff Creek	FR/FS/SC	There are flooded buildings and road structures along the Imhoff Creek stream corridor due to increasing development over the years and lack of channel capacity to contain the flows.	360	---
IC-5	Imhoff Creek	Local	LD	The intersection at Lindsey Street and McGee Drive and Lindsey Street heading East flood after moderate storm events.	---	---
LR-1	Little River	Little River	SE	350 LF of severe bank erosion along the right bank located approximately 2,000 LF upstream of 12th NE Avenue. The bank erosion is approximately 70 LF from a residential structure.	---	350
LR-2	Little River	Little River	FS	There are approximately 40 mobile homes within the future 100-year floodplain located West of the BNSF Railroad and North of Indian Hill Road.	40	---
TGLR-1	Trib. G to Little River	Trib G to Little River	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 10.5-x-7-ft CMP pipe arch culvert system at Franklin Street.	0	---
WC-1A	Woodcrest Creek	Woodcrest Creek	FR/FS/SC	There are flooded buildings and road structures along the Woodcrest Creek stream corridor due to increasing development over the years and lack of channel capacity to contain the flows.	20	---
WC-1B	Woodcrest Creek	Woodcrest Creek	FS/SC	The existing channel downstream of Sequoyah Trail lacks the capacity to contain the future flows. Several buildings along the right side of the stream corridor and one on the left are in the 100-year future floodplain.	10	---
WC-2	Woodcrest Creek	Woodcrest Creek	FR/FS	10-year, 50-year, and 100-year future flows are overtopping the existing two 8-x-7-ft RCBs at Sequoyah Trail.	2	---

Table 5-2, cont'd

Project ID	Watershed	Stream	Problem Type*	Problem	100-Year Floodplain Structures In	Stream Length Eroded (ft)
WC-3	Woodcrest Creek	Woodcrest Creek	SE	200 LF of bank erosion along both banks in the park south of Sequoyah Trail.	---	200
MC-1	Merkle Creek	Merkle Creek	FS	There are structures on both sides of the stream corridor located upstream of 24th Street in the future 100-year floodplain. There are currently three 10-x-11-ft RCBs underneath 24th Street.	15	---
MC-2	Merkle Creek	Merkle Creek	FR/FS	Crestmont and Iowa Streets are being overtopped by the 10-year, 50-year, and 100-year future flows due to backwater from the existing three 10-x-11.5-ft RCB system at Main Street. There are structures upstream of Main Street in the future 100-year floodplain.	14	---
MC-2A	Merkle Creek	Merkle Creek	FR/FS	10-year, 50-year, and 100-year future flows are overtopping the existing three 10-x-7.5-ft RCB at Crestmont Street.	21	---
MC-2B	Merkle Creek	Merkle Creek	FR/FS	10-year, 50-year, and 100-year flows are overtopping the existing two 10-x-5-ft RCBs at Iowa Street.	1	---
RC-1	Rock Creek	Rock Creek	FR/FS	10-year, 50-year, and 100-year future flows are overtopping the existing two 9-ft CMP culverts at Robinson Road.	1	---
RC-2	Rock Creek	Rock Creek	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 10-ft RCP culvert at 36th Avenue NE.	0	---
RC-3	Rock Creek	Trib C to Rock Creek	FR/FS	10-year, 50-year, and 100-year future flows are overtopping the existing 6-ft CMP culvert at 36th Ave NE.	1	---
TMF-1	Ten Mile Flat Creek	Local	LD/SC	The earthen channel through Cambridge Addition West of 48th Avenue NW and North of Main Street is undersized. The 100-year flows have been known to extend into property owners' backyards.	---	---
				Totals	830	10,050

* Problem Types:

- FS – Flooded Structures
- SE – Stream Erosion
- FR – Flooded Roadway
- SC – Stream/Channel Capacity
- LD – Local Drainage (e.g., Storm Sewer, Detention, Channel Conveyance)

The localized problem along Lindsey Street between College Avenue and Tributary A (BC-16) is caused by the inadequate capacity of the roadway's storm sewer system and is significant since the street and building flooding recurs often which impacts vehicular, bicycle, and pedestrian traffic in the University of Oklahoma campus area.

Brookhaven Creek

Ten problems have been identified in Brookhaven Creek as shown in Table 5-2 and Exhibits 6-3, 6-4a, and 6-4b. Of the ten problems identified, one has flooded structures, four have one or more flooded roadways, four result from stream erosion, and three are localized drainage problems. These problems are scattered throughout the urbanized watershed.

Overall in the watershed, there are 276 buildings/structures in the baseline floodplain, four flood prone road crossing structures, and 3,150 ft of stream experiencing erosion. Six of the ten problems occur along the mainstem of Brookhaven Creek with two in Tributary A and three in various localized areas. The most significant problem along the mainstem is a stream flooding and erosion problem, BHC-1, in which 276 homes (including numerous mobile homes and residences north of Main Street and west of the creek) are located in the baseline (100-year) floodplain. In this problem area, flows overtop the Main Street pipe arch opening and spread out over a large area on the west side of the creek due to capacity limitations of the opening and the downstream creek. Some home flooding also occurs east of the creek downstream of Main Street. Since this area transitions into the Canadian River floodplain, it is generally wide and flat resulting in shallow flooding over a large area. Once flows exit the creek, especially on the west side, they may not return to the main channel as they spread out over the floodplain and flow toward the Canadian River.

Stream erosion downstream of Main Street – Brookhaven Creek

In addition to having inadequate flow capacity in this most downstream natural reach (BHC-1), the Brookhaven Creek mainstem is also experiencing significant stream erosion alternating from one side of the creek to the other over a distance of about 2,000 ft. Three other stream erosion problems (BHC-2, BHC-3, and BHC-4) are located between Main Street and 36th Avenue NW further revealing such problems in the lower stream reaches of the watershed.

Eroding stream and drainage outfall downstream of 36th Street NW – Brookhaven Creek

Clear Creek

No stream flooding or stream erosion were identified in the watershed. However, one localized problem area was identified in this primarily undeveloped watershed located along 120th Avenue SE south of Highway 9 and near Lake Thunderbird. The culvert system near the entrance to the Norman Zoo is undersized and the road profile is very near the adjacent road grade, which increases its flood prone nature. Further, the creek parallels the 120th Avenue SE roadway downstream of the culvert system and its limited capacity in this reach causes flood levels to inundate the roadway regardless of the culvert capacity limitations.

Canadian River

The investigation of problems along the Canadian River was not a primary consideration for this SWMP. Floodplains developed by FEMA provide the basis of describing flooding along the river with that floodplain being reflected in Exhibit 4-4 located in a map pocket in this report.

One localized problem area was identified in a small drainageway that drains into the Canadian River. This problem resides at Westbrooke/Terrace Road and Hollywood Street intersection where a traffic calming circular island was installed in the past. Storm water generated from developed areas flows into the intersection from the north, west, and south directions and floods the area before slowly draining off. The traffic island likely slows the flow of water exacerbating the problem but flooding would likely occur even without the island.

Dave Blue Creek

The Dave Blue Creek watershed is primarily undeveloped although urbanization is occurring in its north and western areas. Slopes are relatively steep compared to Norman watersheds in its urban core and western areas. Only two problems were identified in this watershed and both (DBC-1 and DBC-2) are related to stream flooding caused by inadequate road crossing culvert systems along 48th Avenue SE.

Culverts upstream of 48th Avenue SE – Dave Blue Creek

No stream erosion or localized problems were identified in the watershed.

Imhoff Creek

Numerous significant problems were identified in the Imhoff Creek watershed. In fact, the full scope of problems in this watershed outweigh the collective problems in other individual watersheds. This watershed is fully developed and generates high runoff rates and volumes that, in turn, cause stream flooding, stream erosion, and local drainage problems in numerous locations along the creek and at specific areas in the watershed. Although only six problem areas were originally identified, many of them cover long stretches of the creek and/or large localized areas. Five out the six problem areas are located along the mainstem of Imhoff. One of the problem areas (IC-3) has been subdivided into eight contiguous sub-reaches (IC-3A through IC-3H) due to its length, significance, and need to have phased improvements as it extends from the upper reaches of the creek near Andrews Park to a point downstream of the watershed’s middle, approximately 1,200 ft downstream of Lindsey Street. When looked at in this context, dividing IC-3 into eight sub-reaches results in Imhoff Creek watershed having 13 problem areas. Table 5-2 as well as Exhibits 6-7a, 6-7b, and 6-8 provide descriptions of the problems and their locations. Problems IC-4 and IC-4A are being considered as two “problems” although they both primarily relate to the need to reduce flows throughout Imhoff Creek and reflect the need for a one- or two-celled storm water detention facilities in and around Andrews Park to accomplish that purpose.

Overall in the watershed, there are 360 buildings/structures in the baseline (100-year) floodplain footprint (although the finished floor of many structures could well be above the baseline flood levels), 15 flood prone road crossing structures, and 5,000 ft of stream length with erosion problems. Of the 13 problems identified, nine relate to flooded structures (two being generally related to reducing flows using storm water detention), seven have one or more flooded roadways, two depict stream erosion, and one identifies a very large localized drainage problem in the Lindsey Street-McGee Drive intersection area.

WPA channel in Andrews Park – Imhoff Creek

From a stream flooding standpoint there are problems in the lower, middle, and upper reaches of the creek. In the lower natural channel reaches of the creek, 154 structures are located in the baseline (100-year) floodplain near Highway 9 with 49 structures being downstream of the highway (40 of which are east of the creek) and 105 located immediately upstream of the highway and on the east side of the creek. This problem area has been identified as, or linked to, IC-4/IC-4A as these structures can be removed from the floodplain with sufficient detention provided in the Andrews Park area in combination with the diversion of flow in the Lindsey – McGee intersection area proposed as solution IC-5. Exhibit 6-7a shows these flooded structures as well as the IC-4 and IC-4A proposed detention facilities. These structures were not historically shown in the floodplain by FEMA but SWMP corrections to the hydraulic model previous used in FEMA studies along the creek resulted in these structures being located in the floodplain footprint. Finished floor elevations for many of these structures are likely above the flood elevations since flood waters only exceed the creek top of bank by small amounts in the affected areas and spread out over the flat floodplain area at shallow depths. This problem reach of creek is co-located with stream erosion problems IC-1 and IC-2 that are subsequently discussed below.

Stream flooding problems in the middle and upper reaches are depicted by IC-3 and its A through H sub-reaches that extend from about 1,200 ft below Lindsey Street up to Webster Avenue near Andrews Park. The IC-3 problem can best be described by looking at the sub-reach problems as discussed below and shown in Exhibits 6-7a and 6-8 in Section 6.

IC-3A (From near the Elmwood Drive dead end upstream, about 1,200 ft downstream of Lindsey St., to near Madison St. dead end, including a road crossing upgrade at W. Lindsey St.)

This most downstream sub-reach of IC-3 includes a triangular shaped cross section with a concrete pilot channel. Flooding caused by medium sized events, such as a 10-year event, and large events, such as the 100-year (baseline) event, exceeds the creek's flow capacity and extends onto properties adjacent to the creek. In this sub-reach, 14 structures (homes) are located in the baseline floodplain footprint although a majority of these structures are on the fringe or edge of the floodplain with finished floor elevations likely higher than the baseline flood elevation. The Lindsey Street culvert system comprised of three 8-x-6-ft reinforced box culverts (RCBs) is undersized and flood prone as indicated in the flood profiles shown in Exhibit 6-8. This is an important east-west traffic carrier which results in potentially dangerous conditions and significant inconvenience when flooded.

IC-3B (From near the Madison St. dead end upstream to a location about 150 ft downstream of W. Boyd Street, including a crossing at W. Brooks Street)

The triangular shaped cross section with a concrete pilot channel continues for a majority of this sub-reach upstream to a point about 300 ft below W. Boyd Street where the concrete bottom continues but the side slopes become vertical masonry block walls. Flooding caused by medium sized events, such as a 10-year event, and larger events, exceeds the creek's flow capacity and extends onto properties adjacent to the creek. In this sub-reach, 32 structures (homes) are located in the baseline (100-year) floodplain footprint although a few of these structures are on the fringe or edge of the floodplain with finished floor elevations likely higher than the baseline flood elevation. The existing W. Brooks Street bridges spans 30 ft and is undersized and flood prone as indicated in the flood profiles shown in Exhibit 6-8.

Concrete-lined channel upstream of Lindsey Street – Imhoff Creek

Concrete lining and vertical walls downstream of Boyd Street – Imhoff Creek

IC-3C (From a location about 150 ft downstream of W. Boyd St. upstream to just below McNamee St., including road crossing upgrades to W. Boyd Street and S. Pickard Ave.)

The undersized creek channel in the IC-3C sub-reach consists of a concrete bottom with vertical mortared rock sides built as a WPA project over 70 years ago. Flooding caused by small sized events, less than a 10-year event, and larger events exceeds the creek's flow capacity and extends onto properties adjacent to the creek. In this sub-reach, 13 structures (homes) are located in the baseline (100-year) floodplain footprint with only a few of these located on the fringe or edge of the floodplain with finished floor elevations higher than the baseline flood elevation. The Boyd Street concrete slab bridge is only 12 ft wide with a 6 ft height and is undersized and flood prone as indicated in the flood profiles shown in Exhibit 6-8. This is an important east-west traffic carrier which results in potentially dangerous conditions and significant inconvenience when flooded. The Pickard Avenue crossing over the creek is a 12-x-5-ft concrete slab bridge that is also significantly undersized and floods often as Exhibit 6-8 reveals.

IC-3D (From just below McNamee St. upstream to just upstream of Symmes St., including road crossing upgrades to McNamee St., S. Flood Ave., and W. Symmes St.)

The creek channel in the IC-3C sub-reach is also undersized and consists of a concrete bottom with vertical mortared rock sides built as a WPA project over 70 years ago. Flooding caused by small events, less than a 10-year event, and large events exceeds the creek's flow capacity and extends onto properties adjacent to the creek. In this sub-reach, 29 structures (homes) are located in the baseline (100-year) floodplain footprint with most well inside the floodplain likely with finished floor elevations that are below the baseline flood elevation. The McNamee Street concrete slab bridge is only 12 ft wide with a 5-ft height and is undersized and flood prone as indicated in the flood profiles shown in Exhibit 6-8. The Flood Street and Symmes Street crossings over the creek are both 15-x-5-ft concrete slab bridges that are also significantly undersized and flood often as shown in Exhibit 6-8.

IC-3E (From just upstream of W. Symmes St. upstream to just below Main St.)

The IC-3E sub-reach also consists of a concrete bottom with vertical mortared rock sides built as a WPA project although the channel is somewhat deeper and narrower than in downstream sub-reaches as it is approximately 5 ft deep. As shown in Exhibit 6-7a, properties are flooded by small events with large events causing severe flooding damage in this sub-reach. Twenty five (25) structures (homes) are located in the baseline floodplain footprint with most (such as along Lahoma Avenue and Symmes Street) being well inside the floodplain with finished floor elevations that are below the baseline flood elevation. Many of these structures are in the FEMA floodway and have backyard fences that impede flow in the overbank.

WPA channel downstream of Flood Avenue – Imhoff Creek

IC-3F (A Main St. road crossing upgrade plus a small amount of adjacent channel improvements)

The IC-3F sub-reach consists solely of the Main Street crossing that presently has a 12-x-5.5-ft opening. This opening is much too small and causes overtopping of the roadway for small, medium, and large events as seen in Exhibits 6-7 and 6-8. The creek cross section on both sides of the crossing consist of narrow mortared rock WPA channels less than 10 ft wide and approximately 3–4 ft deep.

IC-3G (From just above Main St. upstream to just above W. Tonhawa St., including road crossing upgrades to W. Gray St., N. Lahoma St., and W. Tonhawa St.)

This relatively short sub-reach consists of a narrow mortared rock WPA channels less than 10 ft wide and approximately 3–4 ft deep. There are three small flood prone road crossing openings built as concrete slabs at Gray Street (10 x 5 ft), N. Lahoma Street (10 x 5.1 ft), and W. Tonhawa Street (10 x 5 ft) as shown in Exhibits 6-7a and 6-8. These road crossings and the small WPA channel do not have near enough capacity and flood often. In this sub-reach, there are 22 structures (homes) that are located in the baseline floodplain and flood often with most being located along W. Tonhawa Street.

IC-3H (From just above W. Tonhawa St. upstream to just above N. Webster Ave., including road crossing upgrades at W. Daws St., N. University Blvd., and N. Webster Ave. – N. Park Ave. crossing upgrade not included as this street is assumed removed as part of the Andrews Park storm water detention modifications)

Sub-reach IC-3H is the most upstream length of IC-3 and, like other downstream reaches, it consists of an undersized narrow and shallow WPA channel that often overflows and floods local residences. Adding to the problems are undersized and flood prone road crossing openings (slab bridges) at W. Daws Street (10 x 4 ft), N. University Boulevard (10 x 4 ft), and N. Webster Avenue (10 x 3 ft) as Exhibits 6-7a and 6-8 indicate. Given these conditions, 64 structures (homes) are located in the baseline floodplain with some of the worst flooding occurring along W. Tonhawa Street west of the creek.

WPA channel downstream of Daws Street – Imhoff Creek

Imhoff Creek has the worst stream erosion problems in Norman that extend approximately 5,000 ft as indicated in Exhibit 6-8. These erosion problems begin approximately 1,000 downstream of Highway 9, near the creek's confluence with the Canadian River, and extend upstream to a point about 2,000 ft upstream of Imhoff Road. Specifically, the IC-1 problem area is located below Highway 9 and IC-2 extends upstream of the highway. These two problem areas are somewhat similar in nature and represent a significant stream degradation process that includes down cutting of the streambed, widening of the creek between its banks through ongoing bank failure and collapse, as well as destruction of numerous trees, backyard fences, and loss of usable property. In the past, many of the fallen trees have trapped other fallen trees, tree branches, and other debris which have periodically blocked the creek flow. These types of creek blockages cause further erosion as flows move around the sides of the blockage and further erode adjacent properties. This erosion process will continue until the creek re-stabilizes in an enlarged condition. These problems are a direct result of upstream urbanization of the watershed including increased impervious cover and more efficient drainage systems which, in turn, have led to increased runoff volumes and rates that the creek is trying to accommodate by enlarging.

Stream erosion and fallen trees upstream of Imhoff Road – Imhoff Creek

One of the biggest problems in the Imhoff Creek watershed is the IC-5 localized problem located in the west-central portion of the Imhoff Creek watershed in the vicinity of Lindsey Street and McGee Drive as located on Exhibit 6-7b. Historically, this problem has been one of the worst flooding problems in Norman as it occurs often and lingers for hours due to the flat nature of the local topography, the high intensity of local development, and the lack of adequate drainage infrastructure. During even small storm events, traffic in the local area can be slowed and brought to a halt due to high water around the intersection. Local businesses flood and suffer from frequent flooding events that drive away potential customers. In this area, storm water flows from the north overland and along McGee Drive and other north-south aligned streets and into the Lindsey Street area in several locations. Behind the shopping center located just south of Lindsey and east of McGee, the City has built a concrete channel that collects excess storm flows and delivers it to a large storm sewer system that then takes the flows to Imhoff Creek, outfalling approximately 1,200 ft south of Lindsey Street. Also, at some point between McGee Drive and Wylie Road, a small storm sewer system along Lindsey picks up some runoff and takes it eastward to Imhoff Creek. Although these two systems help some with drainage in the area, they are significantly undersized resulting in the severe flooding problem in the localized area.

Little River Mainstem

There are two problems (LR-1 and LR-2) that have been identified along the Little River mainstem for which CIP projects have been conceptualized. These two problems are located in Exhibit 6-9 and described in Table 5-2. LR-2 is a stream flooding problem consisting of an approximate 40 unit mobile home park that is flooded by medium and large events thusly endangering residents and causing considerable damage. A majority of the units or lots are in the baseline (100-year) floodplain although a few may be outside of this floodplain.

LR-1 is a severe stream erosion problem located about 2,000 ft upstream of 12th Avenue NW. The river bank has eroded along about 350 ft of river presently although additional erosion is likely in the future. The eroded bank is within approximately 70 ft of a residence and could eventually threaten the structure.

Eroding stream bank upstream of 12th Avenue NW – Little River

No localized problems were identified in the watershed.

However, there are other stream flooding and stream erosion problems beyond these two CIP projects that exist and deserve some consideration. These mainstem problems relate to road crossing flooding or overtopping (see Exhibit 6-10) and stream erosion that appears to be accelerating along the river. The potential flood-related problems were not added to the CIP list since a more comprehensive transportation system upgrade of Franklin Road and its many intersecting roadways is badly needed. This upgrade would include roadways extending from 24th Avenue NW to a point beyond the eastern limit of the Level 1 analysis reach at 48th Avenue NE.

Franklin Road generally parallels Little River between 24th Avenue NW and 48th Avenue NE and is inundated by the river's 100-year baseline floodplain for almost 2.7 miles within in this six mile road length, primarily east of N. Porter Avenue. Additionally, numerous small tributaries cross the roadway, and are a flood hazard to the roadway, as they flow toward the river from the north. To alleviate flooding along Franklin Road and the numerous intersecting streets in this area, a significant road upgrade program well beyond this SWMP, would be required. Such a program would likely be a combination of raising the roadway while also increasing the bridge and/or culvert openings at road crossings. Design for such a roadway upgrade would need to consider the potential for increased peak flows in downstream areas as a result of enlarging a number of upstream bridge and culvert openings as well as reducing river flow capacity due to a raised roadway blocking flows at crossings and where the road runs parallel to, and near, the river.

The Level 1 study reach of Little River is also beginning to reveal significant stream erosion problems as a result of its urbanizing watershed and the related increased runoff peak flows and volumes. All indications are that stream erosion will become an even greater problem along Little River and its tributaries in the future as its watershed further develops. Access is limited along the river due to its rural nature and difficulty in obtaining approvals to enter properties along the river so there are likely undetected erosion problems that exist now and will get progressively worse for a long time in the future.

Little River – Tributary G

As shown in Table 5-2, Tributary G to the Little River has only one significant problem area and it is associated with stream flooding upstream of Franklin Street just west of the IH 35 highway corridor. Flood levels are increased by the IH 35 culvert system which, in turn, increases the flood levels at Franklin Street as shown in Exhibits 6-11 and 6-12 in Section 6. As development occurs in this fast growing area of Norman, traffic along Franklin Street is increasing raising concerns about flooding dangers at this crossing.

No stream erosion or localized problems were identified in the watershed.

Little River – Woodcrest

Four problems (WC-1A, WC-1B, WC-2, and WC-3) have been identified for the Woodcrest tributary to Little River, three of the problems reflect stream flooding and one is a stream erosion problem. Twenty (20) homes are located in the baseline floodplain and Sequoyah Road (WC-2) and E. Rock Creek Road crossings over the creek are flood prone as shown in the floodplains and flood profiles respectively shown in Exhibits 6-13 and 6-14 in Section 6. However, the City is presently upgrading the E. Rock Creek Road crossing so it is not considered further as a problem. WC-1A identifies the fact that peak discharges exceed downstream stream and road crossing opening flow capacities. WC-1B focuses specifically on the lack of stream flow capacity in the overgrown and undersized natural channel downstream of Sequoyah Road. The 200 ft of stream erosion (WC-3) upstream of Sequoyah Road is a moderate problem that will likely get worse in the future although upstream flow control (flood detention) targeting small frequent runoff events could help in controlling the erosion.

Culvert view downstream side of Franklin Road – Trib. G to Little River

Stream erosion downstream of Sequoyah Trail – Woodcrest Creek

No localized problems were identified in the watershed.

Merkle Creek

Four problems have been identified in the Merkle Creek watershed as described and located in Table 5-2 and Exhibits 6-15 and 6-16. Of the four problems identified, all four have flooded structures, two have one or more flooded roadways, although no stream erosion or localized problems were identified. It is noted that a storm water detention facility being constructed during the SWMP project and located immediately upstream of Robinson Street was not considered part of existing conditions but, rather, has been considered as a future (proposed) conditions although no costs will be associated with the privately funded improvements.

Overall in the watershed, there are 51 buildings/structures in the baseline floodplain (see Exhibit 6-15) and two flood prone road crossings (see Exhibit 6-16). The most significant problem along the creek is a stream flooding problem (MC-2) in which the Main Street culvert system and adjacent undersized creek conveyance contributes to flooding of upstream structures (homes) as well as road crossings at Crestmont Street and Iowa Street. In addition to the backwater caused by the Main Street culvert system and adjacent channel, the Crestmont Street (MC-2A) and Iowa Street (MC-2B) crossing are undersized and cause flooding of numerous structures upstream of those crossing openings. These three problem areas are contiguous and somewhat related as their problem identification numbers indicate. Combined, there are 36 structures that are in the baseline (100-year) floodplain in these three problem areas. The MC-1 problem is also significant as 15 structures upstream of 24th Street SW are in the baseline floodplain due to the inadequate capacity of the road crossing opening there as well as creek conveyance limitations that currently exist upstream of the road crossing. Exhibit 6-15 in Section 6 clearly shows the backwater impact of the 24th Street culvert system on the 50- and 100-year flood profiles as water levels increase by 3–4 ft through the culvert system.

Rock Creek

The Rock Creek watershed is primarily undeveloped although it is undergoing urbanization in its headwater (upstream) areas. As shown in Exhibits 6-17a, 6-17b, 6-17c, 6-18a, and 6-18b, three problems (RC-1, RC-2, and RC-3) were identified in the watershed with two being located along the mainstem and one problem (RC-3) located along Tributary C on which one structure was shown to be in the baseline floodplain. All three of the problems relate to stream flooding with all also including flood prone road crossings and one (RC-3) also involving creek capacity problems. Traffic is increasing along the roadways in the watershed making road crossings over creeks much more dangerous to the general public. The Robinson Street (RC-1) and 36th Avenue NE crossings over Rock Creek as well as the 36th Avenue NE crossing over Tributary C to Rock Creek are all overtopped for the 10-year and greater floods under baseline conditions.

No stream erosion or localized problems were identified in the watershed.

Culvert outlet downstream of Main Street – Merkle Creek

No culvert headwall downstream of 36th Street NE – Rock Creek

Ten Mile Flat

With its overall flat slopes, shallow channels, and rural character, the nature of stream flooding, stream erosion, and localized flooding in the Ten Mile Flat watershed is significantly different from that in other Norman watersheds. As shown in Exhibit 6-19 and as presented in a FEMA Floodplain/Floodway Conditional Letter of Map Revision (CLOMR) for Ten Mile Flat Creek (MacArthur Associated Consultants, Ltd., 2005), flooding is a general problem in the watershed but the rural land use results in less flooding damage compared to those Norman watersheds that are predominately urbanized. The flooding, most of which is shallow, occurs from runoff generated within the watershed as well as from periodic Canadian River overflows. Exhibit 6-19 indicates structures that are in the 100-year floodplain according to the FEMA CLOMR (approved by FEMA in 2007), which also shows the lower watershed's flooding from the Canadian River. Many of the structures are farm buildings although there are some residence structures that flood. Given that development in most of this watershed has been projected to be low density in the City's 2025 Land Use Plan, future flooding was assumed to be similar to existing flooding.

Typical broad and flat floodplain area in Ten Mile Flat Creek watershed

According to the MacArthur (2005) report, roadways such as W. Main Street, W. Robinson Street, and W. Rock Creek Road are flooded by the 100-year event. W. Tecumseh and 60th Avenue NW are shown as passing such a large event with little, or no, flooding following the completion of ongoing or scheduled drainage and/or roadway projects by the City or local land developers. Given the work associated with the CLOMR and the ongoing projects, TMF-1, located in Exhibit 6-19, is the only watershed specific storm water problem identified in this SWMP.

5.2 PROBLEM IDENTIFICATION METHODOLOGY

As stated above, Table 5-2 presents a summary description of each problem identified with problem locations tracking with respective solutions in Section 6 exhibits. The methodology for identifying problems associated with stream flooding, stream erosion, water quality, and local drainage is provided below. As discussed previously, water quality conditions are approached on a citywide basis and, therefore, are approached in a more broad manner.

5.2.1 Stream Flooding

The identification of flooding problems is presented on a watershed and stream reach basis according to various levels of study detail consistent with the SWMP objectives. As specified above, there are stream flood related aspects in 34 of the 59 overall problems identified. The identification of flooding problems along the major Level 1 and Level 2 streams utilizes the results of the baseline 100-year floodplain which is based on future full buildout urbanization according to the Norman 2025 Plan. As discussed in Section 1, Level 1 stream reaches were selected by City staff as those reaches in which existing problems need better definition and/or new detailed flooding information is needed in order to assess flooding risks as new development occurs near those stream reaches. Budget limitations prohibited the inclusion of numerous stream development reaches as Level 1 study reaches. Level 2 streams represent those stream reaches in Norman's urban core that have been studied previously and the basic models developed in those earlier studies were used in the SWMP development.

Additional streams presently needing studies at a Level 1 degree of detail are represented as Level 3 stream reaches. Certain Level 4 reaches expected to see local land development may also be in need of detailed analyses. Although specific problem areas were not identified in Level 3 and Level 4 stream reaches, the future 100-year floodplains (also referred to as "Stream Planning Corridors" and discussed in Sections 4 and 7) are presented along those streams for waterways with 40 acres or more of drainage area. These Stream Planning Corridors present a very approximate estimation of the future 100-year floodplain that identifies areas inundated by such an event. A map (Exhibit 4-4) delineating the estimated 100-year floodplain for all study reaches (Levels 1, 2, 3, and 4) is provided in a map pocket in this report. Exhibit 4-4 provides a general overview of areas subject to flooding throughout the City and represents the only extent of flood identification for Level 3 and 4 stream reaches.

An extensive review of the SWMP hydrologic and hydraulic analyses presented in Section 4 allows for the identification of flood related problems for Level 1 and 2 stream reaches. Specifically, these analyses provide a means of estimating where homes, businesses, and other structures lie within the respective stream reach baseline 100-year floodplains as well as where road crossings are inundated by the baseline 50-year flood elevations. Although the baseline (future, full development buildout) provides the basis of identifying flood related problems, the existing floodplains and flood profiles have also been reviewed and included in the overall problem identification process. The baseline 100-year floodplains and 50-year flood profiles for Level 1 and 2 stream reaches are presented in Section 6 so that they can be viewed concurrently with the respective floodplains and profiles that correspond with the recommended solutions developed. These floodplains and flood profiles are presented together for each Level 1 and 2 stream reaches to present the flooding locations within each watershed.

5.2.2 Stream Erosion

Stream erosion is a major problem in several stream reaches in the City. The identification of stream erosion problems are based on existing conditions although it should be considered that new problems will likely surface in the future due to increased runoff rates and volumes associated with Norman's urbanization. The watershed assessments (Section 3) provided excellent data and information to locate stream erosion problems. The field reconnaissance, review of the new aerial photography, and spatial analysis of the land use, impervious cover, and soils associated with the watershed assessments allows for the determination of the location and severity of the major stream erosion problem sites in the City. Thirteen (14) of the 59 problems identified have a stream erosion component some of which are very severe threatening homes, fences, roadways, utilities, and trees. Such locations include the downstream portions of Bishop Creek, Imhoff Creek, and Brookhaven Creek which are all streams draining areas that have been urbanized or urbanizing over the last few decades. Lower Merkle Creek just downstream of W. Lindsey Street also had an emerging erosion problem until a local development project added rubble/riprap to in an attempt to stabilize the area. This location will need to be monitored to see whether this riprap protection will be adequate and the modified stream reach remains stable.

5.2.3 Water Quality

Water quality problems have been determined to exist in Norman's storm water systems located in its "urbanized areas" by the United States Environmental Protection Agency's (EPA) National Pollutant Discharge Elimination System (NPDES) program. These urban storm water systems are referred to as municipal storm water separate storm sewer systems (MS4s). In Oklahoma, mandatory compliance with this program is being implemented by the Oklahoma Department of Environmental Quality (ODEQ) and its Oklahoma Pollutant Discharge Elimination System (OPDES) program. The City of Norman has initiated a storm water quality monitoring program targeting numerous locations to assist in identifying water quality problems in the city. A listing of the monitoring and visual screening sites shown in Figure 5-1 is provided below. In an effort to better define water quality conditions in the City and to assist in meeting their regulatory obligations, the City is presently providing quarterly sampling for total suspended solids, chemical oxygen demand, ammonia, phosphate, and nitrate at the monitoring locations. Pesticides and metals scans are also run once a year for samples taken at these locations. Further, the City has started sampling Bishop Creek for fecal coliform in response to the recent Total Maximum Daily Load (TMDL) study for the Canadian River (which includes Bishop Creek as a tributary and possible contributor to the bacteria problem) and added two sample points at tributaries of Little River coming from the Moore and Oklahoma City urbanized areas. The City should also consider expanding their monitoring program to include other creeks that contribute runoff and pollutants into Lake Thunderbird such as Hog Creek, Rock Creek, and Dave Blue Creek.

ODEQ also recently completed a Total Maximum Daily Load (TMDL) study for the Canadian River that identified Norman and the University of Oklahoma as contributors to non-attainment for fecal coliform in Bishop Creek, a local tributary to the Canadian River. Additionally, ODEQ is also concerned that urban development, without appropriate mitigation of its environmental impact, will further degrade Lake Thunderbird's water quality. The agency is

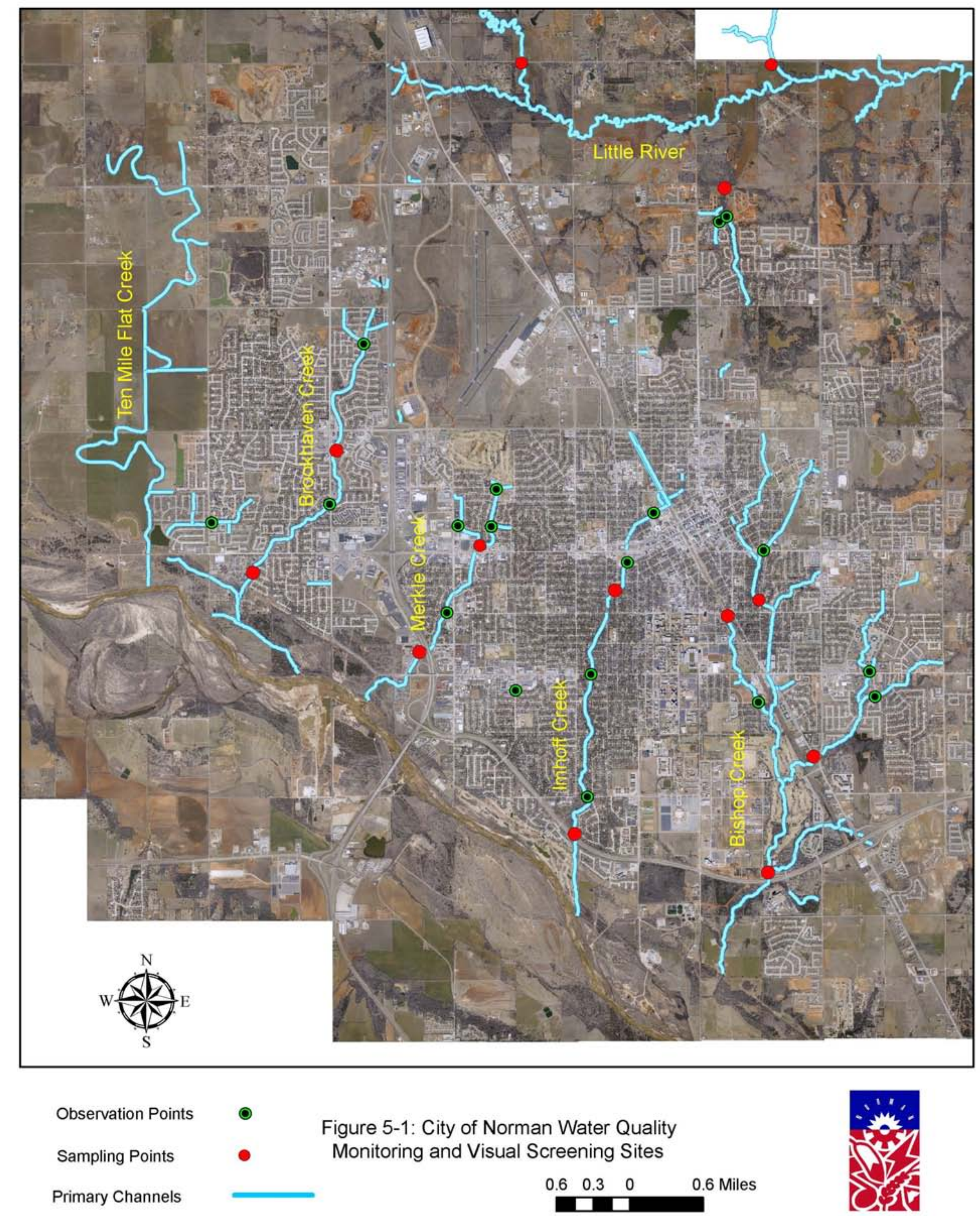


Figure 5-1: City of Norman Water Quality Monitoring and Visual Screening Sites

Monitoring Station	Locations
Bishop 1	Bishop Creek @ Marshall Avenue
Bishop 2	Bishop Creek @ Classen Boulevard
Bishop 3	Bishop Creek @ Boyd Street
Bishop 4	Bishop Creek @ Oklahoma Avenue
Imhoff 1	Imhoff Creek @ SH 9
Imhoff 2	Imhoff Creek @ Flood Street
Merkle 1	Merkle Creek @ Lindsey Street
Merkle 2	Merkle Creek @ Main Street
Brookhaven 1	Brookhaven Creek @ G Street
Brookhaven 2	Brookhaven Creel @ Havenbrook Street
Woodcrest 1	Woodcrest Creek @ Tecumseh Road
Little River 1	Little River @ 1600 West Franklin Road
Little River 2	Little River @ 600 East Franklin Road

presently developing a watershed management plan that will identify management practices and their implementation in the lake's watershed to help achieve beneficial uses of the lake waterbody.

Existing studies as well as determinations made by EPA and OPDES provide the determination of water quality problems in Norman. The existing studies considered include a Rock Creek watershed study for the Central Oklahoma Master Conservancy District (Vieux, 2006), a Lake Thunderbird Watershed modeling and analysis for the Oklahoma Conservation Commission (Vieux, 2007), an ongoing watershed plan developed by the Oklahoma Department of Environmental Quality for Lake Thunderbird (ODEQ, 2008a), and the recently completed Canadian River Bacteria TMDL (ODEQ, 2008b). As part of this master plan development effort, Vieux has provided an overview of these past studies entitled Storm Water Quality Assessment, which is included in Appendix G. A brief summary, much of it verbatim, of that overview is provided below.

Rock Creek Watershed Study

This analysis and water quality evaluation study was performed for the Rock Creek watershed, a significant tributary to Lake Thunderbird, by Vieux for the Central Oklahoma Master Conservancy District (Vieux, 2006). This study estimated the potential impact of land use changes in Rock Creek on nutrient and sediment loading from storm water runoff to Lake Thunderbird. Rock Creek, with an area of 11.9 square miles, drains to the Little River arm of the lake, located entirely within the corporate limits of the City and the Lake Thunderbird watershed. COMCD supplies drinking water derived from the reservoir to the City and two other communities, Del City and Midwest City. Sampling of the water quality in the lake was conducted and reported by OWRB (2001, 2002, 2004a, 2004b, and 2005) in fulfillment of state water quality programs and for COMCD. Lake eutrophication caused by persistent nutrient loading and consequent algae proliferation is a serious concern because the waterbody is designated as a sensitive water supply (SWS) by the State of Oklahoma. The lake exceeds the SWS chlorophyll *a* water quality standard (WQS), 10 µg/l, by as much as three fold due to algae growth. Some species of algae found in the lake can produce toxins. Though toxins have not been found in the lake as reported by OWRB (2004), incidence of toxins produced by these species is known to increase as chlorophyll *a* concentrations exceed the WQS of 10 µg/l (Downing et al., 2001). Besides the risk of toxins in the finished drinking water, excessive algae production also leads to taste and odor complaints about the finished water product.

In support of the COMCD (Vieux, 2006) study, local sampling of tributary runoff in Rock Creek was performed by the OWRB in conformance with EPA standards. The constituents and concentrations were monitored and used to assess the impacts from urbanization within Rock Creek where there is a range of undeveloped to highly developed land use. This study revealed significant differences between locally sampled data and National Stormwater Quality Database (NSQD) constituent concentrations. In general, nutrients and TSS were elevated significantly in comparison to expected values based on land use in the NSQD database.

Oklahoma Conservation Commission Lake Thunderbird Watershed Study

Since water quality in Lake Thunderbird currently does not meet water quality standards, chlorophyll *a* and turbidity, the Oklahoma Conservation Commission (OCC) completed a study that assessed and quantified the impact of projected future land development on storm water quality loadings to the lake as well as targeted management practices within the watershed that would reduce loadings from nonpoint source pollution and achieve water quality standards established for this Sensitive Water Supply. Watershed modeling and analyses for the OCC was performed using the Soil Water Assessment Tool (SWAT) and reported by Vieux (2007). Both baseline (2000) and projected (2030) water quality impacts were modeled to assess the impacts of land use conversion through urban development. The major findings can be summarized as follows:

- Both runoff and constituent concentration affects the annual load of nutrients or suspended solids that storm water conveys to the lake. Increase in runoff is partially driven by impervious cover.
- Algae growth in Lake Thunderbird is increased by nutrients, in particular, phosphorus. Total phosphorus (T-P) loadings were determined to increase with urban land development. Algae growth and chlorophyll *a*. concentrations are a major concern of ODEQ, OCC, COMCD and the water supply users. Since T-P is a limiting nutrient for algae growth and resulting concentrations of chlorophyll *a*, increases in T-P would very likely exacerbate those problems.
- T-N is a source of nutrients that can also accelerate algal growth in the lake, but is not considered a limiting nutrient.
- SWAT modeling revealed considerable potential for reducing phosphorus loadings into Lake Thunderbird using structural and non-structural water quality controls. Structural controls included detention basins, constructed wetlands, retention basins, and bio-retention filters. Non-structural controls included voluntary and mandatory urban fertilizer use restrictions. As discussed in Section 7.2, wetlands, stream buffers, and other means of using protected vegetation for water quality protection are often categorized together as they use many of the same soil, water, and vegetative processes to enhance water quality (see Table 7-4).

ODEQ Lake Thunderbird Study

An ongoing study by the ODEQ (2008a) is developing a watershed plan that assesses the water quality in watershed tributaries, as well as, the impacts of nutrient and sediment loading on water quality in the lake. Lake Thunderbird is listed on the State's 2006 303(d) list for impaired uses of aesthetics and warm water aquatic community. The causes of the impairments are low dissolved oxygen (DO) and high turbidity. The draft 2008 303(d) awaits EPA approval, but does list Lake Thunderbird as being impaired for chlorophyll *a*, DO, and turbidity. The sources of these impairments are listed as "unknown." While there are no permitted point sources of discharge, nutrients and sediment loadings from nonpoint sources discharging during runoff events through tributary streams are believed to be the major cause of the impairments. Another factor, though of lesser importance, is good agricultural practices in rural areas that can affect the lake's water quality. The goal of the watershed study is to determine acceptable loading rates for nutrients and suspended solids that will help allow the intended beneficial use of Lake Thunderbird to be achieved. In light of the unique challenges associated with reducing nonpoint source contributions, ODEQ intends to use a watershed-based plan in lieu of a TMDL for Lake Thunderbird.

Several agencies are cooperating in the development of this watershed plan. The partner agency/organization that ODEQ will work with to develop the plan are the Oklahoma Conservation Commission (OCC) and the COMCD. OCC is the state's main agency for nonpoint source pollution control, and COMCD is the lake's managing organization. OCC will perform watershed stream monitoring in its Priority Watershed Program, and COMCD will fund the data collection effort in the lake through their ongoing contractual agreement with the Oklahoma Water Resources Board (OWRB) and a legal settlement with the ODEQ regarding a storm water permit in the watershed. ODEQ will perform the modeling work using the data collected by OCC and OWRB.

Lake Thunderbird

Water quality modeling goals for this study will be used to establish key nutrient (phosphorus and nitrogen) and turbidity reduction goals for the watershed. The modeling work will also provide information on sources of loadings and potential management options implemented in the watershed. When the ODEQ establishes the watershed management plan the Cities of Oklahoma City and Norman could be required to implement management practices to reduce nutrients and sediment in storm water runoff that drains to the lake.

ODEQ Bacteria TMDL for the Canadian River

Recently, ODEQ (2008b) completed a Total Maximum Daily Loads (TMDL) study for the Canadian River. Elevated levels of pathogen indicator bacteria in aquatic environments indicate that receiving water is contaminated with human or animal feces and that there is a potential health risk for individuals exposed to the water. Pollutant load allocations for indicator bacteria in the Canadian River are currently being established. Waterbodies in the study area are listed on the ODEQ 2004 303(d) list because there is evidence of nonsupport of primary body contact recreation (PBCR), resulting in the development of a TMDL for the Canadian River and certain tributaries including Bishop Creek. Bishop Creek failed to support PBCR due to fecal coliform (FC) concentrations. Seventy-five percent of samples collected at Bishop Creek and Jenkins Avenue exceeded permissible FC concentrations for single samples. The MS4 permit for small communities in Oklahoma became effective on February 8, 2005. Two such MS4 permit holders discharge to Bishop Creek; they are the City of Norman and the University of Oklahoma. The major contribution of FC to Bishop Creek is believed to be from nonpoint sources, though point sources have been identified from sanitary sewer overflows (SSO) that have occurred in Bishop Creek. The estimated FC loads for the four major nonpoint source categories, which contribute to elevated bacteria concentrations in Bishop Creek are estimated to be Commercially Raised Farm Animals (82.26%), Pets (17.66%), Deer (0.04%), and Septic Tanks (0.04%) (ODEQ, 2008b, pg. 3–20 ff).

Compliance with the TMDL requirements under the MS4 program will require that storm water permit holders develop strategies designed to achieve progress toward meeting the reduction goals established in the TMDL. The City of Norman and the University of Oklahoma may be required to participate in a coordinated monitoring program or develop their own for purposes of documenting the effectiveness of the selected best management practice (BMP) and for demonstrating progress toward attainment of water quality standards. Reporting requirements include documentation of actions taken by the permittee that affect MS4 storm water discharges to the impaired waterbody segment (ODEQ, 2008b).

5.2.4 Local Drainage

The identification and location of local drainage problems were provided by the City of Norman based on citizen complaints and observation of the various problems. These problems typically result from inadequate drainage system infrastructure including inlets, street gutters, storm sewers, and/or channels that are undersized. Each problem is distinct in its causes with some being relatively small and straightforward while some are more complex such as the West Central Imhoff Creek watershed (Lindsey Street-McGee Drive intersection) problem. Descriptions of the local problems are provided in Table 5-2 organized by the watershed in which each is respectively located. Numerous photographs were taken in each of these problem areas; the photos will be made available to the City as a separate project deliverable.

6.0 STORM WATER SOLUTIONS

A variety of conceptual solutions have been developed for the stream flooding, stream erosion, water quality, and local drainage problems identified in Section 5. It is anticipated that many of these solutions will be included in a City capital improvement program (CIP) as outlined in this section and in Section 8 for the financial planning requirements. To the extent possible, integrated solutions were developed in order to address storm water issues in the most comprehensive way possible. In most but not all instances, the problems tended to be of one major type such as stream flooding and the primary emphasis of the solution primarily addressed that storm water aspect. However, in solving such one-dimensional problems or in instances in which more than one type of problem occurred in one location, care was taken to develop a solution that further improved other storm water aspects. For instance, if a conceptual stream flooding solution was developed, it was done so in a manner to also protect the stream from future erosion.

Other considerations were also made to incorporate items such as improving and/or protecting the stream's environmental integrity by using bio-engineering and natural channel design techniques, preserving the historical character of an existing solution type such as a WPA channel found in the upper Imhoff and Bishop Creek watersheds, improving water quality, and/or identifying greenway opportunities. Solutions were developed in a way to recognize and respect the conditions and character of the respective watershed in which the problem exists. In addition to considering the opportunities of preserving or enhancing environmental and recreational conditions, the solution development process included the consideration of possible alternatives or options and reviewing preliminary findings with City staff as well as the project Task Force to obtain their feedback and guidance.

As with the identification of problems, a watershed-specific approach in developing conceptual solutions was followed to respect the conditions that exist in the various watersheds. Solutions were developed for Level 1 and 2 streams as well as local drainage problems considering that the potential exists to positively or negatively affect other locations within that respective watershed. Solution development targeted future watershed development conditions projected in the City's 2025 Land Use Plan. In this manner, solutions and programs developed will better serve the City of Norman in addressing their storm water needs in the future and will provide a more complete "blue print" for managing storm water.

Similar to the approach for identifying water quality problems and due to their "non-point source" nature, solutions for water quality problems were evaluated on a citywide scale consistent with what is required for cities throughout the country. This citywide approach to addressing water quality involves using a programmatic approach which is now ongoing with the City's MS4 Program with the potential to be expanded due to Canadian River TMDL concerns as well as the ODEQ Watershed Plan that is being developed for the basin area draining to Lake Thunderbird.

Other important aspects of developing solutions included the development of cost estimates for the improvements as well as the prioritization of the many solutions. While the cost estimates are general in nature to match the conceptual design level of the solutions, they were developed to provide a good approximation of the costs that can be expected to design, permit, construct, and implement the solutions. Details of project cost estimating and prioritization development are subsequently provided in Section 6.2 that follows the summary of results provided immediately below. Comprehensive financial planning associated with the City's overall storm water needs is provided in Section 8.

6.1 SUMMARY OF SOLUTIONS

Conceptual solutions for the 59 flood-related and stream erosion problems have been developed for the Level 1 and 2 streams evaluated as well as specific local drainage area problems identified. Estimated costs for these projects or solutions totaled $82.6 million, which can be rounded to $83 million. As discussed in Section 5, approximately 84% of the problems were located in the urban watersheds of Bishop Creek, Brookhaven Creek, Imhoff Creek, Merkle Creek, and Woodcrest Creek. Solution costs for these same urban watersheds represent over 90% of the total citywide costs. Table 6-1 provides a breakdown of watershed costs listed in order of costs as well as the percentage of total costs that each watershed represents.

Table 6-1
Watershed Capital Improvement Project Costs

Watershed	Costs ($M)	% of Total Cost
Imhoff Creek	$43.7	52.91
Bishop Creek	$11.9	14.41
Merkle Creek	$8.9	10.78
Brookhaven Creek	$6.0	7.26
Woodcrest Creek	$3.3	4.00
Rock Creek	$3.1	3.75
Clear Creek	$1.8	2.18
Dave Blue Creek	$1.8	2.18
Trib G, Little River	$1.0	1.21
Little River	$0.4	0.48
Canadian River Area	$0.4	0.48
Ten Mile Flat	$0.3	0.36
Totals	$82.6	100.00

The solution locations are spread over a large part of the City but, like the problems that they solve, are located along, or west of, 48th Avenue East. Each solution (and matching problem), also referred to as a "project," has been given an identification number such as "IC-1" which provides, in this case, a reference name for a specific solution (and problem) in the Imhoff Creek watershed. Again, the solution identification numbers match those for the respective problems presented in Section 5. As discussed above and in Section 5, water quality problems are dispersed throughout the City, including the urban core area as well as the area that drains into Lake Thunderbird. Due to the nature of the water quality problems, as defined by federal and state regulations, solutions to address them are applied to the City as a whole and need to be implemented as a program or overall plan. This is discussed further below.

Certain solutions address overlapping problems, such as stream flooding and stream erosion. Mirroring the problems identified and considering the 59 solutions developed:

- 34 (58%) address stream flooding along Level 1 and 2 streams,
- 14 (24%) involve stream erosion along Level 1 and 2 streams, and
- 12 (20%) resolve local drainage problems.

Table 6-2 highlights the problems and solutions on a watershed basis that is discussed further below. On a citywide scale and as totaled at the bottom of Table 6-2, the collective performance of all solutions:

- removes 652 of 830 structures in the 100-year baseline floodplain,
- removes 36 out of 36 flood prone road crossings, and
- stabilizes 10,050 ft of eroding streams

The solution for BHC-1 along Brookhaven Creek targets flood related as well as stream erosion aspects, both as primary solutions. Recognizing that many consist of multiple problem types, of the 34 flood related solutions on Level 1 and 2 streams:

- 26 target structure or building flooding,
- 29 include road crossings that are flooded (overtopped by floodwaters), and
- 12 have a structure/parcel buyout component.

Although varying approaches, methods, and analytical tools were used to develop solutions for flooding, stream erosion, and water quality, these solutions were also looked at on a watershed, ward, and City-wide basis to better understand their relationships on various spatial, environmental, and political scales. Table 6-2 concisely presents the following summarized information for each of the individual solutions (or projects):

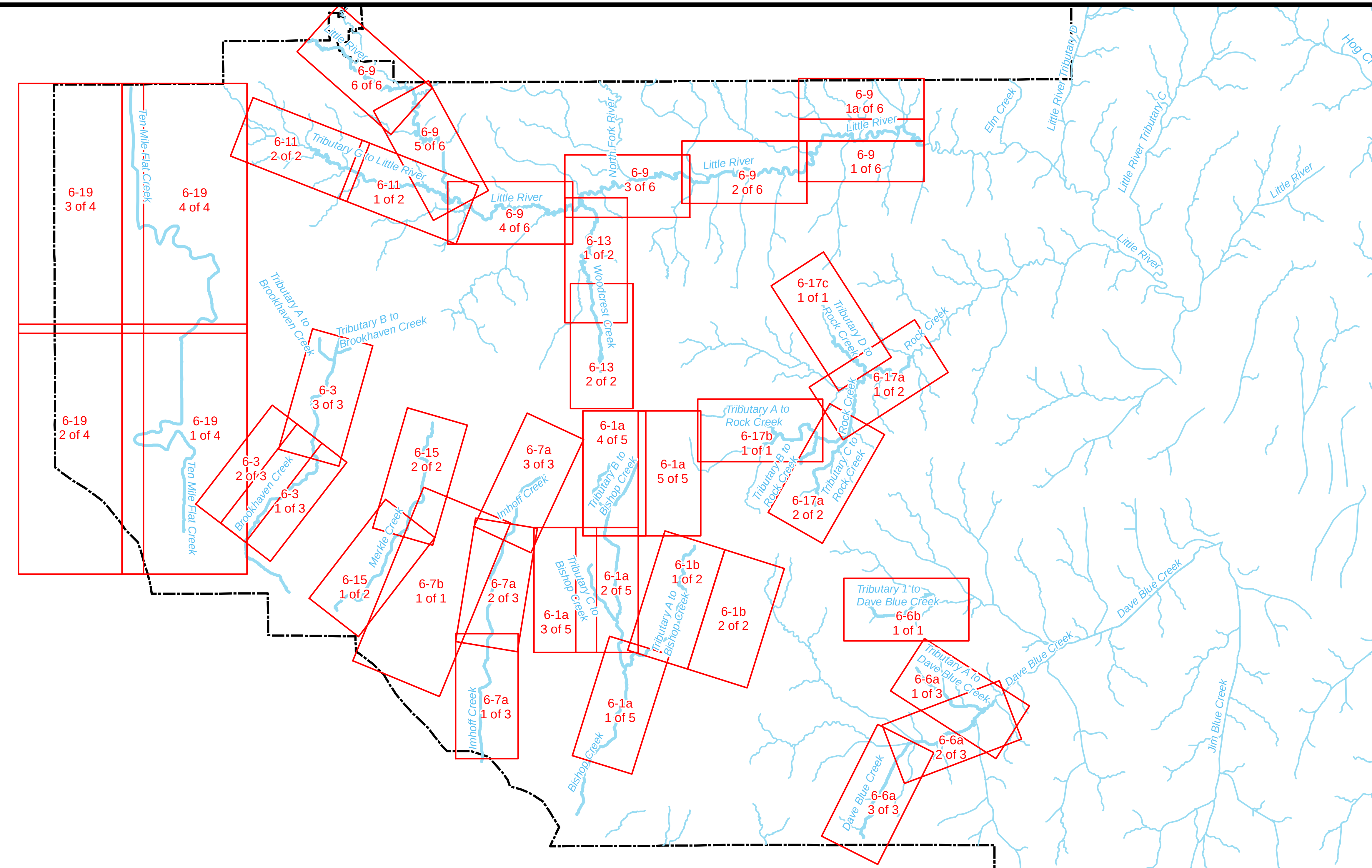
- general location within the City, watershed, and ward,
- solution type(s) including the integration of solution types,
- problem description,
- solution overview,
- key items in defining problem elements and solution results in terms of flood control (structures removed from 100-year baseline floodplain and roadway crossings protected from flooding), stream stabilization (length stabilized), and greenbelt integration opportunities,
- conceptual level cost estimate (see Appendix H for more detail),
- prioritization score (see Appendix I for prioritization spreadsheets of individual problems/solutions), and
- prioritization score ranking within the City, respective watershed, and respective ward(s).

In addition to Table 6-2 and on a watershed basis, Exhibits 6-1a through 6-19, respectively, present the location and extent of stream flooding solutions for those watersheds within which a Level 1 or 2 analyses were carried out. It is pointed out that Table 6-2 includes the number of proposed buyouts in the solution values given for structures removed from the baseline floodplain although the exhibits do not identify the buyouts in the color coding for structures removed from the floodplain. **Solution flood profiles are only provided in this report section for those Level 1 or 2 streams in which a solution is being proposed that alters the flood profile. However, sets of flood profiles (10-, 50-, 100-, and 500-year) are presented in Appendix J for existing as well as baseline or future (full build-out) conditions for all Level 1 and 2 streams.** The odd numbered exhibits provide a very good watershed-specific overview (plan view) of the flooding conditions before and after solutions are in place by delineating and overlaying the floodplains for 100-year baseline (full buildout or future watershed conditions) as well as 100-year solutions conditions.

In addition to showing the differences that the solutions make in the floodplain, the exhibits presented show the structures that are in the baseline and solutions floodplains thusly outlining the problem and the effect of the proposed solution. **Figure 6-A provides a map index that shows the layout of the respective exhibits throughout the city**. The even numbered exhibits provide watershed-specific flood profiles for baseline and post-solution conditions as well as show the difference that the solutions make in the 100-year and 50-year flood profiles. The 50-year profile was included since City design criteria (i.e., no roadway overtopping) for culverts are based on this event. Additionally, these exhibits provide the respective locations of stream erosion and local drainage solutions in the various watersheds. When Table 6-2 is used in conjunction with these exhibits, a clear picture emerges on each project's location, type or character, magnitude, and comparison with other solutions within its respective watershed, its ward(s) as well as the City as a whole. For easy reference, the listing below presents the exhibit numbers for the various watersheds.

	Exhibit Numbers	
Watershed	Plan	Profile
Bishop Creek (Mainstem)	6-1a	6-2
Tributary A	6-1b	6-2a
Tributary B	6-1a	–
Tributary C	6-1a	6-2b
Brookhaven Creek (Mainstem)	6-3	6-4a
Tributary A	6-3	6-4b
Tributary B	6-3	–
Dave Blue Creek (Mainstem)	6-5a	6-6a
Tributary A	6-5a	–
Tributary 1	6-5b	6-6b
Imhoff Creek	6-7a	6-8
Imhoff/Canadian Area	6-7b	–
Little River	6-9	6-10 (reserved)
Tributary G	6-11	6-12
Woodcrest Creek	6-13	6-14
Merkle Creek	6-15	6-16
Rock Creek	6-17a	6-18a
Tributaries A and B	6-17b	–
Tributary C	6-17a	6-18b
Tributary D	6-17c	–
Ten Mile Flat Creek	6-19	–

Watershed	Exhibit Number
Bishop Creek (Mainstem)	6-1a
Tributary A	6-1b
Tributary B	6-1a
Tributary C	6-1a
Brookhaven Creek (Mainstem)	6-3
Tributary A	6-3
Tributary B	6-3
Dave Blue Creek (Mainstem)	6-5a
Tributary A	6-5a
Tributary 1	6-5b
Imhoff Creek	6-7a
Imhoff/Canadian Area	6-7b
Little River	6-9
Tributary G	6-11
Woodcrest Creek	6-13
Merkle Creek	6-15
Rock Creek	6-17a
Tributaries A and B	6-17b
Tributary C	6-17a
Tributary D	6-17c
Ten Mile Flat Creek	6-19



0 3,500 7,000 14,000 Feet

Aerial Photography: 2007
Coordinate System: Oklahoma State Plane, South Zone
Horizontal Datum: NAD 1983
Vertical Datum: NAVD 1988

Legend

City Boundary

Stream Centerlines
- Level 1 and 2 (Detailed)
- Level 3 and 4 (General)

Sheet Outline

NORMAN

PBS&J

Storm Water Master Plan

Figure 6-A
Index Map
Exhibits 6-1a through 6-19

Job No.: 044194100 | Date: 2-16-09 | Scale: 1 inch = 7000 Feet

File: W:/WR/proj/441941_Norman/Report/Figures/Storm_Water_Master_Plan_Index_Map.mxd

Table 6-2
Summary of Proposed Storm Water Projects

Project ID	Watershed	Stream	Ward	Solution Type*	Problem	Solution	100-Yr Floodplain Structures		Flood Prone Road Crossings		Stream Length Stabilized (ft)	Scoring	Watershed Rank	City Rank	Ward Rank	Greenbelt Opportunities	Estimated Cost
							In	Mitigated	In	Protected							
BC-1	Bishop Creek	Bishop Creek	7	E	400 LF of bank erosion located approximately 400 LF upstream of SH 9. 300 LF of the bank erosion is on the left bank of the creek and gets close to an existing parking lot. 100 LF of the bank erosion is on the right bank.	Bank stabilization, MSE wall and rock toe protection.	---	---	---	---	400	71	11	23	5	---	$436,894
BC-2	Bishop Creek	Bishop Creek	7	E	200 LF of severe bank erosion downstream of the confluence of Tributary C and the mainstem. The bank erosion occurs on the left side of the stream.	Bank stabilization, MSE wall, rock riprap protection, and rock toe protection.	---	---	---	---	200	73	9	19	3	---	$353,422
BC-3	Bishop Creek	Bishop Creek	4	FS/FR/G	50-year and 100-year future flows are overtopping the existing three 8-x-4-ft RCB system at Alameda Street. Structures upstream of Alameda Street are in the future 100-year floodplain.	Approximately 220 LF of channel improvements downstream of Alameda Street. Widen channel bottom to 30 LF and side slope = 3:1.	2	1	1	1	---	74	5	13	5	Y	$447,829
BC-4	Bishop Creek	Bishop Creek	4	FB/G	Structures are flooded by the 10-year and 100-year future flows between Symmes Street and Main Street.	Buy 15 structures in the future 10-year floodplain.	49	15	---	---	---	78	2	7	2	Y	$1,846,598
BC-5	Bishop Creek	Trib A to Bishop Creek	7	E/G	300 LF of bank erosion located downstream of Constitution Road. There is severe bed and bank erosion located along the left bank downstream of Constitution. The bank erosion along the right bank occurs approximately 150 LF downstream of Constitution Road.	Bank stabilization, MSE wall and rock toe protection.	---	---	---	---	300	55	15	48	7	Y	$374,045
BC-6	Bishop Creek	Trib A to Bishop Creek	1	FS	Structures located approximately 450 LF northwest of the intersection of Classen Street and 12th SE Street are in the future 100-year floodplain.	Flood protect the structures by building a flood retaining wall on the South and East side of the property.	4	4	---	---	---	58	13	45	6	---	$569,538
BC-7	Bishop Creek	Trib A to Bishop Creek	1	E	Outfall located along the right bank approximately 175 LF upstream of 12th SE Street has failed due to bank erosion around the headwall.	Repair outfall structure.	---	---	---	---	50	52	16	51	8	---	$58,243
BC-8	Bishop Creek	Trib A to Bishop Creek	1	FR/G	10-year, 50-year, and 100-year future flows are overtopping the existing two 72-inch CMP structure at Lindsey Street.	Replace the existing structure with two 10-x-6-ft RCB system.	1	1	1	1	---	75	4	12	2	Y	$450,692
BC-9	Bishop Creek	Trib A to Bishop Creek	1	E	200 LF of bank erosion along the right bank located approximately 400 LF upstream of Lindsey Street.	Bank stabilization and rock toe protection.	---	---	---	---	200	65	12	37	4	---	$63,139
BC-10	Bishop Creek	Trib A to Bishop Creek	1	FS/FR/G	50-year and 100-year future flows are overtopping the existing 10-x-6-ft RCB system at Sinclair Drive and the 8-x-5-ft RCB system at Beaumont Drive. Structures upstream and downstream of Sinclair Drive are in the future 100-year floodplain.	Add one 10-x-6-ft RCB at Sinclair Drive and replace the existing culvert at Beaumont Drive with two 12-x-5-ft RCBs. Approximately 1200 LF of channel conveyance improvement downstream of Beaumont Drive. Proposed channel shall be a benched trapezoidal channel with 3:1 side slopes and 15-ft bottom width.	7	7	2	2	---	80	1	4	1	Y	$1,703,776
BC-11	Bishop Creek	Trib C to Bishop Creek	7	E	200 LF of severe bank erosion and steep bed slope along the right bank located approximately 75 LF upstream of the confluence between Tributary C and the mainstem. The top of the right bank is close to the maintenance building for a local apartment complex.	Bank stabilization, MSE wall, grade control structures, and rock toe protection.	---	---	---	---	200	73	9	19	3	---	$531,505
BC-12	Bishop Creek	Trib C to Bishop Creek	7	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 10-x-4.5-ft RCB system at Brooks Street. Structures located upstream of Brooks Street are located in the future 100-year floodplain.	Replace the existing structure with two 10-x-5-ft RCBs.	6	5	1	1	---	74	5	13	2	---	$329,375

Table 6-2, cont'd

Project ID	Watershed	Stream	Ward	Solution Type*	Problem	Solution	100-Yr Floodplain Structures In	100-Yr Floodplain Structures Mitigated	Flood Prone Road Crossings In	Flood Prone Road Crossings Protected	Stream Length Stabilized (ft)	Scoring	Watershed Rank	City Rank	Ward Rank	Greenbelt Opportunities	Estimated Cost
BC-13	Bishop Creek	Local	1	L/FB	The existing detention pond southeast of 12th Ave SE and Alameda Street intersection is not large enough to detain the existing runoff.	Upsize the existing detention pond to the northeast that is located along Triad Village Drive. Buyout parcel for proposed detention pond (1 parcel).	---	---	---	---	---	74	5	13	3	---	$401,588
BC-14	Bishop Creek	Local	1	L	Two existing ditches located northwest of Tahoe Street and 24th SE Street currently do not contain the existing flows.	1,400 LF of channel conveyance improvement.	---	---	---	---	---	36	17	52	9	---	$30,000
BC-15	Bishop Creek	Local	7	L	The existing ditch between Stinson Road and Fleetwood Road floods frequently.	Ditch conveyance improvement and storm sewer improvements. The proposed ditch shall be a maximum 30-ft top width, with 4:1 side slopes and 10-ft bottom width. The outfall pipe shall be a 36-inch RCP.	---	---	---	---	---	58	13	45	6	---	$292,974
BC-16	Bishop Creek	Local	7	L/G	The existing storm sewer system between College Street and Tributary C to Bishop Creek along Lindsey Street is not adequate to handle the 10-year storm event.	Install a parallel storm sewer system.	---	---	---	---	---	77	3	8	1	Y	$3,628,513
BC-17	Bishop Creek	Local	4	L	The existing two 8-x-4-ft RCB system at Mockingbird Lane is frequently overtopped during rain events.	Replace the existing culvert system with three 8-x-5-ft RCB and raise the roadway elevation by 1.5 feet.	---	---	1	1	---	74	5	13	5	---	$366,981
						Subtotal	**69**	**33**	**6**	**6**	**1350**					**Subtotal**	**$11,885,111**
BHC-1	Brookhaven Creek	Brookhaven Creek	3	FS/FR/FB/E/G	10-year, 50-year, and 100-year future flows are overtopping the existing two 9.5-x-6.4-ft arch pipes at Main Street. Structures located downstream of Main Street are in the future 100-year floodplain. The existing channel for approximately 2,000 LF downstream of Main Street lacks capacity to contain the future 100-year flows.	Replace existing culvert system at Main Street with four 12-x-8-ft RCBs. 2,000 LF of channel improvements and bank stabilization downstream of Main Street. Buyout mobile homes (10 structures). The proposed channel improvements shall include 3:1 side slopes with an additional 20-ft bottom width added to the existing channel. The bank stabilization will require MSE wall, riprap protection, rock toe protection, and rock grade control structures.	276	266	1	1	2,000	84	1	3	1	Y	$3,250,365
BHC-2	Brookhaven Creek	Brookhaven Creek	3	E/G	125 LF of bank erosion on both banks of the channel located approximately 265 LF upstream of Main Street.	Bank Stabilization, rock riprap protection, and rock toe protection.	---	---	---	---	125	69	4	28	2	Y	$101,620
BHC-3	Brookhaven Creek	Brookhaven Creek	3	E/G	225 LF of severe bank erosion along the right bank located approximately 400 LF upstream of Willow Branch Road. Properties located along the right bank are close to the top of bank.	Bank stabilization, MSE wall, rock riprap protection, and rock toe protection.	---	---	---	---	225	69	4	28	2	Y	$156,118
BHC-4	Brookhaven Creek	Brookhaven Creek	3	E/G	800 LF of channel bank erosion located along both banks just downstream of 36th Avenue NW. Approximately 275 LF downstream of 36th Avenue NW the bank erosion gets close to an existing parking lot.	Bank stabilization, MSE wall, rock riprap protection, and rock toe protection.	---	---	---	---	800	69	4	28	2	Y	$593,145
BHC-5	Brookhaven Creek	Brookhaven Creek	8	L/G	Channel underneath Robinson Road is constricted due to concrete riprap rubble.	Channel side slope improvement underneath Robinson Road.	---	---	---	---	---	64	9	38	10	Y	$50,000

Table 6-2, cont'd

Project ID	Watershed	Stream	Ward	Solution Type*	Problem	Solution	100-Yr Floodplain Structures In	100-Yr Floodplain Structures Mitigated	Flood Prone Road Crossings In	Flood Prone Road Crossings Protected	Stream Length Stabilized (ft)	Scoring	Watershed Rank	City Rank	Ward Rank	Greenbelt Opportunities	Estimated Cost
BHC-6	Brookhaven Creek	Brookhaven Creek	8	FR/G	10-year, 50-year, and 100-year future flows are overtopping the existing 60-inch RCP structure at Rock Creek Road.	Add three 60-inch RCP to the existing culvert system.	0	0	1	1	---	70	2	25	5	Y	$254,667
BHC-7	Brookhaven Creek	Trib A to Brookhaven Creek	8	FR/G	50-year and 100-year future flows are overtopping the existing 10-x-7-ft RCB structure at Pendleton Road.	Add one 48-inch RCP to the existing culvert system.	0	0	1	1	---	68	7	32	7	Y	$105,716
BHC-8	Brookhaven Creek	Trib A to Brookhaven Creek	8	FR/G	10-year, 50-year, and 100-year future flows are overtopping the existing 72-inch RCP structure at Rock Creek Road.	Add two 72-inch RCP to the existing culvert system.	0	0	1	1	---	70	2	25	5	Y	$259,009
BHC-9	Brookhaven Creek	Local	8	L	The existing storm sewer system near the Rambling Oaks and Tall Oaks intersection is not adequate.	Increase the size of the existing system to a 60-inch RCP and extend the storm sewer line to outfall at an existing channel.	---	---	---	---	---	61	10	43	12	---	$314,264
BHC-10	Brookhaven Creek	Local	8	L	The existing storm sewer system near the Rambling Oaks and Havenbrook intersection is not adequate.	Increase the size of the existing storm sewer system main trunkline to a 60-inch RCP to carry future flows. The secondary trunklines that tie into the main line shall be 24-inch RCPs.	---	---	---	---	---	67	8	36	9	---	$914,698
						Subtotal	**276**	**266**	**4**	**4**	**3150**					**Subtotal**	**$5,999,601**
CC-1	Clear Creek	Local	5	L	The existing four 36-inch CMP structure at 120th SE Avenue is frequently overtopped during rain events.	Replace the existing primary and secondary roadway culverts with three 10-x-5-ft RCBs and two 10-x-4-ft RCBs respectively. 120th SE Avenue will be raised 2.5 ft to prevent the 10-year future flows from overtopping.	---	---	1	1	---	58	1	45	5	---	$1,794,023
							---	---	1	1						**Subtotal**	**$1,794,023**
CR-1	Canadian River	Local	2	L	The intersection at Westbrooke Terrace Road and Hollywood Street has deep water after heavy rains.	Replace the existing storm sewer system at the intersection with 36-inch RCP and a 7-x-2-ft RCB.	---	---	---	---	---	59	1	44	6	---	$400,645
						Subtotal	---	---	---	---						**Subtotal**	**$400,645**
DBC-1	Dave Blue Creek	Dave Blue Creek	5	FR	10-year, 50-year, and 100-year future flows are overtopping the existing two 10-ft CMPs on the mainstem and the 10-ft CMP on the tributary at 48th Ave SE.	Replace existing road culverts with three 13-x-11-ft RCBs on main stem and three 13-x-11-ft RCBs on tributary. The existing road elevation will be raised 2 ft.	0	0	1	1	---	64	2	38	2	---	$1,542,635
DBC-2	Dave Blue Creek	Trib 1 to Dave Blue Creek	5	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 54-inch CMP at 48th Ave SE.	Replace existing road culvert with two 10-x-6-ft RCBs.	0	0	1	1	---	68	1	32	1	---	$244,098
						Subtotal	**0**	**0**	**2**	**2**						**Subtotal**	**$1,786,733**
IC-1	Imhoff Creek	Imhoff Creek	2	E	800 LF of bank erosion on both banks downstream of SH 9. The erosion along the banks have caused trees to fall into the creek.	Bank stabilization and rock toe protection.	---	---	---	---	800	79	2	5	2	---	$253,418
IC-2	Imhoff Creek	Imhoff Creek	2&4	E	4,200 LF of severe bank erosion along both banks beginning at the upstream face of SH9 to approximately 2,000 LF upstream of Imhoff Rd. The erosion along the banks have caused property fences and trees to fall into the creek.	Bank stabilization, MSE wall, rock riprap protection, rock grade controls, and rock toe protection.	---	---	---	---	4,200	79	2	5	2&1	---	$6,563,091

Table 6-2, cont'd

Project ID	Watershed	Stream	Ward	Solution Type*	Problem	Solution	100-Yr Floodplain Structures In	100-Yr Floodplain Structures Mitigated	Flood Prone Road Crossings In	Flood Prone Road Crossings Protected	Stream Length Stabilized (ft)	Scoring	Watershed Rank	City Rank	Ward Rank	Greenbelt Opportunities	Estimated Cost
IC-3A	Imhoff Creek	Imhoff Creek	4	FS/FR/G	Reach from Elmwood Drive dead end to Madison Street dead end. The 10-year storm event and larger events are overtopping the existing three 8-x-6-ft RCB culvert system at Lindsey Street. Structures are located within the future 100-year floodplain due to lack of channel capacity.	Replace the existing culvert system at Lindsey Street with a 20-inch-deep box beam bridge consisting of two-30 ft. spans. The proposed channel varies from a 1.5:1 side slope with a 15- to 20-ft bottom width to vertical mortared rock banks with a 40-ft bottom width or rock/earth channel equivalent. Benched overbanks are proposed when adequate space is provided.	14	11	1	1	---	74	6	13	5	Y	$2,613,208
IC-3B	Imhoff Creek	Imhoff Creek	4	FS/FR/G	Reach from Madison Street dead end to a location approximately 150 LF downstream of W. Boyd Street. Storm events larger than the 10-year event are overtopping the existing 30-x-8.5-ft concrete lined slab bridge at Brooks Street. Structures are located within the future 100-year floodplain due to lack of channel capacity.	Replace the existing structure at Brooks Street with a 20-inch-deep box beam bridge consisting of 1-50 ft span. The proposed channel varies from a 1.5:1 side slope with a 20-ft bottom width to a transition to vertical walls with concrete bottom and mortared rock walls and a 30-ft bottom width or rock/earth channel equivalent.	32	19	1	1	---	74	6	13	5	Y	$3,722,131
IC-3C	Imhoff Creek	Imhoff Creek	4	FS/FR/G	Reach from 150 LF downstream of W. Boyd Street to just below McNamee Street. The 10-year storm event and larger events are overtopping the existing 12-x-6-ft slab bridge at Boyd Street and the 12-x-5-ft slab bridge at Pickard Street. Structures are located within the future 100-year floodplain due to lack of channel capacity.	Replace the existing structure at Boyd Street with a 20-inch-deep box beam bridge consisting of 1-50 ft span and the existing structure at Pickard Street with four 10-x-6-ft RCB culvert system. The proposed channel will be expanded to a bottom width of 40 ft. The sides shall be constructed as mortared rock, WPA-type channel or rock/earth channel equivalent.	13	6	2	2	---	74	6	13	5	Y	$3,158,147
IC-3D	Imhoff Creek	Imhoff Creek	4	FS/FR/FB/G	Reach from just downstream of McNamee Street to just upstream of Symmes Street. The 10-year storm event and larger events are overtopping the existing McNamee Street (12-x-5-ft slab bridge), Flood Avenue (15-x-5-ft slab bridge), and Symmes Street (15-x-5-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	Replace the existing culvert systems at McNamee Street (four 10-x-6-ft RCB), Flood Avenue (three 10-x-6-ft RCB), and Symmes Street (three 10-x-6-ft RCB). The proposed channel will be expanded to a bottom width of 30 ft. The sides shall be constructed as mortared rock, WPA-type channel or rock/earth channel equivalent. Proposed buyouts upstream of Flood Avenue (4 structures).	29	17	3	3	---	74	6	13	5	Y	$3,191,106

Table 6-2, cont'd

Project ID	Watershed	Stream	Ward	Solution Type*	Problem	Solution	100-Yr Floodplain Structures In	100-Yr Floodplain Structures Mitigated	Flood Prone Road Crossings In	Flood Prone Road Crossings Protected	Stream Length Stabilized (ft)	Scoring	Watershed Rank	City Rank	Ward Rank	Greenbelt Opportunities	Estimated Cost
IC-3E	Imhoff Creek	Imhoff Creek	4	FS/FR/FB/G	Reach from just upstream of Symmes Street to just downstream of Main Street. The 10-year storm event and larger events are overtopping the existing school footbridge (10-x-6-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	Replace the existing culvert system at the school footbridge with a 20-inch-deep box beam bridge consisting of one 30-ft span. The proposed channel will be expanded to a bottom width of 30 ft. The sides shall be constructed as mortared rock, WPA-type channel or rock/earth channel equivalent. Proposed buyouts throughout the reach (12 structures).	25	21	0	0	---	74	6	13	5	Y	$3,459,651
IC-3F	Imhoff Creek	Imhoff Creek	4	FR/G	Reach from just downstream of Main Street to just upstream of Main Street. The 10-year storm event and larger events are overtopping the existing 12-x-5.5-ft slab bridge at Main Street. Structures located upstream of this reach are within the future 100-year floodplain due to lack of channel capacity.	Replace the existing structure at Main Street with three 10-x-6-ft RCBs and a channel bottom lowered by two ft.	0	0	1	1	---	74	6	13	5	Y	$1,645,157
IC-3G	Imhoff Creek	Imhoff Creek	4	FS/FR/FB/G	Reach from just upstream of Main Street to just upstream of W. Tonhawa Street. The 10-year storm event and larger events are overtopping the existing W. Gray Street (10-x-5-ft slab bridge), N. Lahoma Street (10-x-5.1-ft slab bridge), and W. Tonhawa Street (10-x-5-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	Replace the existing culverts at W. Gray Street (three 9-x-5-ft RCBs), N. Lahoma Street (three 9-x-5-ft RCBs), and W. Tonhawa Street (three 7-x-5-ft RCBs). The proposed channel will be expanded to a bottom width of 25 to 30 ft. The sides shall be constructed as mortared rock, WPA-type channel or rock/earth channel equivalent. Proposed buyouts upstream of W. Gray Street (3 structures).	22	12	3	3	---	74	6	13	5	Y	$1,658,975
IC-3H	Imhoff Creek	Imhoff Creek	4	FS/FR/FB/G	Reach from just upstream of W. Tonhawa Street to just upstream of N. Webster Avenue. The 10-year storm event and larger events are overtopping the existing W. Daws Street (10-x-4-ft slab bridge), N. University Boulevard (10-x-4-ft slab bridge), N. Park Avenue (10-x-3.5-ft slab bridge), and N. Webster Avenue (10-x-3-ft slab bridge). Structures are located within the future 100-year floodplain due to lack of channel capacity.	Replace the existing culvert systems at W. Daws Street (three 7-x-4-ft RCBs), N. University Boulevard (three 7-x-4-ft RCBs), N. Webster Avenue (three 7-x-3-ft RCBs). The proposed channel will be expanded to a bottom width of 25 ft. The sides shall be constructed as mortared rock, WPA-type channel or rock/earth channel equivalent. Proposed buyouts throughout reach (2 structures).	64	48	4	4	---	74	6	13	5	Y	$1,474,082
IC-4	Imhoff Creek	Imhoff Creek	4	FS/FR/FB/G	There are flooded buildings and road culvert systems along the Imhoff Creek stream corridor due to increasing development over the years and lack of channel capacity to contain the flows.	Opt 1: A proposed 9 acre detention pond in Andrews Park. Buyouts for proposed detention pond (5 parcels).	360	---	15	---	---	76	5	11	4	Y	$2,126,249
IC-4A	Imhoff Creek	Imhoff Creek	4&8	FS/FR/FB	There are flooded buildings and road culvert systems along the Imhoff Creek stream corridor due to increasing development over the years and lack of channel capacity to contain the flows.	Opt 2: A proposed 9 acre detention pond in Andrews Park plus additional detention storage North of park. Buyouts for proposed detention pond (8 parcels).	360	131	15	---	---	77	4	8	3&2	---	$3,517,101

Table 6-2, cont'd

Project ID	Watershed	Stream	Ward	Solution Type*	Problem	Solution	100-Yr Floodplain Structures In	100-Yr Floodplain Structures Mitigated	Flood Prone Road Crossings In	Flood Prone Road Crossings Protected	Stream Length Stabilized (ft)	Scoring	Watershed Rank	City Rank	Ward Rank	Greenbelt Opportunities	Estimated Cost
IC-5	Imhoff Creek	Local	2	L/G	The intersection at Lindsey Street and McGee Drive and Lindsey Street heading East flood after moderate storm events.	Proposed storm sewer diversion to carry the 10-year storm beginning at the Lindsey Street/McGee Street intersection and outfalling into the Canadian River.	---	---	---	---	---	89	1	1	1	Y	$12,461,087
						Subtotal	**360**	**265**	**15**	**15**	**5000**					**Subtotal**	**$43,717,155**
LR-1	Little River	Little River	6	E	350 LF of severe bank erosion along the right bank located approximately 2,000 LF upstream of 12th NE Avenue. The bank erosion is approximately 70 LF from a residential structure.	Bank stabilization, rock bendway weir structures, and rock toe protection.	---	---	---	---	350	74	2	13	1	---	$123,682
LR-2	Little River	Little River	8	FB	There are approximately 40 mobile homes within the future 100-year floodplain located West of the BNSF Railroad and North of Indian Hill Road.	Buyout all mobile homes.	40	40	---	---	---	88	1	2	1	---	$305,233
						Subtotal	**40**	**40**	**0**	**0**	**350**					**Subtotal**	**$428,915**
TGLR-1	Trib G to Little River	Trib G to Little River	8	FR/G	10-year, 50-year, and 100-year future flows are overtopping the existing 10.5-x-7-ft CMP pipe arch culvert system at Franklin Street.	Replace existing road crossing culverts with five 10-x-10-ft RCBs. The proposed roadway elevation will be raised 1-x-1.5-ft at the roadway crossing.	0	0	1	1	---	72	1	22	4	Y	$992,182
						Subtotal	**0**	**0**	**1**	**1**						**Subtotal**	**$992,182**
WC-1A	Woodcrest Creek	Woodcrest Creek	6	FS/G	There are flooded buildings and road culvert systems along the Woodcrest Creek stream corridor due to increasing development over the years and lack of channel capacity to contain the flows.	Proposed regional storm water detention facility located upstream of Rock Creek Road.	20	4	2	2	---	70	2	25	3	Y	$2,501,285
WC-1B	Woodcrest Creek	Woodcrest Creek	6	FS/G	The existing channel downstream of Sequoyah Trail lacks the capacity to contain the future flows. Several buildings along the right side of the stream corridor and one on the left are in the 100-year future floodplain.	Increase the capacity of the existing channel for approximately 1,200 LF downstream of Sequoyah Trail. The proposed channel shall be 3:1 side slopes and benched on the right side of the modified channel.	10	10	---	---	---	69	3	28	4	Y	$525,290
WC-2	Woodcrest Creek	Woodcrest Creek	6	FR/G	10-year, 50-year, and 100-year future flows are overtopping the existing two 8-x-7-ft RCBs at Sequoyah Trail.	Add one 8-x-7-ft RCB to the existing culvert system if the upstream WC-1A detention pond not constructed. Provides 10-year protection without the WC-!A solution.	2	1	1	1	---	71	1	23	2	Y	$140,591
WC-3	Woodcrest Creek	Woodcrest Creek	6	E	200 LF of bank erosion along both banks in the park south of Sequoyah Trail.	Bank stabilization, outfall repair, rock riprap protection, and rock toe protection.	---	---	---	---	200	68	4	32	5	---	$110,965
						Subtotal	**20**	**15**	**3**	**3**	**200**					**Subtotal**	**$3,278,130**
MC-1	Merkle Creek	Merkle Creek	2	FS/G	There are structures on both sides of the stream corridor located upstream of 24th Street in the future 100-year floodplain. There are currently three 10-x-11-ft RCBs underneath 24th Street.	Add a 10-x-11-ft RCB and 135 LF of channel conveyance improvements downstream of 24th Street. The proposed channel shall be 3:1 side slopes and 30- to 50-ft bottom width.	15	8	0	0	---	73	2	19	5	Y	$649,869

Table 6-2, cont'd

Project ID	Watershed	Stream	Ward	Solution Type*	Problem	Solution	100-Yr Floodplain Structures In	100-Yr Floodplain Structures Mitigated	Flood Prone Road Crossings In	Flood Prone Road Crossings Protected	Stream Length Stabilized (ft)	Scoring	Watershed Rank	City Rank	Ward Rank	Greenbelt Opportunities	Estimated Cost
MC-2	Merkle Creek	Merkle Creek	2&8	FS/FB/G	Crestmont and Iowa Streets are being overtopped by the 10-year, 50-year, and 100-year future flows due to backwater from the existing three 10-x-11.5-ft RCB system at Main Street. There are structures upstream of Main Street in the future 100-year floodplain.	Replace existing road culvert system with three 12-x-12-ft RCBs and 1,500 LF of channel conveyance improvements upstream of Main Street and 300 LF downstream of Main Street. The proposed channel shall be 3:1 side slopes, benched in areas, and 15- to 18-ft bottom width. Proposed buyouts (4 structures).	14	8	0	0	---	77	1	8	4&2	Y	$6,066,932
MC-2A	Merkle Creek	Merkle Creek	8	FR/FB	10-year, 50-year, and 100-year future flows are overtopping the existing three 10-x-7.5-ft RCB at Crestmont Street.	Replace existing road structure with three 12-x-8-ft RCBs. Proposed buyouts (2 structures).	21	14	1	1	---	68	3	32	7	---	$1,752,070
MC-2B	Merkle Creek	Merkle Creek	8	FR	10-year, 50-year, and 100-year flows are overtopping the existing two 10-x-5-ft RCBs at Iowa Street.	Replace existing road structure with three 11-x-6-ft RCBs.	1	1	1	1	---	64	4	38	10	---	$387,687
						Subtotal	**51**	**31**	**2**	**2**						**Subtotal**	**$8,856,558**
RC-1	Rock Creek	Rock Creek	5	FR	10-year, 50-year, and 100-year future flows are overtopping the existing two 9-ft CMP culverts at Robinson Road.	Replace existing road structure with three 14-x-11-ft RCBs.	1	1	1	1	---	63	1	41	3	---	$1,169,349
RC-2	Rock Creek	Rock Creek	1&5	FR	10-year, 50-year, and 100-year future flows are overtopping the existing 10-ft RCP culvert at 36th Avenue NE.	Replace existing road structure with five 10-x-10-ft RCBs. The proposed roadway elevation will be raised approximately 1.5 ft at the culvert crossing.	0	0	1	1	---	63	1	41	5&3	---	$1,057,541
RC-3	Rock Creek	Trib C to Rock Creek	1&5	FS/FR	10-year, 50-year, and 100-year future flows are overtopping the existing 6-ft CMP culvert at 36th Ave NE.	Replace existing road structure with three 72-inch RCPs and 500 LF of channel conveyance improvements downstream of 36th Ave NE. The proposed roadway elevation will be raised 2.29 ft just north of the culvert crossing. The proposed channel improvements shall be 3:1 side slopes and 10- to 14-ft bottom width.	1	1	1	1	---	54	3	50	7&6	---	$909,221
						Subtotal	**2**	**2**	**3**	**3**						**Subtotal**	**$3,136,111**
TMF-1	Ten Mile Flat Creek	Local	3	L	The earthen channel through Cambridge Addition West of 48th Avenue NW and North of Main Street is undersized. The 100-year flows have been known to extend into property owners' backyards.	Increase channel capacity by reconstructing channel with 5:1 side slopes and 20-ft bottom width.	---	---	---	---	---	55	1	48	5	---	$255,326
						Subtotal	**---**	**---**	**---**	**---**						**Subtotal**	**$255,326**
						Totals	**830**	**652**	**36**	**36**	10,050					**Total (Min)**	**$81,139,638**
																Total (Min)	**$82,530,490**

* Solution Types:

FB – Flooded Structure Buyouts
E – Stream Erosion Stabilization
FR – Flooded Mitigation - Road Crossing Upgrade
FS – Flood Mitigation - Stream Capacity Increase and/or Flood Detention
G – Greenbelt Opportunity
L – Local Drainage Improvements

Discussion beyond that provided above, in Tables 6-1 and 6-2, and in the plan and profile descriptions of the proposed solutions (Exhibits 6-1 through 6-19) is provided below for some of the more significant solutions organized by the various City watersheds. *The stream flooding and stream erosion solutions developed are only for the Level 1 and Level 2 stream reaches studied. Water quality solutions are more programmatic and generally apply broadly across the City as a whole. Localized solutions are scattered throughout the watershed beyond the Level 1 and 2 reaches.*

Bishop Creek

With 17 individual problem areas, Bishop Creek also has that same number of solutions which exceeds the totals in any of the other respective watersheds. These solutions are discussed in Table 6-2 with results shown in Exhibits 6-1a, 6-1b, 6-2a, and 6-2b that cover the range of problem types discussed in Section 5. The proposed solutions in the watershed collectively provide protection for and/or removal of, 33 of the 69 buildings/structures in the baseline floodplain, the six flood prone road crossing structures, and 1,350 ft of eroding stream length. Only four of the 17 solutions occur along the mainstem of Bishop Creek, with six along Tributary A, two within Tributary C, as well as five in various localized areas.

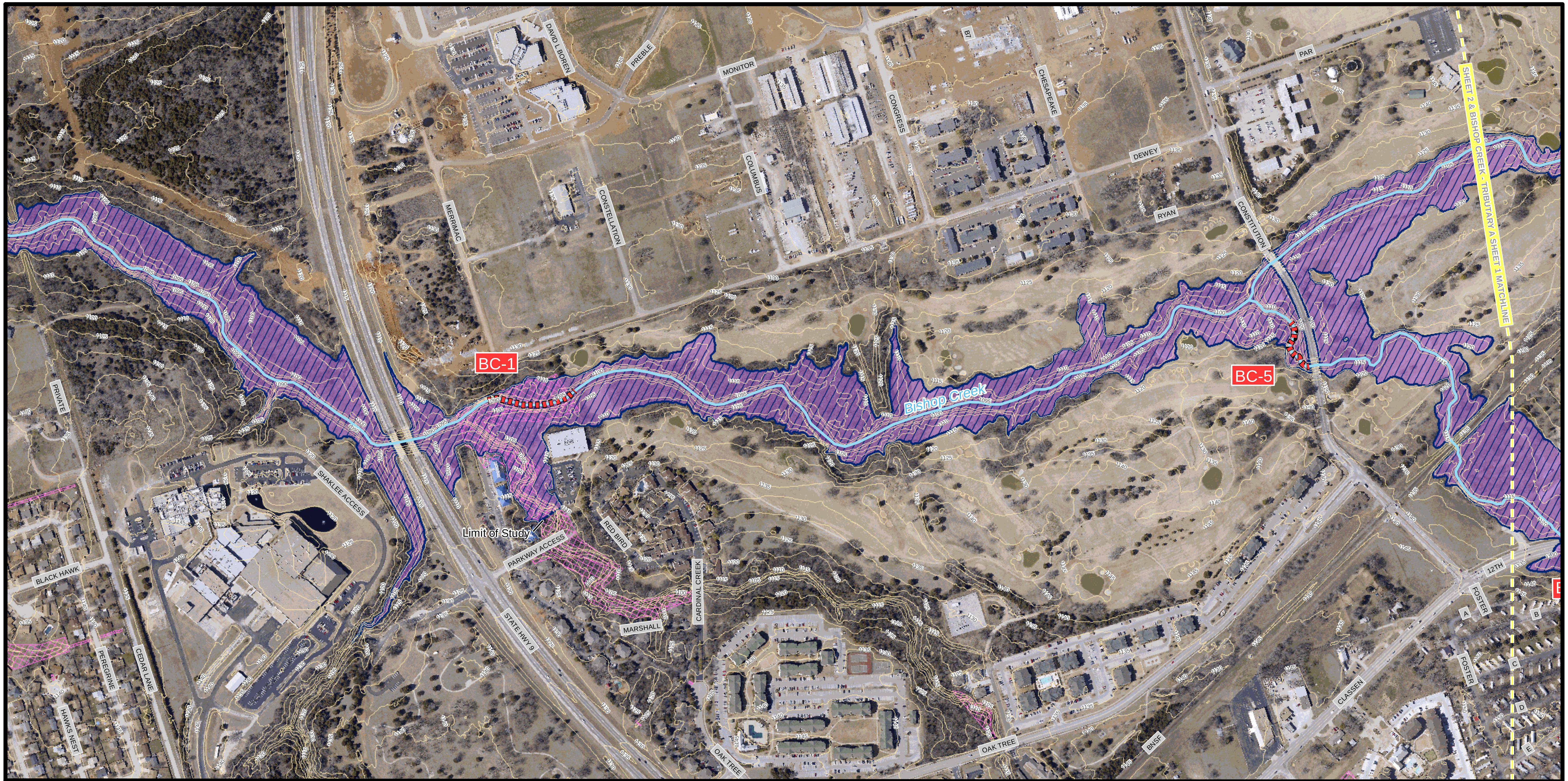
The most significant solution located along the mainstem is BC-4, a stream flooding problem, in which the selected solution calls for 15 of 49 homes to be bought out since many of them flood as a result of small and medium flood events such as the 10-year event. The small mortared rock channel in this upper reach of Bishop Creek is significantly undersized and the floodplain is very flat so overflows spread out over a relatively wide floodplain area. Any channel conveyance improvements would have to be very wide and costly due to the shallow channel and flat overbank area so a solution to buyout the most flood prone 15 structures was selected. By removing these 15 structures that are in the primary flow path of flooding events, the flood levels could be reduced somewhat which will also lessen flooding on the remaining structures. It is recognized that buying out properties is a difficult process and involves significant time, effort, and costs to complete. Therefore, this method of flood protection was used sparingly for this solution and only targeted 15 out of 49 flooded structures for buyout. These 15 structures are those that are the most flood prone in the area and flood significantly from the 10-year future conditions event.

As described in Table 6–2, the solution for BC-10 along Tributary A consists of channel enlargement downstream of Beaumont Drive and the upgrading of road crossing openings at Beaumont Drive and Sinclair Drive. These improvements effectively remove seven homes from the baseline (100-year) floodplain located upstream of the road crossings. Exhibit 6–1a shows the reduction in the baseline floodplain and Exhibit 6-2a displays how the improvements effectively reduce the flood levels at Beaumont Drive by about 4 ft and by over 2 ft at Sinclair Drive preventing overtopping of the roadway crossings. As shown in Exhibits 6-1a and 6-2b, the BC-12 solution along Tributary C at Brooks Street involves enlarging the culvert system as specified in Table 6-2 which reduces flood levels by over 2 ft and removes five of the six flooded apartment buildings located upstream of the road crossing from the baseline (100-year) floodplain.

Stream erosion stabilization solutions BC-1, BC-2, BC-5, BC-7, BC-9, and BC-11 were developed for these six individual locations. Channel widening and/or down cutting in these areas have left unstable channels and the solutions address each of these problems by stabilizing the bank and bottom, where needed, utilizing natural channel design and bio-engineering techniques. More specifically, the stabilization techniques utilized laying back channel side slopes to a more stable angle (3:1 horizontal to vertical) or using mechanically stabilized structures that use geogrid soil reinforcement depending on the situation and the local restraints. These design techniques are discussed subsequently in Section 6.2 which discusses methodologies for developing solutions.

Stream erosion threatening wastewater infrastructure

Stream stabilized with MSE design; wastewater infrastructure protected



0 250 500 1,000 Feet

Aerial Photography: 2007
Coordinate System: Oklahoma State Plane, South Zone
Horizontal Datum: NAD 1983
Vertical Datum: NAVD 1988

Legend

City Boundary
Existing Drainage Easement

Stream Centerlines
- Level 1 and 2 (Detailed)
- Level 3 and 4 (General)

Floodplains
- 100-year Baseline
- 100-year Solution

Buildings in Floodplain
- 100-year Baseline
- 100-year Solution

Recommended Solutions
- Road Crossing Upgrade
- Property Buyouts
- Floodwall
- Channel Stabilization
- Channel Improvements
- Storm Sewer Improvements
- Storm Water Detention

PBS&J

Storm Water Master Plan

Exhibit 6-1a

Baseline Floodplain and Recommended Solutions Overview Bishop Creek Plus Tributaries B and C

Sheet 1 of 5

Job No.: 044194100	Date: 12-11-08	Scale: 1 inch = 500 Feet

File: W:/WR/proj/441941_Norman/Report/Figures/Bishop_1.mxd