CLEAR RUN OUTFALL DRAINAGE IMPROVEMENT STUDY

PREPARED FOR:

CITY OF WILMINGTON

PREPARED BY:

MOFFATT & NICHOL
1616 E. MILLBROOK ROAD, SUITE 160
RALEIGH, NC 27609

EXECUTIVE SUMMARY

The City of Wilmington requested a preliminary engineering study for the existing storm sewer outfall network located in the vicinity of New Centre Drive and South College Road. The areas served by the storm network have experienced significant periodic flooding which is a concern for residents and commercial tenants in the area, as well as the City of Wilmington. The site is located in the Cape Fear River Basin.

The Clear Run outfall network consists of catch basins that discharge into pipe systems along New Centre Dr., S. College Rd., College Acres Dr. and Racine Drive. The street-side storm networks drain to the main outfall system which runs east parallel to the south side of Clear Run Drive. The main outfall consists of a combination of underground storm sewers, natural open channels and road crossing culverts. The network outfall discharges to a floodplain at a point east of the intersection of Clear Run Dr. and Mallard Street.

Moffatt & Nichol performed hydrologic and hydraulic modeling of the existing system, which is currently unable to convey the 25-year storm. Several alternatives were examined to reduce flooding at specific areas of concern within the study area. The probable costs, construction feasibility and hydraulic performance for each alternative were taken into consideration when selecting the preferred alternative.

The preferred solution, **Alternative 3**, installs/upgrades a pipe system west to east along New Centre Drive, north to south along Racine Drive, and includes improvements to the natural stream channels, a culvert upgrade underneath an existing apartment complex, and upgrades for (3) three road culvert crossings. This solution provides additional capacity to the existing system, and diverts flow away from the South College Road pipe network and the upstream portion of the main line run. This alternative lessens the potential for traffic congestion by minimizing construction within the major thoroughfare ROW of South College Road, and simultaneously lessens concerns of construction feasibility compared to other alternatives.

The probable cost for **Alternative 3 is $9,570,000.**

The GIS inventory survey provided for this phase of design was sufficient for preliminary and conceptual design and selection of a preferred alternative. The construction level design will require collection of additional survey data and detailed modeling to ensure the viability of the chosen alternative. Full topographical and existing utility survey will be required along the alignment of the proposed improvements for construction level design.

In conjunction with the topographical and existing utility survey, collection of feature elevation data for select BMP's and underground storm pipes will be needed to supplement detailed hydraulic modeling and construction-level design. The additional survey data will ensure that the final hydraulic model is more accurate and that the proposed

improvements are not under or over-designed. The additional information will allow the engineer to modify the final design to introduce cost savings where applicable. The cost savings potential for the final design outweighs the expenditure required to compile the additional data.

TABLE OF CONTENTS

I. PROJECT BACKGROUND 1
II. SCOPE OF WORK 3
III. DATA COLLECTION 3
A. Aerial Photography 3
B. Topography 3
C. Stormwater Infrastructure 5
D. Field Survey Data 5
E. Utilities 7
IV. FIELD OBSERVATIONS 8
V. MODELING OF HYDRAULIC / HYDROLOGIC BEHAVIOR 8
A. Modeling Scope 8
B. Existing Conditions 9
C. Existing Model Results & Discussion 20
D. Proposed Alternatives 21
VI. OPINIONS OF PROBABLE COST 46
VII. SELECTION OF PREFERRED ALTERNATIVE 49
VIII. HYDRAULIC & HYDROLOGIC IMPROVEMENT SUMMARY 50

APPENDICES

A. XPSWMM NODE/LINK DATA SUMMARY

B. ALTERNATIVE COST SUMMARIES

I. LIST OF FIGURES

FIGURE 1: CLEAR RUN OUTFALL VICINITY MAP 2
FIGURE 2: CLEAR RUN OUTFALL TOPOGRAPHY 4
FIGURE 3: EXISTING STORMWATER INFRASTRUCTURE 6
FIGURE 4: CLEAR RUN OUTFALL DRAINAGE SUB-BASINS 10
FIGURE 5: DURATION FLOODING/SURCHARGE FOR 10-YEAR RAINFALL EVENT - EXISTING 14
FIGURE 6: DEPTH OF FLOODING FOR 10-YEAR RAINFALL EVENT- EXISTING 15
FIGURE 7: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT - EXISTING 16
FIGURE 8: DEPTH OF FLOODING 25-YEAR RAINFALL EVENT - EXISTING 17
FIGURE 9: DURATION FLOODING/SURCHARGE FOR 50-YEAR RAINFALL EVENT - EXISTING 18
FIGURE 10: DEPTH OF FLOODING 50-YEAR RAINFALL EVENT - EXISTING 19
FIGURE 11: IMPROVEMENTS FOR MAIN LINE RUN - PARALLEL SYSTEM – ALTERNATIVE 1 26
FIGURE 12: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 1 27
FIGURE 13: DEPTH OF FLOODING 25-YEAR RAINFALL EVENT – ALTERNATIVE 1 28
FIGURE 14: IMPROVEMENTS FOR MAIN LINE RUN – UPGRADE/REPLACEMENT – ALTERNATIVE 2 32
FIGURE 15: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 2 33
FIGURE 16: DEPTH OF FLOODING 25-YR RAINFALL EVENT– ALTERNATIVE 2 34
FIGURE 17: IMPROVEMENTS FOR NEW CENTRE DIVERSION – UPGRADE/REPLACEMENT – ALTERNATIVE 3 38
FIGURE 18: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 3 39
FIGURE 19: DEPTH OF FLOODING 25-YR RAINFALL EVENT – ALTERNATIVE 3 40
FIGURE 20: IMPROVEMENTS FOR NEW CENTRE DIVERSION & PINCH POINTS – ALTERNATIVE 4 43
FIGURE 21: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 4 44
FIGURE 22: DEPTH OF FLOODING 25-YR RAINFALL EVENT – ALTERNATIVE 4 45
FIGURE 23: NODE REFERENCE MAP ..... APPENDIX A

II. LIST OF TABLES

TABLE 1: 24-HOUR DURATION RAINFALL DEPTHS FOR SELECTED RETURN EVENTS 9
TABLE 2: OPINIONS OF PROBABLE COST 47
TABLE 3A: HYDRAULIC AND HYDROLOGIC IMPROVEMENT SUMMARY – OVERALL NETWORK 50
TABLE 3B: HYDRAULIC AND HYDROLOGIC IMPROVEMENT SUMMARY – RACINE & NEW CENTRE 50
TABLE 3C: HYDRAULIC AND HYDROLOGIC IMPROVEMENT SUMMARY – NEW CENTRE (WEST OF COLLEGE) 5250

I. PROJECT BACKGROUND

The Clear Run Outfall Drainage Improvement Study was completed for the City of Wilmington, North Carolina. The study area lies within the Cape Fear River Basin. The drainage area within the study limits is approximately 468 acres (**See Figure 1**), which drains to Bradley Creek, and ultimately to the Intracoastal Waterway. The study area consists mostly of highly impervious commercial and multi-family residential zones, as well as some lesser impervious single-family residential and recreational open space zones. The primary areas of concern for flooding are within the upstream section of the outfall, located North and West of College Acres Drive. This is the portion of the drainage basin consisting of the majority of the commercial and multi-family developments, which drain through a combination of open and closed conveyances. The downstream section of the drainage basin consists mainly of open, natural channel reaches, within which only two closed drainage structures (culvert crossings under roads) are subject to flooding.

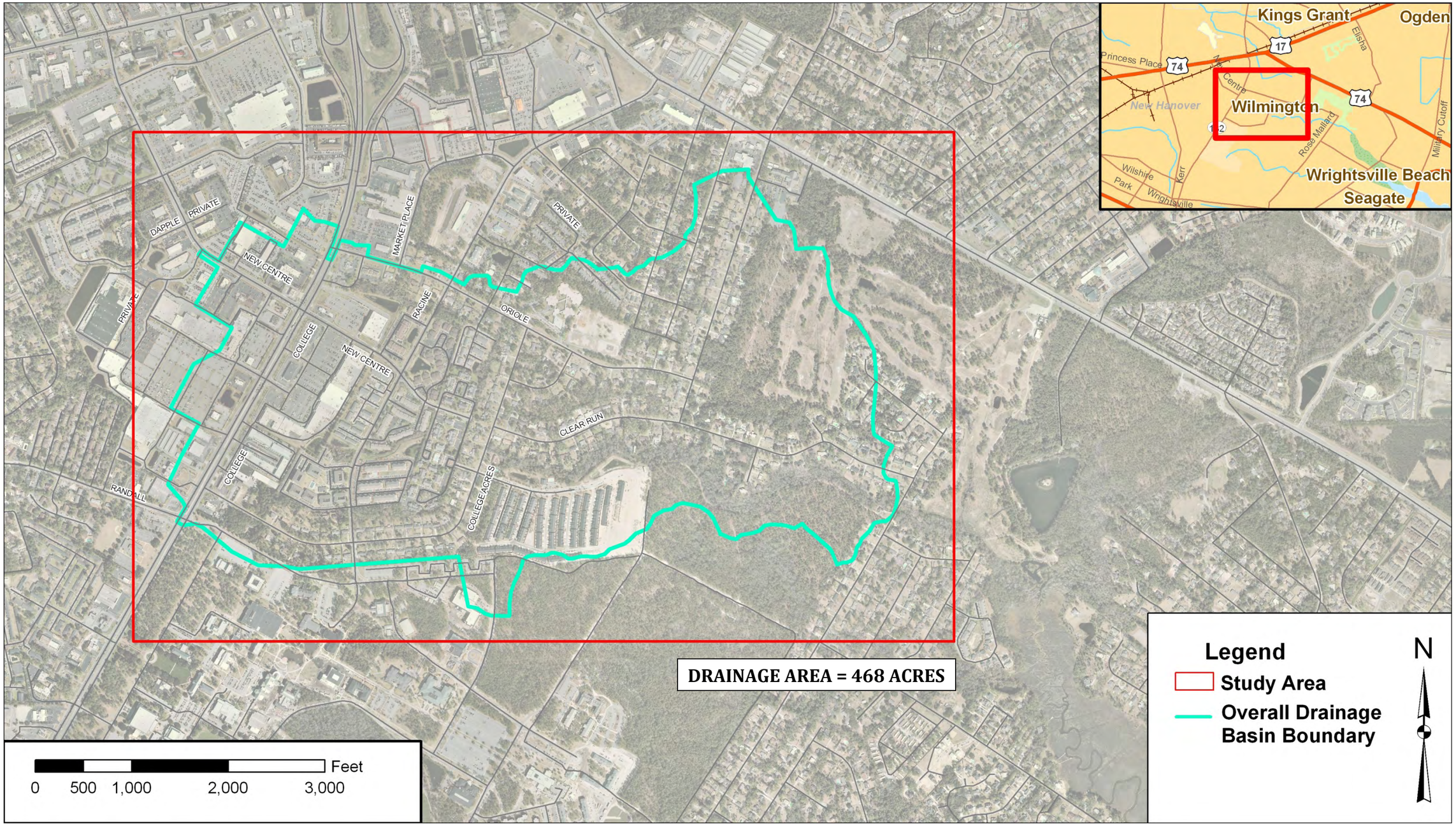

FIGURE 1: CLEAR RUN OUTFALL VICINITY MAP

II. SCOPE OF WORK

The Clear Run watershed has experienced significant periodic flooding, which is a concern for the City and for residents. This has prompted the City of Wilmington to authorize a hydrologic and hydraulic (H&H) study of the existing system to analyze the causes of flooding and propose viable alternatives. The viability of alternatives at the conceptual stage of planning and design is driven primarily by hydraulic performance and projected estimates of cost. The drainage study utilized the XPSWMM modeling software to develop the H&H model using field data collected in 2010 and the existing stormwater inventory provided by the City of Wilmington. The model was used to analyze the existing system conditions for the 10-, 25- and 50-yr rainfall events. The results were used to support preliminary design efforts to ensure that the system improvements in the proposed alternatives would accommodate the 25-year rainfall event. Opinions of probable costs were taken into consideration along with hydraulic performance results when selecting the preferred improvement alternative. A higher level engineering analysis coupled with comprehensive survey will be required to determine the construction feasibility of the proposed alternatives. However, key assumptions regarding anticipated construction limitations were used to aid the development of preliminary design alternatives.

III. DATA COLLECTION

Data was available for the Clear Run Outfall study from various sources. Aerial photography, topography, stormwater infrastructure, and field surveys were used to produce an existing conditions model in XPSWMM.

A. AERIAL PHOTOGRAPHY

Aerial photography from 2006 was downloaded from the NC OneMap website (http://www.nconemap.com/). The aerials were obtained as SID files with associated world files for use in GIS. The aerials were projected in NAD 1983 State Plane (feet).

B. TOPOGRAPHY

Bare Earth data was downloaded from the NC Floodplain Mapping website (http://www.ncfloodmaps.com/). This data was received in an ASCII file format and was used to create a TIN surface in GIS. The vertical datum of the bare earth files was NAVD 88 (feet) and the horizontal datum was NAD 83 State Plane (feet). Two foot contours were created from the TIN surface. A representation of the topographic data is shown in **Figure 2**.

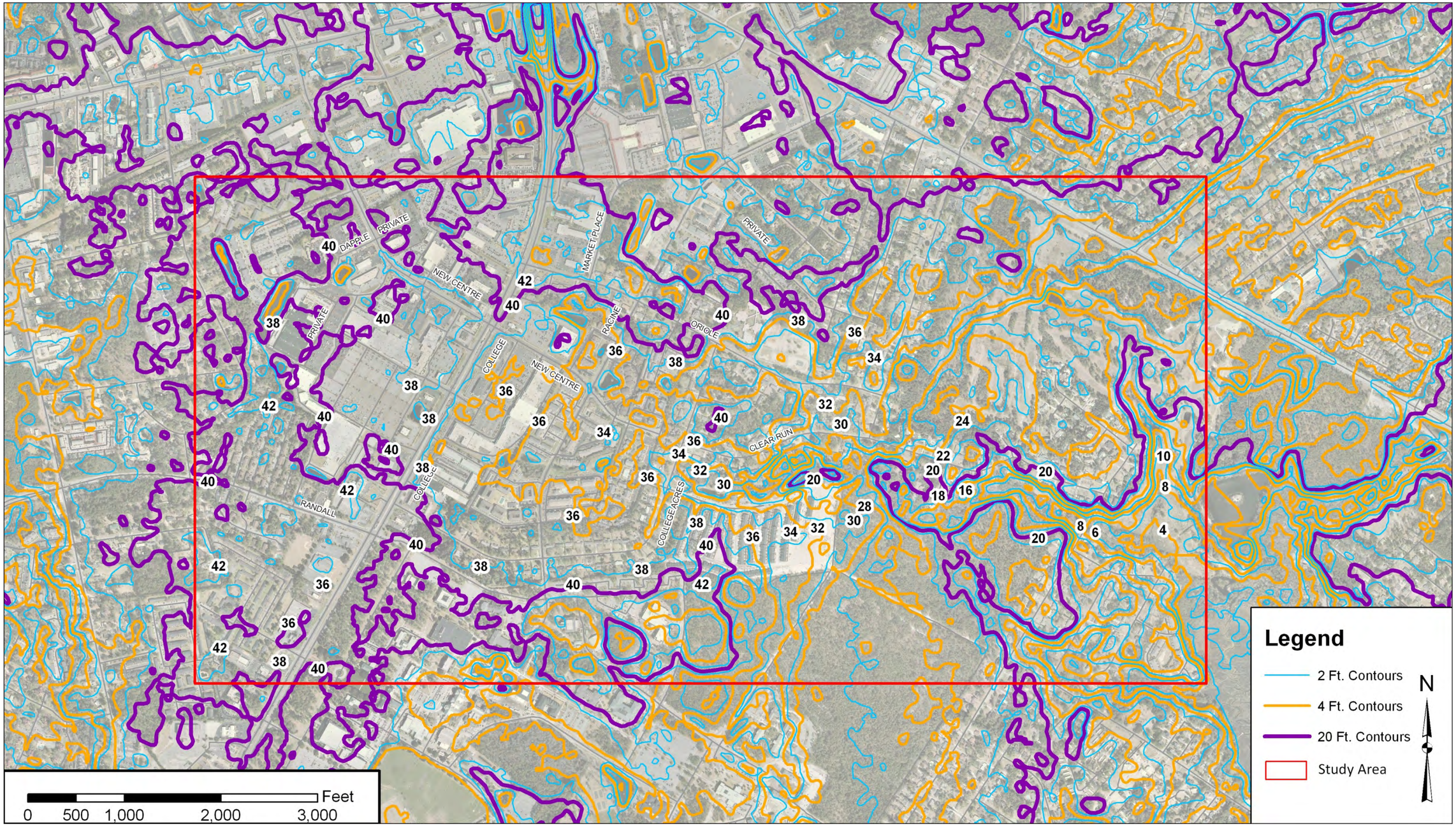

FIGURE 2: CLEAR RUN OUTFALL TOPOGRAPHY

C. STORMWATER INFRASTRUCTURE

The City of Wilmington provided GIS survey of the stormwater infrastructure via ESRI GeoDatabase, which included stormwater pipes, channels, inlets, manholes, junction boxes, and outlets. Attributes for the conduits (pipes and channels) included: length, size of pipe, type of material, top width, bottom width, and side slopes for channels. Attributes for stormwater junctions included: inlet type, structure depth, and type of material. Approximately 70% of the conduits and junctions had upstream and downstream "depth from surface" information provided. Assumptions of depths were made where that information was not available. These assumptions were made such that drainage features had positive slope from the upstream to downstream end. Surrounding topography and adjacent stormwater features with known data were used to assume missing data.

The Clear Run watershed is at or very near full build-out. The drainage system consists of street inlets, pipe networks, small ditches/swales and larger man-made and natural drainage channels located throughout the system. The upstream section of the drainage basin contains the majority of the street collection systems draining to the main outfall run. The main run consists primarily of large underground storm sewers (interspersed with open channels) in the area upstream of College Acres Drive. In the basin area downstream of College Acres Drive, the main run consists primarily of a natural open channel with one road culvert crossing. Figure 3 illustrates the existing stormwater network.

D. FIELD SURVEY DATA

Surveys were completed by the City of Wilmington's contract surveyors in 2010. The survey data consisted of structure inverts for pipes and inlets at select locations throughout the Clear Run Drainage network. The horizontal datum used was NAD 83 State Plane (feet) and the vertical datum was NAVD 88 (feet). In addition to the survey, four (4) channel cross sections were surveyed by Moffatt & Nichol staff along the open channel portion of the Clear Run network. The four cross sections were assumed to represent separate reaches of the 6100LF of natural channel and were used to set key assumed elevation points along the outfall. Additionally, one elevation data point was surveyed (NAVD88 vertical datum) for each of the (3) three roadway crossing culverts underneath Racine Drive, College Acres Drive and Mallard Street (each point surveyed where centerline of the roadway surface intersects the centerline of the culvert structure). Surveyed elevation data at each of the (3) three critical crossings, along with elevation data associated with the (4) stream cross sections provided the basis by which all other

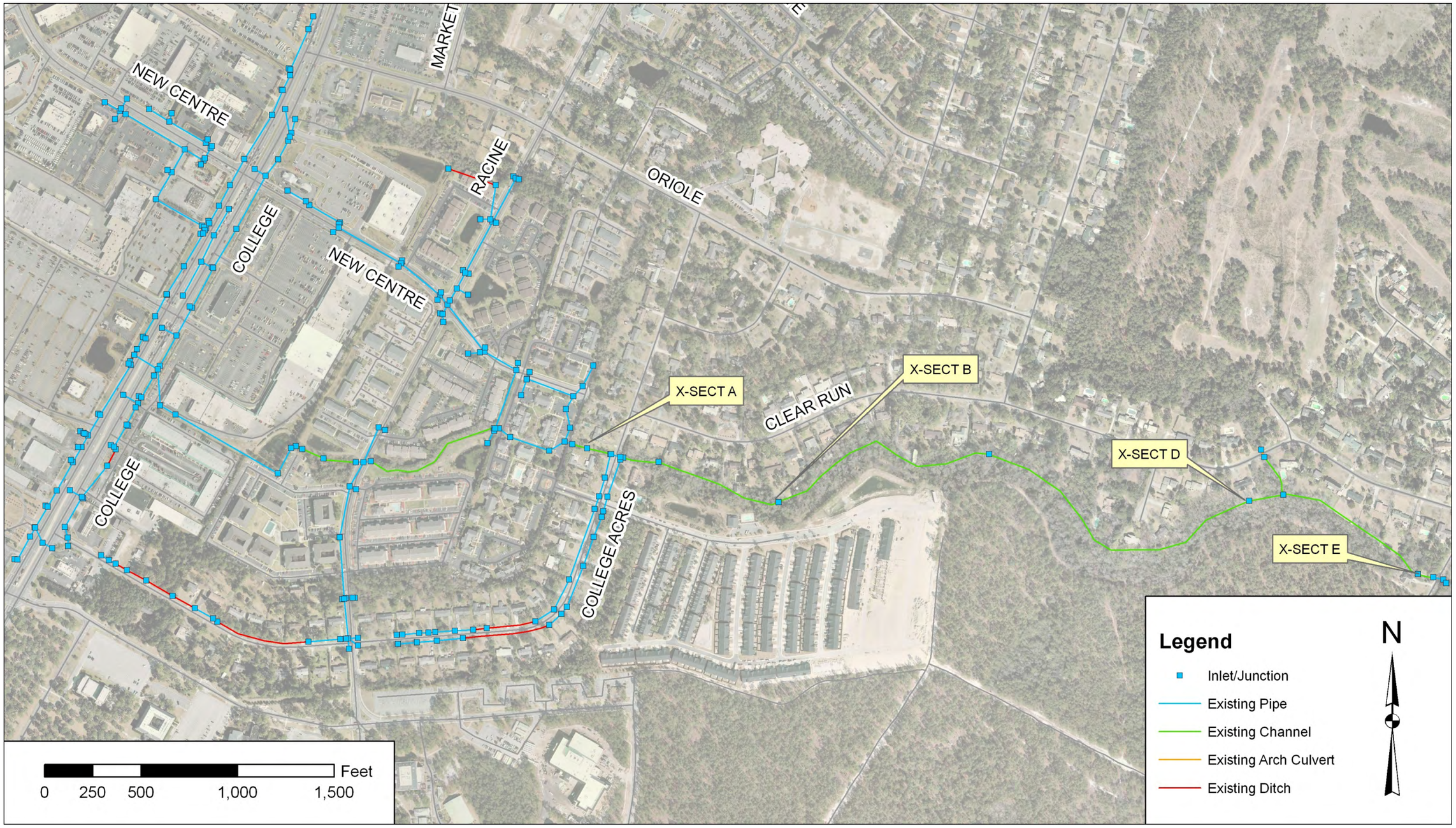

FIGURE 3: EXISTING STORMWATER INFRASTRUCTURE

elevation and connectivity assumptions were made. Slopes for the open channel outfall were interpolated between known elevation data points, and any upstream structure(s) with missing elevation data were assumed to flow to the outfall while maintaining positive gravity slope.

E. UTILITIES

Survey information for existing water distribution, sanitary sewer, electric, gas, and data utilities were unavailable for this study. As the primary goal of this study is the determination of a design alternative based on H & H feasibility and projected cost, detailed survey utility information was not pursued for this level of analysis.

IV. FIELD OBSERVATIONS

Moffatt and Nichol personnel completed a field reconnaissance in late 2010, to verify the stormwater infrastructure system connectivity and flooding concerns.

The contributing drainage area is at or very near full build-out. The existing topography is flat with average slopes of less than 1%. Overall, the existing drainage pattern flows northwest to southeast towards a floodplain of the intra-coastal waterway.

V. MODELING OF HYDRAULIC / HYDROLOGIC BEHAVIOR

A. MODELING APPROACH

The XPSWMM software was utilized to simulate the existing hydrologic and hydraulic conditions for the Clear Run Outfall Drainage Improvement Study and to evaluate drainage improvement alternatives. The modeled alternatives were evaluated on the basis of hydraulic performance and opinions of probable cost. Using survey data and the GIS stormwater inventory provided by the City of Wilmington, the existing stormwater network was modeled to estimate flows during the 10- and 25-yr and 50-yr storm events.

According to the USDA Web Soil Survey, the watershed consists of primarily of Hydrologic Soil Group (HSG) Class 'B' and Class 'A' soils. Although Class 'A' soils exist within the study area, they are present in a smaller percentage compared to Class 'B' soils.

Assuming all soils as Class 'B' provides a factor of safety for designing the proposed improvements as runoff curve numbers (and the resulting peak flows) are slightly higher for Class 'B' soils than are for Class 'A'. All soil areas in the study area basin were assumed to be Class 'B' for the purpose of estimating hydrologic runoff curve numbers. This assumption provides a 5-10% factor of safety in the calculated peak flows utilized for designing the stormwater conveyances.

Rainfall depth data was used from the City of Wilmington Technical Standards and Specifications Manual dated March 2007. A NRCS (SCS) unit hydrograph was developed using the Type III, 24-hour rainfall depth amount of 8.01 inches for the 25-yr rainfall event. The performance of the existing system was also analyzed under lower-flow (10-yr) and higher-flow (50-yr) conditions. A summary of 24-hour rainfall depths utilized in the H&H model can be found in Table 1. The existing conditions model was used to analyze the chosen storm events, and to examine several viable solutions which were considered to provide relief to the flood impacted areas.

Return Period (Yr)	Rainfall Design Depths City of Wilmington Stormwater Manual (inches)
10	6.72
25 (Design Event)	8.01
50	9.01

TABLE 1: 24-HOUR DURATION RAINFALL DEPTHS FOR SELECTED RETURN EVENTS

B. EXISTING CONDITIONS

1. Hydrology

As previously mentioned, a NRCS (SCS) Type III, 24-hour unit hydrograph was developed to calculate the volumetric and peak discharges for the 25-year storm event. Using the topography, stormwater infrastructure inventory, and aerial photography, the watershed was divided into 168 sub-basins, as shown in **Figure 4**. Each sub-basin was analyzed to develop a time of concentration (Tc) based on size, topography, and cover conditions. Small (<3 acres) sub-basins were assigned a minimum Tc of 5 minutes. Sub-basins between (3-20) acres were assigned Tc values based on the Kirpich method. Sub-basins larger than 20 acres were assigned values for the Tc based on the TR-55 segmented calculation approach.

Sub-basins draining to suitably sized BMP's were assumed to benefit from 10-yr peak flow detention only in the XPSWMM model. BMP's in question were evaluated using the TR-55 design methodologies to determine if the existing footprint was large enough to detain the calculated volumes resulting from the 10-yr and 25-year rainfall event at a reasonable depth. In the absence of storage and outfall structure data for the BMPs, nodes at the outlets of suitable BMP's were modeled with a runoff curve number to mimic pre-development cover conditions. The pre-development cover condition was assumed to be a wooded (good condition) with HSG type 'B' soils.

2. Nodes

Nodes were developed in XPSWMM to represent access points to the stormwater system. Inlets, manholes, junction boxes, and open-ended pipes were taken from the stormwater inventory and survey data provided. When available, "depth from surface" data was used with LIDAR Bare Earth ground elevation data to determine node invert and rim (ground) elevations. For nodes that did not have "depth from surface" data provided, surrounding topography and adjacent structures with known depth or elevation data were used to estimate inverts. Missing elevations were interpolated such that the system maintained a positive gravity slope from the upstream to the downstream end. The model was set to

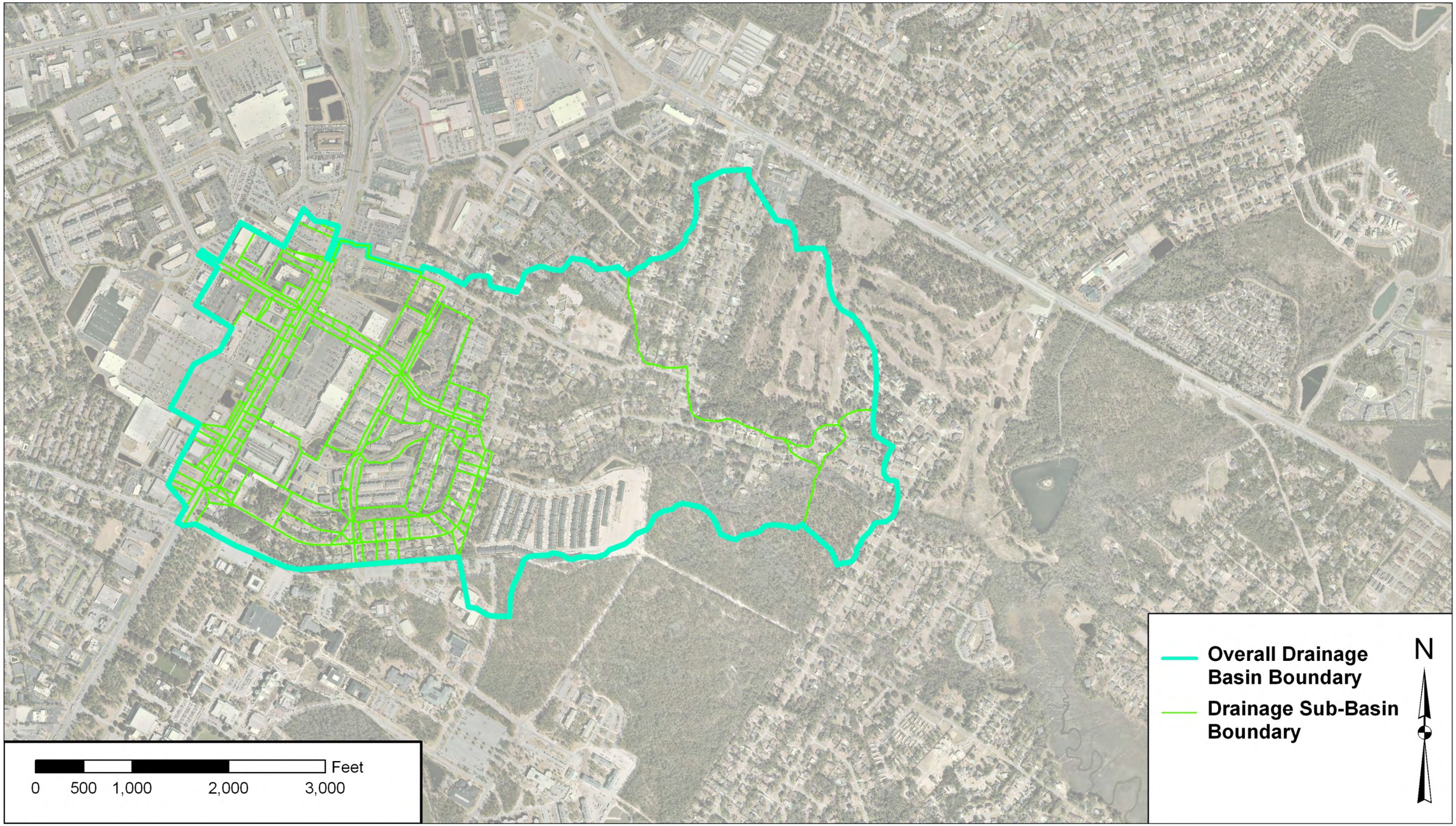

FIGURE 4: CLEAR RUN OUTFALL DRAINAGE SUB-BASINS

allow ponding at all non-channel nodes to maintain total volume continuity, and the assumption was also made that ponding is allowed to occur at all inlet nodes in the contributing drainage system. This approach helps to mimic the existing conditions, as it is believed undersized piping in the network is the primary cause for node flooding, not insufficient inlet capacity. In the absence of the inlet ponding assumption, the model could demonstrate a significant loss of water, potentially leading to undersized pipes in the proposed design alternatives. Individual node parameters are tabulated in **Appendix A**.

3. Conduits

Conduits were developed in XPSWMM to represent pipes, ditches, and open channels in the stormwater system. When available, "depth from surface" data was used with LIDAR Bare Earth ground elevation data to determine conduit invert elevations. For nodes that did not have "depth from surface" data provided, surrounding topography and adjacent structures with known elevation or depth data were used to estimate inverts. Missing elevations were interpolated such that the system maintained a positive gravity slope from the upstream to the downstream end. Sufficient ground cover was provided (as measured from ground elevation to crown of pipe) and appropriate slope ranges were utilized for any conduit with an assumed invert. The four (4) surveyed cross sections provided from the field survey were input as user-defined, natural channels in the existing conditions model. The surveyed cross section data points were "spliced" into the LIDAR ground elevation data to provide reasonable assumptions for profiles of the left and right overbanks for each portion of the open channel system. Individual conduit parameters are tabulated in **Appendix A.**

4. Outfall

The outfall location was assumed to be a point in Bradley Creek ~25 downstream (East) of the culvert crossing at Mallard Street. The FEMA Flood Insurance Study #37129CV000A (April 3, 2006) was used to determine downstream flooding effects at this outfall. Flood elevations were verified to ensure the modeled tailwater conditions had a higher elevation than shown on Flood Profile Sheet 01P in the FEMA report.

5. XPSWMM Model Results

For the Clear Run drainage system, the existing model demonstrates multiple areas with surcharged pipes and flooded storm junctions under the 10- and 25- and 50-yr return events. Most of the critical issues are along New Centre Drive, South College Road, and Racine Drive. A run of undersized piping along New Centre Drive (west of College Road) and along South College Road (north of main line junction) appears to be the cause of flooding at the most upstream points in the network. This undersized run of pipe is causing flooding along the aforementioned section of New Centre Dr. and at inlets along the West side of College Road. High tailwater in the dual pipes crossing South College Road (at a point on South College Road, approximately 1500 feet southwest of the intersection with New Centre Drive) appears to worsen the flooding adjacent to this area of the network. High tailwater in the dual pipes crossing South College Road also appears to be a cause for

flooding and pipe surcharging along College Road in areas immediately north and south of the crossing. Undersized piping along College Road at the southwest corner of the network appears to be causing flooding, and is heightened by the high tailwater at the aforementioned dual pipe crossing.

Undersized piping on New Centre Drive (East of College Road) and Racine Drive appear to be the cause of surcharging and flooding in the areas adjacent to the intersection of these two streets. High tailwater in downstream conduits does not appear to be cause of the flooding in these areas, based on available data.

Surcharging and flooding along College Acres Drive was evident in the model, yet was expected due to the undersized street-side pipe/ditch networks serving the area. High tailwater along the main line (at the point where drainage culvert passes under College Acres Drive) only appears to impact the conduits running parallel along College Acres Drive immediately adjacent to that crossing.

The existing model demonstrates that some junctions and culvert crossings along the main line of Clear Run do not flood in response to the 25-yr storm event. Undersized pipes along upstream portions of the main line appear to be constricting flow to a sufficient degree to prevent inundation of downstream structures for the 25-yr storm event. However, the culvert crossing on the main line at Mallard Street (most downstream crossing) does overtop the road during the 25-yr storm event.

Figure 5 shows the duration of inlet flooding and the duration of conduit surcharging for the 10-yr rainfall event. Figure 6 shows the depth of inlet flooding for the 10-yr event. Figures 7-8 demonstrate similar results for the 25-yr design event, as Figures 9-10 do for the 50-yr storm event. Tabulated results for the existing conditions can be found in **Appendix A**. The four (4) hydraulic parameters of concern are depth of flooding (per node average), duration of flooding (per node average), maximum ponded volume (per node average and overall), and total ponded volume (per node average and overall). The parameters will be tabulated for the existing model and compared to the results from each proposed alternative on a percentage difference basis.

6. Comparison To Anecdotal Evidence

In the interest of assessing the accuracy of the model, a simulation was performed for an actual storm event for which anecdotal accounts of the magnitude of flooding in the study area exist. On July 6, 2009 NOAA rainfall records from the Wilmington Airport show a storm with 6.51 inches of rain in the event, and the Wilmington Star reported that as much as 10 inches of rain fell in the vicinity of Racine and New Centre Drive. City stormwater staff was at that intersection immediately following the rainfall event and reported ponding at that location to depths of 3-4 feet (Personal Communication, Dave Mayes).

The XPSWMM model of the Clear Run watershed was run with 10 inches of rain to simulate the July 6 event and the model resulted in ponded depths of up to 2 feet at the intersection

of Racine and New Centre Drive. While the model did not project the same depths reflected in the anecdotal account, it is important to note that this model does not project the extents or boundaries of flooding/ponding on a physical surface, but rather, relies on the hypothetical uniform surface generated by XPSWMM in the absence of actual surface data. In addition, overland flow was not simulated in this model, so real-world ponding at the actual trough of the valley is slightly underestimated. More importantly, the results did show the intersection to be one of the worst surcharging and ponding nodes in the model.

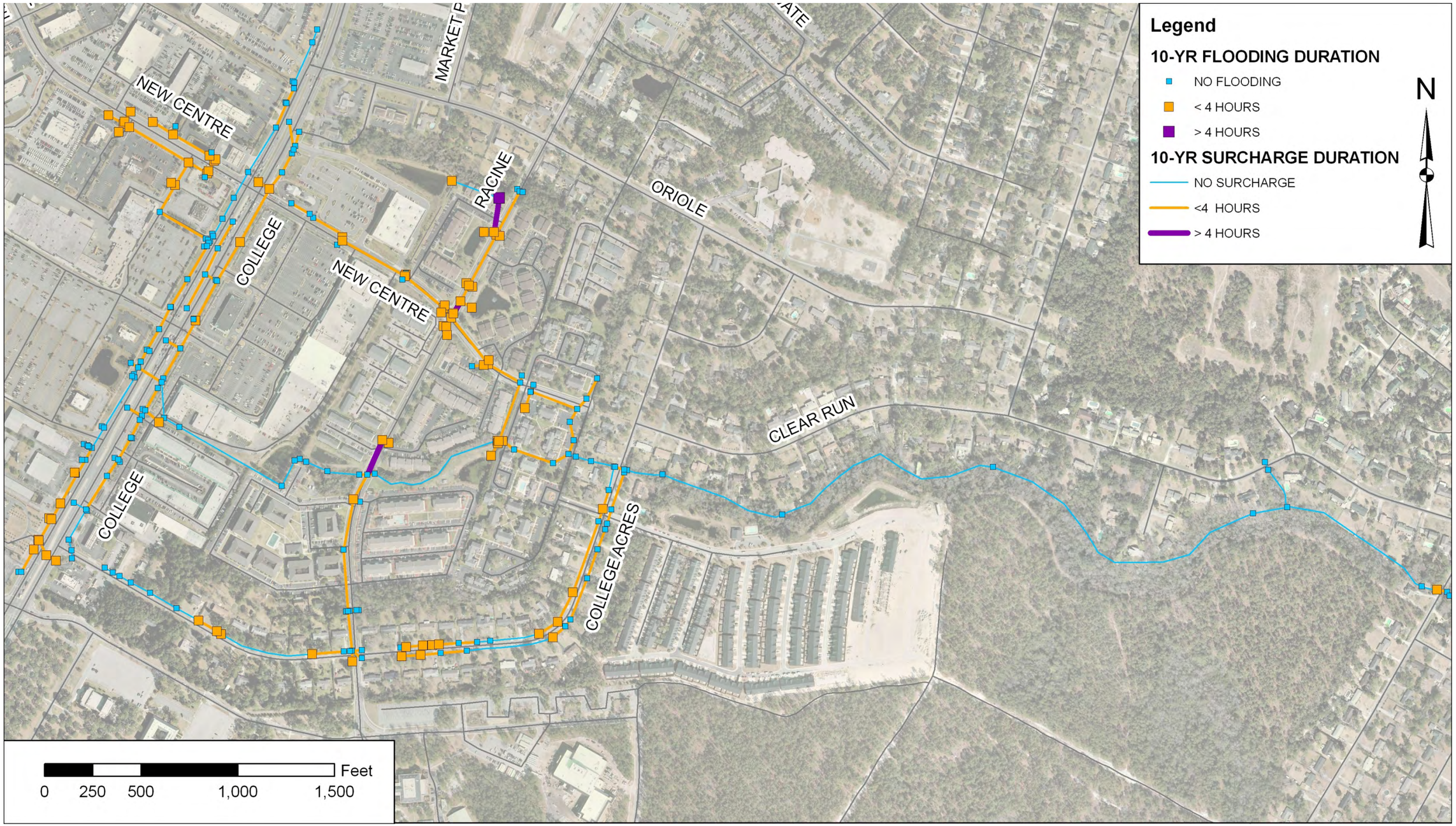

FIGURE 5: DURATION FLOODING/SURCHARGE FOR 10-YEAR RAINFALL EVENT - EXISTING

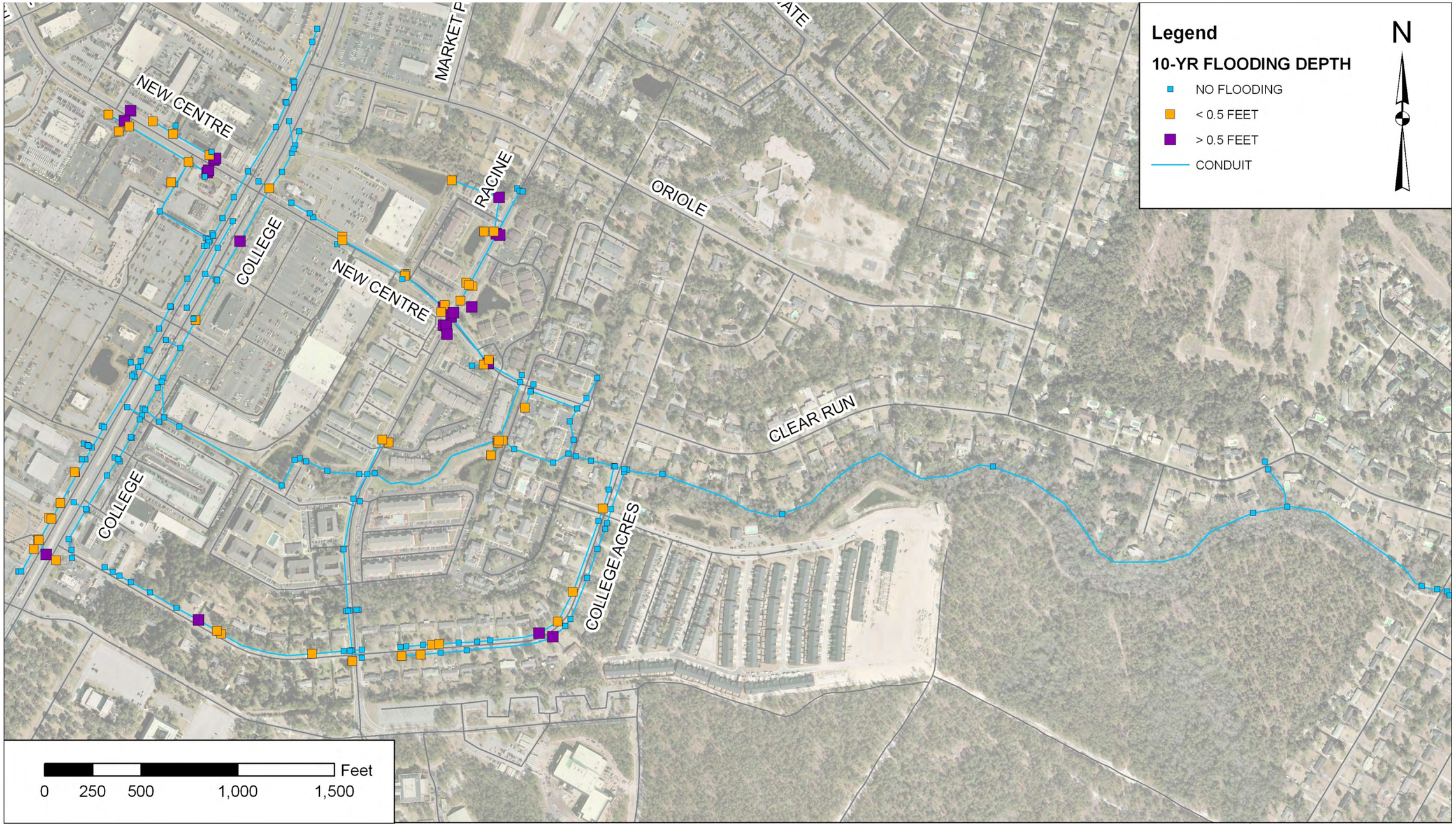

FIGURE 6: DEPTH OF FLOODING FOR 10-YEAR RAINFALL EVENT - EXISTING

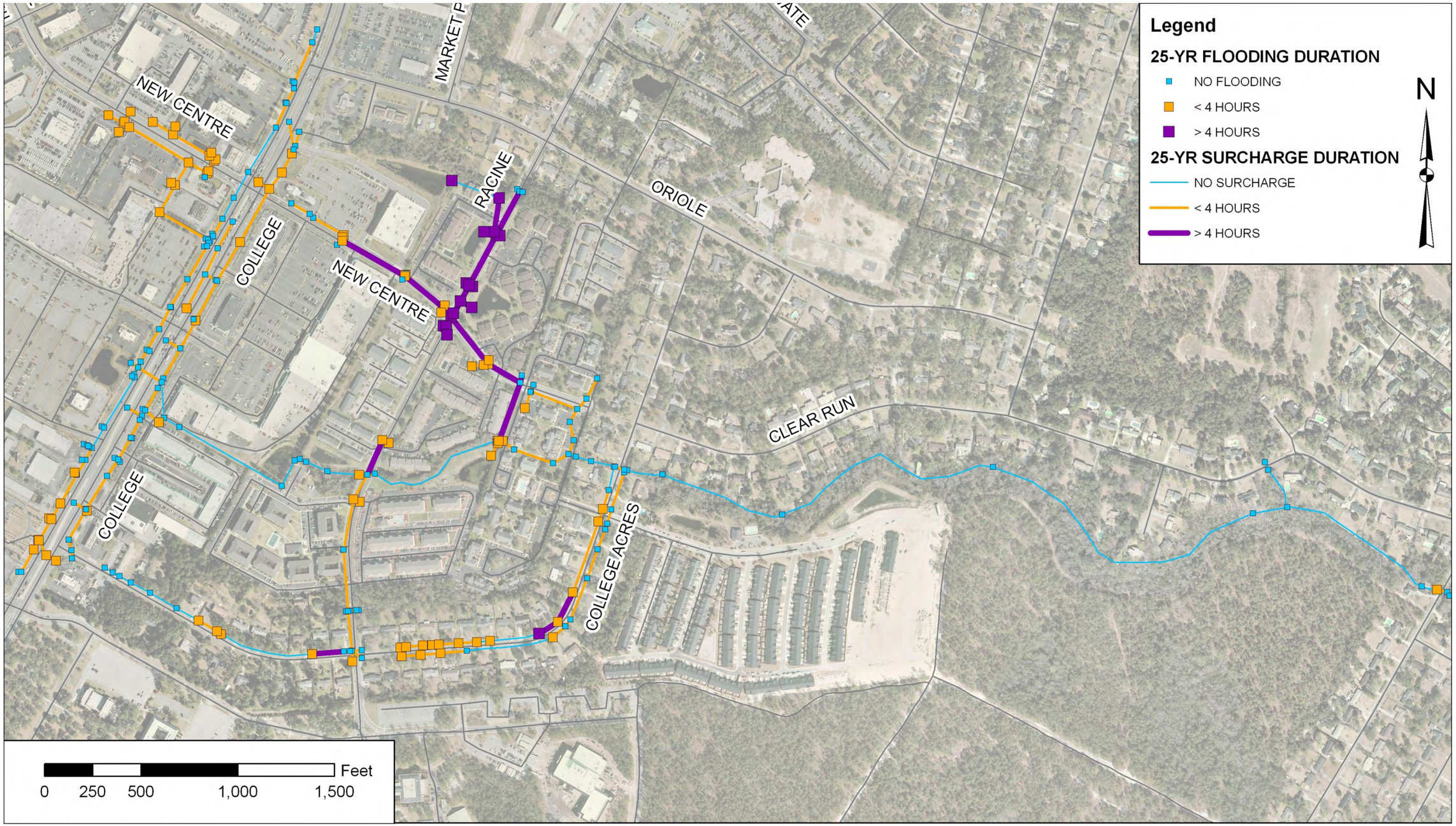

FIGURE 7: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT - EXISTING

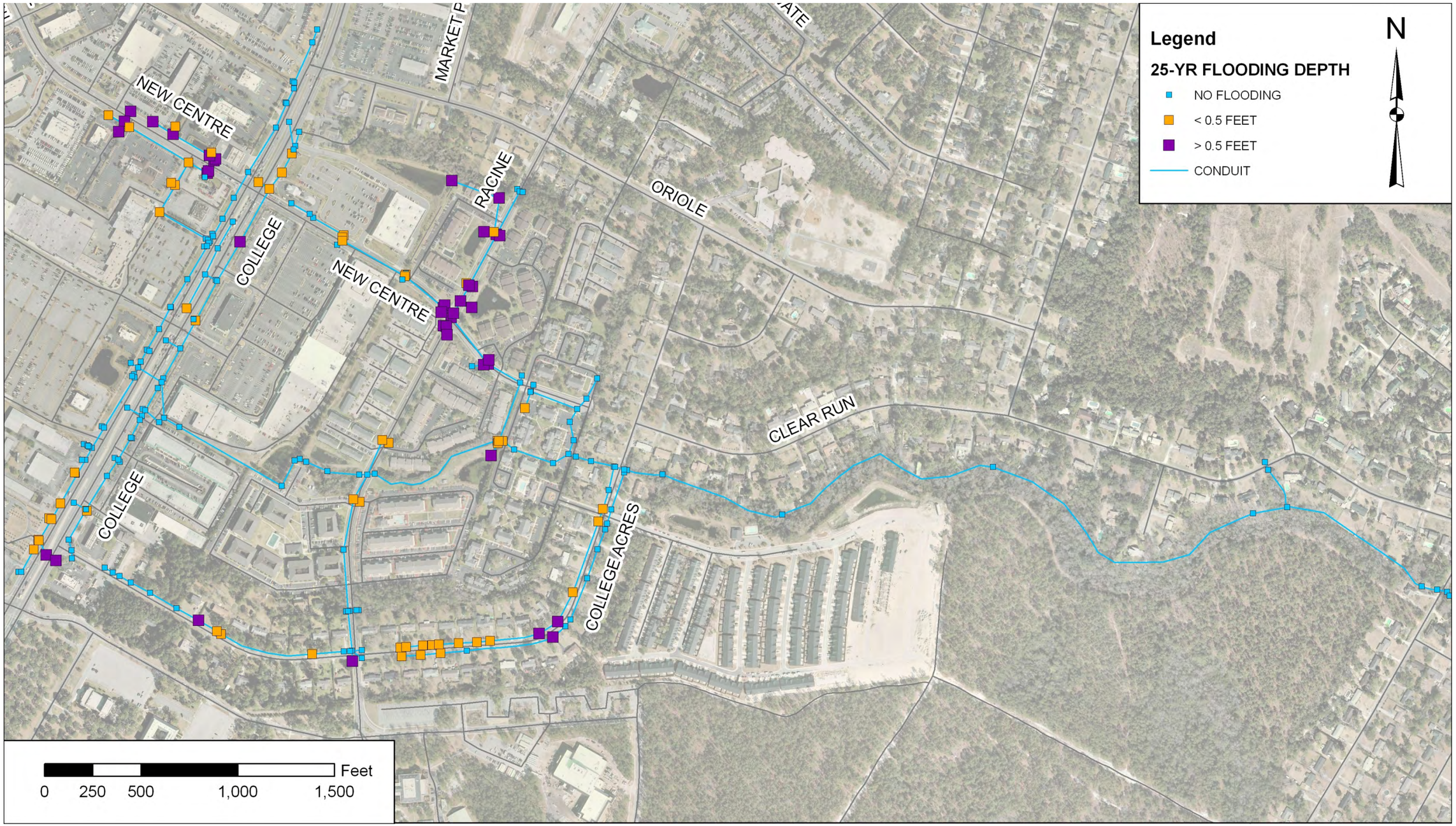

FIGURE 8: DEPTH OF FLOODING 25-YEAR RAINFALL EVENT - EXISTING

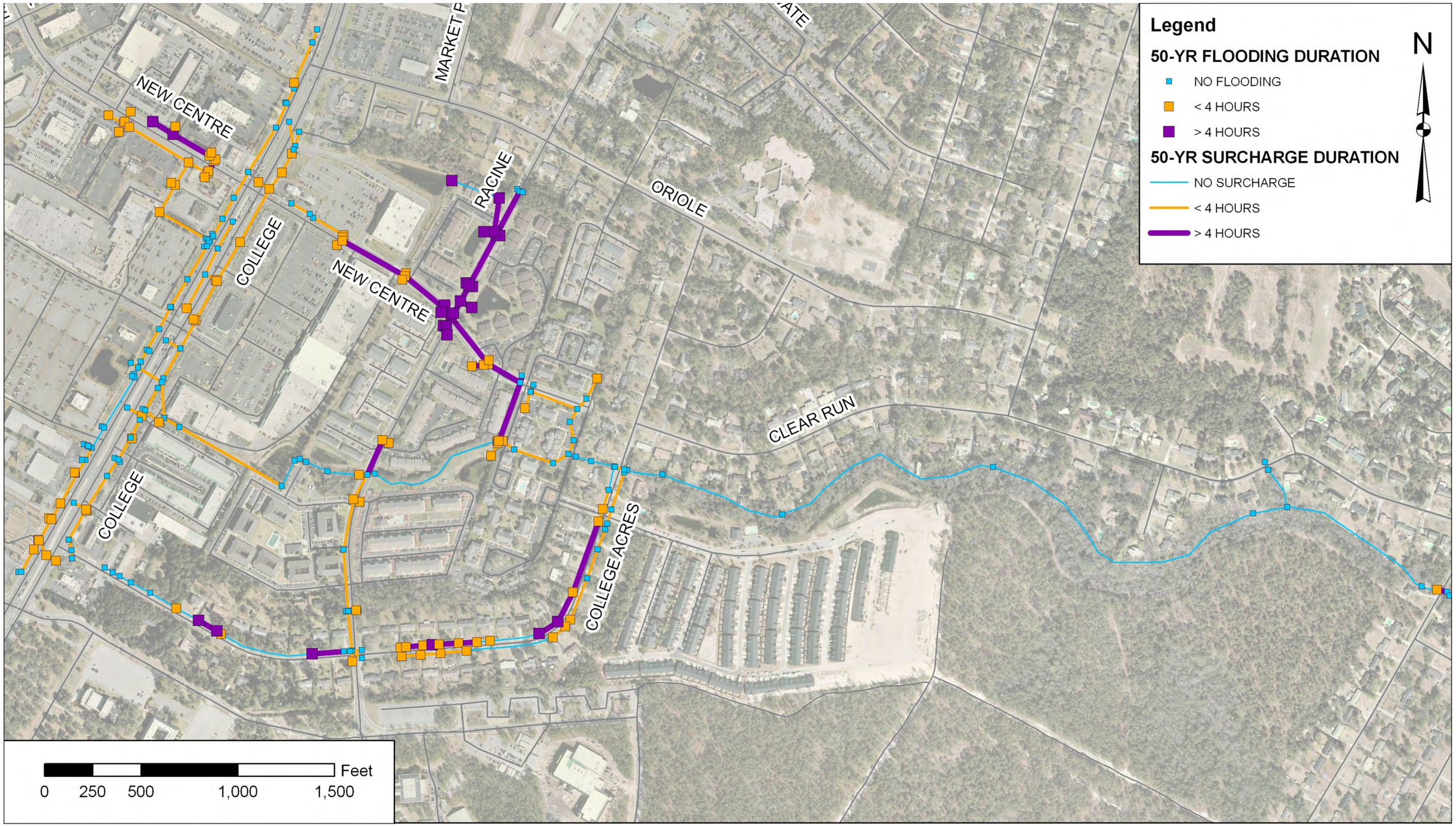

FIGURE 9: DURATION FLOODING/SURCHARGE FOR 50-YEAR RAINFALL EVENT - EXISTING

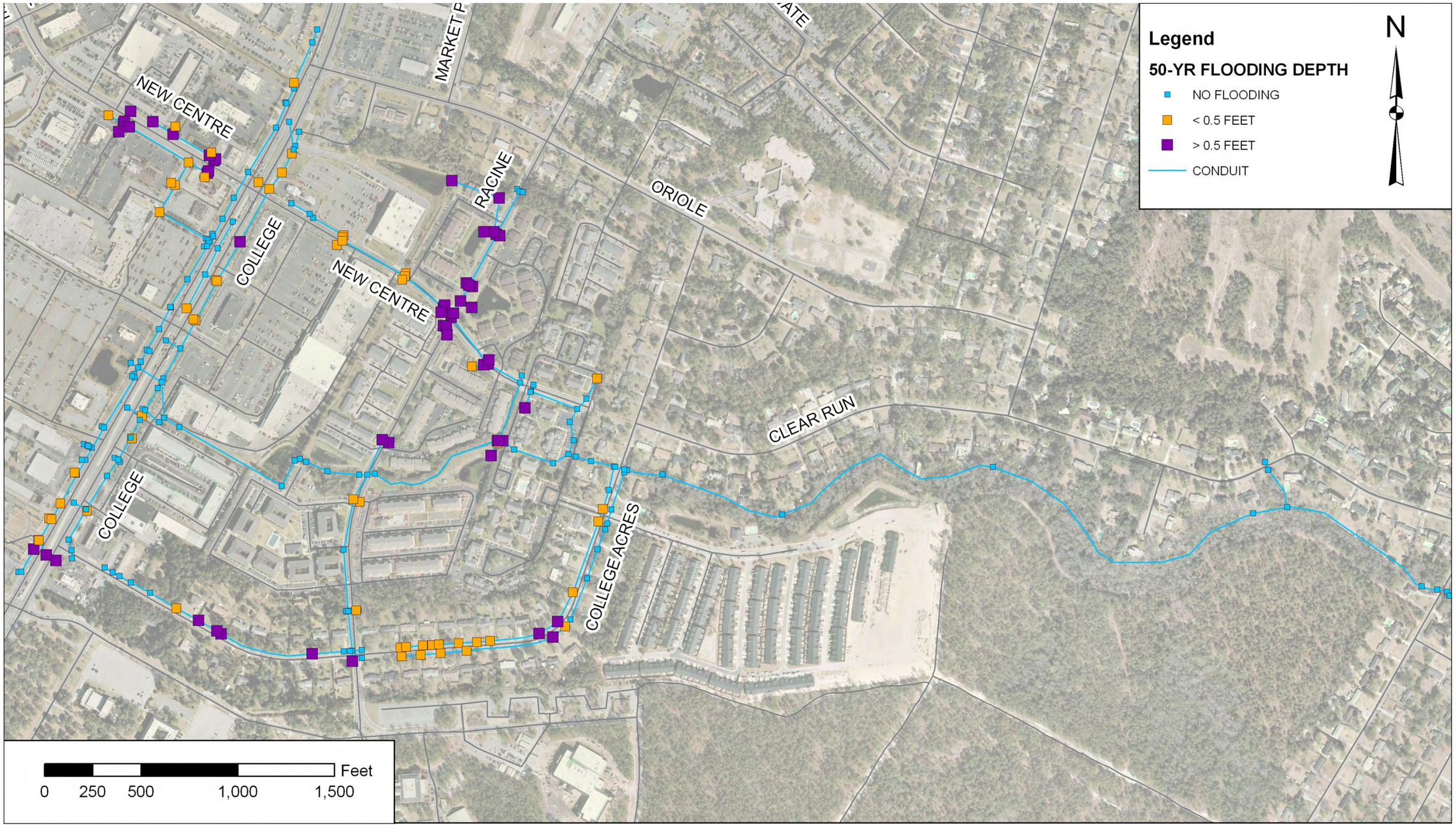

FIGURE 10: DEPTH OF FLOODING 50-YEAR RAINFALL EVENT - EXISTING

C. EXISTING MODEL RESULTS & DISCUSSION

The results of the existing conditions model for the 10-, 25- and 50-yr rainfall event indicate the presence of significant flooding within the network. Flooded nodes and surcharged pipes from the 10-yr rainfall event results indicate areas most susceptible to flooding. These areas are the upstream end of New Centre Drive (West of intersection with South College Road), adjacent to the intersection of Racine Drive and New Centre Drive along both streets, at the Southwest end of the system along South College Road, and along College Acres Drive (West and East of intersection with Racine Drive). The model indicates that the main line (originating at a junction midway along College Acres Drive and travelling East towards the downstream system outfall) is not sufficiently sized to convey the 10-yr rainfall event.

In addition to demonstrating flooding in the aforementioned areas, results from the 25-yr rainfall event show flooded nodes and surcharged pipes along South College Road (North and South of intersection with New Centre Drive), along the full length of New Centre Drive, along Racine Drive (North of the intersection with New Centre Drive), and along the main line outfall under an existing multi-family apartment complex. The model indicates a significant amount of flooded volume stored in the upstream portion of the storm network, which results in the majority of downstream main line structures not becoming inundated as was expected with this rainfall event.

The results of the 50-yr rainfall event demonstrate increases in flooded volumes and durations of node flooding/pipe surcharge throughout the system in the same areas impacted by flooding from the 25-year rainfall event.

Proposed alternatives will be designed to convey the peak flows resulting from the 25-year rainfall event. It is important to summarize the (4) four previously mentioned (H&H) parameters used to analyze the existing model 25-year rainfall event results. The existing model results show that the average depth of node flooding is 0.20 feet, the average duration of node flooding is 0.87 hours, the maximum stored volume is an average of 1,619 cubic feet per node and 9.22 ac-ft. overall and the total ponded volume is an average of 2,241 cubic feet per node and 12.76 ac-ft. overall. Overall H&H results can be found in **Table 3A** on Page 50. A breakdown of the H&H results specific to the Racine Dr./New Centre Dr. intersection and the section of New Centre Drive to the west of South College Road can be found in Table 3B on Page 51 and 3C on Page 52, respectively.

D. PROPOSED ALTERNATIVES

❖ DEVELOPMENT OF ALTERNATIVES

In conceptualizing proposed alternatives, it was important to use the information available regarding physical site conditions to develop feasible solutions. The feasibility of proposed alignments was dictated the topography of the site (for ensuring positive slope and connectivity), presence of commercial and residential parcels (for reduction of impact to existing structures and hardscapes) and layout of roads and streets. To reduce costs and mitigate the impact to traffic flow and businesses, it was important to develop alternatives that could be constructed mostly within existing storm easements and public right of way.

Given that the study area is or is very near fully developed, there are only two available routes/alignments that can be utilized for improving the storm network and effectively reducing the impact of flooding. The first option is an alignment for improvements primarily along South College Road and the second is an alignment that runs along New Centre Drive. In the following sections, there will be a discussion regarding the proposed improvements for every alternative. Each section will include a description of the proposed improvements, potential positive/negative impacts to existing infrastructure, and a discussion/summary of benefits to flood impacted areas (including comparisons of key H&H parameters to the existing model results).

❖ HYDRAULIC & HYDROLOGIC DESIGN METHODOLOGIES

All improvements proposed for the following alternatives are designed to convey the peak flows resulting from the 25-year storm event without surcharging, unless otherwise noted. It is important to note that the entire storm drainage system is not being upgraded to convey the design storm event, only those junctions and conduits proposed for improvement at this time. The proposed improvements are however sized based on the assumption that the entire watershed/storm system is upgraded to City of Wilmington standards for the 25-year storm event. This assumption assures that improvements proposed for each alternative will be sufficiently sized to account for the eventual improvement of the entire storm network.

All underground pipes proposed for improvement meet the City of Wilmington standards for materials, minimum sizes, slopes, unless otherwise noted. Piped improvements are designed based on Manning's equation for gravity pipe flow. All improved pipes are designed as Class III reinforced concrete pipe (RCP) with a manning's n-value of (0.012). The rational method was utilized to obtain the 25-year design flow for each section of pipe based on the following assumptions. Sub-basins were classified into three (3) categories, commercial & industrial (allowable range C = 0.50-0.95, used 0.85), multi-family residential (allowable C = 0.45-0.60. used 0.60) and single family residential (allowable C = 0.30-0.45, used 0.40). Sub-basins draining less than 3 acres were assigned a (Tc) of 5.0 minutes. Sub-

basins draining 3-20 acres were assigned a Tc based on the Kirpich method. The intensity was developed with the IDF equation (I=G / (H+Tc)) using 25-year coefficients for G & H. Available slopes for pipes were determined based on physical site constraints, and a theoretical pipe size was calculated using the Manning's equation. Pipe sizes were selected based on the next largest commercially available pipe size. Pipes were also checked to ensure that a minimum velocity of 2.5 FPS was met for sediment scouring purposes. Pipes were also located to meet the minimum cover requirement of 2.0 feet.

All culverts proposed for improvement meet the City of Wilmington standards for materials, minimum sizes, slopes, unless otherwise noted. Culverts are designed based on the 25-year peak flows generated using the NRCS TR-55 method. All soils in the upstream section of the study area were assumed to be SCS type 'B' soils for generation of runoff curve numbers. Sub-basins were split into (5) categories, commercial/business/industrial (CN = 92), residential 1/8 acre lots/townhouses (CN=85), residential ½ acre lots (CN = 70), residential 1 acre lots (CN = 68) and wooded good condition (CN = 55). Drainage areas to each culvert were calculated, and a composite runoff CN was developed for each. Time of concentration was developed with the TR-55 segmented approach using the following assumptions; maximum sheet flow length is 75 feet with a "smooth" surface, shallow concentrated flow occurs on a "paved" surface, concentrated pipe flow is in concrete pipes and concentrated open channel flow occurs in a grass-lined channel. Flows were generated by utilizing an SCS Type III, 24HR hydrograph with a rainfall depth of 8.01" for the 25-year event (Chart E-5, City of Wilmington design manual). Culverts were analyzed for the 25-year peak flows with the provided inlet control (Chart F-6) and outlet control (Chart F-11) nomographs to determine the controlling condition. Culverts were sized based on the controlling condition such that the resulting Headwater/Depth (HW/D) ratio was less than 1.20. Culverts were also designed such that a minimum freeboard to the roadway/embankment crest of 1.0 feet was attained for the design event.

Open channels proposed for improvement meet the City of Wilmington standards for capacity, velocity, and appropriate linings, unless otherwise noted. Open channels are designed based on the 25-year peak flows generated using the NRCS TR-55 method. The channels are also analyzed for the 50-year peak flows with respect to the same parameters. All assumptions for peak flow generation apply just as outlined in the culvert design section above. Open channel geometry was developed to conform to the four (4) existing surveyed cross sections 'A', 'B', 'D' and 'E'. All proposed channel utilize a 5' bottom width, 4:1 H:V side slopes and a 5' horizontal channel bench on each side above the main channel section. Proposed cross section utilize a 4:1 H:V side slope above the bench to tie in to existing grade. Channels were designed utilizing the Manning's equation for open channel gravity flow in an iterative "trial and error" design process. The channel sections were analyzed as trapezoidal sections with a 5' bottom width, 4.5:1 slopes (to account for 5' bench) and a tall fescue lining. The permissible velocity was assumed to be 4.50 FPS given the assumptions of a (0-5%) channel slope, "easily erodible, non-plastic soils" and a "tall fescue" lining. Channels were analyzed to determine the maximum depth (based on minimum average channel slope) and maximum velocity (based on maximum average

channel slope) were within the permissible values for the 25-year event. The channels were also analyzed for the 50-year event to determine maximum depths and velocities. Channels were sized sufficiently to ensure the maximum design depth was less than 6.0'. Any channel with velocities above the permissible values was analyzed for the 25-year event to ensure that a permanent synthetic lining would result in a stable channel section. In these cases, the channels were analyzed to ensure the shear stresses were below the permissible values for the synthetic lining.

It should also be noted that the preliminary design parameters of the main Clear Run channel segment below College Acres Drive were also informed by the process of natural channel design (Rosgen, 1996). The channel dimensions and the projected easement needs were based on a conceptual combination of Priority 2 and Priority 3 approaches to stream restoration using natural channel design principles. The Priority 2 restoration approach involves excavation of a new floodplain bench at the stream's existing grade, allowing the stream to access a greater cross-sectional area and dissipate erosive velocities during storm events. In Priority 3 restoration, bio-engineered structures such as cross vanes and J-hooks are utilized to dissipate erosive stream velocities and maintain channel stability when significant floodplain areas cannot be achieved. In a Priority 3 approach, the stream restoration is designed to mimic the pattern and profile of a mountain steam where energy is dissipated by stepping the stream down through a series of pools maintained by hardened structures placed in the channel, constructed with logs or natural stones. This combination of Priority 2 and Priority 3 restoration is often employed where lateral room in the alluvial valley is insufficient to allow for construction of a new stream channel at the streams historic grade.

It is assumed that any future re-development within the basin will be required to meet the City of Wilmington standards for peak and/or volumetric detention.

Alternative 1 IS NOT designed to the aforementioned standards, but rather represents a Capital Improvement Project (CIP) planning design schematic provided by the City of Wilmington for an upgrade to the main storm outfall alignment. Alternative 2 provides an improvement scheme along the same alignment as Alternative 1, but IS designed to the above minimum standards. Alternatives 3 & 4 ARE also designed to the above minimum standards. Alternatives 3 & 4 propose to divert flow along New Centre Drive via a new main outfall alignment. Detailed discussions for each alternative are outlined in the following sections.

❖ PROPOSED IMPROVEMENTS: ALT 1

The first alternative, **Alternative 1 (Main Line Parallel System/CIP Planning Design)** involves upgrading the existing pipe/channel system starting from the most upstream junction on south side of New Centre Drive and continuing southeast along New Centre Drive to the intersection with South College Road. The upgrades continue southwest along South College Road to the junction (main line junction point) where the existing pipe

system crosses South College Road. The improvements then continue east along the main line through alternating runs of pipe, natural channels and culvert crossings to the downstream outlet of the system (see Figure 11).

Pipe improvements for **Alternative 1** were input into the model as outlined by a CIP planning design schematic provided by the City of Wilmington. The CIP planning document only represents improvements based on very rudimentary preliminary design flow calculations. It is important to note that the improvements as detailed in this alternative DO NOT meet the City of Wilmington's minimum hydraulic design guidelines.

Pipe upgrades for this alternative involve the installation of a parallel run of RCP conduit of increasing size along the aforementioned route, as well as culvert upgrades at key road crossings. Open stream channels along the main line have been designed with improved capacity to facilitate heavier outflows along the run. These increases in flow occur because this alternative relieves a significant amount of upstream flooding in the network, resulting in higher flows moving downstream. The improved channels serve to increase storage within the system and reduce the hydraulic grade lines of underground pipe and culverts along this run. Key road crossing culverts at Racine Drive, College Acres Drive and Mallard Street have also been upgraded to handle the additional flows along the main line. It is recommended that a synthetic liner be installed along the improved open channels from the stream section immediately upstream of the College Acres Drive crossing down to the outfall of the system.

Figure 11 graphically represents the layout of proposed improvements for **Alternative 1**. A summary of the costs and quantities required for the construction of this alternative can be found in **Appendix B**.

❖ FLOOD REDUCTION: ALT 1

As shown in Figures 12-13, **Alternative 1** reduces the flooding impact primarily at nodes along or directly adjacent to the upgraded pipes. These upgrades reduce the ponded volume and duration of node flooding and pipe surcharge along the south side of New Centre Drive and the West side of South College Road. However, this alternative does not alleviate flooding in the vicinities of New Centre Drive/Racine Drive, College Acres Drive or at the southwest edge of the network along College Road.

Although not evident in the model due to flow restriction from upstream, undersized conveyances, critical structures along the main line run will flood as a result from the calculated design flow for this alignment. All three culvert crossings as shown in this alternative would overtop the roadway if the entire network was improved to convey the 25-year peak flows without surcharge. All improved parallel pipe runs would surcharge, if not already evidenced in Figure 12. Peak flow calculations demonstrate that the entire scope of improvement for pipes and culverts in this alternative is inadequate.

❖ HYDRAULIC PERFORMANCE: ALT 1

The (H&H) parameters of concern are depth of node flooding, duration of node flooding, maximum stored volume, and total ponded volume. The results for each parameter were averaged across all network nodes, and were compared to the existing conditions model in terms of a percent difference reduction for each. The **Alternative 1** model results show that the average depth of node flooding is 0.13 feet per node (-34%), the average duration of node flooding is 0.60 hours per node (-31%), the maximum stored volume is an average of 1,008 cubic feet per node and 5.74 ac-ft. overall (-38%) and the total ponded volume is an average of 1,391 cubic feet per node and 7.92 ac-ft. overall (-38%). The 25-year rainfall event (H&H) results for **Alternative 1** are tabulated in **Table 3A** on Page 50. A breakdown of the H&H results specific to the Racine Dr./New Centre Dr. intersection and the section of New Centre Drive to the west of South College Road can be found in Table 3B on Page 51 and 3C on Page 52, respectively.

❖ CONSTRUCTION FEASIBILITY/IMPACT TO PRIVATE PROPERTY: ALT 1

It should first be noted that **Alternative 1** DOES NOT meet the City of Wilmington design standards and thus should NOT be considered a feasible design scheme. The following discussion is not meant to imply that this alternative should be constructed, but is rather only intended to convey potential drawbacks of the proposed alignment and parallel piping improvements.

Alternative 1 demonstrates several disadvantages relative to construction feasibility. A significant portion of the construction for this alternative involves work along the South College Road ROW, a major multi-lane thoroughfare for the City of Wilmington. Construction along this ROW can potentially pose major negative impacts in terms of traffic congestion and additional cost. The proposal for a parallel pipe network also increases the width of ROW and private easement area required for construction. The likelihood for conflict with existing utilities is increased due to the necessity of a wide storm easement. **Alternative 1** requires upgrades to all three (3) road crossing culverts to facilitate heavier outflows along the main line. This alternative also requires construction within privately owned commercial and residential properties, which potentially increases the likelihood of additional land acquisition for easements and conflicts with existing utilities and/or BMP's. Construction of a parallel network as proposed will also require modifications at existing junctions to hydraulically connect the proposed system to the existing outfall, which could result in additional costs and reduced hydraulic performance.

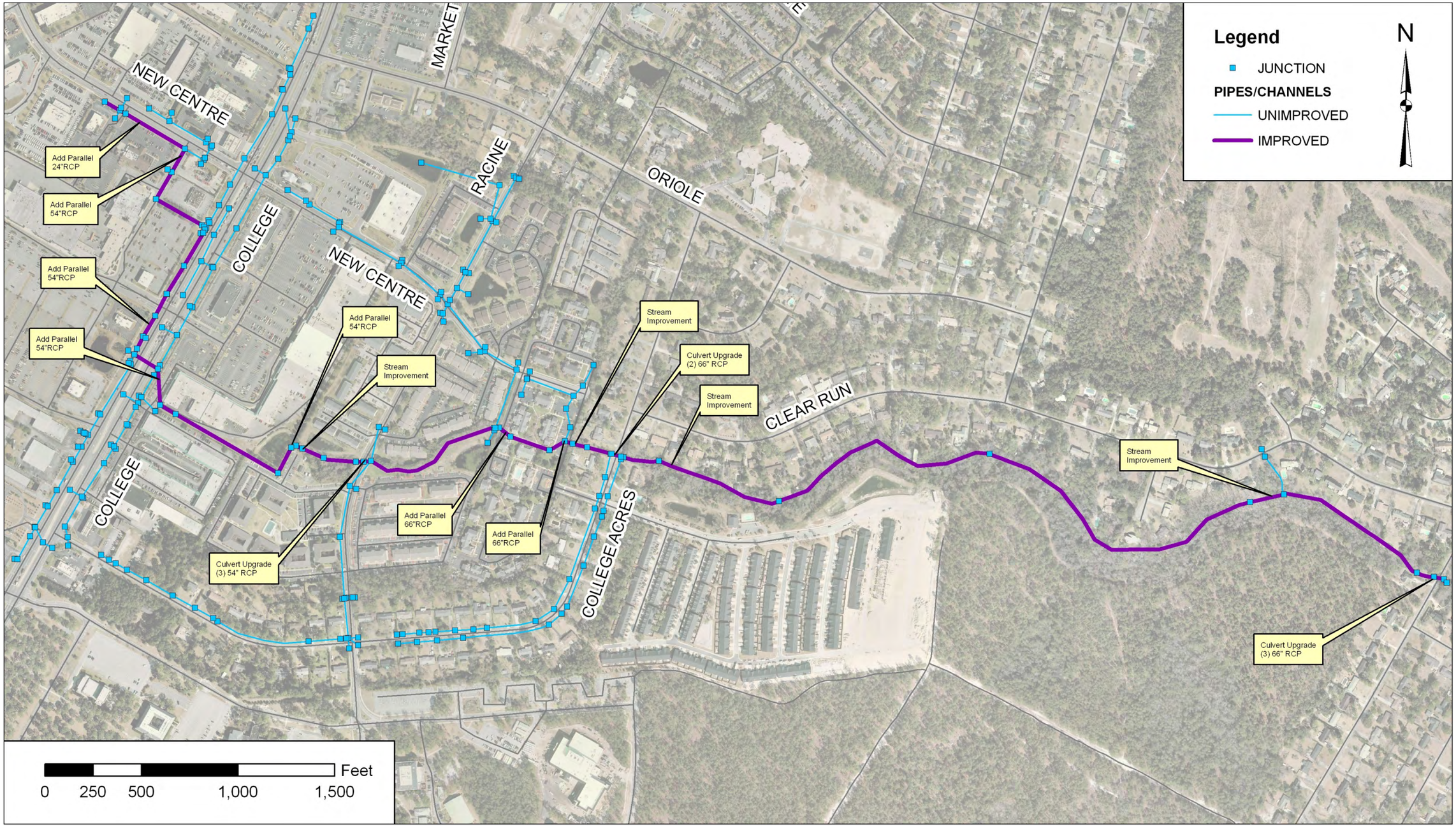

FIGURE 11: IMPROVEMENTS FOR MAIN LINE RUN - PARALLEL SYSTEM – ALTERNATIVE 1

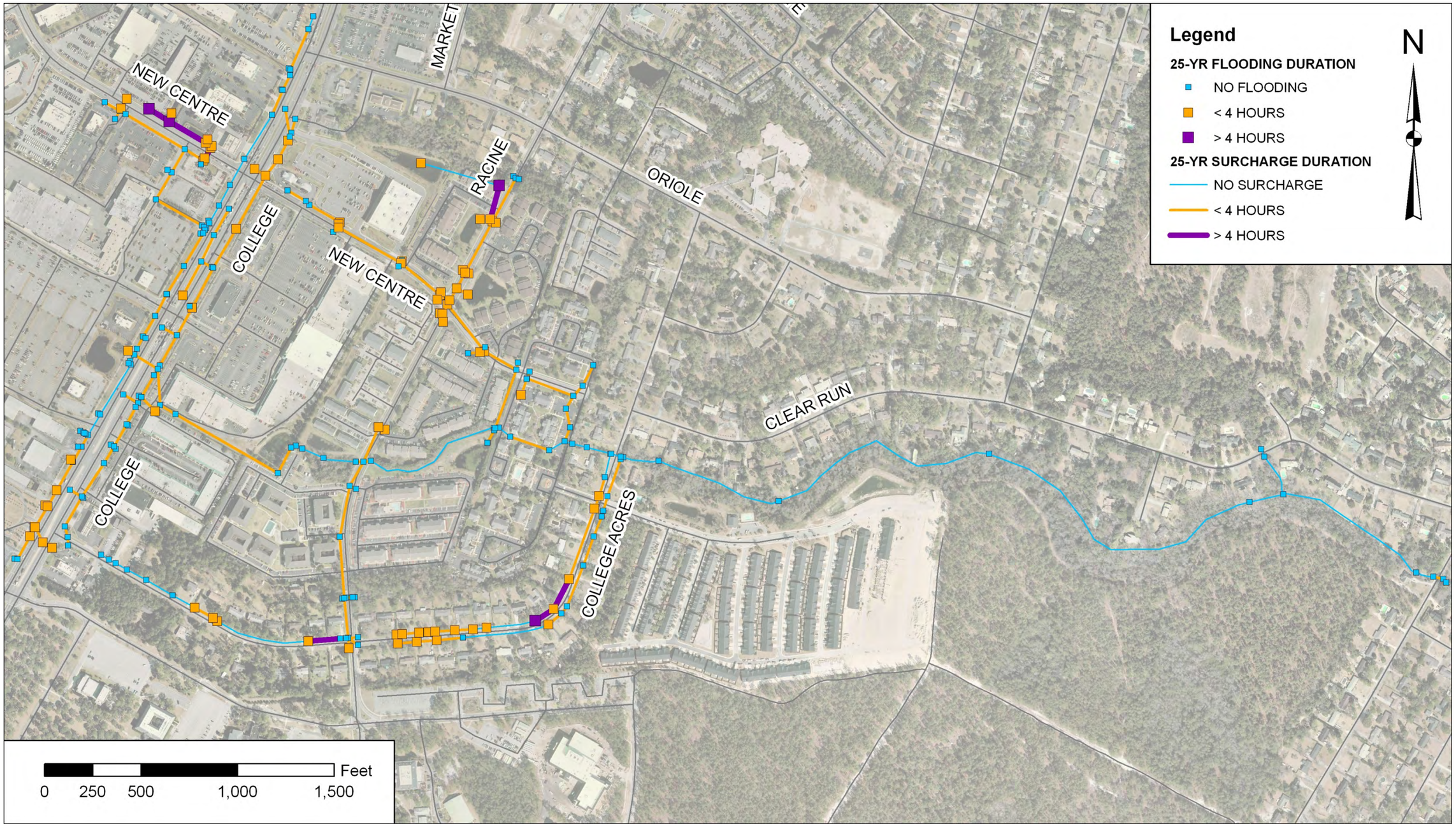

FIGURE 12: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 1

FIGURE 13: DEPTH OF FLOODING 25-YEAR RAINFALL EVENT – ALTERNATIVE 1

❖ PROPOSED IMPROVEMENTS: ALT 2

The improvements outlined in **Alternative 2** ARE sized to meet the City of Wilmington minimum hydraulic design standards.

Alternative 2 (Main Line Upgrade/Replacement of Existing System) involves upgrading the existing pipe/channel system starting from the most upstream junction on south side of New Centre Drive and continuing southeast along New Centre Drive to the intersection with South College Road. The upgrades continue southwest along South College Road to just upstream of the junction (main line junction point) where the existing pipe system crosses South College Road. The improvements then continue east along the main line through alternating runs of pipe, natural channels and culvert crossings to the downstream outlet of the system (see Figure 14).

Pipe and culvert improvements for **Alternative 2** are designed with the assumption that the entire storm network is upgraded to convey the 25-year event. Pipe improvements were designed such that the 25-year peak flow could be conveyed without surcharge. Culverts were designed to convey the 25-year peak flow with a maximum HW/D of 1.20. The culverts conform to City of Wilmington standards such that there is >1.0 feet of freeboard to the embankment elevation during the 25-year event. Proposed open channel/stream cross sections were designed from the four (4) existing surveyed sections, and checked to ensure compatibility with City of Wilmington design guidelines for 25-year peak flow capacity and velocity limitations. It should be noted that open channel/stream improvements do not differ for any of the proposed alternatives.

Pipe upgrades for this alternative involve the upgrade/replacement of the run of RCP conduit along the aforementioned route, a bore and jack installation across South College Road, and culvert upgrades at all three (3) roadway culvert crossings. Open stream channels along the main line are improved to provide capacity for facilitating heavier outflows in this alternative. Increased flows occur as this alternative relieves a significant amount of upstream flooding in the network, resulting in higher flows downstream. The improved channels serve to increase storage within the system and reduce the hydraulic grade lines of underground pipe and culverts along the system. It is recommended that a synthetic liner be installed along the improved open channels from the stream section immediately upstream of the College Acres Drive crossing down to the outfall of the system as the design velocities exceed the City of Wilmington's permissible values for a grass-lined open channel conveyance.

Figure 14 graphically represents the layout of proposed improvements for **Alternative 2**. A summary of the costs and quantities required for the construction of this alternative can be found in **Appendix B**.

❖ FLOOD REDUCTION: ALT 2

As shown in Figures 15-16, **Alternative 2** reduces the flooding impact primarily at nodes along or directly adjacent to the upgraded pipes. These upgrades reduce the ponded volume and duration of node flooding/pipe surcharge along the south side of New Centre Drive and the West side of South College Road. However, this alternative does not alleviate flooding in the vicinities of New Centre Drive/Racine Drive, College Acres Drive or at the southwest edge of the network along College Road.

Even with upgrades to natural channels to provide additional storage and flow capacity, a majority of the underground pipes and culverts along the main line required upgrading to handle the additional flows from upstream improvements. This alternative proposes upgrades to all three (3) road crossing culverts.

❖ HYDRAULIC PERFORMANCE: ALT 2

The (H&H) parameters of concern are depth of node flooding, duration of node flooding, maximum stored volume, and total ponded volume. The results for each parameter were averaged across all network nodes, and were compared to the existing conditions model in terms of a percent difference reduction for each. The **Alternative 2** model results show that the average depth of node flooding is 0.10 feet per node (-47%), the average duration of node flooding is 0.50 hours per node (-43%), the maximum stored volume is an average of 805 cubic feet per node and 4.58 ac-ft. overall (-50%) and the total ponded volume is an average of 1,081 cubic feet per node and 6.15 ac-ft. overall (-52%). The 25-year rainfall event (H&H) results for Alternative 2 are tabulated in **Table 3A** on Page 50. A breakdown of the H&H results specific to the Racine Dr./New Centre Dr. intersection and the section of New Centre Drive to the west of South College Road can be found in Table 3B on Page 51 and 3C on Page 52, respectively. **Alternative 2** demonstrates a greater hydraulic improvement advantage than Alternative 1, as conveyances are adequately sized to meet the City of Wilmington hydraulic design guidelines.

❖ CONSTRUCTION FEASIBILITY/IMPACT TO PRIVATE PROPERTY: ALT 2

Alternative 2 demonstrates several of the same issues relative to construction feasibility as evidenced in Alternative 1. A significant portion of the construction for this alternative involves work along the South College Road ROW, a major multi-lane thoroughfare for the City of Wilmington. Construction along this ROW can potentially pose major negative impacts in terms of traffic congestion and cost. **Alternative 2** requires upgrades to three (3) road crossing culverts to facilitate heavier outflows along the main line.

The alignment for **Alternative 2** does not contain a significant amount of fall from the upper-most improved inlet to the point where the alignment traverses perpendicularly

across South College Road. The flat slope along this alignment results in the requirement for large diameter piping (60" and 72") along this run. This alternative has several conduits and junction boxes (along New Centre Drive and private driveways) that do not meet the City of Wilmington's requirement for more than two (2) feet of cover.

The requirement for large diameter piping on the run upstream of the South College Road crossing results in the potential for a difficult bore & jack installation of a large diameter (72" RCP) pipe. An installation of a pipe this size may prove infeasible due to unforeseen obstacles or conflicts at the construction design phase.

The comparative construction benefit of this alternative is that the improvements are proposed along the original alignment of the existing storm system, thus reducing the potential for additional ROW/easement acquisition and potential conflict with existing utilities. This alternative requires construction within privately owned commercial and residential properties, but to a lesser degree compared to Alternative 1.

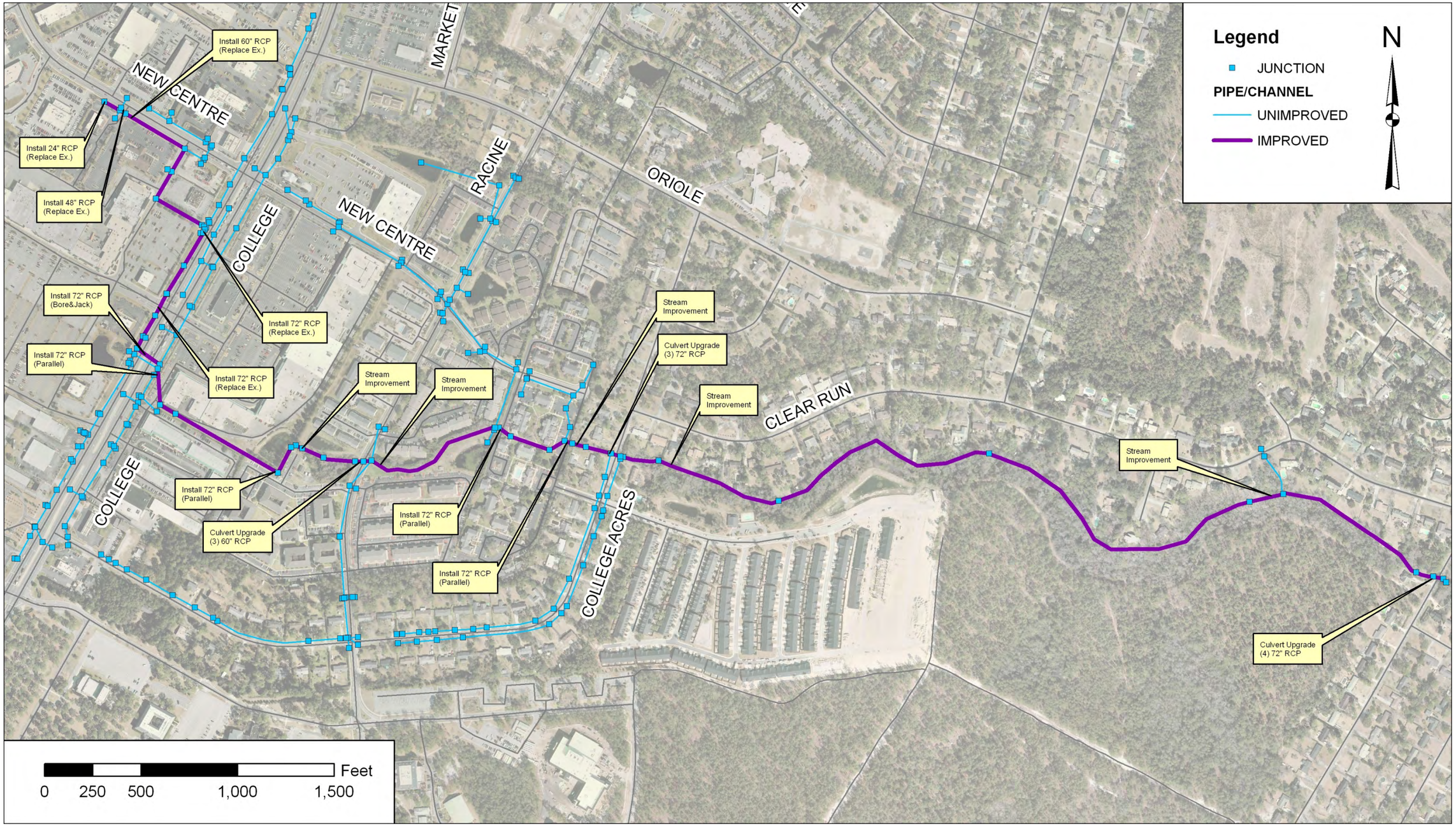

FIGURE 14: IMPROVEMENTS FOR MAIN LINE RUN – UPGRADE/REPLACEMENT – ALTERNATIVE 2

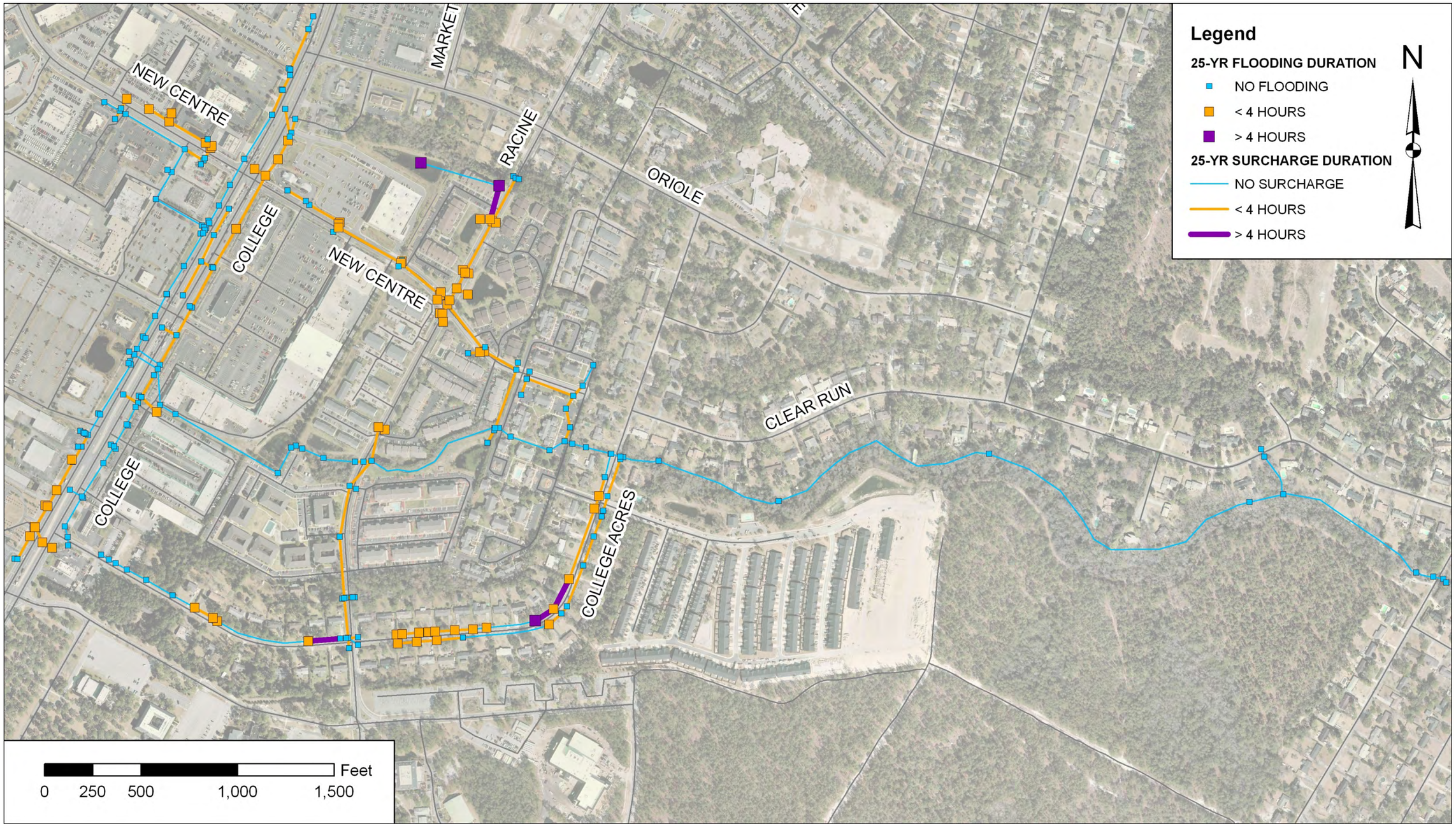

FIGURE 15: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 2

FIGURE 16: DEPTH OF FLOODING 25-YR RAINFALL EVENT– ALTERNATIVE 2

❖ PROPOSED IMPROVEMENTS: ALT 3

Alternative 2 alleviated several construction feasibility concerns with upgrades along existing alignments. As many potential construction issues are related to the proposal of work within the South College Road ROW and the improvement of key main line run structures, it was determined these issues could be avoided by diverting a significant portion of the upstream network flow along New Centre Drive, and then connecting to the existing main line at a junction much further downstream in the network, which is the solution represented by the following alternative.

The improvements outlined in **Alternative 3** ARE sized to meet the City of Wilmington minimum hydraulic design standards.

Alternative 3 (New Centre Dr. Pipe Upgrade), involves upgrading the existing pipe system starting from the most upstream junction on both the north and south sides of New Centre Drive, continuing southeast along New Centre Drive and traversing across South College Road via bore and jack pipe installation. The improvements continue southeast along New Centre Dr. to the terminus of the street at a cul-de-sac. The upgraded line then continues south until a point that ties to the existing main line (just upstream of the College Acres Drive road culvert crossing). Three (3) existing pipes will be abandoned adjacent to and west of South College Road in this alternative, which will serve to reduce flow along several of the South College Road pipe runs as well as the portion of the main line upstream of the junction EX-205. (See Node Reference Map in **Appendix A**). Further improvements to the pipe system are proposed along Racine Drive, both north and south of the intersection with New Centre Drive.

Pipe and culvert improvements for **Alternative 3** were designed with the assumption that the entire storm network is upgraded to convey the 25-year event. Pipe improvements were designed such that the 25-year peak flow could be conveyed without surcharge. Culverts were designed to convey the 25-year peak flow with a maximum HW/D of 1.20. The culverts conform to City of Wilmington standards such that there is >1.0 feet of freeboard to the embankment elevation during the 25-year event. Proposed open channel/stream cross sections were designed from the four (4) existing surveyed sections, and checked to ensure compatibility with City of Wilmington design guidelines for 25-year peak flow capacity and velocity limitations. It should be noted that open channel/stream improvements do not differ for any of the proposed alternatives.

Pipe upgrades for this alternative involve installing/replacing the run of RCP conduit along the aforementioned route, a bore and jack installation across South College Road, and culvert upgrades at the College Acres Drive and Mallard Street road crossing. In this alternative, open stream channels along the main line run have been designed with improved capacity to facilitate heavier outflows along the main line downstream of the new junction point. Increased flows occur because this alternative relieves a significant amount of upstream flooding in the network, resulting in higher flows moving downstream. The

improved channels upstream of the new junction point serve to increase storage within the system and reduce the hydraulic grade lines of underground pipe and culverts along this portion of the run. The increased storage and diverted flow pattern will reduce the scope of improvements required at the upstream road culvert crossing at Racine Drive. It is recommended that a synthetic liner be installed along the improved open channels from the stream section immediately upstream of the College Acres Drive crossing down to the outfall of the system.

Figure 17 graphically represents the layout of proposed improvements for **Alternative 3**. A summary of the costs and quantities required for the construction of this alternative can be found in **Appendix B**.

❖ FLOOD REDUCTION: ALT 3

As shown in Figures 18-19, **Alternative 3** reduces the flooding impact primarily at nodes along or directly adjacent to the upgraded pipes. These upgrades reduce the ponded volume and duration of node flooding/pipe surcharge along the north and south side of New Centre Drive, along the majority of the west and east sides of South College Road and in the vicinity of New Centre Drive/Racine Drive.

This alternative does not address flooding along the majority of College Acres Drive or at the southwest edge of the network along the west side of South College Road. The improvements proposed in Alternative 3 would facilitate additional storm system upgrades in these areas at a later time. Please refer to the discussion of proposed improvements in the Alternative 4 analysis on Pages 41-42.

❖ HYDRAULIC PERFORMANCE: ALT 3

The (H&H) parameters of concern are depth of node flooding, duration of node flooding, maximum stored volume, and total ponded volume. The results for each parameter were averaged across all network nodes, and were compared to the existing conditions model in terms of a percent difference reduction for each. The **Alternative 3** model results show that the average depth of node flooding is 0.04 feet per node (-79%), the average duration of node flooding is 0.26 hours per node (-71%), the maximum stored volume is an average of 302 cubic feet per node and 1.53 ac-ft. overall (-82%) and the total ponded volume is an average of 383 cubic feet per node and 1.94 ac-ft. overall (-83%). The 25-year rainfall event (H&H) results for **Alternative 3** are tabulated in **Table 3A** on Page 50. A breakdown of the H&H results specific to the Racine Dr./New Centre Dr. intersection and the section of New Centre Drive to the west of South College Road can be found in Table 3B on Page 51 and 3C on Page 52, respectively.

Alternative 3 demonstrates a significantly greater overall hydraulic benefit than Alternative 2.

❖ CONSTRUCTION FEASIBILITY/IMPACT TO PRIVATE PROPERTY: ALT 3

In addition to a more substantial reduction of flooding within the model compared to the previous alternative, Alternative 3 demonstrates fewer issues in terms of construction feasibility. A significant portion of the construction for this alternative involves work along the New Centre Drive ROW, and avoids almost all construction within the more heavily travelled South College Road ROW. **Alternative 3** requires upgrades to three (3) road crossing culverts (at College Acres Drive, Racine Drive and Mallard Street), which will result in a similar level of traffic interruption as the previous alternatives.

Alternative 3 proposes the least amount of construction within privately owned commercial and residential properties, which in comparison to the other alternatives, decreases the need for additional land acquisition and reduces the potential for conflict with existing utilities and/or BMP's. This alternative demonstrates the least overall potential among all proposed alternatives for interruption to traffic flow within the study area.

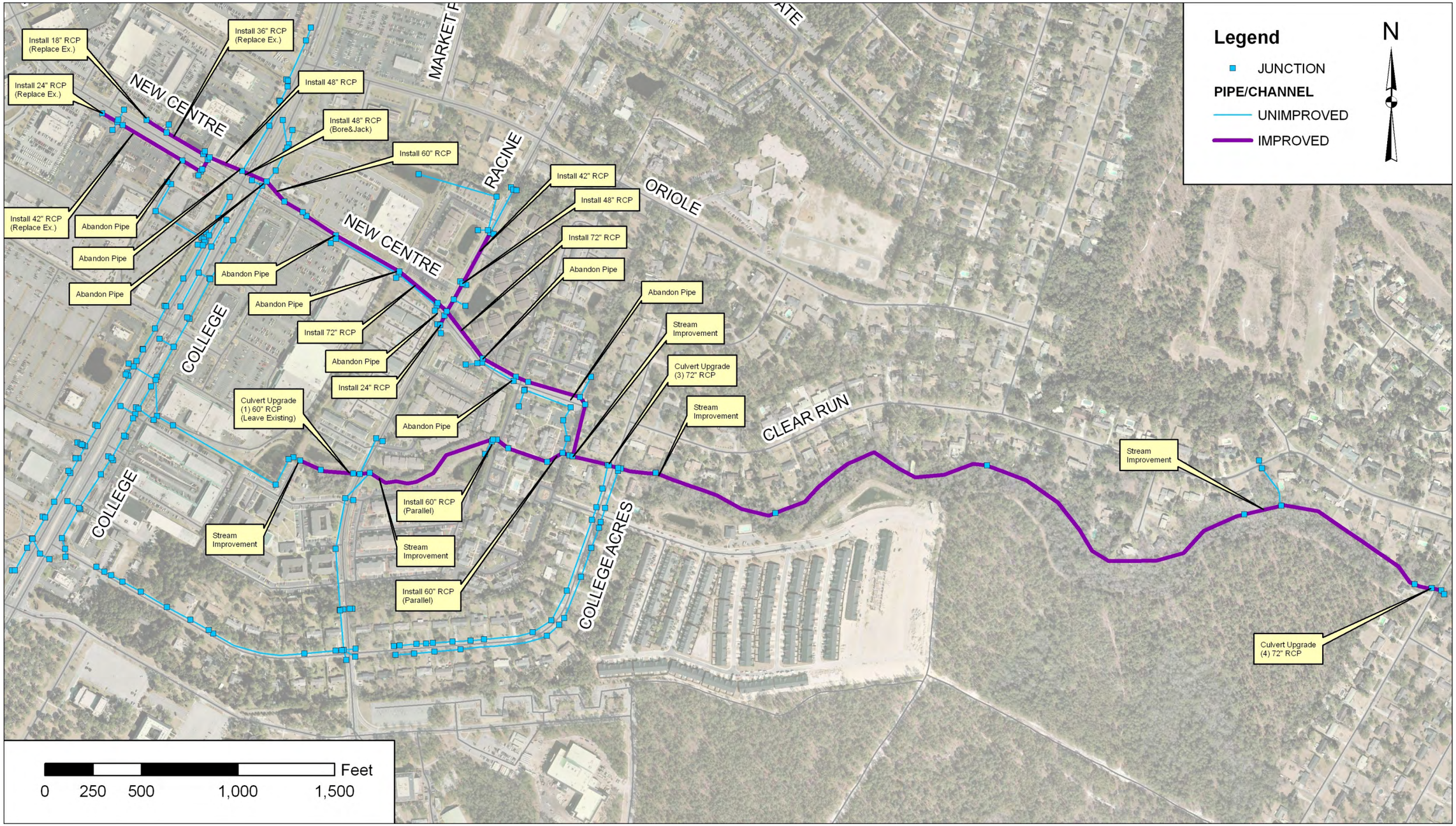

FIGURE 17: IMPROVEMENTS FOR NEW CENTRE DIVERSION – UPGRADE/REPLACEMENT – ALTERNATIVE 3

FIGURE 18: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 3

FIGURE 19: DEPTH OF FLOODING 25-YR RAINFALL EVENT – ALTERNATIVE 3

❖ PROPOSED IMPROVEMENTS: ALT 4

Although Alternative 3 has shown the least potential for construction feasibility problems and the greatest reduction in flooding, it still does not address flooding along the majority of College Acres Drive or at the southwest edge of the network along College Road. This shortfall in Alternative 3 was the impetus for the fourth and final alternative.

The improvements outlined in **Alternative 4** ARE sized to meet the City of Wilmington minimum hydraulic design standards.

Alternative 4 (New Centre Drive Pipe Upgrade & Pinch Points), involves upgrading the underground piping system along New Centre Drive and improving the natural channels along the main line just as proposed in Alternative 3. This alternative (considered a supplementary to Alternative 3) also addresses apparent flooding concerns in the (2) two areas described in the above paragraph. Figure 20 graphically represents the layout of the proposed improvements for this alternative. A summary of the costs and quantities required for the construction of this alternative can be found in **Appendix B**.

Pipe and culvert improvements for **Alternative 4** were designed with the assumption that the entire storm network is upgraded to convey the 25-year event. Pipe improvements were designed such that the 25-year peak flow could be conveyed without surcharge. Culverts were designed to convey the 25-year peak flow with a maximum HW/D of 1.20. The culverts conform to City of Wilmington standards such that there is >1.0 feet of freeboard to the embankment elevation during the 25-year event. Proposed open channel/stream cross sections were designed from the four (4) existing surveyed sections, and checked to ensure compatibility with City of Wilmington design guidelines for 25-year peak flow capacity and velocity limitations. It should be noted that open channel/stream improvements do not differ for any of the proposed alternatives. It is recommended that a synthetic liner be installed along the improved open channels from the stream section immediately upstream of the College Acres Drive crossing down to the outfall of the system.

❖ FLOOD REDUCTION: ALT 4

Figures 21-22 illustrate that **Alternative 4** significantly reduces node flooding for the 25-year storm just as in Alternative 3. In addition to the flooding improvements demonstrated in Alternative 3, this alternative significantly reduces flooding along College Acres Drive and the southwest edge of the network along South College Road.

❖ HYDRAULIC PERFORMANCE: ALT 4

The (H&H) parameters of concern are depth of node flooding, duration of node flooding, maximum stored volume, and total ponded volume. The results for each parameter were averaged across all network nodes, and were compared to the existing conditions model in terms of a percent difference reduction for each. The **Alternative 4** model results show that the average depth of node flooding is 0.006 feet per node (-97%), the average duration of node flooding is 0.02 hours per node (-97%), the "maximum ponded volume" is an average of 70 cubic feet per node and 0.39 ac-ft. overall (-96%) and the "total ponded volume" is an average of 51 cubic feet per node and 0.29 ac-ft. overall (-98%). The 25-year rainfall event (H&H) results for **Alternative 4** are tabulated in **Table 3A** on Page 50. A breakdown of the H&H results specific to the Racine Dr./New Centre Dr. intersection and the section of New Centre Drive to the west of South College Road can be found in Table 3B on Page 51 and 3C on Page 52, respectively.

Alternative 4 demonstrates a greater hydraulic improvement than all previous alternatives.

A short segment of improved piping just west of the Racine Drive and College Acres Drive intersection remains surcharged due to the high tailwater in the downstream, unimproved pipe run on Racine Drive. Improvements are appropriately sized for the design event, and serve to mitigate flooding adjacent to this intersection.

❖ CONSTRUCTION FEASIBILITY/IMPACT TO PRIVATE PROPERTY: ALT 4

In addition to a more substantial reduction of flooding within the model, **Alternative 4** demonstrates the second fewest issues in terms of construction feasibility. However, additional improvements along South College Road and College Acres Drive have the potential to increase traffic congestion in those areas compared to Alternative 3.

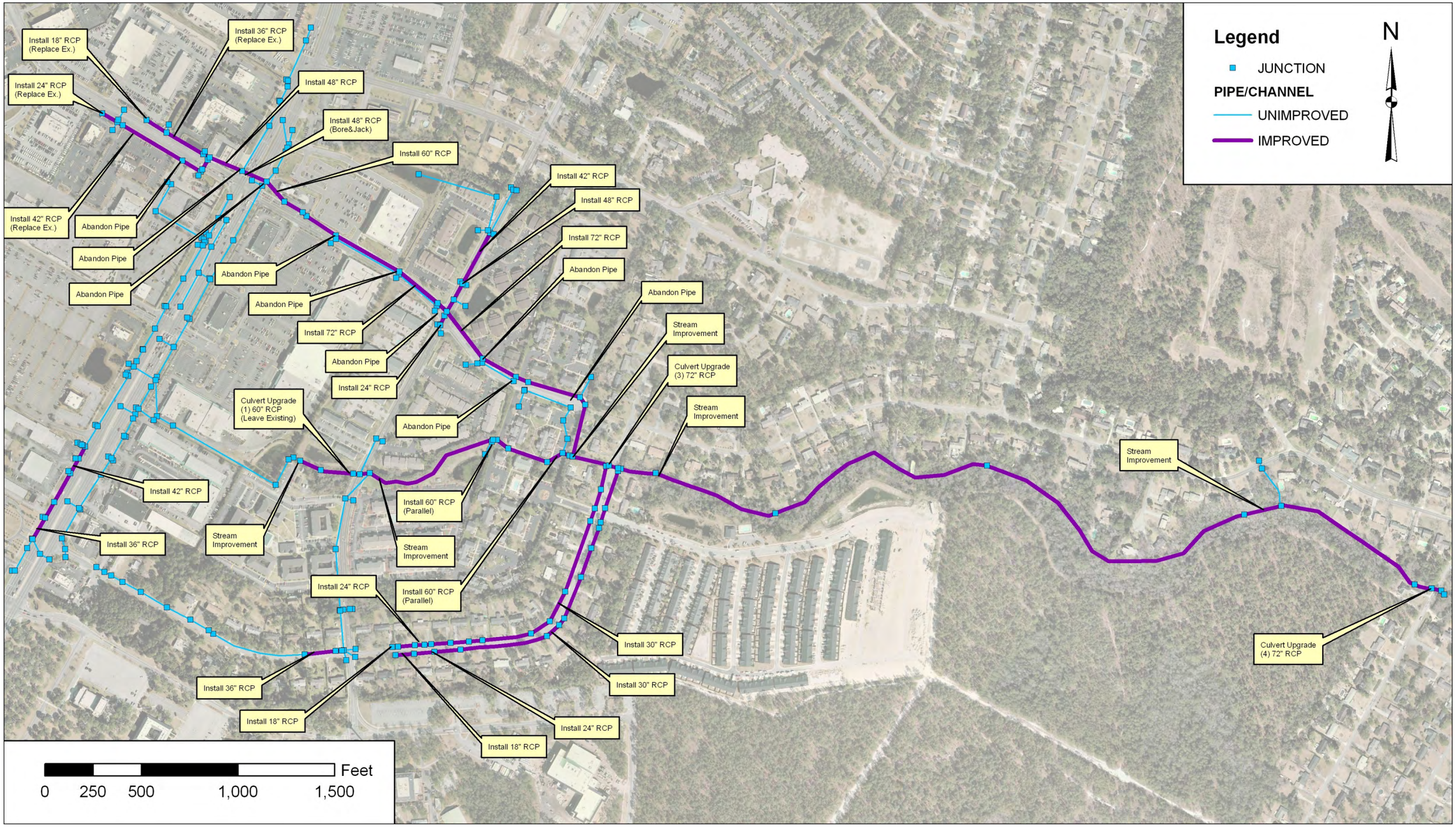

FIGURE 20: IMPROVEMENTS FOR NEW CENTRE DIVERSION & PINCH POINTS – ALTERNATIVE 4

FIGURE 21: DURATION FLOODING/SURCHARGE FOR 25-YEAR RAINFALL EVENT – ALTERNATIVE 4

FIGURE 22: DEPTH OF FLOODING 25-YR RAINFALL EVENT – ALTERNATIVE 4

VI. OPINIONS OF PROBABLE COST

Preliminary opinions of probable costs were developed for each of the modeled alternatives. Unit costs were based on available data from *2010 RSMeans Heavy Construction Cost Data.* A summary of the costs estimates for each Alternative can be found in **Table 2**. Cost estimates taken from *RSMeans 2010* that are used in the probable cost summary in **Appendix B** include the following:

- RCP Pipe Installation
- Inlet/Junction Structures
- Pipe Bedding
- Riprap
- Cast-in-Place Headwalls
- Grading/Trenching
- Demolition/Repair
- Erosion Control – Inlet Protection/Silt Fence

It should be noted that *RSMeans* data are nationally averaged, but include regional adjustment factors which would account for lower labor costs in the Wilmington area. However, the regional adjustment factors were not applied in order to allow a margin of safety to account for increased costs on this project stemming from construction in a landscape with extensive infrastructure already built. It should also be noted that *RSMeans* unit costs already include costs for equipment, overhead and profit, so these items are not reflected separately in the enclosed Opinions of Probable Cost.

Some cost assumptions for data not available in *RSMeans* were taken from the City of Wilmington's preliminary cost estimate spreadsheet and from input from City Staff based on costs associated with recent projects in the Wilmington. Such cost line items and assumptions include the following:

- Cost per linear foot of Bore & Jack installation of metal pipe casing
- $10,000/AC for Clearing & Grubbing
- 1% City of Wilmington Construction Administration Cost
- 2% City of Wilmington Engineering Inspections Cost

Cost estimates for easement acquisitions were based on the assumption that pipe easements would be 20 feet wide and the stream easements would be 50 feet wide. The somewhat narrow stream easement was utilized in order to keep the land acquisition costs at a minimum. However, the functional construction corridor may be greater than 50 feet in some areas due to the presence of an existing sewer line along the stream. Based on recent acquisitions of easements by the City and on an assessment of current market values for the various categories of real estate within the study area, the following unit area costs were used to calculate the costs of easements:

- $3.75/SF for Single Family Residential areas
- $10.00/SF for Multi-Family Residential areas
- $10.00/SF for Retail/Business areas with no major thoroughfare frontage
- $15.00/SF for Retail/Business areas with major thoroughfare (primarily College Road) frontage

A GIS analysis of tax parcels was utilized to divide the parcels affected by easement acquisition needs into the various cost categories above, and to eliminate projected easement acquisitions where exiting easements and rights-of way could be utilized for the planned drainage system improvements.

Remaining cost estimates were made on the basis of the experience of M&N staff with construction of similar projects in urban areas of North Carolina:

- $200,000 Traffic Control
- $100,000 Lump Sum for Mobilization
- $100,000 for Erosion Control
- $100,000 for Erosion Control Restoration
- $75, 000 for Pump Around arrangements during stream construction
- $172,000 Utility/Coordination (Detailed Breakdown in **Appendix B**)
- $300/LF for stream restoration (inclusive of all excavation and take-offs)
- 20% Construction Cost Contingency Factor
- 10% Engineering Fees

Finally the Opinions of Probable Cost were adjusted to account for inflation. According to the most recent data from the US Bureau of Labor Statistics, annual inflation rates have averaged 2.2% per year over the last five years. The Opinions of Probable Cost herein were increased by applying five successive years of inflation at 2.5% annually, assuming a moderate degree of improvement in the economy over the next five years.

ALTERNATIVE	DESCRIPTION	PROBABLE COST
1	Main Line Parallel Network (Wilmington Schematic)	$9,226,000
2	Main Line Upgrade/Replacement	$9,572,000
3	New Centre Drive Diversion	$9,570,000
4	New Centre Drive Diversion & Pinch Points	$10,732,000

TABLE 2: OPINIONS OF PROBABLE COST

The cost estimate of $6,416,000 provided in the original City of Wilmington spreadsheet for Alternative 1 was deemed too low due to the exclusion of various lump sum and unit cost items. After the inclusion and adjustment of items such as construction surveying, traffic control, utility coordination, demolition/replacement, Riprap, CIP concrete headwalls/endwalls, pipe bedding and backfill, the probable cost for Alternative 1 in

Appendix B is estimated at **$9,226,000**. This probable cost estimate reflects a cost discrepancy of $2,810,000 more than the original sum, but is considered to be a more detailed, accurate estimate of construction costs. It should also be noted that the City's original estimate only included simple assumptions for per square foot cost allowances for acquisition of easements and/or additional ROW, which resulted in a total cost of approximately $800,000 for easements, whereas the more detailed estimate of easement acquisition costs presented herein yielded an estimate of $1,900,000 for Alternative 1. As a result, over $1,000,000 in the discrepancy in cost estimates between the two efforts is accounted for by easement costs.

VII. SELECTION OF PREFERRED ALTERNATIVE

Alternative 3 is the preferred initial alternative in terms of relative ease of construction and hydraulic performance. **Alternative 3** is more feasible than Alternative 2, as the majority of construction occurs along New Centre Drive and Racine Drive instead of South College Road. **Alternative 3** thereby minimizes the impact to traffic flow in the area compared to either Alternative 2.

Alternative 1 is not considered a viable alternative, as it is only a representation of a City of Wilmington design schematic. Alternative 1 is thus not considered as it does not meet critical City of Wilmington design guidelines for hydraulic capacity.

Additionally, **Alternative 3** provides the greatest overall reduction of flooding/tailwater at critical junctions in the system. Flooding is mitigated at the Racine Drive & New Centre drive intersection and the section of New Centre Drive to the west of South College Road to a much greater extent than any other alternative. The three culvert crossings along the Clear Run outfall will be designed to meet the 25-year criteria for all alternatives, however the overall cost of culvert replacement is lowest in Alternative 3 given that a portion of the drainage area will be diverted away from the upstream end of the outfall. **Alternative 3** is slightly less expensive than Alternative 2, with a greater comparative hydraulic benefit in critical areas and the overall system watershed.

If flooding persists upstream of areas with undersized pipes as described in Alternative 4, additional improvements can be installed at a later date to maximize the reduction of flooding in those areas. These additional improvements are most feasible in terms of hydraulic benefit through the construction of **Alternative 3.** Alternative 4 shows the additional pipe upgrades that can be beneficial in areas of localized system flooding.

The GIS inventory survey provided for this phase of design was sufficient for preliminary and conceptual design and selection of a preferred alternative. The construction level design will require collection of additional survey data and detailed modeling to ensure the viability of the chosen alternative. Full topographical and existing utility survey will be required along the alignment of the proposed improvements for construction level design.

In conjunction with the topographical and existing utility survey, collection of feature elevation data for select BMP's and underground storm pipes will be needed to supplement detailed hydraulic modeling and construction-level design. The additional survey data will ensure that the final hydraulic model is more accurate and that the proposed improvements are not under or over-designed. The additional information will allow the engineer to modify the final design to introduce cost savings where applicable. The cost savings potential for the final design outweighs the expenditure required to compile the additional data.

VIII. HYDRAULIC & HYDROLOGIC IMPROVEMENT SUMMARY

CLEAR RUN OUTFALL – OVERALL NETWORK

HYDRAULIC PERFORMANCE ANALYSIS & ALTERNATIVE COMPARISON

25-YEAR DESIGN RAINFALL EVENT

		Scenario	EXISTING	ALT 1	ALT 2	ALT 3	ALT 4
	UNITS						
PER NODE AVG. "DEPTH OF FLOODING"	FEET		0.20	0.13	0.10	0.04	0.006
PER NODE AVG. "DURATION OF FLOODING"	HOURS		0.87	0.60	0.50	0.26	0.02
PER NODE AVG."MAX. STORED VOLUME"	CUBIC FEET		1,619	1,008	805	302	70
OVERALL "MAX. STORED VOLUME"	ACRE-FEET		9.22	5.74	4.58	1.53	0.39
PER NODE AVG. "TOTAL PONDED VOL"	CUBIC FEET		2,241	1,391	1,081	383	51
OVERALL "TOTAL PONDED VOL"	ACRE-FEET		12.76	7.92	6.15	1.94	0.29

PERCENT REDUCTION						
		Scenario	ALT 1	ALT 2	ALT 3	ALT 4
	UNITS				PREFERRED	
PER NODE "DEPTH OF FLOODING"	FEET		-34%	-47%	-79%	-97%
PER NODE "DURATION OF FLOODING"	HOURS		-31%	-43%	-71%	-97%
PER NODE "MAXIMUM STORED VOLUME"	CUBIC FEET		-38%	-50%	-82%	-96%
PER NODE "TOTAL PONDED VOLUME"	CUBIC FEET		-38%	-52%	-83%	-98%
OPINION OF PROBABLE COST	USD		$ 9,226,000	$ 9,572,000	$ 9,570,000	$ 10,732,000

TABLE 3A: HYDRAULIC AND HYDROLOGIC IMPROVEMENT SUMMARY – OVERALL NETWORK

CLEAR RUN OUTFALL – INTERSECTION OF RACINE DRIVE & NEW CENTRE DRIVE

HYDRAULIC PERFORMANCE ANALYSIS & ALTERNATIVE COMPARISON (NODES 143-145, 156-161, 169-171).

25-YEAR DESIGN RAINFALL EVENT

		Scenario	EXISTING	ALT 1	ALT 2	ALT 3	ALT 4
	UNITS						
PER NODE AVG. "DEPTH OF FLOODING"	FEET		0.92	0.59	0.52	0.00	0.00
PER NODE AVG. "DURATION OF FLOODING"	HOURS		4.49	2.43	1.97	0.00	0.00
PER NODE AVG."MAX. STORED VOLUME"	CUBIC FEET		8,198	4,484	3,811	24	24
OVERALL "MAX. STORED VOLUME"	ACRE-FEET		2.26	1.24	1.05	<0.01	<0.01
PER NODE AVG. "TOTAL PONDED VOL"	CUBIC FEET		9,998	5,600	4,705	0.00	0.00
OVERALL "TOTAL PONDED VOL"	ACRE-FEET		2.75	1.54	1.30	0.00	0.00

PERCENT REDUCTION		**Scenario**	ALT 1	ALT 2	ALT 3	ALT 4
	UNITS				PREFERRED	
PER NODE "DEPTH OF FLOODING"	FEET		-35%	-43%	-100%	-100%
PER NODE "DURATION OF FLOODING"	HOURS		-46%	-56%	-100%	-100%
PER NODE "MAXIMUM STORED VOLUME"	CUBIC FEET		-45%	-54%	-99.7%	-99.7%
PER NODE "TOTAL PONDED VOLUME"	CUBIC FEET		-44%	-53%	-100%	-100%
OPINION OF PROBABLE COST	USD		$ 9,226,000	$ 9,572,000	$ 9,570,000	$ 10,732,000

TABLE 3B: HYDRAULIC AND HYDROLOGIC IMPROVEMENT SUMMARY – RACINE & NEW CENTRE DRIVE

CLEAR RUN OUTFALL – NEW CENTRE DRIVE, WEST OF S. COLLEGE AVENUE

HYDRAULIC PERFORMANCE ANALYSIS & ALTERNATIVE COMPARISON (NODES 4-5, 35-36, 57-58, 61-63, 67-74).

25-YEAR DESIGN RAINFALL EVENT

		Scenario	EXISTING	ALT 1	ALT 2	ALT 3	ALT 4
	UNITS						
PER NODE AVG. "DEPTH OF FLOODING"	FEET		0.59	0.39	0.17	0.03	0.03
PER NODE AVG. "DURATION OF FLOODING"	HOURS		2.03	1.44	0.60	0.07	0.07
PER NODE AVG."MAX. STORED VOLUME"	CUBIC FEET		4,752	3,113	1,167	253	253
OVERALL "MAX. STORED VOLUME"	ACRE-FEET		1.85	1.21	0.46	0.10	0.10
PER NODE AVG. "TOTAL PONDED VOL"	CUBIC FEET		6,115	4,456	1,609	347	347
OVERALL "TOTAL PONDED VOL"	ACRE-FEET		2.39	1.74	0.63	0.14	0.14

PERCENT REDUCTION						
		Scenario	ALT 1	ALT 2	ALT 3	ALT 4
	UNITS				PREFERRED	
PER NODE "DEPTH OF FLOODING"	FEET		-34%	-71%	-94%	-94%
PER NODE "DURATION OF FLOODING"	HOURS		-29%	-71%	-97%	-97%
PER NODE "MAXIMUM STORED VOLUME"	CUBIC FEET		-34%	-75%	-95%	-95%
PER NODE "TOTAL PONDED VOLUME"	CUBIC FEET		-27%	-74%	-94%	-94%
OPINION OF PROBABLE COST	USD		$ 9,226,000	$ 9,572,000	$ 9,570,000	$ 10,732,000

TABLE 3C: HYDRAULIC AND HYDROLOGIC IMPROVEMENT SUMMARY – NEW CENTRE DRIVE (WEST OF S. COLLEGE AVE.)