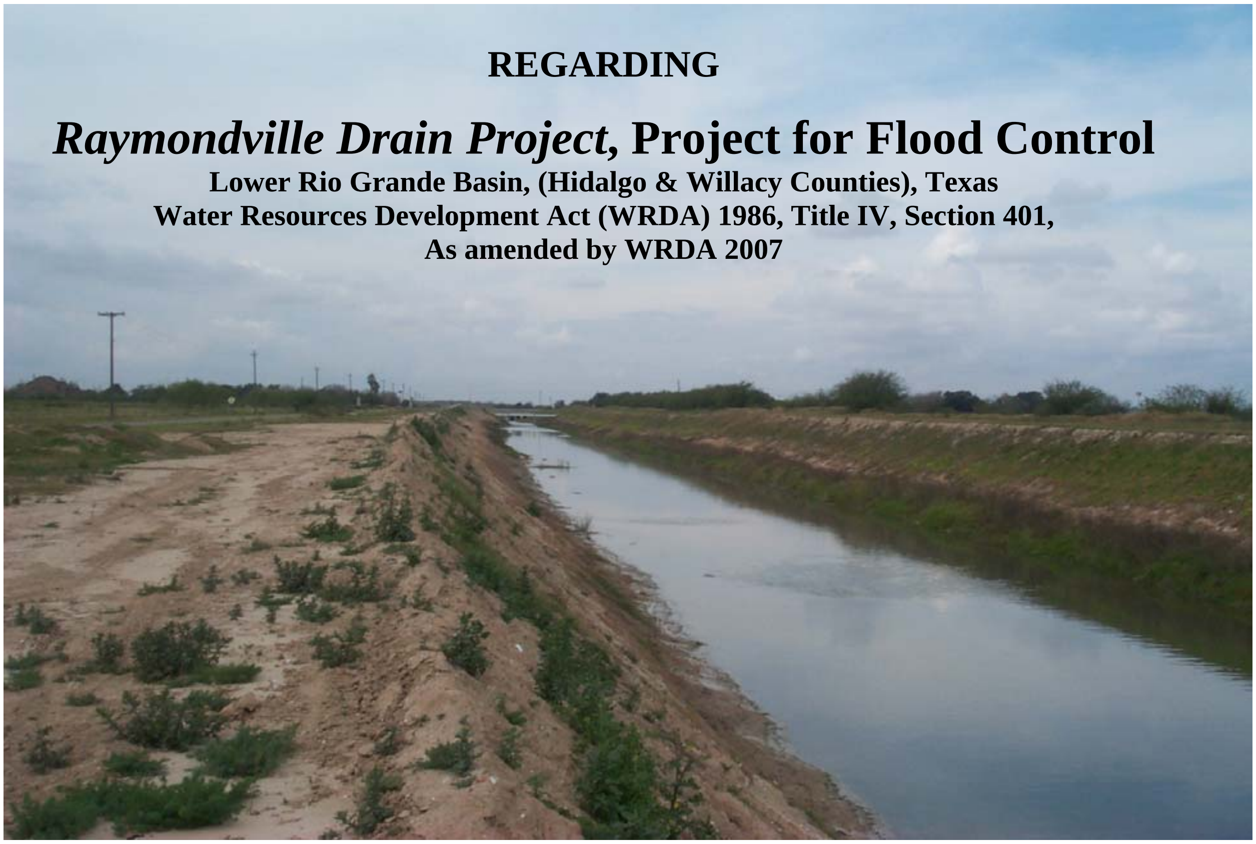
RAYMONDVILLE DRAIN PROJECT
ENGINEERING APPENDIX A1

ATTACHMENT C

H&H QUALITY ASSURANCE REPORT

FINAL TECHNICAL MEMORANDUM
SUMMARY OF QUALITY ASSURANCE REVIEW
HYDROLOGY & HYDRAULIC BASE MODELS



REGARDING

Raymondville Drain Project, Project for Flood Control

Lower Rio Grande Basin, (Hidalgo & Willacy Counties), Texas
Water Resources Development Act (WRDA) 1986, Title IV, Section 401,
As amended by WRDA 2007

Prepared for:

Hidalgo County / Hidalgo County Drainage District No. 1

Under:
Contract No. 2010-164-04-20

THE COUNTY OF HIDALGO TEXAS

HCDD#1

November 30, 2011

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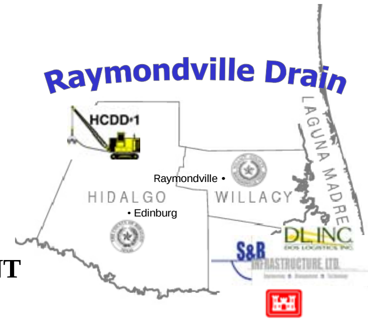

Technical Memorandum

HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT
Design and Expansion of the Raymondville Drain
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SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE HYDROLOGIC / HYDRAULIC (H&H) BASE MODELS

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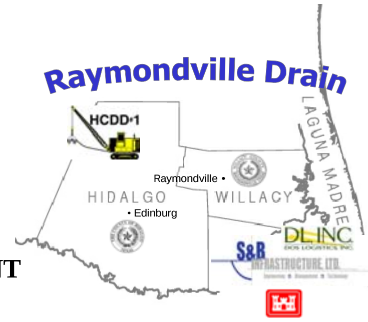

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SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE HYDROLOGIC / HYDRAULIC (H&H) BASE MODELS

Executive Summary

Hydrologic and hydraulic (H&H) models have been prepared for the Raymondville Drain, Project for Flood Control by S&B Infrastructure, Ltd (S&B). This technical memorandum attempts to address the independent assurance review performed by Civil System's Engineering, Inc. (CSE) whose original scope of work was to review S&B's H&H models to ensure S&B's compliance with the US Army Corps of Engineers (USACE) requirements and the methodologies approved for the Project (see **Appendix A** for Project Methodologies).

Throughout the course of the QA Review (September ~ October 2011), approximately four iterations / exchanges occurred between CSE and S&B. **Table 1**, starting on page 2 of the main text of the Technical Memorandum, outlines the review comments by CSE and how they were addressed by S&B. **Appendix B** includes all of the interim technical memorandums generated by CSE and S&B during this review process. Approximately 21 comments were generated and resolved, as all in all, S&B was able to illustrate their methodologies met USACE criteria and the Project's agreed-to methodologies.

However, there was a concern by CSE and HCDD1 regarding the 100-year peak discharge identified in **S&B's 2011 North Main Drain Model (10-day HMS)** and it's relation / comparison to the May 17, 2001 Federal Emergency Management Agency (FEMA) Letter of Map Revision (LOMR) (**FEMA LOMR (May 17, 2001)**). S&B performed further investigations to determine the causes of the differences between the **S&B 2011 North Main Drain Model (10-day HMS)** and the HEC-1 Model utilized by TC&B for the **FEMA LOMR (May 17, 2001)**. This Technical Memorandum serves as a comparison of the modeling methodologies by comparing the peak flow rates at the Seminary Road crossing of the North Main Drain. This location was chosen due to its proximity to the proposed North Main Drain to Raymondville Drain diversion structure.

- In **June 2000, FEMA issued a Flood Insurance Study (FIS)** that affected the Flood Insurance Rate Map (FIRM) community panels near the unincorporated areas adjacent to City of Alton, Edinburg, Elsa-Edcouch-La Villa, and the City of Weslaco, as well as the incorporated panels of the City of Edinburg. The majority of the affected area was along Hidalgo County Drainage District No. 1's (HCDD1) North Main Drain. In general, the 2000 FIRM substantially changed zone conditions, and incorporated a wide regulatory floodway (up to two miles in width, in sections) and floodplain boundaries. Specifically, at Seminary Road, the 100-year peak discharge was identified to be **11,228 cubic feet per second (cfs)**.

- In response, HCDD1 hired S&B, in association with the JE Saenz & Associates, Melden & Hunt, Inc., Sesin Engineering, PLLC, and Quintanilla, Headley & Associates, Inc. to provide a summary report (entitled ***"Summary Report, Evaluation of FEMA FIRM (June 6, 2000), Hidalgo County, Texas"*, dated 11-28-2000**) (S&B 2000 Summary Report) outlining findings of any discrepancies and/or omissions within the 2000 FIS and FIRM. It was noted in the report that the information contained in the report did not include final analysis or design for a map revision or amendment, but should provide a basis for review by FEMA.

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

- Ultimately, information within the report prompted FEMA to advise Hidalgo County to prepare a Letter of Map Revision (LOMR), and in response, Turner, Collie & Braden, Inc. (TC&B), as FEMA's FIRM Contractor, prepared the technical data, including a revised hydrologic and hydraulic analysis, to forward to FEMA, which they did on May 9, 2001. Based on review of the **TC&B** data, FEMA issued a LOMR to reflect the revised hydrologic and hydraulic analyses. The **LOMR (May 17, 2001)** revised areas adjacent to the North Main Drain from Monte Cristo Road to the Donna Drain. Specifically, at Seminary Road, the 100-year peak discharge was revised to be **4,178 cfs**.
- Although the S&B Team prepared a summary report of discrepancies and/or omissions within the 2000 FIS and FIRM prior to TC&B's preparation of a revised H&H analyses for the LOMR, it appears TC&B, based on their TWDB report prepared for HCDD1 entitled ***"Flood Protection Plan for the North Main and Raymondville Drain, December 2001"*** (TC&B December 2001 Study) did not use all of the findings identified in the 2000 Summary Report, including the fact that TC&B appears to have prepared lag time calculations using the Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS), Lag Equation, which was identified in the Melden and Hunt, Inc. critique, included in Appendix A of the S&B 2000 Summary Report, as a method **not** to be used, as basins must be less than 2,000 acres to utilize these types of calculations. The TC&B December 2001 Study identified at Seminary Road, the 100-year peak discharge as **4,178 cfs.**
- **S&B's 2011 North Main Drain Model** indicates at Seminary Road, the 100-year peak discharge should be **5,889 cfs.**
- In summary, it is felt that the 4,178 cfs identified in **LOMR (May 17, 2001) / TC&B Model** is not conducive of today's existing conditions and standards, and that S&B's 2011 North Main Drain Model's 5,889 cfs correctly reflects current watershed conditions; specifically:

  (1) **Age of Data.** The **LOMR (May 17, 2001) / TC&B Model** is 10 years old and used data as old as 1995; S&B is utilizing data more current, and is actually supplementing that data, with on-the-ground field surveys obtained as recently as 2010 and 2011.

  (2) **Modeling Software Programs.** The **LOMR (May 17, 2001) / TC&B Model** utilized the USACE's HEC-1 software program; S&B used a more up-to-date and detailed modeling software (HEC-HMS).

  (3) **Sub-Basin Delineation.** The **LOMR (May 17, 2001) / TC&B Model** identified only **44 sub-basins over 668 square miles** with USGS Quad Maps; the S&B 2011 North Main Drain Model was exceptionally more detailed, and delineated **125 sub-basins over 590 square miles** with 2004~2008 LiDAR and additional site reconnaissance and field surveys.

  (4) **Time of Concentration.** The **LOMR (May 17, 2001) / TC&B Model** utilized the NRCS Lag Equation to calculate time of concentration (even though the S&B 2000 Summary Report included recommendations **not** to utilize this method, and industry standards state that this equation should only be used for sub-basins less than 2,000 acres or at the most 19 square miles) even though 19 of the TC&B sub-basins are larger than 19 square miles. S&B's 2011 North Main Drain Model utilized the velocity method, as required by the US Army Corps of Engineers, and stated by the *"National Engineering Handbook (NEH), Part 630 – Hydrology"* as "the best method for calculating time of concentration for an urbanizing watershed."

  (5) **Slope Determination.** The **LOMR (May 17, 2001) / TC&B Model** utilized an average watershed slope of 0.05% over the entire North Main Drain basin; whereas, S&B's 2011 North Main Drain Model determined the slope for each individual sub-basin utilizing the 2004~2008 LiDAR.

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

(6) **Land Use.** The **LOMR (May 17, 2001) / TC&B Model** appears to have utilized 1995 land use values and 10-foot contours from USGS Quad Maps; the S&B 2011 North Main Drain Model utilizes 2004~2008 LiDAR with 2-foot contours and confirmed / calibrated to actual field surveyed cross-sections. Due to the increased land development over time, S&B's use of the 2007 aerial photography will yield higher peak flow rates throughout the watershed.

(7) **Survey Data (HEC-RAS).** There are also substantial differences between the TC&B 2001 HEC-RAS model and the current S&B HEC-RAS model. Per the TC&B December 2001 Study, the 2001 HEC-RAS model cross section data was based on a 1995 digital terrain model. Moreover, no on-the-ground channel survey was conducted to determine actual flowlines of the channel. There is also no detailed bridge crossing information. TC&B felt that the losses through the bridge structures would be minimal due to the flow velocities within the channel. However, structure overtopping would greatly increase the base flood elevation depending on the height of the structure. The S&B HEC-RAS model utilized 2004~2008 LiDAR data to obtain more accurate cross section data. Additional on-the-ground survey of the channel was conducted to ensure that the minimum channel elevations in the model were accurate. Bridge structures were modeled using information from both field surveys and as-built drawings.

Note: Additional differences are included in the main text of the report, as well as further collaboration and correlation (see Sections 5 and 6).

- **FEMA LOMR.** A LOMR should be requested within 6 months of completion of the proposed improvements (per 44CFR65.3). Even though the 100-year peak discharge identified in the S&B 2011 North Main Drain Model is more than the **LOMR (May 17, 2001) / TC&B Model** (approximately 40%), it does not affect insurance rates at this time. Flood insurance rates are based solely on the current effective Flood Insurance Rate Map (FIRM). A Conditional Letter of Map Revision (CLOMR) may be requested in an effort to obtain any technical comments from FEMA prior to the construction of the proposed improvements, since the proposed project would justify a map revision (44CFR65.8). Note that flood insurance rates are not adjusted based on information provided by a CLOMR.

- **Diversion Analysis.** Incorporating the hydrology from the **S&B 2011 North Main Drain Model**, diversion flows were determined from the North Main Drain to the North Main Drain Diversion Channel. For this analysis, a 40-foot lateral weir was placed downstream of Seminary Road (at User Point 11). It must be noted that the flow in the proposed Diversion Channel ultimately consists of three components: (1) diverted flow from the North Main Drain, (2) runoff that would have flowed to the North Main Drain but is intercepted by the proposed Diversion Channel, and (3) runoff that would have flowed to the existing Raymondville Drain system but is intercepted by the proposed Diversion Channel. The first two components will be utilized for the remainder of this discussion only, as component (3) occurs much further north along the proposed Diversion Channel.

  The **S&B 2011 North Main Drain Model**, during the base conditions, identifies 5,889 cfs during the 100-year, 24-hour storm event in the North Main Drain at point of the future diversion. Utilizing the 40-foot lateral weir, 1,751 cfs is diverted by the weir to the proposed Diversion Channel. An additional 2,286 cfs that would have flowed to the North Main Drain is intercepted by the proposed Diversions Channel. There is 1,852 cfs of remnant flow to the North Main Drain downstream of the proposed Diversion Channel. These peak flow rates may be revised upon further coordination with the USACE.

  Figure 1 on page 17 of the Technical Memorandum provides a schematic overlaid on an aerial to illustrate the diverted flows. Ultimately, it is estimated that the proposed Diversion Channel could reduce the flow to the North Main Drain by approximately 69%.

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

- **Summary of Models and Changes.** Prior to this latest quality assurance review, S&B worked closely with the USACE to prepare the hydrologic and hydraulic criteria to be utilized for the flood damage assessment. At the request of the client, S&B attempted to correlate the hydrology from the 2001 FEMA LOMR to the current hydrology used by S&B. As such, S&B prepared a separate HMS model for correlation purposes only. Any revisions during this phase would be incorporated into the final HMS model for use in the FDA, as long as any revisions complied with the previously agreed upon USACE criteria. The table below summarizes the hydrologic models utilized for this correlation effort and for use in the preparation of the FDA to be submitted to the USACE.

Methodology	**2001 HEC-1 FEMA LOMR**	**2011 S&B HEC-HMS for Correlation to FEMA**	**2011 S&B HEC-HMS for FDA**
Computation Software	HEC-1	HEC-HMS	HEC-HMS
# of Sub-Basins	44	125	125
Loss Method	NRCS Curve Number	Initial/Constant Loss	Initial/Constant Loss
Land Use	1995 Development	2007 Development	2007 Development
Lag Time	NRCS Lag Equation	Velocity Method	Velocity Method
Storm Duration	24-Hour	24-Hour	10-Day
Precipitation Data	USGS 98-4044	USGS Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas, 2004	TP-40/TP-49 with Depth/Area Reduction

In conclusion, this Technical Memorandum provides a summary of the QA review performed by CSE of the Hydrologic and Hydraulic base models for the Raymondville Drain Project and the *"Preliminary Engineering Report – Alternatives Analysis thru Hydrologic & Hydraulic Analysis For the Proposed Typical Sections, From Edinburg Lake to the Guerra Detention Facility"*, dated 5-25-2011 (Supplemented 6-9-2011)". All in all, the S&B 2011 models and data reflect USACE criteria and project methodology. Additionally, the increase from the peak flow rates found in the **LOMR (May 17, 2001) / TC&B Model** are justified by the age in data and up-to-date methodologies, and once the project is constructed, the FIRM can be updated through a future LOMR.

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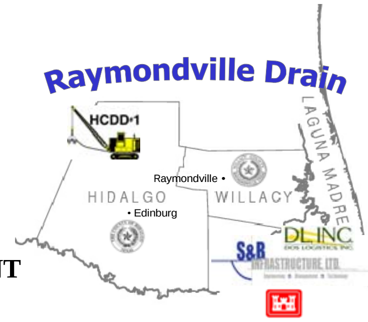

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SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE HYDROLOGIC / HYDRAULIC (H&H) BASE MODELS

1. PURPOSE OF TECHNICAL MEMORANDUM

This technical memorandum is an overall summary of the quality assurance review performed by Deren Li, PE of Civil Systems Engineering, Inc. (CSE) and S&B Infrastructure, Ltd.'s (S&B) responses to the comments received regarding the hydrologic and hydraulic base models for the Raymondville Drain Project and the "Preliminary Engineering Report – Alternatives Analysis thru Hydrologic & Hydraulic Analysis For the Proposed Typical Sections, From Edinburg Lake to the Guerra Detention Facility", dated 5-25-2011 (Supplemented 6-9-2011)".

At the request of Hidalgo County Drainage District No. 1 (HCDD1), CSE was to perform an independent review, and the review should reflect complete professional independence regarding US Army Corps of Engineers (USACE) project criteria, including peak flow rate computations based on standard design storms required by the USACE and the methodologies approved by the USACE for the Raymondville Drain Project. This project criteria was provided to CSE and is included in **Appendix A** of this technical memorandum.

2. BACKGROUND

The development of the Raymondville Drain Project must be in accordance with USACE criteria. Part of USACE criteria includes extensive planning documentation, particularly a General Re-evaluation Report (GRR). A significant part of the GRR is the preparation of a Flood Damage Assessment (FDA) for the North Main Drain & Raymondville Drain to justify drainage improvements needed to alleviate existing flooding along the North Main Drain. This analysis would be presented to the USACE for approval and to obtain funding for the project. The FDA is critical in the determination of a benefit-cost ratio, which must be greater than 1.0 in order for the project to be approved for final design and construction and to receive funding from the USACE. As a result, S&B coordinated with the USACE concerning modeling methodology to be used for the H&H analysis. S&B worked closely with staff from the USACE over a span of several years to jointly prepare the methodology to be utilized in this analysis. This methodology is summarized in the Raymondville Drain Pre-Project Condition Report prepared by the USACE in 2006 and in the S&B Hydrology & Hydraulics (Without Project Conditions) Report dated October 2007.

3. CSE QUALITY ASSURANCE (QA) REVIEW AND S&B RESPONSE

Throughout the course of the QA Review (September ~ October 2011), approximately four iterations / exchanges occurred between CSE and S&B. **Table 1**, starting on the next page, outlines the review comments by CSE and how they were addressed by S&B. **Appendix B** includes all of the interim technical memorandums generated by CSE and S&B during this review process. All in all, S&B was able to illustrate their methodologies met USACE criteria and the Project's agreed-to methodologies.

However, there was a concern by CSE and HCDD1 regarding the 100-year peak discharge identified in **S&B's 2011 North Main Drain Model (10-day HMS)** and it's relation / comparison to the May 17, 2001 Federal Emergency Management Agency (FEMA) Letter of Map Revision (LOMR) (**FEMA LOMR (May 17, 2001)**). A copy of **FEMA LOMR (May 17, 2001)** is included in **Appendix C**. At the request of HCDD1, S&B

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

performed further investigations to determine the differences between the **S&B 2011 North Main Drain Model (10-day HMS)** and the HEC-1 Model utilized by TC&B for the **FEMA LOMR (May 17, 2001)**.

Table 1: Summary of Review Comments / Responses			
No.	**CSE Comment:**	**S&B Response:**	**Was USACE Criteria Met?**
September 7 ~ September 14, 2011 - HEC-HMS Hydrologic Modeling			
1	1. Recommend the use of USGS/TxDOT Atlas in lieu of the TP-40/TP-49 rainfall data.	USGS / TxDOT Atlas was discussed with the USACE, but the USACE felt the area contained "questionable depth-duration frequency values" with substantial inconsistencies; TP-40/TP-49 was required by USACE and incorporated into the project methodology.	**Yes**
2	2. There are no HEC-HMS models developed to compute future (2061) conditions peak flows and hydrographs.	No separate hydrology model is required for the development of the projected peak flow rates; growth factors from TWDB were utilized, and a factor of 1.35 was selected to calculate the peak flow rates for the projected year (2061).	**Yes**
3	3. The Modified Pulse Method was used for flood routing along various channel reaches and that for all channel reaches that uses Modified Pulse Method, Subreach is assumed "1". This probably overestimates the storage effect for some of the reaches.	A subreach value of "1" is valid; the HEC-HMS Technical Reference Manual states that this value "is used commonly for routing through ponds, lakes, wide, flat floodplains, and channels in which the flow is heavily controlled by downstream conditions."	**Yes**
4	4. Some of the storage-outflow relations from HEC-RAS modeling appear to be overestimated.	Storage-outflow tables were taken directly from HEC-RAS, and much of the data was obtained directly from the models provided to S&B by the USACE; significant coordination with the USACE during 2007 occurred regarding storage values; the extremely flat terrain found in these areas, there will be significant storage in the overbanks once the water surface has risen above the banks of the channel.	**Yes**
5	5. Percent Imperviousness parameter is not explicitly modeled in the HEC-HMS model. It is not clear whether it was considered in the CN and time of concentration calculations.	The percent imperviousness parameter was not utilized in the hydrologic model. Instead, the composite CN based on land use and hydrologic soil group was utilized. The impervious cover is included in the final CN values, thus no additional impervious cover percentages should be added separately to the HEC-HMS model.	**Yes**
6	6. The NRCS standard initial loss method of 0.2S (potential maximum retention) is used in the HEC-HMS model. Since the 10-day storm event is assumed for the study, initial loss has very minor impact to the peak flows. The average initial loss used in the model is approximately 2 inches. Even increase to 5 inches, there is very minor changes in peak flows.	The Initial/Constant Loss method is appropriate for long duration storm events; the NRCS CN Method assumes that after the initial loss, all losses go to zero. As a result, the NRCS CN Method should not be used for storms with significant duration. Additionally, composite CN were utilized to determine the initial loss value; CN were adjusted to AMC I (dry condition) prior to calculating the initial loss. This resulted in higher initial loss values.	**Yes**

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

Table 1: Summary of Review Comments / Responses

No.	CSE Comment:	S&B Response:	Was USACE Criteria Met?
7	7. Overall peak flows appear significantly high. At Station 64591, the 100-year peak flow is 12,494 cfs in this study, which is 3 times of the FEMA effective 100-year peak flow of 4,175 cfs. With consideration of the difference of the 24-hr and 10-day storm event, there is a maximum of 10 percent increase. It appears that the rainfall depth makes the most of the differences.	The NRCS CN were adjusted to AMC I. The adjusted CN were kept below 60 to account for depressions and storage found throughout the watershed. The unit hydrographs were flattened to account for the flat terrain found in the watershed. The velocity method was used for time of concentration calculations. This method is more detailed since average velocities for each subbasin were calculated, as opposed to using a general approximation.	**Yes**
8	8. A constant Peaking Rate Factor of 150 is used in calculating the Unit Hydrograph. It seems variable PRF should be used with the consideration of the subbasin physical conditions such as slopes and depressions.	Due to the generally flat topography of these watersheds as compared to the average U.S. watershed, the PRF was adjusted from 484 to 150; 150 was chosen to properly model the slopes and depressions found within these watersheds.	**Yes**
9	9. Detailed documentation is needed to clearly discuss the relationship between the area reduction calculation using spreadsheets and HEC-HMS modeling results.	Due to inherent limitations with the HEC-HMS program, manual adjustments were needed for each storm event to provide valid results at each junction node. For each storm event, multiple runs were created for storm area values from 0 square miles to 400 square miles in 50 square mile intervals. The peak flow rate for each simulation was recorded. Subsequently, the peak flow rate was calculated based on the actual watershed area at each node. It was this peak flow rate that was subsequently input into the HEC-RAS hydraulic model. This methodology was presented to, and agreed with by the USACE. Further documentation will be provided in the hydraulics section of the flood damage assessment report.	**Yes**
September 7 ~ September 14, 2011 - HEC-RAS Hydraulic Modeling			
10	1. In the base HEC-RAS model, at Station 64591, the 100-year peak flow is 12,494 cfs. In the Alt 1B model, the 100-year peak flow is 9,089 cfs. Where the 9,089 came from?	The peak flow rates for the Alt 1B HEC-RAS models were developed using separate HEC-HMS models with manual calculation of the storm area reduction. These models were provided with the base HEC-HMS models.	**Yes**
11	2. As discussed earlier, there is no future conditions HEC-HMS models developed for the project. It is understood there is factor of 1.35 used to obtain future conditions peak flows. What is the justification of 1.35?	As stated earlier, no separate hydrology model is required for the development of the projected peak flow rates; growth factors from TWDB were utilized, and a factor of 1.35 was selected to calculate the peak flow rates for the projected year (2061).	**Yes**
12	3. Why only 645 cfs is used in the RVD HYD Model Alt 1B for the 100-year (2061)? The Preliminary Engineering Report states a 100-year 1,390 cfs flow	The RVD HYD Model Alt 1B is based on the 10-day storm event as required by USACE for development of the Flood Damage Assessment (FDA) needed to obtain federal funding. The Preliminary Engineering	**Yes**

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

Table 1: Summary of Review Comments / Responses

No.	CSE Comment:	S&B Response:	Was USACE Criteria Met?
	is proposed diverted. Alternative diversion flows should be considered to optimize the design of the diversion weir structure, channel, and detention basins.	Report incorporated a 24-hr storm duration calibrated to the peak flows found in the FEMA LOMR of 5-17-2001. The difference is due to the amount of runoff that is intercepted by the diversion channel. Once the FDA is finalized, and the design storm event is selected, the design of the diversion weir structure, channel and detention basin can be further optimized.	
13	4. The combined peak flow (diversion/interception) does not reflect the 1.35 factor.	S&B is assuming that this comment is referring to the actual diversion weir. For the purposes of our initial analysis, the amount of flow diverted was kept constant. As a result the combined diversion / interception peak flow rate for the Year 2011 and Year 2061 will not directly correspond to the 1.35 factor.	**Yes**
14	5. With consideration of the very flat nature of the drainage channels, the HEC-RAS Unsteady Flow Module is recommended for this study. The hydraulic routing technique within the HEC-RAS Unsteady Flow Module is based on the solution of the partial differential equations (dynamic wave equations) of unsteady channel flow. The hydraulic routing method provides the most accurate solutions calculating an outflow hydrograph while considering the effects of channel storage and wave shape. The Modifed Puls hydrologic routing method does not work properly when the channel slope is very flat (< 3 ft/mile). The storage-discharge relations calculated using steady flow profiles produce errors when out-of-bank flows occur over wide floodplains.	Modified Puls was used; this methodology is specifically useful in areas with wide floodplains and where there is significant backwater that will influence the discharge hydrograph. Additionally, this method is valid from slopes ranging from 10 to 2 ft/mile. (Chapter 9, EM1110-2-1417). All storage routing and flow attenuation was calculated using the methods found in HEC-HMS. The HEC-RAS steady state model was utilized to prepare the storage-outflow curves that were used in HEC-HMS. This methodology produced individual hydrographs that were attenuated through their corresponding reaches. It was these attenuated peak flows that were input into the HEC-RAS steady flow model.	**Yes**
15	6. US 281 crossing structure seems oversized.	The US 281 culvert structure was sized for use in determining alternatives for the FDA. The preliminary design was selected to provide a headloss through the structure that closely mimics the proposed bridge solution. Once the design storm event has been selected, the detailed design will be performed to provide an efficient solution that complies with TxDOT design requirements and meets the design constraints set forth by the HCCD1.	**N/A**
16	7. US 281 crossing structure seems oversized.	Once the flood damage assessment is finalized, and the design storm selected, the final design of the diversion channel will be optimized. This preliminary channel geometry is being utilized to determine and	**N/A**

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

Table 1: Summary of Review Comments / Responses

No.	CSE Comment:	S&B Response:	Was USACE Criteria Met?
		evaluate alternatives during this feasibility phase for the entire Raymondville Drain and North Main Drain watersheds.	
17	8. Several reaches show 10+ feet of freeboard.	See response to Comment 7 above.	**N/A**
September 19 ~ September 20, 2011 - Follow-up: HEC-HMS Hydrologic Modeling			
Note:	S&B received an e-mail from CSE on September 19, stating the two most critical comments above were under the **HEC-HMS Hydrologic Modeling Comments No. 2 and 7** regarding the peak flows for the project:		
18	Regarding Comment 2 - A future conditions HEC-HMS is needed for the project with consideration of the sizes of the studied watersheds. The application of an averaged factor of 1.35 will not be able to properly reflect the variations of developments within the entire watersheds and therefore will not be able to properly simulate the hydrologic responses of the watersheds to the projected developments.	The average factor of 1.35 was correctly utilized to account for the increase in peak runoff due to economic growth factors obtained from the TWDB and projected population as determined by the Texas State Data Center, the Office of the State Demographer and Census 2000 data. Concerning “variation of developments”, this is not applicable due to USACE direction to S&B. According to USACE guidelines sent to S&B, “The existing land use pattern will be assumed to continue in the same proportion throughout the watershed”. Since the majority of the watershed is not zoned for future development, it is not feasible to attempt to prepare detailed development estimates over these watersheds. Any attempt will be solely based on the preparer’s opinion. As such, S&B did make preliminary estimates based on population forecasts for a selective number of sub-basins. A hydrologic analysis was performed to determine the average increase in runoff. The increase varied in range from 28% to 32%. Based on the analysis, a factor of 35% was selected as an estimate on the increases in future peak flow runoff.	**Yes**
19	Regarding Comment 7 - Based on flows in the HEC-RAS model for North Main Drain, at Seminary Road (SX 65691), the 100-year peak discharges are 12,501 cfs (existing conditions) and 16,976 cfs (1.35x12,501). The estimated existing 100-peak flow is 3 times the FEMA effective 100-year peak flow of 4,175 cfs, and 4 times of the estimated peak flow of 3,077 cfs by Melden and Hunt, Inc. (Critique of the Flood Insurance Study, 2000). Since the differences in rainfall data between 10-day and 24-storm events, as	During the base conditions HEC-HMS development of the Raymondville Drain and North Main Drain watersheds, a discussion was conducted with the USACE concerning the methodology for determining the lag time of the individual sub-basins. In S&B’s original analysis, the SCS CN lag time equation was utilized. However, the USACE felt that an “accurate SCS lag was needed because the unit hydrograph was already flattened by the adjustment to the peaking factor. If an excessively long lag was used in combination with a reduced peaking factor, then an unrealistically low peak flow rate would likely occur for each sub-basin.” The USACE felt that by using the velocity method to calculate the lag time, the shorter times of	**Yes**

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

Table 1: Summary of Review Comments / Responses

No.	CSE Comment:	S&B Response:	Was USACE Criteria Met?
	well as between TP40/TP49 and USGS rainfall data, don't make a 3 to 4 times differences in peak flow discharges, I have further investigated the methods of estimating Tc or LAG. By comparing the travel time method with the SCS LAG equation $L^{0.8} [(1000/CN)-9]^{0.7} / (1900 \times S^{0.5})$, there is a significant difference in time of concentration for most of the subbasins. With the SCS LAG equation, the unit hydrograph (UH) could be more than doubled for some subbasins. Please email me a copy of the USACE' comments in dealing with the travel time method.	concentration would yield "more reasonable HMS results". Additionally, the USACE assumed an open channel flow velocity of 0.6 fps for their analysis. In order to refine this further, S&B utilized LIDAR data to prepare actual typical sections to calculate the open channel flow velocity for each sub-basin. It was because of this extra step that the USACE chose to utilize S&B values. This information included in the USACE Raymondville Pre-Project Report dated 04-11-2006, which was provided to CSE at the meeting in S&B's office on August 23. We will email you another copy. (A copy of the USACE Raymondville Pre-Project Report dated 4-11-2006 is provided in **Appendix A** of this report.)	
September 22 ~ September 23, 2011 - CSE Final Tech Memo			
Note:	**CSE provided a Final Technical Memorandum (FTM) to S&B on September 22, 2011; within the FTM there was a need to follow-up with a response to the following comment:**		
20	A comparison table <in Attachment E of CSE's Final Technical Memorandum> is presented to further demonstrate our concerns. First, comparison is made between the Lag Time values computed by S&B and the SCS Equation (North Main Drain subbasins were used for this comparison). As shown in column LAG(SCS) / LAG (S&B), the LAG values based on SCS lag equation are 1 to 8 times of the LAG (S&B) values. The ratios are reflected in the Unit Hydrograph peak flows (for PRF 150). Also comparison is made for Unit Hydrograph Peak flows between Qp based on PRF 150 and S&B's lag values and Qp based on standard PRF 484 and SCS lag equation. Column Qp(S&B)/Qp(SCS484) shows that even with the much lower PRF 150 for the project, for most of the subbasins, the computed peak flows are much greater than peak flows based on the standard PRF 484 (3.2 times of 150). It should be noted that the above comparison results do not disqualify the	As stated previously, the basis for the individual sub-basin times of concentration were based on the Velocity Method utilizing LIDAR data to determine individual channel typical sections and velocities. The USACE recognized that this level of detail was superior to the previous assumed velocities that were used by the USACE in their analysis. As shown by CSE, the substantial differences between the times of concentration calculated by this method versus the SCS lag equation simply reinforces that the SCS lag equation over-simplifies this crucial calculation, when utilized on such a large, varied watershed. Based on the data received from CSE using the SCS lag equation, many of the sub-basins do not achieve a peak unit discharge of 10 cfs/sq.mile, which is very low amount of runoff for single square mile of area. Attached is Exhibit "A" (note: see **Appendix B** of this Technical Memorandum) which calculates the unit discharges for each sub-basin based on S&B's methodology and using the SCS lag equation.	**Yes**

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

Table 1: Summary of Review Comments / Responses			
No.	**CSE Comment:**	**S&B Response:**	**Was USACE Criteria Met?**
	Tc or travel time METHOD used for this study. However, estimates of parameters need to be revised to ensure that lag time values and peak flows are more representative of the local watershed conditions		
October 31, 2011 ~ November 11, 2011 **Final CSE Comment**			
Note:	Recent discussions with CSE (November 11, 2011) included the following comment / response:		
21	CSE questioned no depth-area reductions for the precipitation values were used for S&B 100-year, 24-hour storm event, and requested any documentation regarding this methodology.	S&B used the USGS rainfall data, which does not provide for depth-area reductions. Only rainfall data from TP-40 and TP-49 provided for the use of the depth-area reduction. This information included in the USACE Raymondville Pre-Project Report dated 04-11-2006, which was provided to CSE at the meeting in S&B's office on August 23.	**Yes**

4. HISTORY OF DEVELOPMENT OF THE 100-YEAR PEAK DISCHARGE AT SEMINARY ROAD

The following is an outline of events regarding the development of a 100-year peak discharge at Seminary Road:

- In **June 2000, FEMA issued a Flood Insurance Study (FIS)** that affected the Flood Insurance Rate Map (FIRM) community panels near the unincorporated areas adjacent to City of Alton, Edinburg, Elsa-Edcouch-La Villa, and the City of Weslaco, as well as the incorporated panels of the City of Edinburg. The majority of the affected area was along Hidalgo County Drainage District No. 1's (HCDD1) North Main Drain. In general, the 2000 FIRM substantially changed zone conditions, and incorporated a wide regulatory floodway (up to two miles in width, in sections) and floodplain boundaries. Specifically, at Seminary Road, the 100-year peak discharge was identified to be **11,228 cubic feet per second (cfs)**.

- In response, HCDD1 hired S&B, in association with the JE Saenz & Associates, Melden & Hunt, Inc., Sesin Engineering, PLLC, and Quintanilla, Headley & Associates, Inc. to provide a summary report (entitled ***"Summary Report, Evaluation of FEMA FIRM (June 6, 2000), Hidalgo County, Texas", dated 11-28-2000***) (S&B 2000 Summary Report) outlining findings of any discrepancies and/or omissions within the 2000 FIS and FIRM. It was noted in the report that the information contained in the report did not include final analysis or design for a map revision or amendment, but should provide a basis for review by FEMA.

- Ultimately, information within the report prompted FEMA to advise Hidalgo County to prepare a Letter of Map Revision (LOMR), and in response, Turner, Collie & Braden, Inc. (TC&B), as FEMA's FIRM Contractor, prepared the technical data, including a revised hydrologic and hydraulic analysis, to forward to FEMA, which they did on May 9, 2001. Based on review of the **TC&B** data, FEMA issued a LOMR to reflect the revised hydrologic and hydraulic analyses. The **LOMR (May 17, 2001)** revised areas adjacent to the North Main Drain from Monte Cristo Road to the Donna Drain. Specifically, at Seminary Road, the 100-year peak discharge was revised to be **4,178 cfs**.

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

- Although the S&B Team prepared a summary report of discrepancies and/or omissions within the 2000 FIS and FIRM prior to TC&B's preparation of a revised H&H analyses for the LOMR, it appears TC&B, based on their TWDB report prepared for HCDD1 entitled ***"Flood Protection Plan for the North Main and Raymondville Drain, December 2001"*** (TC&B December 2001 Report) did not use all of the findings identified in the 2000 Summary Report, including the fact that TC&B appears to have prepared lag time calculations using the Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS), Lag Equation, which was identified in the Melden and Hunt, Inc. critique, included in **Appendix A** of the S&B 2000 Summary Report, as a method **not** to be used, as basins must be less than 2,000 acres to utilize these types of calculations. The TC&B December 2001 Report identified at Seminary Road, the 100-year peak discharge as **4,178 cfs.**
- **S&B's 2011 North Main Drain Model** indicates at Seminary Road, the 100-year peak discharge is **5,889 cfs.**

5. CORRELATION OF S&B'S 2011 NORTH MAIN DRAIN MODEL (10-DAY HMS) AND FEMA

The primary issue has been the magnitude of the peak flow rates within the North Main Drain (specifically at Seminary Road), as calculated by **S&B's 2011 North Main Drain Model (10-day HMS)**, when compared to the **FEMA LOMR (May 17, 2001) Model (24-hr HEC-1)**. In developing this correlation, the general methodology and input variables still needed to comply with the guidelines set forth by the USACE.

The issues that the USACE had with the FEMA Flood Insurance Study (FIS) models were two-fold. First, according to the phone conversation with Eric Sheibe (USACE), the USACE felt that the FEMA FIS models underestimated the peak flow rates since the results did not appear to duplicate what local experience was claiming. Second, the USACE did not feel that the 24-hour storm duration and the US Geological Survey (USGS) Rainfall Data (Report 98-4044) were justifiable, based on the type of flooding that local experience was suggesting. This is confirmed by the statement in a report by Turner, Collie & Braden, Inc. (TC&B), who also provided the data to FEMA for the **LOMR (May 17, 2001) Model (24-hr HEC-1),** entitled ***"Flood Protection Plan for the North Main and Raymondville Drain, December 2001"*** (TC&B December 2001 Report), which states, *"Due to a lack of stream gage stations located within the project study reaches, the calibration processes did not include a comparison of model computed peak discharges with measure flow values"*.(TC&B December 2001 Report, Section III). The HEC-RAS model used also did not include any detailed bridge crossing information. Although TC&B felt that the losses through bridges would be minor based on the flow velocities, any bridge overtopping would greatly affect the water surface elevations, and thusly, the floodplain elevations. The TC&B December 2001 Report states that *"As part of the LOMR submittal, project HEC-1 and HEC-RAS models would need to be modified to reflect more detailed topographic and bridge crossing information"*(TC&B December 2001 Report, Section I).

The following is an outline of the differences in the 100-year peak discharges at Seminary Road between the **S&B 2011 North Main Drain Model (10-day HMS)** and the S **FEMA LOMR (May 17, 2001) Model (24-hr HEC-1)**:

a. **Age of Data.** The **LOMR (May 17, 2001) / TC&B Model** is 10 years old and used data as old as 1995; S&B is utilizing data more current, and is actually supplementing that data, with on-the-ground field surveys obtained as recently as 2010 and 2011.

b. **Modeling Software Programs.** The **LOMR (May 17, 2001) / TC&B Model** utilized the USACE's HEC-1 software program; S&B used a more up-to-date and detailed modeling software (HEC-HMS).

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c. **Sub-basin Delineation.** A substantial difference between the **S&B 2011 North Main Drain Model (10-day HMS)** and the **LOMR (May 17, 2001) / TC&B Model** was the number of sub-basins and the methodology used for their delineation. The **LOMR (May 17, 2001) / TC&B Model** contained only **44 sub-basins** for a hydrologic model that encompassed over 668 square miles. These sub-basins were delineated using USGS Quadrangle maps. For the **LOMR (May 17, 2001) / TC&B Model,** S&B utilized HEC-GeoHMS to delineate the sub-basins. The background data for this analysis was 2004~2008 LIDAR data obtained from Hidalgo County, additional site reconnaissance and field surveys, and USGS mapping. S&B delineated **125 sub-basins** for the North Main Drain hydrologic model. Using this more recent and detailed information, the total drainage area for the North Main Drain system is 590 square miles.

d. **Precipitation Data.** A primary difference between the two hydrologic models was the storm duration utilized for the analysis. The **LOMR (May 17, 2001) / TC&B Model** utilized a 24-hour storm event and precipitation values from USGS. Because of the USACE criteria n the development of FDA, S&B prepared a hydrologic model using a 10-day storm duration and rainfall data from TP-40/TP-49 with depth-area reductions. This was done in coordination with the USACE during the initial H&H modeling effort. In 2004, during phone conversation with Eric Scheibe (USACE), USACE discussed the requirement to utilize the TP-40/TP-49 rainfall data. The USACE believed that there was not enough data on the depth-area curves for this method. The only data the USACE could find was for a 24-hr duration storm, and this depth-area curve was only valid for Dallas, Austin, and Houston. Therefore, **the USACE felt that a more appropriate method would be to use the TP-40/TP-49 rainfall depths and the corresponding depth-area curves**. <u>However, in an effort to correlate the S&B 2011 North Main Drain Model with the LOMR (May 17, 2001) / TC&B Model, S&B prepared a meteorological model that utilized USGS rainfall data. Additionally, the storm duration was reduced from 10-days to 24-hours. This model is only for correlation purposes and cannot be used for analysis in the FDA</u>.

e. **Time of Concentration and NRCS Lag Equation.** The **LOMR (May 17, 2001) / TC&B Model** utilized the NRCS Lag Equation to calculate time of concentration (even though the S&B 2000 Summary Report included recommendations <u>**not**</u> to utilize this method, and industry standards state that this equation should only be used for sub-basins less than 2,000 acres or at the most 19 square miles) even though 19 of the TC&B sub-basins are larger than 19 square miles. S&B's 2011 North Main Drain Model utilized the velocity method, as required by the US Army Corps of Engineers, and stated by the "*National Engineering Handbook (NEH), Part 630 – Hydrology*" as "the best method for calculating time of concentration for an urbanizing watershed." This methodology for time of concentration calculations differs significantly from what was used in the **LOMR (May 17, 2001) / TC&B Model**. Additionally, S&B utilized the best available LIDAR data to extract the slope and topographic data used to prepare these calculations for each sub-basin within the watershed. In contrast, the **LOMR (May 17, 2001) / TC&B Model** used the NRCS Lag Equation to determine the lag times for each sub-basin. **The NRCS Lag Equation was developed using data from only 24 watersheds ranging from 1.3 acres to 9.2 square miles with a majority of watersheds less than 2,000 acres.** A re-study concluded that a reasonable limit "may be" 19 square miles ((NEH), Part 630 – Hydrology, Chapter 15). However, a review of the sub-basins within the **LOMR (May 17, 2001) / TC&B Model** (HEC-1) found that all but one sub-basin was larger than 2,000 acres. Additionally, there are 19 sub-basins that are larger than 19 square miles, the "may be" upper limit of application for this equation. There is also one sub-basin totaling 94.54 square miles, which far exceeds the applicability of the NRCS Lag Equation.

SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

f. **Slope Determination.** The **LOMR (May 17, 2001) / TC&B Model** utilized an average watershed slope of 0.05% over the entire North Main Drain basin; whereas, the **S&B 2011 North Main Drain Model** determined the slope for each individual sub-basin utilizing the 2004~2008 LiDAR.

g. **Land Use.** To determine the precipitation losses within each sub-basin, it is necessary to determine the hydrologic soil types and land uses within each sub-basin. The hydrologic soil groups for the **LOMR (May 17, 2001) / TC&B Model** were obtained from a Bureau of Reclamation Study dated November 1956. For S&B's hydrologic model, the individual soil types were delineated and measured using the US Department of Agriculture (USDA) Soil Surveys for Hidalgo and Willacy County and utilized USDA mapping software. The land use data utilized in the **LOMR (May 17, 2001) / TC&B Model** was prepared using USGS digital orthophoto quadrangles (DOQQ's) that were dated February 1995. In contrast, S&B used available aerial photography in 2007 to determine the existing land uses within the watershed. Thus, **since the LOMR (May 17, 2001) / TC&B Model was based on the land use in February, 1995, S&B's model will yield higher flow rates throughout the watershed.**

h. **Loss Methodology.** The loss method utilized in the **LOMR (May 17, 2001) / TC&B Model** was the NRCS Curve Number (CN) method. However, in addition to the losses based on the standard CN, the HEC-1 model also included elevated initial losses for the individual sub-basins. According the TC&B December 2001 Report, initial losses of 2-inches was applied unilaterally over areas with elevated canals and 3-inches was applied over all cropland and pasture land. This was also in addition to the storage values included in the model to account for storage behind the elevated canals. Additional storage was also added by delineating areas within the Zone A floodplains as specified by the 1997 FIS and assuming a depth of 2 feet. This had the effect of substantially reducing excess runoff. For the **S&B 2011 North Main Drain Model** (10-day HMS), the initial/constant loss method was utilized to determine losses throughout the watershed. The selection of this method was coordinated with the USACE, since the NRCS CN method would not properly account for precipitation losses during a 10-day storm event. For S&B's model, the standards CN were reduced to AMC I to account for the typically dry conditions within the watershed. When these CN are reduced, the minimum value used in hydrologic modeling is typically 60, according to Soil Conservation Service (SCS) Engineering Technical Note 210-18-TX5. However, since S&B understood that there are numerous minor depressions and storage areas, no minimum value was set, and the actual calculated CN were used, and in many cases the values were far below 60. Once the CN were calculated, the initial loss was calculated using standard NRCS methodology, as recommended by the USACE where;

$$I = 0.2S$$

$$S = \frac{1000\text{-}CN}{CN}$$

and where:

I = Initial loss (in)
S = Potential Maximum Retention
CN = Curve Number

The constant loss was calculated using the most recent available soil surveys from the USDA to determine the percentage of each type of soil group for each sub-basin. Table 2 below shows the SCS soil groups and the infiltration (loss) rates.

Table 2: SCS Soil Groups / Infiltration (Loss) Rates		
Soil Group	**Description**	**Range of Loss Rates (in/hr)**
A	Deep sand, deep loess, aggregated silts	0.30-0.45
B	Shallow loess, sandy loam	0.15-0.30
C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05-0.15
D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0.00-0.05

For **S&B 2011 North Main Drain Model** the highest allowable loss rate for each soil group was utilized. These values were presented in the S&B *Hydrology & Hydraulics (Without Project Conditions) Report* dated October 2007.

i. Hydraulics (HEC-RAS). There are also substantial differences between the TC&B 2001 HEC-RAS model and the current S&B HEC-RAS model. Per the TC&B December 2001 Report, the 2001 HEC-RAS model cross section data was based on a 1995 digital terrain model (DTM). Moreover, no on-the-ground channel survey was conducted to determine actual flowlines of the channel. There is also no detailed bridge crossing information. As stated previously, TC&B felt that the losses through the bridge structures would be minimal due to the flow velocities within the channel. However, structure overtopping would greatly increase the base flood elevation depending on the height of the structure. The S&B HEC-RAS model utilized 2004 ~ 2008 LIDAR data to obtain more accurate cross section data. Additional on-the-ground survey of the channel was conducted to ensure that the minimum channel elevations in the model were accurate. Bridge structures were modeled using information from both field surveys and as-built drawings. An overall comparison of the modeling methodologies has been tabulated and is included in **Appendix D** of this technical memorandum.

j. Population Growth. In an effort to further correlate the differences between **S&B 2011 North Main Drain Model** and the **LOMR (May 17, 2001) / TC&B Model**, S&B conducted a study to determine the population growth for the project area. Based on US Census Bureau data, Hidalgo County experienced a population growth of 48% from the 1990 Census to the 2000 Census. This same area experienced a growth of 36% over the next 10 years based on the 2010 Census data. Over this 20-year period, the total county population experienced an overall population growth of 102%. Additionally, S&B checked the population growth for two cities within the North Main Drain watershed; Edinburg and Alton. The table below illustrates the population growth for these individual cities and for Hidalgo County as a whole. Note that the values presented are based on actual census data and not projections.

Table 3: Population Growth						
Location	**P1990 Census**	**P2000 Census**	**% Growth 1990-2000**	**P2010 Census**	**% Growth 2000-2010**	**% Growth 1990-2010**
City of Alton	3,069	4,384	43%	12,341	182%	302%
City of Edinburg	29,885	48,465	62%	77,100	59%	158%
Hidalgo County	383,545	569,463	48%	774,769	36%	102%

Source: U.S. Census Bureau

6. FURTHER COLLABORATION EFFORT BETWEEN CSE AND S&B REGARDING CORRELATION OF S&B'S 2011 NORTH MAIN DRAIN MODEL (10-DAY HMS) AND FEMA

S&B and CSE have conducted numerous discussions during the QA review process in an attempt to correlate the current **S&B 2011 North Main Drain Model (10-day HMS)** and the results from the **LOMR (May 17, 2001) / TC&B Model**. Recent correlations included:

a. Edinburg Lake Reservoir. During the teleconference of October 20, 2011, the 100-year, 24 hr peak flow rate at Junction User Point 11, (Seminary Road) was identified as approximately 5500 cfs. According to the **LOMR (May 17, 2001) / TC&B Model**, the peak flow rate at this location was 4,178 cfs. However, CSE had a final comment concerning the modeling of the storage areas in the vicinity of Edinburg Lake. According to CSE, there was concern that the storage losses were excessive at element Reservoir-3. In order to address this comment, S&B agreed to revisit the storage calculations for both Reservoir-3 and an upstream element entitled Reservoir-1.

The HEC-HMS output the previous iteration showed that the peak storage for both of these elements was as shown in the table below:

Table 4: Reservoir-1 & 3 Peak Storage / Inflow			
Element	**Peak Storage (ac-ft)**	**Total Inflow (in)**	**Total Outflow (in)**
Reservoir-1	1,747	4.80	3.31
Reservoir-3	7,953	4.53	2.58

S&B revisited the storage calculations for both of these areas using currently available LIDAR data and on-the-ground survey. This was especially crucial in the areas around Edinburg Lake, since there is currently on-going construction around Edinburg Lake. (Current pictures of the Edinburg Lake area are included in **Appendix E**.) Once these new areas were measured and the data input into the HEC-HMS model, the output was re-calculated to determine the revised storage output values. Below is the revised output data for the adjusted elements.

Table 5: Reservoir-1 & 3 Peak Storage / Inflow with Updated Survey at Edinburg Lake			
Element	**Peak Storage (ac-ft)**	**Total Inflow (in)**	**Total Outflow (in)**
Reservoir-1	974	4.81	4.05
Reservoir-3	7,090	4.72	4.29

As these results suggest, it appears that the previous iteration, without the detailed survey, overestimated the amount of available storage within these two elements. As a result of the net loss of available storage, it is no surprise that the peak flow rate at User Point 11 (Seminary Road) increased to 5,889 cfs.

b. Routing Reach Methodology. S&B also made other minor changes to the model. The first revision was to revise more reaches from Muskingum-Cunge 8-point to Modified Puls. The additional reaches revised were R1890, R2670, R2620, R2680, and R1620.

c. **Relocation of Reach-3.** Additionally, the location of element Reach-3 was revised. The previous iteration had this reach located upstream of Reservoir-1. However, after further study of the location of Reservoir-1, S&B felt that this reach should actually be located after Reservoir-1.

d. **LOMR (May 17, 2001) / TC&B Model FIS Profile.** According to the TC&B December 2001 Report, the "existing outfall drainage systems were designed to convey agricultural runoff from a 9.5-year storm event". According the **LOMR (May 17, 2001) / TC&B Model**, the 10-year event for the North Main Drain at Seminary Road is only 527 cfs with a watershed area of 264.72 square miles. In an effort to check the design frequency of the existing channel, S&B prepared a 10-year, 24-hour meteorological model with precipitation data from the **LOMR (May 17, 2001) / TC&B Model** (HEC-1). The incremental precipitation data utilized for this check is shown in the table below.

Table 6: Depth of Rainfall Data for North Main Drain

	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
10-year	2.99 in.	3.52 in.	3.76 in.	4.35 in.	4.95 in.	5.23 in.

The storm duration was reduced from 10-days to 1-day and the hydrologic model results entered into the S&B Base North Main Drain HEC-RAS model. The results from this hydraulic analysis showed that the water surface elevation, for the most part, was below the tops of the adjacent berms, although there were areas where the channel capacity was exceeded, and in some cases, road crossings were overtopped. It should be noted however, that the design of the existing channel would have been **based on the 9.5-year storm event, at the time of the channel design**, and thus would likely be exceeded now, due to the continuing development of the watershed since the design of these existing channel. However, according to the current effective LOMR (May 17, 2001) / TC&B Model FIS Profile, the 10-year water surface is over 5-feet below the low chord of Seminary Road and the ditch banks elevations. Thus, if the existing channel were truly designed to only contain the 9.5-year storm event, it would be appear to be greatly over-designed at this point.

e. **LOMR (May 17, 2001) / TC&B Model FIS Summary of Discharges.** An additional check was to review the Summary of Discharges Table within the current effective FIS. According to this table, the North Main Drain at Seminary Road drains a watershed area of 264.72 square miles, which yields a 10-year peak flow rate of 527 cfs. However, the West Main Drain at the junction with the McAllen Lateral, which drains a watershed area of 92.88 square miles, has a 10-year peak flow rate of 1,081 cfs. Thus, an area that is approximately a third of the North Main Drain area at Seminary Road, is yielding a peak flow rate that is almost 100% higher. Based on these results, it appears that the HEC-1 model and the effective FIS underestimate the peak flow rates within the North Main Drain. This also confirms the opinion of the USACE that the effective FIS does not appear to duplicate what local flood experience was claiming.

7. DIVERSION ANALYSIS

Incorporating the hydrology from the **S&B 2011 North Main Drain Model**, diversion flows were determined from the North Main Drain to the North Main Drain Diversion Channel. For this analysis, a 40-foot lateral weir was placed downstream of Seminary Road (at User Point 11). It must be noted that the flow in the proposed Diversion Channel ultimately consists of three components: (1) diverted flow from the North Main Drain, (2) runoff that would have flowed to the North Main Drain but is intercepted by the proposed Diversion Channel, and (3) runoff that would have flowed to the existing Raymondville Drain system but is intercepted by the proposed Diversion Channel. The first two components will be utilized for the remainder of this discussion only, as component (3) occurs much further north along the proposed Diversion Channel.

The **S&B 2011 North Main Drain Model**, during the base conditions, identifies 5,889 cfs during the 100-year, 24-hour storm event in the North Main Drain at point of the future diversion. Utilizing the 40-foot lateral weir, 1,751 cfs is diverted by the weir to the proposed Diversion Channel. An additional 2,286 cfs that would have flowed to the North Main Drain is intercepted by the proposed Diversions Channel. There is 1,852 cfs of remnant flow to the North Main Drain downstream of the proposed Diversion Channel. These peak flow rates may be revised upon further coordination with the USACE.

Figure 1 on page 17 provides a schematic overlaid on an aerial to illustrate the diverted flows. Ultimately, it is estimated that the proposed Diversion Channel could reduce the flow to the North Main Drain by approximately 69%.

8. CONCLUSION

In order to prepare a FDA for approval from the USACE, it was necessary to coordinate with USACE staff to ensure that the hydrologic and hydraulic modeling used for the FDA were acceptable to the USACE. S&B has worked with the USACE staff over a multi-year period to obtain concurrence for the currently used methodology. This methodology differed greatly from the methodology utilized for the preparation of the hydrologic model for the **LOMR (May 17, 2001) / TC&B Model**. However, based on our review of the **LOMR (May 17, 2001) / TC&B Model** (HEC-1) and the corresponding report prepared by TC&B, the **S&B 2011 North Main Drain Model** (HEC-HMS) contains far more detailed and current information to determine the peak flow rates at various locations throughout the watershed. A quick review of a few of the differences between the various hydrologic models can be found in the table below.

Table 7: Methodology Differences (LOMR / S&B)

Methodology	2001 HEC-1 FEMA LOMR	2011 S&B HEC-HMS for Correlation to FEMA	2011 S&B HEC-HMS for FDA
Computation Software	HEC-1	HEC-HMS	HEC-HMS
# of Sub-Basins	44	125	125
Loss Method	NRCS Curve Number	Initial/Constant Loss	Initial/Constant Loss
Land Use	1995 Development	2007 Development	2007 Development
Lag Time	NRCS Lag Equation	Velocity Method	Velocity Method
Storm Duration	24-Hour	24-Hour	10-Day
Precipitation Data	USGS 98-4044	USGS Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas, 2004	TP-40/TP-49 with Depth/Area Reduction

S&B has worked to address the comments and concerns presented by CSE, including the most recent comment that referred to the modeling of the storage area around Edinburg Lake. This revised hydrologic model (included in **Appendix F**) yields a peak flow rate of 5,889 cfs for the 100-year, 24-hour storm event at the Seminary Road crossing over the North Main Drain. The **LOMR (May 17, 2001) / TC&B Model** stated that the 100-year peak flow rate at this point was 4,178 cfs. Based on the updated methodology, more detailed topographic data, and current land use information, S&B feels that the 5,889 cfs more accurately represents the current peak flow rates within the North Main Drain channel for the 100-year, 24-hour storm event. S&B is also confident that the subsequent changes to the base hydrologic model will be acceptable to the USACE for use in the FDA.

Utilizing this revised model, S&B will prepare 10-day storm duration hydrologic models for use in preparing the FDA for final submittal to the USACE. These hydrologic and hydraulic models will be used to prepare the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year water surface profiles that will be entered into the FDA model to determine the expected annual damages along the existing channel reaches. S&B will be utilizing the latest version of HEC-FDA to prepare the FDA analysis for the North Main Drain. It should be noted that the water surface profiles are only a starting point used by the HEC-FDA program. The HEC-FDA will extrapolate past the 500-year event to the 1000-year event for use in the frequency function. Per the HEC-FDA manual, *"It is recommended that graphical frequency function be defined between the 1- and 0.001-exceedance probability events."*(Section 5.3.2, HEC-FDA User's Manual, November 2008). Additionally, the HEC-FDA utilizes not only the entered water surface profile information, but also uses on conducts a statistical analysis of the entered information, to determine the standard deviation along each water surface profile. During the HEC-FDA computation, it will utilize one standard deviation above and below the entered water surface profile information to prepare the exceedance probability function with uncertainty. This function is then used to determine the expected annual damages along each reach. It is this expected annual damage that will be used during the preparation of the benefit-cost ratio that will be used to analyze the effectiveness of each proposed alternative.

Finally, a LOMR should be requested within 6 months of completion of the proposed improvements (per 44CFR65.3). Even though the 100-year peak discharge identified in the S&B 2011 North Main Drain Model is more than the **LOMR (May 17, 2001) / TC&B Model** (approximately 40%), it does not affect insurance rates at this time. Flood insurance rates are based solely on the current effective Flood Insurance Rate Map (FIRM). A Conditional Letter of Map Revision (CLOMR) may be requested in an effort to obtain any technical comments from FEMA prior to the construction of the proposed improvements, since the proposed project would justify a map revision (44CFR65.8). Note that flood insurance rates are not adjusted based on information provided by a CLOMR.

Respectfully Submitted and Released For Planning Purposes Only Under the Authority of:

Texas PE # 88453
Andres Cardenas, PE

Date: 11 / 30 / 2011

Attachments:

Appendix "A" – USACE-Approved Raymondville Drain Project Methodologies
Appendix "B" – Interim Technical Memorandums (CSE and S&B)
Appendix "C" - FEMA LOMR (May 17, 2001)
Appendix "D" - Overall Comparison of Modeling Methodologies
Appendix "E" – Photo Diary of Edinburg Lake
Appendix "F" - S&B 2011 North Main Drain Model (24-Hr HMS)

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SUMMARY OF QUALITY ASSURANCE REVIEW FOR THE H&H BASE MODELS

Figure 1: Schematic of Diverted Flows

Appendix A
USACE-Approved Raymondville Drain Project Methodologies

USACE Methodologies (4-11-2006)
Raymondville Drain Pre-Project Conditions Report

Purpose – This write-up documents hydraulic and hydrologic modeling conducted for the Raymondville Drain study. The modeling presented was performed for the main stream of the Raymondville Drain located in South Texas near the town of Raymondville. The models represent the without-project condition. The Raymondville Drain planning study will consider the feasibility of flood damage reduction and agricultural drainage improvements for the Raymondville Drain watershed as authorized by the Water Resources Development Act of 1986. The primary focus of the study is flood protection for the city of Raymondville and San Perlita, along with agricultural drainage throughout the basin.

Study Coordination – This study was conducted in cooperation with the primary local sponsor, Hidalgo County. Hidalgo County contracted S&B Infrastructure (S&B) to model the Hidalgo County watersheds as well as the upper reaches of the Raymondville watershed. As a result, the US Army Corps of Engineers Galveston District (SWG) initially developed a model of the lower portion of the Raymondville watershed and merged it with the model developed by S&B.

Models - The two models used in the study are the hydrologic (HEC-HMS) and the hydraulic (HEC-RAS) model. The HEC-HMS model covers the entire watershed, but was coded as two separate models, an upstream model and a downstream model. The HEC-RAS model covers the main stem of the Raymondville drain and was coded as an upstream and downstream model. The HEC-HMS and HEC-RAS models were developed by S&B and SWG. S&B developed the upstream models; SWG developed the downstream models.
Models of this area were developed in the past for the purpose of flood insurance studies. The new models were developed to take advantage of new software and new digital topography.

Model Simulations - The models were used to simulate a range of hypothetical flood frequency events. The specific flood frequency events that were simulated were the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year. These hypothetical flood events were modeled to develop the stage and flow frequency results needed for a flood damage analysis.

Pre-Project Base Conditions – The economic analysis for the Raymondville Drain study will consider the economic viability of alternative measures over a 50-year project life. The base year for the analysis will be 2010. The flood frequencies presented in this report represent that year.

Tributary Modeling – The tributaries that drain the town of Raymondville were simulated in order to determine if there was a federal interest. There are three main tributaries that drain the town of Raymondville. One tributary drains the west side of town and conveys the storm water to the South Hargill ditch which then feeds the water into the Raymondville drain. The other two tributaries drain the east side of town. One of these conveys flow to the South Main Drain. The other conveys flows to the Raymondville Drain a few miles east of town. Only the two tributaries that convey water to the Raymondville drain were modeled. The results of the modeling indicated that the tributaries did not generate sufficient flood flow to qualify for federal participation. The required flow limits are described in ER 1165-2-21, Flood Damage Reduction Measures in Urban Areas, as paraphrased below:

> Urban water damage problems associated with a natural stream or modified natural waterway may be addressed under the flood control authorities downstream from the point where the flood discharge of such a stream or waterway within an urban area is greater than 800 cubic feet per second for the 10-percent flood (one chance in ten of being equaled or exceeded in any given year) under conditions expected to prevail during the period of analysis. Those drainage areas which lie entirely within the urban area (as established on the basis of future projections, in accordance with paragraph 5 above), and which are less than 1.5 square miles in area, shall be assumed to lack adequate discharge to meet the above hydrologic criteria. Those urban streams and waterways which receive runoff from land outside the urban area shall not be evaluated using this 1.5 square mile drainage area criterion.

The tributary modeling indicates that there are flood risks in Raymondville caused by inadequate capacity of these tributaries. This is in addition to flooding originating with the Raymondville Drain. The tributary component of the flooding cannot be addressed in the Federal study because the tributaries do not satisfy ER 1165-2-21.

Related studies and models – Several hydrologic studies have been conducted in this region over the past 25 years.

The Federal Emergency Management Agency conducted a study of Hidalgo County titled, "Flood Insurance Study, Hidalgo County, Texas." Although this study was conducted in Hidalgo County, it noted that Hurricane Beulah is considered to be equivalent to the hypothetical 100-yr storm. This document was last revised on June 6, 2000.

The US Army Corps of Engineers, Galveston District conducted extensive studies documented in, "Lower Rio Grande Basin, Texas, Flood Control and Major Drainage Project General Design Memorandum." This report was published in January 1982.

Data Sources – The models, supporting data, and resultant flood elevations are referenced to NAD 83, state plane south zone and NAVD 88. Lidar topographic data along the Raymondville Drain are referenced to the same datum.

Watershed description - The Raymondville drain watershed is located in the Rio Grande valley of South Texas. The majority of the watershed is within Willacy County and a small portion is within Hidalgo County. The watershed encompasses four towns, Raymondville, San Perlita, La Sara, and Hargill. The total watershed area is approximately 490 square miles. The drain flows in an east west direction and empties into the Laguna Madre south of Port Mansfield, TX. The watershed consists of mainly agricultural land and flat coastal prairie with little topographic relief. The drain is a man made ditch sized primarily for the purpose of agricultural drainage. A network of tributaries is also located within the watershed. These tributaries provide flood reduction and drainage.

Stream Gages and Records - There are no stream gage records for the study area. However, there are weather service rain gages at several locations. Historical rainfall records were used in the analysis to infer an apparent flood frequency range associated with two historical flood events. Assigning a frequency range to the two events (Hurricane "Beulah" and the November 2002 storm) was useful for judging the accuracy of model results.

Hydrologic Model

Split Location - The HMS model was coded as two separate models, an upstream model and a downstream model. The junction between the upstream and downstream model is located just downstream of the town of Raymondville, see Figure 1.

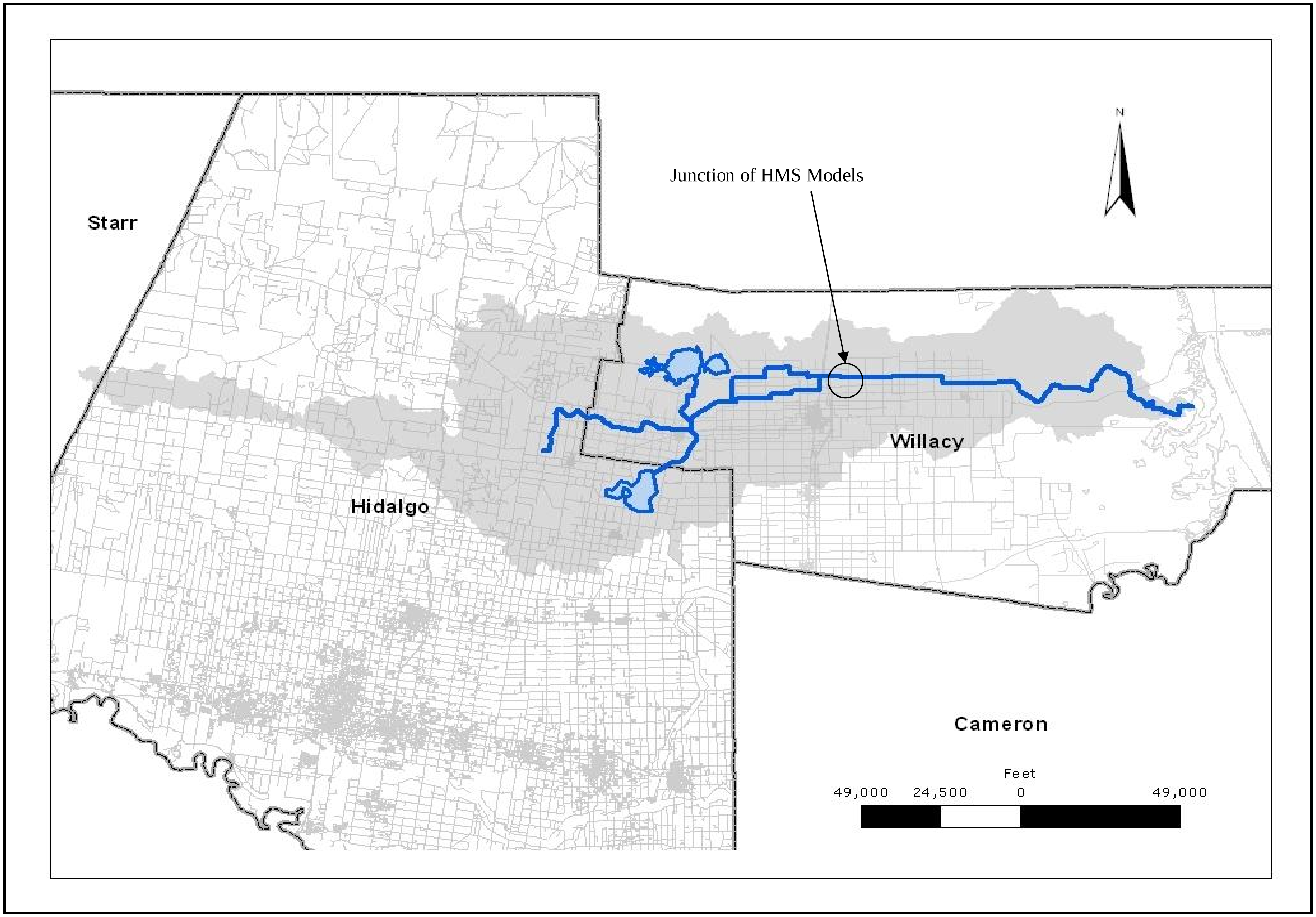

Figure 1: Location of HMS Model Junction

Watershed Delineation Method – Basin and subbasin boundaries are poorly defined for the Raymondville Drain. Tributary alignments cut across natural surface flow patterns so that in-channel flows are diverted along tributaries but revert to natural flow paths when channel capacity is exceeded. Thus, basin and subbasins can be delineated to reflect surface topography or to reflect tributary patterns. A decision was made to base the delineations primarily on surface topography. Testing using both assumptions showed that this method was conservative for Raymondville. Computed stages for the 10 percent flood event would only vary about 0.1 feet for either assumption.

Watershed Delineation - The GIS software ARC-VIEW was used to create the HEC-HMS model using a software extension known as GEO-HMS developed by the Hydrologic Engineering Center (HEC). Geo-HMS enables the user to delineate a watershed using digital terrain. Detailed lidar survey data did not encompass the entire watershed; instead it was collected for the town of Raymondville and a 3000 ft width along the main drain. The remainder of the watershed survey data was obtained from DEM's representing USGS quad maps. The elevations of the DEM file had to be converted from meters to feet so that the DEM could be combined with the lidar digital terrain data. During the process of combining the two data sources, the portion of the DEM that overlapped the lidar digital terrain was deleted. This action was taken to avoid conflicting elevations. The DEM data and the lidar digital terrain data were combined to produce a grid. A 60 ft by 60 ft grid size was used.

Reconditioning the Watershed Grid - Due to the relatively flat topography, the grid was reconditioned or edited in order to ensure that GEO-HMS could identify the correct location of the main drain. A line file was created of the drain. This file was burned into the grid to create a more deeply incised feature along the main drain alignment.

As a result of splitting the hydrologic modeling into an upstream and downstream model, SWG was required to delineate the watershed of the downstream model while at the same time connecting to the upstream S&B watershed delineation. A border or fence was placed along this boundary using Geo-HMS. This process raised the grid along the boundary of the two models, which resulted in the downstream watershed delineation having a border along this boundary.

Geo-HMS to HEC-HMS - GEO-HMS was used to delineate the Raymondville watershed into 22 sub-basins within the downstream model segment. It was also used to create a basin model and background map that could be used in HEC-HMS. GEO-HMS extracted the following HEC-HMS sub-basin characteristics:

Drainage Areas
Watershed Length
Watershed Length to Centroid
Channel Slope
Watershed Slopes
Flow Paths
Elevations

HEC-HMS Basin Model

Sub-basin Rainfall Loss Potential – The initial/constant loss rate methodology was utilized for the infiltration modeling. The initial/constant loss rate method assumes that the initial rainfall increments are absorbed up to a certain initial rainfall loss value specified in inches. All other losses are represented with a constant loss rate specified in inches per hour. No excess precipitation occurs until the initial loss is satisfied. The initial/constant loss rate methodology required parameters are the initial loss and the constant loss rate as described below.

Initial Loss – Initial loss rates were derived with the following soil conservation service (SCS) equations, which relate the initial loss to the soil curve number (CN).

$$I = 0.2S$$

$$S = \frac{1000 - 10CN}{CN} \quad (1)$$

I = Initial loss (in)
S = Potential Maximum Retention
CN = Curve Number

The CN's for the Raymondville sub-basins were estimated as a function of land use, soil type, and antecedent moisture conditions, using tables published by the SCS, in Technical Report 55 (TR-55). For each sub-basin, a series of calculations were made in order to obtain the curve number needed to estimate the initial loss. Twenty cover types and hydrologic conditions contributed curve numbers to four hydrologic soil groups, A, B, C, and D. A weighted curve number was calculated for each soil group type, and then for each sub-basin. An initial loss for each sub-basin was then determined.

Constant Loss Rate – The required constant loss parameter was based on the SCS recommendations for specific hydrologic soil groups, as seen in Table 1 below. Each sub-basin within the watershed contained a percentage of each SCS soil group assigned to it as previously stated. These percentages in combination with Table 1 were used to determine a weighted constant loss rate for each sub-basin.

The suitability of the adopted initial and constant loss values for flood flow frequency simulations were confirmed by comparing the HEC-HMS results with independent methods as discussed later in the calibration section.

Table 1: SCS soil groups and infiltration (loss) rates (SCS, 1996; Skaggs and Khaleel, 1982)

Soil Group	Description	Range of loss Rates (in/hr)
A	Deep sand, deep loess, aggregated silts	0.30-0.45
B	Shallow loess, sandy loam	0.15-0.30
C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05-0.15
D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0.00-0.05

Transform - SCS Unit Hydrograph - The SCS unit hydrograph method was used to compute direct runoff hydrographs from excess precipitation. This method is based on empirical data from small agricultural watersheds across the United States and uses parametric equations to compute the hydrograph peak and the time base from the lag. The SCS UH method incorporates a peaking factor that is representative of an average watershed for the United States. Raymondville drain is much flatter then the average watershed for the U.S., thus the peaking factor was adjusted from 484 to 150 as described in the research document Revisit of NRCS Unit Hydrograph Procedures, Fang, 2005. The UH was adjusted by the recommendations given in the research document in order to maintain one unit of volume under the Unit Hydrograph. HEC-HMS would not accommodate a non-standard peaking factor, so unit hydrograph ordinates were computed for each subbasin and coded into HEC-HMS manually.

Flood Hydrograph Routing and Routing Steps - Routing is the process of accounting for the travel time and attenuation of the flood hydrograph as it traverses a reach. SWG used two methods to calculate the routing for the HMS model. Routing reaches along the mainstream of the Raymondville drain utilized the Modified Puls method. The remaining overland flow reaches were developed using the Muskingum-Cunge 8-point method.

Modified Puls - The modified Puls method requires a storage-outflow curve for each reach. The HEC-RAS model was used to compute the storage-outflow curve for each reach along the Raymondville drain.

Muskingum-Cunge – The Muskingum-Cunge 8-point method was used to represent the overland flow reaches of the watershed because this method would likely produce sufficient results without the need for detailed cross-sections. This method describes the channel with eight station-elevation coordinates describing the typical channel and floodplain shape in the reach. The eight station-elevation coordinates, slope, and length of each reach were determined from the digital elevation grid. The Manning's n-value roughness coefficients for the left over-bank, main, and right over-bank were all set to 0.1.

HEC-HMS Meteorological Model - The precipitation data necessary to simulate the watershed processes are stored in the meteorologic model. The frequency storm method was used to capture the precipitation data. A 10-day storm duration was chosen for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year storm event. The source for the point rainfall data was the National Weather Service (NWS) TP 40 and TP 49. More modern rainfall atlas data are available from USGS publications but aerial adjustment data for all storm durations are only known for TP40/TP49.

HEC-HMS Control Specifications - The control specifications include the start and end dates and times along with the time interval. Testing showed that a one-hour computation interval would provide sufficient definition of each hydrograph. Start and end dates were set to provide 30 days of continuous simulation.

Hydrologic Model Adjustments – Preliminary model tests indicated that the models provided by S&B for the upstream portion of the basin under estimate flood frequency at Raymondville. This was concluded based on flooding accounts from local residents and also based on analysis of the November 2002 flood. Local authorities and residents were interviewed to establish reasonable flooding patterns. These observers reported that the Raymondville Drain fills at least half full almost every year. Furthermore, the observers claimed that the town of Raymondville is impacted by regional storm events every 6 to 7 years on average. The S&B models produced flows that would not flood the town until the 25-yr or 50-yr event, and the main channel would not fill half full until the 10 yr to 25 yr event.

An analysis of the rainfall frequency of the November 2002 flood was made to provide additional clues as to the accuracy of the S&B models at Raymondville. A frequency band was determined by taking the rainfall gage data for this event and determining the peak 1 hour duration up to the peak 10-day duration. This was then plotted with TP-40 rainfall frequency curves for the various durations and frequencies. It was concluded from this comparison that this event was roughly a 2-year to 5-year frequency. Photographic evidence show the main channel at the town of San Perlita was at least bank full. It is therefore likely to assume that the channel was full or near full at the town of Raymondville. The S&B methods produced flows at Raymondville of only 11 cfs for a 2-year event and a minimal stage.

As a result of this evidence it was concluded that the S&B methodologies would need to be adjusted to better replicate the flooding accounts for the town of Raymondville. The adjustments made are listed in Table 2 below and discussed in the following paragraphs.

Table 2: Changes to Upstream HMS model.

Data Type	Original Data From S&B	Adjusted Data
Storm Duration	24- hr Storm Duration	10-day Storm Duration
Loss Method	SCS Loss Method	Initial/Constant Loss Method
Point Rainfall Source	USGS point rainfall from 98-4044 was used with no area adjustment.	TP 40/49 point rainfall with an area adjustment.
Unit Hydrograph	SCS, with a peaking factor of 484.	SCS, with a peaking factor of 150.
Lag Time	Based on the SCS TR-55 CN equation	Longest Flow Path w/ assumed velocities.
Muskingum-Cunge 8-point	Manning's Roughness = 0.06	Manning's Roughness = 0.1

Storm Duration and Loss Method - The changes to the storm duration and the loss method are inter-related. Originally the S&B storm duration was set at 24-hours and the loss method was the SCS curve number method. The storm duration was reviewed and it was found that the duration was too short to be in accordance with the suggestions found in EM 1110-2-1417, which states:

> Associated with application of a hypothetical storm is selection of a storm duration. When a balanced hypothetical storm is used, the duration is generally chosen to equal or exceed the time of concentration for a watershed.

The HEC-HMS technical reference manual, Chapter 4, also suggests the following:

> What duration should the event be? The hypothetical storm options that are included in HEC-HMS permit defining events that last from a few minutes to several days. The selected storm must be sufficiently long so that the entire watershed is contributing to runoff at the concentration point. Thus, the duration must exceed the time of concentration of the watershed; some argue that it should be 3 or 4 times the time of concentration (Placer County, 1990).

Calculations show that the time of concentration for the entire watershed is about five days. This calculation was accomplished by adding up the total travel time from the most upstream point in the HMS model through the town of Raymondville for a bank full event. Thus, the 24-hour storm duration in the original S&B model is too short. Ultimately, a ten day duration was selected. It should be noted that some of the most significant flood events for the Raymondville Drain have had multi-day durations.

The SCS curve number loss method used in the S&B model is not appropriate for storm durations greater than 24 hours. This is documented in the following reference:

> In Practice, the [SCS loss method] procedure has a basic fault in that it theoretically assumes that the infiltration rate eventually goes to zero. Theoretically, the actual infiltration rate should probably approach a constant minimum rate… Thus, the [SCS loss method] curve number method may be slightly conservative when used for predicting runoff from long-duration storms. Because of this limitation, its use is probably questionable for areas greater than perhaps 5 to 10 sq mi since drainage areas that size or larger have times of concentration that may be longer than the time required for the infiltration capacity to reach a minimum. (*Roberson, Cassidy, Chaudry. Hydraulic Engineering. 2nd Ed. Ch.2-6*).

As a result of this documented limitation with the SCS loss method, the initial/constant loss method was adopted. The advantage of using this method rather than the SCS loss method is that the constant loss rate will not deplete to zero during long duration storms. Further support for using a ten-day storm duration and the initial and constant loss method is demonstrated in figure 2. This shows that the computed peak flow for the 100-year event increases with storm duration up to about a ten-day event. Using the SCS-curve number method, the resulting peak flow continues to increase beyond 10 days.

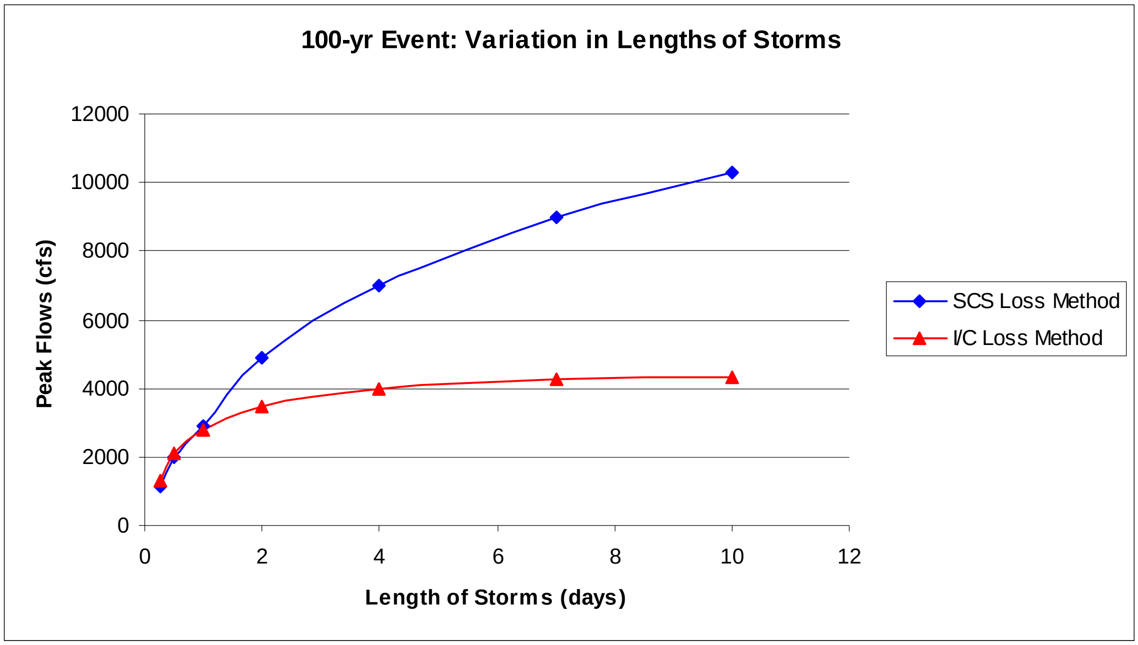

Figure 2: Peak Flow vs. Storm Duration, 100-yr Event.

Point Rainfall Source - The point rainfall in the original S&B model was based on the USGS report 98-4044. This was changed to the National Weather Service (NWS) TP 40 and TP 49. The USGS point rainfall data is more modern; however it does not provide depth-area adjustments for the various storm durations. On the other hand, the NWS TP 40 and TP 49 rainfall atlas has area adjustment factors for all the storm durations. The decision to use TP-40 and TP-49 rainfall was not a critical change because rainfall depths are similar in both sources. However, it was concluded that it would be more consistent to use rainfall and area adjustments from the same source.

Unit Hydrograph – The HMS modeling for the Raymondville drain uses the SCS dimensionless unit hydrograph (UH) method. This method has two variables, the lag time and the peaking factor. In most modeling applications and in the original S&B model the peaking factor is assumed to be 484, representing the typical watershed in the U.S. A decision was made to change the peaking factor to a value of 150. The Raymondville drain watershed is much flatter than the norm; therefore the peaking factor should be adjusted to better represent the type of hydrographs that this area would likely produce. The research document, Revisit of NRCS Unit Hydrograph Procedures, *Fang, 2005*, discusses selection of peaking factors and describes how to develop UH ordinates for non-standard values. These new UH's were computed in a spreadsheet, and manually input into the HEC-HMS model.

Lag Time – The original source of the lag times provided by S&B was unknown. As a result, new lag times were computed using a method for estimating time of concentration. Travel time along the longest flow path in each subbasin is computed based on flow velocity. The method assumes the first 500-ft length is sheet flow, the next length is represented as shallow concentrated flow and is equal to 15% of the total length, and the remaining length is assumed to be channel flow. The time of concentration is then calculated and converted to a lag time by multiplying it by 0.6 as suggested in TR-55. This method produced shorter time of concentrations when compared to the lag times furnished by S&B, as well as more reasonable HMS results. This will be discussed in further detail in the model calibration section.

An accurate SCS lag was needed because the UH was already flattened by the adjustment to the peaking factor. If an excessively long lag was used in combination with a reduced peaking factor, then an unrealistically low peak flow rate would likely occur for each sub basin.

Muskingum Cunge 8-point routing method roughness – The roughness value for the original S&B HMS model was set to 0.06. This was increased to 0.1 to account for the relatively flat terrain, as well as the numerous roads and elevated irrigation ditches that crisscross the watershed.

Hydraulic Model

Junction Location - The hydraulic model or RAS model was coded as an upstream and downstream model. The junction between the upstream and downstream RAS models is in a slightly different location then that of the HMS model junction, see Figure 3.

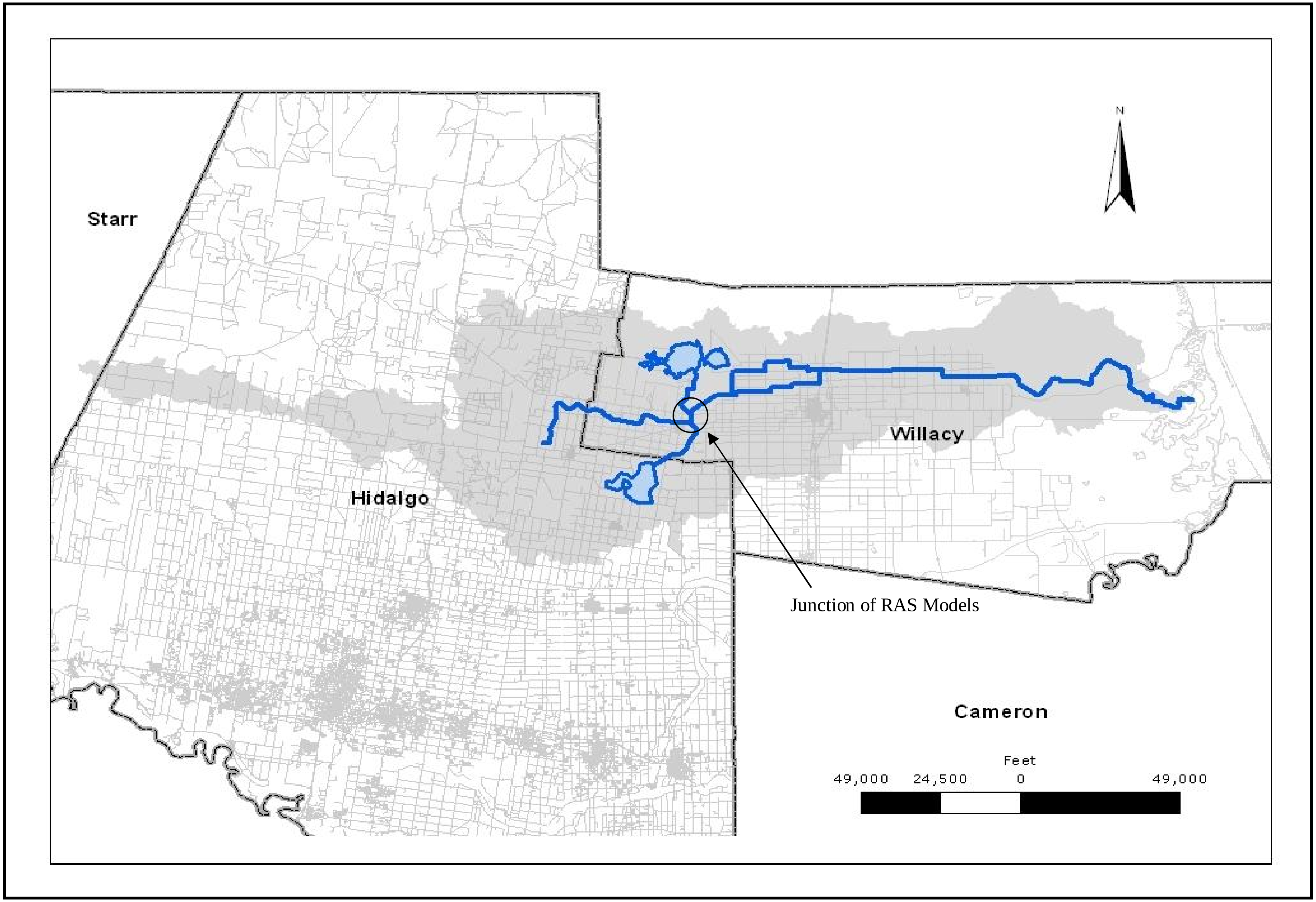

Figure 3: Location of Hydraulic Model Junction.

HEC-GeoRAS - SWG constructed the downstream RAS model. GIS software was used to combine lidar and USGS DEM data into a single TIN. Within the GIS software the TIN was manipulated with the aid of HEC-geoRAS software to create the HEC-RAS geometry file. Discrepancies between the USGS DEM and lidar created problems with the cross-sections in the RAS model. The final geometry file was adjusted to compensate for these discrepancies. This adjustment was accomplished by changing cross-section anomalies to resemble the topography of USGS Quad maps.

Geometry File - The Manning's roughness values were input into the model with the aid of orthophotos. The Manning's roughness values inside the channel ranged from 0.04 to 0.055. The roughness values in the over-banks were set to 0.1. This value was selected due to the fact that the area is extremely flat with numerous obstructions, such as roads and elevated irrigation ditches. It is likely that any water in the over-banks will move very slowly. It was therefore assumed that the most reasonable technique for modeling these characteristics would be to use a high roughness value such as 0.1. The ineffective flow areas were set by viewing USGS quad maps and orthophotos to locate the areas that would not convey any flow in the over banks.

The suitability of these adopted RAS variables were tested for sensitivity and are discussed in the calibration section.

Adjustments to S&B HEC-RAS Model - The upstream RAS model as provided by S&B was modified. The modifications are listed below in Table 3.

Table 3: Adjustments to Upstream S&B RAS model

Method	**Orig. S&B RAS Model**	**Adjustments to S&B RAS Model**
Over-bank Roughness	0.035	0.1
Delta Lake Reach	Not modeled	Modeled

The S&B over bank roughness was 0.035. This was increased to 0.1 to be consistent with the downstream SWG model. Justification for this value is described above.

The reach from Delta Lake to the junction just south of the diversion to La Sal Vieja was originally coded in the S&B HMS model using the kinematic wave routing method rather then the more physically based Modified Puls method. This method was not attenuating the peak in a manner that would likely occur. A flow volume curve was derived with HEC-RAS and input into the HMS model for the Modified Puls routing method. This resulted in more realistic peak flows at the channel junction just downstream of La Sal Vieja.

These adjustments were made to allow the upstream and downstream RAS models to merge more easily and to produce more realistic peak flows. Despite these adjustments, there are still residual differences in the two RAS models.

The main differences that still exist between the upstream and downstream RAS models are listed in Table 4. The first difference is levees (spoil mounds) on each side of the main Raymondville Drain, thus assuming that no water would leave the drain unless these mounds were overtopped. Where as, the downstream model does not consider these spoil mounds to be continuous, confining levees. There are two key reasons that SWG did not model the spoil mounds as levees. First, there is photographic evidence that the main Raymondville drain will flood the over-banks before the spoil mounds overtop, see Figure 4. Second, there are numerous breaks, tributaries, and storm drains all along the spoil mounds suggesting that these spoil mounds do not act as continuous levees.

Table 4: Residual difference between upstream and downstream RAS models.

Upstream Model	**Downstream Model**
Levees were used	No Levees were used
Cross-section width is narrow	Cross-section width is wide


RAYMONDVILLE DRAIN:
VIEW FROM BRIDGE ON FM 2209
APPROXIMATELY 1/2 MILE NORTH OF THE INTERSECTION OF FM 2209 AND FM 3142
SAN PERLITA TEXAS
LOOKING NORTH
PHOTO DATE: 11-04-2002

Figure 4: Photograph of flooding along Raymondville Drain.

A second residual difference in the RAS models is that the upstream model used narrower cross-sections than the downstream model. All cross-sections within the downstream model have a width equal to that of the watershed. This was done to capture all of the storage in the over-banks, which results in a more comprehensive flow-volume curve used for routing reaches in the HMS model. It does not appear that the narrow cross-sections of the upstream model capture all of the available storage.

These two differences did not appear to critically impact the model results so no adjustments were made.

Model Calibration - There were several sources used in the calibration of the hydrologic model. These include the USGS regression equation method from Report 96-4307, Hurricane "Beulah" of 1967, the storm of November 2002, and two reports from local residents in the Raymondville area.

The USGS Regional Equation method from Report 96-4307 was used to compute flows for the 2-yr through the 100-yr frequency events. The equations can be seen on Table 5 below. The resulting peak flows associated with these equations can be seen on Table 6. Of the various frequency equations developed, only the 2-yr and 100-yr are valid equations, because the remaining frequency equations do not possess variables that are within the specified range as shown in Table 5. Therefore, only the peak flows from the 2-yr and 100-yr events were considered. It was reasoned that the peak flows, in the Raymondville watershed, for the 2-yr and 100-yr frequency event should be lower then these two USGS frequency values. The reasoning for this is because these regional equations were developed based on natural streams, and Raymondville drain is not a natural stream. If a natural stream overflows its banks due to heavy rains then the water will spill out into a natural floodplain, which would likely be a confined area close to the stream. On the other hand, when the Raymondville drain overflows due to heavy rains the water will spread out over a much larger area. This would result in increased roughness and more storage volume. This effect is likely more pronounced for large events like the 100-year than for the small in-bank events. Thus, the peak flows within Raymondville drain would likely be significantly smaller for a 100-yr flood event and slightly smaller for a 2-yr event of a natural stream in the same region. The final flow frequency curve can be seen on figure 5 below.

Table 5: Regional Equations for estimating Peak Flow, USGS 96-4307

Frequency	Equation	Variables	Variable Range of Tolerance
2-yr	$Q_2 = 66.2\ A^{.630}\ SH^{-.423}$	A = 325; SH = 1.72	A: (0.36 - 15,4287); SH: (0.011 - 10.9)
5-yr	$Q_5 = 931\ A^{.424}\ SL^{-.410}$	A = 325; SL =1.23	A: (0.36 - 15,4287); SL: (6.88 - 98.9)
10-yr	$Q_{10} = 1720\ A^{.410}\ SL^{-.419}$	A = 325; SL =1.23	A: (0.36 - 15,4287); SL: (6.88 - 98.9)
25-yr	$Q_{25} = 3290\ A^{.398}\ SL^{-.428}$	A = 325; SL =1.23	A: (0.36 - 15,4287); SL: (6.88 - 98.9)
50-yr	$Q_{50} = 4970\ A^{.391}\ SL^{-.434}$	A = 325; SL =1.23	A: (0.36 - 15,4287); SL: (6.88 - 98.9)
100-yr	$Q_{100} = 1780\ A^{.440}$	A = 325	A: (0.36 - 15,4287)

Table 6: USGS Regional equation peak flow results

Frequency	Peak Flow (cfs)
2-yr	2,015
5-yr	*9,953*
10-yr	*16,927*
25-yr	*30,150*
50-yr	*43,685*
100-yr	22,694

Note: Italics represents invalid results

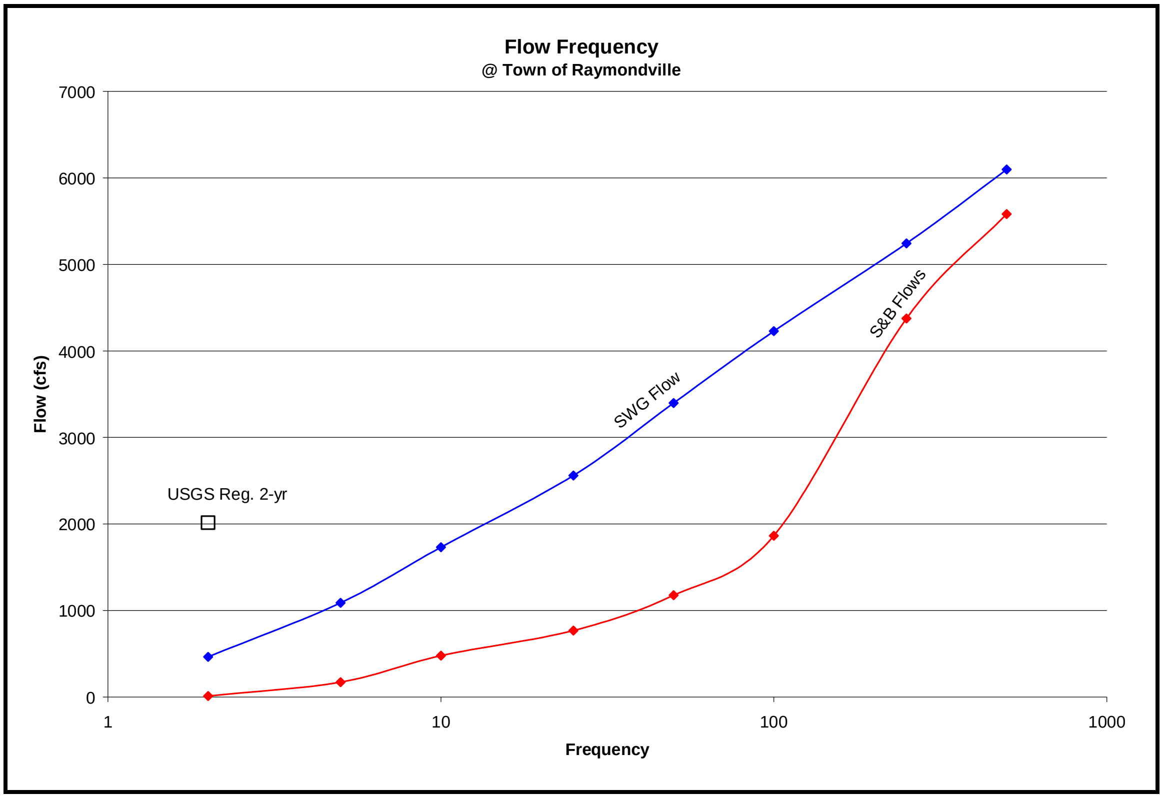

Figure 5: Flow Frequency Curve at the town of Raymondville

The hydrologic and hydraulic models were also calibrated by simulating Hurricane "Beulah" of 1967. The US Army Corps of Engineers developed a report for Hurricane "Beulah". In this report there are high water marks, daily rainfall totals, and flood plain plots. A high water mark of 33 ft was extracted from the report and used for model calibration. A likely corresponding frequency range was then determined. To accomplish this, the daily rainfall totals from the Beulah report were used to develop a rainfall versus storm duration curve that represented Hurricane "Beulah". This curve was then compared to several point rainfall versus duration curves for a range of frequencies from TP 40/49, see figure 6. This comparison helped determine a frequency range for Hurricane "Beulah". When looking at figure 6, one can see that the Beulah frequency fluctuates with storm duration. As the duration approaches 10 days, the storm becomes more representative of a 100-yr or greater event. The critical duration for this watershed is about 7 days. Thus, a frequency range of 100-yr to 250-yr was inferred for the flood produced by this storm.

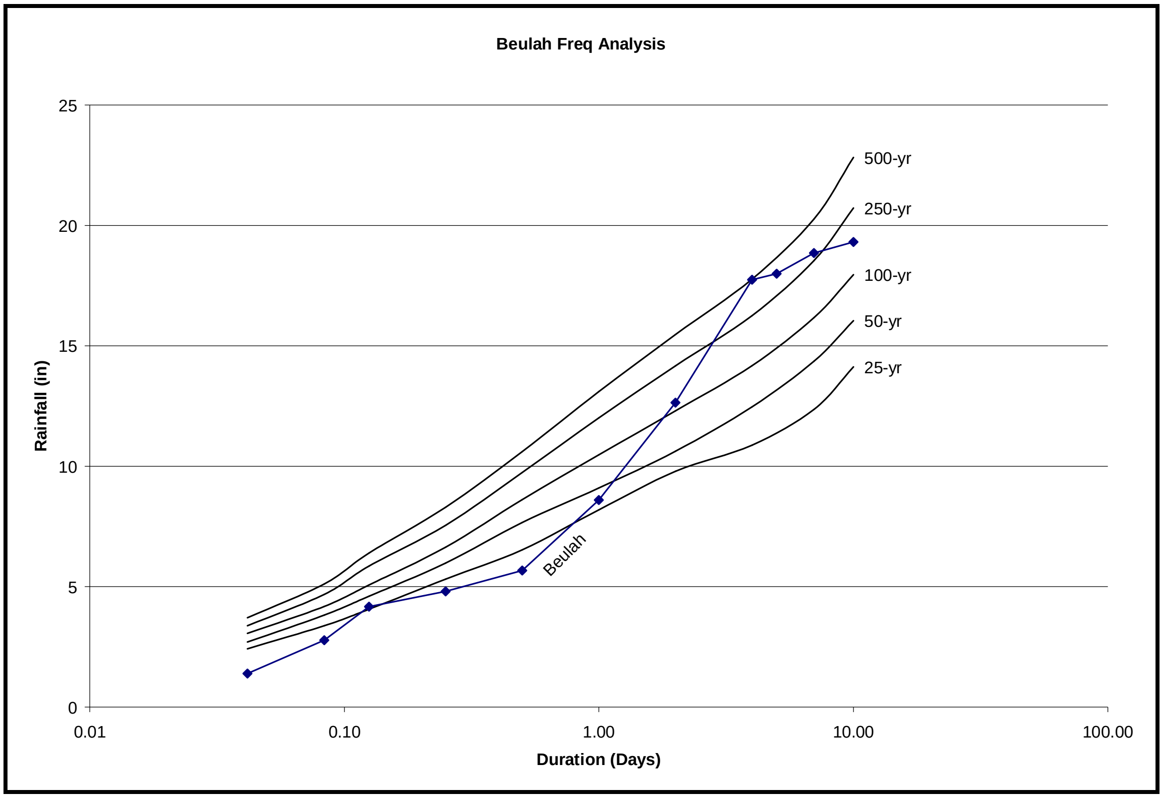

Figure 6: Hurricane "Beulah" Frequency Analysis

A similar technique was used to determine a frequency range for the storm of November 2002. The peak stage for this storm was the result of multiple small events over the period of a month. Therefore the peak rainfall durations were collected for this same one-month period. The results can be seen on figure 7. The frequency range was concluded to be the 2-yr to the 5-yr event. A stage range was based on the photographic evidence near San Perlita, which shows the main Raymondville drain full just north of the town. This suggests that the main Raymondville drain just north of Raymondville was also bank full or near bank full.

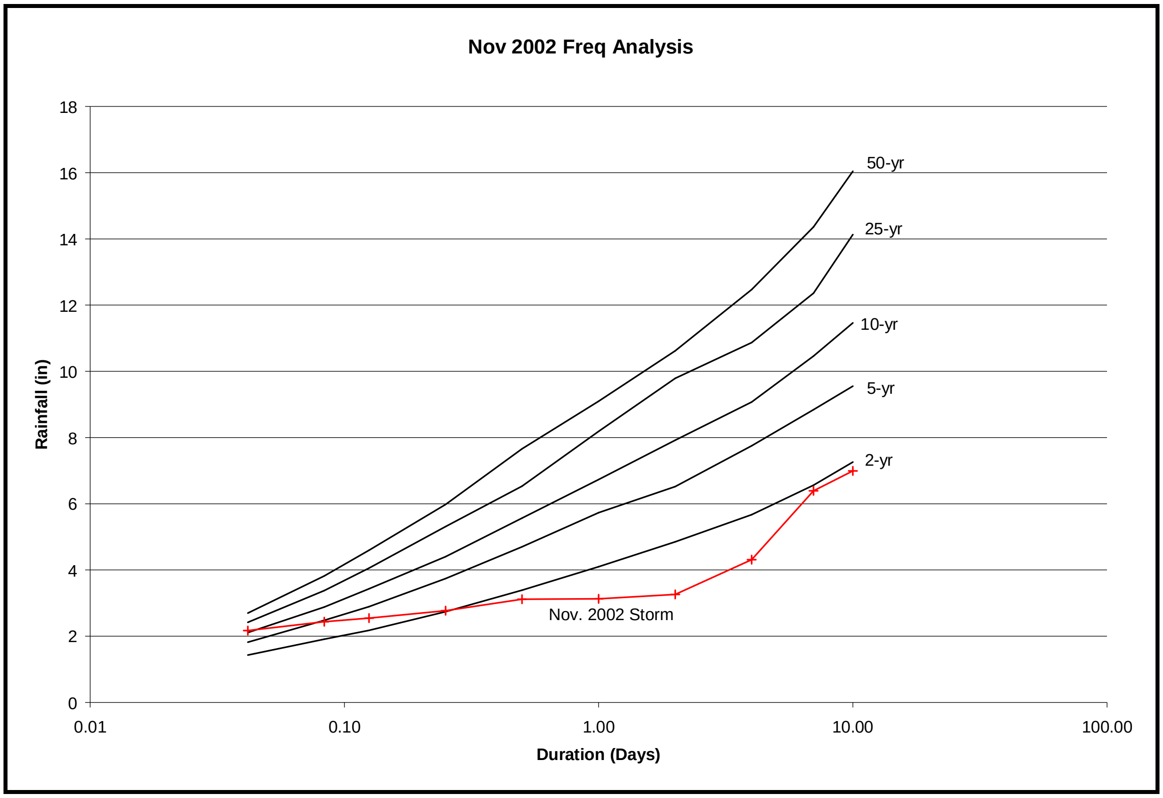

Figure 7: Storm of November 2002 Frequency Analysis

The calibration of the models also considered the reports of two residents in the town of Raymondville. The first resident reported that the main Raymondville drain north of town would fill half full at least once per year. A stage ranging from half full to ¾ full for the 2-yr frequency event was used to represent this report. The second reported that a region wide storm would flood the town of Raymondville every 6-7 years on average. A stage range was selected in order to quantify flooding of the town. The range was set at 28 ft to 30 ft, because 28 ft is the stage at which street flooding would likely occur, and 30 ft is a few feet higher to account for any uncertainty associated with the effects of the tributaries. These reports were then plotted on a stage frequency chart as target windows to determine the level of accuracy in the models. This chart can be seen in figure 8 below. The S&B methodologies produce similar results to the SWG methodologies for the 250-yr event and higher. However, the two models diverge significantly for the more frequent events. The target windows are only crude estimates, but they do lend credence to the SWG results. It should be pointed out that use of the S&B results without adjustment would result in dramatically lower flood damage estimates for Raymondville and thereby lessen the apparent justification for any remedial action. The largest contribution to expected annual damage comes in the 2-year to 25-year flood damage rather than very large but rare floods.

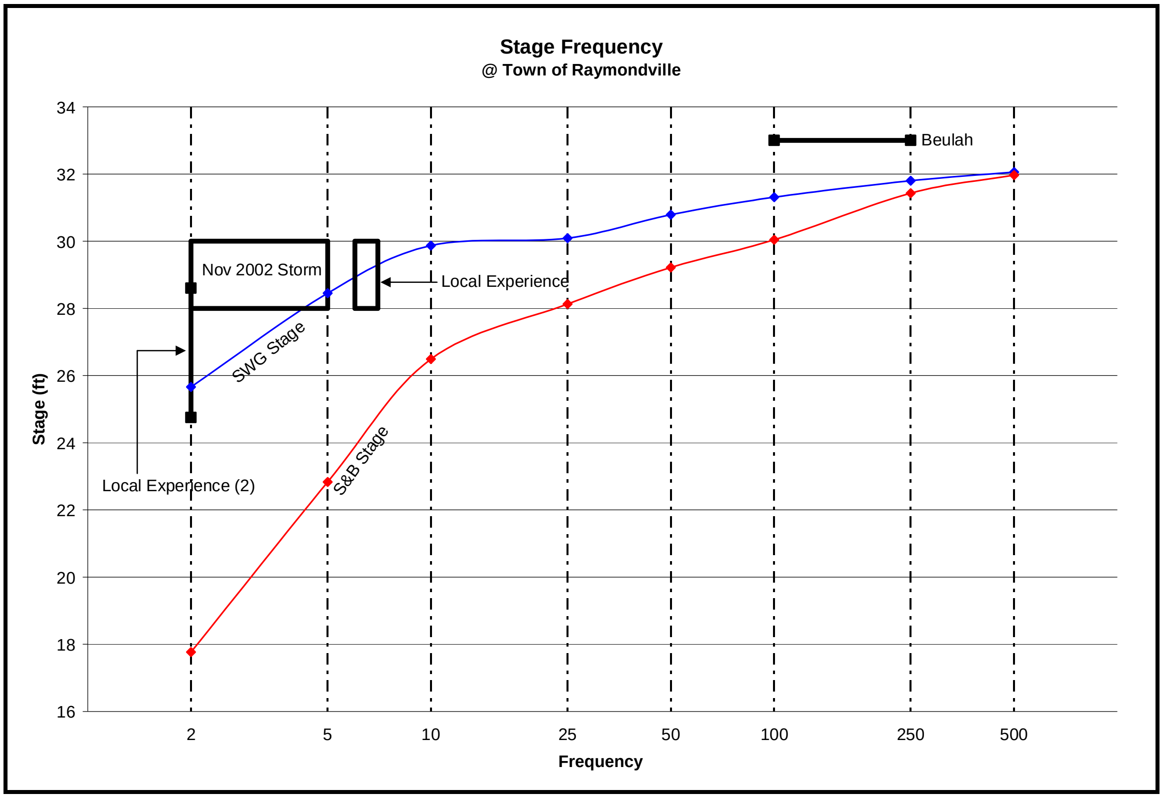

Figure 8: Stage Frequency Curve at the town of Raymondville

The model calibration confirmed the choice of using the initial and uniform loss method and the values selected as previously described. The final loss variable for the upstream and downstream HMS model can be seen on Tables 7 and 8 below, and the final Tc values can be seen on Table 9 and 10 below.

Table 7: Final Loss Variables for Downstream HMS Model.

Sub-Basin	Initial Loss (in)	Constant Loss Rate (in/hr)
R490W490	1.1	0.17
R500W500	1.5	0.23
R130W130	2.7	0.23
R600W600	1.3	0.21
R410W410	1.5	0.15
R230W230	2.3	0.24
R530W530	1.6	0.2
R560W560	3.0	0.2
R470W470	3.0	0.2
R590W590	3.0	0.22
R460W460	2.9	0.19
R370W370	3.0	0.26
R240W240	3.0	0.34
R540W540	3.0	0.2
R210W210	0.8	0.13
R190W190	2.5	0.23
R220W220	1.1	0.15
R250W250	1.3	0.21
R450W450	0.9	0.13

R510W510	1.5	0.26
R480W480	1.2	0.2
R520W520	1.0	0.19

Table 8: Final Loss Variables for the Upstream HMS Model.

Sub-Basin	Initial Loss (in)	Constant Loss Rate (in/hr)
R460W460	3.32	0.16
R470W470	3.22	0.15
R1660W1640	3.02	0.14
R520W520	4.1	0.2
R610W610	4.12	0.2
R730W730	4.55	0.21
R600W600	3.59	0.16
R1640W1620	5.95	0.26
R1630W1610	3.45	0.16
R860W760	4.58	0.21
R830W830	5.36	0.25
R900W890	4.6	0.23
R1030W1030	4.12	0.22
R940W940	4.46	0.22
R1040W1040	4.15	0.21
R1060W1060	3.47	0.21
R1360W1360	4.69	0.23
R1130W1130	4.31	0.21
R1590W1580	4.22	0.22
R1550W1550	4.17	0.22
R1510W1510	4.44	0.22
R1120W1120	3.59	0.2
R1220W1220	4.28	0.22
R1270W1270	3.88	0.22
R930W930	4.01	0.22
R1080W1080	2.32	0.15
R1620W1600	3.13	0.21
R1670W1650	3.77	0.22
R1340W1340	4.19	0.22
R1010W1010	2.14	0.14
R660W660	4.04	0.21
R750W750	3.74	0.19
R770W770	3.79	0.21
R810W810	3.92	0.21
R790W790	3.31	0.21
R630W630	3.07	0.21
R620W620	2.82	0.2
R450W450	3.23	0.22
R370W370	2.35	0.12
R320W320	2.37	0.15
R380W360	2.95	0.18
R280W280	1.65	0.12

R330W330	3.2	0.21
R270W270	3.77	0.22
R1580W180	4.56	0.24
R170W160	3.79	0.2
R130W110	4.31	0.21
R150W150	4.26	0.21
R220W220	4.38	0.21
R230W230	4.37	0.21
R100W100	4.37	0.21
R1610W1590	3.47	0.19
R210W210	4.73	0.23
R240W240	4.56	0.22
R480W480	5.14	0.24
R490W490	4.53	0.22
R1570W1570	2.3	0.12
R560W560	1.74	0.07
R1560W540	1.99	0.12
R570W570	2.28	0.15
R870W870	2.1	0.13
R910W910	2.48	0.12
R300W200	3.22	0.15
R310W310	1.53	0.05
R410W410	2.08	0.13
R260W260	3.49	0.18
R390W390	2.55	0.17
R400W400	1.67	0.09
R1680W1660	1.92	0.12
R650W640	2.25	0.17
R420W420	1.94	0.11

Table 9: Downstream HMS Model Tc Characteristics

Subbasin	Area (Sq. Mile)	Longest Flow Path (ft)	Overland Flow (ft)	Shallow Flow (ft)	Open Channel Flow (ft)	Total Tc (hrs)
R490W490	0.158	5,193	500	779	3,914	6.8
R500W500	9.18	45,686	500	6,853	38,333	39.6
R130W130	13.583	47,000	500	7,050	39,450	40.6
R600W600	3.504	19,984	500	2,998	16,487	18.7
R410W410	7.259	31,507	500	4,726	26,281	28.1
R230W230	3.408	22,515	500	3,377	18,638	20.8
R530W530	1.522	16,308	500	2,446	13,361	15.8
R560W560	5.474	20,615	500	3,092	17,022	19.2
R470W470	6.076	39,423	500	5,913	33,009	34.5
R590W590	10.79	57,152	500	8,573	48,079	48.9
R460W460	0.548	8,292	500	1,244	6,548	9.3
R370W370	0.652	8,728	500	1,309	6,919	9.6
R240W240	11.034	29,250	500	4,387	24,362	26.2
R540W540	2.99	19,199	500	2,880	15,819	18.1
R210W210	2.678	25,972	500	3,896	21,576	23.6
R190W190	8.619	42,501	500	6,375	35,626	37.0
R220W220	3.671	16,625	500	2,494	13,631	16.0

R250W250	4.704	34,194	500	5,129	28,565	30.3
R450W450	5.587	27,170	500	4,076	22,595	24.6
R510W510	9.999	31,083	500	4,662	25,920	27.7
R480W480	14.179	50,789	500	7,618	42,670	43.7
R520W520	9.735	29,036	500	4,355	24,180	26.1

Note: The following was assumed in the calculation of the Time of Concentration:

- Overland Flow Velocity = 0.05 fps
- Shallow Flow Velocity = 0.1 fps
- Open Channel Flow Velocity = 0.6 fps

Table 10: Upstream HMS Model Tc Characteristics

Subbasin	Area (Sq. Mile)	Longest Flow Path (ft)	Overland Flow (ft)	Shallow Flow (ft)	Open Channel Flow (ft)	Total Tc (hrs)
R460W460	4.743	33,485	500	5,023	27,962	29.7
R470W470	2.998	20,097	500	3,015	16,583	18.8
R1660W1640	5.931	39,068	500	5,860	32,708	34.2
R520W520	5.461	37,625	500	5,644	31,481	33.0
R610W610	3.286	33,028	500	4,954	27,574	29.3
R730W730	7.709	57,896	500	8,684	48,711	49.5
R600W600	5.587	42,518	500	6,378	35,640	37.0
R1640W1620	7.136	39,333	500	5,900	32,933	34.4
R1630W1610	1.618	16,518	500	2,478	13,540	15.9
R860W760	3.475	28,281	500	4,242	23,539	25.5
R830W830	2.424	16,635	500	2,495	13,639	16.0
R900W890	4.807	27,882	500	4,182	23,199	25.1
R1030W1030	3.543	26,067	500	3,910	21,657	23.7
R940W940	6.437	29,560	500	4,434	24,626	26.5
R1040W1040	2.832	13,557	500	2,034	11,023	13.5
R1060W1060	2.119	14,855	500	2,228	12,127	14.6
R1360W1360	12.875	44,125	500	6,619	37,006	38.3
R1130W1130	4.553	25,881	500	3,882	21,499	23.5
R1590W1580	3.531	29,124	500	4,369	24,255	26.1
R1550W1550	2.377	21,109	500	3,166	17,443	19.6
R1510W1510	1.199	12,324	500	1,849	9,975	12.5
R1120W1120	4.359	30,036	500	4,505	25,031	26.9
R1220W1220	2.368	18,604	500	2,791	15,314	17.6
R1270W1270	6.444	41,852	500	6,278	35,074	36.5
R930W930	5.342	30,410	500	4,562	25,349	27.2
R1080W1080	5.151	29,585	500	4,438	24,647	26.5
R1620W1600	0.478	7,228	500	1,084	5,643	8.4
R1670W1650	2.463	25,287	500	3,793	20,994	23.0
R1340W1340	11.567	49,720	500	7,458	41,762	42.8
R1010W1010	6.894	33,349	500	5,002	27,846	29.6
R660W660	18.738	76,178	500	11,427	64,252	64.3
R750W750	3.919	23,988	500	3,598	19,889	22.0
R770W770	4.234	30,371	500	4,556	25,315	27.2
R810W810	4.3	27,841	500	4,176	23,165	25.1
R790W790	2.321	25,515	500	3,827	21,187	23.2
R630W630	1.413	11,399	500	1,710	9,189	11.8
R620W620	4.384	20,105	500	3,016	16,589	18.8
R450W450	1.506	15,945	500	2,392	13,053	15.5
R370W370	2.913	24,410	500	3,661	20,248	22.3
R320W320	3.916	18,582	500	2,787	15,294	17.6
R380W360	5.06	22,980	500	3,447	19,033	21.2

R280W280	2.619	12,505	500	1,876	10,130	12.7
R330W330	4.615	35,435	500	5,315	29,620	31.3
R270W270	7.398	35,306	500	5,296	29,510	31.2
R1580W180	5.738	27,468	500	4,120	22,848	24.8
R170W160	2.58	16,770	500	2,516	13,755	16.1
R130W110	5.158	28,296	500	4,244	23,552	25.5
R150W150	2.952	21,559	500	3,234	17,825	20.0
R220W220	3.455	21,807	500	3,271	18,036	20.2
R230W230	5.854	35,171	500	5,276	29,395	31.0
R100W100	6.124	36,532	500	5,480	30,552	32.1
R1610W1590	5.617	24,690	500	3,704	20,487	22.5
R210W210	7.429	27,058	500	4,059	22,500	24.5
R240W240	4.865	17,694	500	2,654	14,540	16.9
R480W480	3.932	19,409	500	2,911	15,997	18.3
R490W490	5.392	29,461	500	4,419	24,542	26.4
R1570W1570	2.527	17,622	500	2,643	14,479	16.8
R560W560	3.165	25,249	500	3,787	20,962	23.0
R1560W540	2.528	23,205	500	3,481	19,225	21.3
R570W570	4.265	21,301	500	3,195	17,606	19.8
R870W870	1.952	17,195	500	2,579	14,116	16.5
R910W910	2.811	18,761	500	2,814	15,447	17.7
R300W200	7.716	33,702	500	5,055	28,147	29.9
R310W310	2.947	18,174	500	2,726	14,948	17.3
R410W410	4.803	33,651	500	5,048	28,103	29.8
R260W260	4.1	28,012	500	4,202	23,310	25.2
R390W390	3.802	25,679	500	3,852	21,327	23.4
R400W400	2.091	16,192	500	2,429	13,263	15.7
R1680W1660	3.085	22,986	500	3,448	19,038	21.2
R650W640	2.597	29,034	500	4,355	24,179	26.1
R420W420	5.342	31,486	500	4,723	26,263	28.1

Note: The following was assumed in the calculation of the Time of Concentration:

- Overland Flow Velocity = 0.05 fps
- Shallow Flow Velocity = 0.1 fps
- Open Channel Flow Velocity = 0.6 fps

The roughness values in the hydraulic model were not calibrated, as there was no stream gage data to calibrate to. However, a sensitivity test was conducted on this model which shows that the model is stable and thus large changes in roughness yield little changes in peak water surface elevations, as can be seen in Table 11. The water surface elevations in Table 11 are along the main Raymondville drain near the town of Raymondville. It should be noted that Table 11 depicts the changes in water surface elevation due to a change in roughness only. Changes to the flow-volume curves and hydrologic routing were not considered. Thus this sensitivity analysis is a conservative estimate. If the flow-volume curves were considered, they would lower frequency rates resulting in even less change in water surface elevation.

Table 11: Affects of changes in roughness on water surface elevation.

Frequency Event	WS EL (ft) with Final Selected n-values	WS EL (ft) with n-values increase by 30%	WS EL (ft) with n-values decrease by 30%	WSEL Diff. (ft) for n-values increased by 30%	WSEL Diff. (ft) for n-values decreased by 30%
2 yr	25.66	26.70	24.15	1.04	1.51
5 yr	28.45	29.29	27.28	0.84	1.17
10 yr	29.87	30.27	28.82	0.4	1.05
25 yr	30.09	30.49	29.92	0.4	0.17
50 yr	30.79	31.12	30.31	0.33	0.48
100 yr	31.31	31.60	31.04	0.29	0.27
250 yr	31.80	31.95	31.61	0.15	0.19
500 yr	32.06	32.21	31.96	0.15	0.1

The lack of sensitivity in the RAS model can be associated with the fact that the watershed is very wide and flat, and thus it would take a tremendous volume of water to cause a significant increase in the water surface elevation for events that are not contained in the main drain. This effect of water spreading out over miles of flat terrain can be seen on plots from the USACE report on Hurricane “Beulah” of 1967, which shows a large portion of Willacy County under water, see Figure 9.

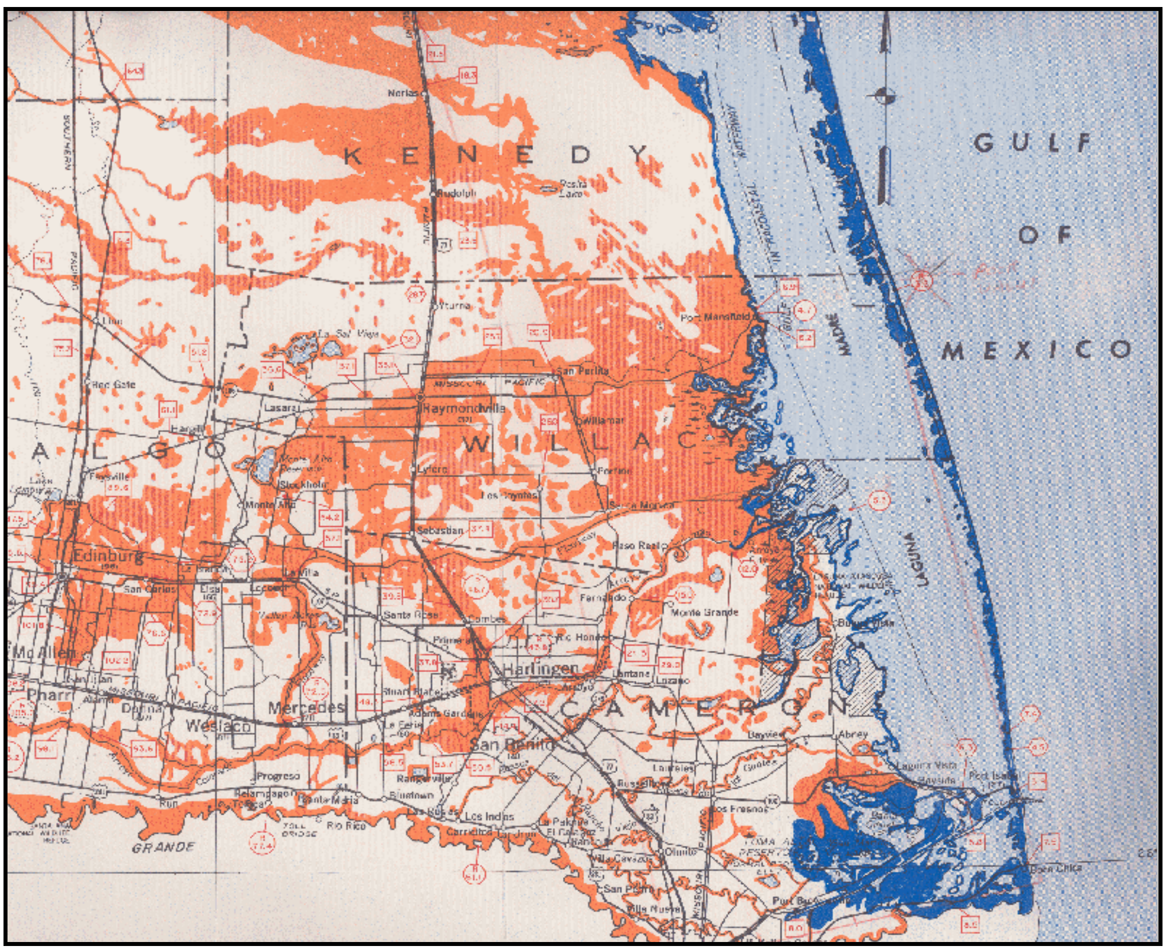

Figure 9: Flooding from Hurricane “Beulah” of 1967 (*Report on Hurricane “Beulah”, US Army Crops of Engineers, 1968*).

Diversion Analysis
There are two important flow junctions located on the Raymondville Drain, see Figure 10. The first is the junction near La Sal Vieja and the second is the junction to the South Hargill drain. Both junctions feature control structures consisting of multiple culverts with sluice gates. The operating criteria for these sluice gates are not known. The S&B model treats La Sal Vieja as a reservoir with discharge to Raymondville Drain only occurring if the reservoir reaches a pool elevation of 41 feet. This is approximately the elevation of the top of the sluice gate structure, a condition that is never reached for any of the events modeled. The model does not consider the case where floods reverse back into the lake thereby reducing flow to the Raymondville Drain. No adjustments were made to this assumption for the SWG analysis. Testing suggests that flooding at the town of Raymondville would be decreased if the sluice gates were removed and floods were allowed to backflow into the lake. For instance, the 10-year flood would be reduced by over half a foot.

The second special flow junction is the South Hargill Junction. This junction is not coded in the S&B HMS model. Thus, the model assumes that flow is neither lost nor gained. This was judged to be a reasonable assumption due to the uncertainty of the control structure operations and the likelihood that both flow paths would be at full capacity during significant floods.

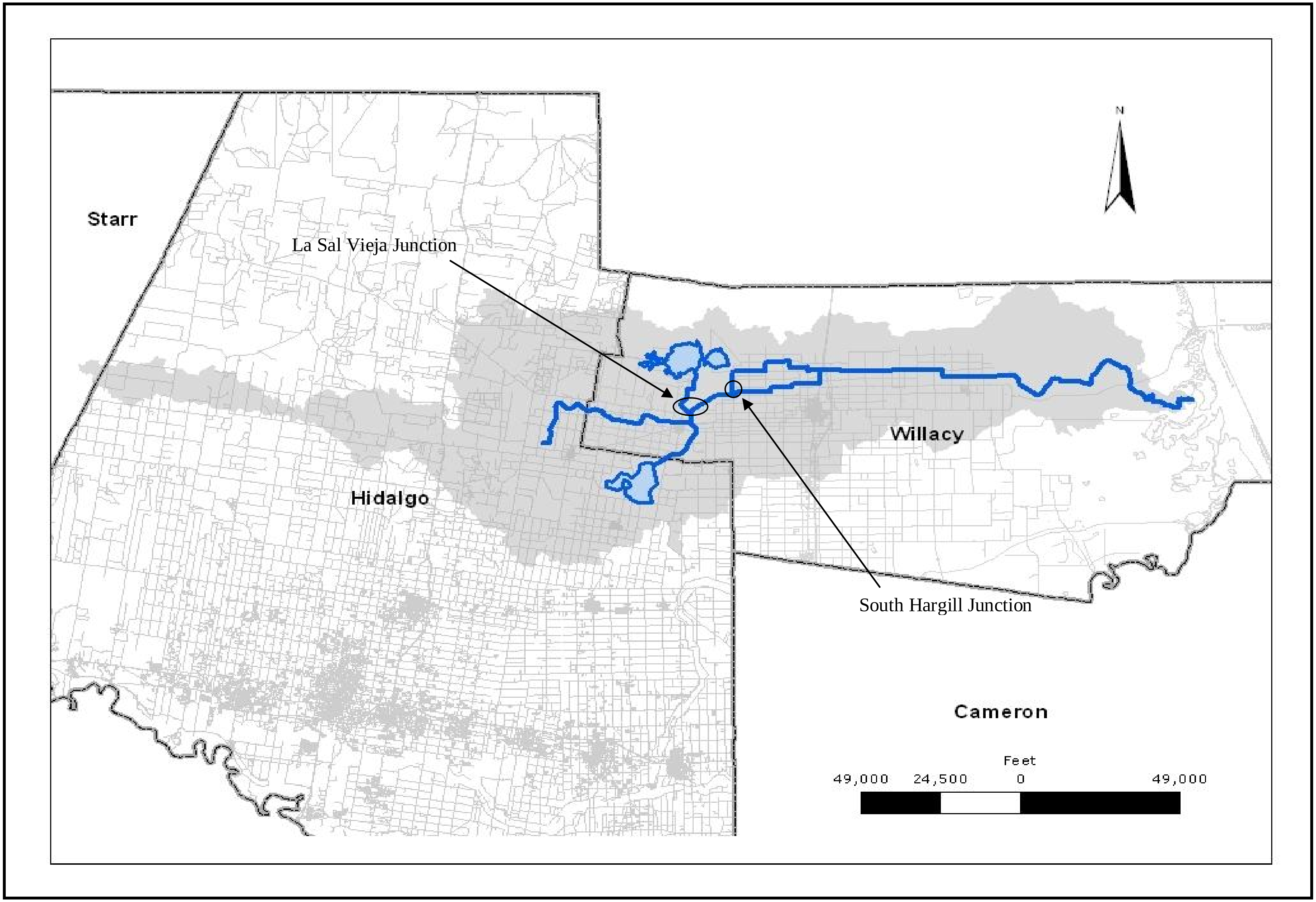

Figure 10: Junction Locations

Risk and Uncertainty Analysis
Flow frequency and stage-discharge relationships from the hydrologic models must be imported into the economics model (FDA) for computation of damages. The FDA model requires uncertainty functions for both relationships. Derivations of each are described in the following paragraphs.

Derivation of Discharge Uncertainty - The uncertainty of flow frequency results can be derived using two approaches. When the flow frequency values are thought to fit a Log Pearson III distribution, the uncertainty can be derived analytically from the mean, standard deviation, skew, and representative record length. Conversely, the order statistics approach is preferred for deriving uncertainty when the log Pearson distribution is not applicable. The order statistics method was adopted because the Raymondville Drain watershed is influenced by regulation in the form of irrigation canals, detention ponds, and many diversions. FDA performs the derivations, but an equivalent record length is required. The equivalent record length was selected using guidance from Table 4-5 of EM 1110-2-1619. A value of 13 years was selected for current conditions. There were two reasons for selecting 13 years, first there was a rainfall-runoff-routing model that contained many handbook or textbook model parameters, and the same model was roughly calibrated to a few events.

Derivation of Stage Uncertainty - The uncertainty of computed flood stages can be attributed to the natural variability of the stream and the hydraulic modeling inaccuracies. Guidance is provided in EM 1110-2-1619 for estimating and combining both components.

Natural variations include such factors as seasonal vegetation changes, debris constrictions, and unsteady flow effects. Equation 5-5 from EM 1110-2-1619 was used to compute the standard deviation of stage uncertainty due to these natural effects. Values were computed for three reaches along the drain with the results averaging to 0.2 ft as shown in Table 12. Figure 5-3 of the EM was used to estimate upper bounds. Upper bound values and adopted values for natural variations are also shown in Table 12.

Table 12
Stage Uncertainty due to Natural Variations

Computed with Equation 5-5, EM 1110-2-1619

Reach	I bed	A basin (sq. km)	H range (m)	Q 100 (cms)	Snatural (m)	Snatural (ft)
Raymondville East Side	3.5	1295	0.16	120	0.062	0.2
Raymondville West Side	3.5	1295	0.16	120	0.062	0.2
San Perlita	3.5	1399	0.15	123	0.062	0.2

Upper Bound From Figure 5-3 EM 1110-2-1619

Reach	Stream Slope (ft/ft)	Upper Bound Snatural (ft)
Raymondville East Side	0.0001	2.5
Raymondville West Side	0.0001	2.5
San Perlita	0.0001	2.5

Adopted Values (Natural Variation)

Reach	Adopted Snatural (ft)
Raymondville East Side	0.2
Raymondville West Side	0.2
San Perlita	0.2
Average	0.2

Hydraulic modeling inaccuracies include errors in estimating roughness values, errors in cross section topography, and errors in defining effective flow area. Minimum values were estimated from Table 5-2 of the EM. The cross-sections for the Raymondville Drain hydraulic model were based on Aerial Lidar Data for the initial 1,500 ft on each side of the channel, and digital terrain data (equivalent to a 5-foot contour map) for the remainder of the cross-sections. Manning's reliability was judged to be fair due to the fact that the hydraulic model is very stable. However, there is only one source of high water marks.

As an additional measure of modeling uncertainty, a series of tests were conducted to determine the sensitivity of the model to the roughness coefficient, Manning's n. The adopted roughness values were increased and decreased by 50% and the resultant profile differences were tabulated. Taking the stage difference between the upper and lower roughness values to be of "reasonable bounds", the standard deviation was then estimated as the difference divided by 4. Table 13 shows the resultant modeling uncertainty values and the adopted values.

Table 13
Stage Uncertainty due to modeling limitations (Table 5-2, EM 1110-2-1619) and from Roughness Sensitivity Testing

Reach	Model Limitations from EM	Roughness Sensitivity from HEC-RAS Testing		Adopted Smodel (ft)
	Smodel Min (ft)	Prof Diff (ft)	Srough (ft)	
Raymondville East Side	0.7	1.99	0.50	0.50
Raymondville West Side	0.7	2.08	0.52	0.52
San Perlita	0.7	1.65	0.41	0.41
Average	0.7	-	-	0.48

The combined stage uncertainty was determined by combining the natural variability and the modeling uncertainty into one value using equation 5-6 from the EM. Final value is 0.52 feet, as seen on Table 14.

Table 14
Stage Uncertainty Combined Total from Equation 5-6, EM 1110-2-1619

Reach	Snatural (ft)	Smodel (ft)	Stotal (ft)
Average	0.2	0.48	0.52

Final Results

The final results of this analysis can be seen in the following table and figure. Table 15 displays the eight frequencies with corresponding stages and flows at the three index locations of West Raymondville, East Raymondville, and San Perlita. Figure 11 shows the water surface profile for each frequency at these same index locations.

Table 15: Stage and Flow frequency for the three index locations

Index #1: Town of Raymondville, West of Railroad

Frequency (yr)	Stage (ft)	Flow (cfs)
2	25.95	412
5	29.27	1233
10	30.79	1873
25	31.19	2649
50	31.5	3498
100	31.77	4313
250	32.11	5321
500	32.32	6180

Index #2: Town of Raymondville, East of Railroad

Frequency (yr)	Stage (ft)	Flow (cfs)
2	25.01	464
5	27.73	1088
10	29.11	1731
25	29.85	2561
50	30.31	3398
100	30.7	4229
250	31.08	5242
500	31.19	6096

Index #3: Town of San Perlita

Frequency (yr)	Stage (ft)	Flow (cfs)
2	17.51	754
5	19.34	1226
10	20.32	1801
25	20.69	2566
50	20.97	3428
100	21.09	4346
250	21.22	5398
500	21.57	6250

Appendix B
Interim Technical Memorandums (CSE and S&B)

Texas Firm No. 1582

Technical Memorandum

HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT
Design and Expansion of the Raymondville Drain
Contract No. C-10-164-04-20

Date: 14 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Draft H&H Assurance Review of September 7, 2011

The following technical memorandum is in response to the comments received from Deren Li, PE of Civil Systems Engineering, Inc. regarding the hydrologic and hydraulic base models for the Raymondville Drain Project and the "Preliminary Engineering Report – Alternatives Analysis thru Hydrologic & Hydraulic Analysis For the Proposed Typical Sections, From Edinburg Lake to the Guerra Detention Facility", dated 5-25-2011 (Supplemented 6-9-2011)". S&B received comments on September 7, 2011 regarding CSE's draft review of the models and document.

A teleconference was held on September 12, 2011 between S&B and CSE regarding the comments. This Technical Memorandum provides written documentation of S&B's response to the comments and additional information required for clarification through the teleconference. Below is a listing of the CSE comments and S&B's formal response:

HEC-HMS Hydrologic Modeling

CSE Comment 1: TP-40/TP 49 rainfall data is used in the HEC-HMS hydrologic modeling analysis. The rainfall depth data based on TP-40/TP-49 is systematically and significantly higher than the rainfall depth data based on the Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas (USGS in cooperation with the Texas Department of Transportation, 2004). With consideration of the USGS/TxDOT Atlas is based on longer precipitation records and better methodology, rainfall data from the USGS/TxDOT Atlas is recommended for the study.

S&B Response: The rainfall depth data was based on TP40/TP49 since areal adjustment data for all storm durations are only known for TP40/TP49. Additionally, rainfall depth data was available for the 10-day storm duration. Furthermore, according the USGS document, this area contained "questionable depth-duration frequency values" with substantial inconsistencies. It was noted that in this region, a large area contained "depths for the 12-hour duration and large recurrence intervals (50 to 500 years) that were larger than the depths for the 1-day duration." The use of TP40/TP49 in lieu of the more modern USGS Atlas was discussed with the USACE, and the agreement was reached during the development of the pre-project hydrology to utilize TP40/TP49 data. This was documented in the USACE white paper titled "Raymondville Drain Pre-Project Conditions Report," April 2006 (USACE RDPR).

As shown in the above tables, once the areal reductions are applied to the precipitation values, the differences are reduced. For the shorter duration events, the precipitation values derived from TP40/TP49 are less than the values obtained from USGS.

Table 1: Comparison Without Areal Reductions

	6-hr		12-hr		1-day		7-day		10-day	
Frequency	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49
50	6.40	6.90	7.25	8.15	8.00	9.58	11.40	14.59	N/A	16.15
100	7.40	7.68	8.50	9.41	9.19	11.07	12.92	16.35	N/A	18.00
250	8.9	8.91	10.5	10.74	11.00	12.65	15.80	18.89	N/A	20.84
500	10	9.76	12.0	11.79	12.50	13.90	17.50	20.75	N/A	22.90

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Technical Memorandum

HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT
Design and Expansion of the Raymondville Drain
Contract No. C-10-164-04-20

Date: 14 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Draft H&H Assurance Review of September 7, 2011

Table 2: Comparison With Areal Reductions (assume 250 sq mile storm area)

	6-hr		12-hr		1-day		7-day		10-day	
Frequency	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49
50	6.40	5.38	7.25	7.25	8.00	8.81	11.40	13.86	N/A	15.67
100	7.40	6.22	8.50	8.37	9.19	10.18	12.92	15.53	N/A	17.46
250	8.9	7.92	10.5	9.35	11.00	11.64	15.80	17.95	N/A	20.21
500	10	8.9	12.0	10.49	12.50	12.72	17.50	19.71	N/A	23.87

CSE Comment 2: There are no HEC-HMS hydrologic models developed to compute future (2061) conditions peak flows and hydrographs. It is not clear how the diversion/intercepted flows were derived without future conditions HECHMS models.

S&B Response: No separate hydrology model is required for the development of the projected peak flow rates. The Texas Water Development Board economic growth factors of were initially utilized as a starting point to determine the level of future development for this area. The impervious cover for several watersheds was increased as a test to determine the average peak flow rate increase. It was found that the on average, the peak flow rate increased between 28%-35% using the growth factors from the Texas Water Development Board. As a result a factor of 1.35 was selected to calculate the peak flow rates for the projected year 2061.

CSE Comment 3: Modified Pulse Method was used for flood routing along various channel reaches. Based on the HEC-HMS model inputs, for all channel reaches that uses Modified Pulse Method, Subreach is assumed "1". This probably overestimates the storage effect for some of the reaches. Reach R1630 is a typical example. There is a 25% flow reduction through this reach.

S&B Response: A subreach value of "1" is valid in all instances that it is used. Per the HEC-HMS Technical Reference Manual this value "is used commonly for routing through ponds, lakes, wide, flat floodplains, and channels in which the flow is heavily controlled by downstream conditions."

CSE Comment 4: Some of the storage-outflow relations from HEC-RAS modeling need to be revisited. It was noticed in the HECRAS model, storage is overestimated (see following cross sections with water surface elevations).

S&B Response: Storage-outflow tables were taken directly from HEC-RAS. Many of these reaches contain data that was obtained directly from the models provided to S&B by the USACE. There was significant coordination with the USACE during 2007 to ensure that storage values utilized by S&B corresponded with the values in the USACE models. Due to the extremely flat terrain found in these areas, there will be significant storage in the overbanks once the water surface has risen above the banks of the channel.

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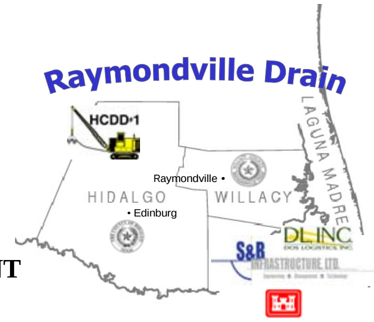


Technical Memorandum

HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT
Design and Expansion of the Raymondville Drain
Contract No. C-10-164-04-20

Date: 14 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Draft H&H Assurance Review of September 7, 2011

CSE Comment 5: Percent Imperviousness parameter is not explicitly modeled in the HEC-HMS model. It is not clear whether it was considered in the CN and time of concentration calculations.

S&B Response: The percent imperviousness parameter was not utilized in the hydrologic model. Instead, the composite CN based on land use and hydrologic soil group was utilized. The impervious cover is included in the final CN values, thus no additional impervious cover percentages should be added separately to the HEC-HMS model.

CSE Comment 6: The NRCS standard initial loss method of 0.2S (potential maximum retention) is used in the HEC-HMS model. Since the 10-day storm event is assumed for the study, initial loss has very minor impact to the peak flows. The average initial loss used in the model is approximately 2 inches. Even increase to 5 inches, there is very minor changes in peak flows.

S&B Response: The Initial/Constant Loss method is most appropriate for long duration storm events. For instance, the NRCS Curve Number Method assumes that after the initial loss, all losses go to zero. As a result, the NRCS Curve Number Method should not be used for storms with significant duration. That is why S&B, in conjunction with the USACE, chose to use the Initial/Constant Loss method. It should be noted that the composite curve numbers were utilized to determine the initial loss value. These curve numbers were adjusted to AMC I (dry condition) prior to calculating the initial loss. This resulted in higher initial loss values. Note that this methodology was agreed to with the USACE and previously documented in the USACE white paper titled "Raymondville Drain Pre-Project Conditions Report," April 2006. (USACE RDPR)

CSE Comment 7: It appears that the overall peak flows are significantly high. At Station 64591, the 100-year peak flow is 12,494 cfs in this study, which is 3 times of the FEMA effective 100-year peak flow of 4,175 cfs. With consideration of the difference of the 24-hr and 10-day storm event, there is a maximum of 10 percent increase. It appears that the rainfall depth makes the most of the differences.

S&B Response: The hydrologic model was prepared in accordance with guidelines agreed to with the USACE. The NRCS curve numbers were adjusted to AMC I. During this process S&B kept the adjusted CN's below 60 to account for depressions and storage found throughout the watershed. The unit hydrographs were flattened to account for the flat terrain found in the watershed. The time of concentration calculations were conducted using the methodology recommended by the USACE. S&B went further by calculating average velocities for each subbasin as opposed to using a general approximation. As a result the USACE adjusted the values in their model to match the values provided by S&B. This was documented in USACE's response to S&B in 2007.

CSE Comment 8: A constant Peaking Rate Factor of 150 is used in calculating the Unit Hydrograph. It seems variable PRF should be used with the consideration of the subbasin physical conditions such as slopes and depressions.

S&B Response: Due to the generally flat topography of these watersheds as compared to the average U.S. watershed, the PRF was adjusted from 484 to 150. The value of 150 was chosen to properly model the slopes and depressions found within these watersheds. This value was discussed and agreed to during negotiations with USACE during the development of the pre-project hydrology. This methodology was agreed to with the USACE and previously documented in the USACE white paper titled "Raymondville Drain Pre-Project Conditions Report," April 2006. (USACE RDPR)

CSE Comment 9: Detailed documentation is needed to clearly discuss the relationship between the area reduction calculation using spreadsheets and HEC-HMS modeling results.

S&B Response: Due to inherent limitations with the HEC-HMS program, manual adjustments were needed for each storm event to provide valid results at each junction node. For each storm event, multiple runs were created for storm area values from 0 square miles to 400 square miles in 50 square mile intervals. The peak flow rate for each simulation was recorded. Subsequently, the peak flow rate was calculated based on the actual watershed area at each node. It was this peak flow rate that was subsequently input into the HEC-RAS hydraulic model. This methodology was presented to, and agreed with by the USACE. Further documentation will be provided in the hydraulics section of the flood damage assessment report.

HEC-RAS Hydraulic Modeling

CSE Comment 1: In the base HEC-RAS model, at Station 64591, the 100-year peak flow is 12,494 cfs. In the Alt 1B model, the 100-year peak flow is 9,089 cfs. Where the 9,089 came from?

S&B Response: The peak flow rates for the Alt 1B HEC-RAS models were developed using separate HEC-HMS models with manual calculation of the storm area reduction. These models were provided with the base HEC-HMS models.

CSE Comment 2: As discussed earlier, there is no future conditions HEC-HMS models developed for the project. It is understood there is factor of 1.35 used to obtain future conditions peak flows. What is the justification of 1.35?

S&B Response: As stated earlier, no separate hydrology model is required for the development of the projected peak flow rates. A factor of 1.35 was applied to the HEC-RAS flow data to account for economic growth based on data available from the Texas Water Development Board during the 50 year study period.

CSE Comment 3: Why only 645 cfs is used in the RVD HYD Model Alt 1B for the 100-year (2061)? The Preliminary Engineering Report states a 100-year 1,390 cfs flow is proposed diverted. Alternative diversion flows should be considered to optimize the design of the diversion weir structure, channel, and detention basins.

S&B Response: The RVD HYD Model Alt 1B is based on the 10-day storm event as required by USACE for development of the Flood Damage Assessment (FDA) needed to obtain federal funding. The Preliminary Engineering Report utilizes the 24-hr storm duration and was calibrated to the peak flows found in the Letter of Map Revision for Hidalgo County dated, 05-17-2001. The difference is due to the amount of runoff that is intercepted by the diversion channel. Once the FDA is finalized, and the design storm event is selected, the design of the diversion weir structure, channel and detention basin can be further optimized.

CSE Comment 4: The combined peak flow (diversion/interception) does not reflect the 1.35 factor.

S&B Response: Based on the comment above, it is unclear where in the model it is believed that the 1.35 factor was not applied. S&B is assuming that this comment is referring to the actual diversion weir. For the purposes of our initial analysis, the amount of flow diverted was kept constant. As a result the combined diversion/interception peak flow rate for the Year 2011 and Year 2061 will not directly correspond to the 1.35 factor.

CSE Comment 5: With consideration of the very flat nature of the drainage channels, the HEC-RAS Unsteady Flow Module is recommended for this study. The hydraulic routing technique within the HEC-RAS Unsteady Flow Module is based on the solution of the partial differential equations (dynamic wave equations) of unsteady channel flow. The hydraulic routing method provides the most accurate solutions calculating an outflow hydrograph while considering the effects of channel storage and wave shape. The Modifed Puls hydrologic routing method does not work properly when the channel slope is very flat (< 3 ft/mile). The storage-discharge relations calculated using steady flow profiles produce errors when out-of-bank flows occur over wide floodplains.

S&B Response: Due to the complicated nature of the existing watershed, S&B does not feel that the HEC-RAS Unsteady Flow modeling is appropriate. All storage routing and flow attenuation was calculated using the methods found in HEC-HMS. The HEC-RAS steady state model was utilized to prepare the storage-outflow curves that were used in HEC-HMS. This methodology produced individual hydrographs that were attenuated through their corresponding reaches. It was these attenuated peak flows that were input into the HEC-RAS steady flow model. The methodology of using HEC-HMS to provide for the flow attenuation and storage routing within the watershed was agreed to with the USACE during the pre-project conditions phase of the analysis. This same above methodology was used in the steady flow HEC-RAS and HMS models that were provided by the USACE to S&B for the Willacy County portion of the Raymondville Drain. Additionally, HEC-RAS unsteady flow models are inherently unstable, especially in watersheds as complex as the Raymondville Drain and the North Main Drain. Per the HEC-RAS Hydraulic Reference Manual,

> "In practice, other factors may also contribute to the non-stability of the solution scheme. These factors include dramatic changes in channel cross-sectional properties, abrupt changes in channel slope, characteristics of the flood wave itself, and complex hydraulic structures such as levees, bridges, culverts, weirs, and spillways. In fact, these other factors often overwhelm any stability considerations…"

Concerning the use of Modified Puls, this methodology is specifically useful in areas with wide floodplains and where there is significant backwater that will influence the discharge hydrograph. Additionally, this method is valid from slopes ranging from 10 to 2 ft/mile. (Chapter 9, EM1110-2-1417)

CSE Comment 6: US 281 crossing structure seems oversized.

S&B Response: The US 281 culvert structure was sized for use in determining alternatives for the FDA. The preliminary design was selected to provide a headloss through the structure that closely mimics the proposed bridge solution. Once the design storm event has been selected, the detailed design will be performed to provide an efficient solution that complies with TxDOT design requirements and meets the design constraints set forth by the HCCD#1.

CSE Comment 7: Channel slope modifications are required to optimize the channel design, especially upstream reaches.

S&B Response: To reiterate, once the flood damage assessment is finalized, and the design storm selected, the final design of the diversion channel will be optimized. This preliminary channel geometry is being utilized to determine and evaluate alternatives during this feasibility phase for the entire Raymondville Drain and North Main Drain watersheds.

CSE Comment 8: Several reaches show 10+ feet of freeboard.

S&B Response: This appears to be similar to comment #7. Please see previous response.

S&B Additional Note Regarding HEC-RAS Modeling: A summary of hydrologic and hydraulic models will be included as documentation of the individual hydrologic and hydraulic models used for the pre-project and post-project conditions modeling. This summary will be included in the appendices of the final H&H report. This summary will serve as a guide and to provide clarification for which computer model was utilized for the pre-project and post-project conditions H&H modeling.

Respectfully Submitted and Released For Planning Purposes Only Under the Authority of:

______________________ **Texas PE # 88453**

Andres Cardenas, PE

Date: 09/14/11

Final Technical Memorandum

To: Sharlotte L. Teague, P.E.

From: Deren Li, PhD, P.E., D.WRE, CFM

Date: September 22, 2011

Re: H&H Assurance Review for the Raymondville Drain Flood Control Project
S&B Project No: U1445; Subcontractor No: U1445-Z003

Civil Systems Engineering Inc. (CSE) has completed our H&H Assurance Review for the H&H reports and models prepared by S&B Infrastructure Ltd. for the Raymondville Drain Flood Control Project. The H&H Assurance Review was performed in accordance with the scope of services defined in the Agreement of Professional Services between Civil Systems Engineering Inc. and S&B Infrastructure, Ltd., signed on September 16, 2011.

CSE prepared a Draft Technical Memorandum dated September 7, 2011 (Attachment A) to summarize the findings and comments to the reports and models prepared by S&B. On September 7, 2011, S&B prepared a Technical Memorandum (Attachment B) dated September 7, 2011 to respond to CSE's comments. Explanations and support documents were provided for each comment. After review of S&B's Technical Memorandum, CSE further discussed Comments 2 and 7 in an email to S&B dated September 19, 2011 (Attachment C), both of these comments are related to the magnitudes of computed peak flows for the project. We believe these two comments address the more influential factors with regards to the modeling analysis among the previously discussed comments in CSE's September 7 draft technical memorandum. On September 20, 2011, S&B provided explanations and justifications of the computations of the project peak flows (Attachment D). With further review of S&B's September 20's response, we still have concerns for the very high peak flows for the project.

A comparison table (Attached E) is presented to further demonstrate our concerns. First, comparison is made between the Lag Time values computed by S&B and the SCS Equation (North Main Drain subbasins were used for this comparison). As shown in column LAG(SCS)/LAG(S&B), the LAG values based on SCS lag equation are 1 to 8 times of the LAG (S&B) values. The ratios are reflected in the Unit Hydrograph peak flows (for PRF 150). Also comparison is made for Unit Hydrograph Peak flows between Qp based on PRF 150 and S&B's lag values and Qp based on standard PRF 484 and SCS lag equation. Column Qp(S&B)/Qp(SCS484) shows that even with the much lower PRF 150 for the project, for most of the subbasins, the computed peak flows are much greater than peak flows based on the standard PRF 484 (3.2 times of 150).

It should be noted that the above comparison results do not disqualify the Tc or travel time METHOD used for this study. However, estimates of parameters need to be revised to ensure that lag time values and peak flows are more representative of the local watershed conditions.

Attachment A

Draft Technical Memorandum

To: Sharlotte L. Teague, P.E.

From: Deren Li, PhD, P.E., D.WRE, CFM

Date: September 7, 2011

Re: H&H Assurance Review for the Raymondville Drain Flood Control Project
S&B Project No: U1445; Subcontractor No: U1445-Z003

This Draft Technical Memorandum was prepared to summarize the findings and recommendations based on our preliminary H&H Assurance Review for the H&H reports prepared by S&B Infrastructure Ltd. for the Raymondville Drain Flood Control Project. The H&H Assurance Review was performed in accordance with the scope of services defined in the Agreement of Professional Services between Civil Systems Engineering Inc. and S&B Infrastructure, Ltd., signed on September 16, 2011.

HEC-HMS Hydrologic Modeling

1. TP-40/TP 49 rainfall data is used in the HEC-HMS hydrologic modeling analysis. The rainfall depth data based onTP-40/TP-49 is systematically and significantly higher than the rainfall depth data based on the Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas (USGS in cooperation with the Texas Department of Transportation, 2004). With consideration of the USGS/TxDOT Atlas is based on longer precipitation records and better methodology, rainfall data from the USGS/TxDOT Atlas is recommended for the study. A comparison of the rainfall depth is made in the following table.

Frequency	1-Day		7-Day	
	USGS	TP40/TP49	USGS	TP40/TP49
50	8.00	9.70	11.40	14.90
100	9.19	11.20	12.92	16.50
250	11.00	12.79	15.80	19.22
500	12.50	14.06	17.50	21.12

2. There are no HEC-HMS hydrologic models developed to compute future (2061) conditions peak flows and hydrographs. It is not clear how the diversion/intercepted flows were derived without future conditions HEC-HMS models.

3. Modified Pulse Method was used for flood routing along various channel reaches. Based on the HEC-HMS model inputs, for all channel reaches that uses Modified Pulse Method, Subreach is assumed "1". This probably overestimates the storage effect for some of the reaches. Reach R1630 is a typical example. There is a 25% flow reduction through this reach.

Modified Puls Routing [NMD_AMC1_FIN]

Show Elements: All Elements

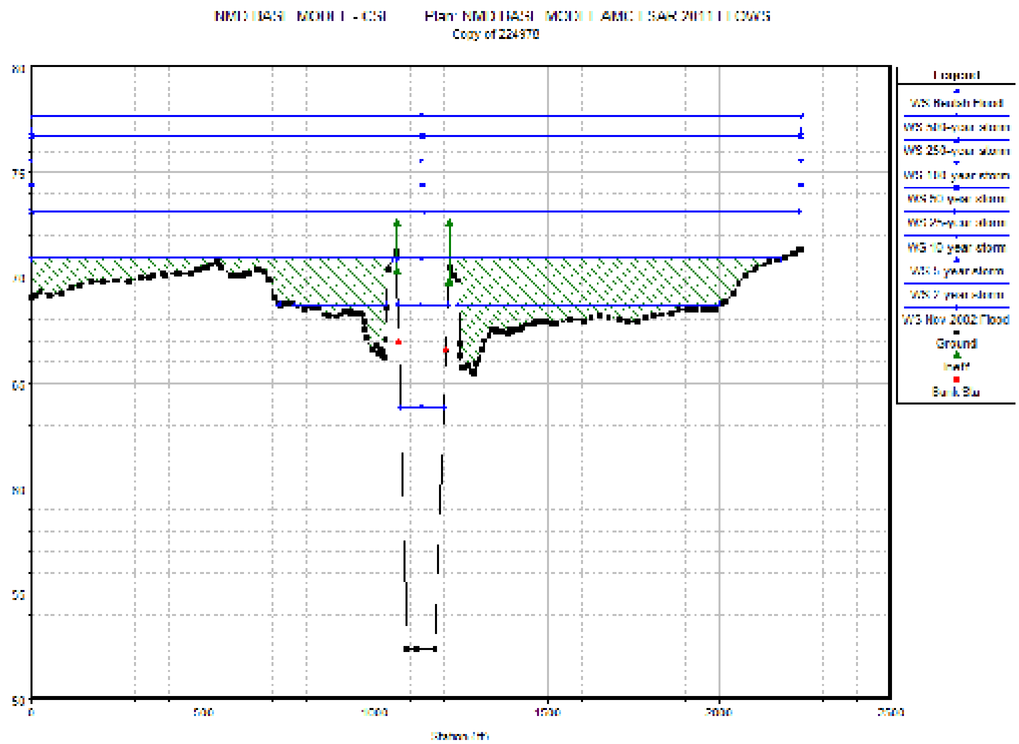
Reach	Stor-Dis Function	Subreaches	Initial
R1190	R1190	1	Inflow = Outflow
R1280	R1280	1	Inflow = Outflow
R1290	R1290	1	Inflow = Outflow
R1450	R1450	1	Inflow = Outflow
R1630	R1630	1	Inflow = Outflow
R1790	R1790	1	Inflow = Outflow
R1800	R1800	1	Inflow = Outflow

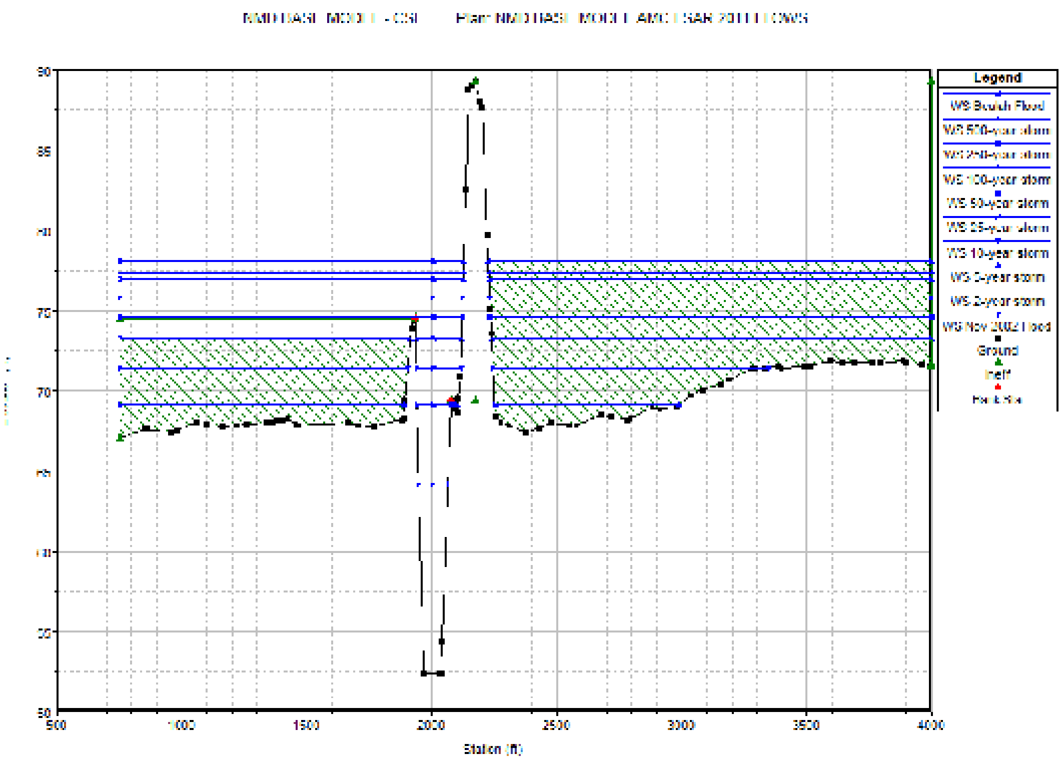
Modified Puls Routing [RVDBASEMODEL]

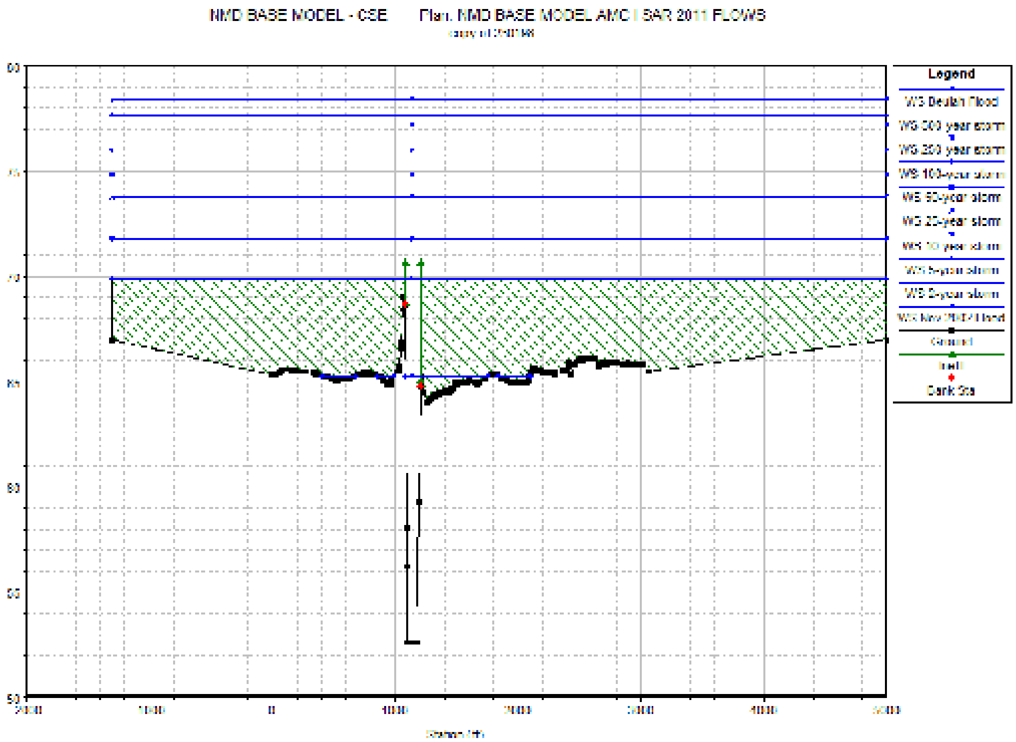
Show Elements: All Elements

Reach	Stor-Dis Function	Subreaches	Initial
R1560	R1560(RVDProj)	1	Inflow = Outflow
R1570	R1570(RVDProj)	1	Inflow = Outflow
R310	R310(RVDProj)	1	Inflow = Outflow
R390	R390(RVDProj)	1	Inflow = Outflow
R400	R400(RVDProj)	1	Inflow = Outflow
R550	R550(RVDProj)	1	Inflow = Outflow
R630	R630(RVDProj)	1	Inflow = Outflow
R770	R770(RVDProj)	3	Inflow = Outflow
R780	R780(RVDProj)	2	Inflow = Outflow
Reach-1	Reach-1(RVDProj)	1	Inflow = Outflow

4. Some of the storage-outflow relations from HEC-RAS modeling need to be revisited. It was noticed in the HEC-RAS model, storage is overestimated (see following cross sections with water surface elevations).


WS Nov 2002 Flood
Ground


Legend
WS 100-year storm
WS 50-year storm
WS 25-year storm
WS 10-year storm
WS 5-year storm
WS 2-year storm
WS Nov 2002 Flood
Ground
Station (ft)



5. Percent Imperviousness parameter is not explicitly modeled in the HEC-HMS model. It is not clear whether it was considered in the CN and time of concentration calculations.

Initial Constant Loss [NMD_AMC1_FIN]

Show Elements: All Elements

Subbasin	Initial Loss (IN)	Constant Rate (IN/HR)	Impervious (%)
R1090W1090	1.509	0.27	0.0
R1140W1140	1.636	0.29	0.0
R1150W1150	1.922	0.29	0.0
R1160W1160	2.545	0.3	0.0
R1180W1180	1.636	0.30	0.0
R1190W1190	1.636	0.28	0.0
R1200W1200	1.922	0.29	0.0
R1210W1210	2.878	0.32	0.0
R1230W1230	1.636	0.30	0.0
R1240W1240	3.128	0.3	0.0
R1250W1250	1.279	0.21	0.0
R1280W1280	1.125	0.18	0.0
R1290W1290	1.774	0.27	0.0
R1310W1310	1.390	0.23	0.0
R1320W1320	1.704	0.29	0.0
R1330W1330	1.636	0.29	0.0
R1380W1300	1.704	0.30	0.0

Initial Constant Loss [RVDBASEMODEL]

Show Elements: All Elements

Subbasin	Initial Loss (IN)	Constant Rate (IN/HR)	Impervious (%)
R100W100	3.000	0.28	0.0
R1010W1010	1.509	0.29	0.0
R1030W1030	1.774	0.30	0.0
R1040W1040	1.704	0.29	0.0
R1060W1060	1.509	0.27	0.0
R1080W1080	1.636	0.29	0.0
R1100W1100	1.125	0.18	0.0
R1110W1110	1.509	0.27	0.0
R1120W1120	1.509	0.28	0.0
R1130W1130	2.545	0.29	0.0
R1220W1220	1.922	0.29	0.0
R1270W1270	1.774	0.30	0.0
R130W110	2.255	0.28	0.0
R1340W1340	1.636	0.30	0.0

6. The NRCS standard initial loss method of 0.2S (potential maximum retention) is used in the HEC-HMS model. Since the 10-day storm event is assumed for the study, initial loss has very minor impact to the peak flows. The average initial loss used in the model is approximately 2 inches. Even increase to 5 inches, there is very minor changes in peak flows.
7. It appears that the overall peak flows are significantly high. At Station 64591, the 100-year peak flow is 12,494 cfs in this study, which is 3 times of the FEMA effective 100-year peak flow of 4,175 cfs. With consideration of the difference of the 24-hr and 10-day storm event, there is a maximum of 10 percent increase. It appears that the rainfall depth makes the most of the differences.
8. A constant Peaking Rate Factor of 150 is used in calculating the Unit Hydrograph. It seems variable PRF should be used with the consideration of the subbasin physical conditions such as slopes and depressions.
9. Detailed documentation is needed to clearly discuss the relationship between the area reduction calculation using spreadsheets and HEC-HMS modeling results.

HEC-RAS Hydraulic Modeling

1. In the base HEC-RAS model, at Station 64591, the 100-year peak flow is 12,494 cfs. In the Alt 1B model, the 100-year peak flow is 9,089 cfs. Where the 9,089 came from?

2. As discussed earlier, there is no future conditions HEC-HMS models developed for the project. It is understood there is factor of 1.35 used to obtain future conditions peak flows. What is the justification of 1.35?

3. Why only 645 cfs is used in the RVD HYD Model Alt 1B for the 100-year (2061)? The Preliminary Engineering Report states a 100-year 1,390 cfs flow is proposed diverted. Alternative diversion flows should be considered to optimize the design of the diversion weir structure, channel, and detention basins.

4. The combined peak flow (diversion/interception) does not reflect the 1.35 factor.

5. With consideration of the very flat nature of the drainage channels, the HEC-RAS Unsteady Flow Module is recommended for this study. The hydraulic routing technique within the HEC-RAS Unsteady Flow Module is based on the solution of the partial differential equations (dynamic wave equations) of unsteady channel flow. The hydraulic routing method provides the most accurate solutions calculating an outflow hydrograph while considering the effects of channel storage and wave shape. The Modifed Puls hydrologic routing method does not work properly when the channel slope is very flat (< 3 ft/mile). The storage-discharge relations calculated using steady flow profiles produce errors when out-of-bank flows occur over wide floodplains.

6. US 281 crossing structure seems oversized.

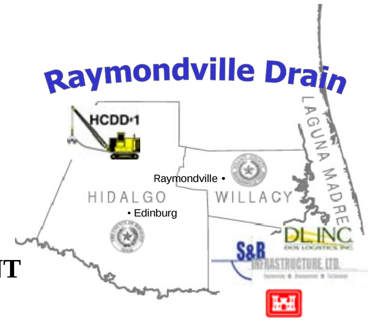
7. Channel slope modifications are required to optimize the channel design, especially upstream reaches.

8. Several reaches show 10+ feet of freeboard.

Attachment B

S&B INFRASTRUCTURE, LTD.
Engineering ⚙ Management ⚙ Technology

Texas Firm No. 1582



Technical Memorandum

HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT

Design and Expansion of the Raymondville Drain

Contract No. C-10-164-04-20

Date: 14 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Draft H&H Assurance Review of September 7, 2011

The following technical memorandum is in response to the comments received from Deren Li, PE of Civil Systems Engineering, Inc. regarding the hydrologic and hydraulic base models for the Raymondville Drain Project and the "Preliminary Engineering Report – Alternatives Analysis thru Hydrologic & Hydraulic Analysis For the Proposed Typical Sections, From Edinburg Lake to the Guerra Detention Facility", dated 5-25-2011 (Supplemented 6-9-2011)". S&B received comments on September 7, 2011 regarding CSE's draft review of the models and document.

A teleconference was held on September 12, 2011 between S&B and CSE regarding the comments. This Technical Memorandum provides written documentation of S&B's response to the comments and additional information required for clarification through the teleconference. Below is a listing of the CSE comments and S&B's formal response:

HEC-HMS Hydrologic Modeling

CSE Comment 1: TP-40/TP 49 rainfall data is used in the HEC-HMS hydrologic modeling analysis. The rainfall depth data based on TP-40/TP-49 is systematically and significantly higher than the rainfall depth data based on the Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas (USGS in cooperation with the Texas Department of Transportation, 2004). With consideration of the USGS/TxDOT Atlas is based on longer precipitation records and better methodology, rainfall data from the USGS/TxDOT Atlas is recommended for the study.

S&B Response: The rainfall depth data was based on TP40/TP49 since areal adjustment data for all storm durations are only known for TP40/TP49. Additionally, rainfall depth data was available for the 10-day storm duration. Furthermore, according the USGS document, this area contained "questionable depth-duration frequency values" with substantial inconsistencies. It was noted that in this region, a large area contained "depths for the 12-hour duration and large recurrence intervals (50 to 500 years) that were larger than the depths for the 1-day duration." The use of TP40/TP49 in lieu of the more modern USGS Atlas was discussed with the USACE, and the agreement was reached during the development of the pre-project hydrology to utilize TP40/TP49 data. This was documented in the USACE white paper titled "Raymondville Drain Pre-Project Conditions Report," April 2006 (USACE RDPR).

As shown in the above tables, once the areal reductions are applied to the precipitation values, the differences are reduced. For the shorter duration events, the precipitation values derived from TP40/TP49 are less than the values obtained from USGS.

Table 1: Comparison Without Areal Reductions

Frequency	6-hr		12-hr		1-day		7-day		10-day	
	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49
50	6.40	6.90	7.25	8.15	8.00	9.58	11.40	14.59	N/A	16.15
100	7.40	7.68	8.50	9.41	9.19	11.07	12.92	16.35	N/A	18.00
250	8.9	8.91	10.5	10.74	11.00	12.65	15.80	18.89	N/A	20.84
500	10	9.76	12.0	11.79	12.50	13.90	17.50	20.75	N/A	22.90

Table 2: Comparison With Areal Reductions (assume 250 sq mile storm area)

	6-hr		12-hr		1-day		7-day		10-day	
Frequency	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49	USGS	TP40/TP49
50	6.40	5.38	7.25	7.25	8.00	8.81	11.40	13.86	N/A	15.67
100	7.40	6.22	8.50	8.37	9.19	10.18	12.92	15.53	N/A	17.46
250	8.9	7.92	10.5	9.35	11.00	11.64	15.80	17.95	N/A	20.21
500	10	8.9	12.0	10.49	12.50	12.72	17.50	19.71	N/A	23.87

CSE Comment 2: There are no HEC-HMS hydrologic models developed to compute future (2061) conditions peak flows and hydrographs. It is not clear how the diversion/intercepted flows were derived without future conditions HECHMS models.

S&B Response: No separate hydrology model is required for the development of the projected peak flow rates. The Texas Water Development Board economic growth factors of were initially utilized as a starting point to determine the level of future development for this area. The impervious cover for several watersheds was increased as a test to determine the average peak flow rate increase. It was found that the on average, the peak flow rate increased between 28%-35% using the growth factors from the Texas Water Development Board. As a result a factor of 1.35 was selected to calculate the peak flow rates for the projected year 2061.

CSE Comment 3: Modified Pulse Method was used for flood routing along various channel reaches. Based on the HEC-HMS model inputs, for all channel reaches that uses Modified Pulse Method, Subreach is assumed "1". This probably overestimates the storage effect for some of the reaches. Reach R1630 is a typical example. There is a 25% flow reduction through this reach.

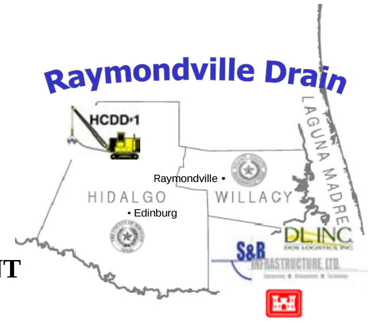
S&B Response: A subreach value of "1" is valid in all instances that it is used. Per the HEC-HMS Technical Reference Manual this value "is used commonly for routing through ponds, lakes, wide, flat floodplains, and channels in which the flow is heavily controlled by downstream conditions."

CSE Comment 4: Some of the storage-outflow relations from HEC-RAS modeling need to be revisited. It was noticed in the HECRAS model, storage is overestimated (see following cross sections with water surface elevations).

S&B Response: Storage-outflow tables were taken directly from HEC-RAS. Many of these reaches contain data that was obtained directly from the models provided to S&B by the USACE. There was significant coordination with the USACE during 2007 to ensure that storage values utilized by S&B corresponded with the values in the USACE models. Due to the extremely flat terrain found in these areas, there will be significant storage in the overbanks once the water surface has risen above the banks of the channel.

Texas Firm No. 1582

Technical Memorandum



HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT
Design and Expansion of the Raymondville Drain
Contract No. C-10-164-04-20

Date: 14 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Draft H&H Assurance Review of September 7, 2011

CSE Comment 5: Percent Imperviousness parameter is not explicitly modeled in the HEC-HMS model. It is not clear whether it was considered in the CN and time of concentration calculations.

S&B Response: The percent imperviousness parameter was not utilized in the hydrologic model. Instead, the composite CN based on land use and hydrologic soil group was utilized. The impervious cover is included in the final CN values, thus no additional impervious cover percentages should be added separately to the HEC-HMS model.

CSE Comment 6: The NRCS standard initial loss method of 0.2S (potential maximum retention) is used in the HEC-HMS model. Since the 10-day storm event is assumed for the study, initial loss has very minor impact to the peak flows. The average initial loss used in the model is approximately 2 inches. Even increase to 5 inches, there is very minor changes in peak flows.

S&B Response: The Initial/Constant Loss method is most appropriate for long duration storm events. For instance, the NRCS Curve Number Method assumes that after the initial loss, all losses go to zero. As a result, the NRCS Curve Number Method should not be used for storms with significant duration. That is why S&B, in conjunction with the USACE, chose to use the Initial/Constant Loss method. It should be noted that the composite curve numbers were utilized to determine the initial loss value. These curve numbers were adjusted to AMC I (dry condition) prior to calculating the initial loss. This resulted in higher initial loss values. Note that this methodology was agreed to with the USACE and previously documented in the USACE white paper titled "Raymondville Drain Pre-Project Conditions Report," April 2006. (USACE RDPR)

CSE Comment 7: It appears that the overall peak flows are significantly high. At Station 64591, the 100-year peak flow is 12,494 cfs in this study, which is 3 times of the FEMA effective 100-year peak flow of 4,175 cfs. With consideration of the difference of the 24-hr and 10-day storm event, there is a maximum of 10 percent increase. It appears that the rainfall depth makes the most of the differences.

S&B Response: The hydrologic model was prepared in accordance with guidelines agreed to with the USACE. The NRCS curve numbers were adjusted to AMC I. During this process S&B kept the adjusted CN's below 60 to account for depressions and storage found throughout the watershed. The unit hydrographs were flattened to account for the flat terrain found in the watershed. The time of concentration calculations were conducted using the methodology recommended by the USACE. S&B went further by calculating average velocities for each subbasin as opposed to using a general approximation. As a result the USACE adjusted the values in their model to match the values provided by S&B. This was documented in USACE's response to S&B in 2007.

CSE Comment 8: A constant Peaking Rate Factor of 150 is used in calculating the Unit Hydrograph. It seems variable PRF should be used with the consideration of the subbasin physical conditions such as slopes and depressions.

S&B Response: Due to the generally flat topography of these watersheds as compared to the average U.S. watershed, the PRF was adjusted from 484 to 150. The value of 150 was chosen to properly model the slopes and depressions found within these watersheds. This value was discussed and agreed to during negotiations with USACE during the development of the pre-project hydrology. This methodology was agreed to with the USACE and previously documented in the USACE white paper titled "Raymondville Drain Pre-Project Conditions Report," April 2006. (USACE RDPR)

CSE Comment 9: Detailed documentation is needed to clearly discuss the relationship between the area reduction calculation using spreadsheets and HEC-HMS modeling results.

S&B Response: Due to inherent limitations with the HEC-HMS program, manual adjustments were needed for each storm event to provide valid results at each junction node. For each storm event, multiple runs were created for storm area values from 0 square miles to 400 square miles in 50 square mile intervals. The peak flow rate for each simulation was recorded. Subsequently, the peak flow rate was calculated based on the actual watershed area at each node. It was this peak flow rate that was subsequently input into the HEC-RAS hydraulic model. This methodology was presented to, and agreed with by the USACE. Further documentation will be provided in the hydraulics section of the flood damage assessment report.

HEC-RAS Hydraulic Modeling

CSE Comment 1: In the base HEC-RAS model, at Station 64591, the 100-year peak flow is 12,494 cfs. In the Alt 1B model, the 100-year peak flow is 9,089 cfs. Where the 9,089 came from?

S&B Response: The peak flow rates for the Alt 1B HEC-RAS models were developed using separate HEC-HMS models with manual calculation of the storm area reduction. These models were provided with the base HEC-HMS models.

CSE Comment 2: As discussed earlier, there is no future conditions HEC-HMS models developed for the project. It is understood there is factor of 1.35 used to obtain future conditions peak flows. What is the justification of 1.35?

S&B Response: As stated earlier, no separate hydrology model is required for the development of the projected peak flow rates. A factor of 1.35 was applied to the HEC-RAS flow data to account for economic growth based on data available from the Texas Water Development Board during the 50 year study period.

CSE Comment 3: Why only 645 cfs is used in the RVD HYD Model Alt 1B for the 100-year (2061)? The Preliminary Engineering Report states a 100-year 1,390 cfs flow is proposed diverted. Alternative diversion flows should be considered to optimize the design of the diversion weir structure, channel, and detention basins.

S&B Response: The RVD HYD Model Alt 1B is based on the 10-day storm event as required by USACE for development of the Flood Damage Assessment (FDA) needed to obtain federal funding. The Preliminary Engineering Report utilizes the 24-hr storm duration and was calibrated to the peak flows found in the Letter of Map Revision for Hidalgo County dated, 05-17-2001. The difference is due to the amount of runoff that is intercepted by the diversion channel. Once the FDA is finalized, and the design storm event is selected, the design of the diversion weir structure, channel and detention basin can be further optimized.

CSE Comment 4: The combined peak flow (diversion/interception) does not reflect the 1.35 factor.

S&B Response: Based on the comment above, it is unclear where in the model it is believed that the 1.35 factor was not applied. S&B is assuming that this comment is referring to the actual diversion weir. For the purposes of our initial analysis, the amount of flow diverted was kept constant. As a result the combined diversion/interception peak flow rate for the Year 2011 and Year 2061 will not directly correspond to the 1.35 factor.

CSE Comment 5: With consideration of the very flat nature of the drainage channels, the HEC-RAS Unsteady Flow Module is recommended for this study. The hydraulic routing technique within the HEC-RAS Unsteady Flow Module is based on the solution of the partial differential equations (dynamic wave equations) of unsteady channel flow. The hydraulic routing method provides the most accurate solutions calculating an outflow hydrograph while considering the effects of channel storage and wave shape. The Modifed Puls hydrologic routing method does not work properly when the channel slope is very flat (< 3 ft/mile). The storage-discharge relations calculated using steady flow profiles produce errors when out-of-bank flows occur over wide floodplains.

S&B Response: Due to the complicated nature of the existing watershed, S&B does not feel that the HEC-RAS Unsteady Flow modeling is appropriate. All storage routing and flow attenuation was calculated using the methods found in HEC-HMS. The HEC-RAS steady state model was utilized to prepare the storage-outflow curves that were used in HEC-HMS. This methodology produced individual hydrographs that were attenuated through their corresponding reaches. It was these attenuated peak flows that were input into the HEC-RAS steady flow model. The methodology of using HEC-HMS to provide for the flow attenuation and storage routing within the watershed was agreed to with the USACE during the pre-project conditions phase of the analysis. This same above methodology was used in the steady flow HEC-RAS and HMS models that were provided by the USACE to S&B for the Willacy County portion of the Raymondville Drain. Additionally, HEC-RAS unsteady flow models are inherently unstable, especially in watersheds as complex as the Raymondville Drain and the North Main Drain. Per the HEC-RAS Hydraulic Reference Manual,

> "In practice, other factors may also contribute to the non-stability of the solution scheme. These factors include dramatic changes in channel cross-sectional properties, abrupt changes in channel slope, characteristics of the flood wave itself, and complex hydraulic structures such as levees, bridges, culverts, weirs, and spillways. In fact, these other factors often overwhelm any stability considerations…"

Concerning the use of Modified Puls, this methodology is specifically useful in areas with wide floodplains and where there is significant backwater that will influence the discharge hydrograph. Additionally, this method is valid from slopes ranging from 10 to 2 ft/mile. (Chapter 9, EM1110-2-1417)

CSE Comment 6: US 281 crossing structure seems oversized.

S&B Response: The US 281 culvert structure was sized for use in determining alternatives for the FDA. The preliminary design was selected to provide a headloss through the structure that closely mimics the proposed bridge solution. Once the design storm event has been selected, the detailed design will be performed to provide an efficient solution that complies with TxDOT design requirements and meets the design constraints set forth by the HCCD#1.

CSE Comment 7: Channel slope modifications are required to optimize the channel design, especially upstream reaches.

S&B Response: To reiterate, once the flood damage assessment is finalized, and the design storm selected, the final design of the diversion channel will be optimized. This preliminary channel geometry is being utilized to determine and evaluate alternatives during this feasibility phase for the entire Raymondville Drain and North Main Drain watersheds.

CSE Comment 8: Several reaches show 10+ feet of freeboard.

S&B Response: This appears to be similar to comment #7. Please see previous response.

S&B Additional Note Regarding HEC-RAS Modeling: A summary of hydrologic and hydraulic models will be included as documentation of the individual hydrologic and hydraulic models used for the pre-project and post-project conditions modeling. This summary will be included in the appendices of the final H&H report. This summary will serve as a guide and to provide clarification for which computer model was utilized for the pre-project and post-project conditions H&H modeling.

Respectfully Submitted and Released For Planning Purposes Only Under the Authority of:

_______________________ **Texas PE # 88453**
Andres Cardenas, PE
Date: 09/14/11

Attachment C

Deren

From: Deren [dli@cseengineers.com]
Sent: Monday, September 19, 2011 3:50 PM
To: 'Teague, Sharlotte'
Cc: 'Cardenas, Andres'; 'Rios, Daniel'
Subject: RE: Draft Technical Memo for H&H Review

Sharlotte,

Regarding the CSE comments and S&B responses, I believe the two most critical ones are Number 2 and Number 7, both dealing with peak flows for the project. Since project peak flows (existing and post conditions) directly dictate the sizes of the project components and economic justification of the project, further documentation and investigations are needed.

Number 2 (Future Conditions 2061 HEC-HMS Modeling)

I believe a future conditions HEC-HMS is needed for the project with consideration of the sizes of the studied watersheds. The application of an averaged factor of 1.35 will not be able to properly reflect the variations of developments within the entire watersheds and therefore will not be able to properly simulate the hydrologic responses of the watersheds to the projected developments.

If it is available, please email me a copy of the sample testing calculations in deriving the 1.35 factor.

Number 7 (Magnitudes of Peak Flows)

Based on flows in the HEC-RAS model for North Main Drain, at Seminary Road (SX 65691), the 100-year peak discharges are 12,501 cfs (existing conditions) and 16,976 cfs (1.35x12,501). The estimated existing 100-peak flow is 3 times the FEMA effective 100-year peak flow of 4,175 cfs, and 4 times of the estimated peak flow of 3,077 cfs by Melden and Hunt, Inc. (Critique of the Flood Insurance Study, 2000).

Since the differences in rainfall data between 10-day and 24-storm events, as well as between TP40/TP49 and USGS rainfall data, don't make a 3 to 4 times differences in peak flow discharges, I have further investigated the methods of estimating Tc or LAG. By comparing the travel time method with the SCS LAG equation $L^{0.8}[(100\text{-}CN)\text{-}9]^{0.7}/(1900xS^{0.5})$, there is a significant difference in time of concentration for most of the subbasins. With the SCS LAG equation, the unit hydrograph (UH) could be more than doubled for some subbasins.

If it is available, please email me a copy of the USACE' comments in dealing with the travel time methd.

Please let me know if you have any questions.

Thanks.

Deren Li, Ph.D., P.E., D.WRE, CFM
Civil Systems Engineering Inc.
9894 Bissonnet St., Suite 404
Houston, Texas 77036
713-298-6819 (c)
713-782-3811 (o)
www.cseengineers.com

From: Teague, Sharlotte [mailto:slteague@sbinfra.com]
Sent: Wednesday, September 14, 2011 3:58 PM
To: Deren
Cc: Cardenas, Andres
Subject: RE: Draft Technical Memo for H&H Review

Deren – Attached is our written response to your comments based on the coordination and teleconference we had on Monday.

Let us know if you have any questions, thanks.
Sincerely,
Sharlotte L. Teague, PE
Senior Project Manager
S&B Infrastructure, Ltd. - McAllen
5408 N. 10th Street, McAllen, TX 78504
ph: 956.926.5000; fax: 956.994.0427
cell: 956.279.7364

From: Deren [mailto:dli@cseengineers.com]
Sent: Wednesday, September 07, 2011 4:27 PM
To: Teague, Sharlotte
Subject: Draft Technical Memo for H&H Review

Sharlotte,

Attached is the draft review comments.

Please let me know if you have any questions.

Deren Li, Ph.D., P.E., D.WRE, CFM
Civil Systems Engineering Inc.
9894 Bissonnet St., Suite 404
Houston, Texas 77036
713-298-6819 (c)
713-782-3811 (o)
www.cseengineers.com

Attachment D

Regarding the CSE comments and S&B responses, I believe the two most critical ones are Number 2 and Number 7, both dealing with peak flows for the project. Since project peak flows (existing and post conditions) directly dictate the sizes of the project components and economic justification of the project, further documentation and investigations are needed.

Number 2 (Future Conditions 2061 HEC-HMS Modeling)

I believe a future conditions HEC-HMS is needed for the project with consideration of the sizes of the studied watersheds. The application of an averaged factor of 1.35 will not be able to properly reflect the variations of developments within the entire watersheds and therefore will not be able to properly simulate the hydrologic responses of the watersheds to the projected developments.

The average factor of 1.35 was correctly utilized to account for the increase in peak runoff due to economic growth factors obtained from the Texas Water Development Board and projected population as determined by the Texas State Data Center, the Office of the State Demographer and Census 2000 data. Concerning "variation of developments", this is not applicable due to USACE direction to S&B. According to USACE guidelines sent to S&B, *"The existing land use pattern will be assumed to continue in the same proportion throughout the watershed".* Since the majority of the watershed is not zoned for future development, it is not feasible to attempt to prepare detailed development estimates over these watersheds. Any attempt will be solely based on the preparer's opinion. As such, S&B did make preliminary estimates based on population forecasts for a selective number of sub-basins. A hydrologic analysis was performed to determine the average increase in runoff. The increase varied in range from 28% to 32%. Based on the analysis, a factor of 35% was selected as an estimate on the increases in future peak flow runoff.

Number 7 (Magnitudes of Peak Flows)

Based on flows in the HEC-RAS model for North Main Drain, at Seminary Road (SX 65691), the 100-year peak discharges are 12,501 cfs (existing conditions) and 16,976 cfs (1.35x12,501). The estimated existing 100-peak flow is 3 times the FEMA effective 100-year peak flow of 4,175 cfs, and 4 times of the estimated peak flow of 3,077 cfs by Melden and Hunt, Inc. (Critique of the Flood Insurance Study, 2000).

Since the differences in rainfall data between 10-day and 24-storm events, as well as between TP40/TP49 and USGS rainfall data, don't make a 3 to 4 times differences in peak flow discharges, I have further investigated the methods of estimating Tc or LAG. By comparing the travel time method with the SCS LAG equation $L^{0.8}[(100\text{-}CN)\text{-}9]^{0.7}/(1900xS^{0.5})$, there is a significant difference in time of concentration for most of the subbasins. With the SCS LAG equation, the unit hydrograph (UH) could be more than doubled for some subbasins.

During the base conditions HEC-HMS development of the Raymondville Drain and North Main Drain watersheds, a discussion was conducted with the USACE concerning the methodology for determining the lag time of the individual sub-basins. In S&B's original analysis, the SCS CN lag time equation was utilized. However, the USACE felt that an "*accurate SCS lag was needed because the unit hydrograph was already flattened by the adjustment to the peaking factor. If* *an excessively long lag was used in combination with a reduced peaking factor, then an unrealistically low peak flow rate would likely occur for each sub-basin."* The USACE felt that by using the velocity method to calculate the lag time, the shorter times of concentration would yield *"more reasonable HMS results"*. Additionally, the USACE assumed an open channel flow velocity of 0.6 fps for their analysis. In order to refine this further, S&B utilized LIDAR data to prepare actual typical sections to calculate the open channel flow velocity for each sub-basin. It was because of this extra step that the USACE chose to utilize S&B values.

If it is available, please email me a copy of the USACE' comments in dealing with the travel time methd.

This information was found in the USACE Raymondville Pre-Project Report dated 04-11-2006, which we believe was provided at the meeting in S&B's office on August 23. We will email you another copy.

Attachment E

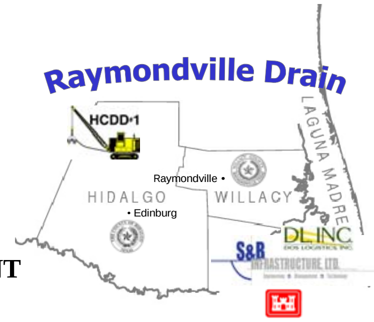
Comparison of Unit Hydrograph Peak Flows (S&B vs. SCS Equation)											
Subarea Name	Square Miles	TC(S&B)	LAG(S&B)	Tp(S&B)	LAG(SCS)[1]	LAG(SCS)/	UH Qp - PRF 150		Qp(S&B)/Qp(SCS)	Qp(SCS)	Qp(S&B)/
		Hrs	Hrs	Hrs	Hrs	LAG(S&B)	Qp(S&B)	Qp(SCS)		PRF 484	Qp(SCS484)
R1000W1000	3.352	11.30	6.78	7.78	17.41	2.57	65	29	2.24	93	0.69
R1070W1070[2]	5.508	20.40	12.24	12.74	29.40	2.40	65	28	2.31	91	0.72
R1090W1090	1.461	18.90	11.34	12.84	17.07	1.51	17	13	1.33	41	0.41
R1140W1140	6.361	18.70	11.22	12.72	24.10	2.15	75	40	1.89	128	0.59
R1150W1150	2.940	4.40	2.64	2.89	11.03	4.18	153	40	3.82	129	1.18
R1160W1160	5.171	32.70	19.62	22.62	28.50	1.45	34	27	1.26	88	0.39
R1180W1180	1.436	19.70	11.82	13.32	21.24	1.80	16	10	1.59	33	0.49
R1190W1190	5.024	22.30	13.38	14.88	42.71	3.19	51	18	2.87	57	0.89
R1200W1200	3.692	9.90	5.94	6.94	21.89	3.68	80	25	3.15	82	0.98
R1210W1210	2.773	7.40	4.44	4.94	18.21	4.10	84	23	3.69	74	1.14
R1230W1230	9.845	10.20	6.12	7.12	37.47	6.12	207	39	5.26	127	1.63
R1240W1240	3.668	11.80	7.08	8.08	18.67	2.64	68	29	2.31	95	0.72
R1250W1250	2.305	17.50	10.50	12.00	24.21	2.31	29	14	2.02	46	0.63
R1280W1280	4.871	6.50	3.90	4.94	26.45	6.78	148	28	5.36	89	1.66
R1290W1290	0.515	9.90	5.94	6.94	16.57	2.79	11	5	2.39	15	0.74
R1310W1310	2.973	32.20	19.32	22.32	38.29	1.98	20	12	1.72	38	0.53
R1320W1320	9.469	8.80	5.28	5.78	36.57	6.93	246	39	6.33	125	1.96
R1330W1330	2.267	12.20	7.32	8.32	15.05	2.06	41	23	1.81	73	0.56
R1380W1300	2.389	9.60	5.76	6.26	15.19	2.64	57	24	2.43	76	0.75
R1390W1390	4.890	12.90	7.74	8.74	34.15	4.41	84	21	3.91	69	1.21
R1410W960	5.494	9.20	5.52	6.02	11.66	2.11	137	71	1.94	228	0.60
R1440W1440	10.172	34.90	20.94	23.94	42.08	2.01	64	36	1.76	117	0.54
R1460W1460	0.946	9.10	5.46	5.96	18.24	3.34	24	8	3.06	25	0.95
R1470W1470	1.181	3.50	2.10	2.35	18.73	8.92	75	9	7.97	31	2.47
R1480W1480	0.592	4.20	2.52	2.77	8.66	3.44	32	10	3.13	33	0.97
R1490W1490	2.445	18.50	11.10	12.60	36.19	3.26	29	10	2.87	33	0.89
R1520W1520	3.557	8.00	4.80	5.30	23.22	4.84	101	23	4.38	74	1.36
R1540W1540	6.025	22.30	13.38	14.88	26.13	1.95	61	35	1.76	112	0.54
R1560W1560	4.210	15.60	9.36	10.36	27.59	2.95	61	23	2.66	74	0.83
R1570W1570	2.444	12.60	7.56	8.06	17.24	2.28	45	21	2.14	69	0.66
R1580W1580	6.803	20.20	12.12	13.62	24.06	1.99	75	42	1.77	137	0.55
R1590W1590	4.653	11.30	6.78	7.78	27.95	4.12	90	25	3.59	81	1.11
R1600W1600	3.150	16.20	9.72	10.72	17.27	1.78	44	27	1.61	88	0.50
R1610W1610	4.941	19.00	11.40	12.90	24.90	2.18	57	30	1.93	96	0.60
R1620W1620	7.311	13.80	8.28	9.28	27.76	3.35	118	40	2.99	127	0.93
R1630W1630	10.162	12.80	7.68	8.68	62.45	8.13	176	24	7.19	79	2.23
R1660W1660	3.894	8.60	5.16	5.66	23.50	4.55	103	25	4.15	80	1.29
R1670W1670	3.367	4.60	2.76	3.01	20.08	7.28	168	25	6.67	81	2.07

Comparison of Unit Hydrograph Peak Flows (S&B vs. SCS Equation)

Subarea Name	Square Miles	TC(S&B)	LAG(S&B)	Tp(S&B)	LAG(SCS)[1]	LAG(SCS)/	UH Qp - PRF 150		Qp(S&B)/Qp(SCS)	Qp(SCS)	Qp(S&B)/
		Hrs	Hrs	Hrs	Hrs	LAG(S&B)	Qp(S&B)	Qp(SCS)		PRF 484	Qp(SCS484)
R1690W1690	3.009	6.60	3.96	4.46	23.94	6.04	101	19	5.37	61	1.66
R1700W1700	10.796	19.40	11.64	13.14	47.82	4.11	123	34	3.64	109	1.13
R1710W1710	2.333	16.20	9.72	10.72	18.81	1.94	33	19	1.75	60	0.54
R1720W1720	7.190	27.80	16.68	18.18	28.81	1.73	59	37	1.58	121	0.49
R1730W1730	1.005	3.60	2.16	2.41	11.86	5.49	63	13	4.92	41	1.52
R1740W1740	0.947	12.10	7.26	8.26	15.35	2.11	17	9	1.86	30	0.58
R1750W1750	7.491	28.60	17.16	18.66	34.71	2.02	60	32	1.86	104	0.58
R1760W1760	2.361	15.30	9.18	10.18	21.56	2.35	35	16	2.12	53	0.66
R1770W1770	4.844	21.60	12.96	14.16	31.05	2.40	51	23	2.19	75	0.68
R1780W1780	0.655	2.00	1.20	1.37	9.21	7.67	72	11	6.72	34	2.08
R1800W1800	2.286	5.40	3.24	3.74	21.31	6.58	92	16	5.70	52	1.77
R1840W1840	1.763	3.80	2.28	2.53	17.13	7.51	105	15	6.77	50	2.10
R1850W1850	3.823	6.60	3.96	4.46	24.13	6.09	129	24	5.41	77	1.68
R1860W1860	6.614	12.00	7.20	8.20	39.41	5.47	121	25	4.81	81	1.49
R1890W1890	2.721	5.90	3.54	4.04	22.90	6.47	101	18	5.67	57	1.76
R1900W1900	8.909	12.20	7.32	8.32	38.93	5.32	161	34	4.68	111	1.45
R1920W1920	1.008	2.92	1.75	1.99	9.26	5.28	76	16	4.65	53	1.44
R1930W1930	6.445	19.80	11.88	13.38	70.91	5.97	72	14	5.30	44	1.64
R1940W1940	4.558	21.60	12.96	14.46	60.90	4.70	47	11	4.21	36	1.31
R1960W1960	1.665	2.70	1.62	1.87	11.24	6.94	134	22	6.01	72	1.86
R1970W1970	2.257	23.20	13.92	15.42	19.47	1.40	22	17	1.26	56	0.39
R1990W1990	3.046	6.80	4.08	4.58	28.22	6.92	100	16	6.16	52	1.91
R2010W2010	3.501	18.20	10.92	12.42	22.37	2.05	42	23	1.80	76	0.56
R2020W2020	11.520	16.50	9.90	11.40	56.79	5.74	152	30	4.98	98	1.54
R2030W2030	6.681	7.00	4.20	4.70	26.03	6.20	213	38	5.54	124	1.72
R2040W2040	6.462	6.10	3.66	4.66	22.75	6.22	208	43	4.88	137	1.51
R2050W2050	0.664	5.90	3.54	4.16	7.78	2.20	24	13	1.87	41	0.58
R2060W2060	7.784	8.30	4.98	5.48	38.15	7.66	213	31	6.96	99	2.16
R2070W2070	2.869	4.50	2.70	2.95	16.00	5.92	146	27	5.42	87	1.68
R2080W2080	3.880	15.60	9.36	10.36	47.85	5.11	56	12	4.62	39	1.43
R2110W2110	1.712	3.20	1.92	2.92	9.70	5.05	88	26	3.32	85	1.03
R2120W2120	8.669	25.30	15.18	16.98	53.20	3.50	77	24	3.13	79	0.97
R2150W2150	14.457	14.50	8.70	9.70	50.40	5.79	224	43	5.20	139	1.61
R2170W2170	2.515	4.60	2.76	3.01	16.71	6.05	125	23	5.55	73	1.72
R2180W2180	1.554	3.60	2.16	2.41	14.49	6.71	97	16	6.01	52	1.86
R2200W2200	3.602	6.90	4.14	4.64	27.76	6.70	116	19	5.98	63	1.85
R2210W2210	1.227	2.83	1.70	2.70	11.89	7.01	68	15	4.41	50	1.37
R2220W2220	1.790	3.50	2.10	2.35	12.08	5.75	114	22	5.14	72	1.59

Comparison of Unit Hydrograph Peak Flows (S&B vs. SCS Equation)											
Subarea Name	Square Miles	TC(S&B)	LAG(S&B)	Tp(S&B)	LAG(SCS)[1]	LAG(SCS)/	UH Qp - PRF 150		Qp(S&B)/Qp(SCS)	Qp(SCS)	Qp(S&B)/
		Hrs	Hrs	Hrs	Hrs	LAG(S&B)	Qp(S&B)	Qp(SCS)		PRF 484	Qp(SCS484)
R2240W2240	4.019	7.30	4.38	4.88	20.01	4.57	124	30	4.10	97	1.27
R2250W2250	5.247	15.50	9.30	10.30	36.81	3.96	76	21	3.57	69	1.11
R2300W2300	4.401	9.20	5.52	6.02	33.95	6.15	110	19	5.64	63	1.75
R2330W2330	0.858	2.20	1.32	1.49	5.67	4.29	86	23	3.80	73	1.18
R2400W2400	3.464	16.80	10.08	11.58	22.61	2.24	45	23	1.95	74	0.61
R2410W2410	2.281	4.70	2.82	3.07	13.43	4.76	111	25	4.37	82	1.36
R2430W2430	9.142	16.50	9.90	10.90	36.34	3.67	126	38	3.33	122	1.03
R2440W2440	0.695	2.10	1.26	1.43	4.17	3.31	73	25	2.92	81	0.90
R2460W2460	3.302	4.30	2.58	2.83	12.47	4.83	175	40	4.40	128	1.37
R2490W2490	2.631	17.70	10.62	12.12	10.82	1.02	33	36	0.89	118	0.28
R2500W2500	3.920	7.30	4.38	4.88	19.88	4.54	120	30	4.07	95	1.26
R2570W2570	4.759	12.90	7.74	8.74	35.16	4.54	82	20	4.02	66	1.25
R2580W2580	2.912	5.10	3.06	3.56	13.09	4.28	123	33	3.68	108	1.14
R2620W2600	4.261	4.20	2.52	2.77	18.76	7.44	231	34	6.77	110	2.10
R2630W2610	5.908	8.60	5.16	5.66	26.55	5.14	157	33	4.69	108	1.45
R2640W2620	7.067	26.30	15.78	17.28	22.77	1.44	61	47	1.32	150	0.41
R2650W2630	6.453	14.00	8.40	9.40	20.32	2.42	103	48	2.16	154	0.67
R2660W2640	0.680	2.10	1.26	1.43	9.08	7.21	71	11	6.35	36	1.97
R2670W2650	3.246	3.40	2.04	2.29	15.69	7.69	213	31	6.85	100	2.12
R2680W2660	2.552	3.70	2.22	2.47	13.78	6.21	155	28	5.58	90	1.73
R2690W1420	0.412	2.80	1.68	1.85	8.32	4.95	33	7	4.50	24	1.39
R2700W1430	1.498	14.00	8.40	9.40	14.26	1.70	24	16	1.52	51	0.47
R2710W2680	3.269	18.60	11.16	12.66	11.02	0.99	39	45	0.87	144	0.27
R2720W2690	2.258	3.90	2.34	2.59	10.27	4.39	131	33	3.97	106	1.23
R2730W2700	0.868	3.50	2.10	2.35	9.81	4.67	55	13	4.18	43	1.29
R2760W2710	3.732	3.20	1.92	2.17	14.67	7.64	258	38	6.76	123	2.09
R2770W2720	1.177	2.60	1.56	1.81	12.90	8.27	98	14	7.13	44	2.21
R2780W2730	1.473	3.30	1.98	2.23	10.74	5.42	99	21	4.82	66	1.49
R710W710	6.306	19.60	11.76	13.26	21.41	1.82	71	44	1.61	143	0.50
R740W740	7.275	24.50	14.70	16.20	27.34	1.86	67	40	1.69	129	0.52
R840W840	10.138	13.70	8.22	9.22	27.55	3.35	165	55	2.99	178	0.93
R950W950	3.838	9.20	5.52	6.02	17.22	3.12	96	33	2.86	108	0.89
R970W970	4.335	9.40	5.64	6.14	21.20	3.76	106	31	3.45	99	1.07

Note: 1 - Lag from SCS equation is assumed as Tp; 2 - S&B UH spreadsheet does not match with the Appendix B Volume 2 0f 2 for North Main Drain Hydrology



Texas Firm No. 1582

Technical Memorandum

HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT
Design and Expansion of the Raymondville Drain
Contract No. C-10-164-04-20

Date: 23 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Final H&H Assurance Review of September 22, 2011

The following technical memorandum is in response to the final comments received from Deren Li, PE of Civil Systems Engineering, Inc. regarding the hydrologic and hydraulic base models for the Raymondville Drain Project and the "Preliminary Engineering Report – Alternatives Analysis thru Hydrologic & Hydraulic Analysis For the Proposed Typical Sections, From Edinburg Lake to the Guerra Detention Facility", dated 5-25-2011 (Supplemented 6-9-2011)".

S&B received follow-up comments on September 19, 2011 from CSE's, and provided responses to the CSE comments on September 20, 2011 by e-mail.

In CSE's Final Technical Memorandum, dated September 22, 2011, CSE still had concerns regarding the peak flows for the project, and provided a comparison table in the Final Technical Memorandum's Attachment E. Below is a listing of the CSE comments and S&B's formal response:

CSE Comment: A comparison table <in Attachment E of CSE's Final Technical Memorandum> is presented to further demonstrate our concerns. First, comparison is made between the Lag Time values computed by S&B and the SCS Equation (North Main Drain subbasins were used for this comparison). As shown in column LAG(SCS)/LAG(S&B), the LAG values based on SCS lag equation are 1 to 8 times of the LAG (S&B) values. The ratios are reflected in the Unit Hydrograph peak flows (for PRF 150). Also comparison is made for Unit Hydrograph Peak flows between Qp based on PRF 150 and S&B's lag values and Qp based on standard PRF 484 and SCS lag equation. Column Qp(S&B)/Qp(SCS484) shows that even with the much lower PRF 150 for the project, for most of the subbasins, the computed peak flows are much greater than peak flows based on the standard PRF 484 (3.2 times of 150).

It should be noted that the above comparison results do not disqualify the Tc or travel time METHOD used for this study. However, estimates of parameters need to be revised to ensure that lag time values and peak flows are more representative of the local watershed conditions.

S&B Response: As stated previously, the basis for the individual sub-basin times of concentration were based on the Velocity Method utilizing LIDAR data to determine individual channel typical sections and velocities. The USACE recognized that this level of detail was superior to the previous assumed velocities that were used by the USACE in their analysis. As shown by CSE, the substantial differences between the times of concentration calculated by this method versus the SCS lag equation simply reinforces that the SCS lag equation over-simplifies this crucial calculation, when utilized on such a large, varied watershed. Based on the data received from CSE using the SCS lag equation, many of the sub-basins do not achieve a peak unit discharge of 10 cfs/sq.mile, which is very low amount of runoff for single square mile of area. Attached is **Exhibit "A"** which calculates the unit discharges for each sub-basin based on S&B's methodology and using the SCS lag equation.

Technical Memorandum

HIDALGO COUNTY PROFESSIONAL ENGINEERING SERVICES AGREEMENT
Design and Expansion of the Raymondville Drain
Contract No. C-10-164-04-20

Date: 23 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Final H&H Assurance Review of September 22, 2011

Concerning the other estimate of parameter, the Curve Numbers, precipitation values, routing reaches, unit hydrographs and Tc calculations were coordinated and reviewed with the USACE to ensure that each sub-basin's characteristics were accurately modeled for the required 10-day storm duration.

Respectfully Submitted and Released For Planning Purposes Only Under the Authority of:

Andres Cardenas, PE **Texas PE # 88453**

Date: 9 / 23 / 2011

Attachment:
Exhibit "A" – Unit Discharges Per Sub-Basin

Date: 23 September 2011

RE: Response to Civil Systems Engineering, Inc.'s Final H&H Assurance Review of September 22, 2011

Exhibit "A"

Comparison of Unit Hydrograph Peak Flows (S&B vs. SCS Equation)					
Subarea Name	Square Miles	Qp(S&B) (cfs)	Unit Discharge (cfs/sq.mi)	Qp(SCS) PRF 150 (cfs)	Unit Discharge (cfs/sq.mi)
R1000W 1000	3.352	65	19	29	9
R1070W 10702	5.508	65	12	28	5
R1090W 1090	1.461	17	12	13	9
R1140W 1140	6.361	75	12	40	6
R1150W 1150	2.940	153	52	40	14
R1160W 1160	5.171	34	7	27	5
R1180W 1180	1.436	16	11	10	7
R1190W 1190	5.024	51	10	18	4
R1200W 1200	3.692	80	22	25	7
R1210W 1210	2.773	84	30	23	8
R1230W 1230	9.845	207	21	39	4
R1240W 1240	3.668	68	19	29	8
R1250W 1250	2.305	29	13	14	6
R1280W 1280	4.871	148	30	28	6
R1290W 1290	0.515	11	21	5	10
R1310W 1310	2.973	20	7	12	4
R1320W 1320	9.469	246	26	39	4
R1330W 1330	2.267	41	18	23	10
R1380W 1300	2.389	57	24	24	10
R1390W 1390	4.890	84	17	21	4
R1410W 960	5.494	137	25	71	13
R1440W 1440	10.172	64	6	36	4
R1460W 1460	0.946	24	25	8	8
R1470W 1470	1.181	75	64	9	8
R1480W 1480	0.592	32	54	10	17
R1490W 1490	2.445	29	12	10	4
R1520W 1520	3.557	101	28	23	6
R1540W 1540	6.025	61	10	35	6
R1560W 1560	4.210	61	14	23	5
R1570W 1570	2.444	45	18	21	9
R1580W 1580	6.803	75	11	42	6
R1590W 1590	4.653	90	19	25	5
R1600W 1600	3.150	44	14	27	9
R1610W 1610	4.941	57	12	30	6
R1620W 1620	7.311	118	16	40	5
R1630W 1630	10.162	176	17	24	2
R1660W 1660	3.894	103	26	25	6
R1670W 1670	3.367	168	50	25	7
R1690W 1690	3.009	101	34	19	6
R1700W 1700	10.796	123	11	34	3
R1710W 1710	2.333	33	14	19	8
R1720W 1720	7.190	59	8	37	5
R1730W 1730	1.005	63	63	13	13
R1740W 1740	0.947	17	18	9	10
R1750W 1750	7.491	60	8	32	4
R1760W 1760	2.361	35	15	16	7
R1770W 1770	4.844	51	11	23	5
R1780W 1780	0.655	72	110	11	17

Comparison of Unit Hydrograph Peak Flows (S&B vs. SCS Equation)					
Subarea Name	Square Miles	Qp(S&B) (cfs)	Unit Discharge (cfs/sq.mi)	Qp(SCS) PRF 150 (cfs)	Unit Discharge (cfs/sq.mi)
R1800W 1800	2.286	92	40	16	7
R1840W 1840	1.763	105	60	15	9
R1850W 1850	3.823	129	34	24	6
R1860W 1860	6.614	121	18	25	4
R1890W 1890	2.721	101	37	18	7
R1900W 1900	8.909	161	18	34	4
R1920W 1920	1.008	76	75	16	16
R1930W 1930	6.445	72	11	14	2
R1940W 1940	4.558	47	10	11	2
R1960W 1960	1.665	134	80	22	13
R1970W 1970	2.257	22	10	17	8
R1990W 1990	3.046	100	33	16	5
R2010W 2010	3.501	42	12	23	7
R2020W 2020	11.520	152	13	30	3
R2030W 2030	6.681	213	32	38	6
R2040W 2040	6.462	208	32	43	7
R2050W 2050	0.664	24	36	13	20
R2060W 2060	7.784	213	27	31	4
R2070W 2070	2.869	146	51	27	9
R2080W 2080	3.880	56	14	12	3
R2110W 2110	1.712	88	51	26	15
R2120W 2120	8.669	77	9	24	3
R2150W 2150	14.457	224	15	43	3
R2170W 2170	2.515	125	50	23	9
R2180W 2180	1.554	97	62	16	10
R2200W 2200	3.602	116	32	19	5
R2210W 2210	1.227	68	55	15	12
R2220W 2220	1.790	114	64	22	12
R2240W 2240	4.019	124	31	30	7
R2250W 2250	5.247	76	14	21	4
R2300W 2300	4.401	110	25	19	4
R2330W 2330	0.858	86	100	23	27
R2400W 2400	3.464	45	13	23	7
R2410W 2410	2.281	111	49	25	11
R2430W 2430	9.142	126	14	38	4
R2440W 2440	0.695	73	105	25	36
R2460W 2460	3.302	175	53	40	12
R2490W 2490	2.631	33	13	36	14
R2500W 2500	3.920	120	31	30	8
R2570W 2570	4.759	82	17	20	4
R2580W 2580	2.912	123	42	33	11
R2620W 2600	4.261	231	54	34	8
R2630W 2610	5.908	157	27	33	6
R2640W 2620	7.067	61	9	47	7
R2650W 2630	6.453	103	16	48	7
R2660W 2640	0.680	71	104	11	16
R2670W 2650	3.246	213	66	31	10
R2680W 2660	2.552	155	61	28	11

Comparison of Unit Hydrograph Peak Flows (S&B vs. SCS Equation)					
Subarea Name	Square Miles	Qp(S&B) (cfs)	Unit Discharge (cfs/sq.mi)	Qp(SCS) PRF 150 (cfs)	Unit Discharge (cfs/sq.mi)
R2690W 1420	0.412	33	80	7	17
R2700W 1430	1.498	24	16	16	11
R2710W 2680	3.269	39	12	45	14
R2720W 2690	2.258	131	58	33	15
R2730W 2700	0.868	55	63	13	15
R2760W 2710	3.732	258	69	38	10
R2770W 2720	1.177	98	83	14	12
R2780W 2730	1.473	99	67	21	14
R710W 710	6.306	71	11	44	7
R740W 740	7.275	67	9	40	5
R840W 840	10.138	165	16	55	5
R950W 950	3.838	96	25	33	9
R970W 970	4.335	106	24	31	7

= unit discharge less than 10 cfs/sq.mile

From: Cardenas, Andres
Sent: Tuesday, September 20, 2011 5:59 PM
To: 'Deren'
Cc: Teague, Sharlotte
Subject: RE: Draft Technical Memo for H&H Review

Deren,

Per the response letter you received earlier today (attached), I am sending you a copy of the "USACE Raymondville Pre-Project Report 4-11-06" for further documentation of the time of concentration calculation. I have also included a calculation page for one of the North Main Drain sub-basins.

Andres Cardenas, P.E., CFM
S & B Infrastructure
807 Brazos, Suite 901
Austin, TX 78701
(512) 542-7426
amcardenas@sbinfra.com

From: Teague, Sharlotte
Sent: Tuesday, September 20, 2011 4:12 PM
To: 'Deren'
Subject: FW: Follow-up Comments from Deren Li on H&H Review

Deren – Attached is Andres' responses. Can you give me a call ASAP, thanks.

Sharlotte L. Teague, PE
Senior Project Manager
S&B Infrastructure, Ltd. - McAllen
5408 N. 10th Street, McAllen, TX 78504
ph: 956.926.5000; fax: 956.994.0427
cell: 956.279.7364

From: Deren [mailto:dli@cseengineers.com]
Sent: Monday, September 19, 2011 3:50 PM
To: Teague, Sharlotte
Cc: Cardenas, Andres; Rios, Daniel
Subject: RE: Draft Technical Memo for H&H Review

Sharlotte,

Regarding the CSE comments and S&B responses, I believe the two most critical ones are Number 2 and Number 7, both dealing with peak flows for the project. Since project peak flows (existing and post conditions) directly dictate the sizes of the project components and economic justification of the project, further documentation and investigations are needed.

Number 2 (Future Conditions 2061 HEC-HMS Modeling)

I believe a future conditions HEC-HMS is needed for the project with consideration of the sizes of the studied watersheds. The application of an averaged factor of 1.35 will not be able to properly reflect the variations of developments within the entire watersheds and therefore will not be able to properly simulate the hydrologic responses of the watersheds to the projected developments.

If it is available, please email me a copy of the sample testing calculations in deriving the 1.35 factor.

Number 7 (Magnitudes of Peak Flows)

Based on flows in the HEC-RAS model for North Main Drain, at **Seminary Road** (SX 65691), the 100-year peak discharges are **12,501 cfs** (existing conditions) and **16,976 cfs** (1.35x12,501). The estimated existing 100-peak flow is **3 times** the FEMA effective 100-year peak flow of **4,175 cfs**, and **4 times** of the estimated peak flow of **3,077 cfs** by Melden and Hunt, Inc. (Critique of the Flood Insurance Study, 2000).

Since the differences in rainfall data between 10-day and 24-storm events, as well as between TP40/TP49 and USGS rainfall data, don't make a 3 to 4 times differences in peak flow discharges, I have further investigated the methods of estimating Tc or LAG. By comparing the travel time method with the SCS LAG equation $L^{0.8}[(100\text{-}CN)\text{-}9]^{0.7}/(1900xS^{0.5})$, there is a significant difference in time of concentration for most of the subbasins. With the SCS LAG equation, the unit hydrograph (UH) could be more than doubled for some subbasins.

If it is available, please email me a copy of the USACE' comments in dealing with the travel time methd.

Please let me know if you have any questions.

Thanks.

Deren Li, Ph.D., P.E., D.WRE, CFM
Civil Systems Engineering Inc.
9894 Bissonnet St., Suite 404
Houston, Texas 77036
713-298-6819 (c)
713-782-3811 (o)
www.cseengineers.com

Regarding the CSE comments and S&B responses, I believe the two most critical ones are Number 2 and Number 7, both dealing with peak flows for the project. Since project peak flows (existing and post conditions) directly dictate the sizes of the project components and economic justification of the project, further documentation and investigations are needed.

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The average factor of 1.35 was correctly utilized to account for the increase in peak runoff due to economic growth factors obtained from the Texas Water Development Board and projected population as determined by the Texas State Data Center, the Office of the State Demographer and Census 2000 data. Concerning "variation of developments", this is not applicable due to USACE direction to S&B. According to USACE guidelines sent to S&B, *"The existing land use pattern will be assumed to continue in the same proportion throughout the watershed".* Since the majority of the watershed is not zoned for future development, it is not feasible to attempt to prepare detailed development estimates over these watersheds. Any attempt will be solely based on the preparer's opinion. As such, S&B did make preliminary estimates based on population forecasts for a selective number of sub-basins. A hydrologic analysis was performed to determine the average increase in runoff. The increase varied in range from 28% to 32%. Based on the analysis, a factor of 35% was selected as an estimate on the increases in future peak flow runoff.

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During the base conditions HEC-HMS development of the Raymondville Drain and North Main Drain watersheds, a discussion was conducted with the USACE concerning the methodology for determining the lag time of the individual sub-basins. In S&B's original analysis, the SCS CN lag time equation was utilized. However, the USACE felt that an "*accurate SCS lag was needed because the unit hydrograph was already flattened by the adjustment to the peaking factor. If* *an excessively long lag was used in combination with a reduced peaking factor, then an unrealistically low peak flow rate would likely occur for each sub-basin."* The USACE felt that by using the velocity method to calculate the lag time, the shorter times of concentration would yield *"more reasonable HMS results"*. Additionally, the USACE assumed an open channel flow velocity of 0.6 fps for their analysis. In order to refine this further, S&B utilized LIDAR data to prepare actual typical sections to calculate the open channel flow velocity for each sub-basin. It was because of this extra step that the USACE chose to utilize S&B values.

If it is available, please email me a copy of the USACE' comments in dealing with the travel time methd.

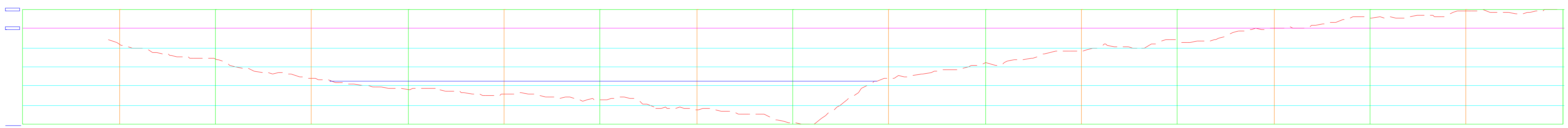
This information was found in the USACE Raymondville Pre-Project Report dated 04-11-2006, which was provided at the meeting in S&B's office on August 23. We will email you another copy.

Sub Watershed

Definitions:

V = Average Velocity
R = Hydraulic radius (ft) and is equal to A/P_w
A = Cross Section Flow Area (ft^2)
P_w = Wetted perimeter (ft)
S = Slope of the hydraulic grade line (ft/ft)
n = Manning Roughness coefficient for open channel flow

R1000W1000



A = 282.1808 ft^2
P_w = 284.0212 ft

R = $\frac{282.1808}{284.0212} = 0.9935$ ft

S = 0.003 ft/ft
n = 0.1

V = $\frac{1.49R^{\frac{2}{3}}\sqrt{S}}{n} = \frac{\left(1.49(0.9935)^{\frac{2}{3}}\sqrt{0.003}\right)}{0.10} = 0.8126\ ft/\sec$

Lengths used:
Total Length: L_{total} = 31596.34 ft
Sheet flow Length: L_{sheet} = 500 ft
Shallow Flow Length: $L_{shallow}$ = 0.15* L_{total} = $0.15 * 31596.34 = 4739.45$ ft
Channel Flow Length: $L_{channel}$ = $31596.34 - 4739.45 - 500 = 26356.89$ ft

Time of Concentration:

$\mathbf{T_{Total}=T_{Sheet}+T_{Shallow}+T_{Channel}}$

$$T_{sheet} = \frac{0.007(nL_{sheet})^{0.8}}{(S^{0.4})\sqrt{2p}} = \frac{0.007(0.1\cdot 500)^{0.8}}{(0.003^{0.4})\sqrt{4.375}} = 0.8Hours$$

$$T_{shallow} = \frac{L_{shallow}}{3600V} = \frac{4739.45}{(3600)(16.1345\sqrt{S})} = \frac{4739.45}{(3600)(16.1345\sqrt{0.003})} = 1.49Hours$$

$$T_{channel} = \frac{L_{channel}}{3600V} = \frac{26356.89}{(3600)(0.8126)} = 9.01Hours$$

$$T_{total} = 0.8 + 1.49 + 9.01 = 11.3Hours$$

Appendix C
FEMA LOMR (May 17, 2001)

Federal Emergency Management **Agency**

Washington, D.C. 20472

MAY 17 2001

CERTIFIED MAIL
RETURN RECEIPT REQUESTED

IN REPLY REFER TO:

Case Number:	01-06-1095P
Community Name:	Hidalgo County (Unincorporated areas), Texas
Community Number:	480334
Panels Affected:	480334 0325 D, 0350 C
Effective Date of This Revision:	MAY 17 2001

102-I-A-C

The Honorable Jose Eloy Pulido
Hidalgo County Judge
P.O. Box 1356
Edinburg, TX 78540

Dear Judge Pulido:

The Flood Insurance Study (FIS) and Flood Insurance Rate Map (FIRM) for the Unincorporated Areas of Hidalgo County, Texas, have been revised by this Letter of Map Revision (LOMR) to reflect revised hydrologic and hydraulic analyses, and more accurate topographic information. The subject area is located along the North Main Drain from Monte Christo Road to the confluence with Donna Drain. This project also affects flood hazard information for the City of Edinburg, Texas. This revision was initiated by [redacted], Floodplain Administrator, Hidalgo County, Texas, with his request dated March 30, 2001.

On January 31, 2001, a LOMR was issued for Hidalgo County, Texas, that removed several areas along the North Main Drain from the Special Flood Hazard Area (SFHA), the area inundated by the flood having a 1% chance of being equaled or exceeded in any given year (base flood). This LOMR supersedes the January 31, 2001, LOMR. On February 28, 2001, another LOMR was issued that removed the regulatory floodway from FIRM number 480334, panels 0290 D, 0295 D, 0325 D, 0350 C, and 0450 C. The floodway removal established by the February 28, 2001, LOMR remains in effect. The maps accompanying this LOMR now reflect the removal of the regulatory floodway from FIRM number 480334, panels 0325 D and 0350 C, as described in the February 28, 2001, LOMR.

We received the revised hydrologic and hydraulic analyses, and updated topographic information that was prepared by Turner, Collie and Braden in support of this revision. All of the data required to perform the technical review of this request were received as of May 9, 2001.

Based on our review of the submitted data, we are issuing this LOMR to reflect the revised hydrologic and hydraulic analyses, and updated topographic information. This LOMR revises areas adjacent to the North Main Drain from Monte Christo Road to its confluence with Donna Drain. As a result of the revised analyses there are increases and decreases in the elevation of the flood having a 1% chance of being equaled or exceeded in any given year. The Base (1% annual chance) Flood Elevation (BFE) will decrease from 92.8 feet to 90.7 feet at the upstream end and from 74.2 feet to 68.7 feet at the downstream end. The maximum decrease, 5.5 feet, occurs just upstream of the confluence with Donna Drain, as shown on the enclosed annotated portions of FIRM number 480338, panels 0325 D and 0350 D, and flood profiles. The BFE increase extends from about 600 feet to 3,500 feet upstream of North Alamo Road. The maximum increase, 0.55 feet, occurs approximately 3,000 feet upstream of North Alamo Road. The width of the SFHA has also decreased along North Main Drain. The maximum SFHA decrease, 13,800 feet, occurs approximately 2,500 feet downstream of North Alamo Road. The submitted information indicated that the corporate limits for your community have changed because of annexations. We have reflected these corporate-limit changes in this LOMR.

This revision is effective as of the date of this letter. However, due to the changes in the BFEs, a 90-day appeal period is required. If FEMA receives an appeal of the BFEs presented in this LOMR during the 90-day appeal period, it may be required that this LOMR be rescinded. If the LOMR is rescinded, any flood insurance policies or building permits issued in the revised areas subsequent to the issuance of this LOMR will have to be re-assessed.

We have enclosed a copy of the public notification of the revised BFEs, which will be published in ***The Monitor*** on or about May 24,2001, and May 31,2001. In addition, we will publish a notice of changes in the ***Federal Register.*** However, we will not print and distribute this LOMR to users, such as insurance agents or lenders. Your community will serve as a repository for the new data. Therefore, we encourage you to supplement the notification to appear in ***The Monitor*** by preparing a news release for publication in your community newspaper that describes the revision and explains how your community will provide data, and help interpret the National Flood Insurance Program (NFIP) maps. In that way, interested persons, such as property owners, insurance agents, and mortgage lenders, can benefit from the information.

Within 90 days of the second publication in ***The Monitor,*** any interested party may request that we reconsider this determination. Any request for reconsideration must be based on scientific or technical data. However, until the 90-day period ,elapses, the revised BFEs presented in this LOMR may be changed.

We based this determination on the 1% annual chance discharges computed in the revised hydrologic model. Future development of projects upstream could cause increased discharges, which could cause increased flood hazards. A comprehensive restudy of your community's flood hazards would consider the cumulative effects of development on discharges and could, therefore, establish greater flood hazards in this area.

This LOMR is based on minimum floodplain management criteria established under the NFIP. Your community is responsible for approving all floodplain development and for ensuring all necessary permits required by Federal or State law have been received. State, county, and community officials, based on knowledge of local conditions and in the interest of safety, may set higher standards for construction in the SFHA. If the State, county, or community has adopted more restrictive or comprehensive floodplain management criteria, these criteria take precedence over the minimum NFIP criteria.

We will not physically revise and republish the FIRM and FIS report for your community to reflect the modifications made by this LOMR at this time. When changes to the previously cited FIRM panels and FIS report warrant physical revision and republication in the future, we will incorporate the modifications made by this LOMR at that time.

Use the map panels listed above and revised by this letter for flood insurance policies and renewals issued in your community.

We have enclosed an updated version of a document titled *List of Current Flood Insurance Study Data,* which includes this letter, to help your community maintain all information for floodplain management and flood insurance. If any of the items in this document are not filed in your community's map repository, please contact the Federal Emergency Management Agency (FEMA) Map Assistance Center at the telephone number listed below for information on how to obtain those items.

We have made this determination pursuant to Section 206 of the Flood Disaster Protection Act of 1973 (P.L. 93-234) and in accordance with the National Flood Insurance Act of 1968, as amended (Title XIII of the Housing and Urban Development Act of 1968, P.L. 90-448), 42 U.S.C. 4001-4128, and 44 CFR Part 65. Pursuant to Section 1361 of the National Flood Insurance Act of 1968, as amended, communities participating in the NFIP are required to adopt and enforce floodplain management ordinances that: meet or exceed minimum NFIP criteria. These criteria, including adoption of the FIS and FIRM, and modifications made by this LOMR, are the minimum requirements for continued NFIP participation and do not supersede more stringent State or local requirements to which the regulations apply.

If you have any questions regarding this LOMR., please contact the FEMA Map Assistance Center, toll free, at 1-877-FEMA MAP (1-877-336-2627).

Sincerely,

William R Blanton Jr

William R. Blanton, Jr., Prqject Officer Hazards Study Branch Mitigation Directorate	For: Matthew B. Miller, P.E., Chief Hazards Study Branch Mitigation Directorate

Enclosures

cc: Mr. Godfrey Garza
Floodplain Administrator, Hidalgo County, Texas,

LIST OF CURRENT FLOOD INSURANCE STUDY DATA

This list is provided to document all information. currently effective for your community for insurance and floodplain management.

Date:	May 17,2001
Community:	Hidalgo County, Texas (Unincorporated Areas)
Community Number:	480334
Page Number:	1 of 2

CURRENT EFFECTIVE FLOOD INSURANCE STUDY DATE: June 6,2000

FLOOD INSURANCE RATE MAP

Index Date: June 6,2000

Panel Numbers	Effective Date
480334 0290D, 480334 0295D, 480334 0300D, 480334 0325D, 480334 0350C, 480334 0450C,	June 6,2000
480334 0400C, 480334 0425C,	November 11,1982
480334 0025B, 480334 0050B, 480334 0075B, 480334 0100B, 480334 0125B, 480334 0150B, 480334 0175B, 480334 0200B, 480334 0225B, 480334 0250B, 480334 0275B, 480334 0375B, 480334 0475B, 480334 0500B, 480334 0525B,	January 2,1981

LETTERS OF MAP REVISION

Panel Numbers	Effective Date
480334 0325 D, 480334 0350 C	May 17,2001
480334 0400 C	February 26,1991
480334 0375 B	September 21,1998

LETTERS OF MAP AMENDMENT AND MAP :REVISIONBASED ON FILL

Panel Numbers	Effective Date
480334 0425C	November 2,1998
480334 0425C	December 20, 1999
480334 0300D	May 11,2000
480334 0450C	August 9,2000
480334 0290D	October 25,2000
480334 0295D	January 17,2001
480334 0290D	January 17,2001
480334 0300D	March 7,1996
480334 0300D	August 26,1996

BEST AVAILABLE DATA LETTERS

None

CHANGES *ARE* MADE IN DETERMINATIONS OF BASE FLOOD ELEVATIONS FOR THE UNINCORFRATED AREAS OF HIDALGO COUNTY, TEXAS, UNDER THE NATIONAL FLOOD INSURANCE PROGRAM

On June 6,2000, the Federal Emergency Management Agency (FEMA) identified Special Flood Hazard Areas (SFHAs) in the Unincorporated Areas of Hidalgo County, Texas, through issuance of a Flood Insurance Rate Map (FIRM). The Mitigation Directorate has determined that modification of the elevations of the flood having a 1-percent chance of being equaled or exceeded in any given year (base flood) for certain locations in this community is appropriate. The modified base flood elevations (BFEs) revise the: FIRM for the community.

The changes are being made pursuant to Section 206 of the Flood Disaster Protection Act of 1973 (Public Law 93-234) and are in accordance with the National Flood Insurance Act of 1968, as amended (Title XIII of the Housing and Urban Development Act of 1968, Public Law 90-448), 42 U.S.C. 4001-4128, and 44 CFR Part 65.

A hydraulic analysis was performed to incorporate revisions to the 1% annual chance discharges and resulting hydraulic analysis and has resulted in, a reduction in SFHA, and increases and decreases to the BFEs for the North Main Drain from Monte Christo Road to its confluence with Donna Drain. The table below indicates existing and modified BFEs for selected locations along the affected lengths of the flooding source(s) cited above.

Location	Existing BFE (feet)*	Modified BFE (feet)*
Just downstream of Monte Christo Road	92.8	90.7
Approximately 3,000 feet upstream of North Alamo Road	81.4	81.9
Just upstream of Donna Drain	74.2	68.7

*National Geodetic Vertical Datum, rounded to nearest whole foot

Under the above-mentioned Acts of 1968 and 1973, the Mitigation Directorate must develop criteria for floodplain management. For the community to participate in the National Flood Insurance Program (NFIP), the community must use the modified BFEs to administer the floodplain management measures of the NFIP. These modified BFEs will also be used to calculate the appropriate flood insurance premium rates for new buildings and their contents and for the second layer of insurance on existing buildings and contents.

Upon the second publication of notice of these changes in this newspaper, any person has 90 days in which he or she can request, through the Chief Executive Officer of the community, that the

Mitigation Directorate reconsider the determination. Any request for reconsideration must be based on knowledge of changed conditions or new scientific or technical data, All interested parties are on notice that until the 90-day period elapses, the Mitigation Directorate's determination to modify the BFEs may itself be changed.

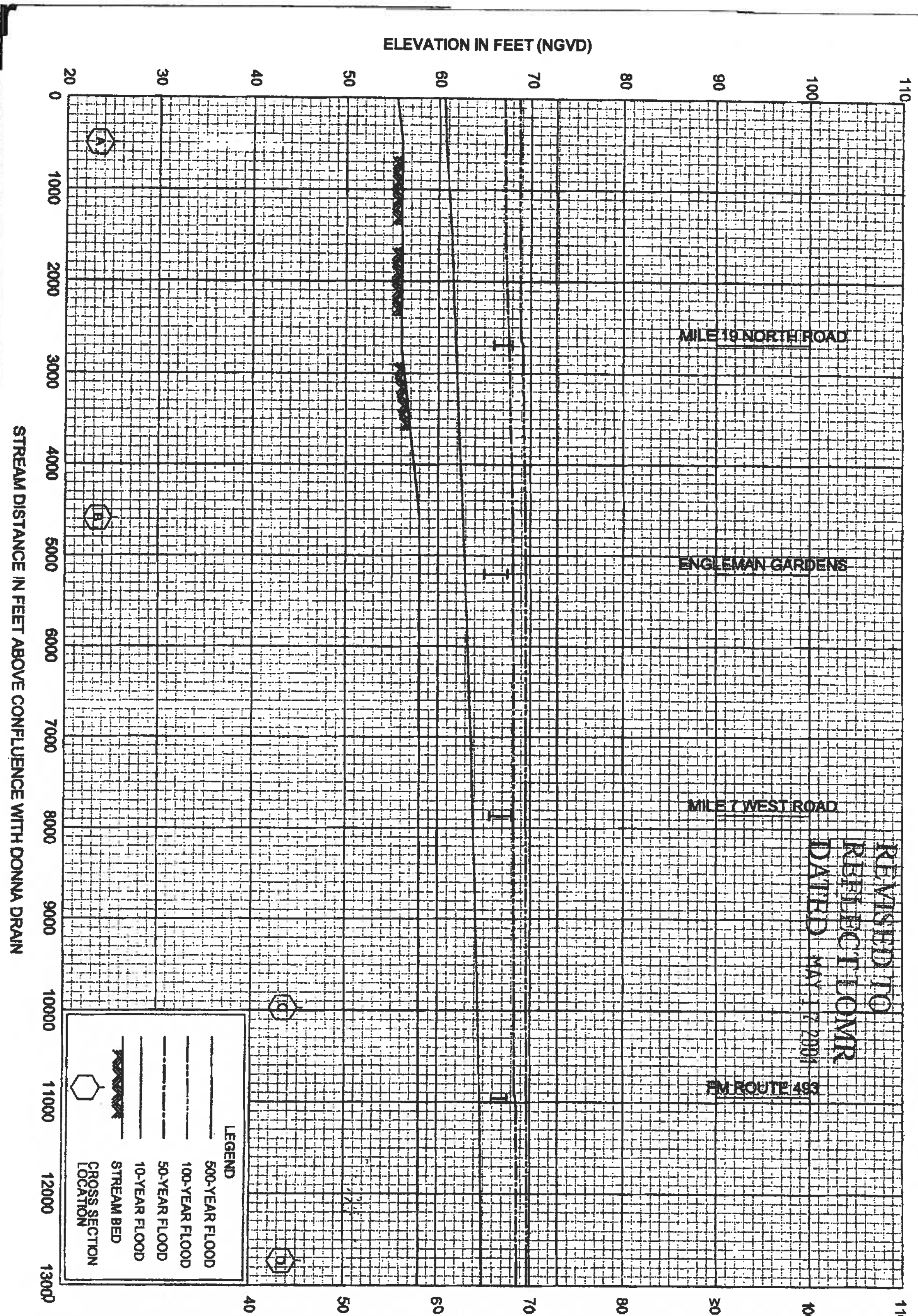
Any person having knowledge or wishing to comment on these changes should immediately notify:

The Honorable Jose Eloy Pulido
Hidalgo County Judge
P.O. **Box** 1356
Edinburg, TX 78540

Table 1. Summary of Discharges

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cfs) 10-Year	50-Year	100-Year	500-Year
East Lateral Drain					
At outfall to IBWC floodway	30.53	1,295	2,036	2,556	3,629
At Mile 2 West Road	24.94	1,415	2,049	2,428	3,267
At elevated canal crossing	13.96	1,172	1,890	2,321	3,278
At Mile 11 North Road	7.36	1,040	1,615	1,953	2,702
North Main Drain					
At downstream Junction of North and South Main Drains	318.10	653	2,565	4,001	10,034
At Doolittle Road	288.78	554	2,258	3,818	10,010
At Ingle Road	277.30	555	2,215	3,804	9,926
At Seminary Road	264.72	527	2,355	4,178	10,347
At FM 1925	208.28	484	1,985	3,435	8,788
At upstream junction of North and South Main Drains	105.61	158	1,051	1,858	4,804
West Main Drain					
At junction of West Main Canal and Mission-McAllen Lateral	92.88	1,081	3,336	0,809	7,700
At confluence of United Irrigation Canal	28.22	148	727	1,128	2,072
At Brushliner Road	18.40	85	468	731	1,346

REVISED TO
REFLECT LOMR
DATED MAY 17 200



04P

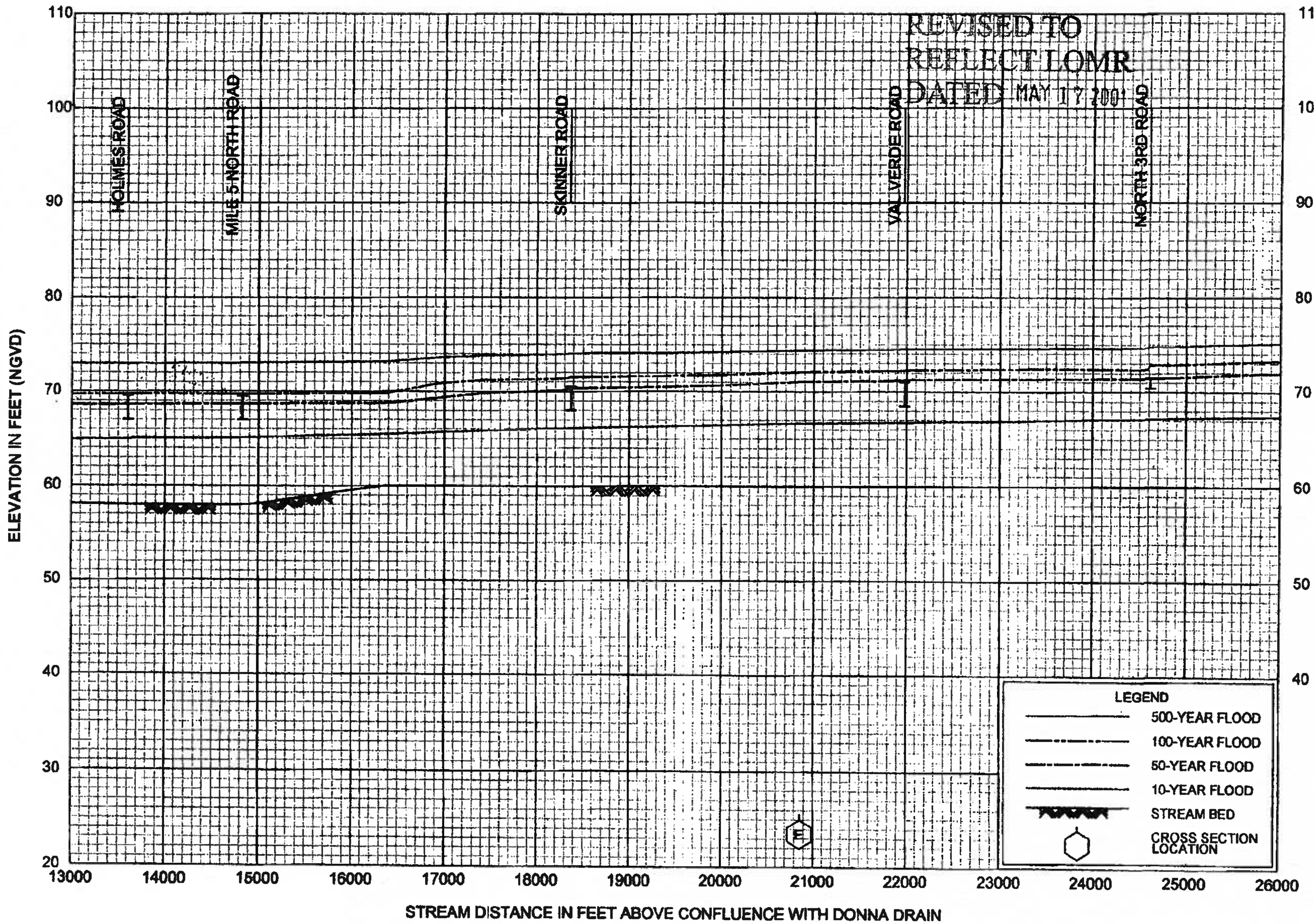
FEDERALEMERGENCYMANAGEMENTAGENCY

HIDALGO COUNTY, TX

(UNICORPORATEDAREAS)

FLOOD PROFILES

NORTH **MAIN DRAIN**



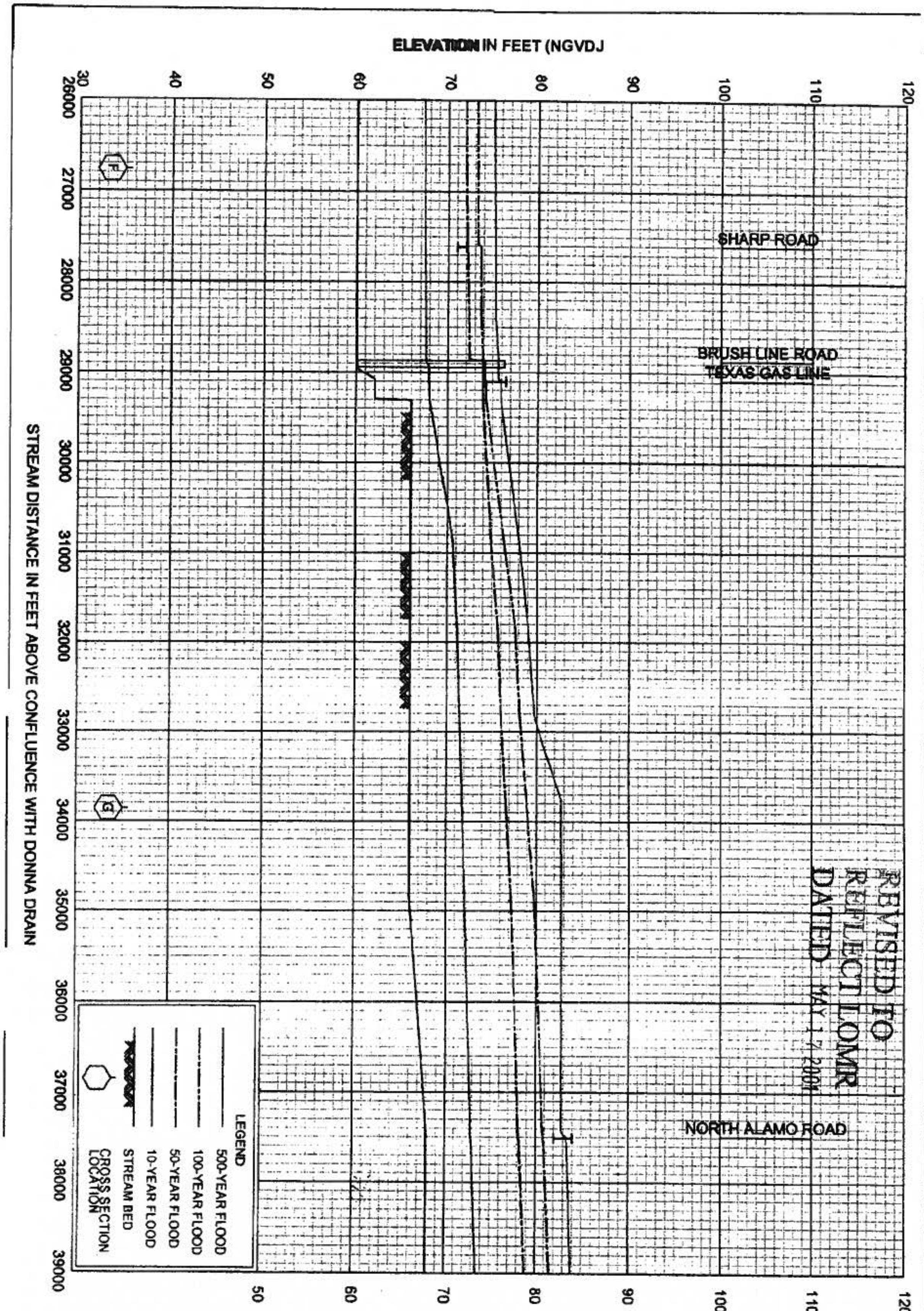
FEDERAL EMERGENCY MANAGEMENT AGENCY

HIDALGO COUNTY, TX
(UNICORWRATEDAREAS)

FLOOD PROFILES

NORTH MAIN DRAIN

05P



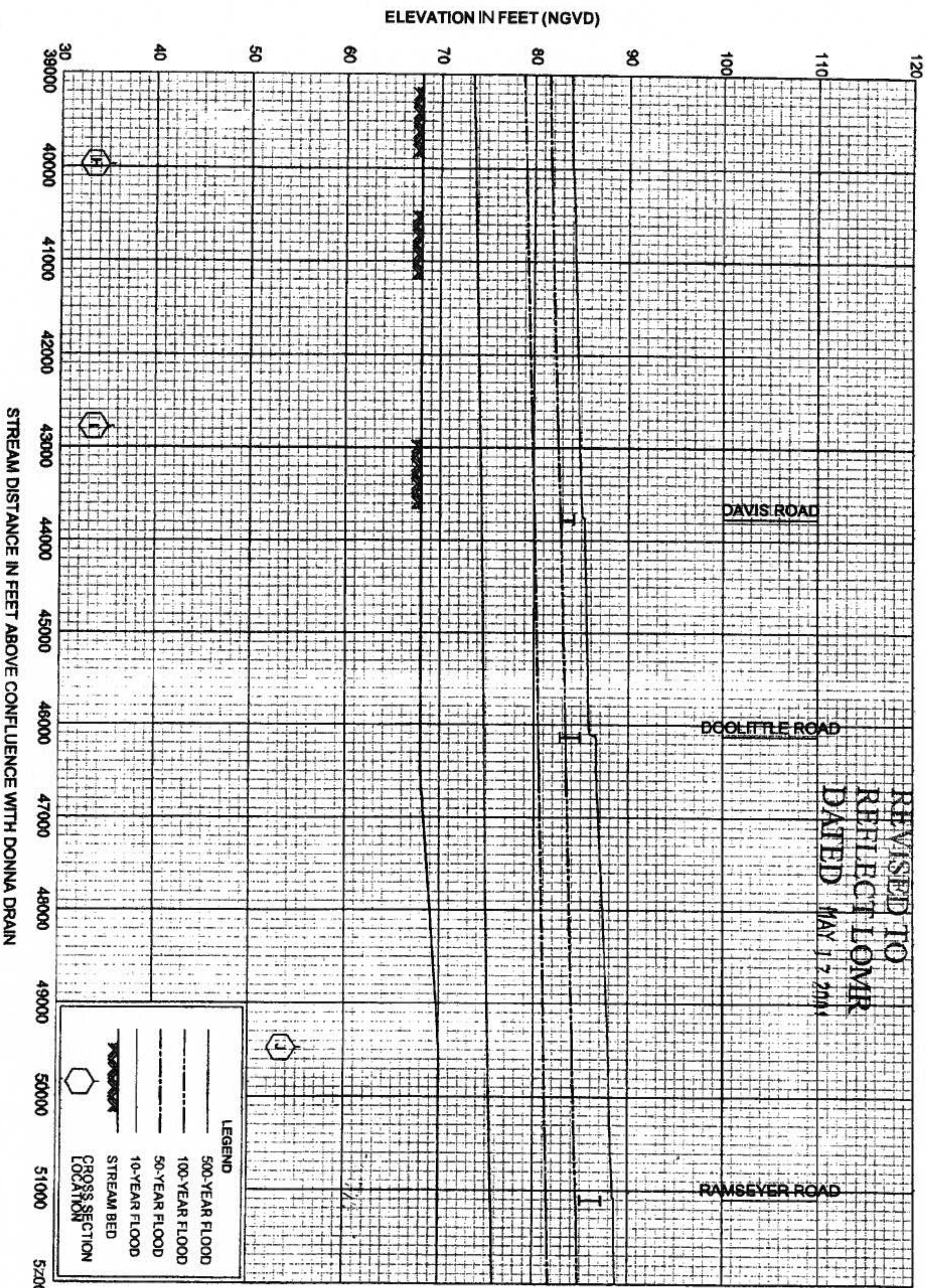
FEDERAL EMERGENCY MANAGEMENT AGENCY

HIDALGO COUNTY, TX

(UNICORPORATED AREAS)

FLOOD PROFILES

NORTH MAIN DRAIN



07P

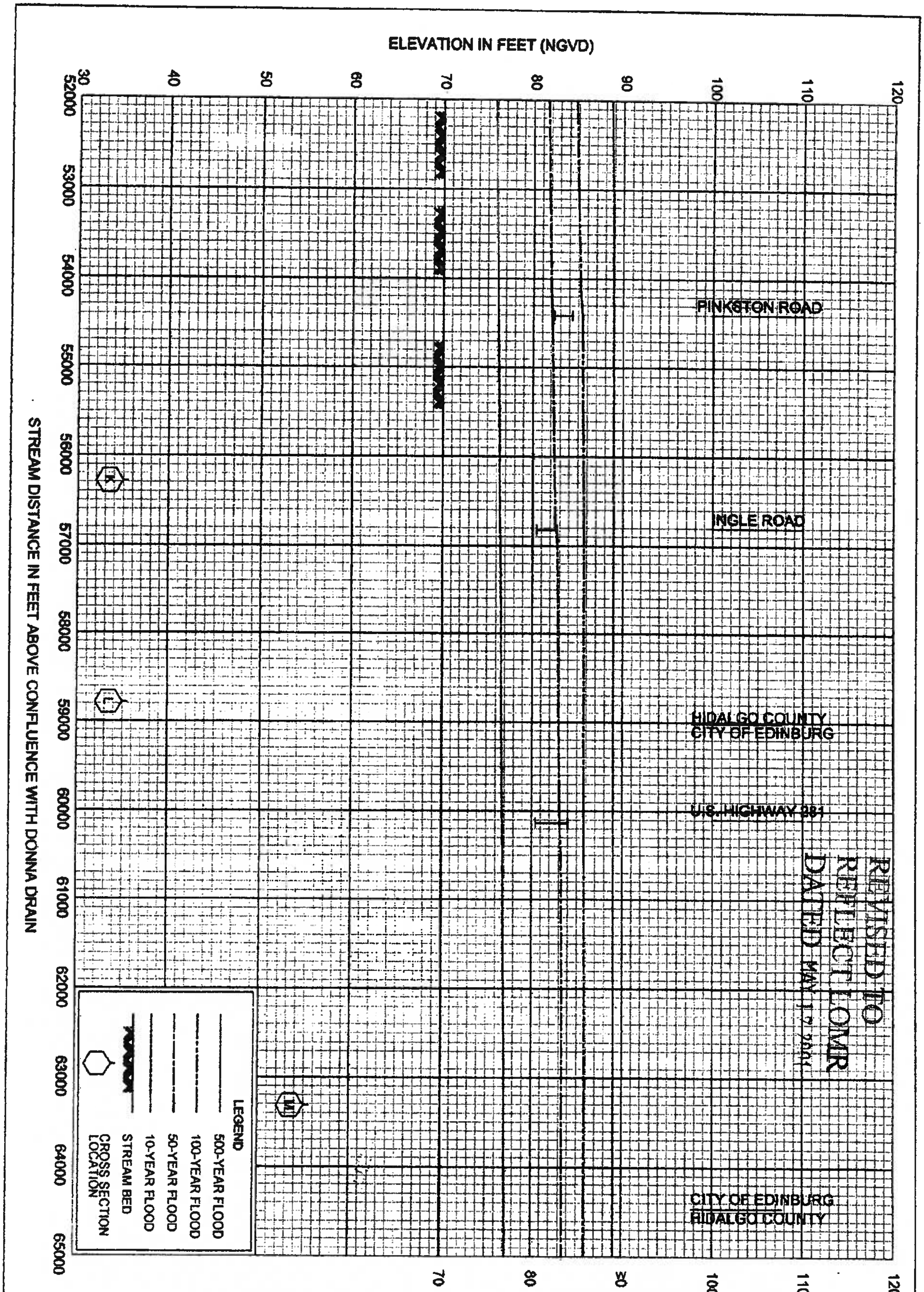
FEDERAL EMERGENCY MANAGEMENT AGENCY

HIDALGO COUNTY, TX

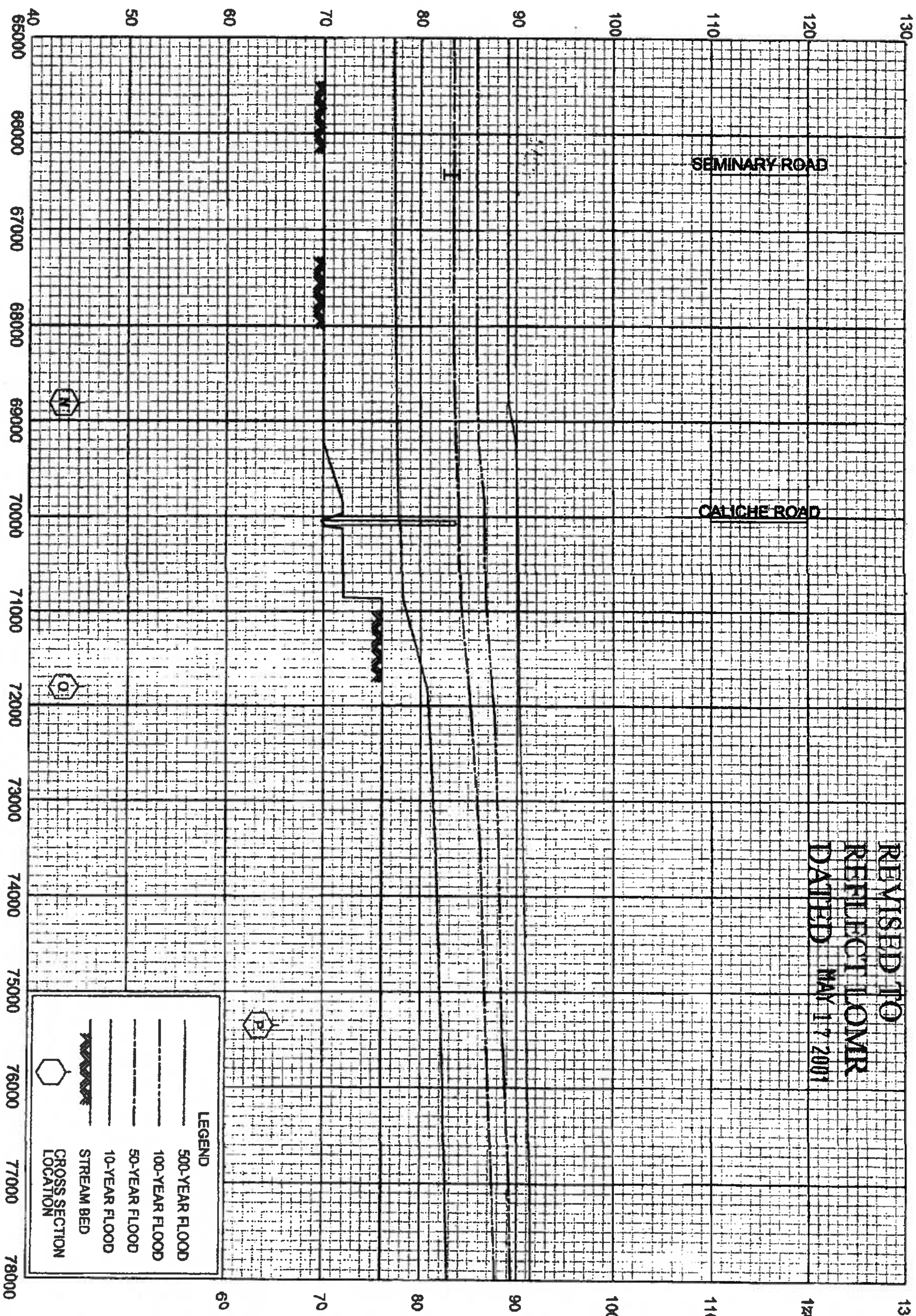
(UNICORPORATED AREAS)

FLOOD PROFILES

NORTH MAIN DRAIN



FEDERAL EMERGENCY MANAGEMENT AGENCY	FLOOD PROFILES
HIDALGO COUNTY, TX (UNICORPORATEDAREAS)	NORTH MAIN DRAIN



FEDERAL EMERGENCY MANAGEMENT AGENCY

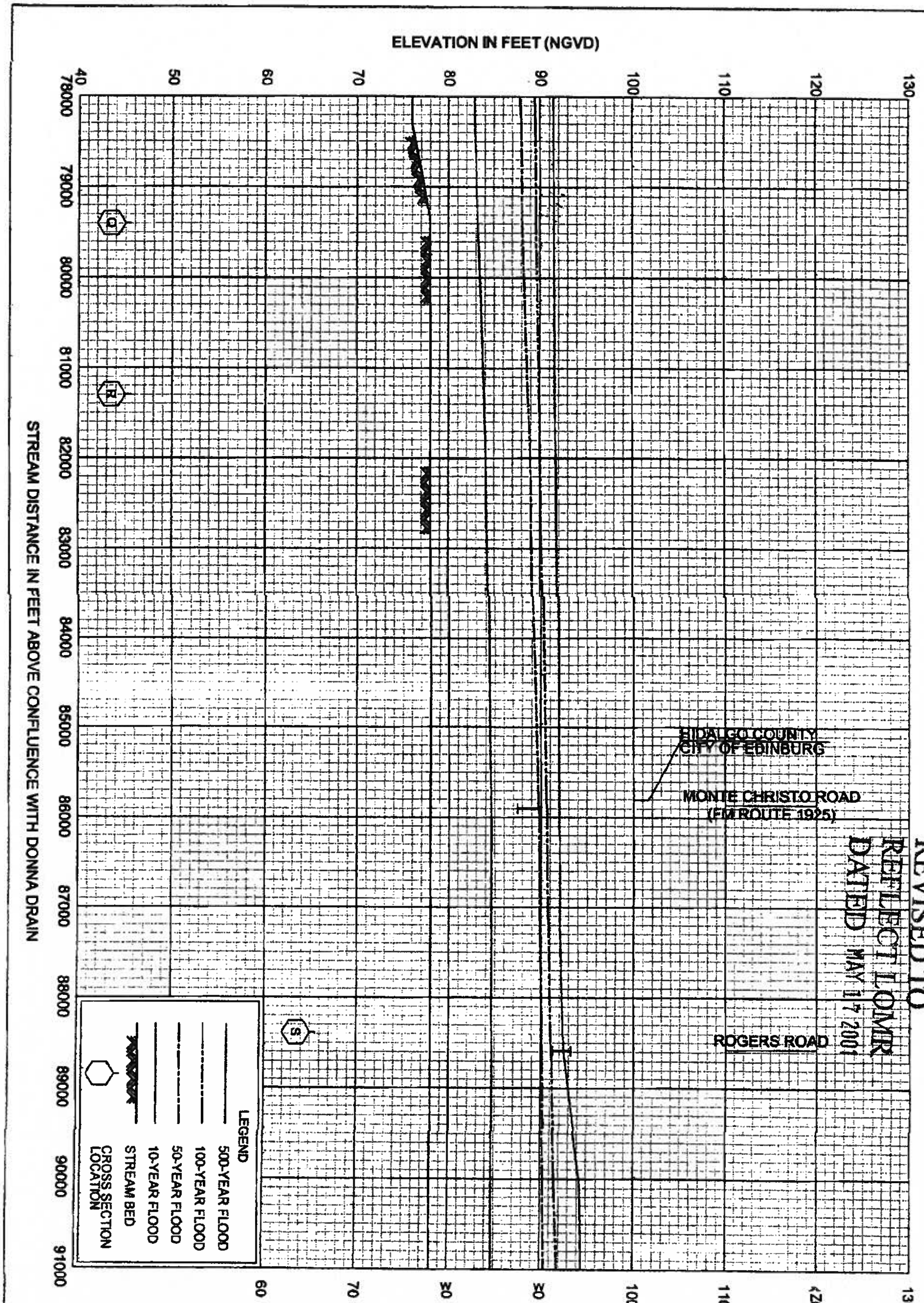
HIDALGO COUNTY,,TX

(UNICORPORATEDAREAS)

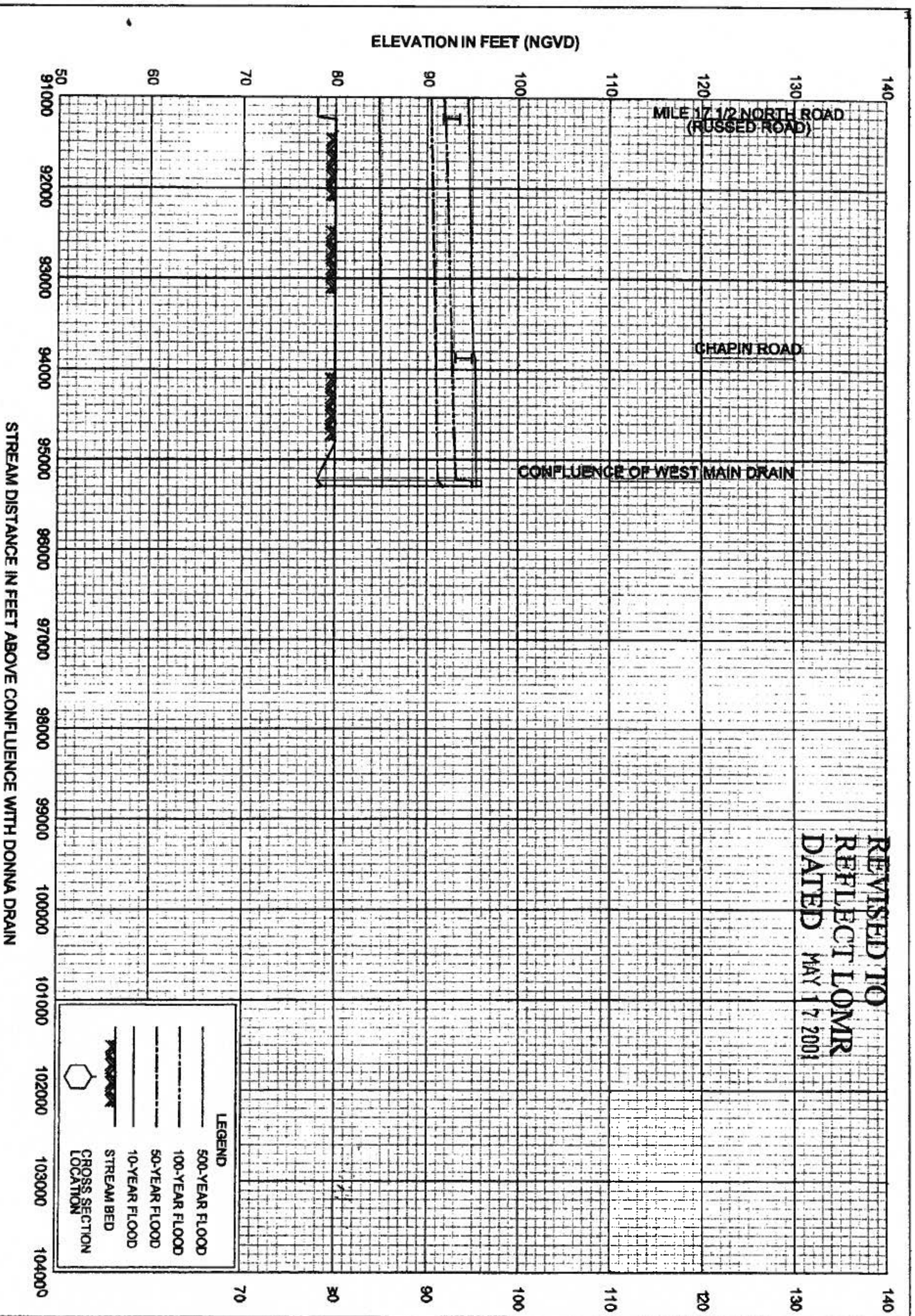
FLOOD PROFILES

NORTH MAIN DRAIN

09P



10P	FEDERALEMERGENCYMANAGEMENTAGENCY **HIDALGO COUNTY, TX** (UNICORPORATEDAREAS)	FLOOD PROFILES **NORTH MAIN DRAIN**



FEDERAL EMERGENCY MANAGEMENTAGENCY

HIDALGO COUNTY, TX

(UNICORPORATEDAREAS)

FLOOD PROFILES

NORTH MAIN DRAIN

11P

NOTICE TO SUBSCRIBERS

SOME ATTACHMENTS TO THIS LETTER OF MAP REVISION WERE TOO LARGE TO BE INCLUDED IN THIS PACKAGE. FOR COPIES OF THESE ATTACHMENTS, FEE OF ADDITIONAL CHARGE, PLEASE CONTACT THE LOMC DISTRIBUTION COORDINATOR AT THE ADDRESS BELOW:

LOMC DISTRIBUTION COORDINATOR
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12101 INDIAN CREEK COURT
BELTSVILLE, MARYLAND 20716

Appendix D
Overall Comparison of Modeling Methodologies

COMPARISON BETWEEN 2001 AND 2011 METHODOLOGY
FOR
HYDROLOGY AND HYDRAULIC MODELING

HYDROLOGY					
MODEL	**VARIABLE**	**2000 S&B REPORT RECOMMENDATIONS**	**2001 TCB**	**2011 S&B**	**2011 S&B FDA**
PROGRAM SOFTWARE	HEC	N/A	HEC-1	HEC-GeoHMS & HEC-HMS	HEC-GeoHMS & HEC-HMS
SUB-AREA DELINEATION	A	N/A	**1927 DATUM** U.S.G.S. QUADRANGLE MAPS	GEO-HMS USED 2004-2008 HIDALGO COUNTY LIDAR DATA, SITE RECONNAISSANCE, FIELD SURVEYS, USGS MAPPING	GEO-HMS USED 2004-2008 HIDALGO COUNTY LIDAR DATA, SITE RECONNAISSANCE, FIELD SURVEYS, USGS MAPPING
LAND USE	N/A	N/A	FEBRUARY 1995 USGS QUAD DOQQ'S (AERIALS)	2010 AERIALS AND COMMUNITY ZONING MAPS FOR PROJECTED LAND USE	2010 AERIALS AND COMMUNITY ZONING MAPS FOR PROJECTED LAND USE
HSG SOILS	TYPES A,B,C,D	NOVEMBER 1956 BUREAU OF RECLAMATION STUDY AND MELDEN AND HUNT PERMEABILITY MAP (EXHIBIT 5)	NOVEMBER 1956 BUREAU OF RECLAMATION STUDY AND MELDEN AND HUNT PERMEABILITY MAP-BLANKET 3" FOR CROPLAND, AND PASTURELAND. 0" FOR URBAN	DELINEATED SOIL PERCENTAGE BY SUBAREA AND CALCULATED ON USDA MAPPING SOFTWARE	DELINEATED SOIL PERCENTAGE BY SUBAREA AND CALCULATED ON USDA MAPPING SOFTWARE
CURVE NUMBER	CN	N/A	SCS AMCI (DRY)	SCS AMCI (DRY)	SCS AMCI (DRY)
SUB-AREA SLOPE	S	N/A	,05% FOR ENTIRE WATERSHED	GEO-HMS CALCULATED FLOW PATHS, LENGTHS AND SLOPE FOR EACH SUBAREA	GEO-HMS CALCULATED FLOW PATHS, LENGTHS AND SLOPE FOR EACH SUBAREA
FLOW LENGTHS/PATHS	L	N/A	U.S.G.S. QUADRANGLE MAPS	GEO-HMS CALCULATED FLOW PATHS, LENGTHS AND SLOPE FOR EACH SUBAREA	GEO-HMS CALCULATED FLOW PATHS, LENGTHS AND SLOPE FOR EACH SUBAREA
TIME OF CONCENTRATION	TC	N/A	LAG EQUATION	VELOCITY METHOD WITH SHEETFLOW, SHALLOW CONCENTRATED, AND CHANNEL FLOW (PULLED XS FROM LIDAR AND ITERATED FLOWS FOR VELOCITY)	VELOCITY METHOD WITH SHEETFLOW, SHALLOW CONCENTRATED, AND CHANNEL FLOW (PULLED XS FROM LIDAR AND ITERATED FLOWS FOR VELOCITY)

INITIAL ABSTRACTION	IA	J.E. SAENZ AND MELDEN AND HUNT WATERSHED FEATURES MAP (EXHIBITS 6 AND 7)	HEC1 DAM BREACH STORAGE AREAS AT ELEVATED CANALS (SAENZ AND HUNT EXHIBITS 6 AND 7)	NRCS HEC-HMS **0.2*S** STANDARD METHODOLOGY (SOIL RETENTION ***S*** =1000/CN-10)	NRCS HEC-HMS **0.2*S** STANDARD METHODOLOGY (SOIL RETENTION ***S*** =1000/CN-10)
STORAGE ROUTING	N/A	N/A	8 POINT XS MUSKINGUM CUNGE IN AREAS WITHOUT HYDRAULIC MODEL MODIFIED PULS IN HECRAS LOCATIONS	8 POINT XS MUSKINGUM CUNGE IN AREAS WITHOUT HYDRAULIC MODEL AND HECRAS LOCATIONS WHERE SLOPE WAS LESS THAN .0004, MODIFIED PULS IN HECRAS LOCATIONS AND WESTERN HMS REACHES	8 POINT XS MUSKINGUM CUNGE IN AREAS WITHOUT HYDRAULIC MODEL AND HECRAS LOCATIONS WHERE SLOPE WAS LESS THAN .0004, MODIFIED PULS IN HECRAS LOCATIONS AND WESTERN HMS REACHES
RAINFALL DATA SOURCE	N/A	BEULAH WITH RAINFALL PEAK TOTAL OF 7.52"	USGS 2001 DATA	USGS 2004 DATA	TP40 AND TP49
RAINFALL DISTRIBUTION	N/A	N/A	PRECIPITATION DATA INCREMENTS (HEC1)	PRECIPITATION DATA INCREMENTS (FREQUENCY STORM HEC-HMS)	PRECIPITATION DATA INCREMENTS (FREQUENCY STORM HEC-HMS)
RAINFALL DURATION	D	N/A	24 HOUR	24 HOUR	10-DAY
STORM AREA REDUCTION	N/A	N/A	N/A	N/A	AT POINTS OF INTEREST
PARTIAL/ANNUAL 2YR,5YR,10YR RAINFALL ADJUSTMENT	N/A	N/A	N/A	N/A	2,5, AND 10YR
HYDRAULICS					
MODEL	**VARIABLE**	**2000 S&B REPORT**	**2001 TCB**	**2011 S&B**	**2011 S&B FDA**
PROGRAM SOFTWARE	N/A		HEC-RAS	HEC-RAS & HEC-GeoRAS	HEC-RAS & HEC-GeoRAS
FLOW/DISCHARGES	Q		FROM HEC-1	FROM HEC-HMS	FROM HEC-HMS WITH STORM AREA REDUCTIONS
CROSS SECTIONS	N/A	RECOMMENDED FIELD CROSS SECTIONS OF CHANNEL	1995 DIGITAL TERRAIN MODEL WITHOUT CHANNEL SURVEY (FLOWLINE IS WATER SURFACE)	2004 AND 2008 LIDAR AND 2006-2010 FIELD SURVEY OF CHANNELS	2004 AND 2008 LIDAR AND 2006-2010 FIELD SURVEY OF CHANNELS
BRIDGES/CULVERTS	N/A		FEBRUARY 1995 DOQQ'S (AERIALS) AND ASBUILTS (1970) (MODEL ASSUMED BRIDGE LOSSES MINIMAL AND NOT INCLUDED)	FIELD SURVEY & AS-BUILTS (1970-2008)	FIELD SURVEY & AS-BUILTS (1970-2008)
ROUGHNESS COEFFICIENTS	N		FROM FIELD DATA	FROM FIELD DATA	FROM FIELD DATA

Appendix E
Photo Diary of Edinburg Lake Area

Photo 001
Looking South at Relief Channel

Photo 002
Looking North at Relief Channel

Photo 003

Looking Northwest at 24-inch Pipe Outlet

Photo 004
Looking Southwest at 24-inch Pipe Inlet

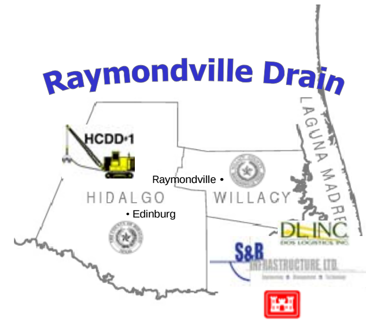

Photo 005
Looking West Near Curve of Relief Channel

Raymondville Drain Project, **Project for Flood Control**
Lower Rio Grande Basin, (Hidalgo & Willacy Counties), Texas
Water Resources Development Act (WRDA) 1986, Title IV, Section 401, As amended by WRDA 2007

Hidalgo County Contract No. 2010-164-04-20

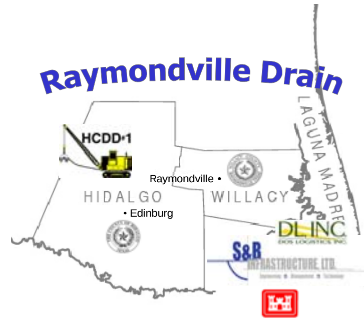

Photo 006
Looking South at Access Road

Photo 007
Looking East at Berm of Lake

Photo 008
Looking North at Access Road

Photo 009

On Road, Looking West at Channel Near Curve of Channel

Appendix F
S&B 2011 North Main Drain Model (24-HR HMS)