COA Review Comments
Euclid / Wilson Storm Drain Improvement Project, 90% Submittal
Subproject I.D. No. 5789.035

Comments by Sergio Mendoza, P.E. (Local Flood Hazard Mitigation Group)

StormCAD Model: LAN_reroute System A PROFILES.stc

1. The following conduits do not meet the minimum allowable velocities for the 25-Year storm event: LT-A4-04 and LT-A8-05. See DCM 5.3.1.
2. The following inlets do not meet the minimum allowable size of 10 ft: IN-CIPP-A20, IN-A4-A11b2 and IN-A4-A11b1. See DCM 4.1.0.B.

StormCAD Model: Line_B_60%.stc

3. Conduits SD-B1-05, SD-X3-01 and LT-B1-14 do not meet the maximum allowable velocity of 20 ft/sec. See DCM 5.3.2.
4. Transitions PC-B1-02 and PC-B1-03 are modeled with no headlosses. Transition PC-B1-03 is modeled as having an absolute headloss of 0 ft. Please revise the model or justify why there are no headlosses at the transitions. See DCM 5.5.2.
5. The headloss method for inlet IN-B1-B06b is listed as absolute with a headloss of 0 ft. See DCM 5.5.2.
6. The time of concentration for DA-B11 varies between storm events.

Excel Workbook: 2011 01 31 CB Spread Analysis.xlsx

7. The calculations for clear width are incorrect, accounting for spread on one side of the street only. The clear width at the crown of the street must take into account the spread on both sides of the roadway. (Repeat comment) See DCM Section 3.

Excel Workbook: 2011 01 31 Drainage Area Map Table.xlsx

8. Drainage areas DA-A11a, DA-A11b1, DA-A11b2, DA-A11c, DA-A21 and DA-A22 are missing from the spreadsheet but are listed in the StormCAD file "LAN_reroute System A PROFILES.stc."
9. Drainage area DA-A11 is listed in the spreadsheet but not found in the StormCAD model.
10. Drainage areas DA-B00, DA-X02 and DA-X03 are missing from the spreadsheet but are listed in the StormCAD file "Line B-90%.stc."
11. Drainage areas DA-B06a and DA-B06b are listed in the spreadsheet but not found in the StormCAD model.
12. The time of concentration for DA-B11 differs from the StormCAD model.
13. The Rational C-value for DA-A17b, DA-A20 and DA-B11 differ from the StormCAD model.

Comments by John Driscoll, P.E. (Local Flood Hazard Mitigation Group)

StormCAD Models, Systems A and B

14. Headloss Coefficients at Junctions: The use of DCM coefficients in the Standard Headloss Method in StormCAD yields headlosses that are too low when compared to the headloss methodology (Eq. 5-6 and Fig. 5-10) in the DCM. Eq. 5-6 in the DCM should be used to calculate headlosses for the specific conditions where the DCM provides the headloss coefficient (Table 5-4). Eq. 5-6 is the same as the generic method in StormCAD where K2 is set equal to 1.0. We noticed several instances where the model needs to be revised for this reason. Among them are structures MH-A2-02, WC-A2-01 (id 3410), MH-A2-03, MH-A2-04, WC-A2-01 (id 3461), WC-A5-01, WC-A5-02, MH-A5-04, WC-A5-03, MH-A5-05, MH-A5-06, WC-A5-04, MH-A5-07, WC-A5-05, MH-A5-07, MH-A1-10, WC-A1-01, MH-A4-01, MH-A4-02, MH-A6-01, MH-A1-15, MH-A9-01, WC-A1-06, WC-B1-01, WC-B1-02, WC-B1-03. All junctions should be checked for proper use of the coefficient and the corresponding methodology using the coefficient. Also, this aspect of the design needs to be corrected in subsection 5.1 of the Engineering Design Report.
15. There are two structures labeled WC-A2-01 (id3410) and WC-A2-01 (id 3461). Is this intentional?
16. There are two structures labeled MH-A5-07. Is this intentional?
17. Headloss coefficients at bends: The coefficients in Table 5-4 for bends are intended for bends where a smooth radius is provided equal to the diameter of the pipe. Is LAN proposing to provide smooth radii within the manholes? If not, then the coefficients for headloss at bends in Table 5-4 are likely too low. The use of the coefficients and methodology provided in HEC-22 appears to be more appropriate where the flow is being deflected at a manhole and a smooth radius is not being provided. Some of the bends where the headloss appears to be too low are MH-A2-01, MH-A5-07 (the structure labeled MH-A5-07 at the south end of Wilson St.), MH-A1-11, Bend A1-03, Bend A1-05, Bend A4-01, Bend WC-A9-01, MH-A6-02, MH-A1-18, WC-CIPP-01, WC_CIPP-02, WC-CIPP-03, WC-CIPP-04, PC-B1-01, PC-B1-05, WC-X1-01, PC-B1-06. All deflections should be checked for use of the proper headloss coefficient.
18. The structure labels in the model should match the structure labels on the drawings. Discrepancies between labels in the model and labels on the drawings make it difficult to confirm that the proper headloss coefficients are being used.
19. Bend A1-03 in the model is called Bend A1-05a in the drawings.
20. MH-A1-11 in the model is called Bend A1-11 on the drawings.
21. MH-A1-12 in the model is called MH-A1-11 on the drawings.
22. Bend A1-06 in the model is called Bend A1-11b on the drawings.
23. MH-A1-13 in the model is called Mh-A1-12 on the drawings.
24. MH-A1-14 in the model is called MH-A1-13 on the drawings.

25. MH-A1-15 in the model is called MH-A1-14 on the drawings.
26. MH-A1-16 in the model is called MH-A1-15 on the drawings.
27. MH-A4-01 is called WC-A4-01 on the drawings.
28. MH-A4-05 in the model is called MH-A4-018 on the drawings.
29. WC-A2-01 in the model is called WC-A2-02 on the drawings.
30. T-108 in the model is called MH-A5-03 on the drawings.
31. MH-A5-03 in the model is called MH-A5-05 on the drawings.
32. MH-A5-07 in the model is called MH-A5-08 on the drawings.
33. MH-A1-17 in the model is called MH-A1-16 on the drawings.
34. Bend A1-08 in the model is called Bend A1-16b on the drawings.
35. MH-A1-18 in the model is called MH-A1-17 on the drawings.
36. J-464 in the model is called MH-A8-04 on the drawings.
37. WC-A8-05 in the model is called WC-A8-02 on the drawings.
38. The headloss coefficient for T-108 seems low.
39. The headloss coefficient for MH-A5-03 seems low.

Engineering Design Report

40. Engineering Design Report, Subsection 1.0, Creek Flood Mission: The discussion on the pond belongs under Water Quality Mission (Water Quality Treatment). The last sentence is confusing because there is only one system draining to the pond under the alternative selected for design.
41. Engineering Design Report, Subsection 3.1, Public Coordination: Public meetings held during the preliminary engineering phase should also be listed.
42. Engineering Design Report, Subsection 5.1: Revise the discussion on the use of DCM coefficients and the appropriate methodology to be used.
43. Engineering Design Report, Subsection 5.2: The reference to Appendix G should read Appendix H. Please provide electronic copies of the spreadsheets used to model pond drawdown, orifice sizing, weir sizing, and outfall pipe design.

Comments by Arthur Romero, P.E. (Local Flood Hazard Mitigation Group)

Construction Plans

Comments not addressed from 60% review

44. Sheet CS-128, Sta 21+94: There is a conflict with a 4” gas line. I don’t see a note to move Gas Line.
45. Sheet CS 129, Sta 23+45: Please show existing SD in the profile.
46. Sheet CS-140 & 141, Roughly Sta 4+10: Please make a note that the existing SD crossing Oltorf will not be tying into the new system, at that point. I see the new

inlet, but do not see a note as to whether the existing SD ties into the new SD. I see the two profiles Not meeting. I see the existing SD south of Oltorf to be removed.

47. CS-164 & 166: There are multiple references to 'Storm Sewer' in connection with curb inlets. Please change these to 'Storm Drain'. See laterals at Sta. 8+00 and 11+62

Comments by Andrea Henry, P.E. (Creek Flood Hazard Mitigation Group)

48. On page 16 of the engineering report, LAN states "The lag time parameter was assumed to be 60% of the longest system travel time for the Current Conditions StormCAD model.". Although I understand the reasoning re: the size of the small drainage areas for System A and System B vs. the existing HMS sub-drainage areas, I still think using the StormCAD travel time (Rational Method), even with a 60% adjustment, would be too fast to accurately portray the impacts to the main stem of East Bouldin. Additionally, it is inconsistent with the rest of the model.

49. Please add the 10-yr storm to the Peak Flow Summary Table.

50. Comments by Kristin Kasper Pipkin, P.E. (Stream Restoration Program)

51. Sheet SB-102:

- Note Section:
  - Note 4.2 - Provide openings in grout for water to free drain through boulder wall.
  - Note 9 - Do not use grout to secure rocks - use a sufficient size material so rock riprap is stabilize without grout.
  - Note 9 - Rock riprap will not be placed below or above tree roots, correct?
  - Note 10 - Why do we need gravel within 5' of tree? Avoid working around trees or in the root zone. Confirm that design does not negatively impact tree with John Gleason (974-3543).

52. Sheet SB-103:

- What is the bank slope at the rock riprap transition? In 60% response, it was mentioned that rock riprap is necessary due to a steep bank slope but soil lifts can be used at steeper slopes than rock riprap. Rock riprap should only be used at slope less than or equal to 1.5H:1V. Potentially tie in soil lifts directly into limestone headwall.

53. Sheets SB-105:

- STA 10+75 and 11+00 - Channel flowline is at the same elevation as the foundation of the retaining wall. Lower wall below the potential channel scour depth. If this section of the reach is bedrock, adjust the design for a bedrock channel (remove the footers and place the foundation boulder at grade)

54. Sheets SB-106:

- STA 13+25 - Use soil retention blanket instead of rock riprap above flood bench. Soil retention blankets, such as Class 2, Types G or H can be used on slopes up to 1:1.
- STA 13+77 - Boulder wall is designed above the natural grade behind the wall. Adjust design so top of wall ties into natural grade behind wall.
- Is the rock riprap still vegetated? If so, the landscape plans do not indicate it (as mentioned in the responses).

55. Sheet SB-501:

- Detail 3 & 4 - Show contact between boulders and soil lifts
- Detail 7 & 9 - Will detail be used ONLY if exposed roots are not in area? Rock will not be placed above or below exposed roots? Please contact John Gleason about design in these areas.
- All details - Use a permanent fabric such as Class 2, Type G or H (based on channel hydraulics) to stabilize slopes. Contact Kristin Pipkin (974-3315) if you would like to discuss.

Comments by Tom Franke, E.I.T. (Storm Water Treatment Program)

56. On Sheet WQ-102: ECM 1.6.5(A)(4), <u>Underdrain Piping</u>, states, The maximum spacing for the laterals should be ten (10) feet between laterals and five (5) feet from a wall or side. It appears that the underdrain piping nearest the sedimentation chamber is too close to the sedimentation chamber and hedgerow. To avoid potential short circuiting please eliminate the underdrain piping located between the hedgerow/rock berm and the 5 inch trunk line nearest the sedimentation chamber.

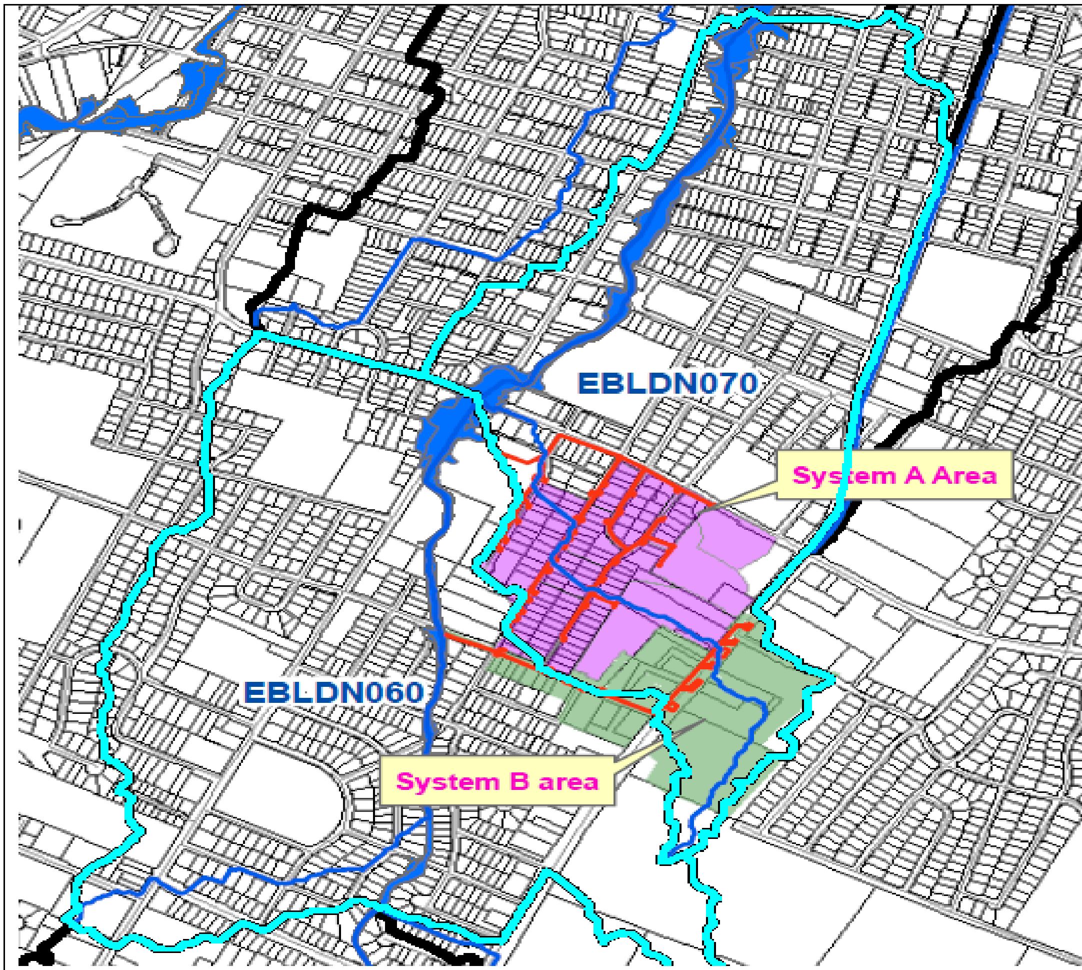
57. On Sheet WQ-103: The Appendix R table, for the proposed conditions, shown on this sheet appears to have the incorrect drainage area and capture depth to the proposed Water Quality pond. Please verify and make the necessary corrections where appropriate.

58. On Sheet L103: Please contact and coordinate the landscape design and proposed plantings selection with John Gleason (974-3543).

Comments by Fang Yu, P.E., PhD (Value Engineering Team)

Project Item: 90% Design Plans and Report / Euclid_Wilson SDIP
Data Receiving Date: 2/1/2011
Comments Submitted Date: 2/15/2011

Brief Project Summary and Background Information: This is a mission-integration project with flood control as the primary mission. The project area is bounded by W. Oltorf on the north, S. Congress Avenue on the East, Cumberland Road on the South and East Bouldin Creek on the west. The existing storm drainage systems were installed in the 1960s and their flow capacities are rather limited. Significant local flooding occurs in the area under relatively small storm events.
VE Comment #1: On Page 16 of the 90% design report, it says; "it was noted that basin EBLDN060 of the original FIS model was split into basins 'EBLDN060' and 'Euclid-Wilson Area' to take into account and evaluate the effects of the proposed storm water development in the area. The AECOM PER left all variables unchanged in the model, except for area (due to basin split) and lag time (due to storm sewer improvements)." In addition, LAN stated: "The only variable in the HMS model that LAN modified from AECOM's PER is the lag time for the Euclid-Wilson area for both existing and proposed conditions'. According to the City's latest FIS model (HMS), the Euclid_Wilson area is essentially located within the subbasin EBLDN070 (see the map shown below). Therefore, the subbasin EBLDN070 should have been split into two subbasins instead of Subbasin EBLDN060. A check of the revised model modified by AECOM/LAN, subbasin EBLDN070 remains unchanged with the same drainage area as the original FIS model while subbasin EBLDN060 is indeed split into two subbasins and both discharge to East Bouldin Creek at the same junction location (JEBLDN060). **The split subbasins and their relative discharge locations in model appear incorrect as compared with the original FIS model as well as the original GIS delineation for the two related subbasins. Please check the models and the subbasin GIS data and explain the discrepancy.**



Estimated Potential Savings: **N/A**

PS/PM Response to #1:

PS/PM Decision on VE's Recommendation:	Agree: ☐	Disagree: ☐	Partially Agree: ☐

VE Comment #2:
The 90% design plan shows that a rather dense inlet layout is proposed. For example, along the Wilson Street, within 908 feet of street length, 12 storm drain inlets are proposed. This appears somewhat excessive when comparing with many other cities' design practices. Depending on local topography, short distance inlet layouts sometimes are necessary but typically not as many as 12 inlets in 908 feet or one inlet for every 1.5 lots. The dense-inlet design may also raise an aesthetic issue for the neighborhood.

The VE team recommends an internal (and/or external) technical discussion/evaluation on the storm-drain design criteria be conducted to find ways of avoiding dense inlet layout in the future.

PS/PM Response to #2:

Estimated Potential Savings: N/A

PS/PM Decision on VE's Recommendation:	Agree: ☐	Disagree: ☐	Partially Agree: ☐

VE Comment #3:

Existing storm drain size on S. Congress Avenue varies from 18" to 24". This storm-water collection line collects storm water from the east side of S. Congress Ave. using 7 curb inlets and then discharges into a downstream drain with a pipe diameter of 36" (See the data for the downstream 3 pipe segments B-P-27, BP-26 and BP-25 in the PER report). The 36" storm drain is apparently too small to drain storm water from the upstream area on the east side of S. Congress Ave. Under a 100-year storm event, a portion of the upstream storm water has to flow across the road to the west side of S. Congress, passing through the Euclid-Wilson area (flowing across yards and streets), which causes the identified local flooding problems. The 90% design plan proposed a 5'x4' box culvert (which is **nearly 3 times** larger in cross-section area than the existing downstream drains) to drain the upstream storm water directly to East Bouldin Creek along the Cumberland Road. As a result, a significant flow increase will likely occur at the downstream boundary of this project due to the storm-drain size upgrade along this Cumberland discharging line. This may be an issue under the City's floodplain management policy and land development code. It appears that no adverse impact analysis was performed along this Cumberland discharge line, (e.g., the peak flow and volume changes between pre-development and post-development conditions at the discharge point to East Bouldin Creek should be analyzed).

Please check the issue and make sure that adverse impact analysis is performed along this Cumberland Road discharging line. Any additional peak flow and volume increase at the discharge point to East Bouldin Creek needs to be properly mitigated.

Estimated Potential Savings: N/A

PS/PM Response to #3:

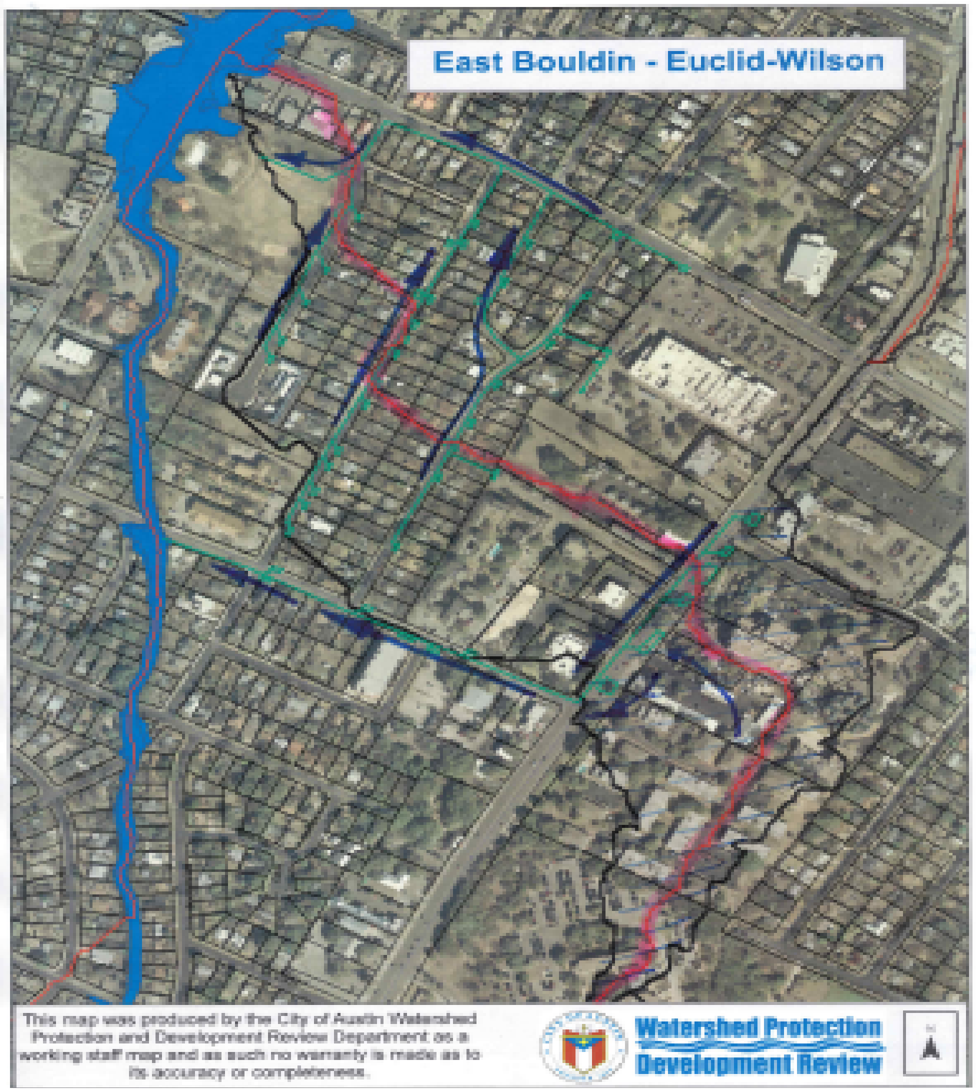
PS/PM Decision on VE's Recommendation:	Agree: ☐	Disagree: ☐	Partially Agree: ☐

VE Comment #4:

Similar to other storm-drain improvement designs, the proposed hydraulic grade lines (HGL) within the project area are below street level under a 100-year storm event. As a result, the longest flow path for Tc estimation between the pre-development and the post-development conditions should be dramatically different. The City's FIS model assumes that under the 100-yr flood, most storm-drains are surcharged and the Tc calculation is based on surface runoff with the longest flow path determined primarily by topography (see the red line shown in the map below). This assumption appears to be reasonable for existing conditions for the 100-yr storm event and is supported by the fact that local storm drains were undersized and local flooding has been severe in this area. However, as a result of the proposed design, the Tc flow path should be changed accordingly and the storm-drain network becomes a major portion of the longest flow path.

However, the report indicates that the post-project Tc is reduced by only 1.06 minutes as compared with the pre-development condition and there is no adverse impact from the project. A brief check over the available Tc calculation data indicates that the Tc estimation by the design engineer for the **pre-development conditions** is not appropriate. The 90% design report states "the time of concentration used for the Euclid-Wilson area (consistent with AECOM's PER approach) was to use the longest travel time of the System A and System B StormCad models. System A is the longest and existing conditions travel time is computed to be 15.87 min and the 90% design proposed conditions travel time is computed at 14.81 min." In fact, part of the storm water runoff from System B as well as from the upstream drainage area on the east side of the S. Congress was used to flow through System A area and can not be separated from System B under predevelopment conditions. By using the floodplain model, the Tc has been estimated to be approximately 25.53 minutes [sheet flow length has been adjusted to be the same as TCB's (150 ft) for comparison purposes] using the original FIS model and flow path while the 90% design report shows the pre-development Tc is about 15.87 minutes.

The VE team recommends that the design team re-check the pre-development Tc calculation to reflect the existing flow conditions and take the flow travel time in the area on the east side of S. Congress Ave. into account before comparing with post-development flows. If adverse impact exists, proper mitigation design is needed to meet the City's "no adverse impact" requirements.



Estimated Potential Savings: N/A

PS/PM Response to #4:

PS/PM Decision on VE's Recommendation:	Agree: ☐	Disagree: ☐	Partially Agree: ☐

Additional General Comment from the VE team:

The longest flow path of a floodplain model (pre-development) and the longest flow path of a storm-drain model (post-development) are generally different as a result of a proposed storm drain improvement project. In most cases, both the existing floodplain model and the post-development model are reasonable and acceptable by national or local standards. However, the post-development Tc would likely be significantly shorter than that of pre-development. Consequently, the peak flow from the project-impacted area would likely increase significantly and adverse impact to downstream area may occur unless proper detention/retention design is made to accommodate the increased flow and volume from the project-impacted area. Similar to other major storm-drain improvement projects such as the Little Shoal Creek and Blarwood projects, these issues demonstrate the need for possible DCM revisions to ensure the project goals, assumptions, and design criteria are clear and consistent between storm-drain design criteria and floodplain mapping and management policies. Although subbasin size, better geometric data and many other model parameters and configuration do affect a model's accuracy, the key factor for a substantial difference between a floodplain model and a storm drain model is believed to be the changes of flow path and conduit size. The general engineering dilemma here is that a storm-drain improvement project aims to eliminate/reduce local flooding for up to a 100-yr storm by adding, enlarging, and/or replacing existing storm drain systems while floodplain management policies aim to maintain or reduce peak flow rate and total flow volume from any new project/development for selected flood frequencies. The difficulty for meeting the requirements from both ends are sometimes rather demanding, especially for urban watersheds where detention sites are likely not available. Thus, collective efforts are needed to discuss, evaluate and resolve the issues in the near future.