Travel Time Estimation of the Lower Rio Grande Water Transportation Model

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Abstract: **The prediction of a pollutant spill on downstream water quality is dependent on the water velocity, longitudinal mixing, and seasonal effects on water inflows, where velocity is the most important, but most difficult, aspect. This paper provides a study on the calculation of travel-time from one bank discharge to another by using a one dimension Lagrangian river model. A one dimension Lagrangian river model was developed and applied to the 210 plus miles for the lower Rio Grande River Basin from the Falcon Dam to the head water of Brownsville that pours onto the Gulf of Mexico. The lower reach includes two dams, seven inflows into the river, various wastewater treatment plants, and five main diversions (either stems or city extracts). A set of time-series flow hydrographs was obtained as an input to the model that was simulated to give an output of velocities, stage, and flow hydrographs in order to obtain the travel time. The numerical treatment of series of dams and spillway (that included uncontrolled overflow spillway, gate-controlled ogee spillway; and underflow gates) were created to give a realism to the modeling. Special attention was focused on the high spatial and temporal variation of flows in the river basin, a result of the large variation in inflows, and channel geometry due to dams and reservoirs along the river. For the reach there were slopes greater than about 0.0002, travel times could be predicted by computing the active flow area using the average Manning equation with n = 0.032 and assuming a constant inactive area for each reach. Predicted and measured spatial and seasonal variation of flow profiles along the river show good agreement. The simulated travel time of the modeled output was compared against gauge data for both water elevation and flow data provided by the IBWC from the dates of 2006-2010. Being that this is a Lagrangian model, this is computationally efficient model that has the potential to become a tool that could be used for long term simulation for water resource planning, management and operation decision making in a large and complex river basin system.**

Keywords: **Travel time, numerical model, stormwater drainage, water flow rate.**

1. BACKGROUND

The Lower Rio Grande Basin is comprised of the Falcon Dam, a dam on the Rio Grande between Starr County in the U.S. state of Texas and the city of Nueva Ciudad Guerrero in the Mexican state of Tamaulipas extends along the U.S./Mexico border, to the Gulf of Mexico. As the Rio Grande is the fifth largest river in North America, it forms an International Boundary between the United States and Mexico for a 1200 mile distance from its source in the Colorado. Its main source of water inflow are Río Conchos from Mexico, and the Pecos River from the United States. (USBR, 2013) The Amistad-Falcon Reservoir System contributes as a water resource for South Texas as assigned by the Texas Water Development Board. The Falcon Reservoir is about 275 miles upstream of the Gulf of Mexico, providing the source of water for the Lower Rio Grande River of a drainage area of approximately 13,000 square miles, near Rio Grande City, Texas. The size of the Lower Rio Grande Basin is 54,000 square miles, of which 8,100 square miles of the section, or sub-basins, do not contribute any water flow to the River basin. The Falcon International Reservoir has storage for over 8.0 million acre-feet of water that is controlled by both the United States and Mexico, where it has a designated purpose of 2.25 million acre-feet for flood control purposes while 6.05 million acre-feet are reserved for water supply. (Carter et.2015), Seelke, Shedd, 2015)

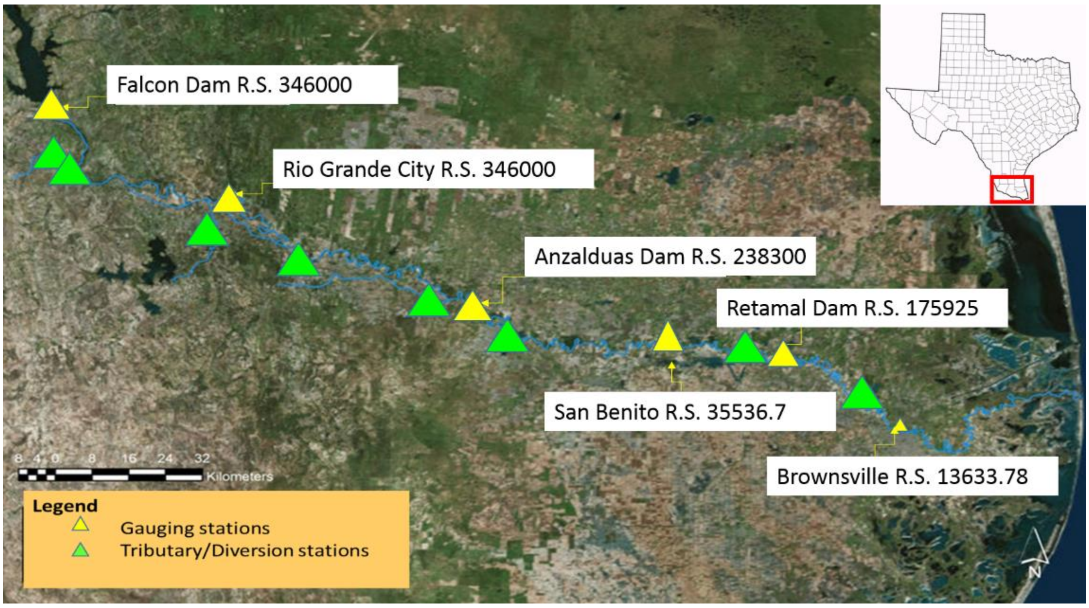

Fig 1: Lower Rio Grande watershed and tributaries

Due to the increment of human interference as a source of water and other developments in Colorado, New Mexico and West Texas, the Rio Grande runs dry South of El Paso during much of the year. Lower Rio Grande Basin's climate is semi-dry that brings a somewhat barren condition to support vegetation. The estimated amount of precipitation that does fall within that reaches the Rio Grande is about four percent, where any runoff provided only caused by a temporary excess watershed loss of infiltration and evaporation during heavy rainfall. (USBR, 2013) The contribution of heavy precipitation are generally caused by storms and tropical occurrences creates various storm hydrographs to work with, but the rainfall amounts are lowered the farther away the travel from the Gulf of Mexico.

To understand the complex interactions of these factors and make informed decisions regarding stream ecosystem management, natural resource managers can use computer simulations to water transportation. Regression models have the advantage of being computationally simple and applicable to locations where streamflow data is available. While regression modeling can be used effectively to predict water flows at discrete locations, they have problems when trying to project empirical relationships into the future or to locations where measurements were not actually made (Liu, L., & Xu, Z., 2015). This level of predictive capability calls for a deterministic model that represents the processes influencing water flow in a realistic manner. In order to understand the influences upon the water flow, a water-balance between the main stem of the river and tributaries and irrigation channels diversions must be applied. With the use of drainage canals that were obtained from a digital elevation model (DEM) was obtained from the United States Geological Survey for automated GIS (Geographic Information System) and the canals delineations were obtained. River sections were obtained and identified to obtain the cross-sectional structure that would be the basis for the geometric files.

Table 1: Breached banks cross-sections in LRG

Beginning RS of Breached Location	End RS of Breached Location	Country of Breach
Falcon Dam (RS 416280)	Rio Grande City (RS 346000)	U.S. & Mexico U.S. & Mexico
Los Ebanos (RS 299820.1)	Anzalduas Dam (RS 238300)	
Anzalduas Dam (RS 238300)	Retamal Dam (RS 175925)	U.S. & Mexico
Gateway Bridge (RS 45425.97)	Gateway Bridge (RS 45412.9)	U.S. & Mexico
Matamoros, MX (RS 35536.7)	Matamoros, MX (RS 35021.6)	Mexico
Brownsville (RS 13633.78)	End of Reach (RS 414.206)	U.S. & Mexico

Spatial variable data for the lower Rio Grande and its floodplain include a wide array of topographical, geomorphological, hydrographical data sets. The available data includes detailed digital terrain models (DEM), topographic mapping, field survey data such as river cross sections. These data bases have been incorporated into the 1-D data input files. HEC-RAS has the flood routing capability to account for spatial variation and as more detailed floodplain data sets become available, the model resolution and accuracy will improve.

Steady and unsteady state flow modeling can help establish a modeling stream system endpoints, i.e., upstream and downstream. Water surface profile computations begin upstream for subcritical flow or downstream for supercritical flow (Brunner, 2000). Soluble dye or chemical, tracers have been commonly used in stream tracer investigations (Hubbard et al. 1982; Jobson 1996, 1997). Since the mid-1960s, numerous dye tracer studies have been performed on streams in the United States for travel time studies. For example, in a study of travel-times, Jobson (1997) analyzed tracer data from nearly a thousand sub reaches of United States streams. However, the values used with these methodologies can be uncalibrated estimates for travel time studies (Hubbard et al. 1982).The IBWC-US Section occasionally measures current velocities in the Lower Rio Grande/Rio Bravo using Acoustic Doppler Current Profilers, but these measurements are scattered and infrequent can provide only snapshots of velocity at single points in the river at random flow conditions. Nevertheless this data could be used to perform some level of hydraulic calibration for travel time studies, but according to TCEQ (2015), reliable data of this type is either lacking within frequent years, or data has not been an official statement on their reliability.

The main objective of this study is to contribute to a better understanding on the hydrodynamics of the river that can be used as management strategy for future waste load management for the lower Rio Grande. This may fall into various strategies such as promoting elevation stability, alignment stability, maintaining channel capacity, and managing water supply to reaches based on the association between a hydraulic and hydrological model for a comprehensible model for flow to estimate travel time. To identify reaches with the highest potential to incorporate these strategies a one-dimensional (1-D) hydrological model. This study will focus on the 1D model in which the channel adjustments are made in the vertical with no change in width or channel alignment done by calibration for flow to estimate travel time of the Lower Rio Grande using one dimensional channel dynamic routing model.

2. PURPOSE AND SCOPE

The purpose of this journal is to determine travel times, streamflow velocities, and longitudinal dispersion rates for a reach of the Rio Grande River for various unsteady streamflow conditions. Predicting the effect of a mass transportation on the downstream water quality is a complex problem. The accurate modeling of travel time requires accurate modeling of transport that can be obtained by various methodologies such as: a Lagrangian model and Eulerian models, but the often suffer from numerical diffusion. Devkota and Imberger (2009) reviewed various Lagrangian and semi-Lagrangian models (Fischer, 1972, Manson et al., 2001). The drawbacks of these models were that these were not coupled with the Lagrangian flow and transport; instead they used an externally supplied reach average velocity field at fixed Eulerian grid points to drive the pollutants further and faster downstream of the river. While others have studied the possibility of using velocity-prediction equations where the main drive is using the mean river velocity as variable to calculate the travel time, but have been inaccurate at with velocities (Graf 1986; Jobson 1997). Predicting the effect of a pollutant spill may be dependent on the ability to predict the speed of movement downstream and the rate of longitudinal mixing. An optimal way to study the effect of mixing and distribution is to obtain the time of travel for a hydraulic output. However the results may vary with the use only limited to a flow condition that exists within the margin of an existent flow condition. This study plans to consider the use of extrapolating the travel time of water from the Rio Grande River from a high to low within flow bank.

The use of a one-dimensional Dynamic channel routing and the development of the numerical model would build fundamental insights about the effect of a hydraulic control on nutrient concentrations disperse, where the flow would be a feature to study as the flow of the water of a normal condition would not flush out the concentrations of nutrients to study the effect of water transportation. With the use of a one dimensional model there would be an inclusion of a channel morphology, where the cross-sectional river mean velocity would be the factor that controls the travel time. Unfortunately the estimation of a mean cross-section velocity within a long reach has been studied before as being a difficult feat, where the measurement of travel time through a river is usually limited to various techniques, instead of dye injection being the most prevalent (Kilpatrick and Wilson, 1989). The (USGS) has conducted various travel time studies where there has been various equation have been proposed for the prediction of velocities, but all of having a poor prediction accuracy (Jobson, 1996). At the existent river discharge, the travel time study tries to give an accurate measurement for the average reach water velocity, but because the velocity varies there has to be an extrapolation of velocity from flow to another. Within this study we will explore the possibility of applying the principals of geometric morphology and the uniform unsteady flow distribution to predict the water velocity of a varying discharges. This would include an analysis of the flow data to determine streamflow velocities and longitudinal dispersion rates, and comparisons of the data to estimates from the transportation study developed Texas Commission of Environmental Quality.

2.1 Modeling Plan:

In this study the development of a numerical model was used to build fundamental insights about the effect of a critical hydrologic control on nutrient concentrations, where the flow would be a feature to study as the flow of the water of a normal condition would not flush out the concentrations of nutrients to study the effect of water transportation. Using a one-dimensional river analysis program of Hydraulic Engineering Center River Analysis System (HEC-RAS), the use of one-dimensional advection-dispersion equation water model implementing the principle of mass conservation using a control volume approach. (HEC-RAS, 2010). The use for the hydrological approaches the use of the principle of conservation of mass as a change in volume of water over the channel reach with a linear approach to discharge with the storage-continuity equation. This would have the need to determine the hydrologic parameters of recorded data for the dates of simulation (January 2006 to December 2010) of both upstream and downstream sections of the river.

Observed inflow and outflow hydrographs can be used to compute channel storage by an inverse process of flood routing. When both inflow and outflow are known, the change in storage can be computed, and from that a storage vs. outflow function can be developed. In order to have a water balance volume and flow changes must be estimated accurately for the reach, that is why a calibration can be done to the hydraulic parameters, as flow velocities are generally higher at higher flows. This can be done the inclusion of tributary inflow and outflow must also be accounted for in this calculation. The inflow and outflow hydrographs can also be used to compute routing criteria through a process of iteration in which an initial set of routing criteria is assumed, the inflow hydrograph is routed, and the results are evaluated. The process is repeated if necessary until a suitable fit of the routed and observed hydrograph is obtained

2.2 River Geometry:

Cross-sectional information for the drainage canals were obtained from a digital elevation model (DEM) was obtained from the United States Geological Survey for automated GIS (Geographic Information System) and the canals delineations were obtained. River sections were obtained and identified to obtain the cross-sectional structure that would be the basis for the geometric files as previously seen in Table 1 for a total of 2042 cross-sections. Longitudinal segments with vertical layers in each segment obtained by the DEM data. Water surface elevations at each individual segment were to represent the overall distribution of water caused by the critical inflow of the upper-most section at a steady-state condition. During each individual normal profile computation, fundamental hydraulic properties of the flow, wetted area, average velocity, the Froude number for each cross-section are computed.

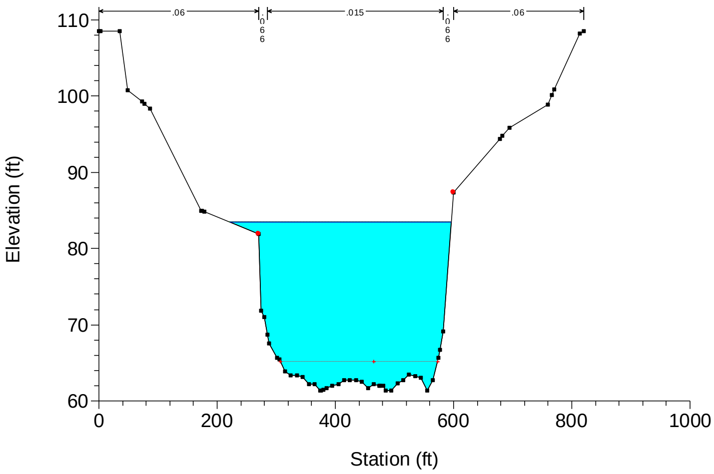

Fig 2: Lower Rio Grande Geo-referenced River Cross-section

2.3 Flow Data:

Through the information obtained by the International Boundary Water Commission (IBWC) the FLO-2D model provided a hydrograph of various high flow sates within the river that extends from Falcon Dam to Progresso. For an unsteady state modeling a flow that will be applied to 86 miles reach of the Lower Rio Grande. Various dates of high flow times were chosen to seek the effect of nutrient transportation within the river. The goal of the project is to design a seasonal high flow hydrograph that will inundate selected areas of the floodplain in this reach. One flow hydrograph will be simulated while three other can be based as a control for a comparative flow to the simulated flow: 6 month average hydrograph located at Falcon Dam, Anzalduas, and Progresso.

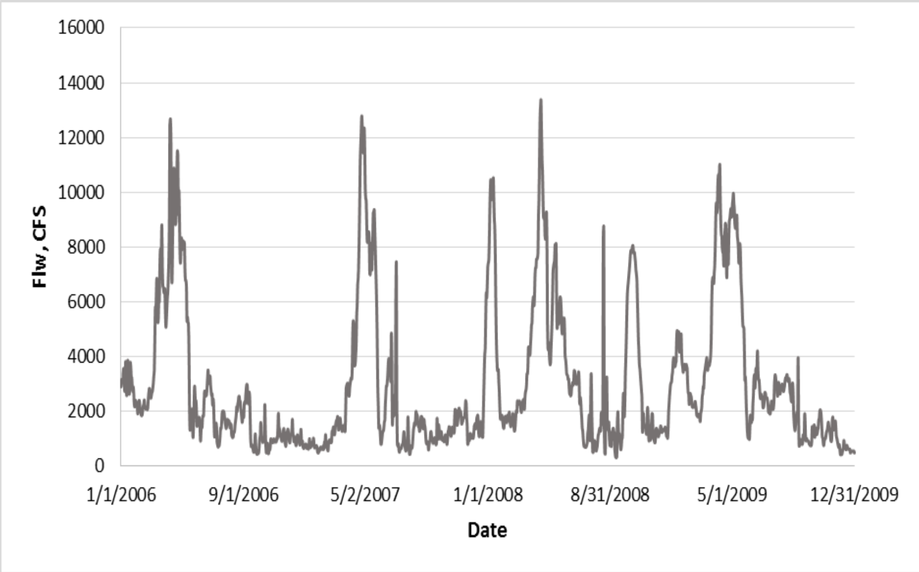

Fig 3: LRG River Flow Hydrograph

It is important part of this project to have an implementation to existing data bases for a base control. In several locations along this reach, the water surface elevation may be foot below the top-of-bank. This channel bed response is difficult to predict because it depends on sediment supply as well as flow hydraulics. Generally, the volume of the water stored in the river at a given time is relatively minor compared to the total volume in the flood hydrograph.

While the existing the model has relatively large grid elements, it is sufficiently detailed and accurate to conduct flood studies for a variety of projects such as levee design, river restoration, hydrograph routing, and flood inundation. The model will provide accurate estimates of in-channel discharge, area of inundation and water surface elevations. Estimated water losses include in-flow and diversion flow from the channel and floodplain. This journal discusses model development, new components, calibration and applications.

3. MODEL IMPLEMETATION

Dataset consisted of river morphology was provided information on the topography of the land surface and the gradient of the river, supplied information on the elevations, structures, and distances was acquired concurrent binational hydrography data set developed through a cooperative effort between the United States Geological Survey (USGS) and Mexico’s Instituto Nacional de Estadística y Geografía e Informática (INEGI). The locations include major confluences of tributaries and drains to the river, major diversion points such as diversion dams, irrigation pumps, wastewater outfalls, historical and synoptic water quality monitoring stations, and flow gages. Major lateral structures were models for three locations; Anzalduas Canal, U.S. Floodway, and Mexican Floodway, along the LRG. The starting point of the upstream river station (RS) is labeled as 416280 where the ending of the most downstream RS is labeled 414.27as seen in Fig. 1.

Channel flow is simulated one-dimensionally with the channel geometry represented by either by natural shaped, rectangular or trapezoidal cross sections. Secondary currents, elevation in bends and vertical velocity distribution are computed by the channel component. Local flow hydraulics such as hydraulic jumps and flow around bridge piers are also not simulated with the model. The model does distinguish between subcritical and supercritical flow because the momentum equation is used and it has no restrictions when computing the transition between the flow regimes. Channel overbank flow is computed when the channel capacity is exceeded. An interface routine calculates the channel to floodplain discharge exchange including return flow to the channel. Once the flow overtops the channel, it will disperse to other overland grid elements based on topography, roughness and obstructions.

The equation of motion is solved by computing the average flow velocity across a grid element boundary one direction at a time. Each velocity computation is essentially one-dimensional in nature and is solved independently of the other directions. The individual pressure, friction, convective and local acceleration components in the momentum equation are retained. More discussion of model solution and constitutive equations is presented in the journal. The goal is to correlate a calibrated hydrological the model to that of theory based, numerical calculation, and simplistic travel time calculation to predict the travel-time velocity.

The calculation of velocity used are used in sites adjacent to each other, so the travel time can be considered the time it takes for water to travel from each upstream sampling site to an adjacent downstream sampling site. The travel time, is calculated by taking the distance traveled and dividing it by the velocity of the river. The velocity of the river was not directly measured, but can be estimated from flow rate measurements using various equations based on control measured flows, and compared to model computations. The following relationship seems to be a generally accepted form for relating velocity to flow:

$$V = aQ^b \tag{1}$$

where is the velocity (m/s), are the coefficients, and Q is the flow (m3/s).

This relationship is used in the QUAL2E model (EPA, 1997) and has been adopted by several others (TCEQ, 2015). The watershed surrounding the river becomes one of determining parameters ("a" and "b") for the relationship. Several sources were examined that provide information from which parameters could be derived (W.E.Gates and Associates, 2007, EPA, 2007)

Equation 1 relates the velocity of the river to the flow rate using two coefficients, a and b, to account for the hydraulic characteristics of the river. The method for deriving velocity-related coefficients “a” and Exponents “b” was used by a combination of the available stream geometry

To derive these coefficients and exponents, we used a combination of available stream geometry flow, flow rating curve data, Manning’s equation. With the river characteristics for this stretch Texas Commission of Environmental Quality (TCEQ, 2015), the river coefficients for the stretch of the Rio Grande were found to be a=0.0758 and b=0.5.60. The general average river flow of the sample that was taken was 116 cubic feet per second (cfs), or 4.7 cubic meters per second (m3/s) (TCEQ, 2015). The velocity is estimated as the coefficient a multiplied by the flow raised to the power of the coefficient b.

3.1 Initial and boundary conditions:

The initial condition, for the boundary, one upstream and one downstream condition are required to solve the water equations for flow. The common upstream boundary conditions are either known depth or discharge both as a function of time:

$$y(x,0) = y^0(x) \text{ and } q(x,0)=Q^0 \tag{2}$$

For this particular simulation a stage-discharge relationship, or Q(h) as a single-valued function based on the relationship between the stage and discharge for the location of Brownsville station RS 414.206. The internal boundary condition used for the simulation were used only as either a flow through spillways, contributions through tributaries, or diversion of lateral structures as seen in Table 2**.**

Table 2: Diversion Station Numbers

Diversion Channel	R.S. Number
Ciudad Mier Exctract	392700
Camargo Extract	349000
Banker Weir	239870
Mexican Irrigation	238465
Mexican Floodway	176568

For a fully functional model to achieve a realistic unsteady simulation would need to specify the exchanges between basin storage areas and the river flow, where surface water interactions can be quantified using exchanging storage models that describe surface water flow with an advection-dispersion equation, and incorporate storage zones to simulate water stored in low–velocity zones such as pools and storages (Runkel et al., 1998). The one-dimensional transport with inflows and storage is a common method that simulates systems where continuous and exchange between the stream and a storage zone in subsurface flows. In order to achieve the exchange of flows, two variables could be implemented: the cross-sectional storage area, or the exchange rate between the stream channel and the storage section (Runkel et al., 1998).

In the model, the main channel and flood plains of a river use two physical laws: the principle of conservation of mass continuity, and the principle of conservation of momentum bases on the principles presented by James A. Ligget from the book "Unsteady flow in Open Channels" (Brunner, 2010a). The model uses a control volume within a distance "x" measured along the channel, at the midpoint of the control volume, the flow and total flow area are denoted as Q(x,t) and AT, respectively. The total flow are is the sum of active area A and an off- channel storage area S (Brunner, 2010b).

The control volume is divided into a number of sections that moves with the mean flow velocity. The model can simulate the flow and lateral interactions between the main channel and flood plains. Devkota and Imberger (2009b) and (Brunner, 2010b) have described the modeling framework and the governing equations for flow and transport in open channels. The conservation of volume (incorporating the dams, inflows, storage, diversion or withdrawals), the longitudinal momentum, and the new position of the control volume. The rate of inflow to control volume, the rate of out flow, and the rate of storage change is given as

$$\frac{\partial}{\partial t}(AL) = [Q_{in} - Q_{out} \pm Q_{structure}] \tag{3}$$

$$\rho \frac{\partial A}{\partial t}\Delta x = \rho[(Q - \frac{\partial Q}{\partial x}\frac{\Delta x}{2}) - (Q + \frac{\partial Q}{\partial x}\frac{\Delta x}{2}) \pm Q_l] \tag{4}$$

Where Q_l is the lateral flow entering or leaving the control volume and ρ is the fluid density. Symplifying and dividing the change in distance and fluid density yields the form of the continuity equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} \pm q_l \tag{5}$$

Where q_1 is the lateral outflow or inflow per unit length.

3.2 Flow boundary conditions:

By employing the numerical method, the time history of water depth and discharge can be computed for each point in the river. River stage observations collected during the years of 2006 to the 2010 flood were used to calibrate the 1-D model. Several simulations were carried out by using: the flow hydrograph observed at LRG upstream boundary condition of the Falcon Dam along with two other interflow locations, Anzalduas and Progresso, to the overall tendency of flows within the river as seen in Fig. 3. In order to apply the water balance the conditions for the final model did not include the internal flow condition, instead differences in gauge flows and recorded data for inflows/outflows were used to account for the water balance. At the downstream section where the information is known, such as a control section or a stage-hydrograph a stage-discharge relationship was created, or Q(h) as a single-valued function based on the relationship between the stage and discharge for the location of Brownsville station RS 414.206. The internal boundary condition used for the simulation were used only as either a flow through spillways, contributions through tributaries, or diversion of lateral structures as seen in Table 2**.**

Common one-dimensional open channel flow models behave poorly in terms of flow distribution across a section in a natural meandering channel with vegetated flood plains (Martín-Vide, et. 2008). Therefore, model calibration remains a critical step in numerical modeling (Vidal, et. 2005). The model presented in this study is developed in a manner to enable multiple calibrating options with the goal of determining the most appropriate approach. The considered approaches are calibration with an implementation of contributed flow provided by the surrounded basins, or diverted flows provided by either diversion lateral weirs, or extracted water used by cities that were recorded by the International Boundary Water Commission (IBWC). Although the simulations are conducted for the 5 years in the verification process, to retain clarity of all the considered methods, only segments of these results are presented. The criteria to evaluate the considered methods is the requirement of the calibration process, the physical justification of the considered approach as well as the results deviation from the measurements.

3.3 Inflows and Diversions:

Lateral inflows the Ciudad Mier Extract from Mexico (R. S. 397200), Arroyos La Minita and Los Negros (R. S. 384182.3 & 381590.1) and Diversions of Cuidades Mier and Miguel Aleman (R. S. Rio Alamo Diverted to Banker weir (R.S. 239870),and the Mexican Flood way (R.S. 179820.54) can be used as a calibration method.

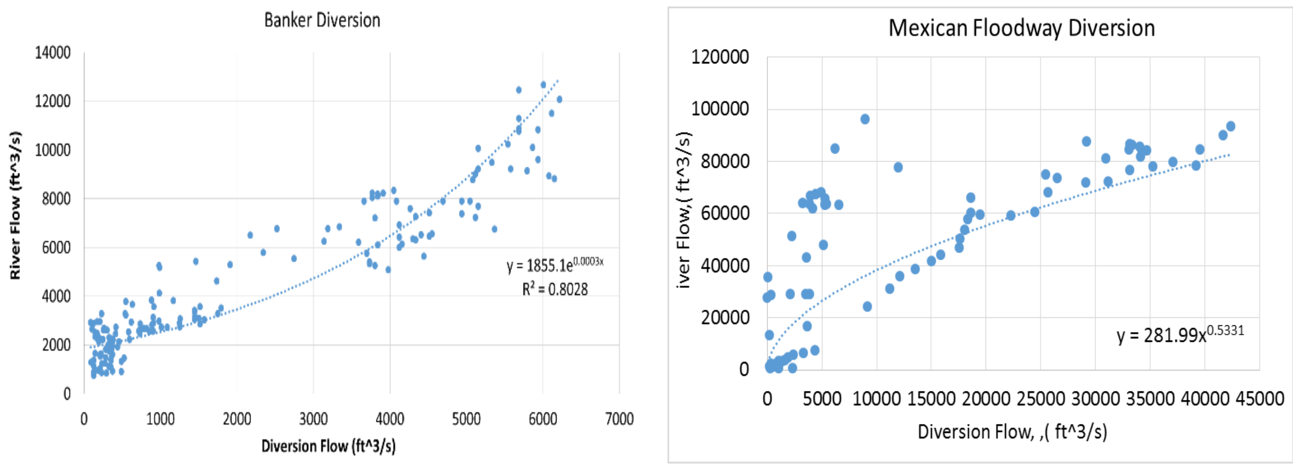

Fig 4: Relational Curves between river flow and diversions

The relation curves from which both diversion and inflows are derived were defined by the five year points for 2006-2010 years from flow measurements provided by the IBWC as seen in **Fig. 4**. The curves as defined by the points do not, however, cover the range required for all of the estimations of inflow. To extend the curves, at both the high and low ends, mean monthly inflow for the five years was computed for the relation of diversion relation to the river flow. The use of relational curves as a calibrating method was used as a methodology used in making estimates of inflow, the report presents considerable data on drainage basins and on streamflow patterns. (Bue, 1968) presented its relation curves as reference with only a limited number of values.

3.4 Steady non-uniform flow:

The model was validated for a 10-day steady state flow for approximately 257 mile reach from the Falcon Dam to the downstream section pouring into the Gulf of Mexico. The point and diffuse inflows to the river are presented in Table 2, showing the location of the inflows and the steady state flow rate. The upstream boundary condition, the flow from the Falcon Dam, was 6038.81 ft^3/s and a rating curve comparative of stage and flow at the downstream boundary. Comparison between the simulated and the measured steady state discharges are presented in Fig. 5. The results show great agreement between the field data and the model result, with the only difference being in the first few miles in Rio Grande City of the reach where the errors reached approximately 10%, which can be attributed to the diversion of water through the specified tributary of Table 3. The steady state simulations show that the system appears to be dominated by the diversions, as the flow decreases from 5155.941 ft.^3/s to approximately 1737.4 ft.^3/s, which is caused by the inclusion of the Anzalduas Dam and water diversion through irrigation canals located at the dam. The implication of this for modeling was that particular attention must be paid to the diversion boundary conditions.

Table 3: Contributions and Diversion of LRG Reach and proportional flow contribution

Tributary Contributions	**Length (m)**	**Proportion**	**Station**
Arroyo Morteros	7509.05	0.16	R.S. 408931
Arroyo La Minita	15698.01	0.34	R.S. 397764.8
Ramirez Creek	10507.49	0.23	R.S. 384182.3
Arroyo Los Negros	4615.54	0.1	R.S. 381590.1
Arroyo eEl Coronel	8354.00	0.18	R.S. 405122
Diversions	**Length (m)**	**Proportion**	**Station**
Ciudad Mier Extract	195463.59	0.36	392743.50
Ciudad Miguel Aleman	354739.7	0.64	349277.10

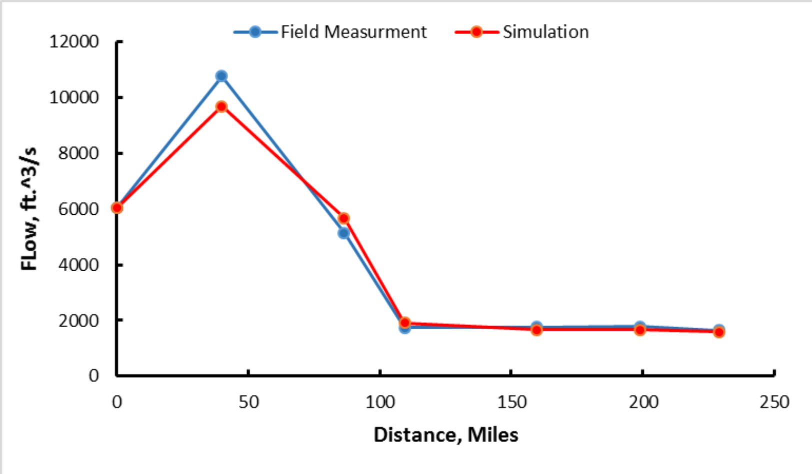

Fig 5: Comparison of field data and predicted simulation result for steady state flow.

4. RESULTS

4.1 Unsteady Flow:

First, the model was simulated for the Rio Grande River flow for the reach from the Falcon Dam to the downstream into the Gulf of Mexico for a 365-day simulation from January 01, 2007 to December 31, 2007 (Fig. 6) with flow recordings and tributary inflows provided by IBWC. The time step of the model was 6 minutes. The channel distances were varied between 100 ft and 3 miles.

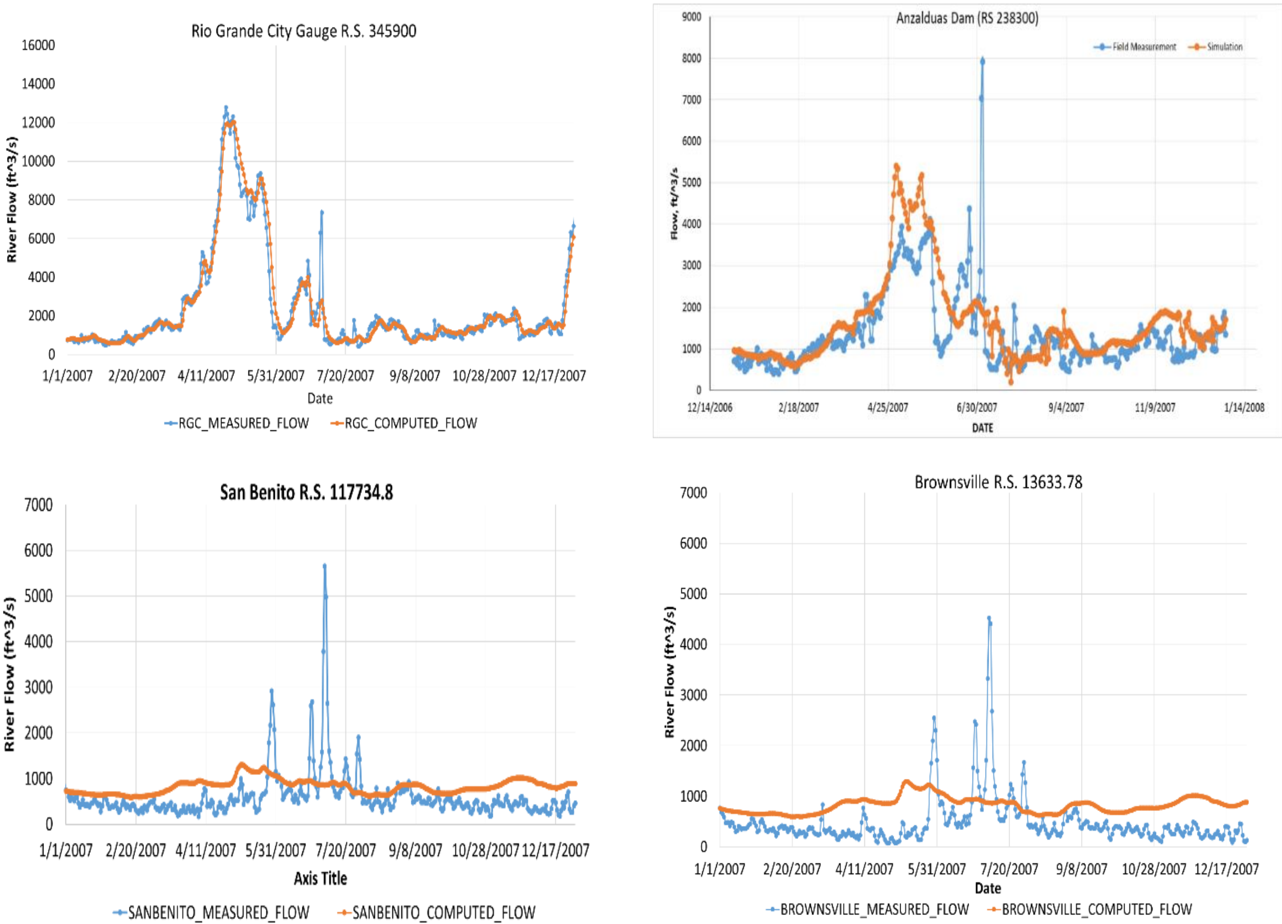

Fig 6: Comparison between measured flow with given lateral flows provided by IBWC and flow simulation of stations Rio Grande City (RS 346000), Anzalduas Dam (RS 238300), San Benito (RS 35536.7), and Brownsville (RS 13633.78)

As seen in Fig. 6, the initial flow simulation has a considerable amount of error, that is because the initial model only has the attributed upper and lower boundary conditions, but when we implement the attributed changes found from steady state, where the diversion of water changes the flow for the reach, there can be calibration model attempt, for the water contributions and water diversions for the reach. They were used for the creation of a rating curve for lateral structures, where if the river's water elevation hit the elevation of the lateral structure, that water would be diverted by a certain amount, or the alternative would be for river flow comparison to that of recorded diversion flows for a specific lateral structure.

4.2 Calibration of Model:

Calibration can be considered a continuous process where the input parameters that control modeled processes are adjusted during calibration to obtain better agreement between model output and actual observations. For this study, model iterations were made to improve predictions with, prior to calibration, initial boundary conditions were established for discharge, geometric and hydraulic parameters. The streamflow boundary conditions were established using the actual daily-value discharge hydrograph for the Rio Grande River below the Falcon Dam streamgage as the upstream boundary condition and a stage-discharge rating for daily-value streamflows from the Brownsville, Texas as the downstream boundary condition **(Fig. 1)**, with the addition of IBWC recorded lateral flows to 4 specific locations as seen in **Table 2.** As previously mentioned, instantaneous and daily-mean discharges files and stage-discharge-rating curves were retrieved or developed. Each file was tested in the model and the simulation result that yielded the best hydraulic performance (matching normal stage elevations) was selected. For the hydraulic boundary conditions, the initial Manning's n values were modified during calibration based on examination of cross section bed movement. Various periods from January 2006 to December 2009 had recorded two to three storm events each year and were used for model calibration and evaluation. The model was calibrated by manually adjusting the Manning's n values to minimize the differences between the predicted and observed as flow hydrographs at Rio Grande City (RS 346000), Anzalduas Dam (RS 238300), San Benito (RS 35536.7), and Brownsville (RS 13633.78). The initial Manning's n values were determined in reference to Chow (1959). The following sections shows the calibration parameters for the lateral flow (contribution and diversion) and calibration of Manning's n. The average Manning's n for all cross sections was 0.034, ranging from .03 (level beds) to .6 (very rough bedrock and boulders near channel banks and just downstream of each dam). In general, gate openings for Anzalduaz Dam and the Retamal Dam were acquired from real life measured hourly gate operations and averaged for every 6 hour changes. Internal boundaries for the dams were set using time-series of gate openings.

4.2.1 Inflows and Diversions:

As mentioned before the lateral inflows are the Ciudad Mier Extract from Mexico (R. S. 397200), Arroyos La Minita and Los Negros (R. S. 384182.3 & 381590.1) and Diversions of Cuidades Mier and Miguel Aleman (R. S. Rio Alamo Diverted to Banker weir (R.S. 239870),and the Mexican Flood way (R.S. 179820.54) were used as a calibration method.

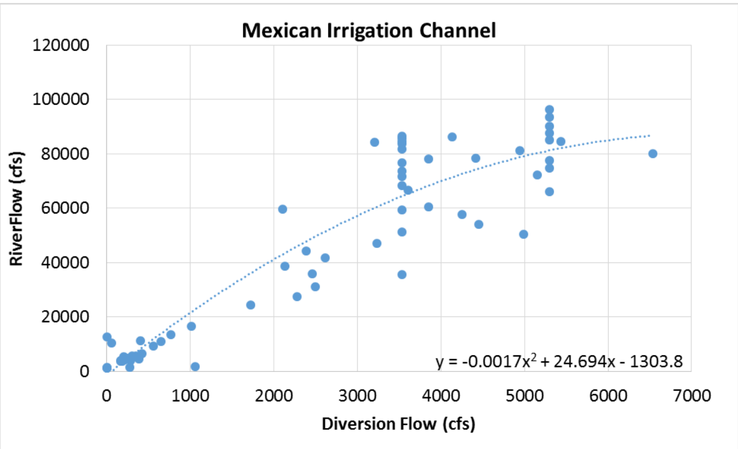

Fig 7: Relational Curve for the Mexican irrigation Diversion

For the purposes of automation, relational points between inflow/outflow within channel flows converted into equations by fitting a line through the table values in logarithmic space as seen for this study. Streamflow records for the 5-year period show that the patterns of streamflow around diversion areas can vary considerably from year to year. The use of a relational curve can be made as a calibration parameter where the estimates of inflow at inflow/diversion sections are estimates of total surface inflow to the river, which theoretically would equal the inflow to the river if adjustments were made for all diversions as on **Figures 4 and 7**. During very low months when evaporation and precipitation might be significant items in the water budget, adjustments can be estimated on basis water being pumped out by the cities, in fact records show (TCEQ, 2015) that water extracts from cities are more common during the dry seasons.

Groundwater inflow is largely an unknown quantity, as no comprehensive estimate of it has ever been made. Groundwater inflow consists of two main components: (1) direct seepage from water-table aquifers, and (2) upward leakage into the river from artesian aquifers lying beneath it. The U.S. Geological Survey has estimated the upward leakage to be within river waters between 50 to 250 cfs,(although they vary considerably from river to river) qualifying the estimate as possibly being in error by an order of magnitude but has made no estimate of the direct seepage (E. G. Otton, 1967). The attributed changes found from steady state, where the diversion of water changes the flow for the reach, was used for a pre-calibrated model attempt for the water contributions and water diversions for the reach, with flow recordings and tributary inflows provided by IBWC are. With further reviews for further model calibration, changes in diversion can be made to both allow further diversion of water during storm events and vice versa, obtain realistic results of minimal extraction during low flow events, as long as they agree with the relational curves.

4.2.2 Calibration of the model using Manning's Roughness Coefficient 'n'

Using the flows for years 2006 through 2009 there were attempts for calibration of the model through the Manning's roughness coefficient; by using the coefficient to establish a "standard 'n'" for a flow event at one section of length for the lower Rio Grande River from the Falcon dam to Brownsville (reach end). Due to the long length of the reach and having various changes in slope, dams, bridges, and various types of industrial/natural sections along the river that taking single n value for simulation of flow in the whole reach would not be best approach. There was a calibration of the Manning's roughness coefficient for a point using the storm data and then different values have been used to justify their use for a simulation of flow in the study reach. Various single values used in calibration by the recommended guidelines for river morphology for whole reach for floods of years 2006 through 2009 that are shown in Table 4. The table 5, also, shows the flow duration and data for various gauging stations for calibration.

Table 4: Manning's roughness coefficients for natural streams, Chow (1959)

ROUGHNESS COEFFICIENT 'n' FOR MANNING EQUATION				
			n Value	
	Natural Streams	Minimum	Design	Maximum
28	(a) Clean, straight bank, full stage, no rifts, or deep pools	0.025		0.033
29	(b) Same as (a) above but some weeds and stones	0.030		0.040
30	(c) Winding, some pools and shoals, clean	0.035		0.050
31	(d) Same as (c), lower stages, more ineffective slopes and sections	0.040		0.055
32	(e) Same as (c), some weeds and stones	0.033		0.045
33	(f) Same as (d), stony sections	0.045		0.060
34	(g) Sluggish river reaches, rather weedy or with very deep pools	0.050		0.080
35	(h) Very weedy reaches	0.075		0.150

Table 5: Flow duration, Manning's n and gauge station used for calibration.

Flow Year	Roughness coefficient Manning's 'n'	Storm events	Guage station used for calibration
2006-09	0.03, 0.032, 0.034, 0.036, and 0.04	15	RGC (RS 346000), ANZ DAM (RS 238300), San Benito (RS 35536.7), and Brownsville (RS 13633.78)

The model of the lower Rio Grande River has been used to simulate the stages for different single roughness coefficients for floods 2006 through 2009. Different values of Manning's n have been used, as shown in Table 4, in order to achieve a correlation between the measured flow and the simulated flow. The simulated stage hydrographs were compared with observed stage hydrograph at Rio Grande City (RS 346000), Anzalduas Dam (RS 238300), San Benito (RS 35536.7), and Brownsville (RS 13633.78) stations. Simulation periods used for floods of various storm events are also shown on the Table 5. Root mean squared error (RMSE) has been used for comparison of simulated stage with observed flow for various Manning's 'n' listed in Table 4. RMSE can be defined as:

$$RMSE = \sqrt{\frac{\sum_{i}^{n}(Q_O - Q_O)^2}{n}} \tag{6}$$

where Q_o = observed water flow in cubic ft. per second, Q_s = observed water flow in cubic ft per second and n = total no. of reference data points. Comparison of observed and simulated flow hydrographs using storm events found from 2006 through 2009 are shown on Table 6.

There were various attempts to create a simulation based on the varying 'n' numbers, from which individual Manning's n numbers were used for the entire reach, while those with the most prevalent RMSE were kept for their specific section were used. From Table 6, RMSE flow comparison of various roughness coefficients, we can observe that it's not necessarily the best option to place a single value of Manning's 'n' represents the whole reach that in order to create a better performing model. For example, n = .032 creates a greater performance at the Anzalduas Dam (RS 238300) while for a n = .030 gives better performance at San Benito (RS 35536.7). Similarly, Brownsville (R.S. 4963.44) of n= .04 creates a better performance than the n = .030 that gave Anzalduas Dam (RS 238300) a better performance. And so, with the river that has various topographical features along the reach, two dams that interfere progressive flow, and expansion as it pours into the gulf, in can be assessed that no single value of 'n' can be chosen for the entire reach, instead a combination of the various n's can be applied to the simulated reach in order to achieve a greater performance.

Table 6: RMSE flow comparison of different roughness coefficients at various gauging stations

Station no.	Change in time (Days)	n	∑R.M.S. Discharge
Rio Grande City (RS 346000)	1	0.03	414.52
		0.032	71.54
		0.034	106.68
		0.036	79.38
		0.04	109.75
Anzalduas Dam (RS 238300)	1	0.03	570.68
		0.032	71.20
		0.034	130.32
		0.036	365.80
		0.04	322.66
San Benito (RS 107700)	1	0.03	93.08
		0.032	134.23
		0.034	90.98
		0.036	202.97
		0.04	178.17
Brownsville (RS 49634.44)	1	0.03	104.40
		0.032	119.62
		0.034	81.92
		0.036	188.66
		0.04	68.72

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According to their respective performance outcome, the combination of roughness coefficients of n=.0032/0.036 were used up to and between stations of Rio Grande City (R.S. 346000) and Anzalduas Dam (RS 238300). Meanwhile the variations of n=0.03/0.034/0.036/0.04 were used for the stations below Anzalduas Dam (RS 238300) at San Benito and Brownsville. Their respective station performance compared to measurements of flow can be seen on Fig. 8.

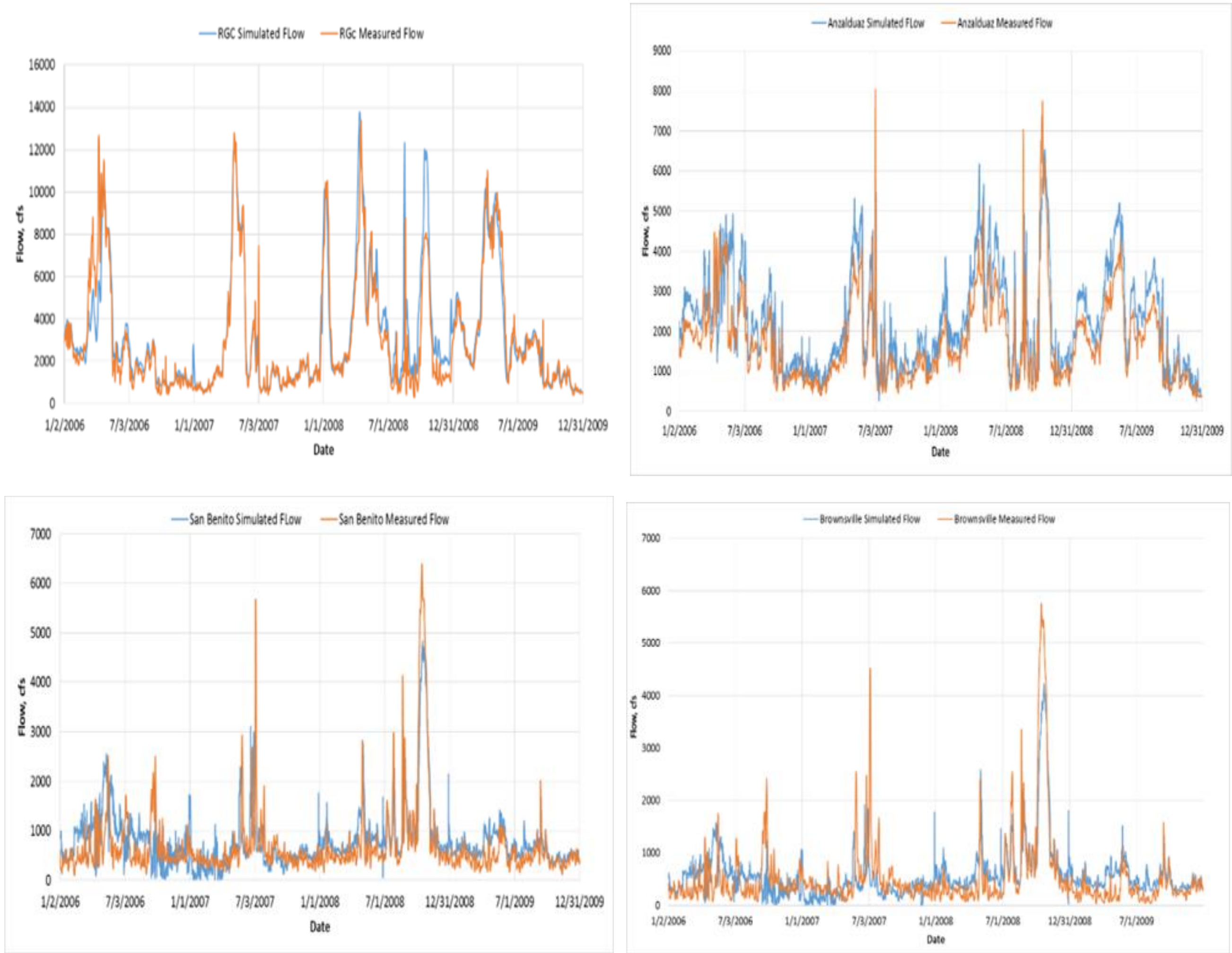


Fig 8: Comparison between measured flow and Roughness Coefficient calibrated flow simulation with tributary inflow and diversion accounted of stations Rio Grande City (RS 346000), Anzalduas Dam (RS 238300), San Benito (RS 35536.7), and Brownsville (RS 13633.78)

The calibrated model based model has been used to simulate the storm event in 2006. The comparison of observed and simulated flows at Rio Grande City (RS 346000), Anzalduas Dam (RS 238300), San Benito (RS 35536.7), and Brownsville (RS 13633.78). Also, the RMSE has been computed to compare the performance of model in flow to their respective measurements for all the gauging stations are shown in Table 6. The results seem to agree with the dispersion of various ‘n’ numbers throughout the reach, where in fact the only station that would need to vary by some standard is station 107700, where the RMSE was its highest at 93.08. This anomaly could be factored as an unpredictable factor such as the operation variability of Anzalduaz gate operation, where the simulation compared to the measured flows vary from time to time.

Having opted for a manning calibrations, the results can be compared to the observations. In figure 9 the observed versus peak flow was graphed from 14 independent storm events over the periods of 2006-2010 throughout the simulated period for the Anzalduaz station. The high peak discharges were reproduced reasonably well, but estimations were slightly worse for small floods where some points which were far from the 1:1 line. This can be attributed to the independent use of gate operation at the Anzalduaz Dam, where based on judgement, the gates might close to let water run off at lateral structures, while the model does account for this issue as a diversion rating curve, the operating gates remain open in the model based on historical data obtained from the dam operators.

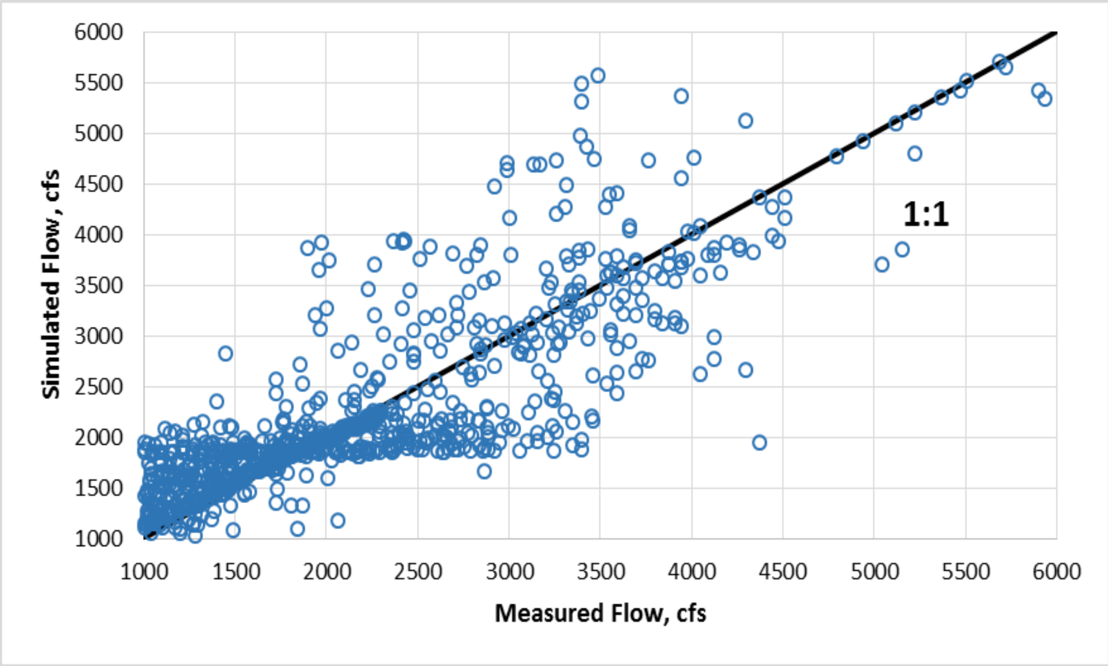
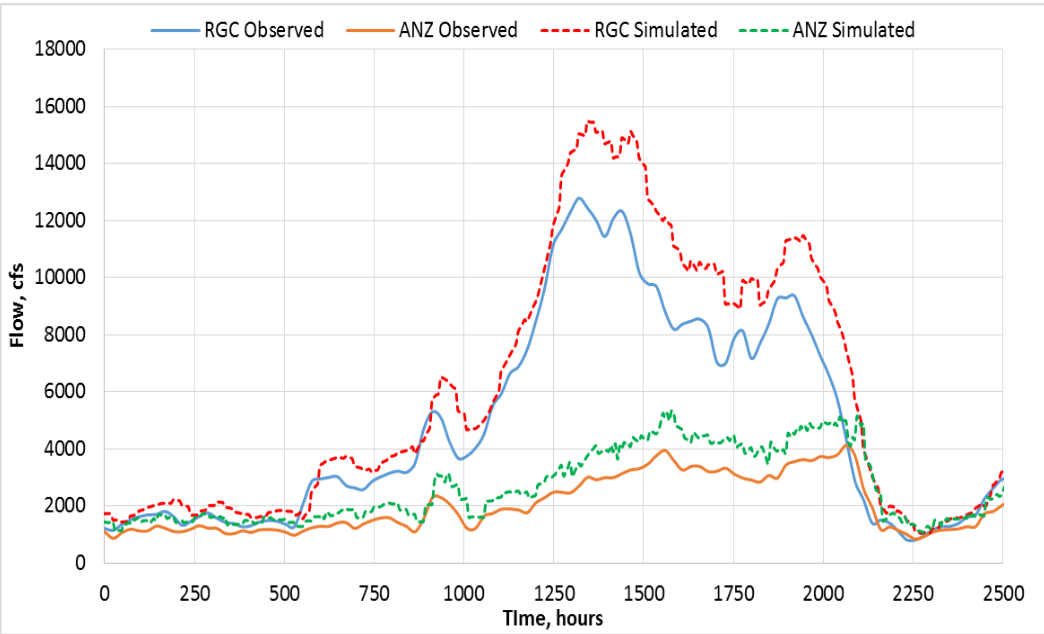


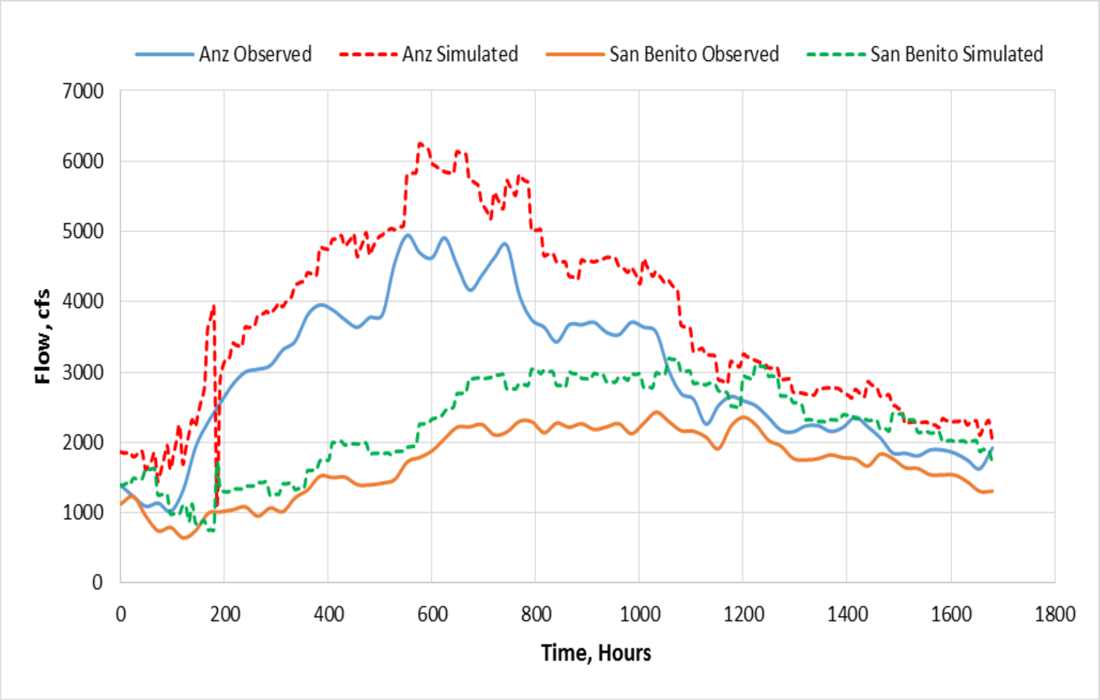
Fig 9: Comparison between measured flow and simulated calibrated flow for Anzalduas Dam (RS 238300).

5. TRAVEL TIME CALCULATION

After we obtained a combination of calibrated parameters n, the calibrated value of n is within the general range for natural river channels, but n also carries the uncertainty of R^2 of 0.81 as seen in Figure 9, therefore, the ranges of calibrated n factors carry an influence that represent a quality that mirrors the real roughness of the channel. The simulated hydrograph and observed hydrograph were used to find the travel times for the sub-watershed area, where the flow measurements between the two cross sections was derived using the difference of the gauging stations. This simulated hydrograph was similar in shape to the flood wave at the upstream cross section, but with a larger peak amplitude volume and with various points of varying differences of discharge values.

6. RESULTS





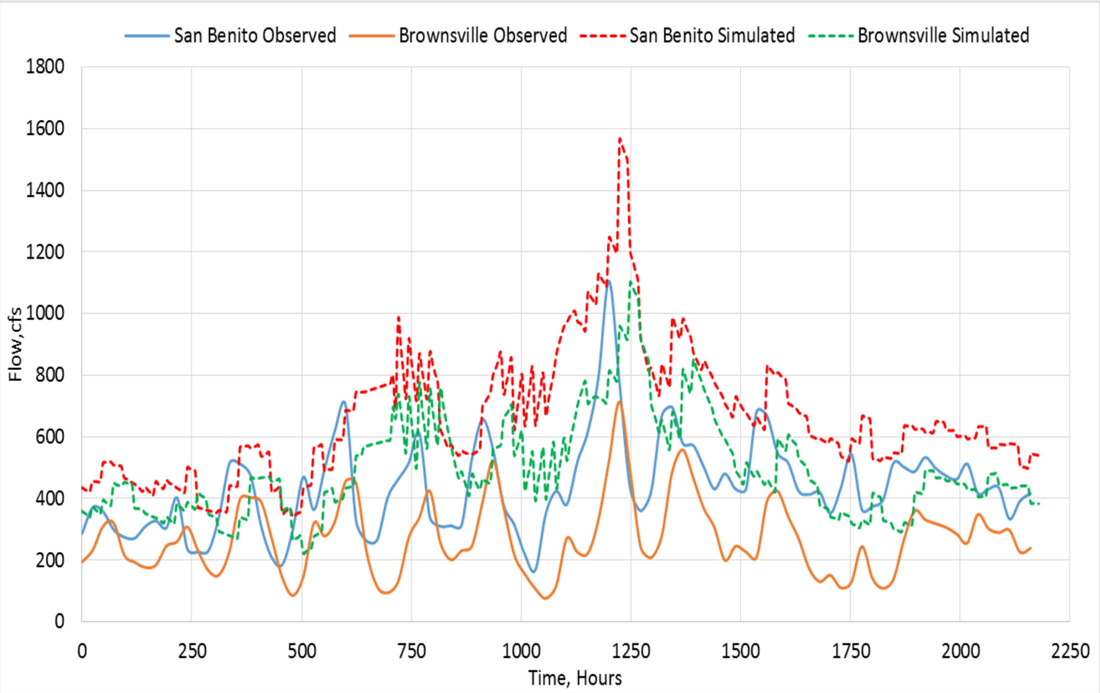


Fig 10: Observed and simulated flood waves. (a) Storm event that encompasses 75 percentile flow for the 4 year period between 2006-2009 of which the river bed roughness was calibrated; (b) Storm event that encompasses 50 percentile flow for the 4 year period between 2006-2009; (c) Storm event that encompasses 25 percentile flow for the 4 year period between 2006-2009

In these applications, the simulated hydrographs match our expectations, as illustrated by the Figure 10. The downstream flow peaks were dominated by the upstream flow peaks, as they strongly influenced by the outflow diversions generated by the irrigations canals and the lateral structures assigned as spillways between the two cross sections (Figs. 3b and 3c). Our simulation shows differences of the flows between the two cross sections, from Rio Grande City (RGC) to Anzalduaz (ANZ) stations, and Anzalduaz stations to San Benito station, and San Benito to Brownsville. With such a large time slot four years (2006-2010), we compared a total of 45 storm events that encompass different flow variations of the historical flow data, that is, obtain flows of 75%, 50%, 25% of the peak flow, in order to find the differences in travel time at variations of flow for the river model.

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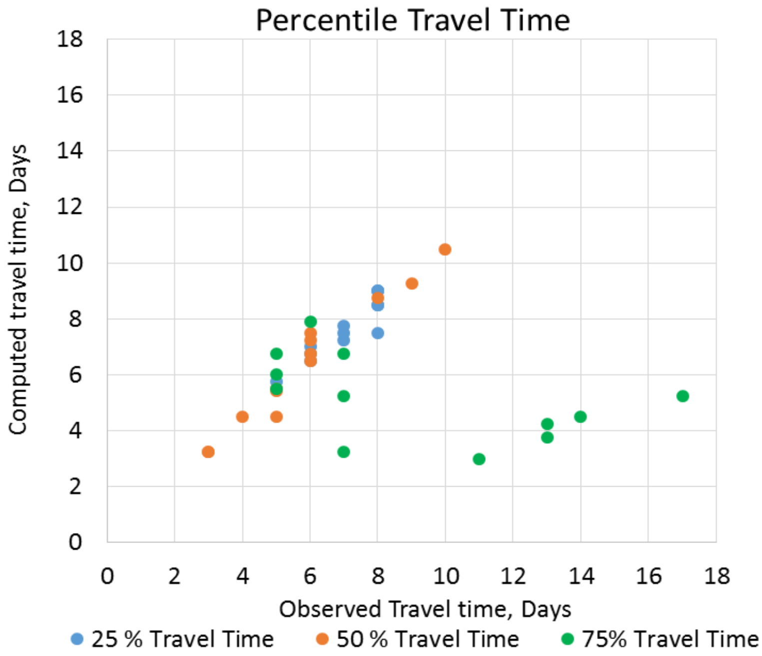

Fig 11: Travel time comparisons between gauge data provided by the IBWC and the simulated model

Our simulation shows that the travel time of the flood waves between various gauging stations for the entire reach varied throughout the change in flow where 25% constituted for an average of 800 to 1200 cfs.50% average of 4000 to 6000 cfs., and 75% average of 12,000 to 9,000 cfs. When taking the 25 % flow variation the travel times average about 7 to 8 days, this can be contributed, in the author's opinion, as an everyday flow where for the vast majority of the year the flow falls within this flow section. Next for the 50 % flow for a travel time calculation of an average of 5 to 6 days; for this spectrum of flow there is large margin of various travel times, this can be attributed to the variation of flow.

7. DISCUSSION

According to TCEQ (2015) the gate operations at the dams for Anzalduaz Dam and Retamal dam (RS 238300 and RS 175925 respectively) are manually operated and their operation is independently judged based on the flow of the river. What this means for the results, as seen in Fig 11, the travel times for this flow regime has a variation from 3 to 10 days due to the fact that when the river flow exceeds a certain flow (no recorded specification) that gates are closed in order to divert water into the lateral irrigation channels or the overflow dam. In order to mimic this diversion within the model, as explained in ***Inflows and Diversions*** section, a relational curve between reach historical data of river flow and diversion flow was established at the dams, where is a certain flow (ranging from 2,000 to 10,000 cfs.) would divert an average of half water flow out into the irrigations channels. Which brings us to the flow regime of 75% of peak water flow, where simulated travel time does not represent the real observational data. Where the simulated travel time averaged 6 to 8 days, the observed travel time varied from 7 to 14 days. This anomaly could be traced to the operations of the gates, where the model depends on diversion curves; and it's limited, due the fact that the real gate operations closed at higher flows in order to prevent flooding downstream. The model itself does not close the gates in order to prevent instability due to the limitations of the models use of equations for conservation of volume, the longitudinal momentum, and the new position of the control volume that depends on constant flow from the upstream sections.

According to McDonnell et. (2014) there has to be a use of routine flow velocity in runoff routing model development in order to improve the understanding of hydrological processes. There is variability on the use of water balance, river and watershed discharges, and flow velocities are not easy to measure or estimate. These flow discharges are dependent on temporary, spatial information for any given watershed, the channel condition, and weather conditions. With the conservation of volume and the use of the historical discharge data information and precipitation's influence on discharge can be used to obtain an idea on the river's travel time. This study approaches on methodology to utilize discharge data from tributaries, diversions, gauging stations in order to extract information about the flow waves in order to find travel times can and is feasible. This has the potential to be applied as a procedure to estimate flow velocity, water balance, and travel times.

8. CONCLUSION

This study explored the application of the travel time distribution formulation in the context of tributary and diversion outflows water interaction with the main water stem for the Rio Grande River. Through the medium of a distributed hydrologic modeling, we constructed the one dimensional, unsteady state hydrological water flow model of the shallow, unconfined aquifer at the Lower Rio Grande watershed, in South Texas by using HEC-RAS. The distribution was characterized with a mean travel time of 6 to 8 days. We were able to find that the time scales over which one can expect to observe the surface water response to distributed along the watershed, were influenced both by dams and diverting/combining flows that vary along the watershed to be of the order of a week.

The determination of the time travel is due to the impact of various control variables on this distribution. The first variable that has to be considered is the variation for cross-sectional analysis for the model; an analysis of the GIS geometry can and does influence the velocity distributions as where the necessity of accurate representation of both river path lines and location of diversions/tributary inflows influence the flow. Secondly, the necessity of attributing the inflows to the main river flow is a representation of the watershed's water catchment due to precipitation that flows into the river. With this in mind, the goal of attaining a water balance was achieved through the representation of storm event that provide water from the watersheds that provide flow waves along the river path-lines. The last control variable are the dams/lateral structures themselves; where their detainments and diversion of water attributed to the water balance. When one looks at a historical hydrograph of the river's flow, there are peak points of flow, where water is either detained. Where their representation in the model can achieve a similar result by recreating the gate operations that are present during a storm event that either detains or releases water through them that creates a similar hydrograph that can be compared to that of historical results.

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