DETERMINING THE EFFECTIVENESS OF CHLORINE-BASED BIOFILM CONTROL ON THE FRICTION FACTORS OF LARGE DIAMETER PIPELINES IN RAW WATER

by

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ABSTRACT

DETERMINING THE EFFECTIVENESS OF CHLORINE-BASED BIOFILM CONTROL ON THE FRICTION FACTORS OF LARGE DIAMETER PIPELINES IN RAW WATER

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Large diameter transmission mains pumping raw water experience capacity loss during the summer months due to the growth of biofilm. The loss of capacity impacts power costs at a time when power is obtained at a premium. Biofilm growth varies with the seasons and varies along the length of the pipeline. Biofilm growth and decay directly impacts the friction factors. Future design work should consider a seasonal friction factor rather than an aged friction factor over time.

The method of controlling biofilm is with the application of chloramines at the lake pump stations. This study compares the difference between little to no chloramines application to increased chloramine dosages as related to the friction factor. Proper chloramines application retards the growth of biofilm and is cost effective.

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CHAPTER 1

INTRODUCTION

The Tarrant Regional Water District (TRWD) is a regional raw water supplier which serves approximately 1.8 million people in the greater Fort Worth, Texas area. Eighty percent of the total Fort Worth water supply is supplied from East Texas. The East Texas system is supplied by two separate reservoirs - Cedar Creek and Richland Chambers Lakes - with single sourced pipelines coming from each. Both pipelines are pre-stressed concrete cylinder pipe (PCCP).

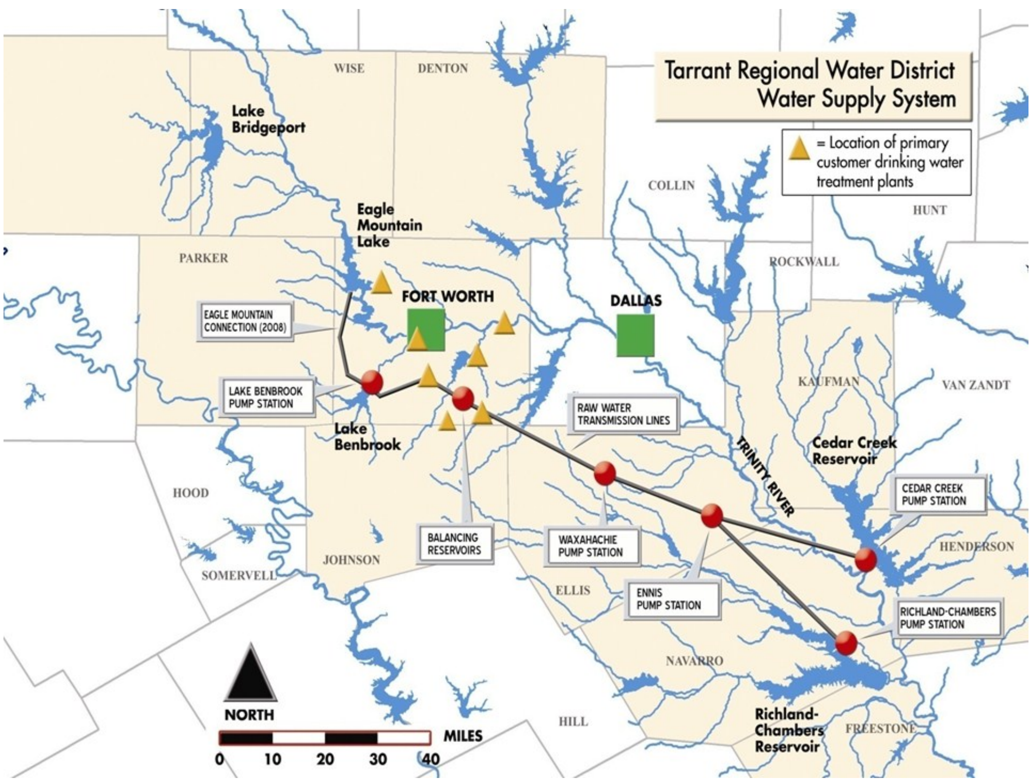


Figure 1.1: Tarrant Regional Water District Pipeline Schematic

The Cedar Creek pipeline was commissioned in 1972 and is approximately 40 years old. The total length of the Cedar Creek line is approximately 68 miles of 72-inch diameter pipe and 5.7 miles of 84-inch diameter pipe. Due to the corrosive nature of the Cedar Creek line, the internal concrete core has been damaged and is very rough. The Richland Chambers pipeline was commissioned in 1989 and is approximately 23 years old. The total length of the Richland Chambers line is approximately 72 miles of 90-inch diameter pipe and 5.9 miles of 108-inch diameter pipe.

There are two booster pump stations located between the top of the system high point, or Midlothian Hill, and the lake pump stations. The main booster pump station is located in Waxahachie, Texas. The high capacity booster pump station is located in Ennis, Texas. The scope of this research is limited to pumping from the lakes to Waxahachie. The pipeline system changes over to a gravity system once the pumped water reaches Midlothian Hill. The final gravity fed endpoint is the City of Fort Worth Rolling Hills Water Treatment Plant.

1.1 Statement of Problem

The District has observed a loss in the water conveyance pipeline capacity during the summer months of operation and considers the reduction in flow is due to bio-fouling of the pipeline. Bio-fouling directly impacts the friction factor seen in the pipeline. The method of controlling biofilm is to treat the raw water with chloramines in an effort to retard biofilm growth.

Forecasting when biofilm starts and stops growing prior to this study is quantified by seeing a direct loss or gain in the flow rate. Furthermore, no other protocols are in place to determine if the biofilm is successfully being controlled during chemical treatment.

The location of the biofilm growth in the pipeline is unknown and could have a direct impact to the chemical treatment dosage levels needed to successfully keep biofilm from impacting the water conveyance capacity.

1.2 Purpose of Study

This study attempts to identify the efficacy of treating the raw water with chloramines to retard growth of biofilm which realizes a smaller friction factor. In addition, the study will also determine if the cost of chemical treatment saves enough energy costs to warrant the use of chloramines.

The location of the biofilm will be determined to establish where chemical treatment impacts the friction factor. Is chemical treatment effective only near the pump station or is it successful all the way up the pipeline?

1.3 Literature Review

The original head loss equation can be traced back to the 1790's and is based on Antoine Chézy's work on flow in open channels. Gaspard Clair Francois Marie Riche de Prony, a student of Chézy, published in Chézy's findings in 1800. Prony (1800) also refined the Chézy findings into an empirical formula.

The Prony equation was the prevalently used head loss equation until 1845 when Jules Weisbach developed the head loss equation with the introduction of a friction factor, f; however, this equation did not further a refined method of determining f. Darcy's work (1857) never did classify a friction factor, but it is his concept that first relates pipe roughness to the diameter of the pipe.

$$H_l = f \frac{L}{D} \frac{\bar{v}^2}{2g} \qquad 1.1$$

where, *L* is the length, *D* is the nominal internal diameter of the pipe, $\bar{v}$ is the mean velocity (ft/s), g is the gravitational acceleration (ft/s^2). The friction factor can be determined using the Colebrook-White equation, or Equation 1.2.

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left(\frac{\epsilon}{3.7\,D} + \frac{2.51}{Re\sqrt{f}}\right) \qquad 1.2$$

The Brett (1980) study documented the effects of the loss of capacity due to biofilm build up. Brett also found that the best method of restoring capacity is to physically desiccate and clean the pipe. The author noted the seasonal variability of pipe capacity due to biofilm growth but did not elaborate upon how much the seasonal effects change. The largest increase in friction over the course of 11 years of data gathering was reported to increase by 40 percent.

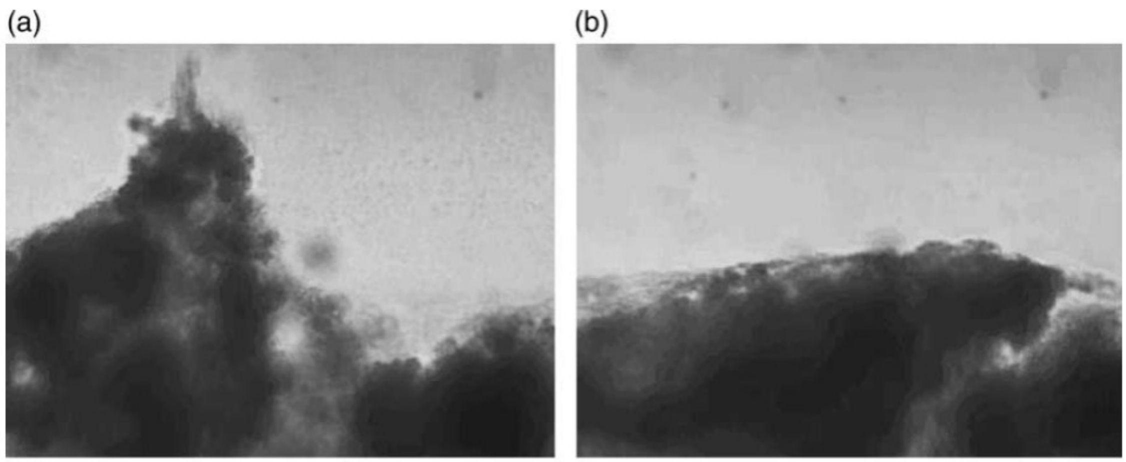
Characklis, (1979) found that biofilm thickness does not increase friction until the thickness exceeds the theoretical viscous sublayer thickness. Picologlou's (1980) study found that friction resistance increases as biofilm thickness increases. Biofilm formation was compared between a roughened pipe and a smooth pipe under the same fluid shear stress conditions. The initial friction factor in the roughened pipe is higher in the roughened pipe; however, the friction factor stays fairly the same until the biofilm thickness exceeds the ridged rough surface of the roughened pipe.

Lewandowski (1995) did a study and found that biofilm affects pressure drop when a critical velocity has been reached. Their laboratory testing also found that biofilm thickness will reach a pseudo steady state at the same time the pressure drop reaches a pseudo steady state status. The velocity directly influences the length of streamers allowed to form. High flows yielded longer streamers. The oscillation of the streamers removes kinetic energy from the bulk liquid. Higher flows induce greater oscillation of the streamers while low flows do not. Lewandowski also found that at low flows, the biofilm formation does not increase friction. The biofilm may impact the roughness of the pipe by filling in-between the peaks of the ridged rough surface, thereby smoothing the roughness coefficient.

Velocity was also found in Stoodley's (1999) study to influence biofilm formation. When the flow regime is laminar, the biofilm thickness and structure were different than biofilm grown under turbulent flow conditions. Turbulence changes the structure of how the biofilm is formed – it becomes filamentous and increases frictional resistance.

The Barton (2008) study looked at the same conduits as Brett studied. Barton documented the friction factor due to biofilm prior to physically cleaning the conduits and found that bio-fouled conduits do not follow the Colebrook-White model in the smooth-rough transition regime. When the conduits were cleaned, the friction factors do follow Colebrook-White. Due to the nature of the conduits being gravity-fed lines, they did not have the opportunity to look at the friction factor at different Reynolds numbers. Barton's study also measured the thickness of the biofilm prior to cleaning and found the equivalent sand grain roughness was larger than what was experimentally determined prior to cleaning the conduit. This difference could be attributed to possible viscoelastic behavior of biofilm. Viscoelastic behavior can be due to biofilm deforming and reducing in physical height due to increased wall shear velocities.

When there is no velocity, the biofilm thickness is larger than when there is velocity (Vo, 2010). Biofilm deforms due to moving fluid. The deformation keeps any anti-microbial application to only be in contact with the top of the biofilm and does not allow penetration of an anti-bacterial towards the roots of the biofilm. The study also looked at biofilm thickness recovery due to velocity. He studied the reaction of biofilm with respect to the cycle of compression. The thickness tended to recover only 80% of the original thickness. Vo also stated the biofilm has many different mechanical properties, such as bacterial strain and biofilm age that could impact the CFD model.



(Courtesy Jeff Heys, Montana State University-Bozeman)

Figure 1.2 Deformation of Biofilm (a) without velocity, (b) with velocity

Kruzic (2012) determined that once biofilm has been established on the wall of a pipe, it was very difficult to remove the biofilm. Visual inspection found the surface of the biofilm looked as though it was killed, but the base of the biofilm still remained firmly attached to the pipe wall.

CHAPTER 2

RESEARCH APPROACH

The limits of the study begins at the lake pump stations continuing up to the next functioning booster pump station located in either Ennis or Waxahachie Texas.

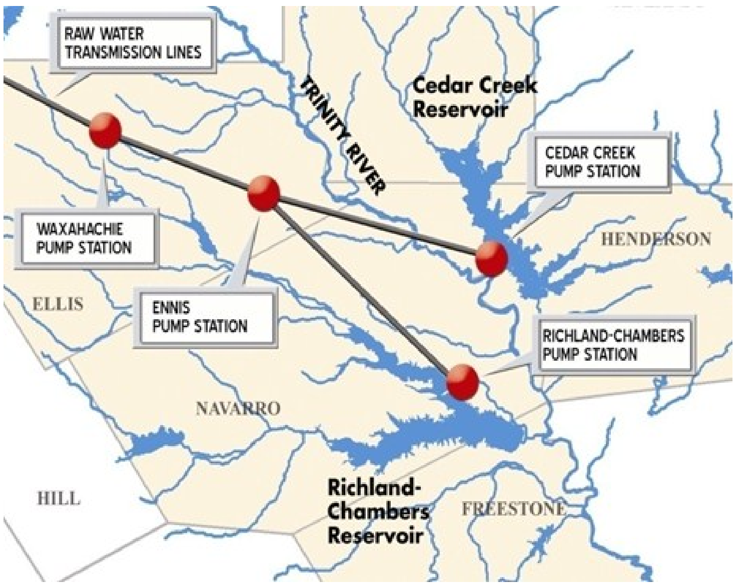


Figure 2.1: Limits of Study - From the Lake Pump Stations to the Waxahachie Pump Station

The efficacy of chemical dosing to retard biofilm growth will be accomplished by measuring the loss or gain of pipeline flow rates. The differential in flow rates between years with little to no chlorine dosage versus years with adequate dosage will help to determine if there are any energy savings in chemically treating the pipeline. The hypothesis being tested is to establish that the cost for chemical dosing to retard biofilm growth is cost effective when compared to energy costs. Two different studies are performed to determine if treating the raw water with chloramines directly impacts the pipeline flow capacity.

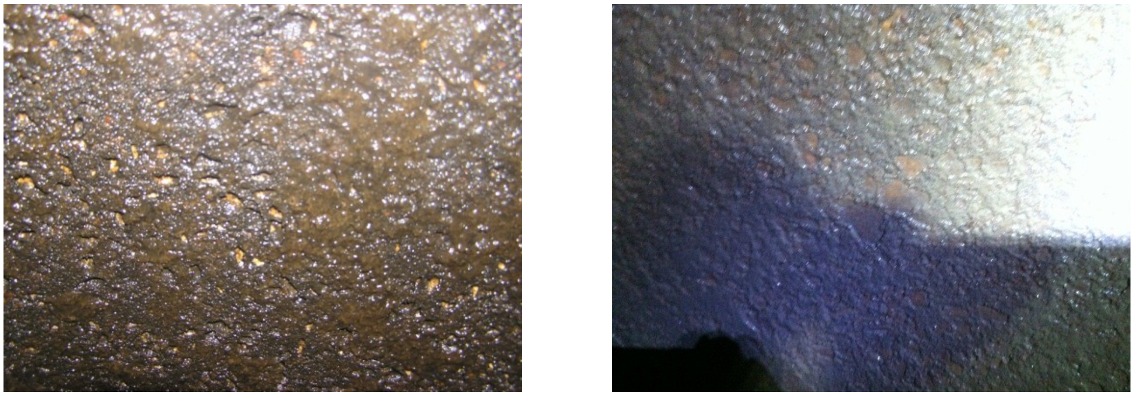
The first study, or desktop study, is based on a daily time step using historical Supervisory Control and Data Acquisition (SCADA) system data for 2010 through 2012. The chemical treatment during the 2010 and a majority of the 2011 desktop study period is considered non-effective as the system was operating with little to no chemicals. This study will serve as a baseline of how biofouling impacts the pipeline flow rates and friction factor. The 2012 data will be evaluated against the 2010/2011 data to document any differences due to adequate chemical feed.

The second study, or field study, goes into more depth and looks at how the friction factor changes along the length of the pipeline. In addition, field data collected during the study period also looks at the effectiveness of chemical treatment on pipeline flow rates.

Finally, an economic evaluation study will compare the energy costs from the 2010/2011 data, which documents low to no chemical treatment, with the energy costs of the 2012 season, which accomplished higher chlorine residual 2012 rates, to determine if chemical treatment is cost effective.

2.1 Estimation of Pipeline Capacity

A loss in the pipeline pumped flow rate is observed on a yearly basis. Based on field observations of the interior of the pipe, it is assumed the loss of capacity is due to the cyclical loss and regrowth of biofilm. Figure 2.2 are photographs of the wintertime biofilm found in both the Cedar Creek and Richland Chamber's pipelines.



(a) Cedar Creek (b) Richland Chambers

Figure 2.2: Observed Wintertime Biofilm on (a) Cedar Creek and (b) Richland Chambers Pipe Interiors.

Note that at the time of the photographs (Winter 2010), the Richland Chambers pipeline had not been dosed with chemicals for over two years. Both pipelines have a lake pump station and two booster pump stations. Up to five pumps at each station can be used to deliver water. There are two different modes of operation: low capacity and high capacity. Low capacity occurs when the lake pump stations pump water to the Waxahachie booster pump station, which by-passes the Ennis Booster Station. The pipe class pressures are exceeded when pumping to Waxahachie at flows greater than 68 million gallons per day (MGD) at Cedar Creek and 144 MGD at Richland Chambers, the Ennis booster stations must be used. There are six pump configurations based on 5 pumps and the low capacity/high capacity features of the system. The naming convention is to put the number of pumps operating in front of the letter designating either L for low capacity or H for high capacity. Each pump configuration is matched at each station. For example, when using the 3L pump configuration, 3 pumps at the lake station are on and 3 pumps at the Waxahachie station are on.

Table 2.1: Low Capacity/High Capacity Pump Station Configuration

Pump Configuration	***Lake Pump Station***	***Ennis Booster Pump Station***	***Waxahachie Booster Pump Station***
Low Capacity (1L, 2L, 3L)	X		X
High Capacity (3H, 4H, 5H)	X	X	X

Cedar Creek Lake Pump Station (CC1) consists of six pumps where five pumps maximum can be run at one time due to the pressure limitations of the upstream pipeline. There are two stations CC1 can pump to: CC2, located at Ennis, and CC3, located in Waxahachie. The distance from CC1 to CC2 is 25 miles. The distance from CC2 to CC3 is 17.7 miles. The low capacity pumping configuration is when CC1 bypasses CC2 and pumps directly to CC3 in Waxahachie. High capacity is when all three pump stations are in operation. All pumps on the Cedar Creek pipeline are constant speeds.

Richland Creek Lake Pump Station (RC1) consists of six pumps. Five pumps maximum can be run at one time due to the pressure limitations of the upstream pipeline. There are two stations RC1 can pump to: RC2, located at Ennis, and RC3, located in Waxahachie. RC2 and RC3 both have 5 pumps each. The pumps at RC1 and RC2 are constant speeds pumps. RC3H has 2 pumps with Variable Frequency Drives (VFD) and the other three are constant speed. The distance from RC1 to RC2 is approximately 29.6 miles. The distance from RC2 to RC3 is 17.5 miles. The low capacity pumping configuration is when RC1 bypasses RC2 and pumps directly to RC3 in Waxahachie. High capacity is when all three pump stations are in operation.

To determine if chloramine dosage impacts the flow rate, comparison of the pumped flow rate from year to year will be best done when compared by pump configuration. Ideally, the 2012 flow rates should be less than the 2010 and 2011 rates if chloramines can control biofilm. Another comparison is to look at dosage rates and chlorine residuals with regard to pumped flow rates. Laboratory testing has shown that a minimum chlorine residual of 0.5 mg/L throughout the line can control biofilm growth (Kruzic, 2011). During the field study period, the goal is to target a minimum of a 0.5 mg/L residual as far as Waxahachie.

2.2 Estimation of Friction Factor

2.2.1 Calculation of Head Loss

The energy equation is used to determine the friction factor by calculating the head loss.

$$\frac{P_1}{\gamma} + \frac{\bar{v}_1^2}{2g} + z_1 = \frac{P_2}{\gamma} + \frac{\bar{v}_2^2}{2g} + z_2 + H_L \qquad 2.1$$

where P is pressure (psi), γ is the specific weight of water (lbs/ft^3), z is the height to a horizontal datum (ft), and H_L is the total head loss (ft).

Comparing the percentage of lost capacity to years where chemical treatment was more effective will help determine if chloramines treatment was effective

2.2.1.1 Estimation of Head Loss Based on Historical Data

TRWD's SCADA system stores historic operation data in a database called iHistorian. In order to establish a baseline of evidence of biofouling, a desktop study was performed using the historic data to calculate friction factors and hydraulic roughness from 2010 through summer 2012 from the lake pump stations to the next functioning booster pump station located in either Ennis or Waxahachie Texas. The results from 2010 and 2011 will be compared to the results of the 2012 data.

The total head loss was evaluated from the lake pump station to the next functioning booster pump station for the desktop study and is represented in Figure 2.3

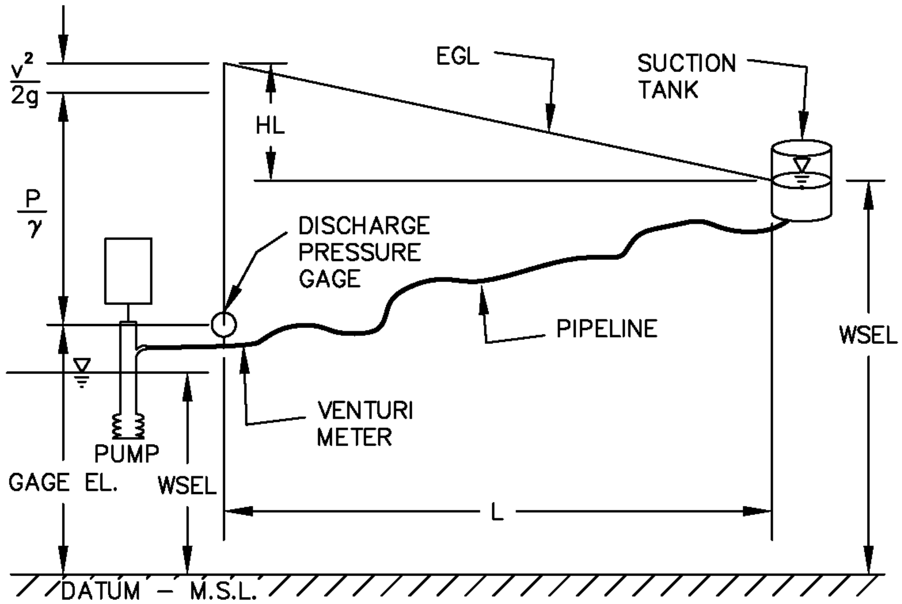


Figure 2.3: Lake Pump Station to Suction Tank Hydraulic Schematic

The head loss is calculated using a transformed Equation 2.1.

$$H_L = \frac{P_{\text{Discharge Pressure Gauge}}}{\gamma} + \frac{\overline{V}^2}{2g} + Z_{\text{Gauge Elevation}} - Z_{\text{WSEL}} \qquad 2.2$$

Pressure (P) is obtained from the existing pressure transducers installed on the discharge side of the pumps. The specific weight of water (γ) is adjusted based on temperature of the water. Temperature is measured at the pump intake structure at the elevation of the pump bells to approximate the temperature of the water entering the pipeline. Mean velocity ($\overline{V}$) is found by using the flow rate obtained from the previously mentioned venturi meters and dividing by the area of the pipe based on the nominal internal diameter. The datum established is based on the North American Vertical Datum of 1988 (NAVD 88). All of the gauge elevations were surveyed and the detail information is located in the Appendix under Field Logs. The only elevations which vary are the water levels located in the upstream suction tanks and are obtained from SCADA's iHistorian database.

2.2.1.2 Estimation of Head Loss Based on Field Data

Twenty-two (22) Omega Engineering OM-CR-PRTEMP-1-300G pressure transducers were installed along both the Richland-Chambers and Cedar Creek pipelines.

Figure 2.4: Transducer Connected to Pipeline – Cedar Creek Sta. 3873+60

The locations for each pressure transducer, shown in Figure 2.5, are selected based on somewhat even spacing along the pipeline and ease of access to the collection site. Table 2.2 lists the discrete sections based on the station and location of each pressure transducer.

Figure 2.5: Pressure Transducer Locations

Table 2.2: Field Study Pressure Transducer Stations and Location Names

Seg.	Cedar Creek			Richland Chambers		
#	Stations	Length (ft)	Locations	Stations	Length (ft)	Locations
1	3895+11 to 3857+00	3,811.15	CC1 Pump Station to SH 274	4123+95 to 3956+72	16,723.29	RC1 Pump Station to FM 2859
2	3857+00 to 3512+05	34,632.16	SH 274 to Gravel Pit #2	3956+72 to 3715+90	24,314.81	FM 2859 to SH 31
3	3512+05 to 3320+00	19,235.93	Gravel Pit #2 to Ready Mix	3715+90 to 3554+40	16,153.65	SH 31 to FM 2100
4	3320+00 to 3182+00	13,786.49	Ready Mix to CR 3250	3554+40 to 3355+13	20,950.15	FM 2100 to FM 1129
5	3182+00 to 3080+00	10,189.16	CR 3250 to Rosewood	3355+13 to 3125+25	22,987.93	FM 1129 to CR 1030
6	3080+00 to 2856+70	22,320.96	Rosewood to Walker Creek	3125+25 to 2901+25	22,399.78	CR 1030 to Barker Road
7	2856+70 to 2600+75	25,614.03	Walker Creek to FM 1181	2901+25 to 2675+83	22,541.70	Barker Road to SH 85
8	2600+75 to 2550+89	4,985.88	FM 1181 to CC2 (Ennis)	2675+83 to 2555+62	10,998.05	SH 85 to RC2 (Ennis)
9	2550+89 to 2390+95	16,472.22	CC2 (Ennis) to Crisp Pike Rd	2555+62 to 2389+30	16,636.32	RC2 (Ennis) to Crisp Pike Rd
10	2390+95 to 2209+13	18,174.48	Crisp Pike Rd to Garrett	2389+30 to 2209+13	18,003.40	Crisp Pike Rd to Garrett
11	2209+13 to 2076+82	13,233.69	Garrett to Ebenezer Rd	2209+13 to 2076+81	13,232.71	Garrett to Ebenezer Rd
12	2076+82 to 1874+16	20,263.37	Ebenezer Rd to Graves	2076+81 to 1874+14	20,264.58	Ebenezer Rd to Graves
13	1874+16 to 1638+24	23,790.46	Graves to CC3 (Waxahachie)	1874+14 to 1636+59	23,755.58	Graves to RC3 (Waxahachie)

Head loss for the field study is evaluated from pressure transducer gauge to the next pressure transducer gauge. Figure 2.6 best represents the parameters used in the field study.

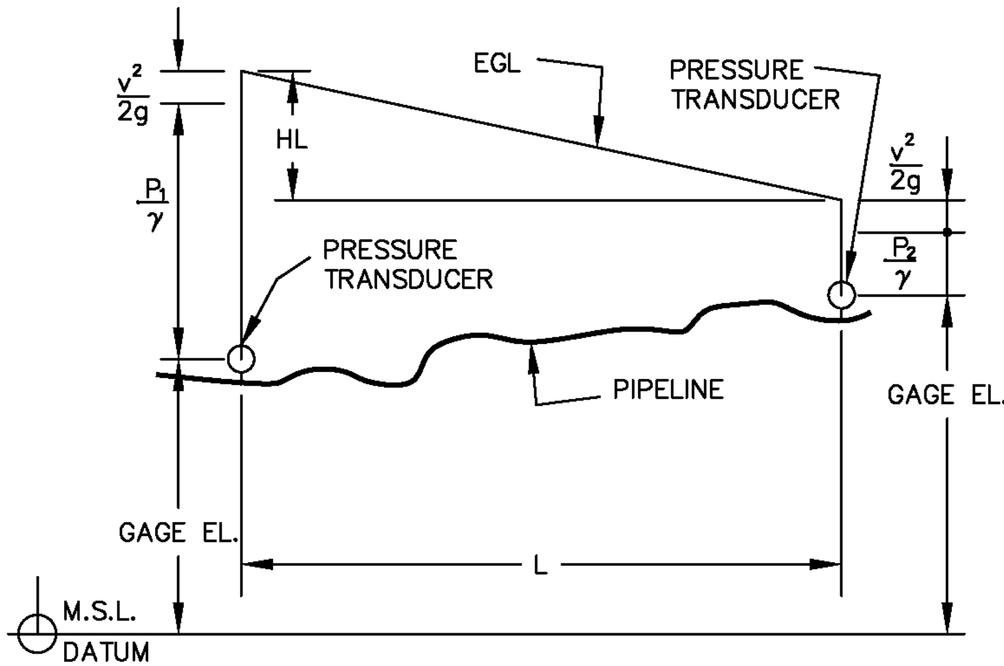


Figure 2.6: Pressure Transducer to Pressure Transducer Hydraulic Schematic

Transforming Equation 2.1 to calculate head loss, the head loss between pressure transducers will help to determine if biofilm growth increases or decreases in specific discrete reaches of the pipeline. When head loss is high, it is anticipated that biofilm is thicker than when the head loss is smaller when pumping at the same pump configuration.

$$H_L = \frac{P_{\text{D.S.Pressure Gauge}}}{\gamma} + Z_{\text{D.S.Gauge Elevation}} - \frac{P_{\text{U.S.Pressure Gauge}}}{\gamma} - Z_{\text{U.S.Gauge Elevation}} \qquad 2.3$$

The pressure (psi) is obtained from the Omega pressure transducers. Each gauge elevation was field surveyed. D.S. means downstream. U.S. means upstream.

2.2.2 *Calculation of Friction Factor*

From the head loss equations previously mentioned, the friction factor can be calculated from Equation 2.4.

$$f = \mathrm{H_L} * \frac{\mathrm{D}}{\mathrm{L}} * \frac{2\mathrm{g}}{\overline{\mathrm{V}}^2} \qquad 2.4$$

where f is the unitless friction factor, $\mathrm{H_L}$ is the head loss calculated from Section 2.2 in feet, D is the nominal internal diameter of the pipe in feet, L is the distance between the gauges/suction tank in feet, and the mean velocity is calculated from lake stations flow rate (ft/s). Note that minor losses of all valves and bends are so small for large diameter pipe that they are neglected because line friction is governs hydraulically.

Finally, Equation 2.5 is the calculated hydraulic roughness and is a translation of the Colebrook-White equation (Equation 1.2).

$$\epsilon = 3.7\mathrm{D}\left[10^{-\frac{1}{2\sqrt{f}}} - \frac{2.51}{\mathrm{Re}\sqrt{f}}\right] \qquad 2.5$$

where, ϵ is the hydraulic roughness coefficient and Re is the Reynolds number as determined by Equation 2.6.

$$Re = \frac{\overline{\mathrm{V}}D}{v} \qquad 2.6$$

where v is kinematic viscosity of water and is adjusted for water temperature.

Since biofilm demonstrates a viscoelastic behavior, the relationship of friction factor with respect to the Reynolds number will help confirm Barton's findings that biofouled conduits do not follow the Colebrook-White model at different Reynolds numbers.

2.3 Estimation of Energy Costs

District records include an electrical profile of how many kilowatt hours each pump station uses on a daily basis. An average daily flow rate is also known. A ratio of gallons per kilowatt hour for each pump configuration can be calculated and compared from year to year. District records also include a total monthly cost associated with a total kilowatt hours used. A cost per kilowatt hour can be calculated, and in turn, a cost per gallon can be calculated. The costs include various tariffs. Energy costs are very labyrinthine due to tariffs and coincidental and non-coincidental peak charges and are beyond the scope of this study.

Since water delivery from the lake pump stations requires boosting, to get a true pumping cost for each pump configuration, calculations will need to include the electrical costs for each booster pump station used.

Table 2.1 shows which pump stations will need to be added to determine pumping costs for each pump configuration. Next determine the cost per gallon with Equation 2.7. There are two different costs per gallon to determine: maximum flow and minimum flow

$$\frac{\$}{gallon} = \frac{\$}{day} \div Q \qquad 2.7$$

where Q is in MGD.

Based on the dose, the pounds per day value can be determined with Equation 2.8

$$\frac{\#}{day} = dose \times Q \times 8.34 \frac{\#}{gallon} \qquad 2.8$$

where $dose$ is in mg/L, Q is in gallons/day.

The chlorine to ammonia ratio is a 4:1 and is determined using Equation 2.9

$$\frac{\#}{day}_{NH_4} = \frac{\#}{day}_{Cl_2} \div 0.76 \qquad 2.9$$

The District has contracted to purchase chemicals at the rates in Table 2.3 and a cost per day can be established

Table 2.3: Current Chemical Costs

Chemical	***Cost per ton***
Chlorine gas	$ 580
19% aqueous NH_4	$ 150

Compare the cost per day in 2012 to the values in 2010 and 2011 to the total chemical costs to determine if properly dosing chemicals are cost effective.

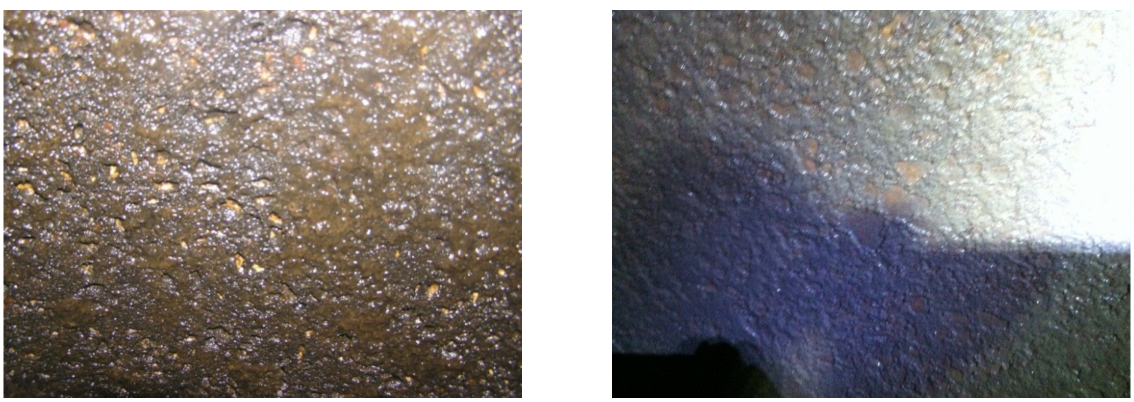
CHAPTER 3

RESULTS AND DISCUSSION

Traditional hydraulic analysis uses a single value for friction factor based on an assumption of the hydraulic roughness of the pipe. Actual operations of large diameter pipelines have found the friction factor to vary seasonally.

3.1 Cedar Creek and Richland Chambers Comparison

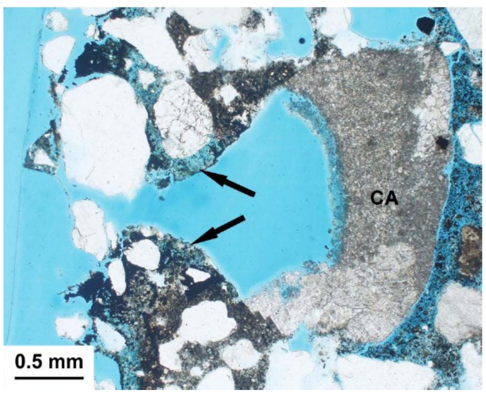
According to studies by Characklis (1979), McCoy and Costerton (1982), pressure drops increase when biofilm grows beyond the theoretical thickness of the viscous sublayer. The rougher the pipe wall, the larger the hydraulic roughness which indicates a higher theoretical thickness of the viscous sublayer.



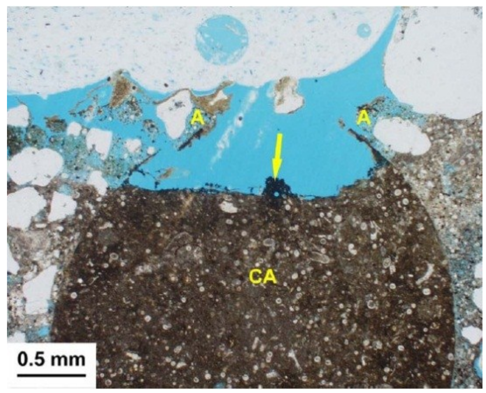
(a) Cedar Creek (b) Richland Chambers

Figure 3.1: Observed Wintertime Biofilm on (a) Cedar Creek and (b) Richland Chambers Pipe Interiors.

Figure 3.1 (a) is a photo of the inside of the mainline of the Cedar Creek pipeline. The wall of the pipe is very rough in the Cedar Creek pipeline as evidenced with the pocked concrete. The Richland Chambers line is not as old and does not have the severe pocking occurring internally. A petrographic analysis performed on concrete taken from the internal concrete core found that the rugosity of the pipe wall is due to the dissolution of the limestone aggregate from the internal concrete lining (Highbridge Materials, 2010). Please note that biofilm does not attach to the aggregate in both cases. The Cedar Creek aggregate (a) is a deeper loss in material compared to the Richland Chambers aggregate (b) and can be attributed to two reasons: aggressiveness of the source water and pipe age. Cedar Creek water is more aggressive than Richland Chambers water and is an older pipe as well.



(a) Cedar Creek



(b) Richland Chambers

Figure 3.2: Aggregate Dissolution in Pipeline Concrete Liner (Highbridge Materials) for (a) Cedar Creek and (b) Richland Chambers

The hydraulic roughness is determined from the historic data using Equation 2.5. Assuming the biofilm is at a minimum in the winter and using the calculated minimum hydraulic roughness as a representation of the pipe roughness, Figure 3.3, obtained from the Bureau of Reclamation, gives a good classification of the condition of the pipe. The calculated ϵ value of 0.0032 feet for Cedar Creek is high enough to be considered “Unusually Rough” and can be attributed to the erosion of the concrete aggregate. The minimum hydraulic roughness for Richland Chambers is calculated to be 0.0006 feet where Figure 3.3 classifies the pipe wall as “Centrifugally cast concrete pipe”.

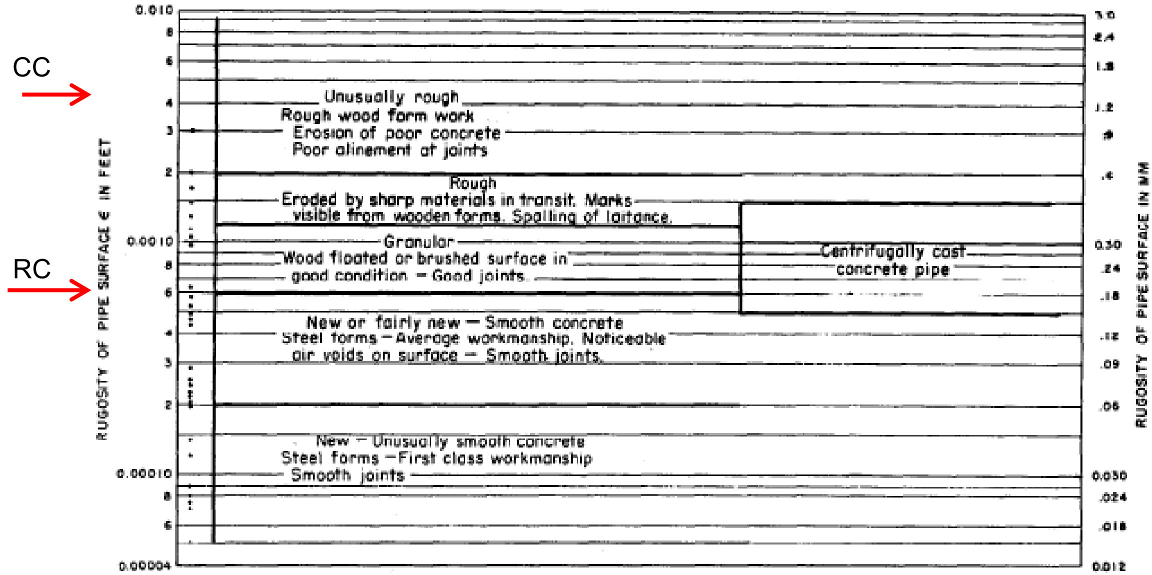


Figure 3.3: Rugosity Values in Concrete Pipe (Bureau of Reclamation, Figure 14)

The physical profile of the pipe directly impacts head loss. The hydraulic roughness of the pipe remains constant in the fully turbulent range in a classical Colebrook-White relationship as shown in a typical Moody Diagram. When plotting the friction factor versus the Reynolds number of the study data, it is found that use of the Moody Diagram to determine friction factor based on an assumed pipe condition breaks down. Figure 3.4 and Figure 3.5 demonstrates the variability of the friction factor and Reynolds number when pumping with three pumps at low capacity for the Richland Chambers pipeline Variability in friction factor is attributed to the growth and regression of biofilm thickness while variability in the Reynolds number is directly linked with the viscosity of the water which is adjusted based on water temperature. The general shape starting in winter has low friction factors and low Reynolds numbers. The values then increase in both cases until the July or August months as demonstrated in Figure 3.4. After the summer months, the opposite occurs and the friction factors and Reynolds numbers return to being low as illustrated in Figure 3.5. Both figures confirm that biofouled pipe friction factor can

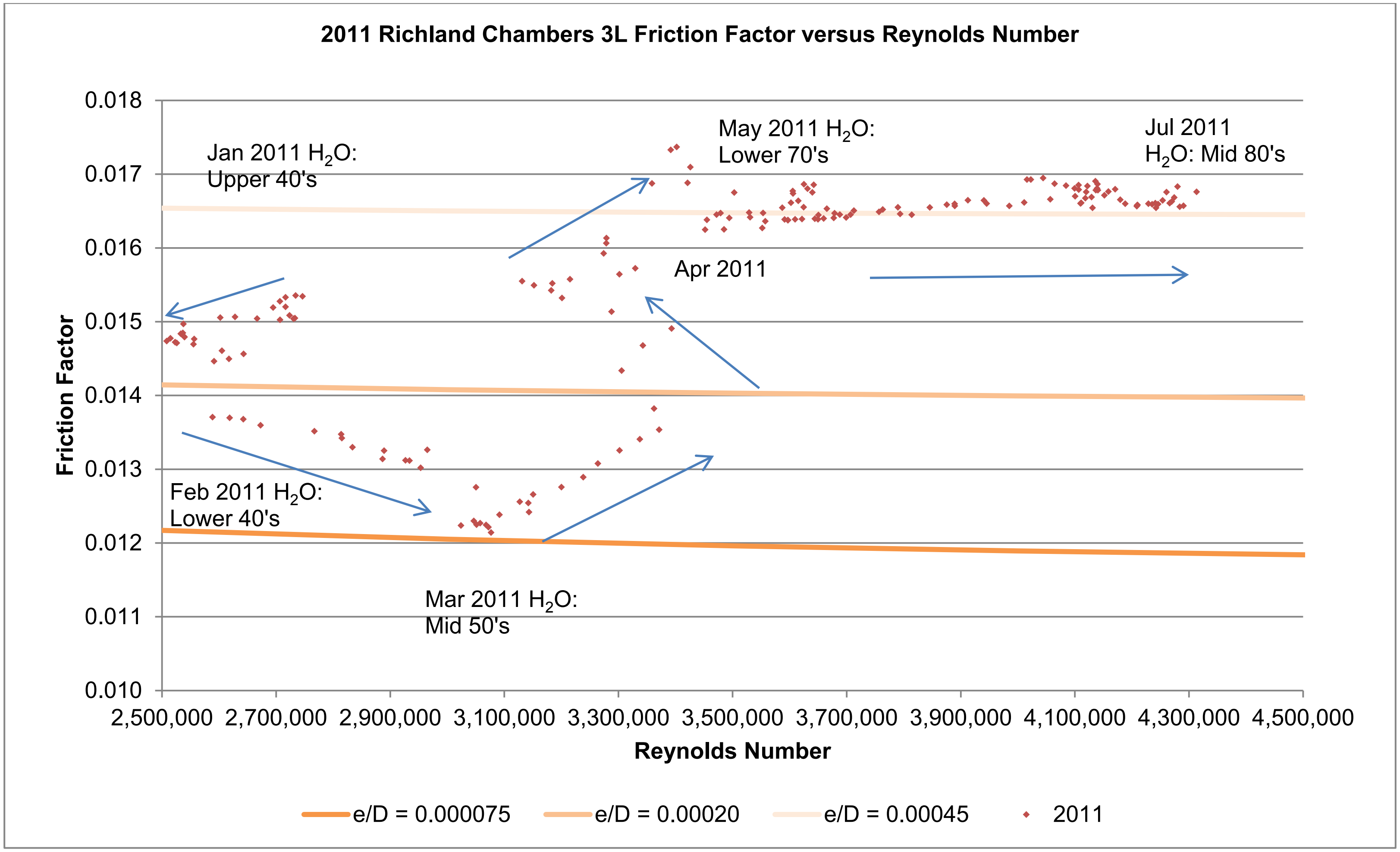


Figure 3.4: Friction Factor versus Reynolds Number for 2011 3L Richland Chambers Pump

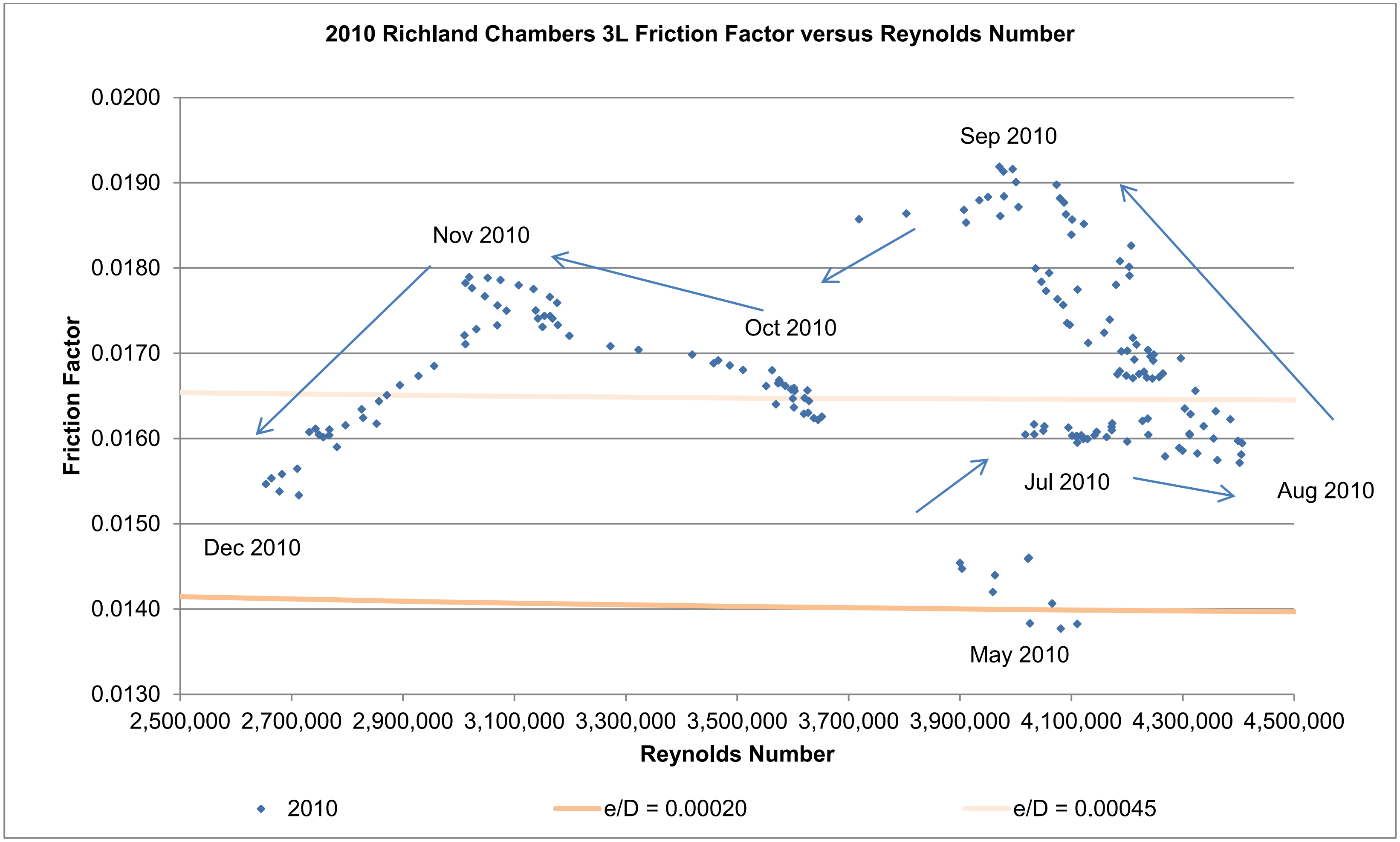


Figure 3.5: Friction Factor versus Reynolds Number for 2010 3L Richland Chambers Pump

no longer be determined using a Moody Diagram and an assumed hydraulic roughness based on pipe condition.

3.2 Wintertime versus Summertime Comparison

The variability of the hydraulic roughness of the pipe wall changes based on the time of year water is delivered due to biofouling. Based on the study performed by Vo (2010), the thickness of biofilm drives the friction factor value. Since the friction factor dynamically changes through the year, it is hypothesized that the dynamically changing friction factor is due to biofilm activity and changes seasonally with higher friction factors seen in the summer and lower friction factors occurring in the winter. Since summarizes the values of both summer and wintertime friction factors. The difference between the friction values for the Cedar Creek pipeline ranges from 0.001 to 0.003 decrease from summertime pumping to wintertime pumping.

Table 3.1: Cedar Creek/Richland Chambers Summer/Winter Friction Factors

	Cedar Creek			***Richland Chambers***		
	Summer	***Winter***	***Difference***	***Summer***	***Winter***	***Difference***
2010	0.019	0.017	0.002	0.018	0.015	0.003
2011	0.021	0.018	0.003	0.017	0.013	0.004
2012	0.018	0.017	0.001	0.014	0.014	0

The daily friction factors for the summer through winter months for the past three years are plotted in Figure 3.6 for the Cedar Creek pipeline. The values describe where the higher friction factors occur in the summer months (July through the end of September) compared to the values occurring in the winter (October through December). The abrupt drop occurring after November 11th corresponds to a 14 hour shut down of the Cedar Creek pipeline. A possible explanation for this could be that when the pumps shut down, the water column changes the direction of flow as a part of transient control by slowly closing pump control valves at the pump stations. The change in direction could peel back the biofilm and diminish its thickness.

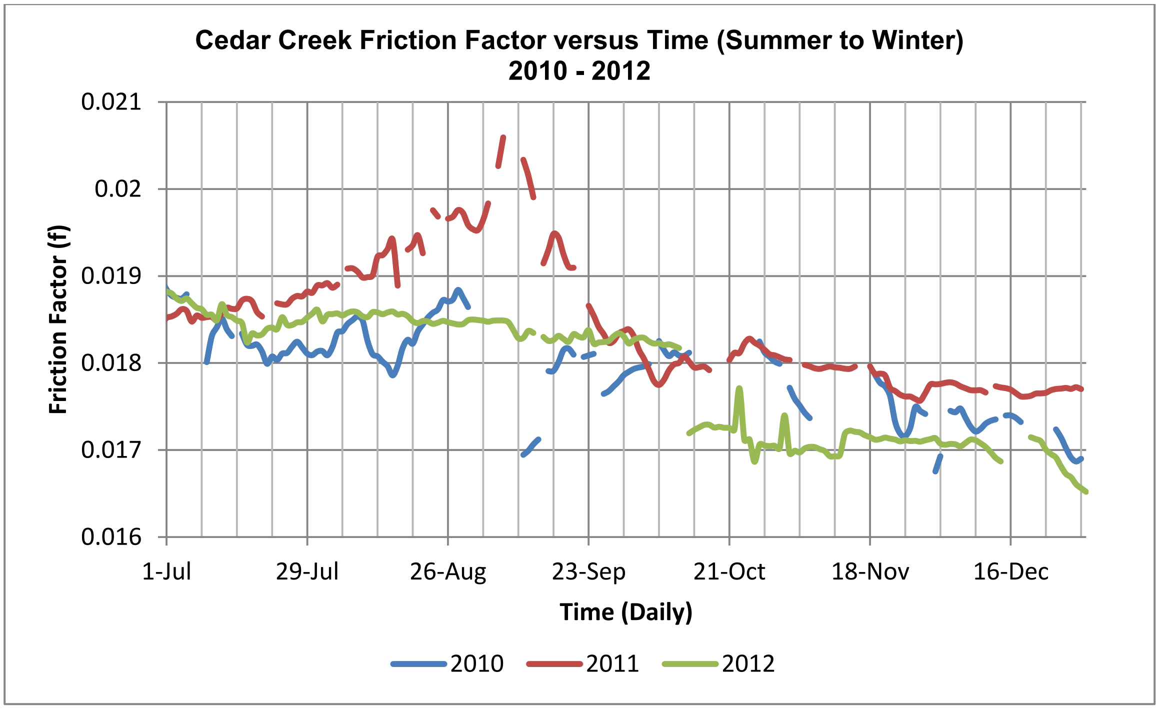


Figure 3.6: Cedar Creek Friction Factor versus Time from Summer to Winter

The Richland Chambers friction factor differences are larger than the Cedar Creek differences with the values ranging between 0 and 0.004. The daily friction factors for the summer through winter months for the past three years are plotted in Figure 3.7 for the Richland Chambers pipeline. The pattern seen in the 2010 and 2012 years show a rise in the friction factor in the October/November time frame and is considered to be possibly linked to the turnover of the lake which would be a food source for biofilm. Determining a link between the lake turnover and the friction factor is beyond the scope of this study. Another observation is the wintertime friction factors for Cedar Creek are very close in value to the Richland Chambers summertime friction factors and may be due to the pipe wall roughness of Cedar Creek being higher in value than the Richland Chambers roughness as discussed in Section 3.1. Since Characklis, (1979) found that biofilm thickness does not increase friction until the thickness exceeds the theoretical viscous sublayer thickness, the higher wintertime friction factors in the

Cedar Creek line may be due to having a thicker theoretical viscous sublayer due to the rougher pipe wall.

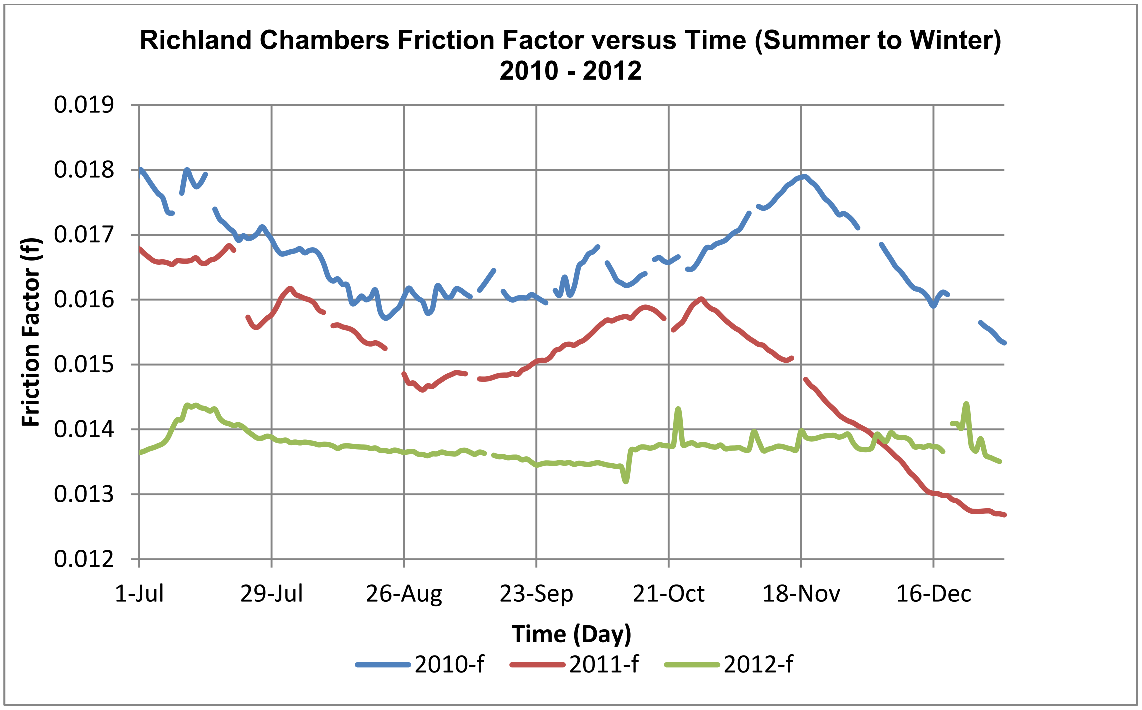


Figure 3.7: Richland Chambers Friction Factor versus Time from Summer to Winter

Since the hydraulic roughness of the pipe wall very slowly changes over the life of the pipeline, there is evidence that the summer and winter friction factor values are likely attributed to seasonal biofilm growth and decline.

3.3 Chloramine Treatment Comparison

A method used to control biofilm growth is through the use of chloramines. As demonstrated in Section 0, there is a difference between winter and summer friction factors. This thesis hypothesizes that proper chloramine application will reduce friction factors by comparing the summer to summer chloramine treatment during the study period.

The 2010 (Figure 3.10) and 2011 (Figure 3.9) Cedar Creek chloramine treatment is considered ineffective as the chlorine residuals at Waxahachie are below the value of 0.05

mg/L. The Kruzic (2011) study determined biofilm will not grow when the chloramine residual is above 0.05 mg/L.

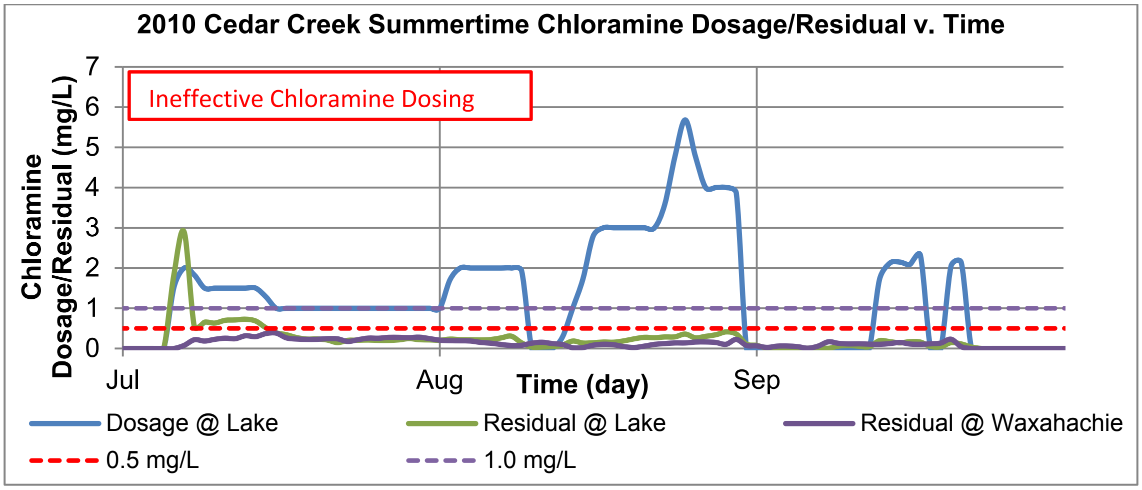


Figure 3.8: 2010 Cedar Creek Summertime Chloramine Dosing

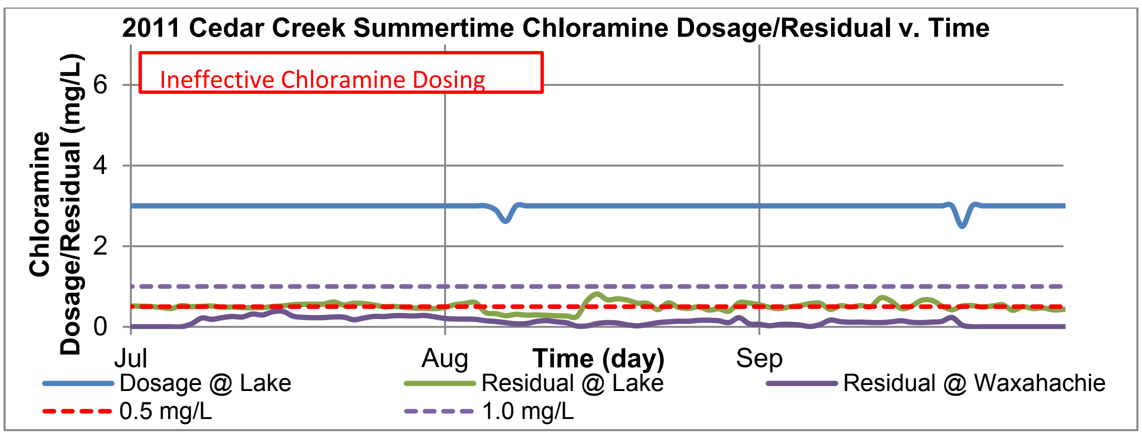


Figure 3.9: 2011 Cedar Creek Summertime Chloramine Dosing

In Figure 3.10, the Cedar Creek 2012 chloramines treatment is considered effective as chlorine residuals above 0.05 mg/L are observed at Waxahachie.

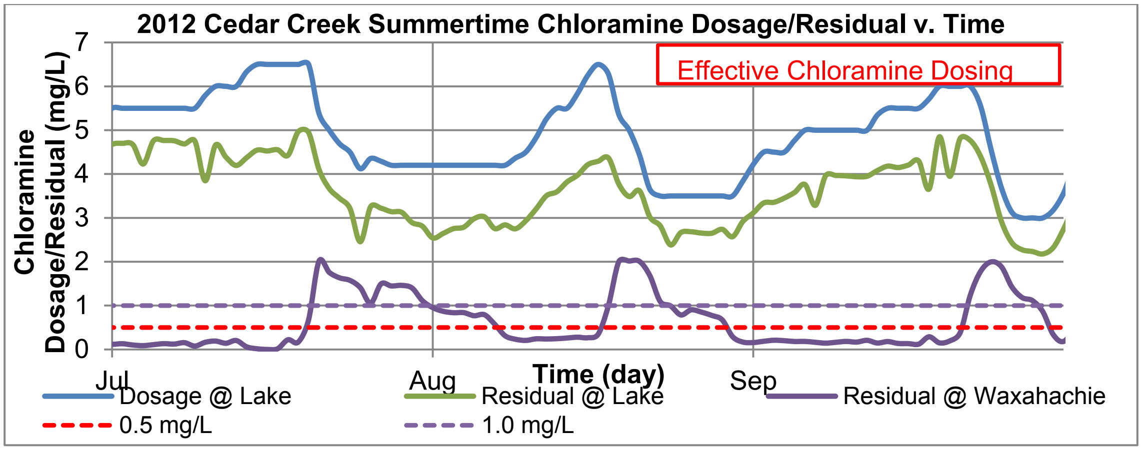


Figure 3.10: 2012 Cedar Creek Summertime Chloramine Dosing

No chloramines were used in the Richland Chambers lines during 2010 and 2011 due to chemical system upgrades. The Richland Chambers chloramine residual at Waxahachie is effective in 2012 with a majority of the values over 1.0 mg/L as seen in Figure 3.11. During the month of July, the aqueous ammonia line scaled closed and only free chlorine was dosed for a period of time. This was remedied in the middle of July and chloramine effectiveness increased as evidenced with seeing a residual at Waxahachie.

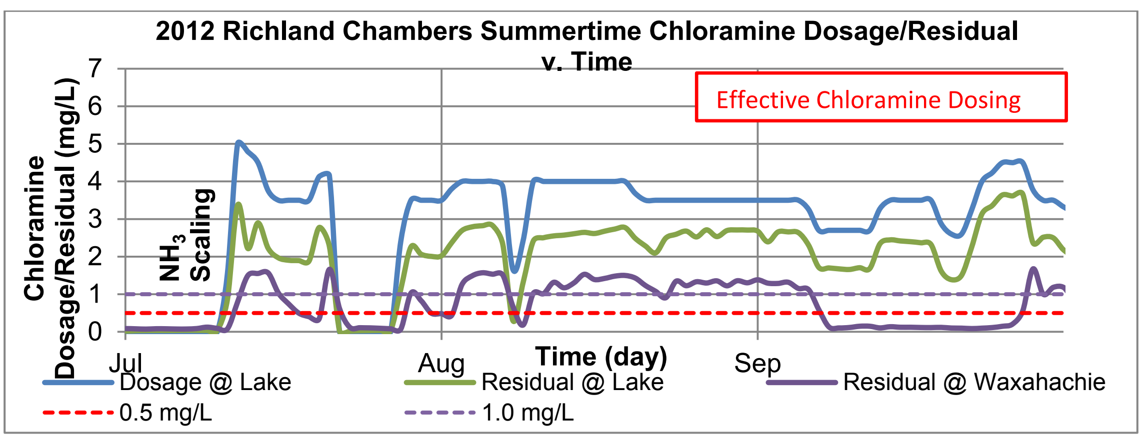


Figure 3.11: Richland Chambers Summertime Chloramine Dosing for 2012

Evidence is found for when chloramines are applied, a discernible impact to friction factors occurs. Using the 2012 numbers as a baseline, Table 3.2 lists the difference in friction factor from the two years where there were little to no chloramine residuals found at Waxahachie.

Table 3.2: Friction Factor Summer/Summer Comparisons to 2012 Values

	Cedar Creek	*Richland Chambers*
2010	0.002	0.004
2011	0.004	0.003

Since both the Cedar Creek and Richland Chambers friction factors decreased in 2012 when chloramines are applied effectively during the summer months, the biofilm thickness did not grow as thick as compared to operating without chloramines. Further work is needed to determine how to consistently apply chloramines so the residual variability can be steadied.

Figure 3.12 is a chart showing the Cedar Creek friction factors against the Reynolds numbers for the whole study period. The greatest difference occurs in 2012, which is attributed to effective chloramine treatment, where the friction factor between May and August of 2012 is somewhat stable around the 0.016 value. Figure 3.13 is the chart for Richland Chambers. The 2010 and 2011 friction factors illustrate the seasonality variation as compared to the 2012 values. The lower friction factors in both the Cedar Creek and Richland Chambers lines during 2012 is an indication that proper chloramines application keeps friction factors down during traditionally high friction factor months.

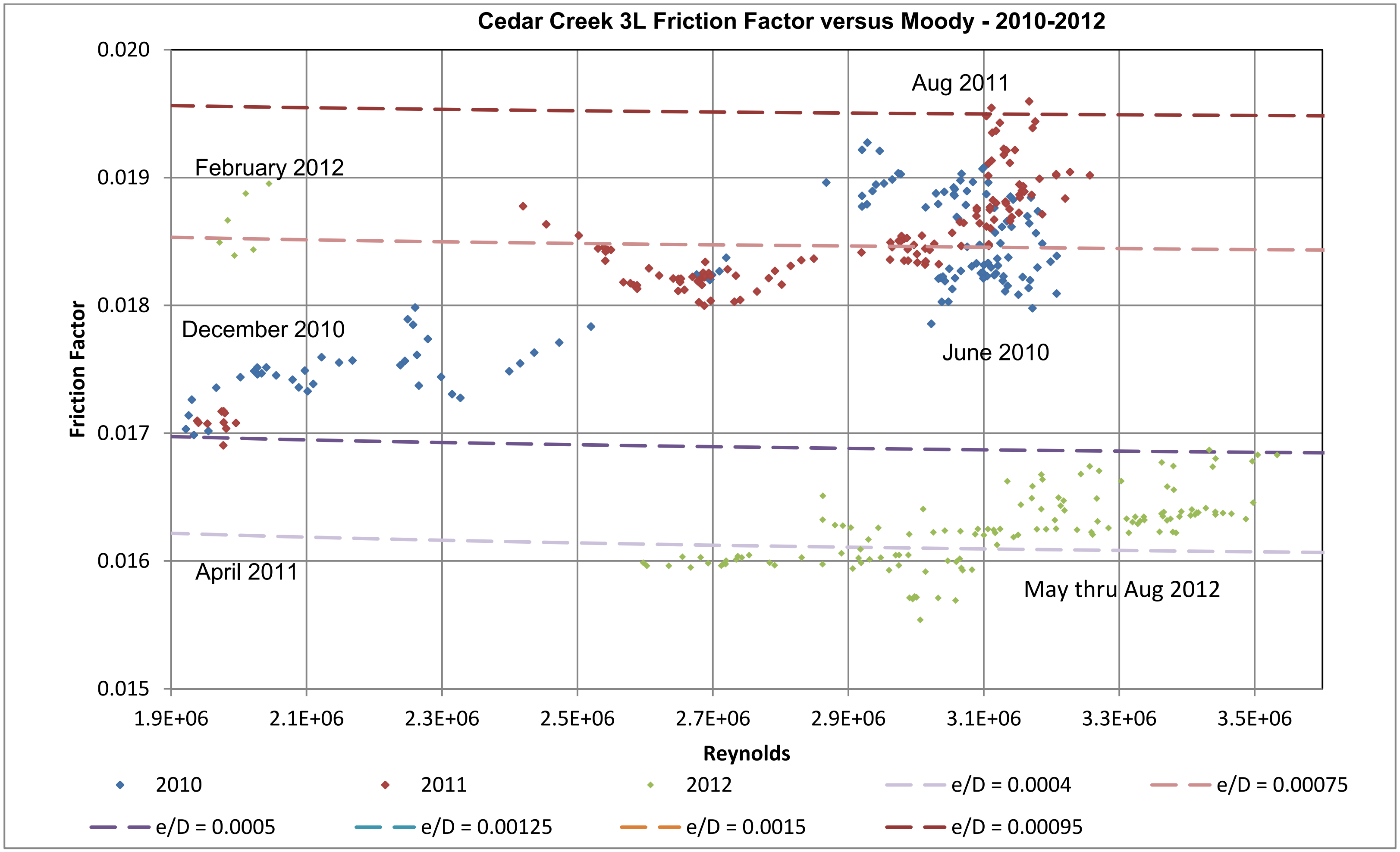


Figure 3.12: Friction Factor versus Reynolds Number for 3L Cedar Creek

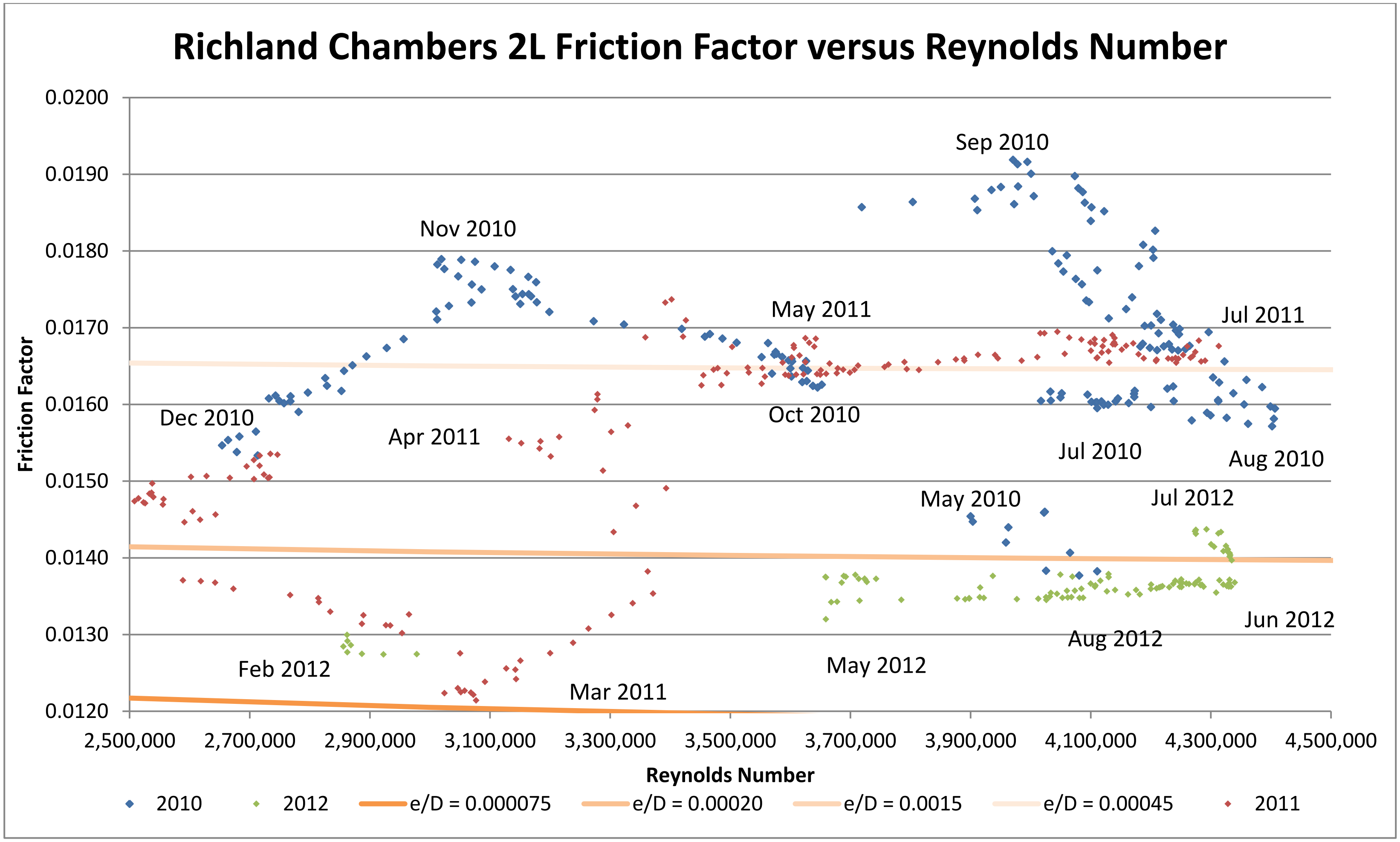


Figure 3.13: Friction Factor versus Reynolds Number for 3L Richland Chambers

3.4 Pump Configuration Comparison

Velocity conversely impacts the thickness of the biofilm as it will deform in shape due to the fluid stress (Vo, 2010). Slower velocities will have lower stress, thus allowing the biofilm to be thicker. Evidence of velocity impacting biofilm thickness is derived from Figure 3.14 and Figure 3.15. Velocity is directly proportional to the flow rate as the area of the pipe does not change. Low flow rates equate to low velocities in the pipe. Biofilm thickness is related to the friction factor in a similarly proportional manner.

In March of 2011 (Figure 3.14), the friction factors for the 2L pump configuration increases significantly; but there are no comparisons available for that month in 2010 or 2012. This thesis cannot draw a conclusion based on the data obtained. Two observations can be made. One, whenever there is a period where a pump is shut off, the friction factor decreases.

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Whenever there is a pump combination change, the friction factor changes also. The friction factor increases from 0.015 to 0.016, for example, when the pump combination increases from 2L to 3L in the April-May 2012 timeframe. The same thing occurs in January/February 2012 when going from the 3H pump combination where the friction factor is 0.018 to the 3L friction factor value of 0.017. The Cedar Creek pipeline finds the lower the velocity, the lower the friction factor. Both timeframes discussed above occurred prior to chloramine treatment. Richland Chambers (Figure 3.15), demonstrates the same type of relationship in 2010. In 2010, the 1L pump configuration has a calculated friction factor value of 0.012 in April. The pump configuration changes to 2L and the friction factor goes up slightly to 0.013. The greatest difference occurs from the 2L to 3L pump change with the friction factor going from 0.014 to 0.019 in June. All these scenarios occurred when chloramines were not being applied. Further investigation in pump configuration and friction factor is needed when the chloramines are being properly applied to determine if there is a similar relationship.

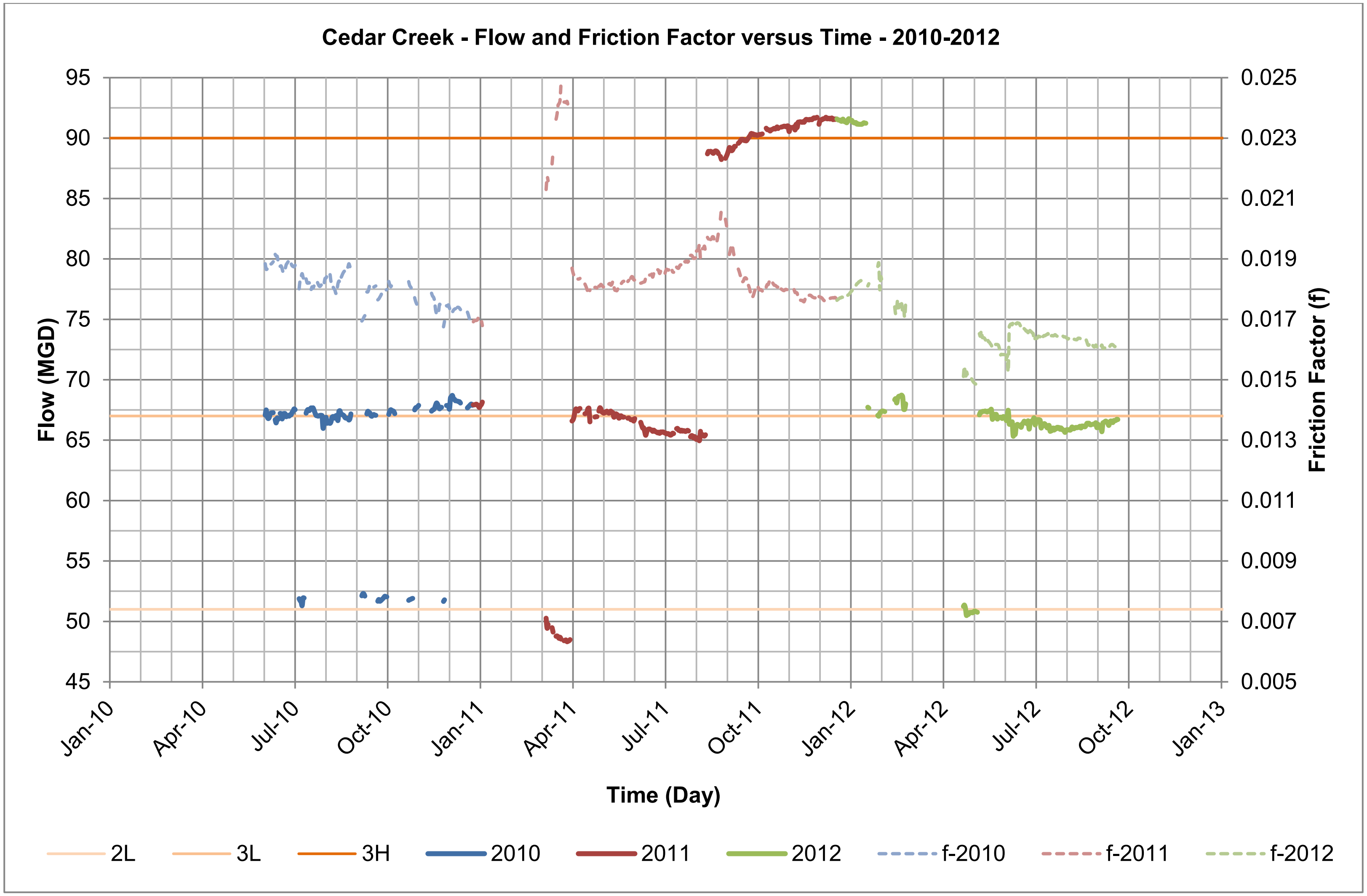


Figure 3.14: Flow Rates and Calculated Friction for the Cedar Creek Lake Pump Station

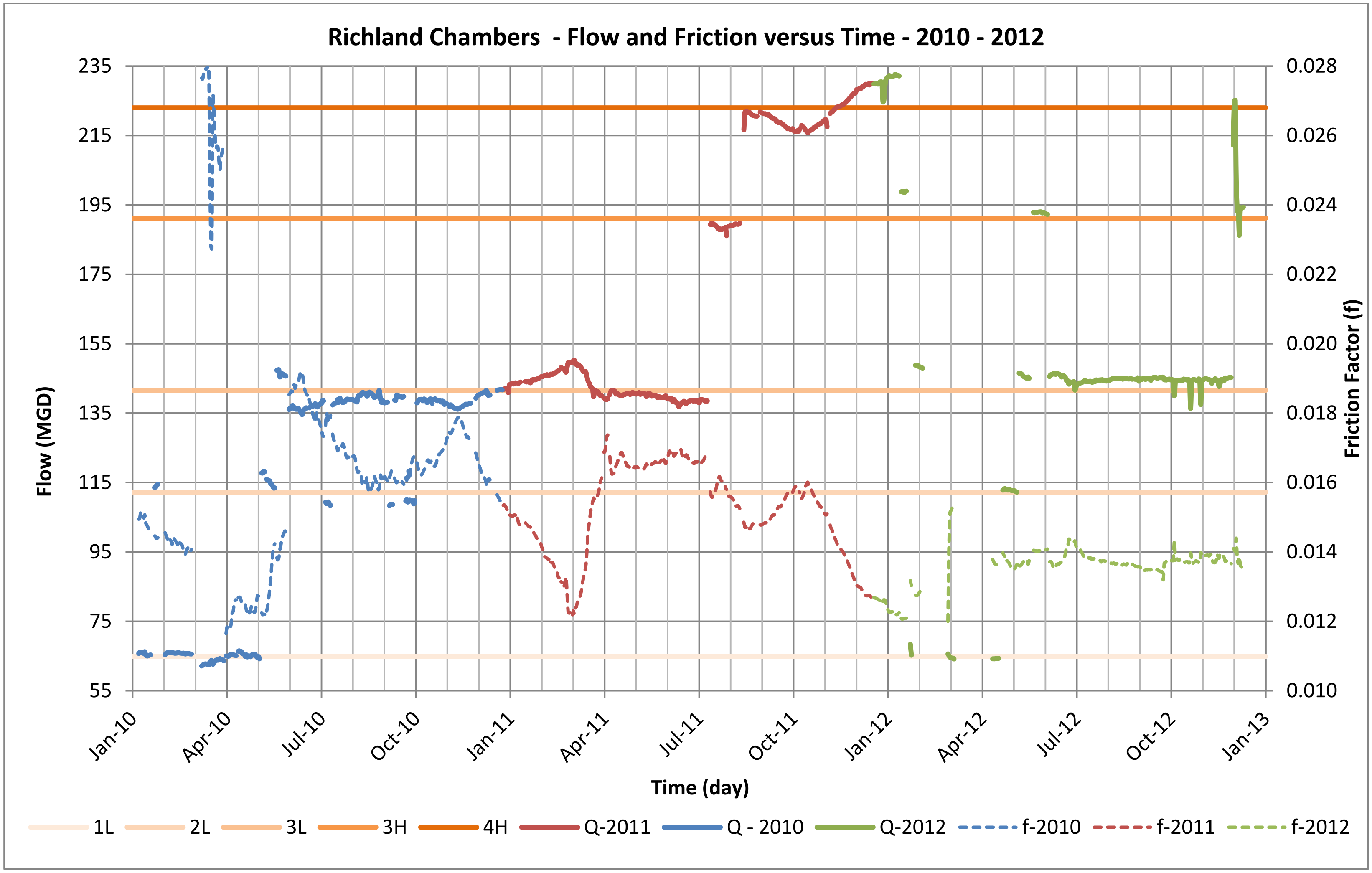


Figure 3.15: Flow Rates and Calculated Friction for the Richland Chambers Lake Pump Station

3.5 Spatial Comparison

Because chloramine decay occurs as the water travels up the line, biofilm thickness may vary in the form of higher friction factor values. The closer to the lake pump station where chloramines are treated could possibly be due to the lower the friction factor because the biofilm is exposed to a higher chloramine residual.

The location in the pipeline impacts the friction factor and by proxy, the thickness of the biofilm. The field study described in Section 2.2.1.2 calculated the friction factor for 13 segments on a 15-minute interval from October 2011 to July 2012. Each pump combination is separated out by location and an average friction factor is calculated.

Figure 3.16 and Table 3.3 illustrates friction factor values found along the Cedar Creek pipe. The friction factor is low between the lake pump station and the pressure transducer located at the Gravel Pit #2 (mile 7.3). The friction factor dips for the 2L and 3L low capacity pump combinations at County Road 3250 (mile 13.5) but the 3H pump combination did not experience the dip. Since these are averages, the values may be influenced by the time of year as the 3H pump combination occurred only during the wintertime when biofilm effects are considered low and both the 2L and 3L pump combinations occur during the spring and summer when friction factors increase. The friction factor spikes at the Ennis pump station for all three pump configurations. The 3H values at the spike are lower than the 2L and 3L configurations. A spike, however, is observed and assumed to be due to biofilm being thicker in that area. Another dip is seen at the 32 mile mark (Garrett) and can be attributed to the failure of the pressure transducer beginning May 17th. Loss of this data impacts the Crisp Pike to Garrett values and the Garrett to Ebenezer Road values. The 2012 data for the 2L and 3L pump configuration for that location has a smaller data set to average and could be the reason for the variability in this location. Further study is warranted to determine if the biofilm thickness varies in these locations.

Table 3.3: Cedar Creek Friction Factor Spatial Comparisons by Pump Configuration

Segment	***Description***	***1L***	***2L***	***3L***	***3H***	***Miles***
1	CC1 Pump Station to SH 274		0.005	0.011	0.012	0.7
2	SH 274 to Gravel Pit #2	0.016	0.018	0.017	0.018	7.3
3	Gravel Pit #2 to Ready Mix	0.025	0.020	0.019	0.018	10.9
4	Ready Mix to CR 3250	0.012	0.016	0.017	0.018	13.5
5	CR 3250 to Rosewood	0.027	0.020	0.019	0.018	15.5
6	Rosewood to Walker Creek	0.016	0.018	0.018	0.018	19.7
7	Walker Creek to FM 1181	0.025	0.021	0.019	0.018	24.5
8	FM 1181 to CC2	0.026	0.025	0.020	0.019	25.5
9	CC2 to Crisp Pike Rd		0.023	0.027	0.021	28.6
10	Crisp Pike Rd to Garrett	0.018	0.018	0.018	0.020	32.0
11	Garrett to Ebenezer Rd	0.019	0.020	0.018	0.019	34.6
12	Ebenezer Rd to Graves	0.015	0.017	0.018	0.018	38.4
13	Graves to CC3	0.015	0.017	0.018	0.019	42.9

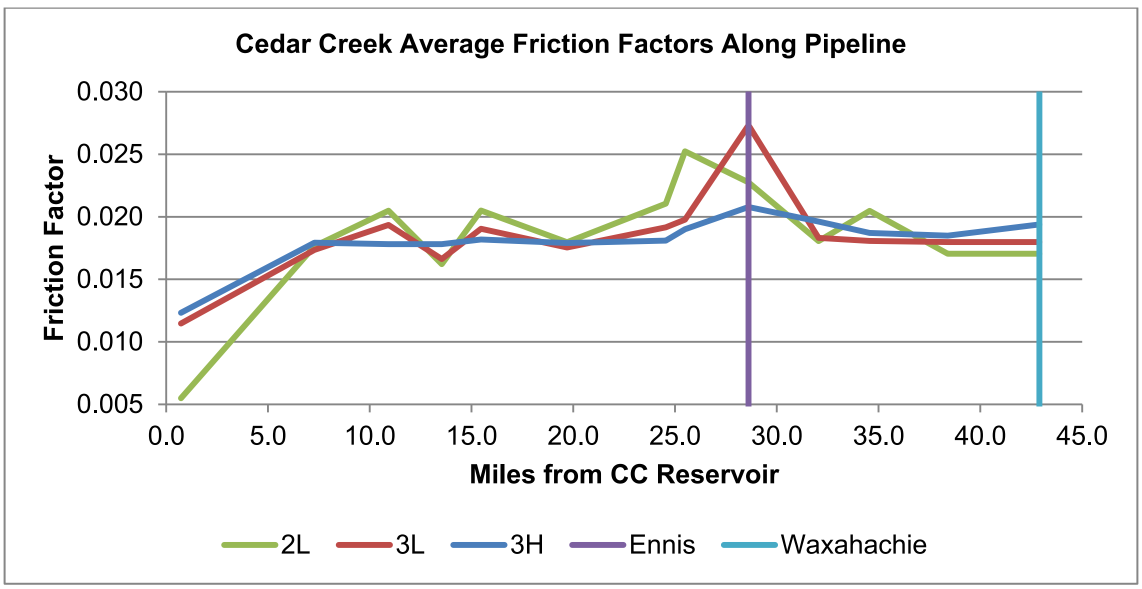


Figure 3.16: Friction Factor versus Distance from Cedar Creek Lake Pump Station

Richland Chambers behaves similarly, but a dip occurs at Garrett and then spikes at Ebenezer Road (mile 38.8) for all three pump combinations. Table 3.4 lists each friction factor along the Richland Chambers pipeline. The friction factors go down at the spike in Figure 3.17. . Contrary to what is previously discussed in Section 3.4 regarding how the velocity impacts biofilm thickness, the slower velocities of 2L yields a friction factor of 0.028 versus a value of 0.015 seen when pumping at a 4H pump combination. Since friction factor is directly related to biofilm thickness, it is possible that the Ebenezer Road location on the Richland Chambers pipe line may have the thickest biofilm growing at that location with lower biofilm thicknesses in the segments before and after the spike. Additional research is needed, however, to test this hypothesis.

Table 3.4: Richland Chambers Friction Factor Spatial Comparisons by Pump Configuration

Seg	*Description*	*1L*	*2L*	*3L*	*3H*	*4H*	*Miles*
1	RC1 to FM 2859	0.018	0.012	0.014	0.013	0.012	3.2
2	FM 2859 to SH 31	0.013	0.015	0.016	0.014	0.014	7.8
3	SH 31 to FM 2100	0.015	0.013	0.012	0.013	0.015	10.8
4	FM 2100 to FM 1129	0.012	0.013	0.013	0.013	0.014	14.8
5	FM 1129 to CR 1030	0.013	0.013	0.014	0.014	0.015	19.2
6	CR 1030 to Barker Road	0.019	0.016	0.015	0.015	0.015	23.4
7	Barker Road to SH 85	0.014	0.015	0.014	0.013	0.014	27.7
8	SH 85 to RC2	0.012	0.013	0.015	0.016	0.016	29.7
9	RC2 to Crisp Pike Rd	0.021	0.014	0.014	0.014	0.014	32.9
10	Crisp Pike Rd to Garrett	0.031	0.006	0.009	0.011	0.011	36.3
11	Garrett to Ebenezer Rd	0.043	0.028	0.023	0.018	0.015	38.8
12	Ebenezer Rd to Graves	0.011	0.010	0.010	0.012	0.014	42.7
13	Graves to RC3	0.020	0.015	0.014	0.015	0.016	47.2

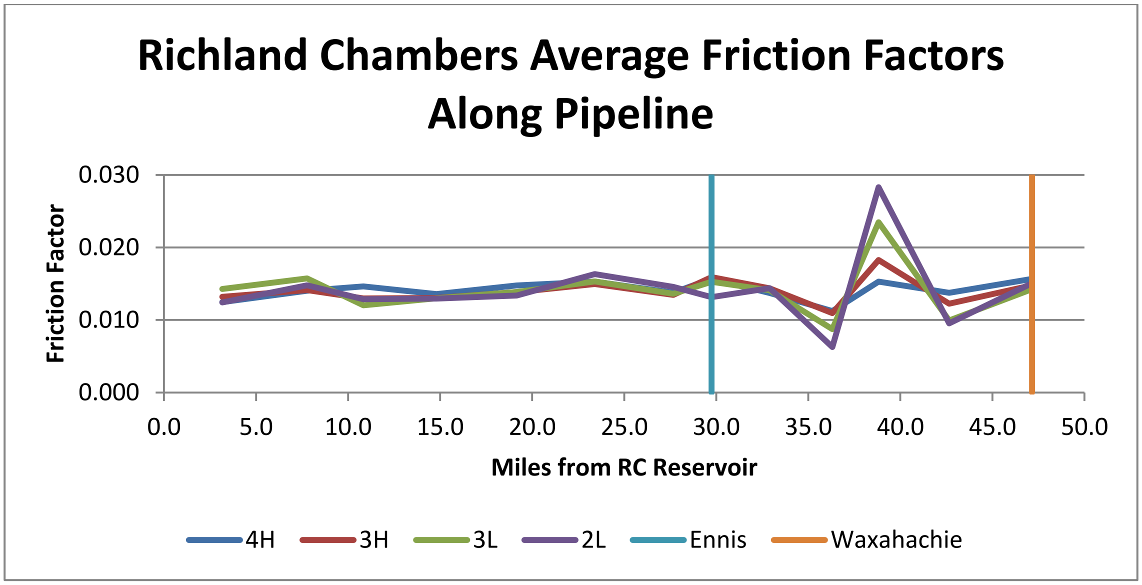


Figure 3.17: Friction Factor versus Distance from Richland Chambers Lake Pump Station

A spatial relationship does exist along the pipeline, but the evidence show there are areas with higher friction factors located away from the lake pump station. There may be other biological and chemical processes influencing thicker biofilm in certain locations, but is beyond the scope of this study.

Chloramines effectiveness is also spatially dependent. The chlorine residual for 2010 and 2011 are not very effective and trace amounts of total chlorine residual are found in the Cedar Creek pipeline as shown in Figure 3.18 and Figure 3.19. Since there was chloramine applied in the 2010 and 2011 years, it is the possible reason why the friction factor is low from the 1 mile to 7 mile mark. Biofilm could not be well established in this area. The chlorine residuals have not been evaluated on the Richland Chambers line and should be considered for further study.

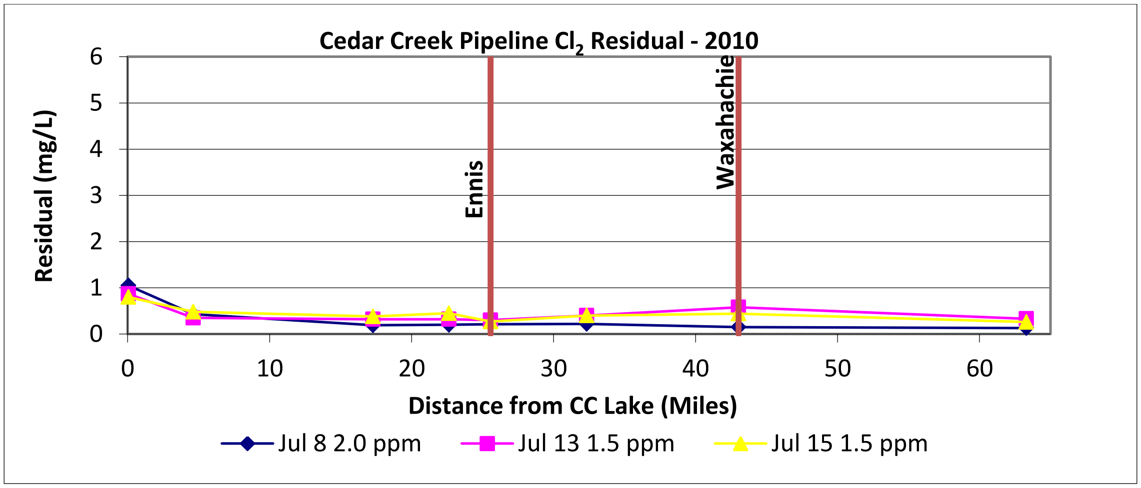


Figure 3.18: Cedar Creek 2010 Chloramine Residual versus Distance

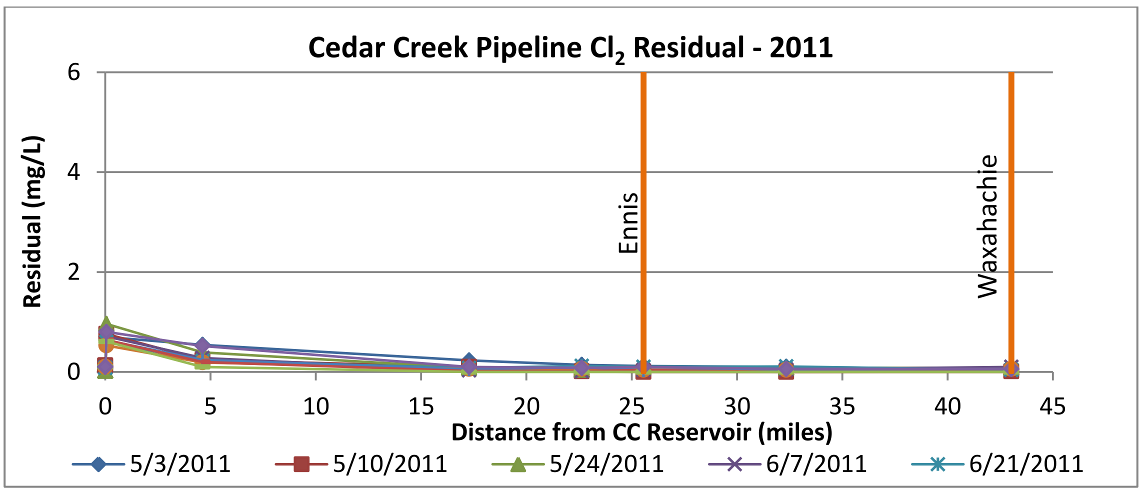


Figure 3.19: Cedar Creek 2011 Chloramine Residual versus Distance

The chlorine residuals in 2012 are appreciably higher at the lake which indicates the chloramines are effectively applied. The chlorine residual decays at a fairly slow rate until Walker Creek (mile 19.7). The chlorine residual decays rapidly from Walker Creek to Waxahachie and it varies seasonally. Please note that the friction factors in Figure 3.16 are all the same value at the same location and increase at Ennis. Once beyond Ennis, however, the friction factors return to the 0.018 value. The averages are most likely skewed due to the loss of data during the May through July timeframe of the Garrett pressure transducer.

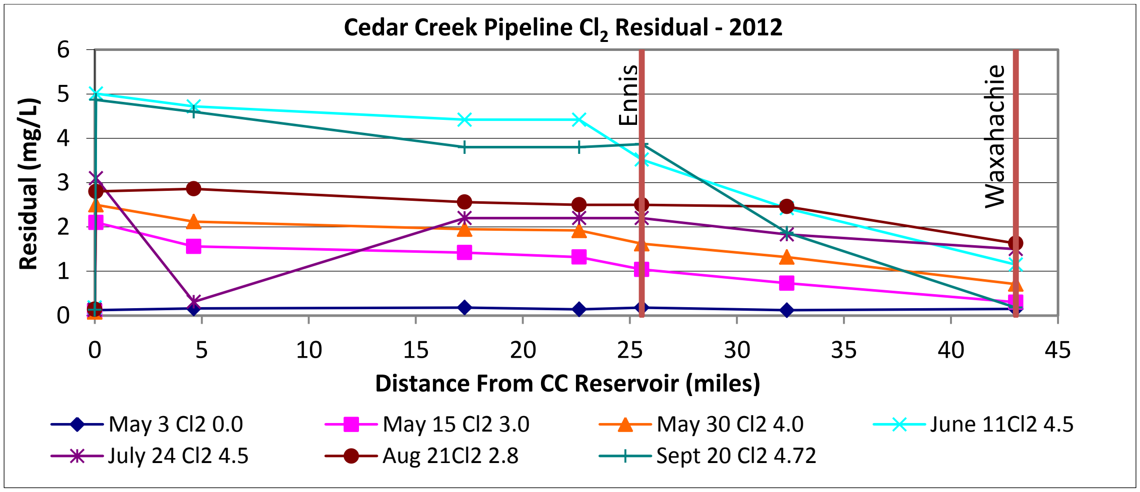


Figure 3.20: Cedar Creek 2012 Chloramine Residual versus Distance

The August 2012 run, shown in Figure 3.21, loses forty-three percent of the chlorine residual from the lake station to Waxahachie. At the time of sampling, the initial chlorine residual at the lake is marginal at 2.56 mg/L and decays down to a value of 1.63 mg/L at Waxahachie. The timing of the sampling occurs when a rapid decrease in the dosage occurs due to high residual values (> 1 mg/L) at Waxahachie. The September run indicates a loss of ninety-six percent of chlorine residual. At the time of sampling, the initial chlorine residual at the lake is somewhat high at 4.6 mg/L and decays down to a 0.18 mg/L at Waxahachie.

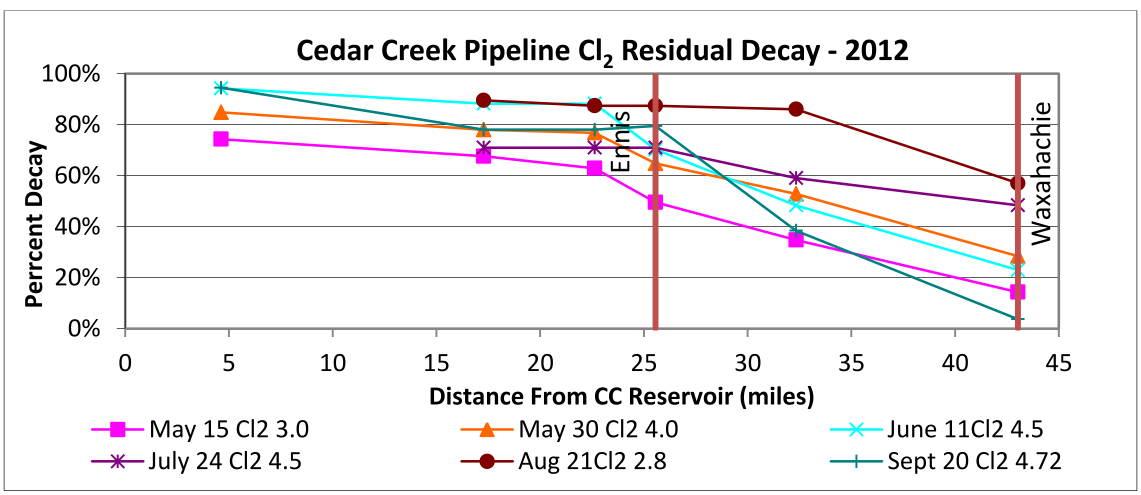


Figure 3.21: Cedar Creek 2012 Chloramine Residual Decay

There are two items to consider when doing a spatial comparison. One is the friction factor down the line and the other is chlorine decay. Friction factor varies with location and it is assumed the biofilm thickness varies throughout the line. The chlorine residuals decay with respect to pipeline location and are influenced by seasonal date of application.

3.6 Energy Analysis

Does the cost for chemicals outweigh the energy cost difference between a high flow rate and a low flow rate? Electrical profiles records for each pump station for each day of operation are available to aid in determining costs.

The 3L pump combination is used to compare between years due to coincident time periods through the study period. The lake stations and the Waxahachie booster pump stations are used to calculate energy costs.

3.6.1 Cedar Creek

The 2010 and 2011 data is compared to 2012 since it is the year where dosing with chloramines is more effective. The total energy and chemical costs for two different time periods are listed for each year in Table 3.5.

Table 3.5: Cedar Creek Energy and Chemical Costs – 2010 – 2012

	Energy Cost	**Chemical Cost**	**Avg $/MGD**	**% Increase**	**Energy Savings**
	June 6 through July 6				
2010	$ 333,773.88	$ -	$ 160.51	20.9%	$ 57,597.82
2011	$ 322,474.98	$ 19,951.53	$ 166.96	16.8%	$ 46,298.92
2012	$ 276,176.06	$ 31,571.54	$ 149.19		
	July 17 through August 21				
2010	$ 439,287.74	$ 10,949.49	$ 182.15	17.3%	$ 64,813.89
2011	$ 474,162.73	$ 10,717.93	$ 205.55	26.6%	$ 99,688.88
2012	$ 374,473.84	$ 37,278.16	$ 173.08		

For both time periods, the total energy and chemical costs are lowest in 2012 when chloramines are used. The average cost per million gallons per day decreases by approximately

$20 for each million gallons. Figure 3.22 displays the flow rates and energy costs during the June 6th through July 7th time period. The increased cost per million gallons for 2010 and 2011 compared to 2012 is an indication that properly dosing with chloramines is cost effective.

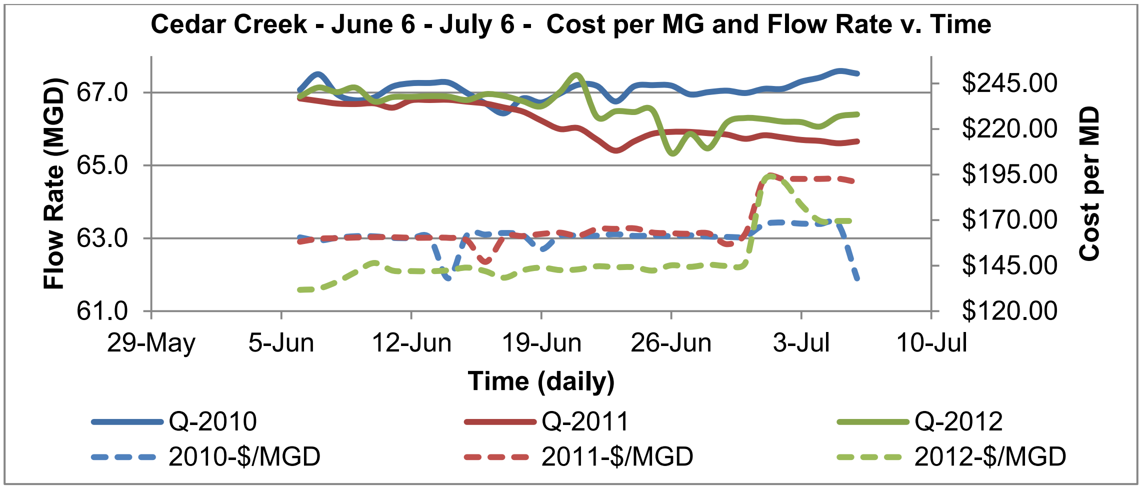


Figure 3.22: Cedar Creek Energy and Chemical Cost and Flow Rate Versus Time

3.6.2 Richland Chambers

There are no coincident times to compare all three years at a time, so two different times are evaluated. Dosing the water with chloramines is found to be effective as seen with lower energy and chemical costs (Table 3.6) just as with Cedar Creek.

Table 3.6: Richland Chambers Energy and Chemical Costs – 2010 – 2012

	Energy Cost	***Chem. Cost***	***Avg $/MGD***	***% Increase***	***Energy Savings***
	July 17 through September 9				
2010	$1,543,212.35	$ -	$ 205.25	17%	$ 221,682.62
2012	$1,321,529.73	$ 79,793.85	$ 179.77		
	June 22 through July 21				
2011	$1,008,440.53	$ -	$ 243.07	42%	$ 299,458.96
2012	$ 708,981.58	$ 21,518.85	$ 168.20		

The flow rates in 2011 shown in Figure 3.23 are lower than the 2012 flow rates. The cost per million gallons for 2012 includes both energy and chemical costs and the values are still lower than the 2011 values which do not have any chemical costs since there was no chloramine treatment that year.

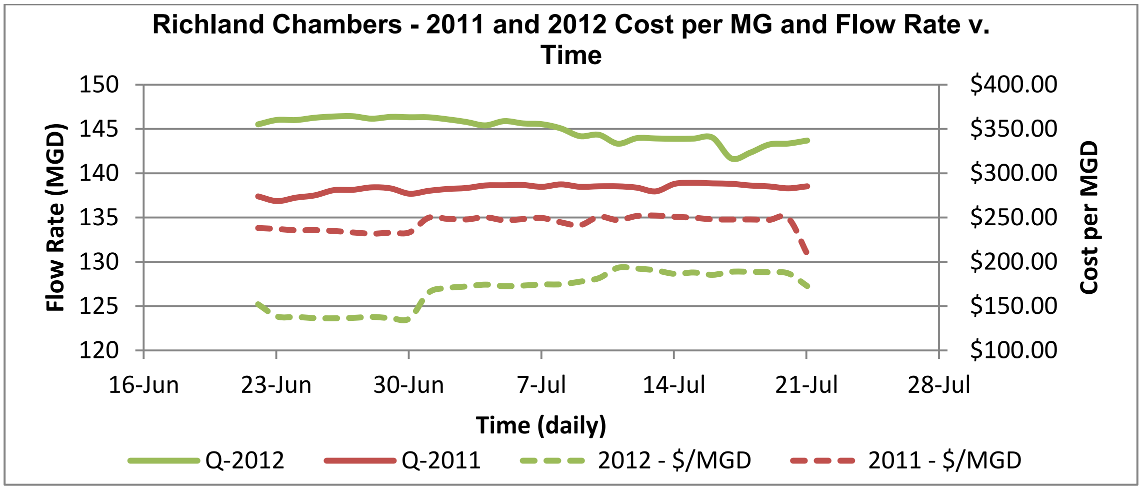


Figure 3.23: Richland Chambers Energy and Chemical Cost and Flow Rate Versus Time

3.7 Flow Rate and Friction Factor Analysis

Based on historic data obtained from the District's SCADA database, the friction factor is calculated as described in Equation 2.2 and plotted with the flow over the course of the study period in Figure 3.24 and Figure 3.25. The flow rate and friction factor are indirectly proportional as when the friction factor is high, the flow rate is low. This relationship usually occurs in the summer. Winter time flow rates increase because the friction factors decline. Table 3.7 lists the flow rates by pump combination broken out by year for Cedar Creek.

Table 3.7: Cedar Creek Flow Rates by Pump Configuration and Year

Cedar Creek	***2L***			***3L***			***3H***		
	2010	**2011**	**2012**	**2010**	**2011**	**2012**	**2010**	**2011**	**2012**
Min	51.3	48.3	48.3	66.0	65.0	65.3		87.9	91.1
Max	52.3	50.6	52.4	68.7	68.2	68.7		91.7	91.6
Average	51.9	49.2	50.0	67.3	66.5	66.6		90.2	91.3

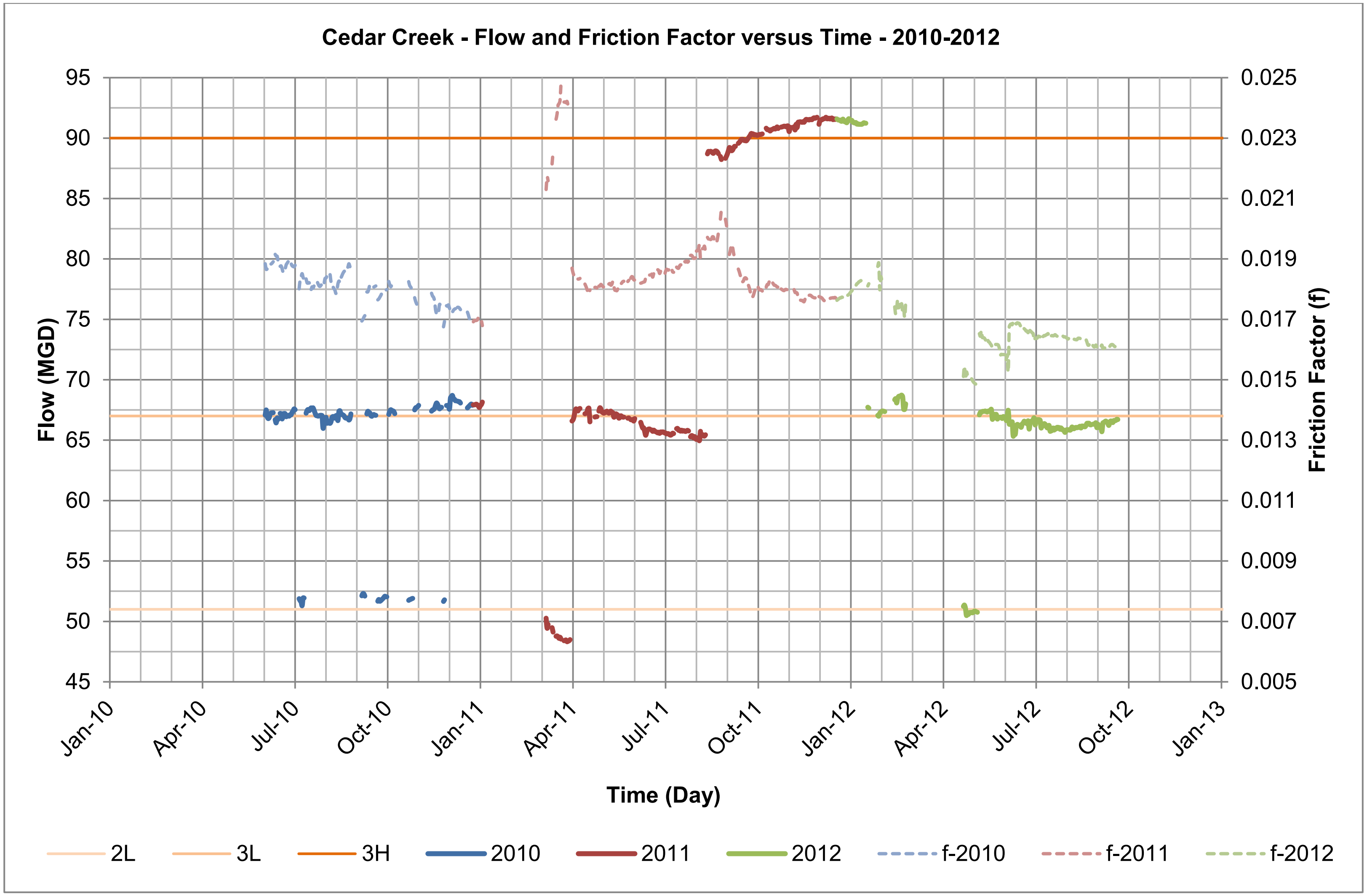


Figure 3.24: Flow Rates and Calculated Friction for the Cedar Creek Lake Pump Station

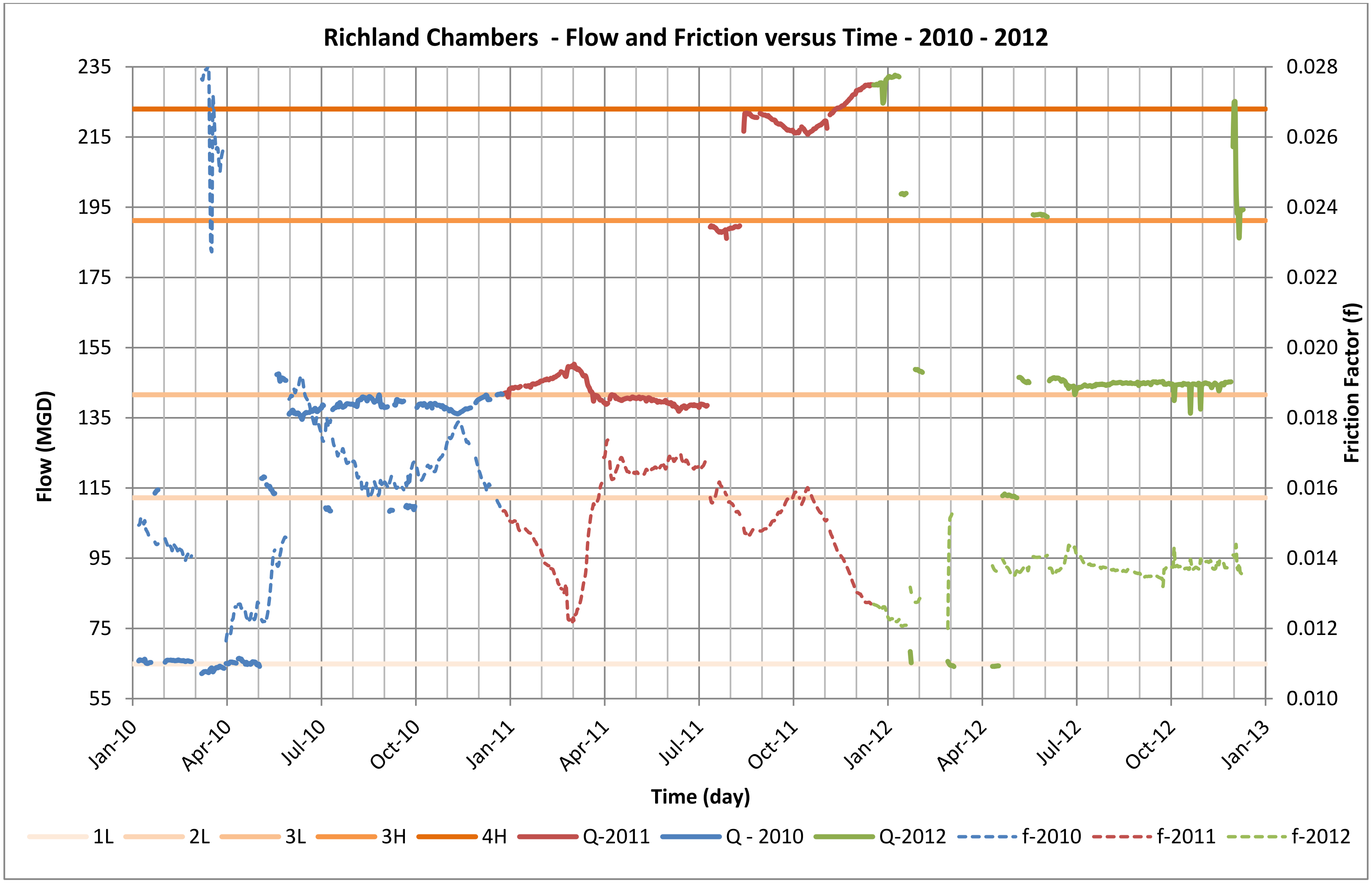


Figure 3.25: Flow Rates and Calculated Friction for the Richland Chambers Lake Pump Station

The Cedar Creek 3L pump configuration is different because 2012 saw the low rate of 65.3 MGD and a maximum of 68.7 MGD. The difference between the two values yields a 5.2 percent flow rate difference. The flow rates for the Cedar Creek 3H pump configuration in 2011 declined by 4.3% compared to 0.5% in 2012.

The same evaluation of Richland Chambers flow rates yields the information found in Table 3.8 and Table 3.9. The 1L pump configuration has the largest loss at 17.1 percent. This may be due to the short intermittent use of the 1L pump combination which does not allow the system to enter into a pseudo steady state. The 2L pump combination also shows 2010 to be the worst year for flow rate loss. 2011 is the worst year for the 3L configuration.

Table 3.8: Richland Chambers Flow Rates by Low Capacity Pump Configuration and Year

Richland Chambers	**1L**			**2L**			**3L**		
	2010	**2011**	**2012**	**2010**	**2011**	**2012**	**2010**	**2011**	**2012**
Min	56.8		64.1	108.3		108.5	134.6	133.2	139.0
Max	66.5		68.4	118.2		113.3	147.5	150.3	148.8
Average	64.9		64.7	112.1		112.5	139.0	142.1	145.0

High capacity did not occur in 2010, but it did in 2011 and 2012 as shown in Table 3.9.

. The 3H pump combination has the year 2012 being the year where most of the flow rate is lost but it is also the year where higher flow rates were seen in comparison to 2011. The flow rates in 2012 are higher than the flow rates found in 2011 but the 2012 losses are less than the 2011 rates.

Table 3.9: Richland Chambers Flow Rates by High Capacity Pump Configuration and Year

Richland Chambers	3H			4H		
	2010	**2011**	**2012**	**2010**	**2011**	**2012**
Min		186.1	192.3		215.8	224.7
Max		189.7	199.0		229.9	232.6
Average		188.8	194.5		221.4	230.7

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

The pumped flow rates see a yearly gain and reduction in values. Two attributes, roughness of the pipe material and roughness due to biofilm, are the main components of head loss due to friction line losses.

Main observations include:

- The conventional method of design considers the friction factor to age with the pipe. This study found the friction factor to change with biofilm growth oscillations and should be considered during design.
- The friction factor changes with the velocity of the water in a proportional relationship. The faster the water, the higher the friction factor.
- Superimposing the calculated friction factor and Reynolds number on a Moody diagram finds a specific relationship develops and can be tied to the time of year, not to a set hydraulic roughness. Generally, from January to August, the Reynolds numbers increase and the friction factors increase. When pumping from August to December, the reverse is true.
- The chloramine chemical cost helps save energy costs.
- Low friction factors in the wintertime occur briefly and biofilm growth occurs fairly rapidly soon after.
- Properly dosing chloramines at the lake can impact biofilm growth as increased pump capacity is observed.

- Rougher pipe sees a smaller swing in friction factor which is due to higher initial hydraulic roughness of the pipe.

4.2 Recommendations

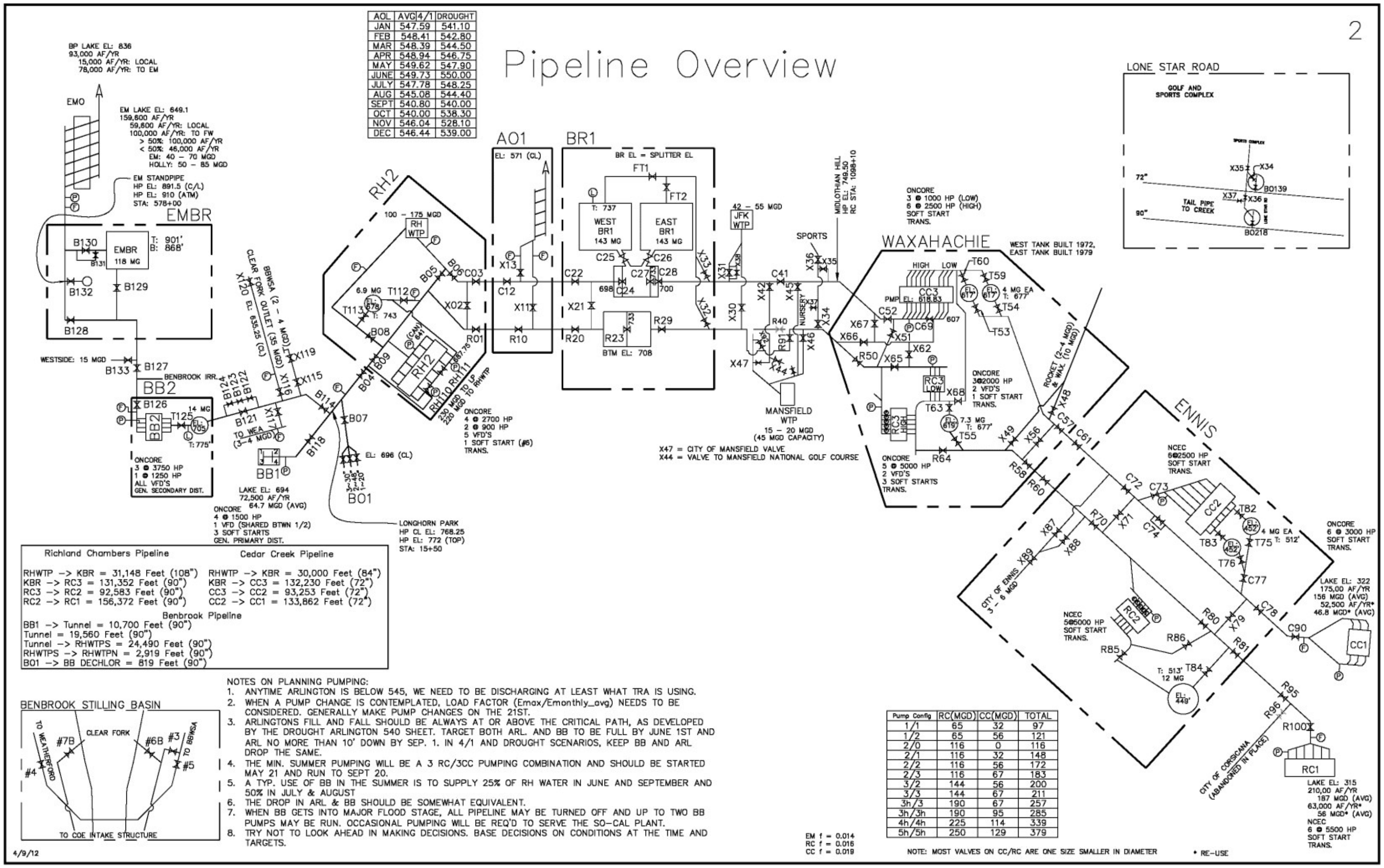
- Monitor friction factor on a daily basis to use as a measure of determining biofilm growth or decay. Consider starting chloramine treatment as soon as friction factor begins to increase.
- Consider year-round chloramine treatment to determine if chloramines can help regress established biofilm.
- Gather data sets along the Richland Chambers pipeline to be able to evaluate chlorine decay.
- Gather more data sets along the pipeline to determine if the spikes in the friction factor are an artifact of instrument error/data corruption or due to thicker biofilm.
- Further study to determine if the lake turnover has an influence on friction factors during the October/November timeframe on the Richland Chambers pipeline.
- A model could be developed for finding friction factor values due to biofouled pipes rather than using the Moody Diagram.
- Develop a chloramine application plan to properly dose the pipelines.

APPENDIX A

MAPS

Pipeline Overview

2



AOL	AVG 4/1	DROUGHT
JAN	547.59	541.10
FEB	548.41	542.80
MAR	548.39	544.50
APR	548.94	546.75
MAY	549.62	547.90
JUNE	549.73	550.00
JULY	547.78	548.25
AUG	545.08	544.40
SEPT	540.80	540.00
OCT	540.00	538.30
NOV	546.04	528.10
DEC	546.44	539.00

Richland Chambers Pipeline

RHWTP -> KBR = 31,148 Feet (108")
KBR -> RC3 = 131,352 Feet (90")
RC3 -> RC2 = 92,583 Feet (90")
RC2 -> RC1 = 156,372 Feet (90")

Cedar Creek Pipeline

RHWTP -> KBR = 30,000 Feet (84")
KBR -> CC3 = 132,230 Feet (72")
CC3 -> CC2 = 93,253 Feet (72")
CC2 -> CC1 = 133,862 Feet (72")

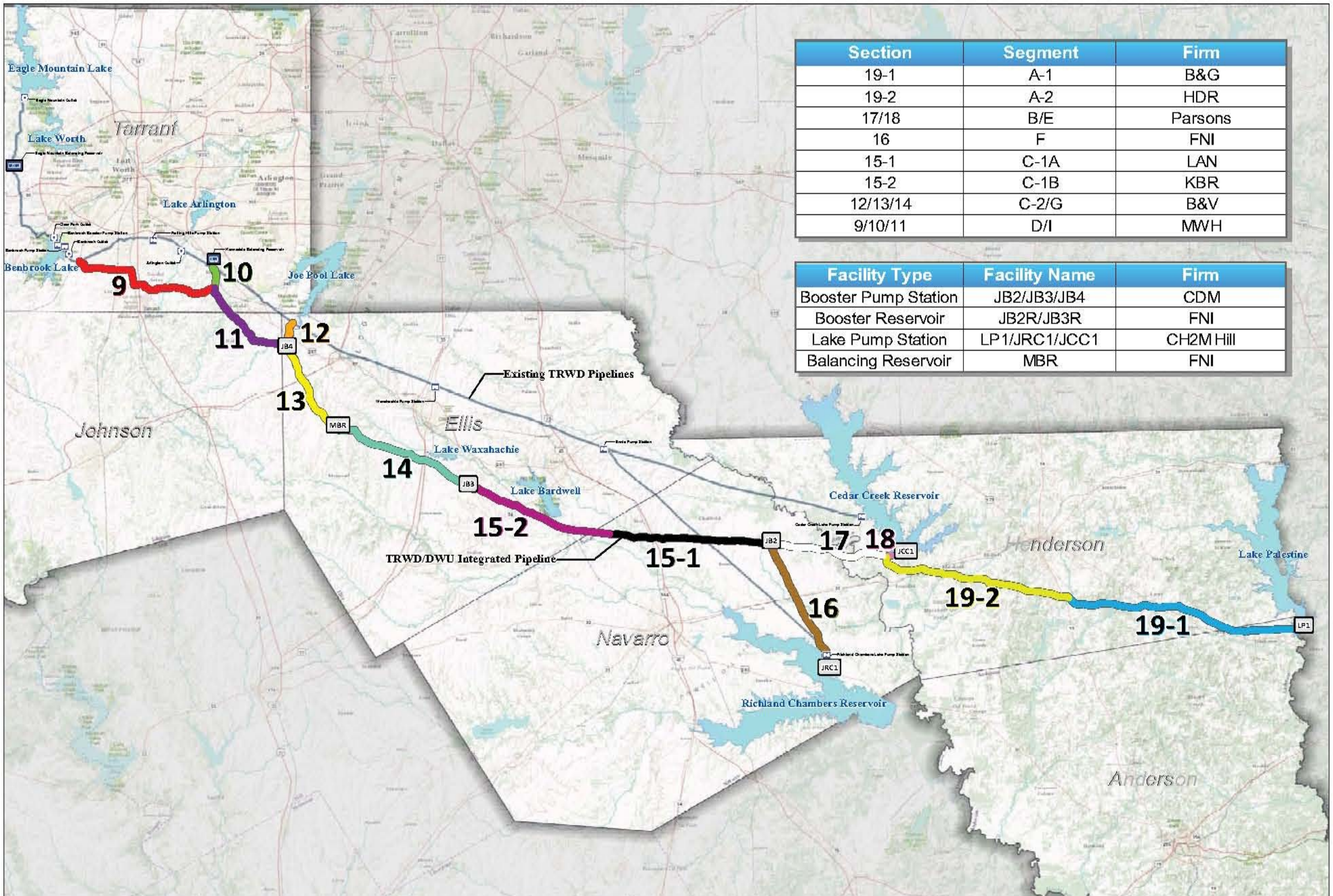
Benbrook Pipeline

BB1 -> Tunnel = 10,700 Feet (90")
Tunnel = 19,560 Feet (90")
Tunnel -> RHWTPS = 24,490 Feet (90")
RHWTPS -> RHWTPN = 2,919 Feet (90")
B01 -> BB DECHLOR = 819 Feet (90")

NOTES ON PLANNING PUMPING:

1. ANYTIME ARLINGTON IS BELOW 545, WE NEED TO BE DISCHARGING AT LEAST WHAT TRA IS USING.
2. WHEN A PUMP CHANGE IS CONTEMPLATED, LOAD FACTOR (Emax/Emonthly_avg) NEEDS TO BE CONSIDERED. GENERALLY MAKE PUMP CHANGES ON THE 21ST.
3. ARLINGTONS FILL AND FALL SHOULD BE ALWAYS AT OR ABOVE THE CRITICAL PATH, AS DEVELOPED BY THE DROUGHT ARLINGTON 540 SHEET. TARGET BOTH ARL. AND BB TO BE FULL BY JUNE 1ST AND ARL NO MORE THAN 10' DOWN BY SEP. 1. IN 4/1 AND DROUGHT SCENARIOS, KEEP BB AND ARL DROP THE SAME.
4. THE MIN. SUMMER PUMPING WILL BE A 3 RC/3CC PUMPING COMBINATION AND SHOULD BE STARTED MAY 21 AND RUN TO SEPT 20.
5. A TYP. USE OF BB IN THE SUMMER IS TO SUPPLY 25% OF RH WATER IN JUNE AND SEPTEMBER AND 50% IN JULY & AUGUST
6. THE DROP IN ARL & BB SHOULD BE SOMEWHAT EQUIVALENT.
7. WHEN BB GETS INTO MAJOR FLOOD STAGE, ALL PIPELINE MAY BE TURNED OFF AND UP TO TWO BB PUMPS MAY BE RUN. OCCASIONAL PUMPING WILL BE REQ'D TO SERVE THE SO-CAL PLANT.
8. TRY NOT TO LOOK AHEAD IN MAKING DECISIONS. BASE DECISIONS ON CONDITIONS AT THE TIME AND TARGETS.

Pump Config	RC(MGD)	CC(MGD)	TOTAL
1/1	65	32	97
1/2	65	56	121
2/0	116	0	116
2/1	116	32	148
2/2	116	56	172
2/3	116	67	183
3/2	144	56	200
3/3	144	67	211
3h/3	190	67	257
3h/3h	190	95	285
4h/4h	225	114	339
5h/5h	250	129	379



Section	Segment	Firm
19-1	A-1	B&G
19-2	A-2	HDR
17/18	B/E	Parsons
16	F	FNI
15-1	C-1A	LAN
15-2	C-1B	KBR
12/13/14	C-2/G	B&V
9/10/11	D/I	MWH

Facility Type	Facility Name	Firm
Booster Pump Station	JB2/JB3/JB4	CDM
Booster Reservoir	JB2R/JB3R	FNI
Lake Pump Station	LP1/JRC1/JCC1	CH2M Hill
Balancing Reservoir	MBR	FNI

Integrated Pipeline Program
Pipeline Sections and Facility Sites

Disclaimer:
Data contained in this map represents the best information available at the time of printing and is intended for general planning level use only. Unless otherwise stated: 1) it does not represent an on-the-ground survey, 2) only represents approximate relative locations, 3) may not be suitable for legal, engineering, or surveying purposes. TRWD/DWU/IPL is not liable for misuse of this information or derivative products resulting from this map.

MAP PRODUCED ON 7/17/2012 BY IPL- KZ

APPENDIX B

FIELD LOGS

Monday, October 03, 2011 - Test run – Station 3783+60

An Omega Engineering OM-CR-PRTEMP-1-300G pressure transducer was installed on the Cedar Creek pipeline at Station 3783+60 in order to test if communication can be established (Serial Number: N61680).

Prior to installation, District personnel modified an existing blind flange that attached to the end of a blow off valve. Figure 1 is a shop drawing of the existing blow off – however, it does not show the 12-inch butterfly valve attached at the end with a blind flange.

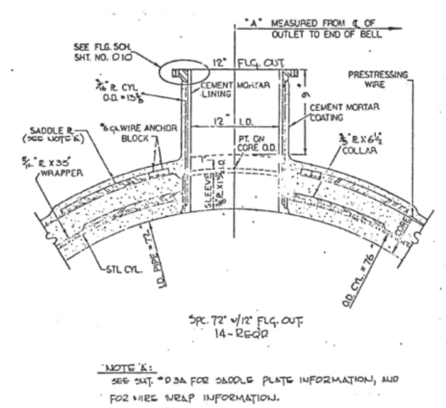


Figure 1: Cedar Creek Shop Drawing - 12-inch Blow Off

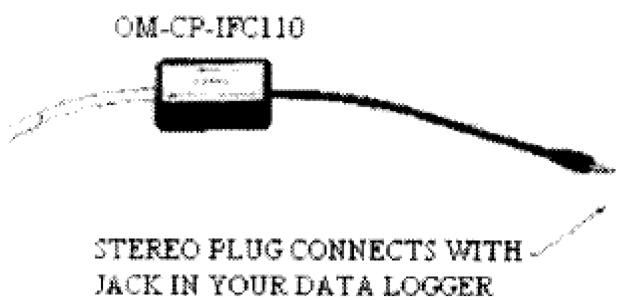
Figure 2 is a photo of the drilled blind flange and welded 1-inch half coupling prior to installing the pressure transducer.

Figure 2: Blind Flange with Half Coupling

A 1” x ¼” reducer was installed to allow the transducer, which has a ¼” NPT, to be attached directly to the top of the blind flange. Figure 3 is a photo of the transducer connected to the pipeline. Pipe dope is added to the threads of the transducer to ensure a water tight fit.

Figure 3: Transducer Connected to Pipeline - Sta. 3873+60

Prior to going to the field, the Omega Engineering OM-CP Data Logging Software (v. 2.05) was installed on a laptop. The USB Drivers were also installed so the software could communicate with the transducer through the data logger (OM-CP-IFC110)



To test the pressure transducer, the recorder was set in “Real Time Chart Recording” mode. The butterfly valve was opened. The pressure went to approximately 152 psig. To see if air could be fouling the pressure transducer, the valve was closed; the transducer was loosened to burp any possible air trapped in the transducer; then the transducer was tightened and the butterfly valve re-opened. The pressure returned to 152 psig, so it was believed any possible air trapped was not impacting the pressure measurements. The reading rate for this test was set to record every 5 seconds. The data recorded was downloaded and the following graph properly documents the events described herein.

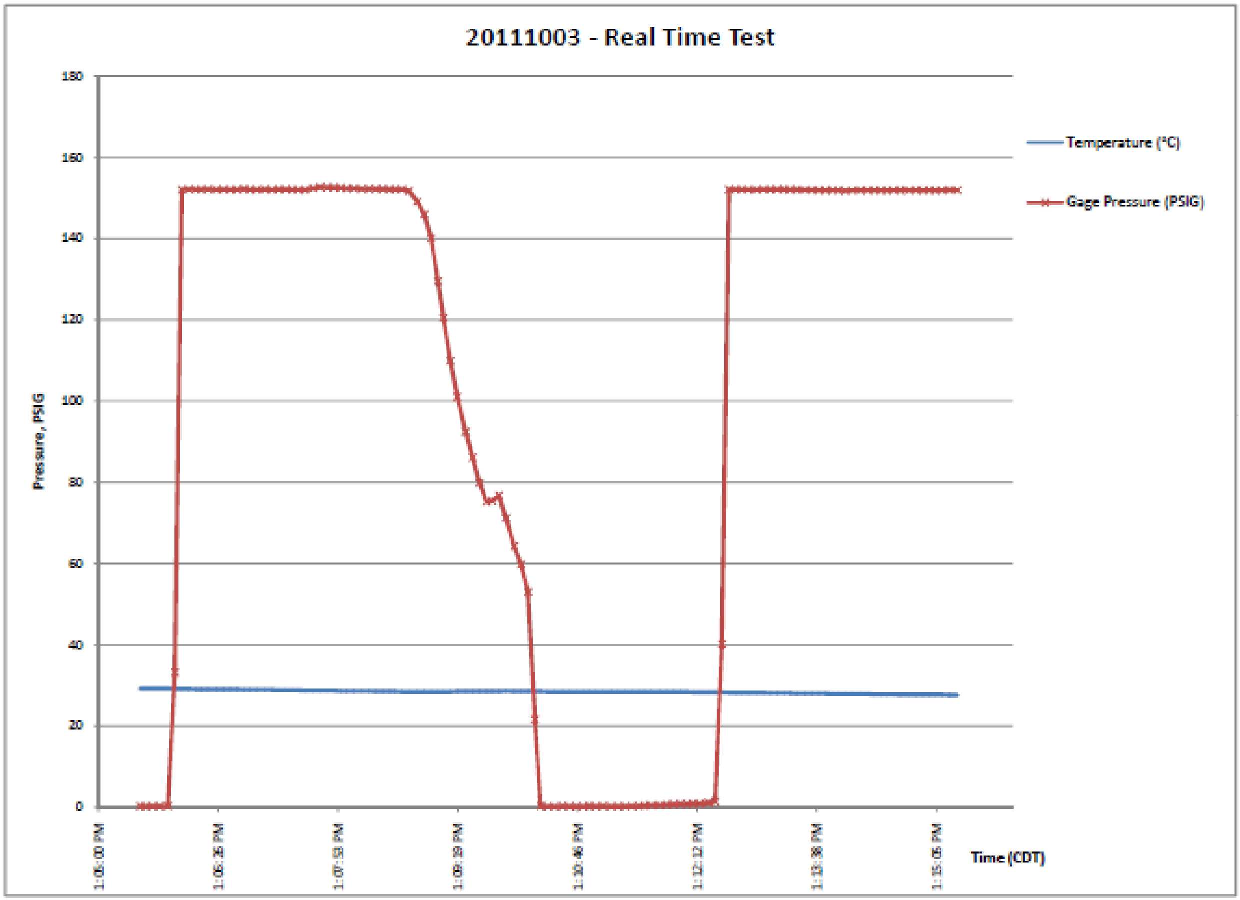


Figure 4: Real Time Test Chart

Upon the success of being able to start, stop, record, and download pressure data, the next test was to see if the transducers could be set to record overnight at a one hour interval.

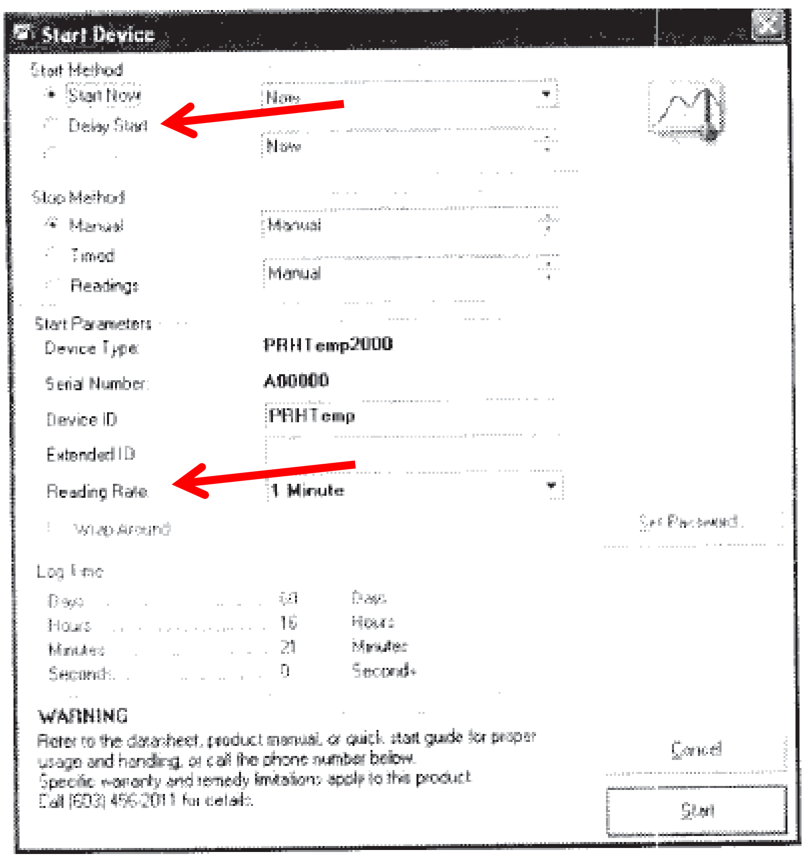
The recorders will be read the next day.

Lessons learned:

The laptop, if going to be used to set all 22 transducers, will need a car charger for long day set ups.

The time in the transducer records on UTC time, but the software uses the date/time of the laptop and adjusts upon download.

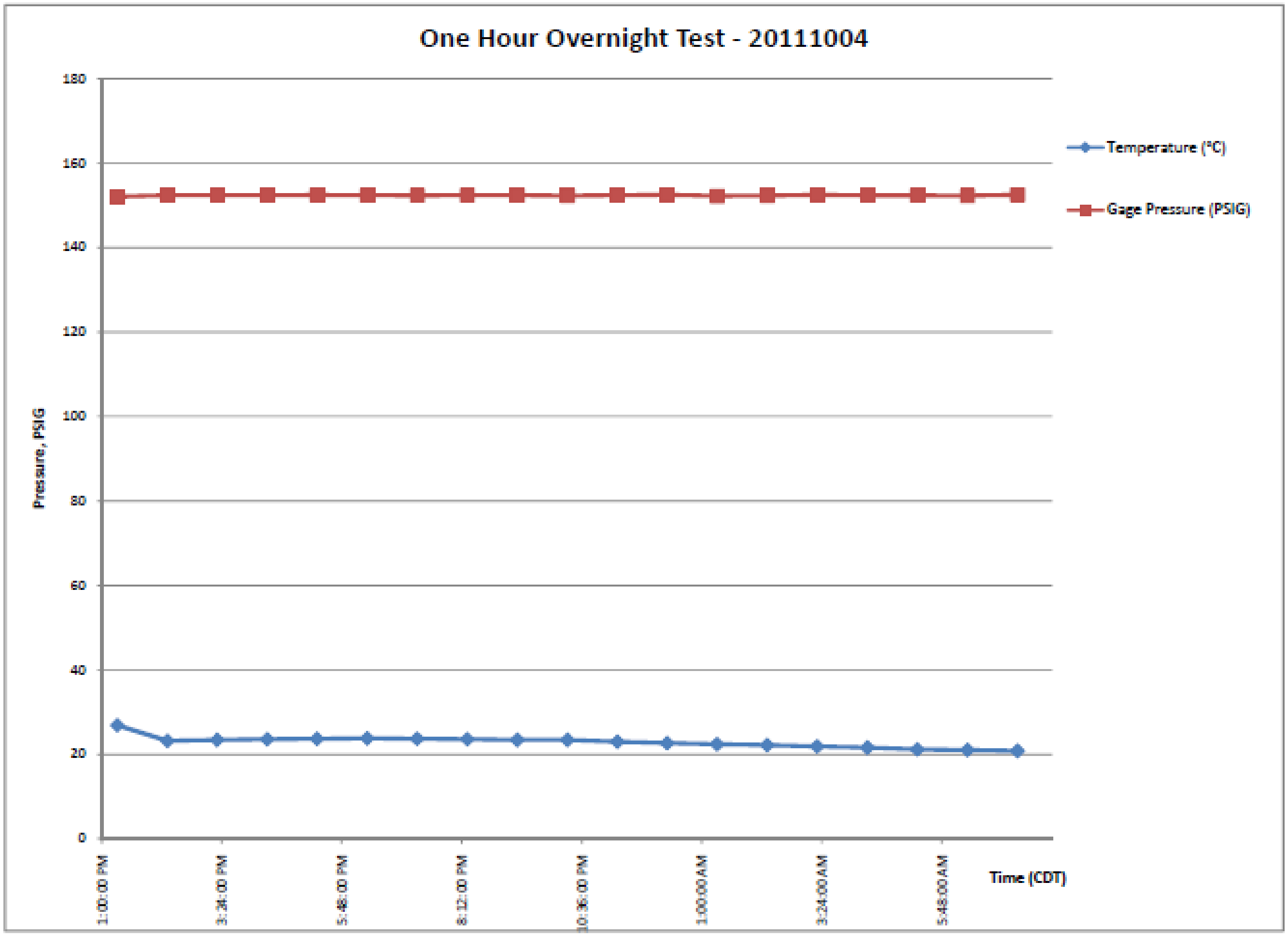
The record reading rate can be set on the devise Start menu:



Calibration is unclear – more information will need to be obtained from Omega Engineering as the calibration information in the Manual refers to Sterilization.

<u>October 4, 2011 – Overnight data recording</u>

The data was downloaded successfully.



Max	152.50
Min	152.08
Median	152.36
Average	152.34
Standard	0.10

Deviation	

The pressure transducer was reset to read every 15 minutes

Lesson learned:

The time also starts exactly when started. There is a delay function and will need to be implemented in order for all data to be collected at the same time.

Next step: Return next Monday (October 10) to restart the transducer to match the time interval for all pressure transducers to be deployed for a pre-cleaning reading. At the same time, the top of the manholes will need to be surveyed in to verify pipe centerline elevation.

October 10, 2011

The procedure established was used to install 14 pressure transducers from the lake pump stations to Ennis. The first transducer that was installed for the test run was downloaded and reset.

The top of the modified blind flange was surveyed in for each blow off location. Data worksheets were tallied. The following chart summarizes the information gathered.

10/10/2011									
	Station	PT SSN	Outlet Size	Top of Flange	Pressure	Start Time	Est. CL EL	Pipeline	
	ft		in	msl	psig	psig		msl	
1	3873+60	61680	12	278.823	152.36	152.34	8:30 AM	274.30	CC
2	3527+28	61664	12	288.092	130.16	130.2	9:15 AM	283.57	CC
3	3334+92	61673	12	297.826	116.25	116.08	10:00 AM	293.31	CC
4	3197+05	61676	12	332.265	94.14	94.04	10:30 AM	327.74	CC
5	3095+16	61679	12	321.679	93.63	93.58	10:45 AM	317.16	CC
6	2871+95	61665	12	317.912	84.1	83.98	11:15 AM	313.39	CC
7	2615+81	61677	12	417.812	27.28	28.2	11:45 AM	413.29	CC
8	2675+83	61661	12	384.865	49.75	49.72	1:15 PM	379.59	RC
9	2901+25	61670	8	400.292	60.52	60.28	2:00 PM	395.19	RC
10	3125+25	61667	8	471.18	47.84	47.76	2:30 PM	466.08	RC

11	3355+15	61682	12	424.256	85.6	86.28	3:00 PM	418.99	RC
12	3564+62	61666	12	394.491	115.98	115.94	3:30 AM	389.22	RC
13	3726+16	61675	12	346.147	149.82	149.6	4:00 PM	340.88	RC
14	3968+13	61681	12	349.47	169.24	169.24	4:15 PM	344.20	RC
10/11/2011									
15	2399+49	61668	8	460.574	138.84	138.84	9:45 AM	455.47	RC
16	2401+19	61662	12	459.239	119.16	119.04	10:00 AM	454.72	CC
17	2209+41	61663	12	454.119	111.68	111.68	10:30 AM	449.60	CC
18	2219+37	61674	12	456.701	127.32	127.32	10:45 AM	451.43	RC
19	2087+04	61678	12	463.095	114.04	113.84	11:15 AM	457.82	RC
20	2087+11	61671	12	465.17	100.4	100.24	11:15 AM	460.65	CC
21	1884+45	61672	12	509.046	71.6	71.56	12:00 PM	504.53	CC
22	1884+40	61669	12	507.299	79.34	79.22	12:00 PM	502.03	RC

Attached are copies of each field observation.

October 11, 2011

Field crews were delayed, so the opportunity to measure each data point was taken. The following observations were made.

RC2 Pump Discharge Pressure Elevation:

RC2 FF elevation + Discharge Pressure Gage CL elevation = 454.0 + 3.167 =

457.021 msl

As-Built FF elevation: 454.00 msl

Associated SCADA Tags:

RC2H_PC.PSIDIS_PMP1_RC2H.F_CV	RC2H DISCHARGE PRESSURE PUMP 1 (F_CV)
RC2H_PC.PSIDIS_PMP2_RC2H.F_CV	RC2H DISCHARGE PRESSURE PUMP 2 (F_CV)
RC2H_PC.PSIDIS_PMP3_RC2H.F_CV	RC2H DISCHARGE PRESSURE PUMP 3 (F_CV)
RC2H_PC.PSIDIS_PMP4_RC2H.F_CV	RC2H DISCHARGE PRESSURE PUMP 4 (F_CV)
RC2H_PC.PSIDIS_PMP5_RC2H.F_CV	RC2H DISCHARGE PRESSURE PUMP 5 (F_CV)

RC2 Station Pressure:

Concrete Vault Elevation +C/L gage to lip of concrete vault = 453.496 + 4.0 ft =

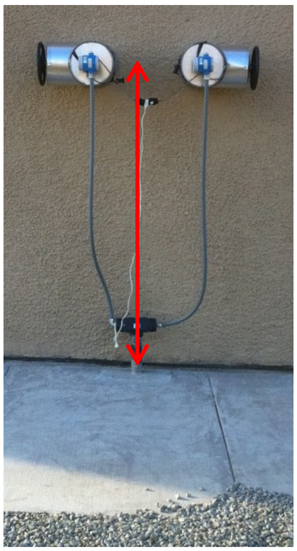
457.496 msl

Associated SCADA Tag (need to verify):

RC2H_PC.PSIDIS_HDR_RC2H.F_CV	RC2H DISCHARGE HEADER PRESSURE (F_CV)

RC2 Tank Level:

The tank is measured at the following location:



Sidewalk Elevation + Sidewalk to C/L level indicator = 448.53 + 5.9 ft = 454.43 ft msl (5 ft mark for SCADA)

Therefore an adjustment must be made to calculate the actual water surface elevation:

Tank Depth – 5 ft + Level Transmitter Elevation (454.43) = Tank WSEL

There are two level instruments which alternate based on SCADA operator's choosing. The level will only start measuring when the water level reaches the level instrumentation level. If below, they will read 5'.

5' = Low Range

64.1 = High Range

SCADA is currently set to cut the pumps off if the level gets to 21.34 ft. The high level is set for 57.06’. The low is set at 22.1’.

Associated SCADA Tag:

RC2H_PC.LVL_TANK_RC2H.F_CV	RC2H TANK LEVEL (F_CV)

RC2 Flow

Each pump for the RC2 site has a Venturi meter located on each pump.

Concrete Pad Elevation + C/L gage = 453.670+ 4.4 ft = 458.07

Associated SCADA Tags:

RC2H_PC.FLOWDIS_PMP1_RC2H.F_CV	RC2H FLOW RATE DISCHARGE PUMP 1 (F_CV)
RC2H_PC.FLOWDIS_PMP2_RC2H.F_CV	RC2H FLOW RATE DISCHARGE PUMP 2 (F_CV)
RC2H_PC.FLOWDIS_PMP3_RC2H.F_CV	RC2H FLOW RATE DISCHARGE PUMP 3 (F_CV)
RC2H_PC.FLOWDIS_PMP4_RC2H.F_CV	RC2H FLOW RATE DISCHARGE PUMP 4 (F_CV)
RC2H_PC.FLOWDIS_PMP5_RC2H.F_CV	RC2H FLOW RATE DISCHARGE PUMP 5 (F_CV)
RC2H_PC.FLOWDIS_RC2H.F_CV	RC2H SUMMED DISCHARGE FLOW (F_CV)

Note: The total station discharge is based on adding all active pumps together.

CC2 Tanks:

CC2 FF Elevation + C/L gage Elevation = 454.7 ft + 1.6 ft = 456.3 msl

As-built FF Elevation = 453.00

2’ = Low

60’ = High

SCADA will cut the pumps off if the level gets to 14.7 ft. The low alarm is set at 16.5 ft. The high alarm is set at 55 ft.

Therefore an adjustment must be made to calculate the actual water surface elevation:

Tank Depth – 2.00 ft = Depth above transmitter

Depth above Transmitter + Level Transmitter Elevation (456.3) = Tank WSEL

Associated SCADA Tag:

CC2H_PC.LVL_RES_CC2H.F_CV	CC2H RESERVOIR TANK LEVEL (F_CV)

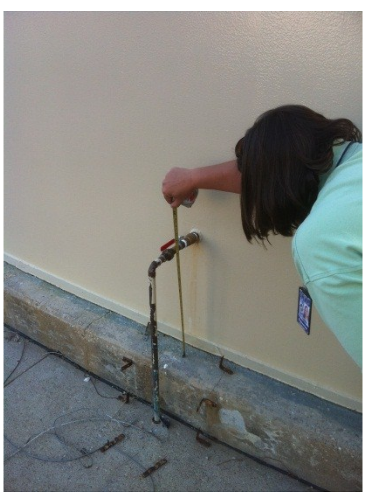
Note: The tank elevations have been verified as follows:

West Tank:

Concrete Curb Elevation + Curb to C/L Level Tap = 453.014 + 1.52 = 454.534

msl

East Tank:



Concrete Curb Elevation + Curb to C/L Level Tap = 453.068 + 1.52 = 454.588 msl

Note: Tank As-Built Elevation: 452.00 msl, Surveyed Elevation: 453.01/453.07. (Difference = 0.99/0.93 ft)

CC2 Station Discharge Pressure:

CC2 FF Elevation + C/L gage Elevation = 455.202 ft + 1.791 ft = 456.993 msl

As-built FF Elevation = 453.00

Associated SCADA Tag:

CC2H_PC.PSIDIS_CC1_CC2H.F_CV	CC2H DISCHARGE PRESSURE CC1 LINE (F_CV)

RC3H Station Discharge Pressure:

Concrete lip of vault elevation + C/L gage distance = 617.033+ 4.1 ft = 621.133 ft msl

Associated SCADA Tags:

RC3H_PC.PSIDIS_HDR_RC3H.F_CV	RC3H DISCHARGE HEADER PRESSURE (F_CV)

RC3H Flow

Concrete pedestal elevation + Distance to C/L gage = 617.820+ 4.45 ft =

622.27 ft msl

Associated SCADA Tags:

RC3H_PC.FLOWDIS_PMP1_RC3H.F_CV	RC3H FLOW RATE DISCHARGE PUMP 1 (F_CV)
RC3H_PC.FLOWDIS_PMP2_RC3H.F_CV	RC3H FLOW RATE DISCHARGE PUMP 2 (F_CV)
RC3H_PC.FLOWDIS_PMP3_RC3H.F_CV	RC3H FLOW RATE DISCHARGE PUMP 3 (F_CV)
RC3H_PC.FLOWDIS_PMP4_RC3H.F_CV	RC3H FLOW RATE DISCHARGE PUMP 4 (F_CV)
RC3H_PC.FLOWDIS_PMP5_RC3H.F_CV	RC3H FLOW RATE DISCHARGE PUMP 5 (F_CV)
RC3H_PC.FLOWDIS_RC3H.F_CV	RC3H SUMMED DISCHARGE FLOW (F_CV)

Note: The total station discharge is based on adding all active pumps together.

RC3H Pump Discharge Pressure:

RC3H Finished Floor Elevation + Distance to C/L gage = 617.648+ 3.75 ft =

621.398 ft msl

As-built RC3H Finished Floor Elevation: 618.00 msl

Associated SCADA Tags:

RC3H_PC.PSIDIS_PMP4_RC3H.F_CV	RC3H DISCHARGE PRESSURE PUMP 4 (F_CV)
RC3H_PC.PSIDIS_PMP5_RC3H.F_CV	RC3H DISCHARGE PRESSURE PUMP 5 (F_CV)
RC3H_PC.PSIDIS_PMP6_RC3H.F_CV	RC3H DISCHARGE PRESSURE PUMP 6 (F_CV)
RC3H_PC.PSIDIS_PMP7_RC3H.F_CV	RC3H DISCHARGE PRESSURE PUMP 7 (F_CV)
RC3H_PC.PSIDIS_PMP8_RC3H.F_CV	RC3H DISCHARGE PRESSURE PUMP 8 (F_CV)

RC3H Tank Elevation:

Sidewalk elevation + Distance to C/L gage = 618.139 + 3.83 ft = 621.969 ft msl

There are two level instruments which alternate based on SCADA operator's choosing. The level will only start measuring when the water level reaches the level instrumentation level. If below the instruments, they will read 3'.

3' = Low

58' = High

SCADA will cut the pumps off if the level gets to 16.5 ft. The low alarm is set at 17 ft. The high alarm is set at 55 ft.

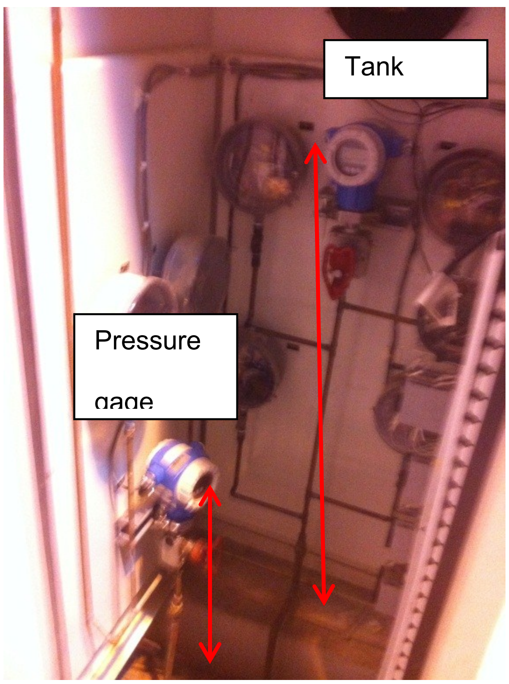
An adjustment must be made to calculate the actual water surface elevation:

Tank Depth – 3.00 ft + Level Transmitter Elevation (621.969) = Tank WSEL

Associated SCADA Tag:

RC3H_PC.LVL_TANK_RC3H.F_CV	RC3H RESERVOIR TANK LEVEL (F_CV)

RC3L Tank Elevation



Tank: RC3L Finished Floor Elevation + Distance to C/L gage = 618.066+ 3.20 ft

= 621.266 ft msl

2’ = Low

58’ = High

As-built RC3L Finished Floor Elevation = 618.00’

An adjustment must be made to calculate the actual water surface elevation:

Tank Depth – 2.00 ft + Level Transmitter Elevation (621.266) = Tank WSEL

SCADA Tag:

RC3L_PC.LVL_RES_RC3L.F_CV	RC3L TANK LEVEL RESERVOIR (F_CV)

Note: These two tank elevations measure the same tank and return different elevations that are off by 2 ft.

RC3L Discharge Pressure:

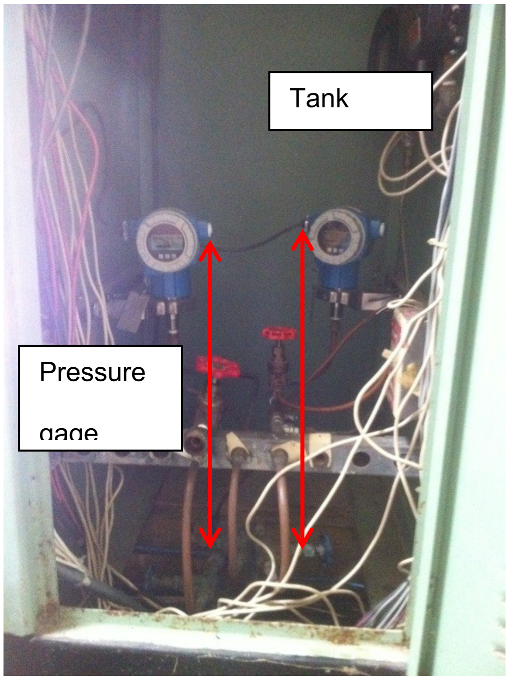
RC3L Finished Floor Elevation + Distance to C/L gage = 618.066+ X.XX ft = xxx.xx ft msl

As-built RC3L Finished Floor Elevation = 618.00’

SCADA Tag:

RC3L_PC.PSIDIS_RC3L.F_CV	RC3L DISCHARGE PRESSURE (F_CV)

CC3 Tank:



Tank: CC3 Finished Floor Elevation + Distance to C/L gage = 616.449+ 1.5 ft = 617.949 ft msl

As-built CC3 FF Elevation: 616.00 msl

0’ = Low Elevation

60’ = High Elevation

SCADA will cut the pumps off if the level gets to 13.7 ft. The low alarm is set at 17 ft. The high alarm is set at 55 ft.

An adjustment must be made to calculate the actual water surface elevation:

Tank Depth + Level Transmitter Elevation (617.949) = Tank WSEL

SCADA Tag:

CC3_PC.LVL_RES_CC3.F_CV	CC3 RESERVOIR TANK LEVEL (F_CV)

The tank elevations have been verified as follows:

West Tank:

Concrete Curb Elevation + Curb to C/L Level Tap = 616.571+ 1.15 = 617.721 ft msl

East Tank:

Concrete Curb Elevation + Curb to C/L Level Tap = 616.101+ 1.25 = 617.351 ft msl

As-built Concrete Curb Elevations: 617.00 msl

CC3 Discharge Pressure:

CC3 Finished Floor Elevation + Distance to C/L gage = 616.449 + 1.5 ft = 617.949 ft msl

As-built CC3 FF Elevation: 616.00 msl

SCADA Tag:

CC3_PC.PSIDIS_CC3.F_CV	CC3 DISCHARGE PRESSURE (F_CV)

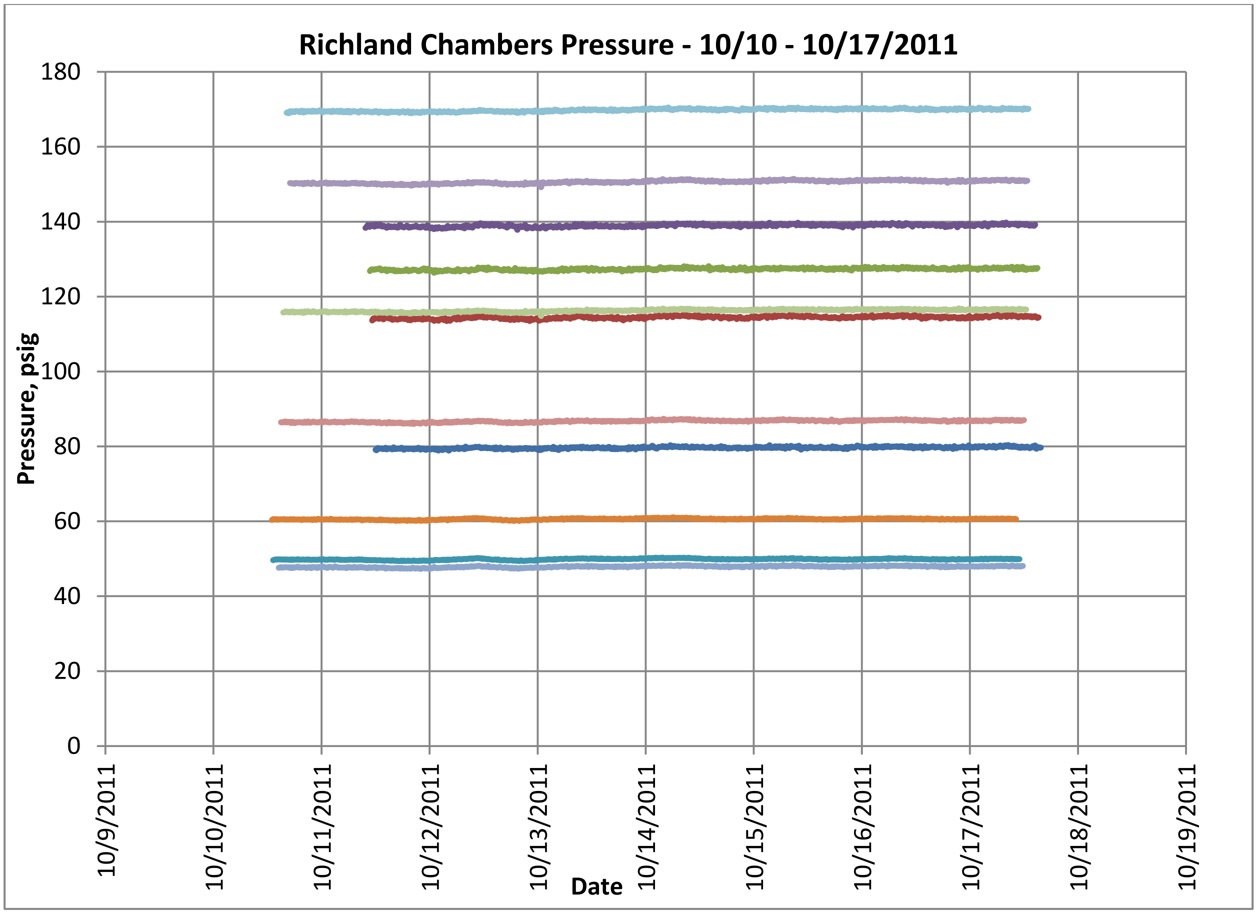
October 17, 2001

Each pressure transducer was downloaded and restarted. Approximately one week of data was downloaded successfully.

Re-starting the pressure transducers worked well until reaching the Walker Creek station. A message from the software showing there was an error starting the pressure transducer. The error was initially ignored and work continued. The error occurred at another station – prior to leaving, the pressure transducer's status was checked and determined it was stopped. It was restarted and the error was not seen. A new step has been added to the procedure prior to closing the vault to check that the pressure transducer is set to start.

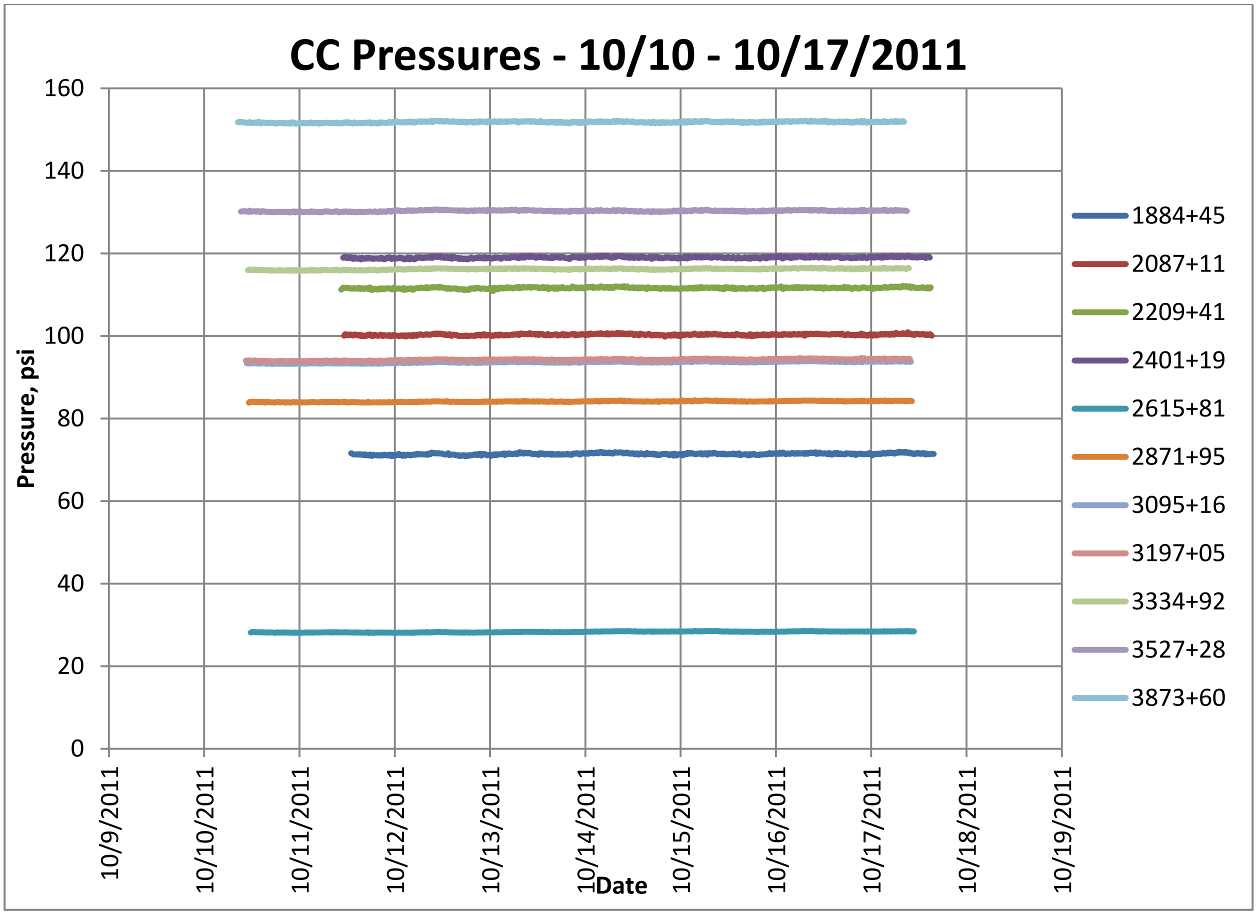
Prior to going out to read the pressure transducers, a visit to the Cedar Creek Lake Pump Station was completed to verify instrument locations.

A one page log was taken to keep track of times when the transducers were restarted.



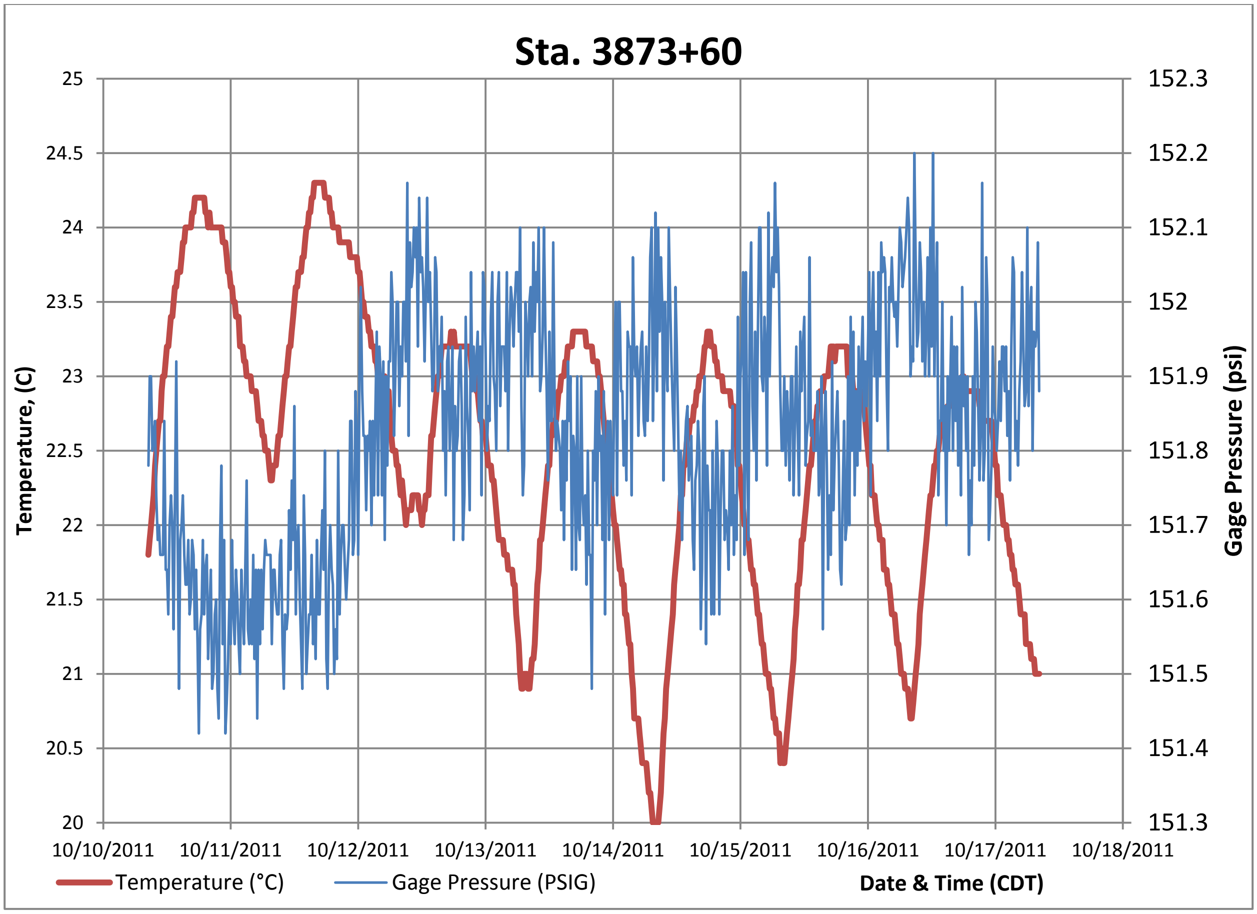
Pre-cleaned Pressure Data Collected on Richland Chambers pipeline between

10/10/2011 to 10/17/2011



Pre-cleaned Pressure Data Collected on Cedar Creek pipeline between 10/10/2011 to 10/17/2011

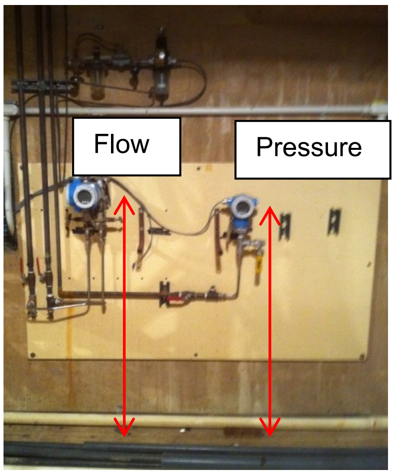
The raw data downloaded showed a particular pattern – when the ambient temperature of the pressure transducer goes up, the pressure goes down. Note that the pressure differences were on a small scale:



Omega Engineering was contacted to determine if the instrument compensated its pressure readings due to the temperature readings. The manufacturer stated that the temperature reading capability of the instrument is to assure the accuracy of the pressure tests are within operating range of the pressure transducer – in other words, the readings are independent of each other. The instrument is accurate when working between 0 and 85 degrees Celsius. The instrument is most accurate at 25 degrees but the maximum transducer error when exposed to other temps beside 25 degrees Celsius is +/- 1 % FS.

CC1 Discharge Pressure & Flow Meter

The discharge pressure is a total station discharge reading.



Cedar Creek Lake Pump Station Discharge Pressures and Flow Meter Instrumentation Location

CC1 Discharge Pressure:

Electrical Chase Elevation + Distance to Centerline Pressure Transducer =

333.68’ + 3.17’ = 336.85’

Centerline of Pipe Elevation at High Point of Venturi = 333.76'

SCADA Tag:

CC1_PC.PSIDIS_CC1.F_CV	CC1 DISCHARGE PRESSURE (F_CV)

CC1 Flow Meter:

Electrical Chase Elevation + Distance to Centerline Pressure Transducer =

333.68 + 3.33' = 337.01'

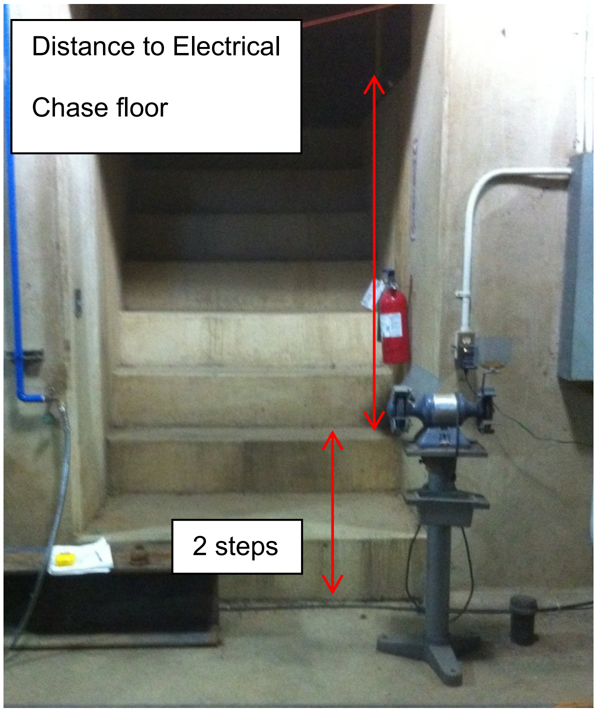
Electrical Chase Elevation = Finished Floor Elevation – Distance to top of wet well + 2 steps + dist to Electrical Chase floor

343.949– 18.17' + 1.96' + 5.94' = 333.68'

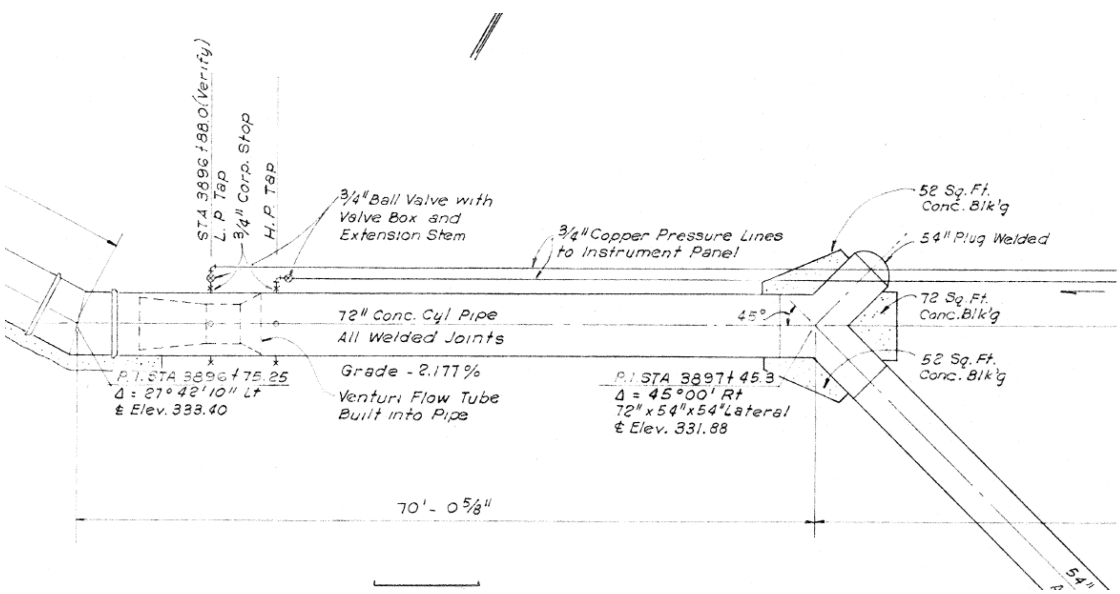
SCADA Tag:

CC1_PC.FLOWDIS_SQRT_CC1.F_CV	CC1 DISCHARGE FLOW SCALED SQUARE ROOT (F_CV)

Distance to top of wet well



2 steps + Distance to Electrical Chase floor



CC1 Lake Elevation:

The lake elevation is determined with a USGS level gage and it is calibrated regularly.

SCADA Tag:

CC1_PC.LVL_RES_CC1.F_CV	CC1 RESERVOIR LAKE LEVEL (F_CV)

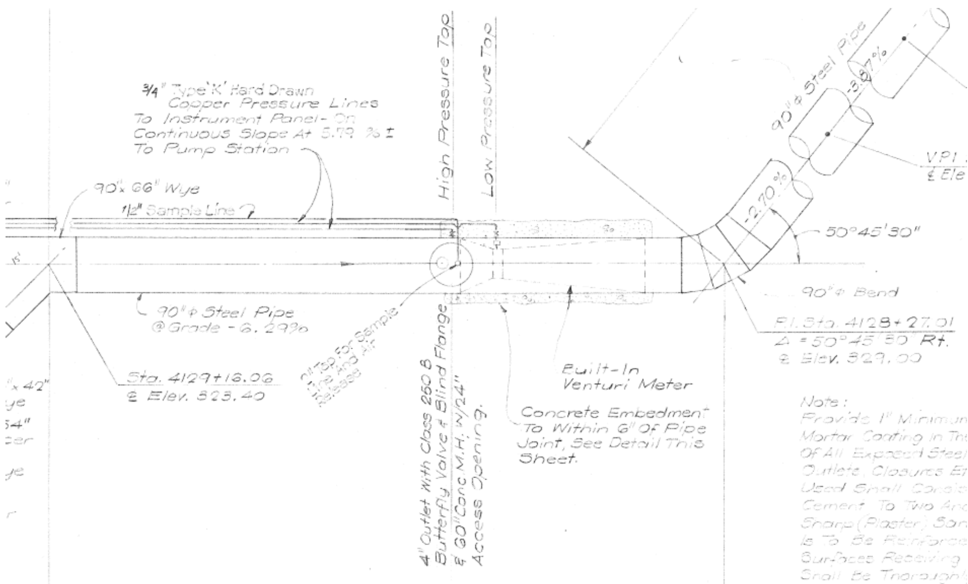
Cedar Creek Lake Elevation USGS Gage

October 20, 2001

A visit was required to verify the instrumentation at the Richland Chambers Lake Pump Station.

RC1 Station Discharge Pressure and Flow Meter

The Station Discharge Pressure is read from the same line as the high pressure side of the flow meter. The flow meter is located outside the main building and has a tap located at the middle of the venturi meter (considered the high pressure line). Note the meter is not level.



RC Venturi Meter Plans Showing Tap Locations

Yard Piping showing venturi tap locations

Measurements were taken to verify the location of the pressure taps on the venturi.

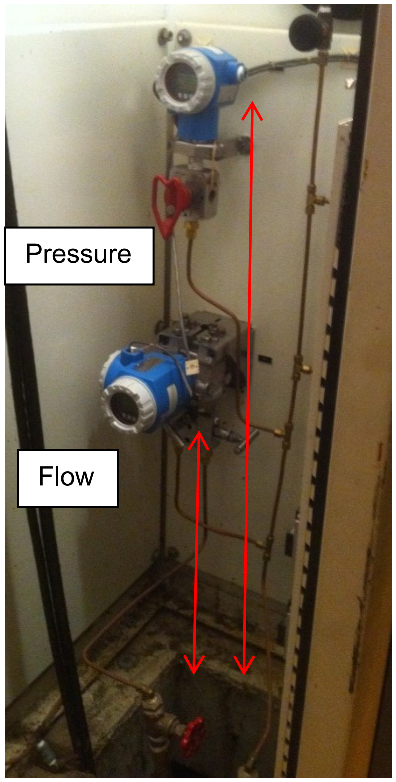
High: Ground Elevation – Tape down Measurement = 340.161 - 12.3 = 327.86' msl

Record Drawing El: 326.75 msl

Low: Ground Elevation – Tape down Measurement = 340.133 - 10.3 = 329.833' msl

Record Drawing El: 327.11 msl

The discrepancy in the values could be due to the venturi meter not being installed as shown in the record drawings.



Richland Chambers Flow and Pressure Discharge Meter

RC1 Pressure Discharge Elevation:

RC1 Finished Floor Elevation + Distance to C/L Gage = 339.459+ 3.375 ft = 342.834 msl

SCADA Tag:

RC1_PC.PSIDIS_RC1.F_CV	RC1 DISCHARGE PRESSURE (F_CV)

RC1 Flow Meter Elevation:

RC1 Finished Floor Elevation + Distance to C/L Gage = 339.459 + 2.00 ft = 341.459 msl

SCADA Tag:

RC1_PC.FLOWDIS_SQRT_RC1.F_CV	RC1 DISCHARGE FLOW SCALED SQUARE ROOT (F_CV)

RC1 Lake Elevation:

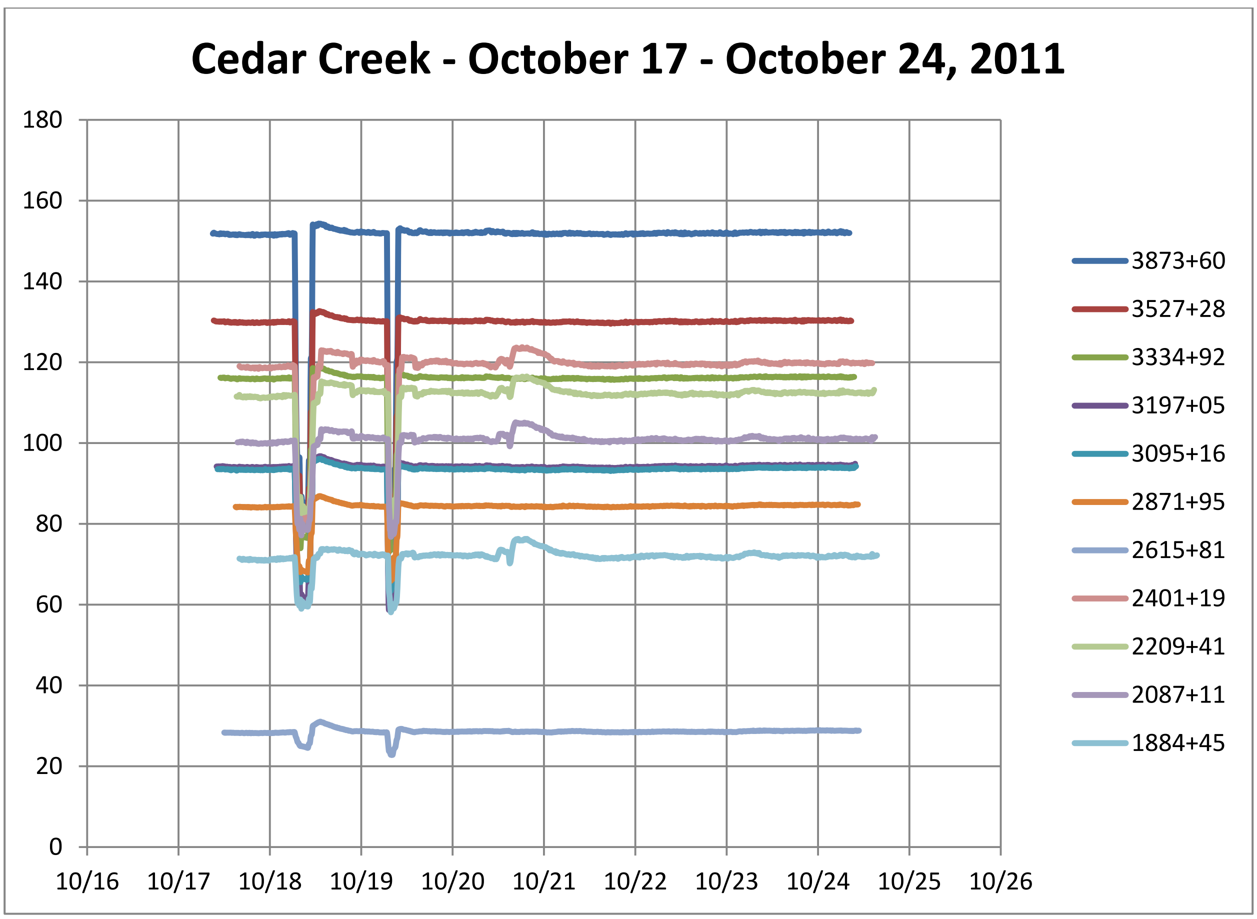
The lake elevation is determined with a USGS level gage and it is calibrated regularly by said organization. The District has a repeater that duplicates the readings.

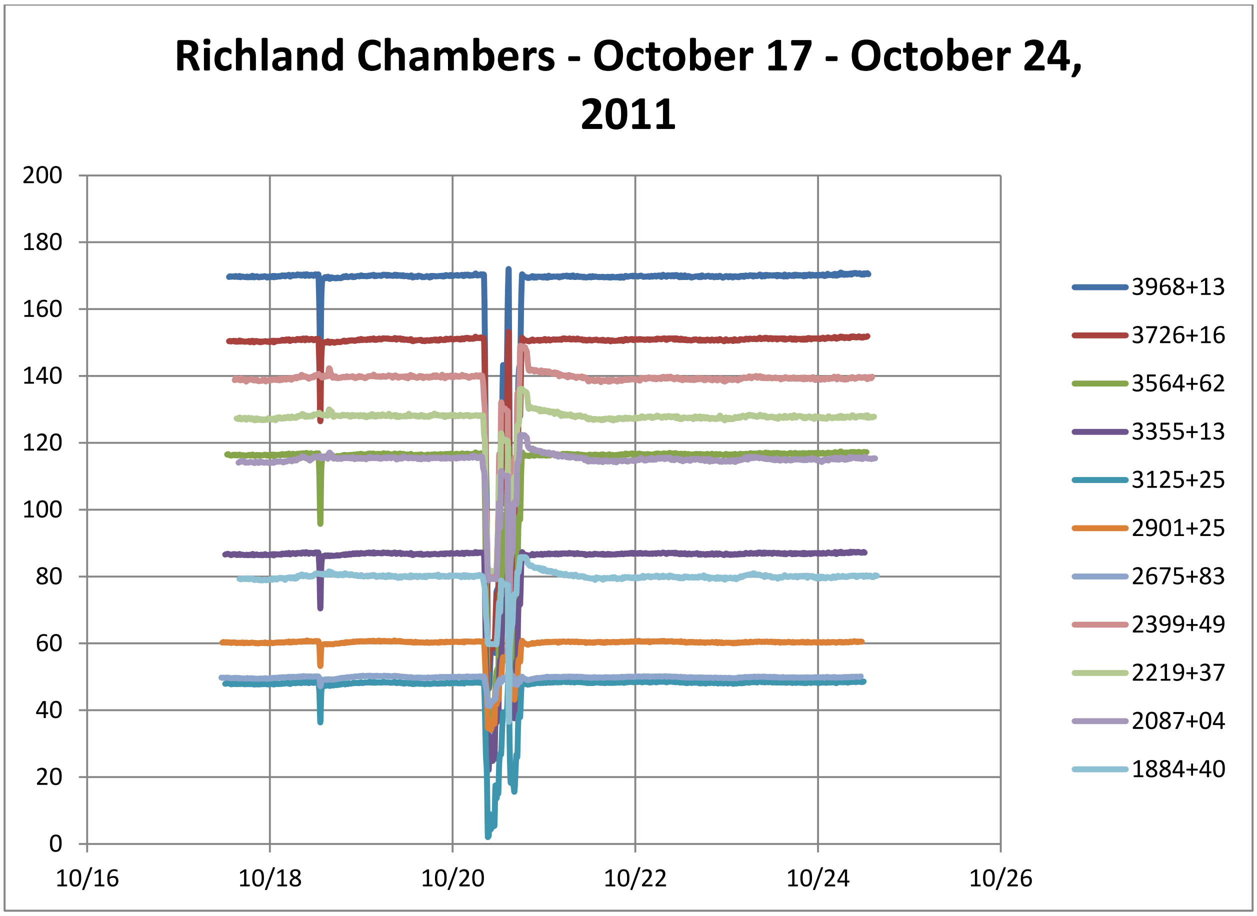
SCADA Tag:

RC1_PC.LVL_RES_RC1.F_CV	RC1 LAKE LEVEL RESERVOIR (F_CV)

October 24, 2011

All the data collectors were downloaded starting at 8 am to approximately 3:30 pm. Starting at 7:30 am, the valve manipulation plan began to be implemented. The plan is to isolate the Cedar Creek line to discharge completely to the Arlington Outlet due to dosing the line with free chlorine and the possible THM formation. No water treatment plants will receive Cedar Creek water while dosing with free chlorine. The plan on dosing free chlorine is to start feeding at a 2 mg/l rate and go up 1 mg/l each day. The dosing started at 12:42 pm. The two dips below coincide with the pumps going down at the Cedar Creek Lake Pump Station and the Richland Chambers line. Any of the "surcharge pressure" that occurred on the 20th happened in the line between Ennis and Waxahachie.





While collecting data, it was observed that not all blow offs were configured as originally thought. Station 3726+16 on the Richland Chambers line (Highway 31) had a 24-inch man-way with a reduction to a 12-inch blow-off and a 12-inch blow off valve. The distance from the top blind flange to the top of man-way flange is 32-inches.

RC 3726+16 Blow-off (Highway 31)

Revised Top of Pipe Elevation: 346.147 – 2.667 = 343.480 msl

Another location which differed is also on Richland Chambers at Station 2087+04 (Ebenezer Road). There were two 12-inch butterfly valves instead of one. The practice along the pipeline is to add a butterfly valve on top of a leaking isolation valve in the time where the valve cannot be removed due to the pipeline being live.

RC Sta. 2087+04 Blow off (Ebenezer Road)

Revised Top of Pipe Elevation: 463.095 - (9+(2*8)+1.25)/12 = 460.908 msl

Because of the above observations, the next time data collection is done, photos of each pressure transducer location will be taken to verify vault contents.

All of the vaults were fairly dry, with the exception of water found at Station 2675+83 on the Richland Chambers line (Highway 85). The pressure transducer did not get submerged at this time and data was successfully downloaded.

Station 2675+83 on the Richland Chambers line (Highway 85)

October 25, 2011

The dosing of the Cedar Creek line went up to 3 mg/l at 12:00 pm.

October 26, 2011

The dosing of the Cedar Creek line went up to 4 mg/l at 13:04 pm. However, due to chemical delivery issues (i.e., we will run out before more is delivered on Tuesday, November 1), the dosing had to be dialed back to 3 mg/l at 16:10 pm. No change in flow rate has been observed at this time.

The chloramine dosing of the Richland Chambers line (with a 4:1 Cl_2/NH_3 ratio) started at 13:14 pm. The chemical feed system was recently upgraded and there were some difficulties upon start up so the chemical feed was on and off all day. The feed stabilized at 2:08 am

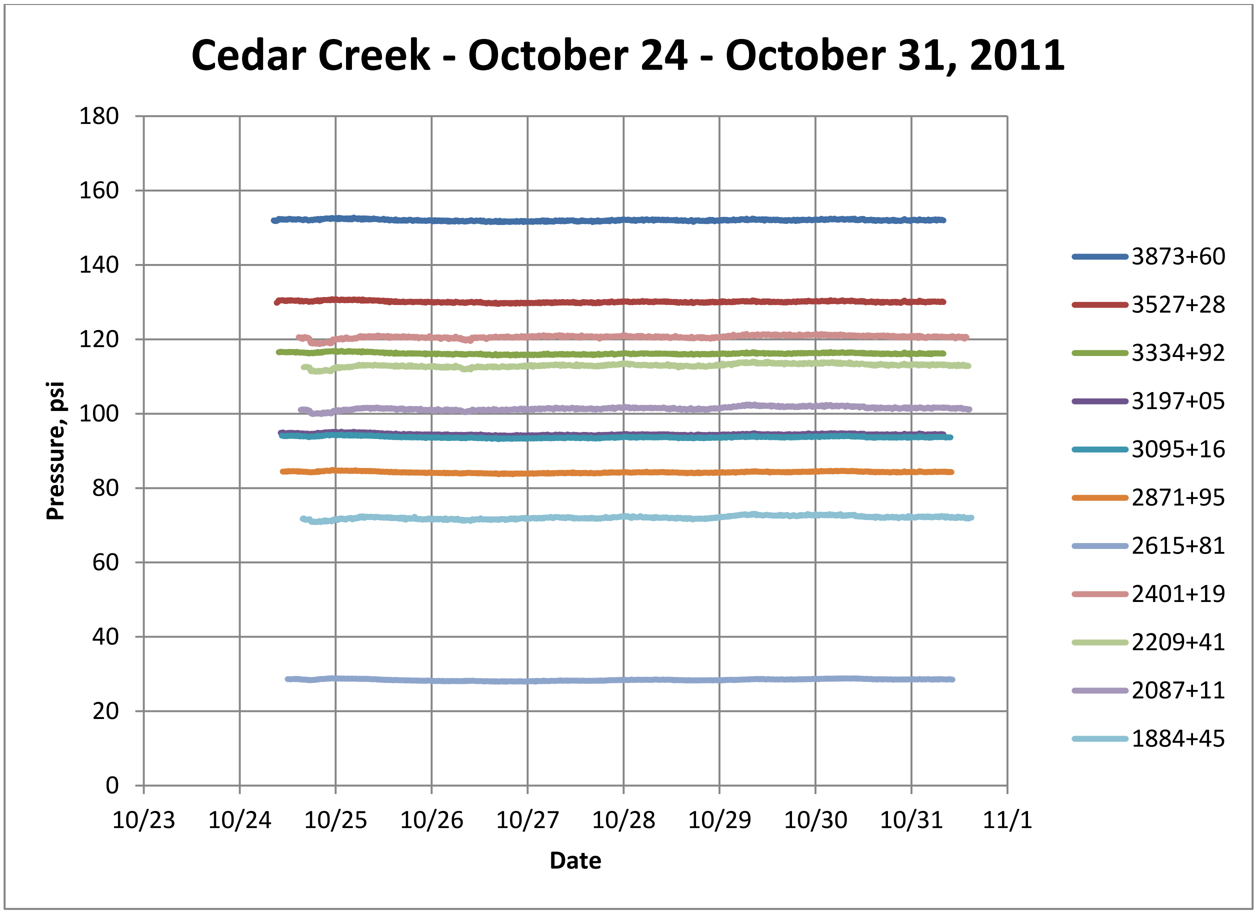
October 27. 2011

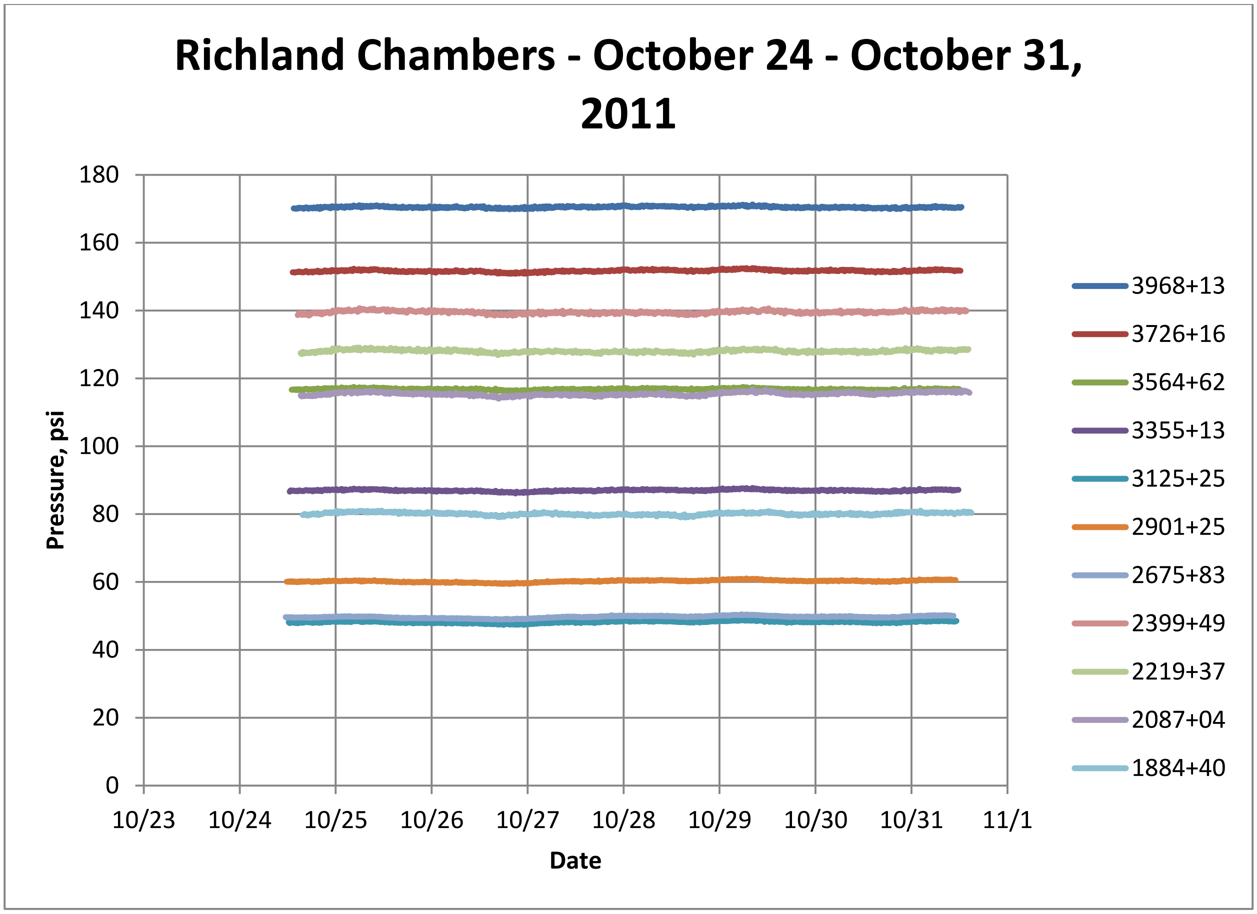
Operations requested the change the Richland Chambers chemical feed rate from 2 mg/l to 2.5 mg/l. This feed rate will remain at this level until more chlorine is delivered on Tuesday, November 1.

October 31, 2011

The pressure readings were gathered this day.

The pressures along the line stayed, for the most part, constant on a macro scale. Because of operational constraints, free chlorine dosing ended at 8:29 am and ammonia was turned on. The District's chlorine evaporators are currently not functioning and the temperatures dipped so low at this time that we could not increase the chloramines dosage rate.


Cedar Creek - October 24 - October 31, 2011
Pressure, psi
0
20
40
60
80
100
120
140
160
180
10/23
10/24
10/25
10/26
10/27
10/28
10/29
10/30
10/31
11/1
Date
3873+60
3527+28
3334+92
3197+05
3095+16
2871+95
2615+81
2401+19
2209+41
2087+11
1884+45



November 14, 2011

Data was gathered with no events. At 17:15, the chloramines were turned off for the season.

November 15, 2011

The Cedar Creek chloramines were stopped for the season at 12:05 pm.

November 28, 2011

Data was gathered with no events

Walker creek loss of data.

The pressure transducers were left in place over the winter gathering data.

March 12, 2012

The Cedar Creek line shuts down due to lower demands and Benbrook being in flood stage.

March 20, 2012

The Richland Chambers line shuts down due to lower demands and Benbrook being in flood stage.

April 4, 2012

Due to wet conditions, access to all the vaults was limited. This trip was just to Ebenezer Road to pull the transducers and assess the condition of the transducers. Data was successfully downloaded and the transducer batteries checked. The batteries still held a good charge and the transducer retained all data to date. The decision was made to keep the existing batteries in the transducers and assess at a later date.

April 24, 2012

Free Chlorine at 7 mg/L begins at RC1 at 12:35 pm. The line is isolated and no customers receive this water.

April 26, 2012

Cedar Creek line starts up with no chemicals (chemical feed system still being modified)

May 1, 2012

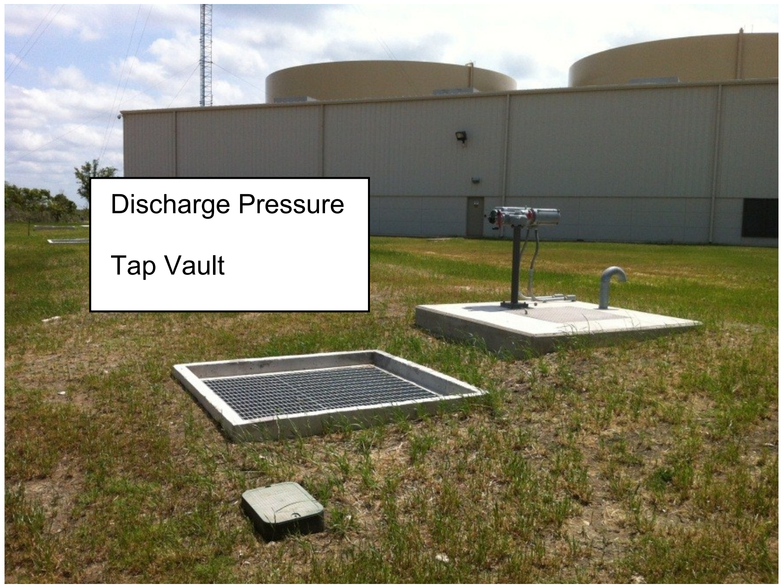
Chloramines begin at 7:34 pm at 3 mg/L. Note that the chlorine residual in the pipeline is now 2.3 - which is in line with lab findings of Dr. Kruzic's work. Data was also gathered on this date. A number of vaults had water in them.

The vaults were pumped down and the data was successfully taken at submerged stations - the transducers remained water tight.

One station did not yield field data - that was the Cedar Creek transducer located at Ebenezer Road. Thus, the only data lost is from April 4th. Cedar Creek was mostly down during that time - only four days of data was lost.

CC2 Discharge Pressure Tap

The tap is located at the top of the pipe at a man way located in an open vault.



May 8, 2012

Begin Chloramines at CC1. 3 ppm at 4:1 (Cl_2:NH_3) ratio.

May 27, 2012

Increase chloramines at both sites to 4 ppm.

June 19, 2012

RC1 goes down from 3H to 3L.

June 21, 2012

Shut down RC1 chloramines. Ammonia line is plugged with Calcium Carbonate.

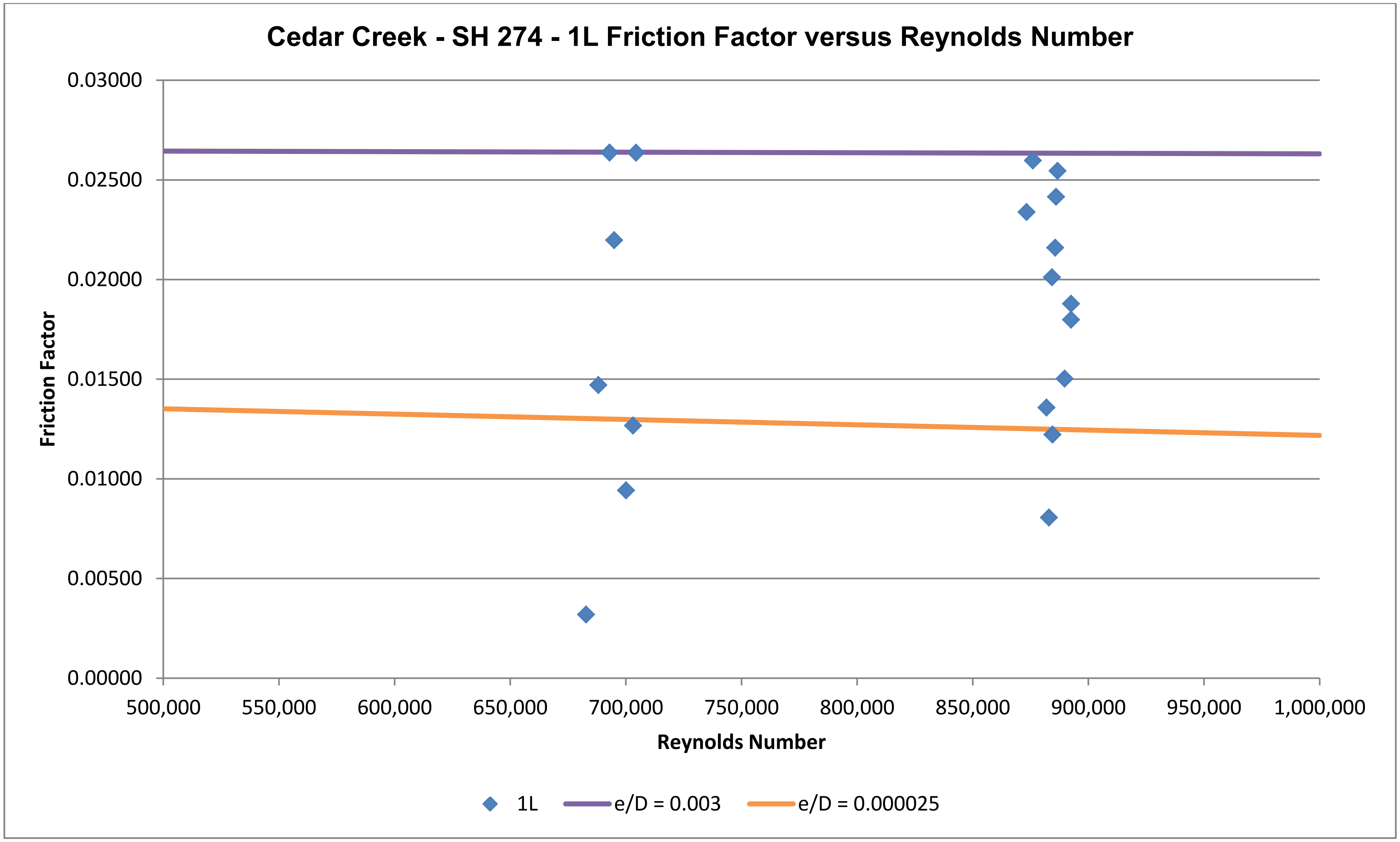
APPENDIX C

CEDAR CREEK FIELD STUDY

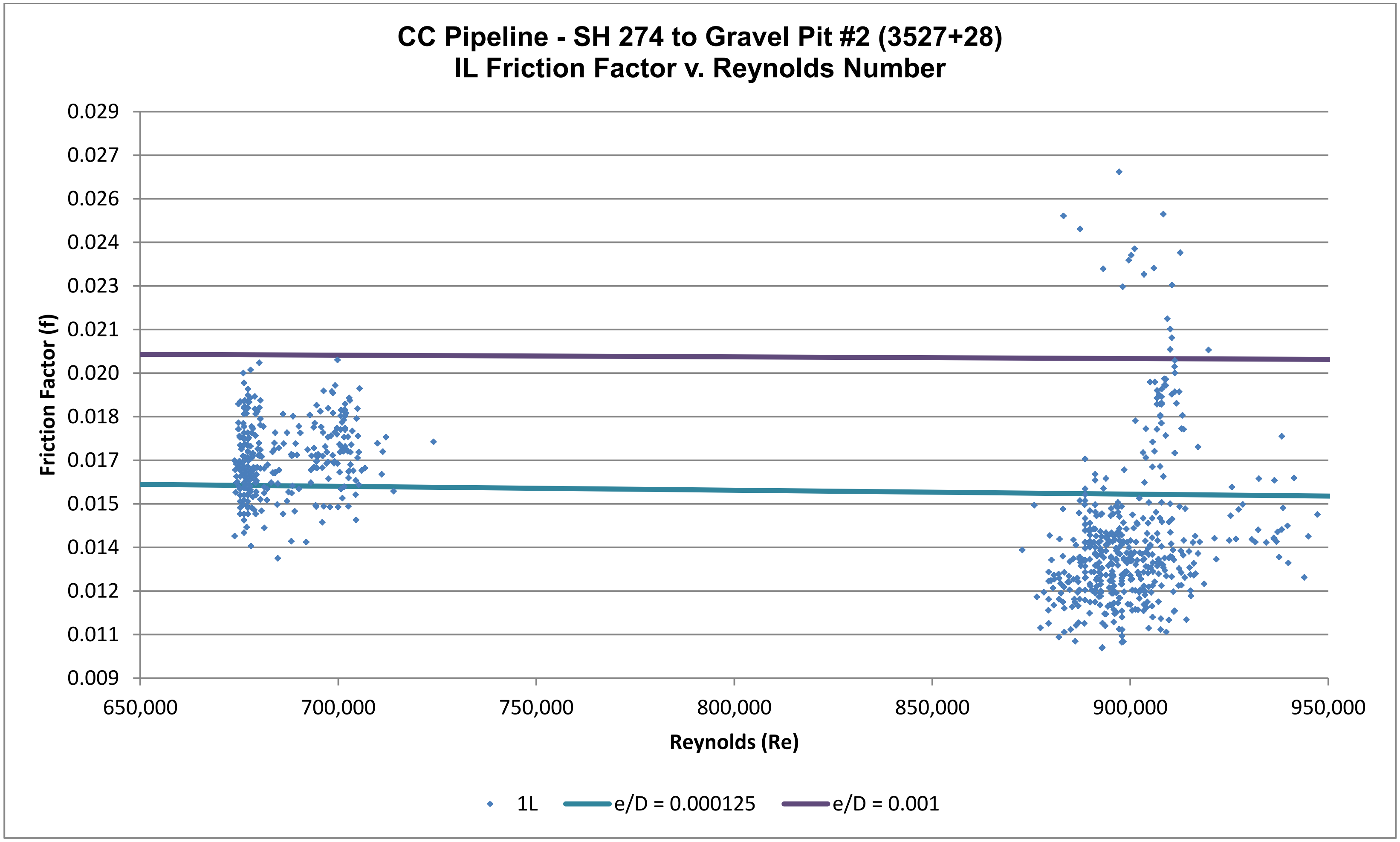
FRICTION FACTOR AND MOODY DIAGRAM RESULTS

Cedar Creek 1L Pump Combination

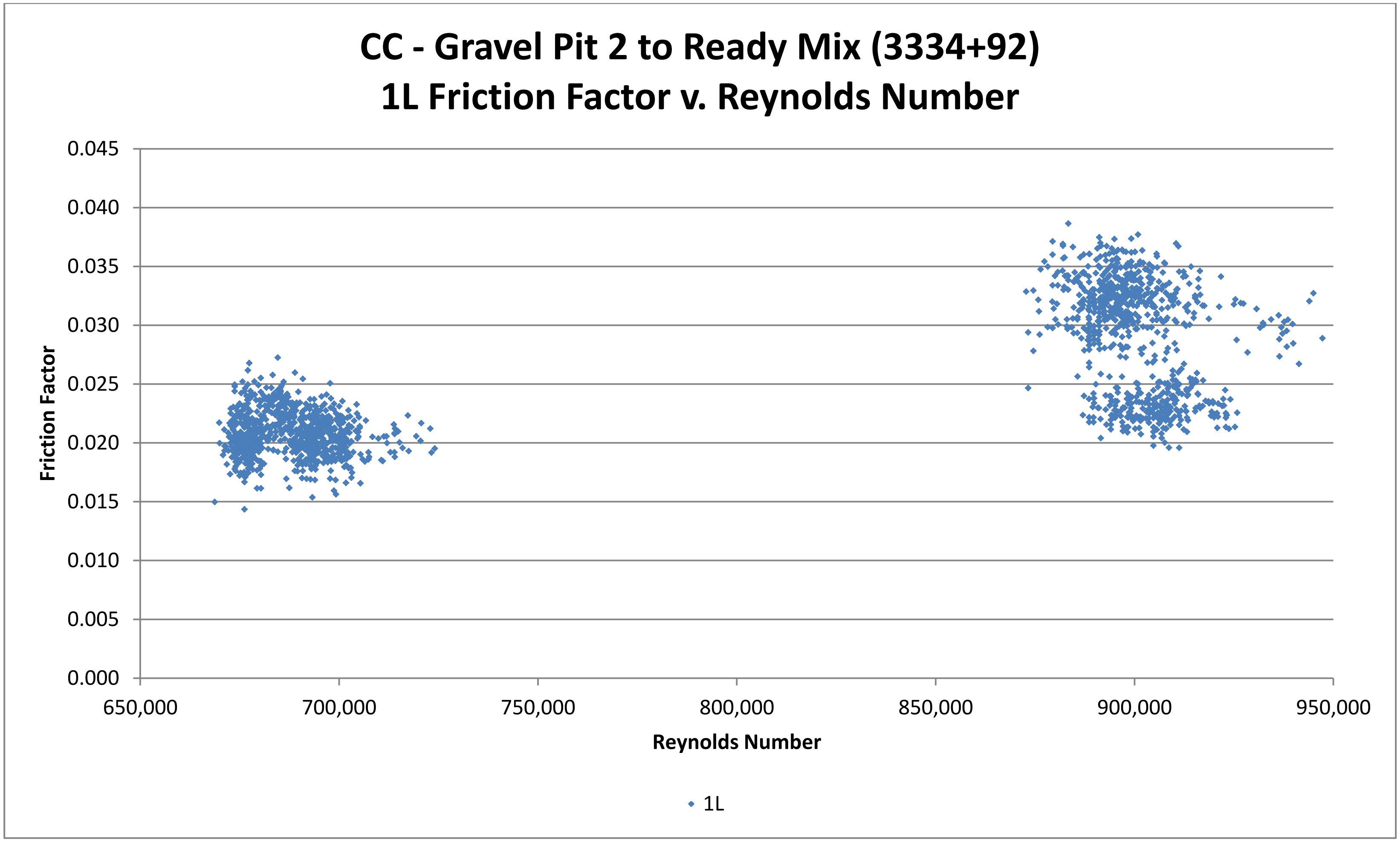
Cedar Creek Lake Pump Station to State Highway 271



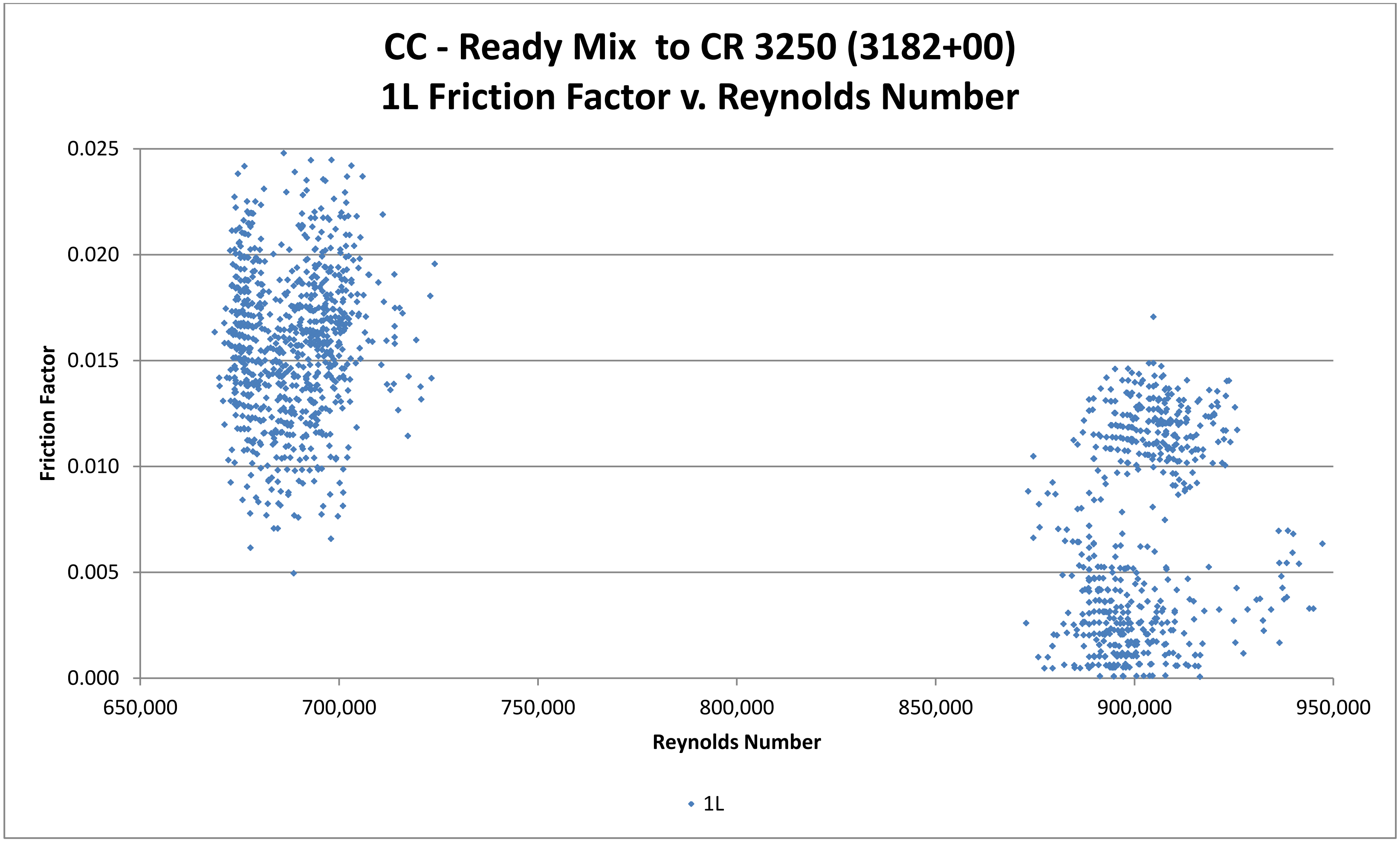
SH 274 to Gravel Pit #2



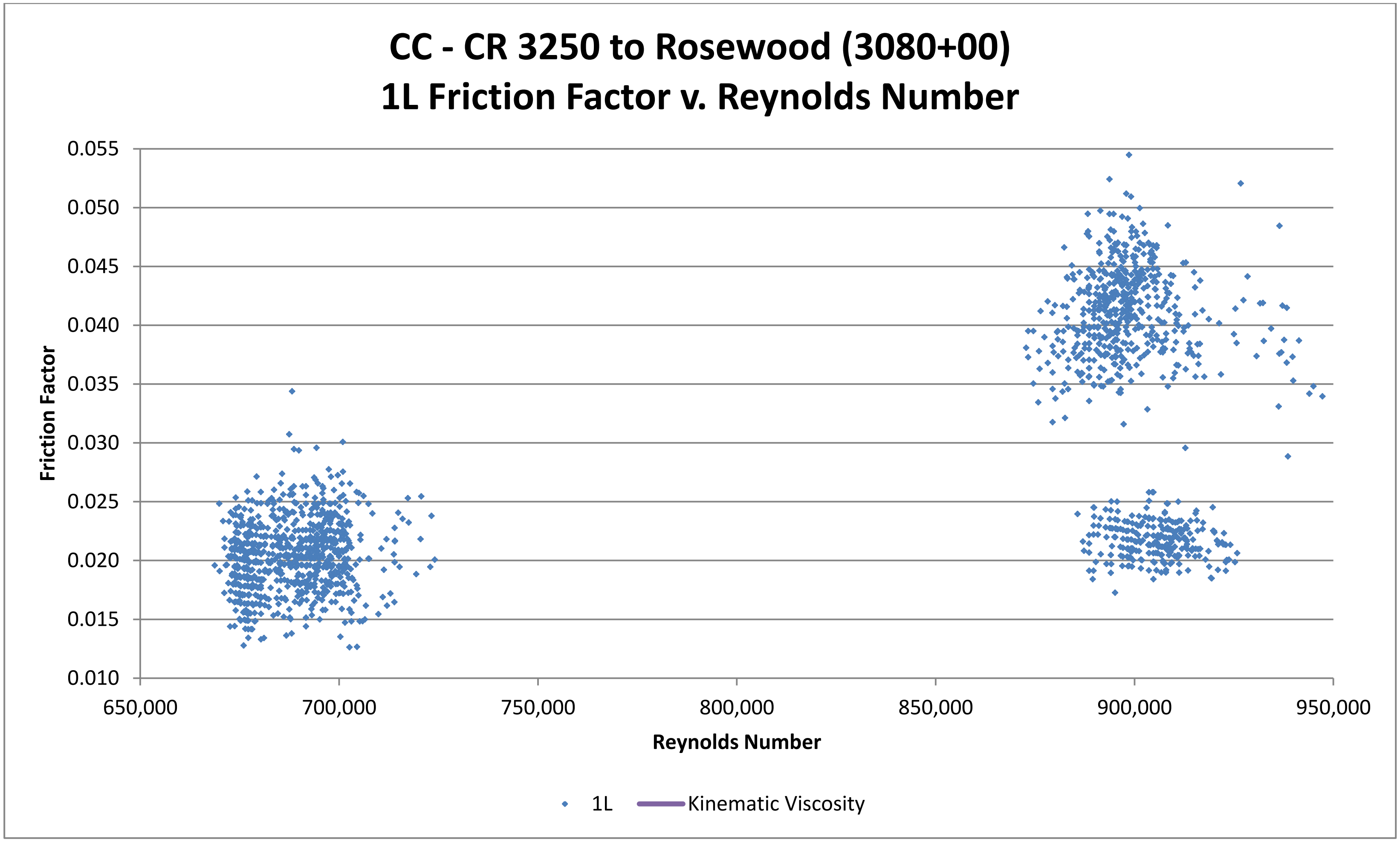
Gravel Pit #2 to Ready Mix



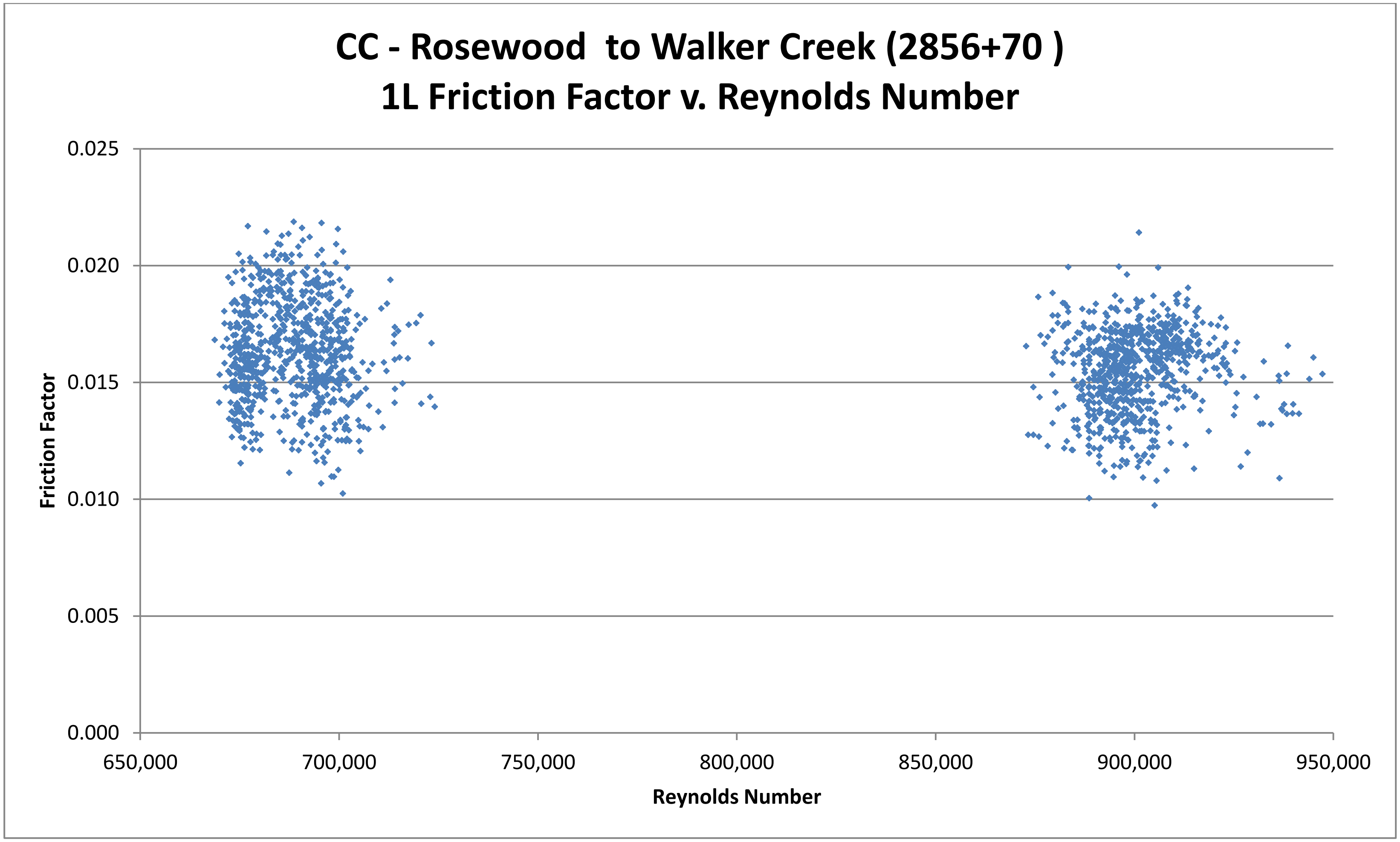
Ready Mix to CR 3250



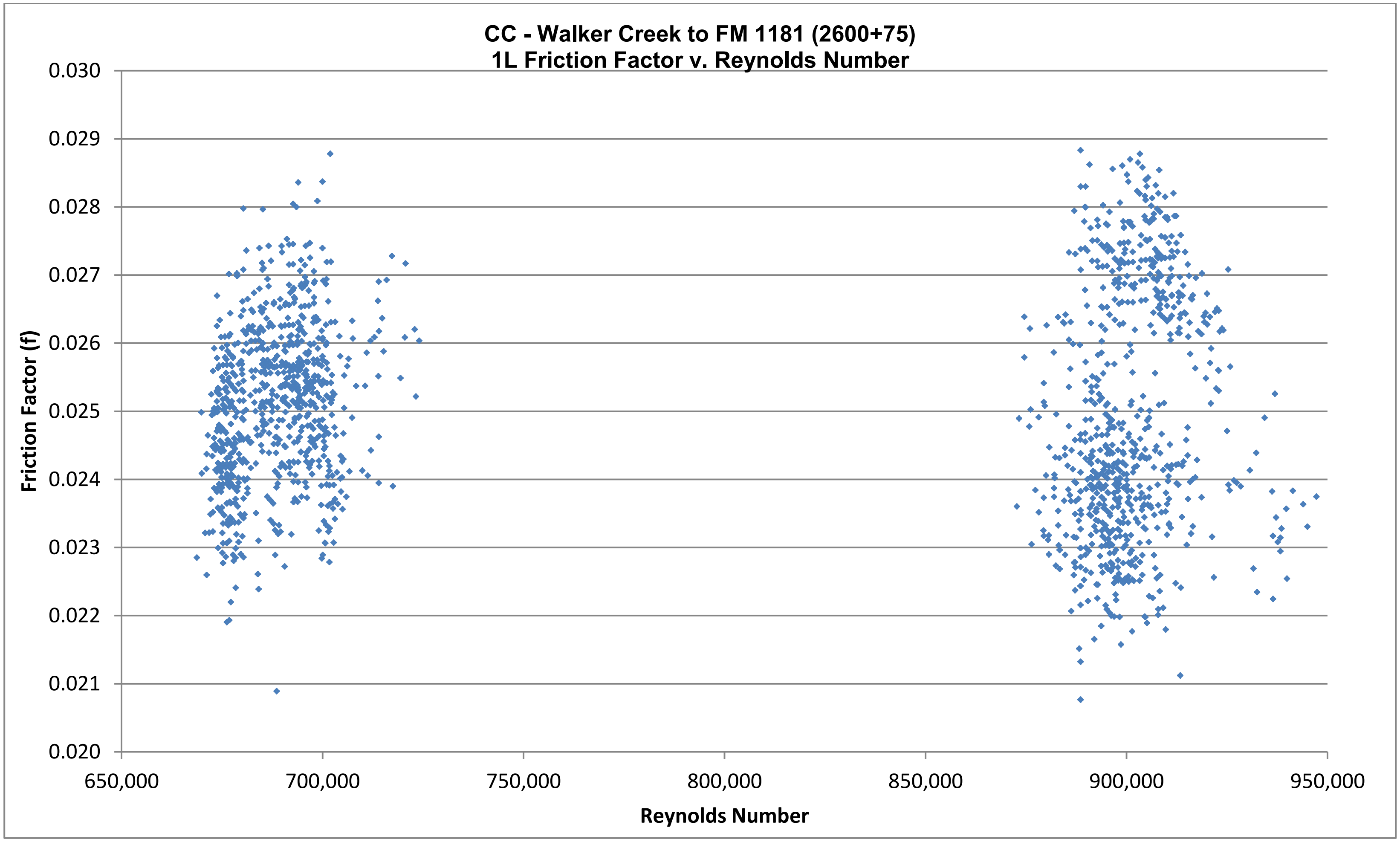
CR 3250 to Rosewood



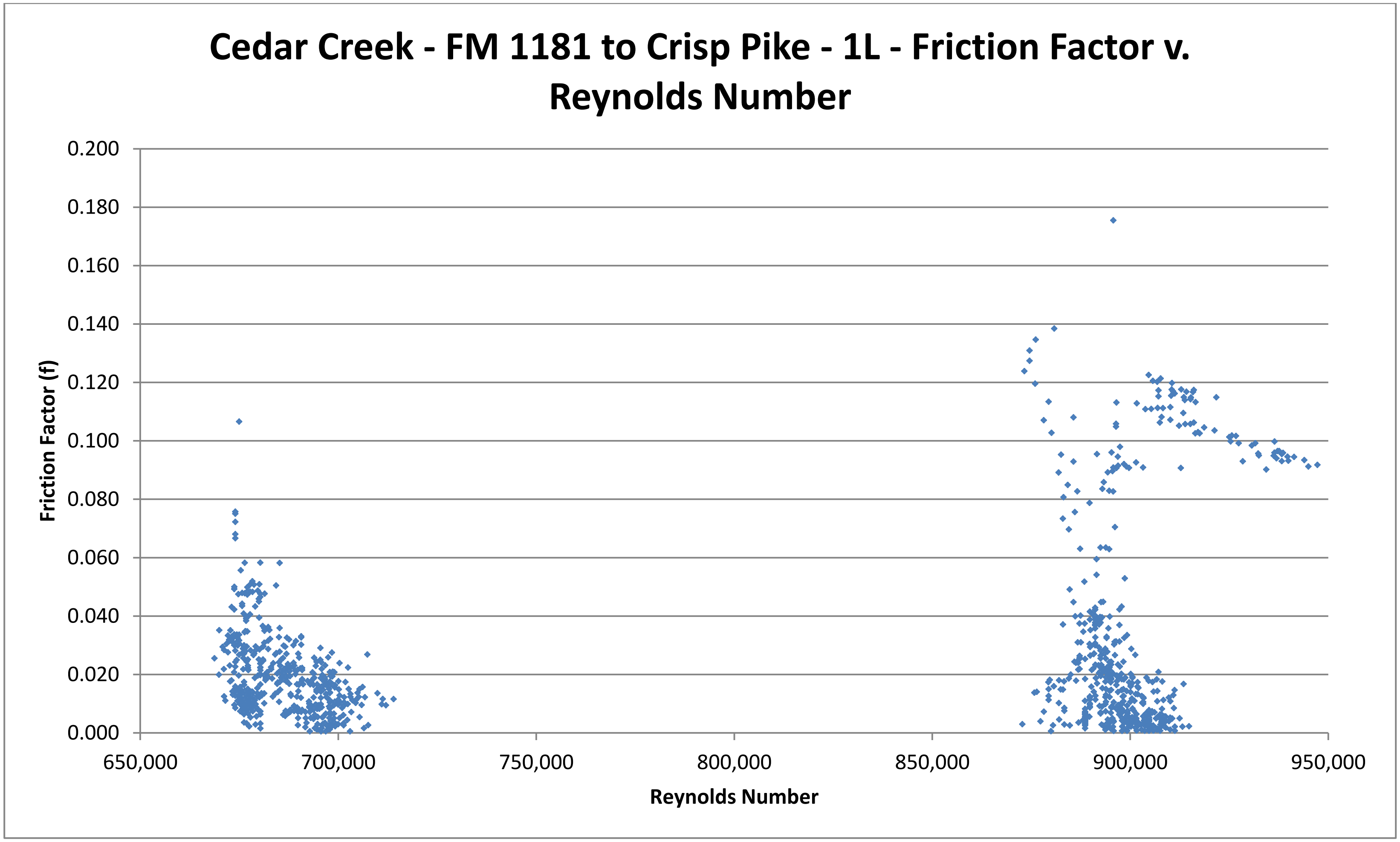
Rosewood to Walker Creek



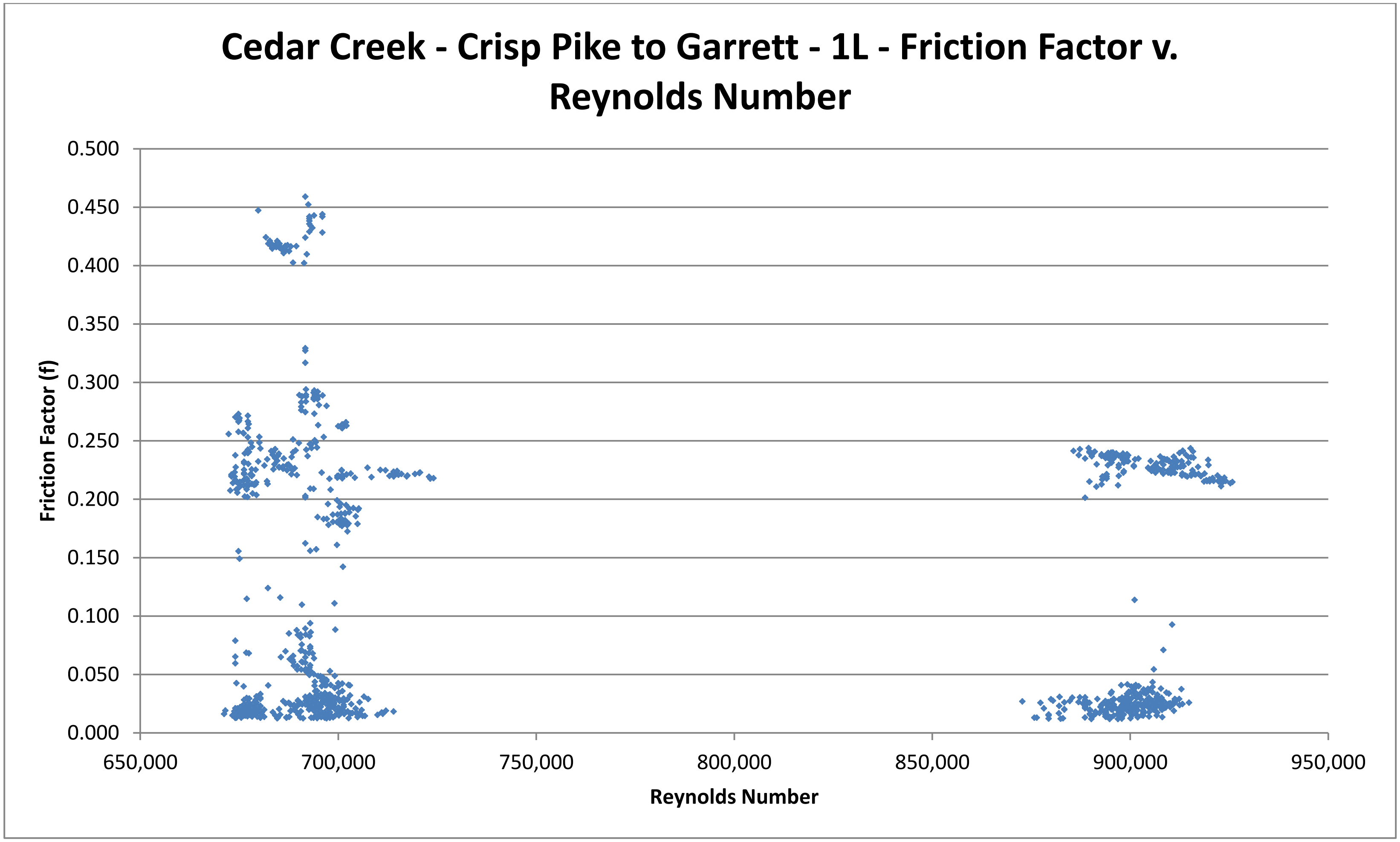
Walker Creek to FM 1181



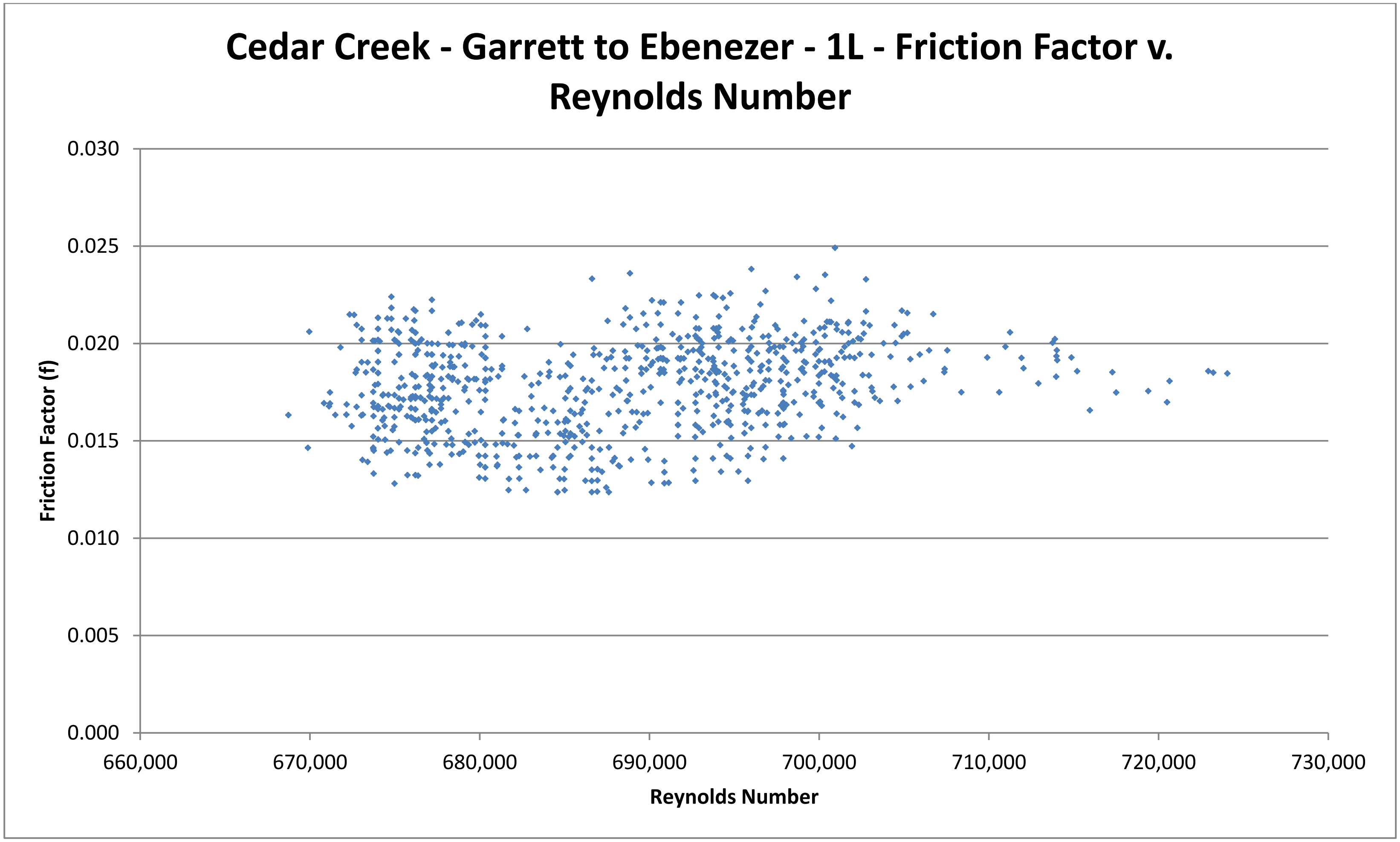
FM 1181 to Crisp Pike



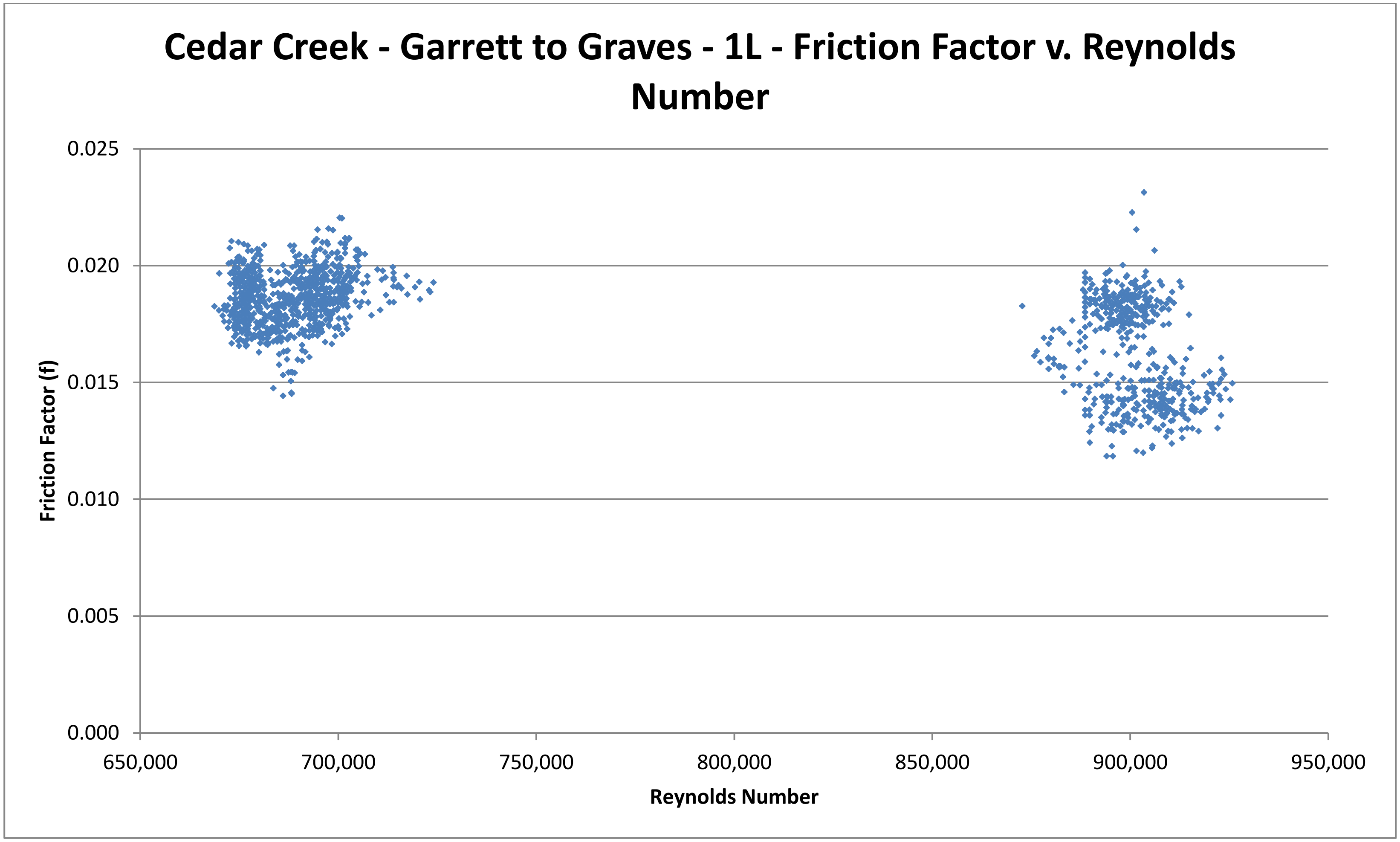
Crisp Pike to Garrett



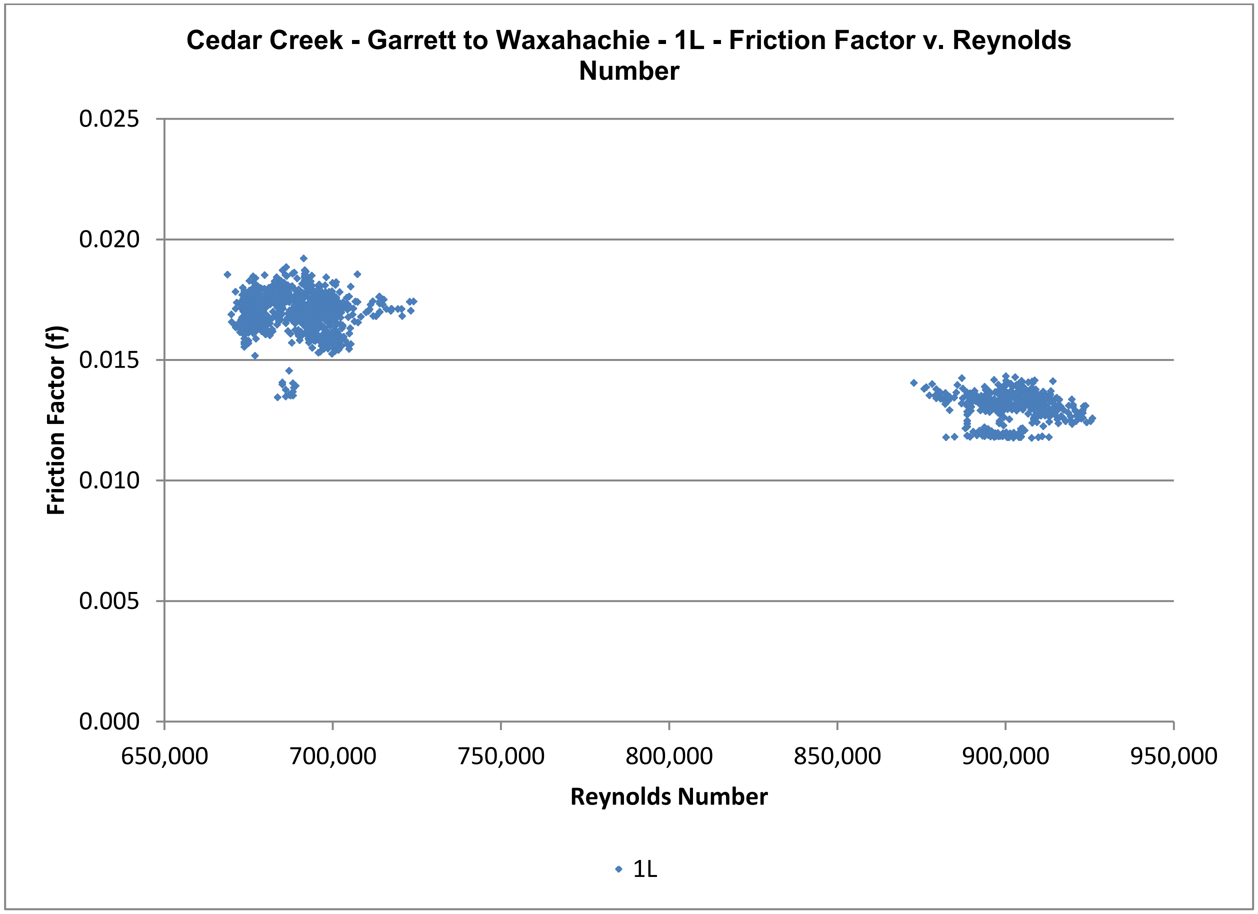
Garrett to Ebenezer



Garrett to Graves

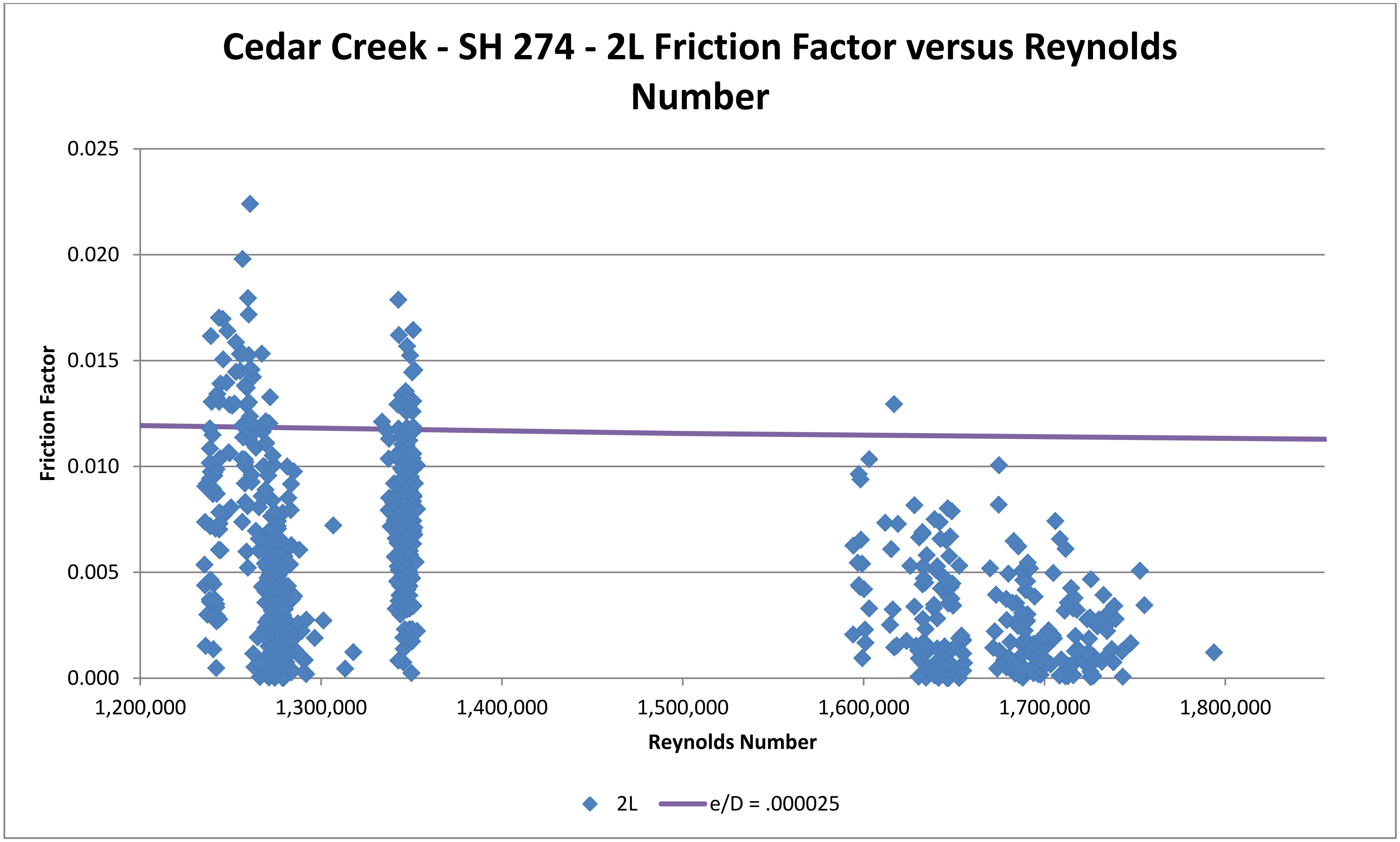


Garrett to Waxahachie

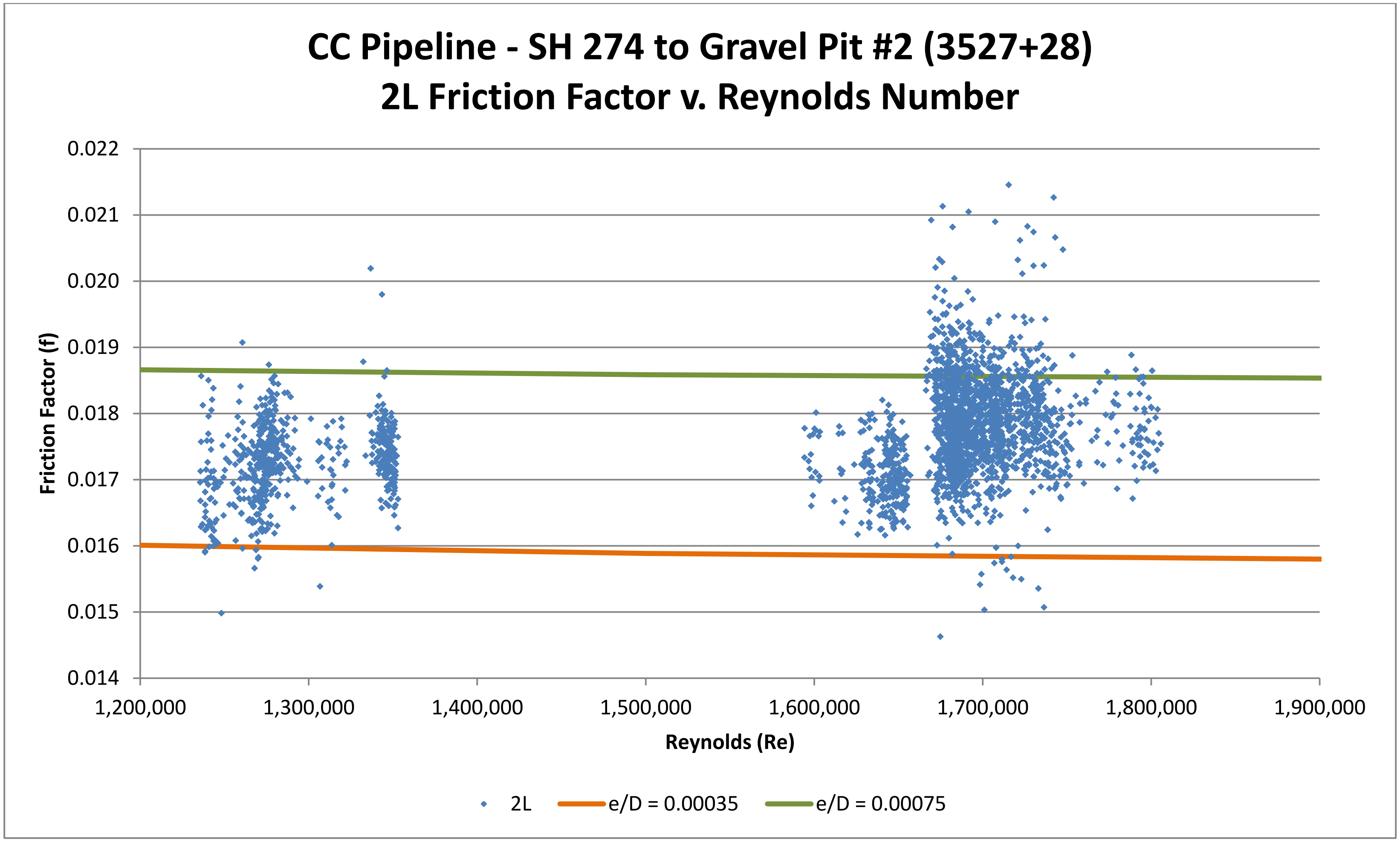


Cedar Creek 2L Pump Combination

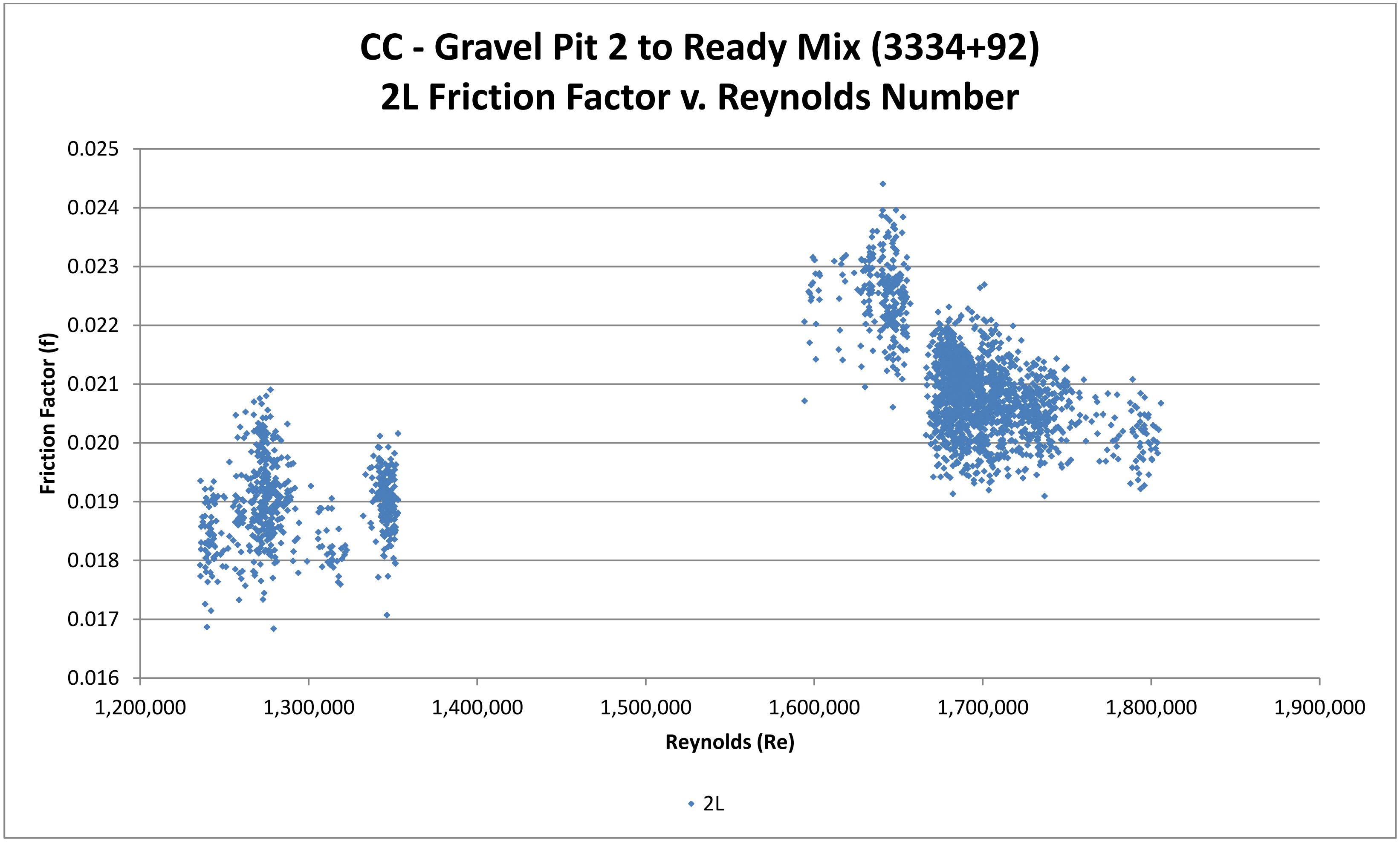
Cedar Creek Lake Pump Station to State Highway 271



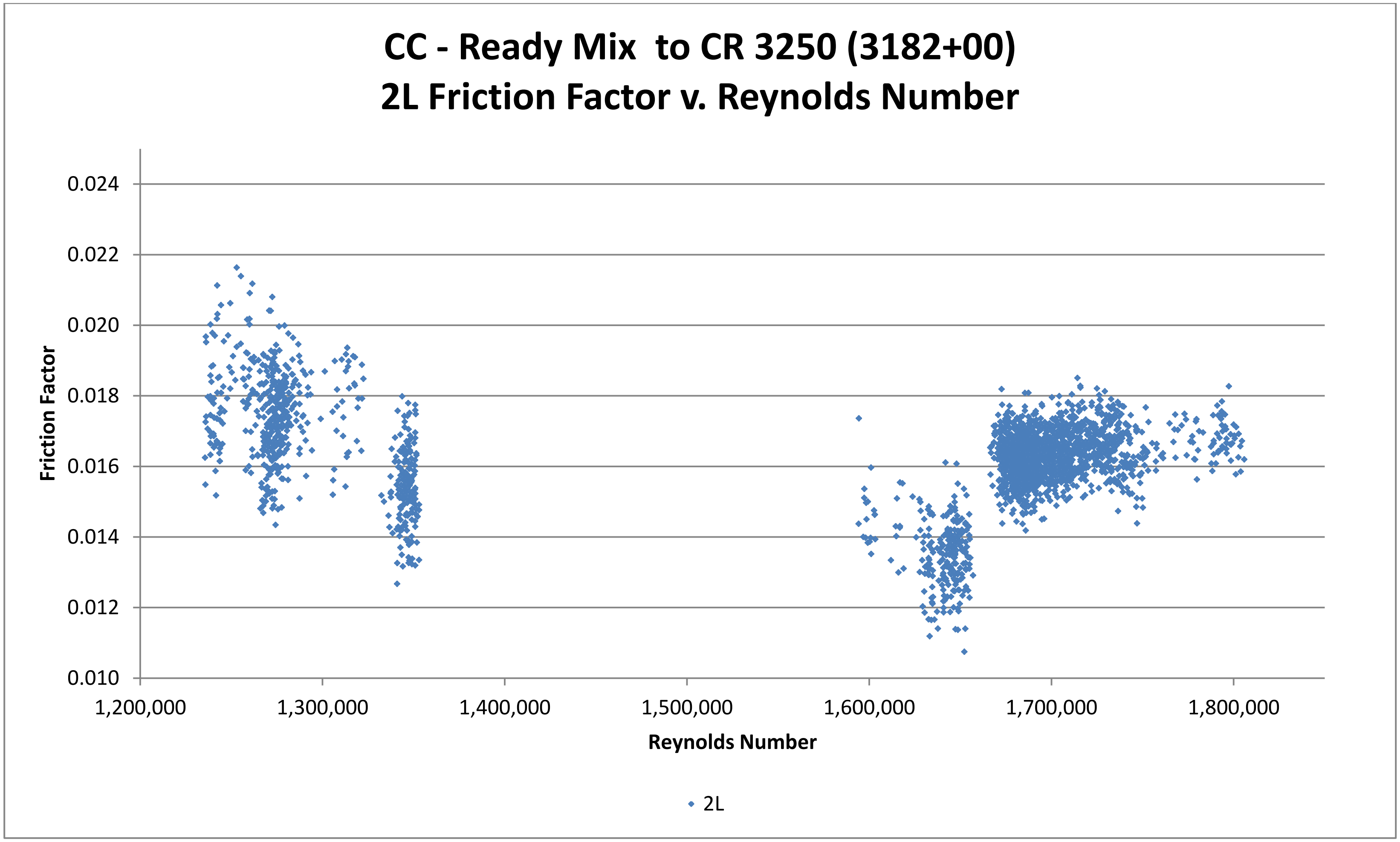
SH 274 to Gravel Pit #2



Gravel Pit #2 to Ready Mix

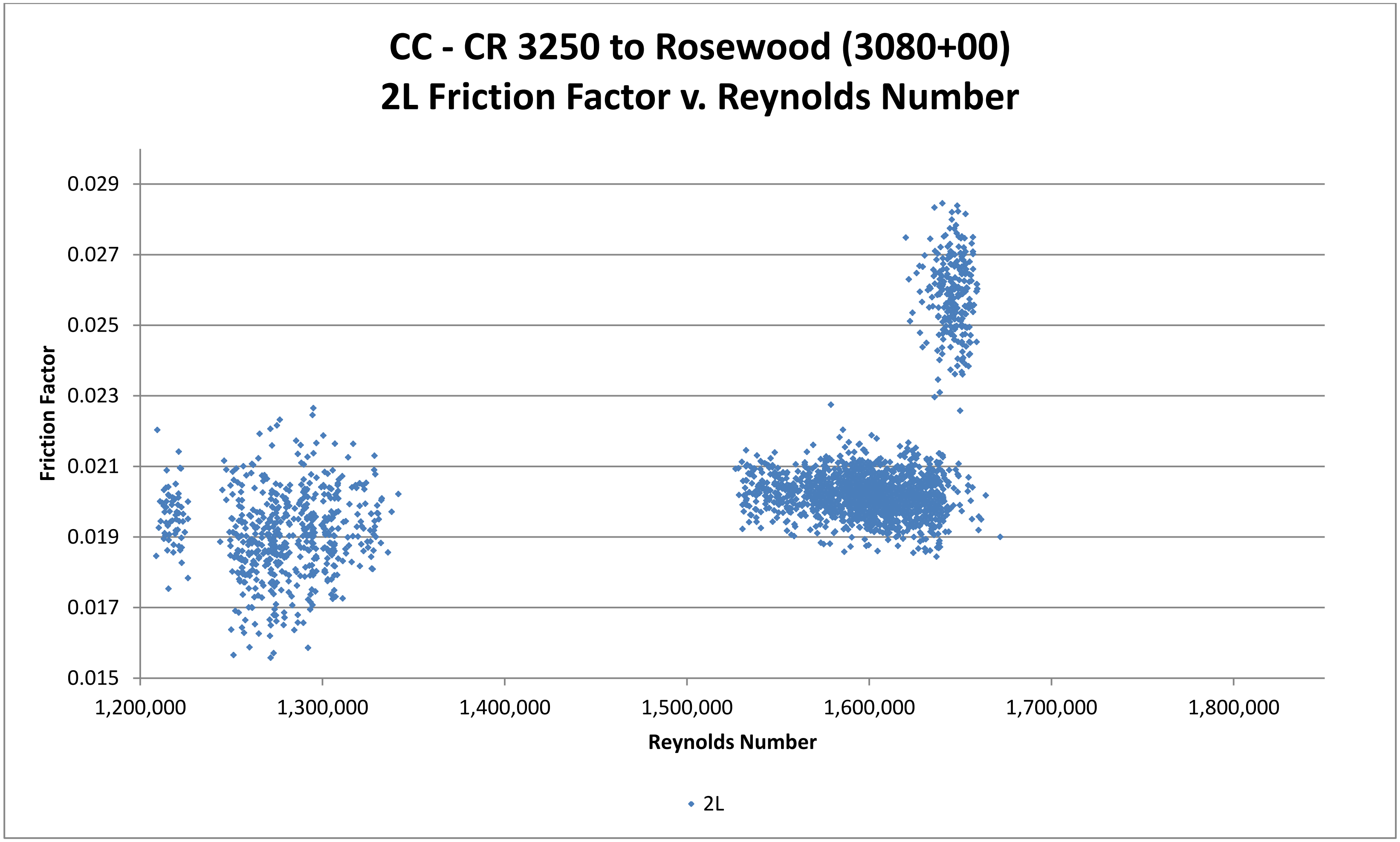


Ready Mix to CR 3250

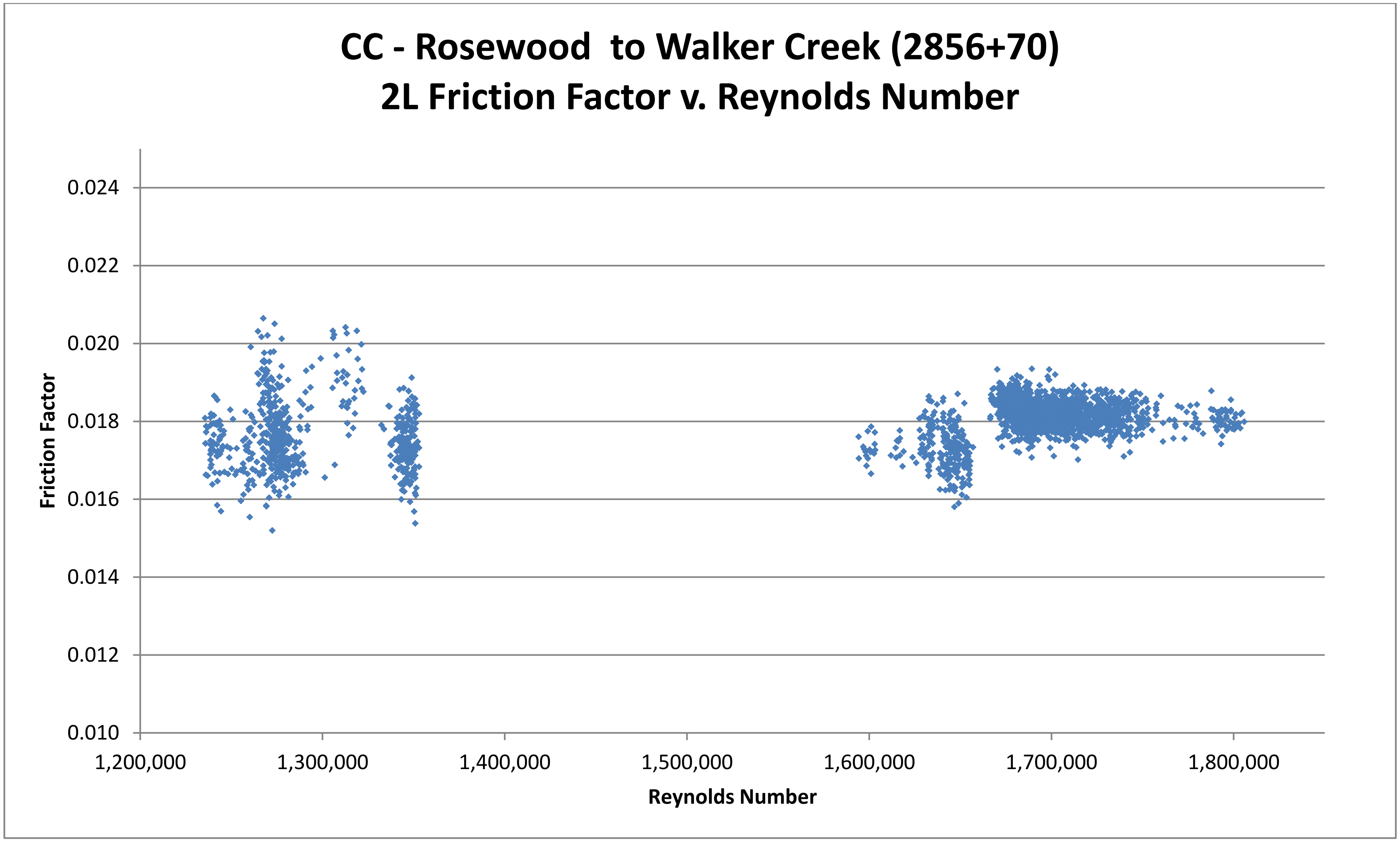


126

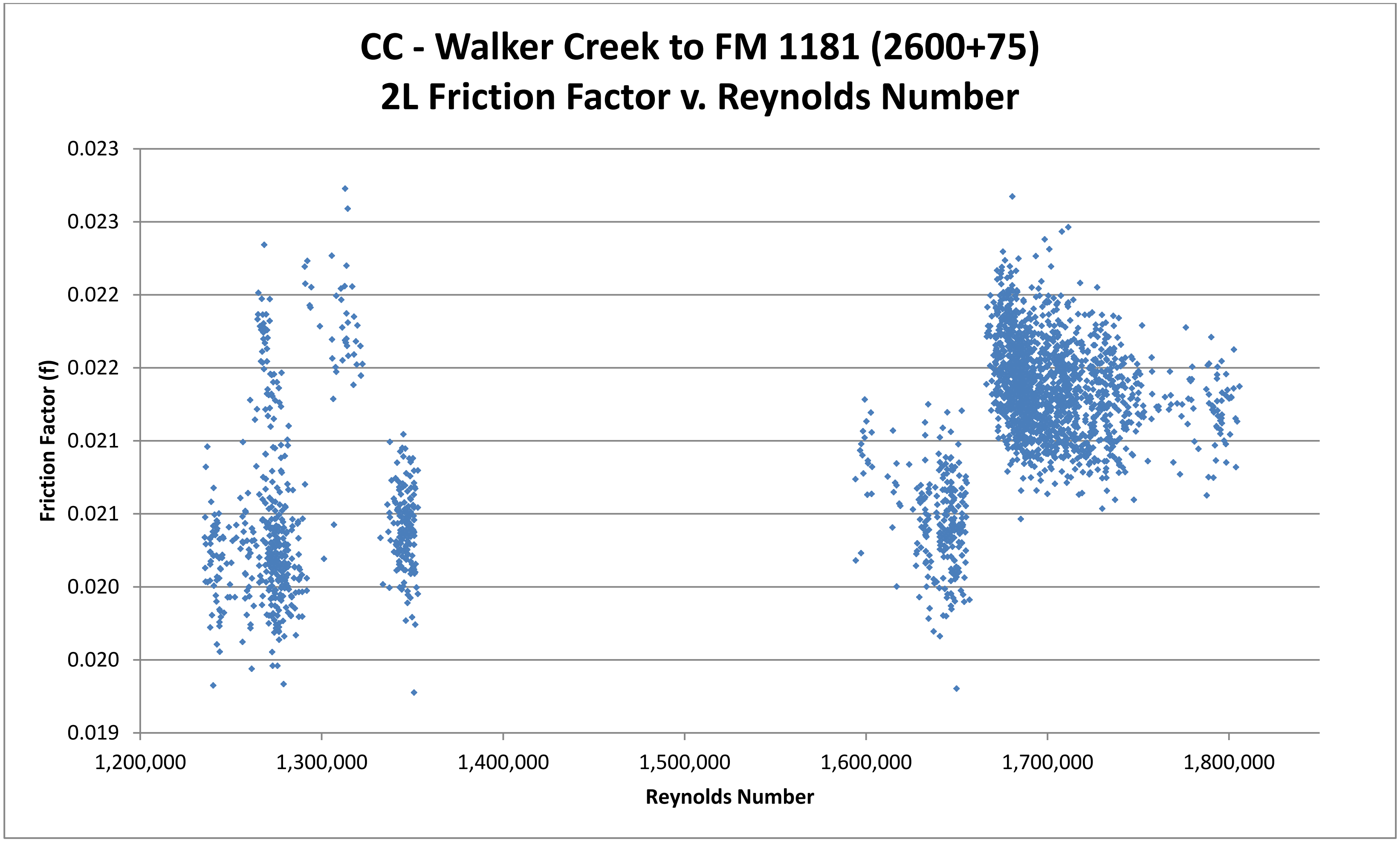
CR 3250 to Rosewood



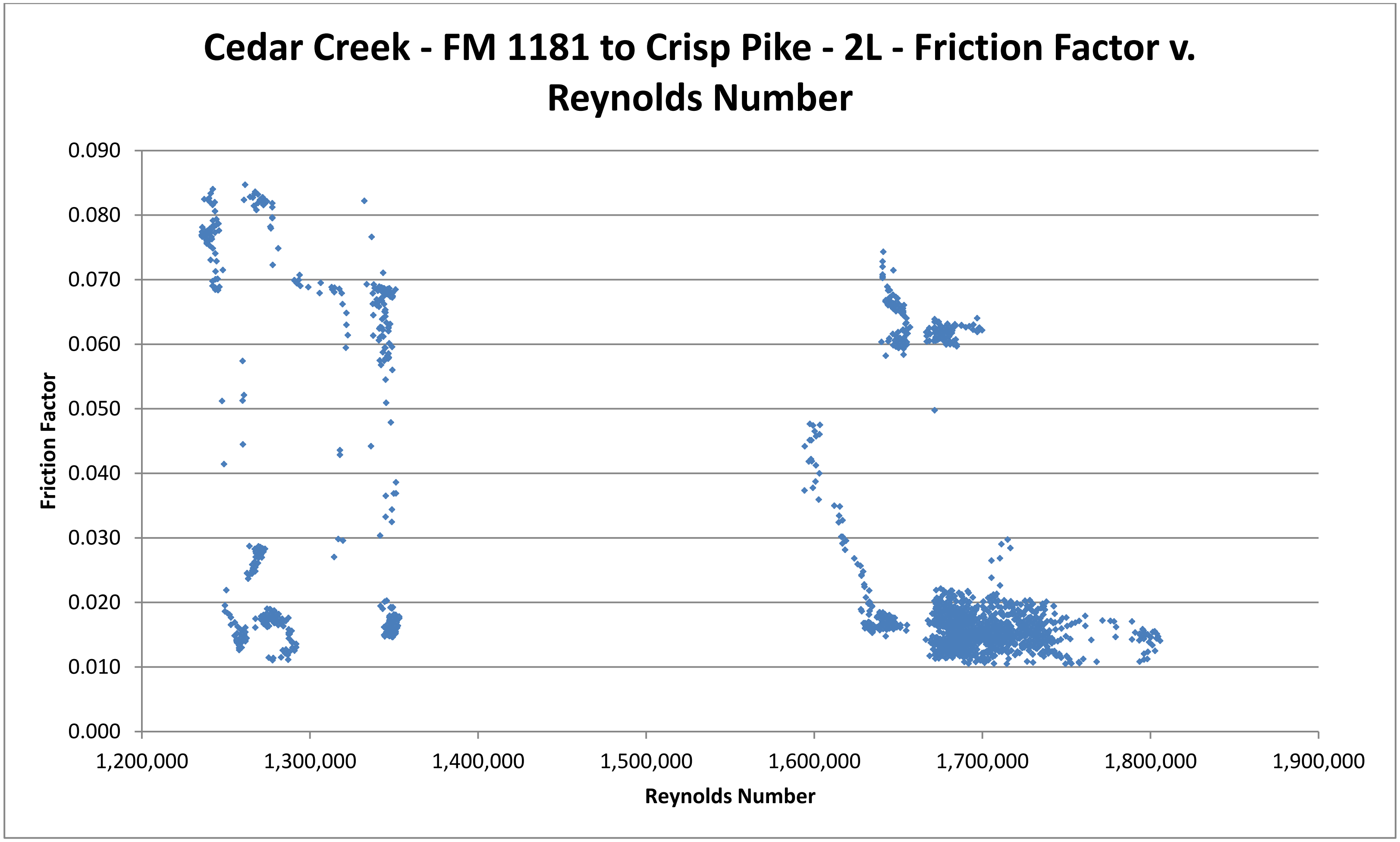
Rosewood to Walker Creek



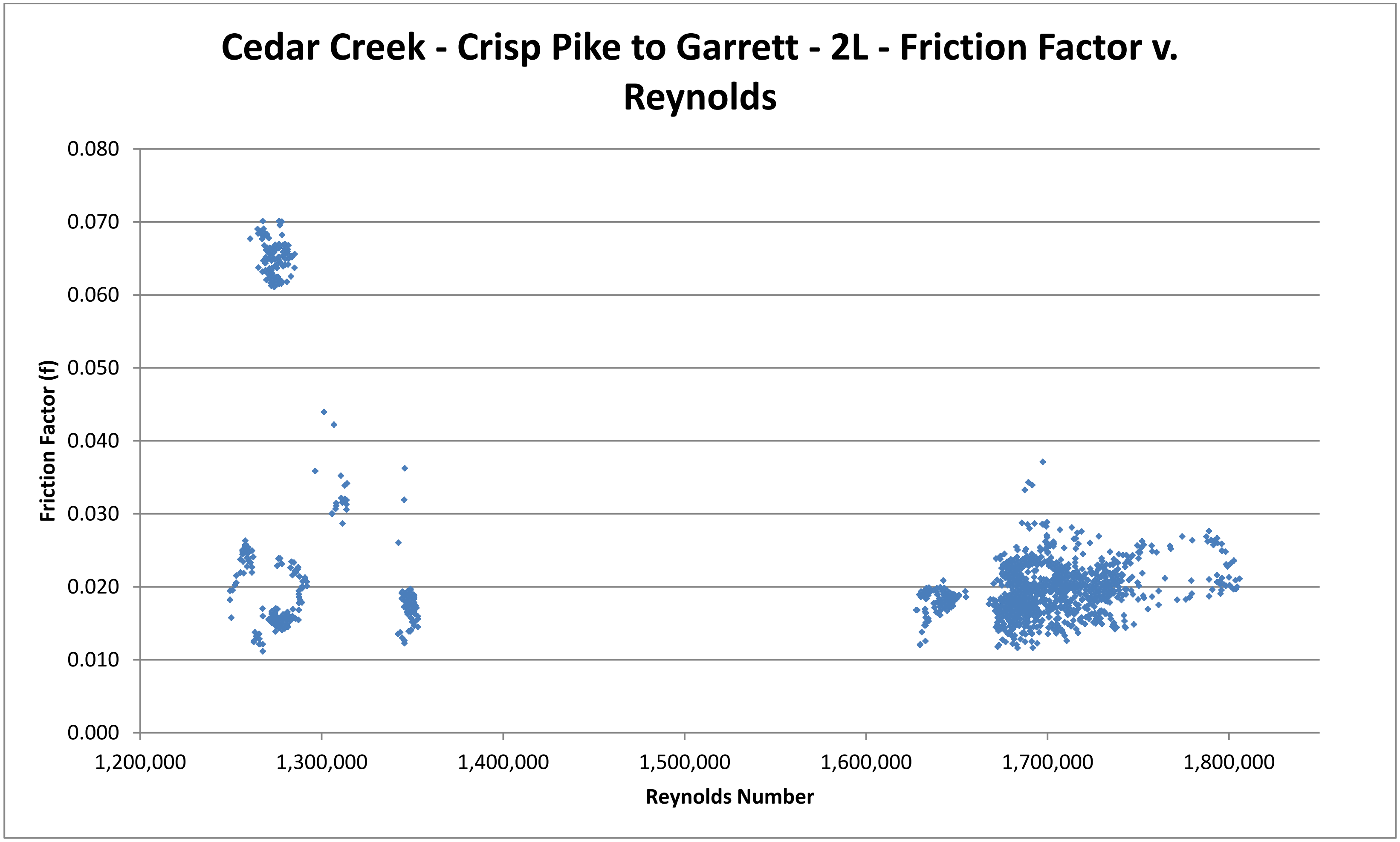
Walker Creek to FM 1181



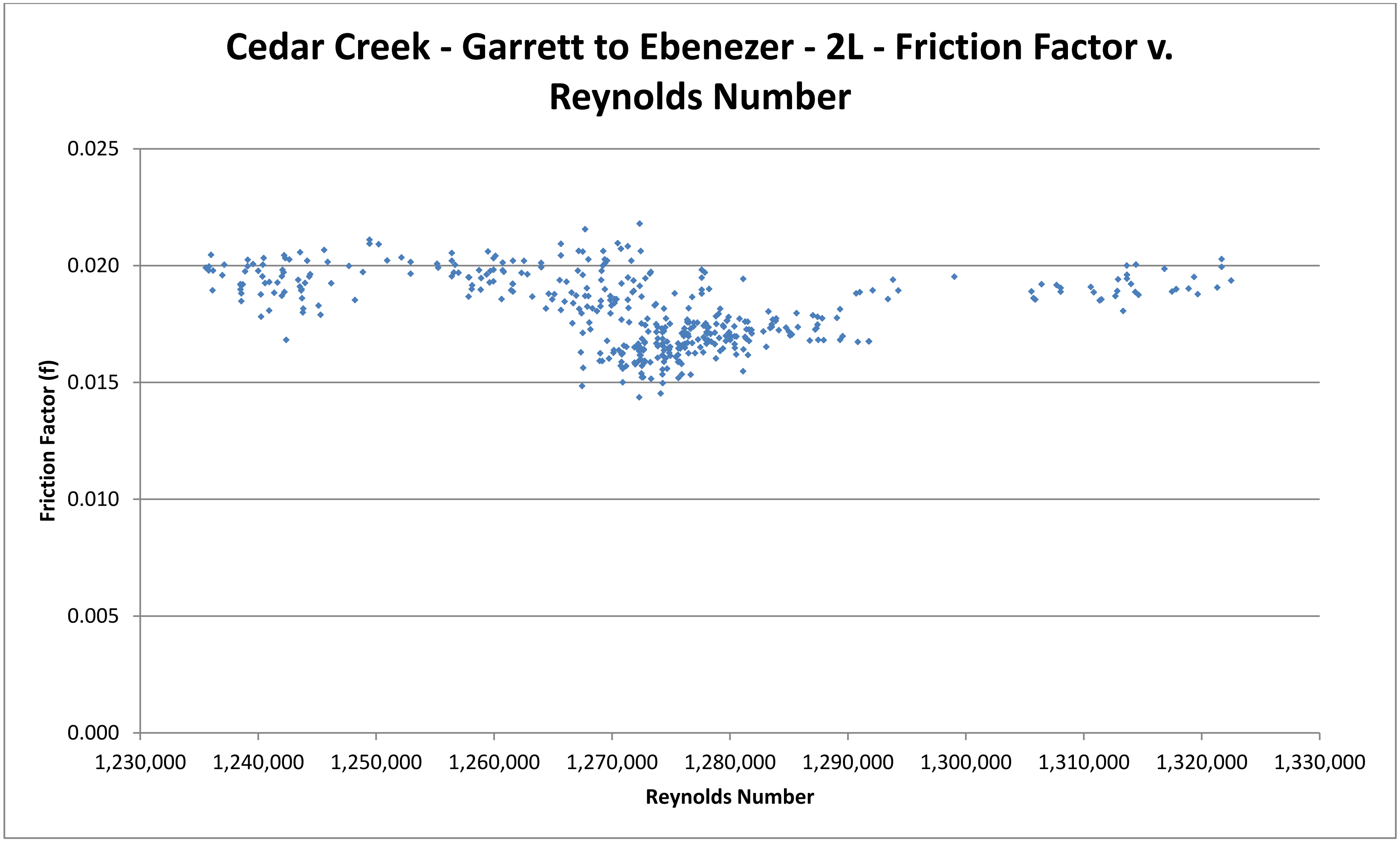
FM 1181 to Crisp Pike



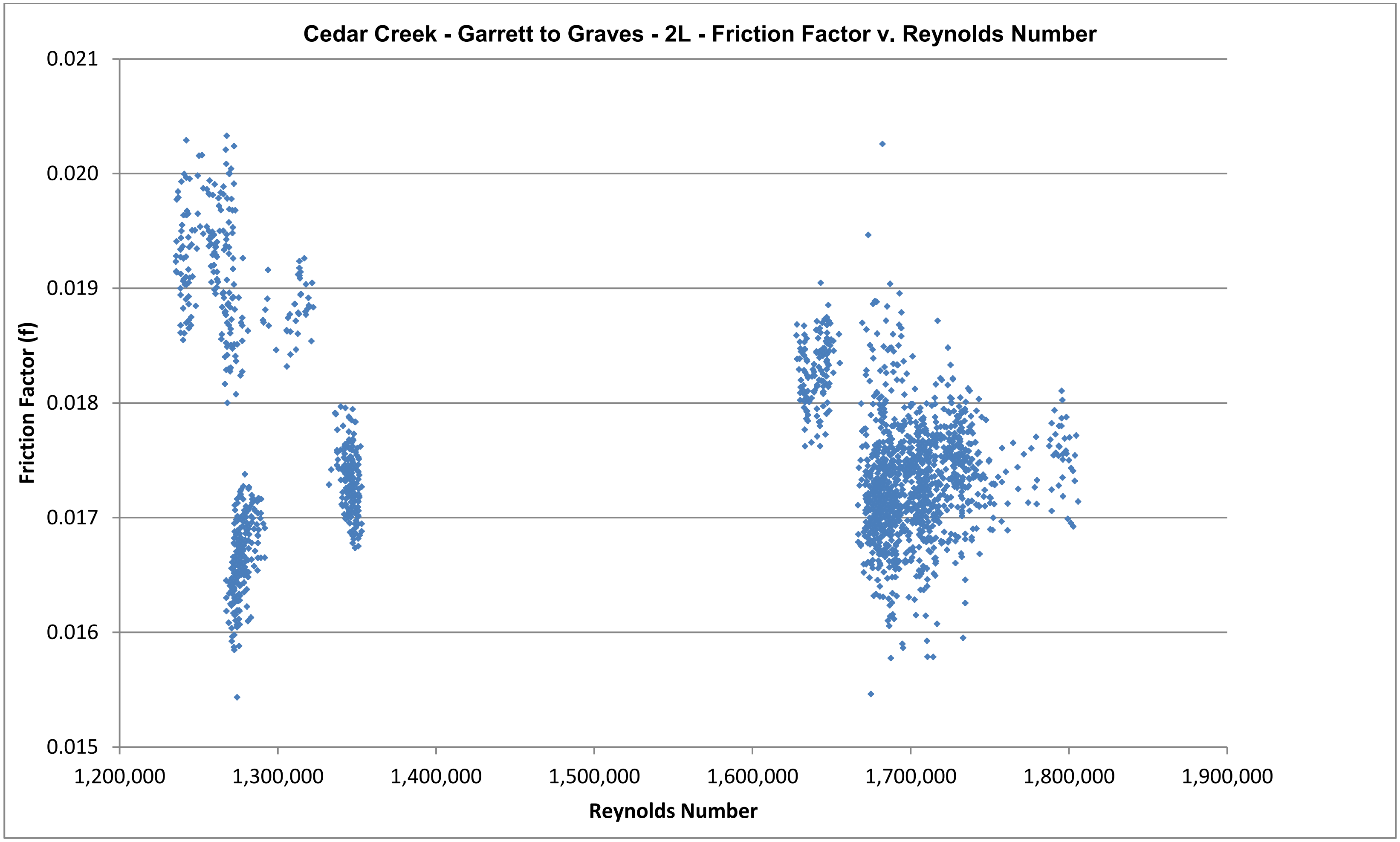
Crisp Pike to Garrett



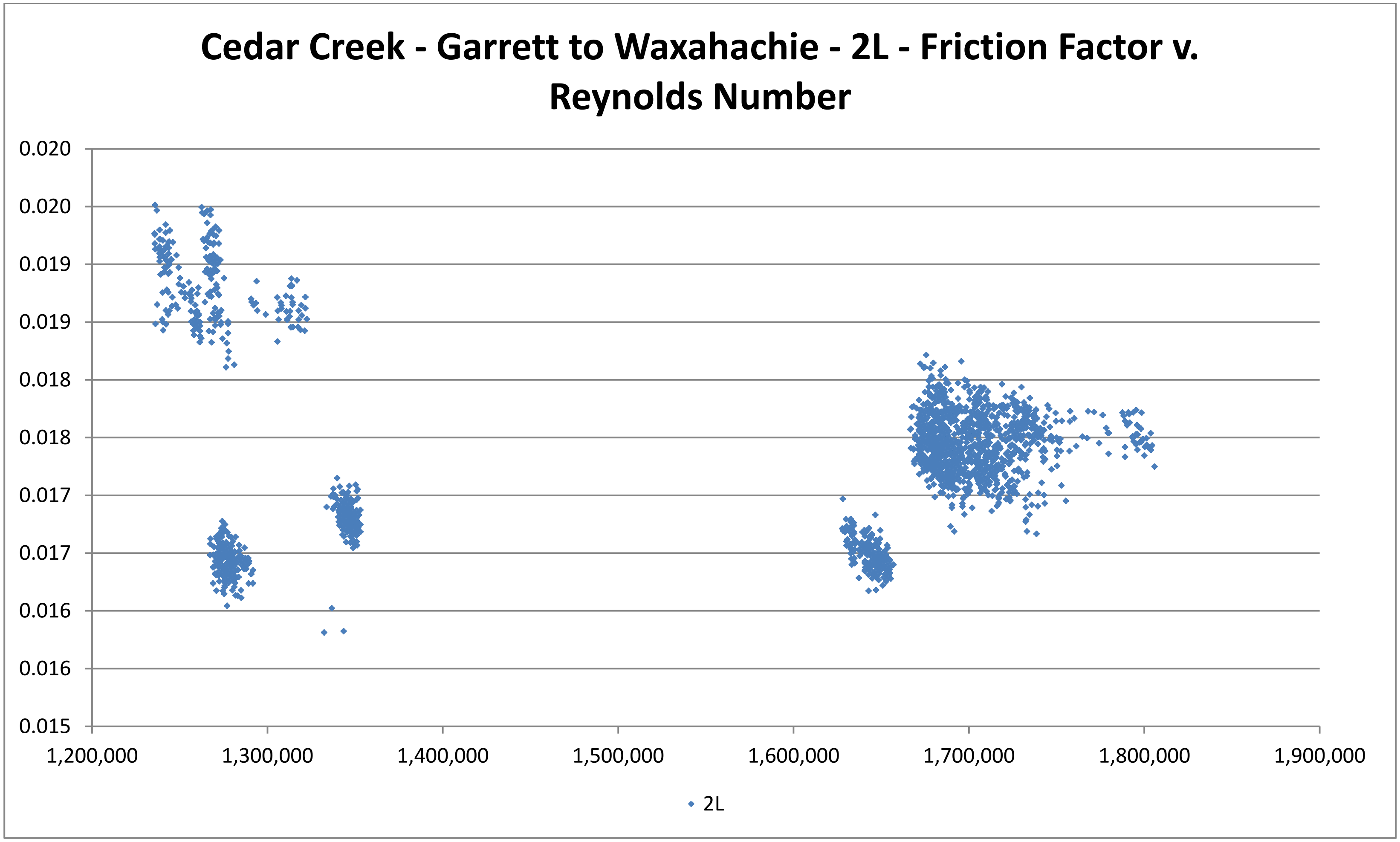
Garrett to Ebenezer



Garrett to Graves

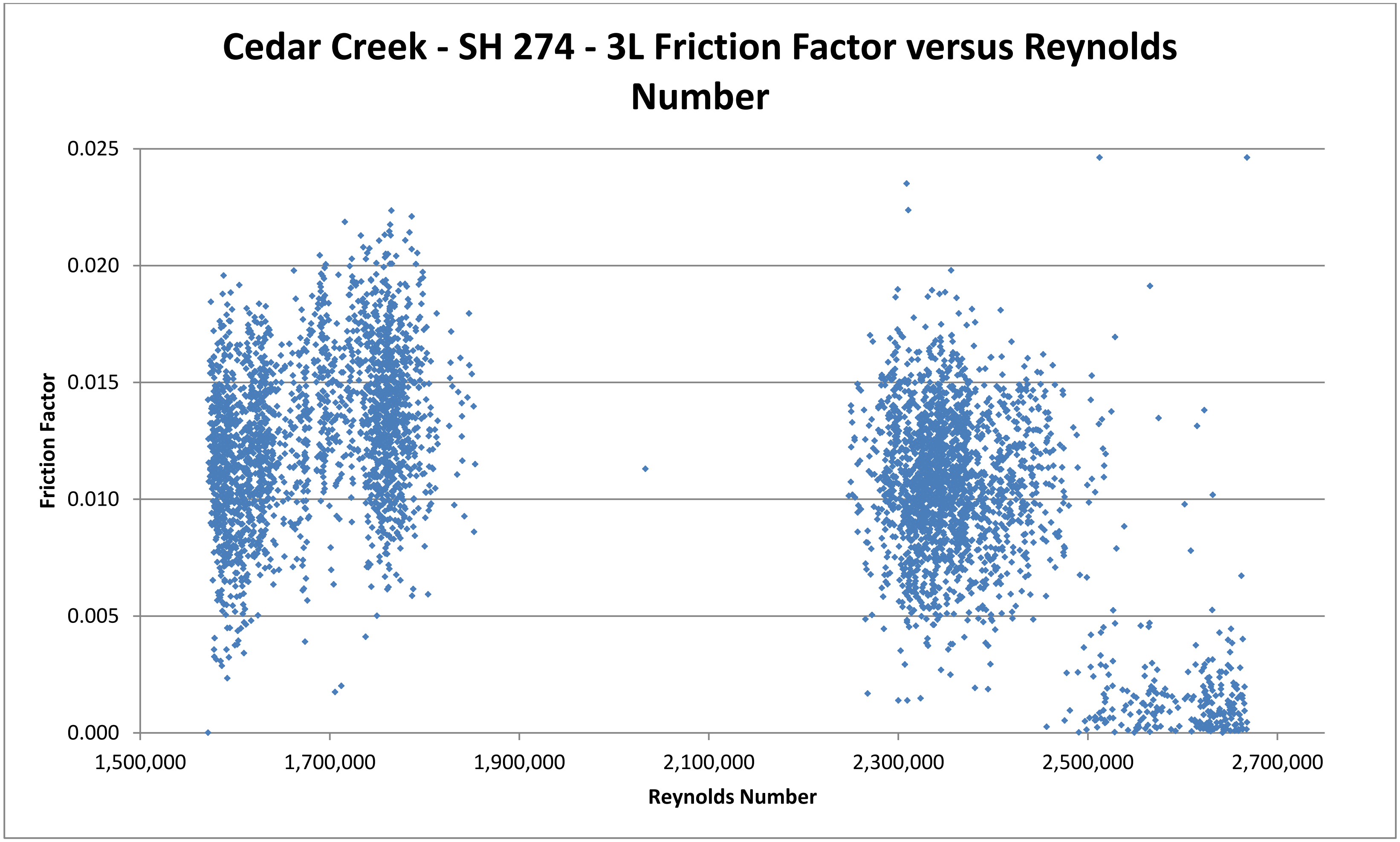


Garrett to Waxahachie

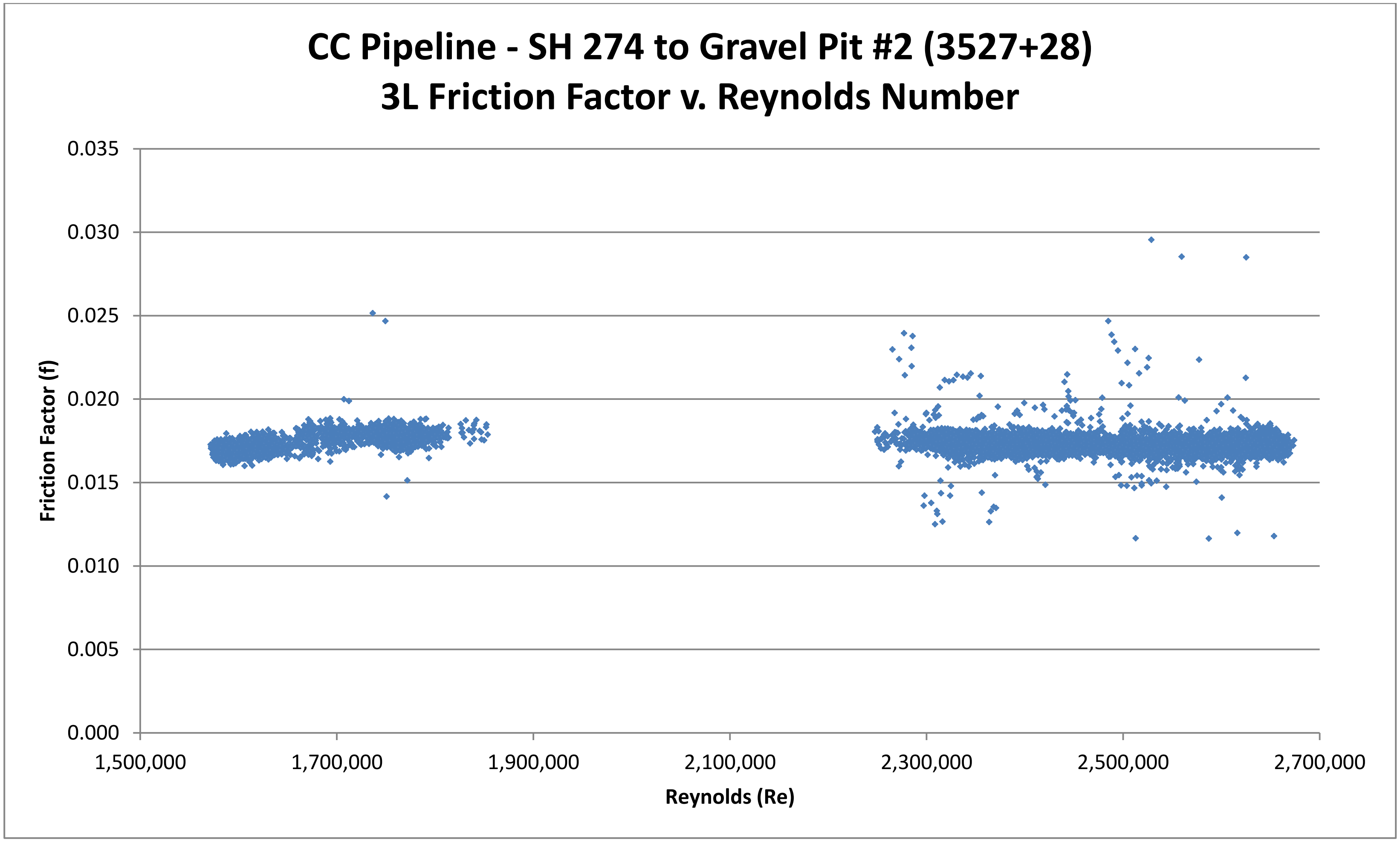


Cedar Creek 3L Pump Combination

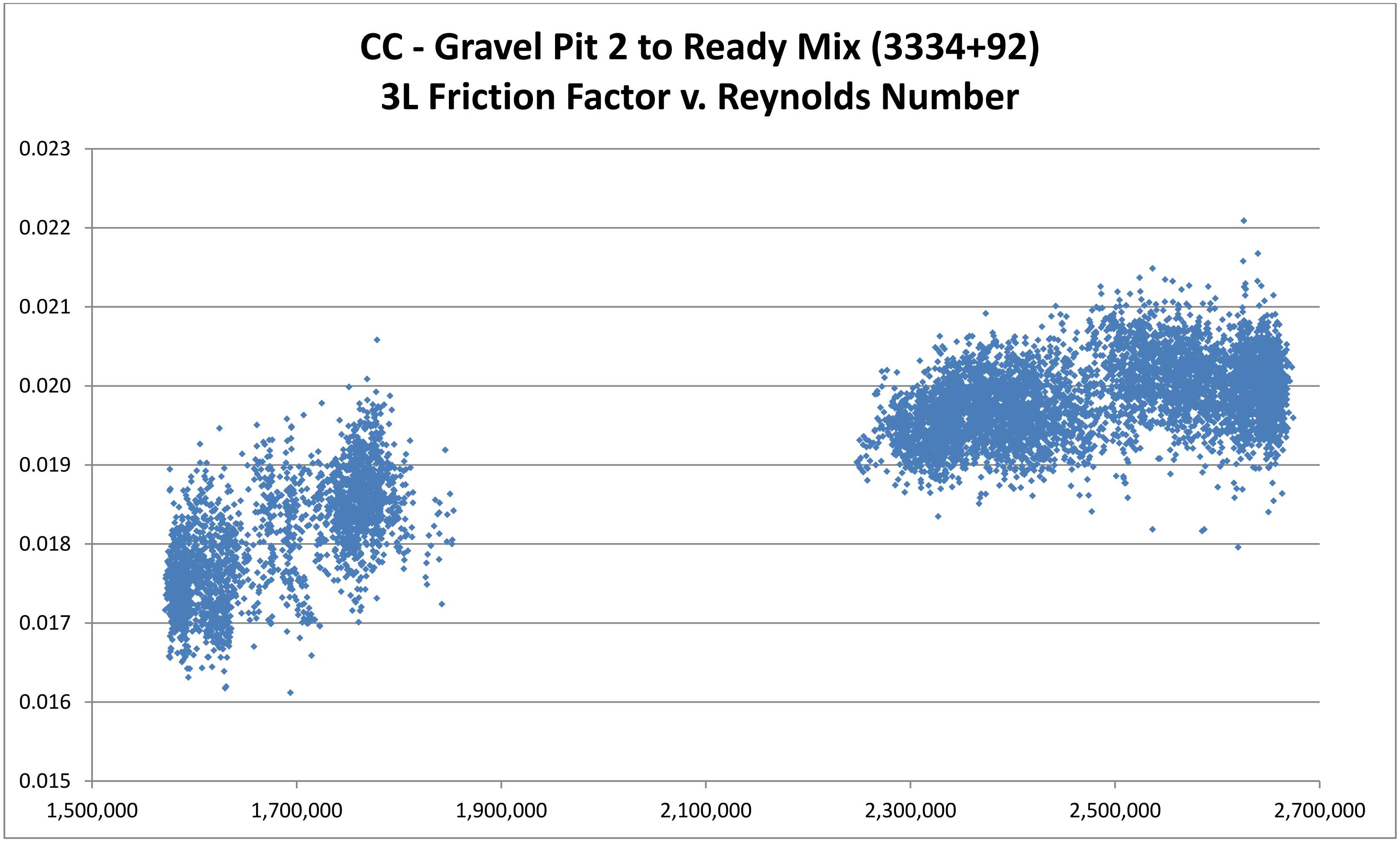
Cedar Creek Lake Pump Station to State Highway 271



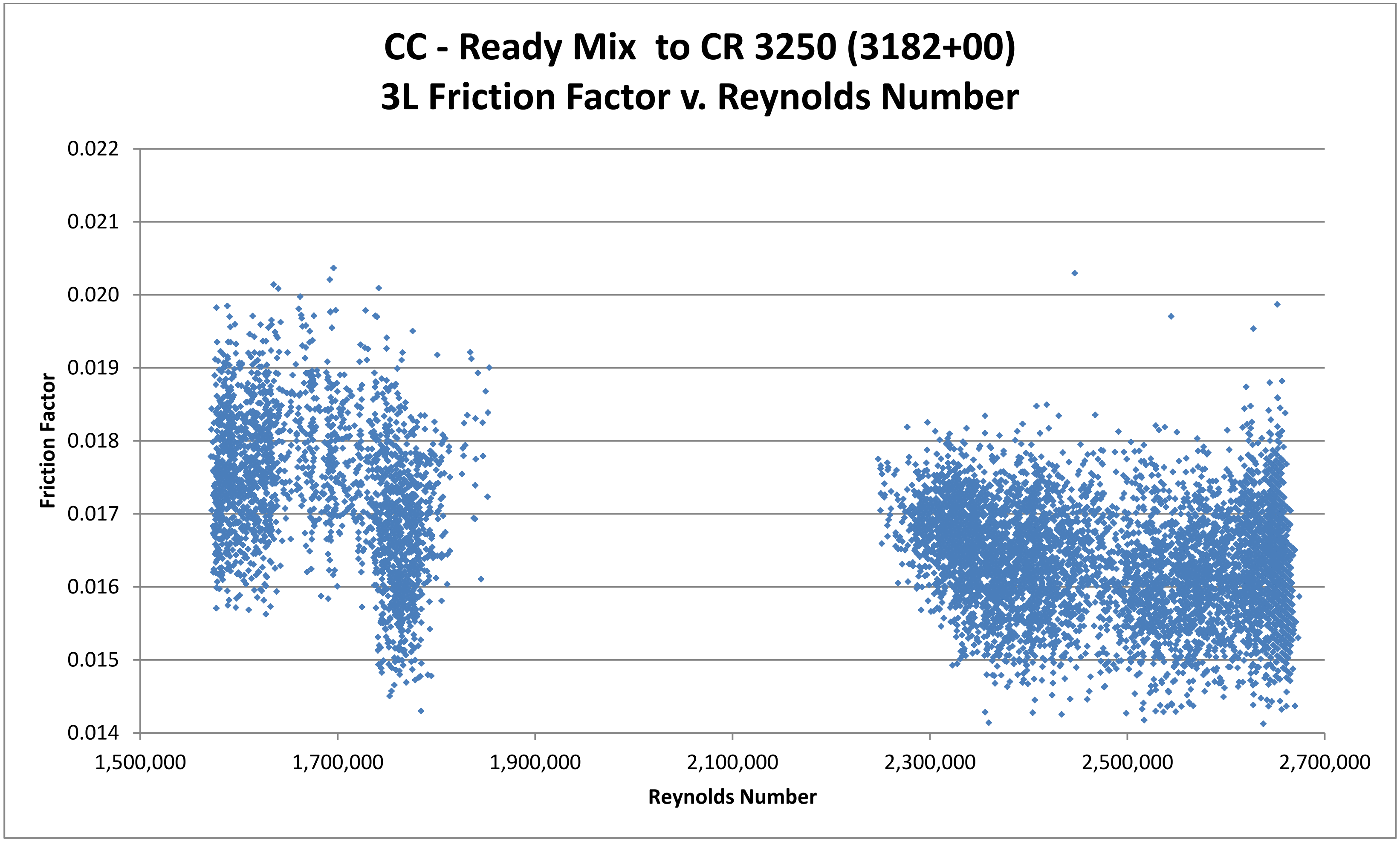
SH 274 to Gravel Pit #2



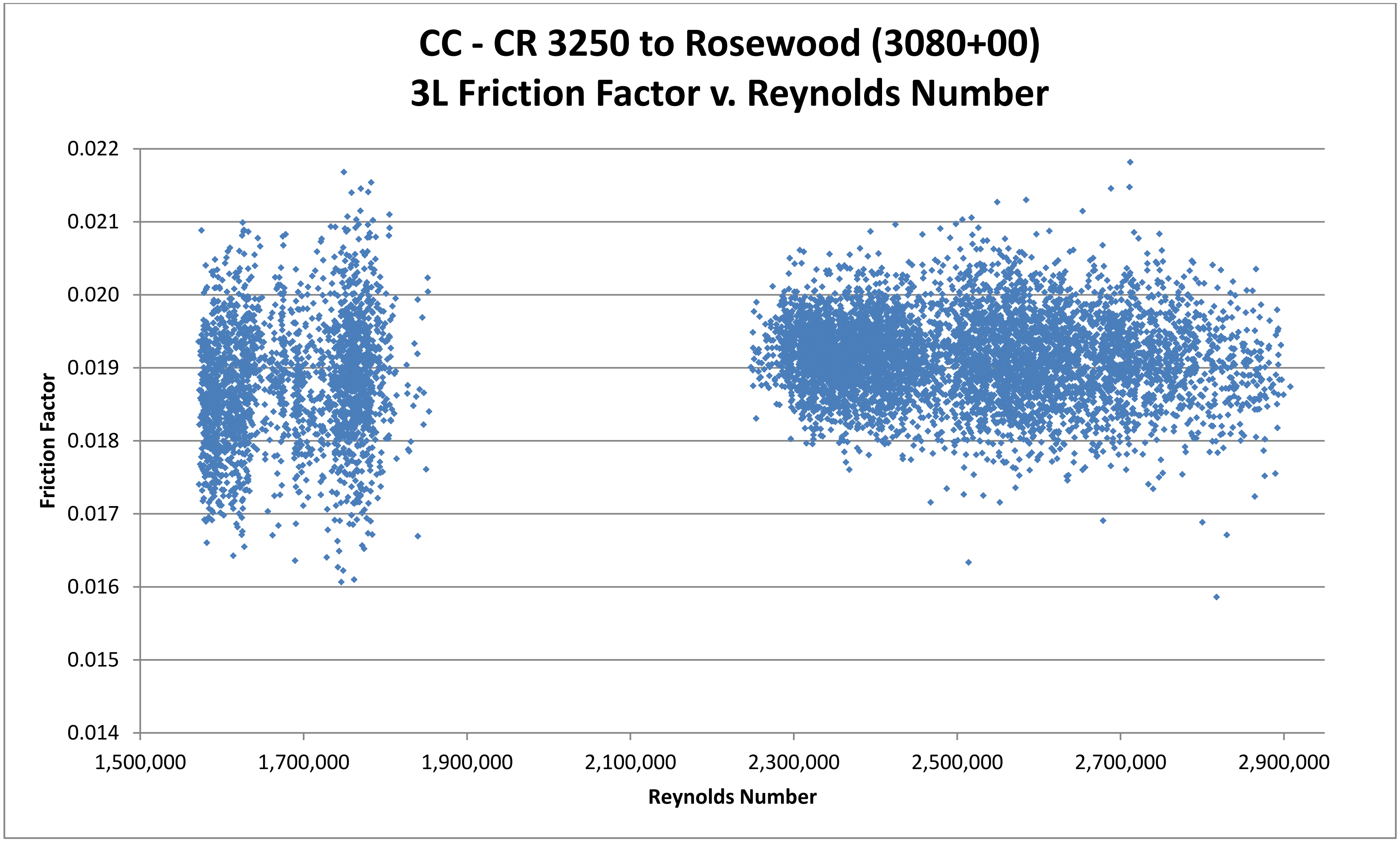
Gravel Pit #2 to Ready Mix



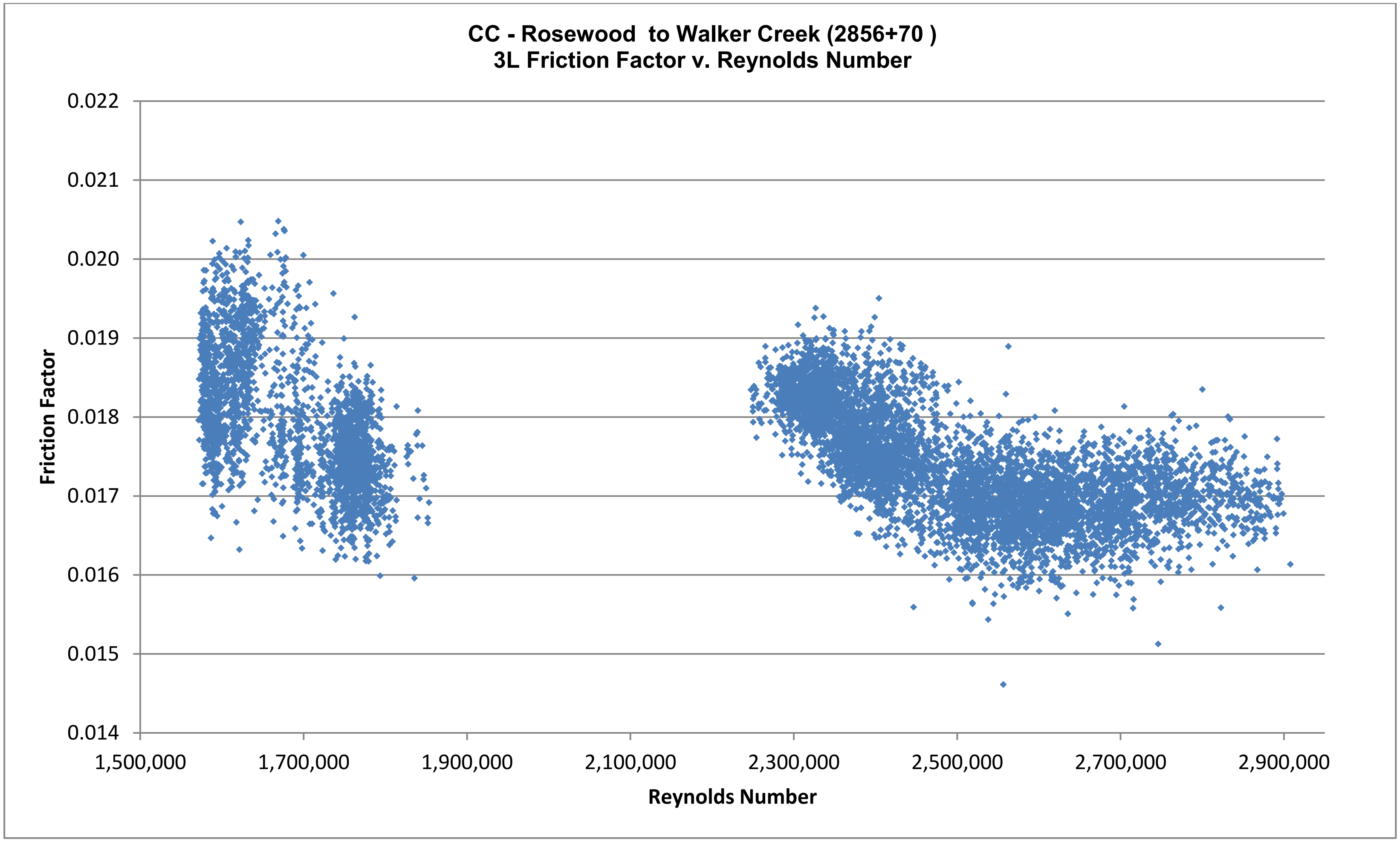
Ready Mix to CR 3250



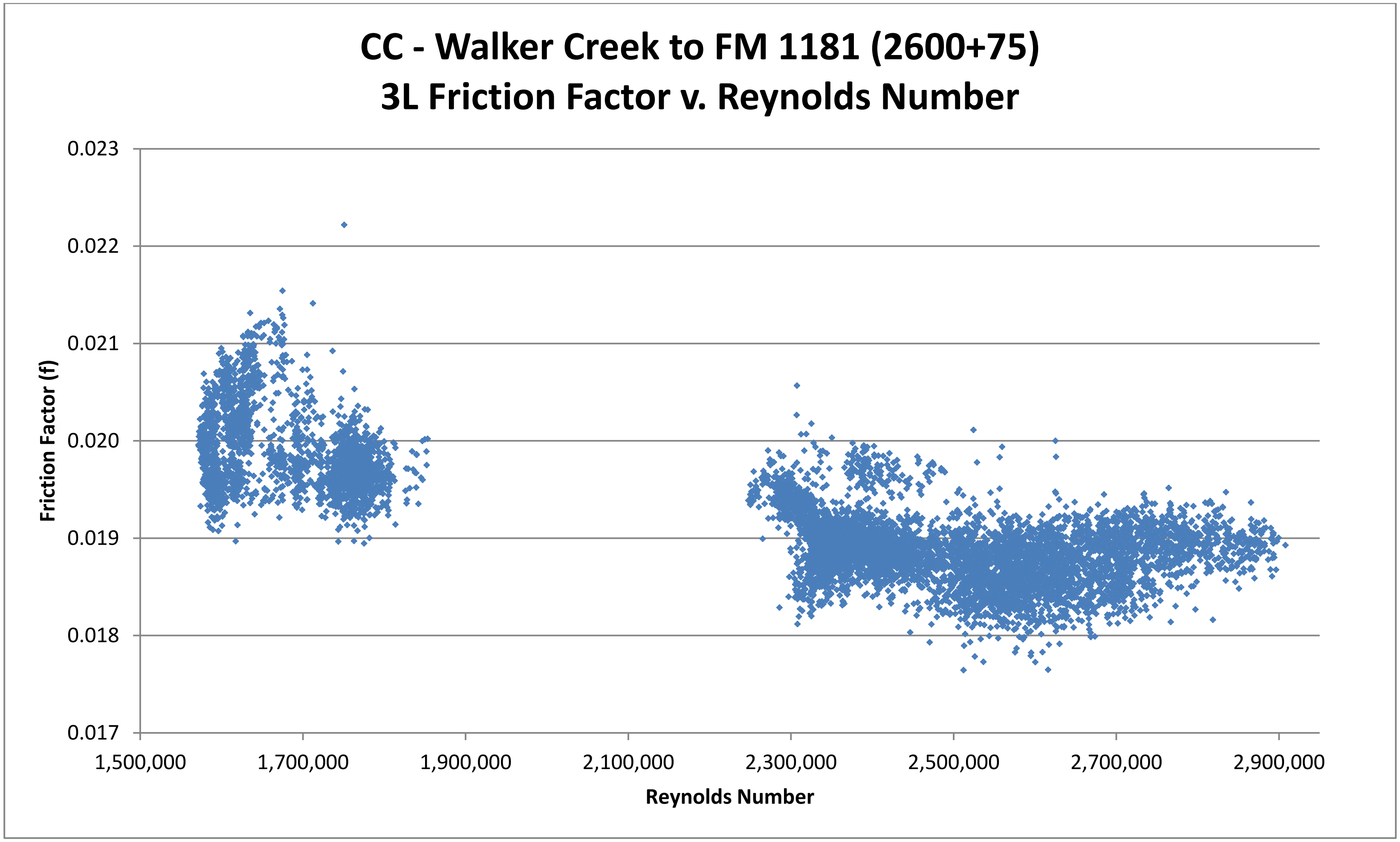
CR 3250 to Rosewood



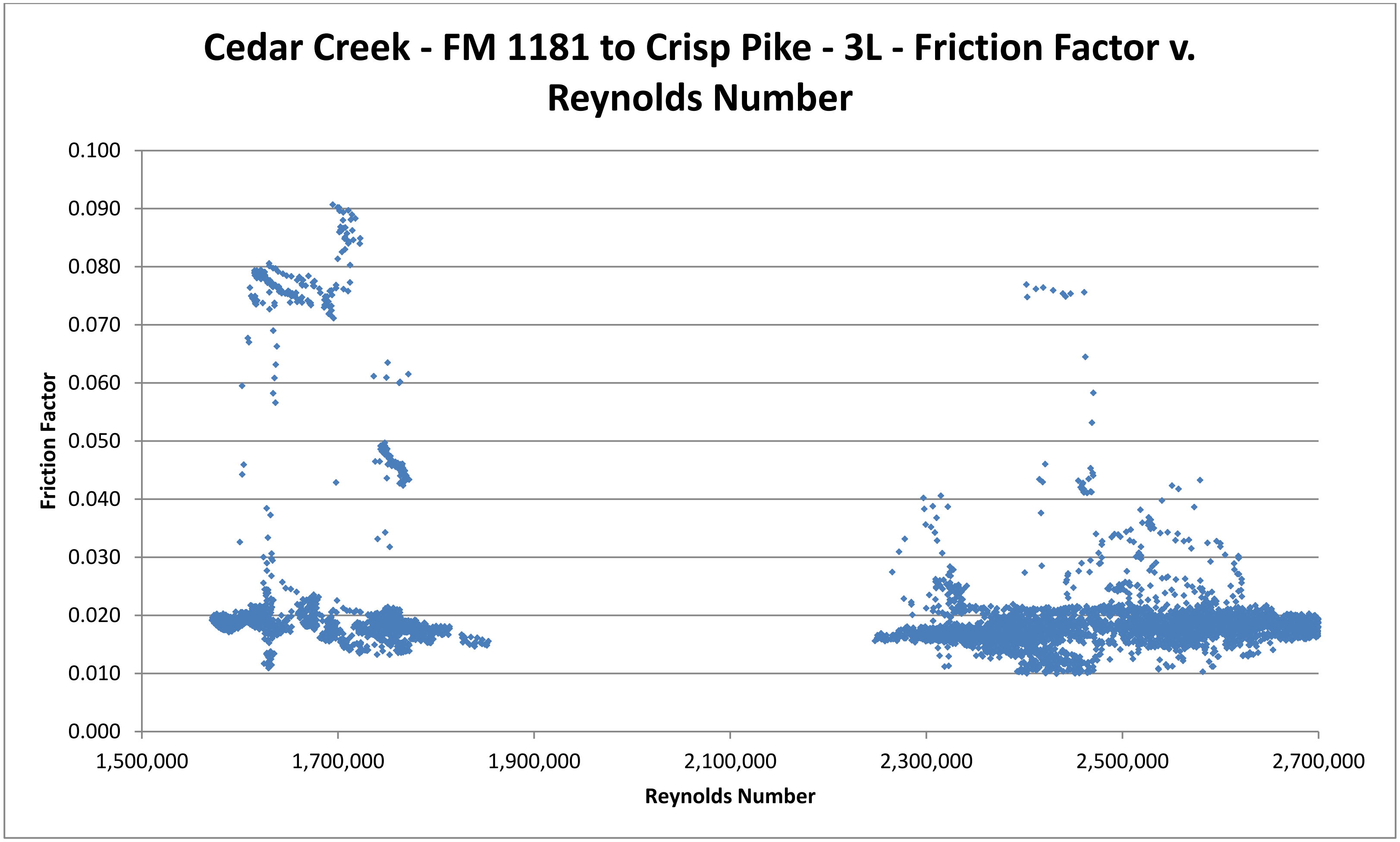
Rosewood to Walker Creek



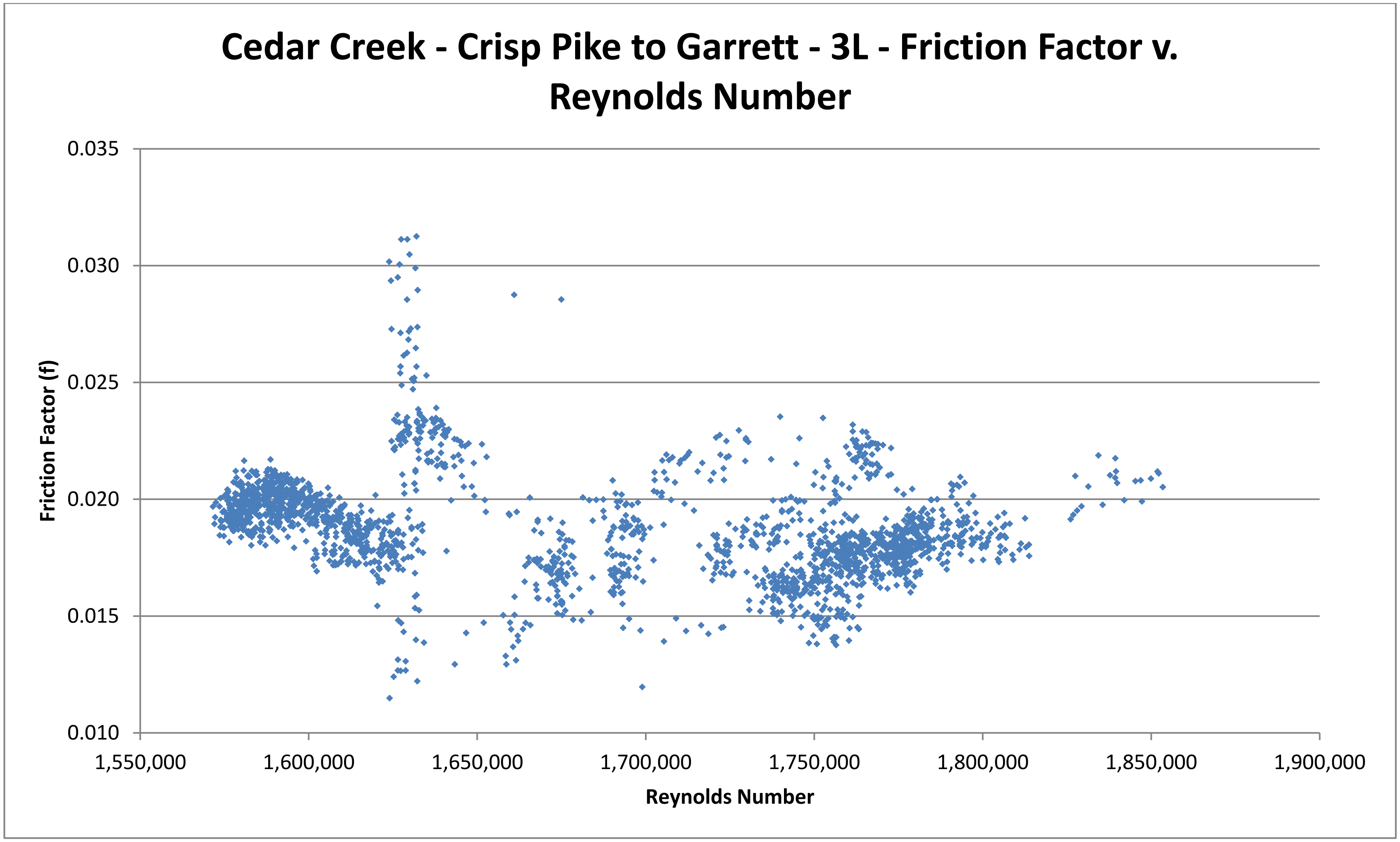
Walker Creek to FM 1181



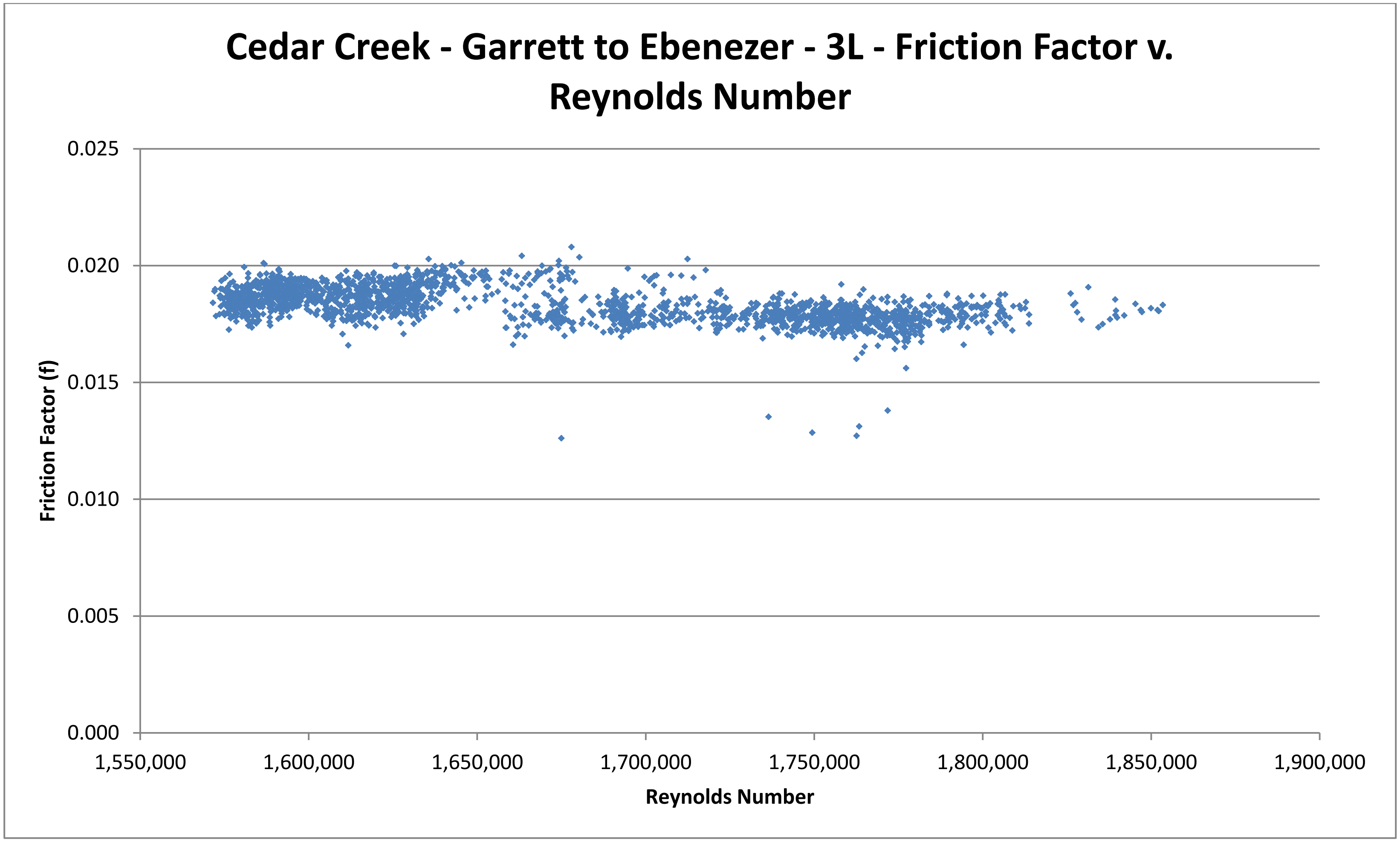
FM 1181 to Crisp Pike



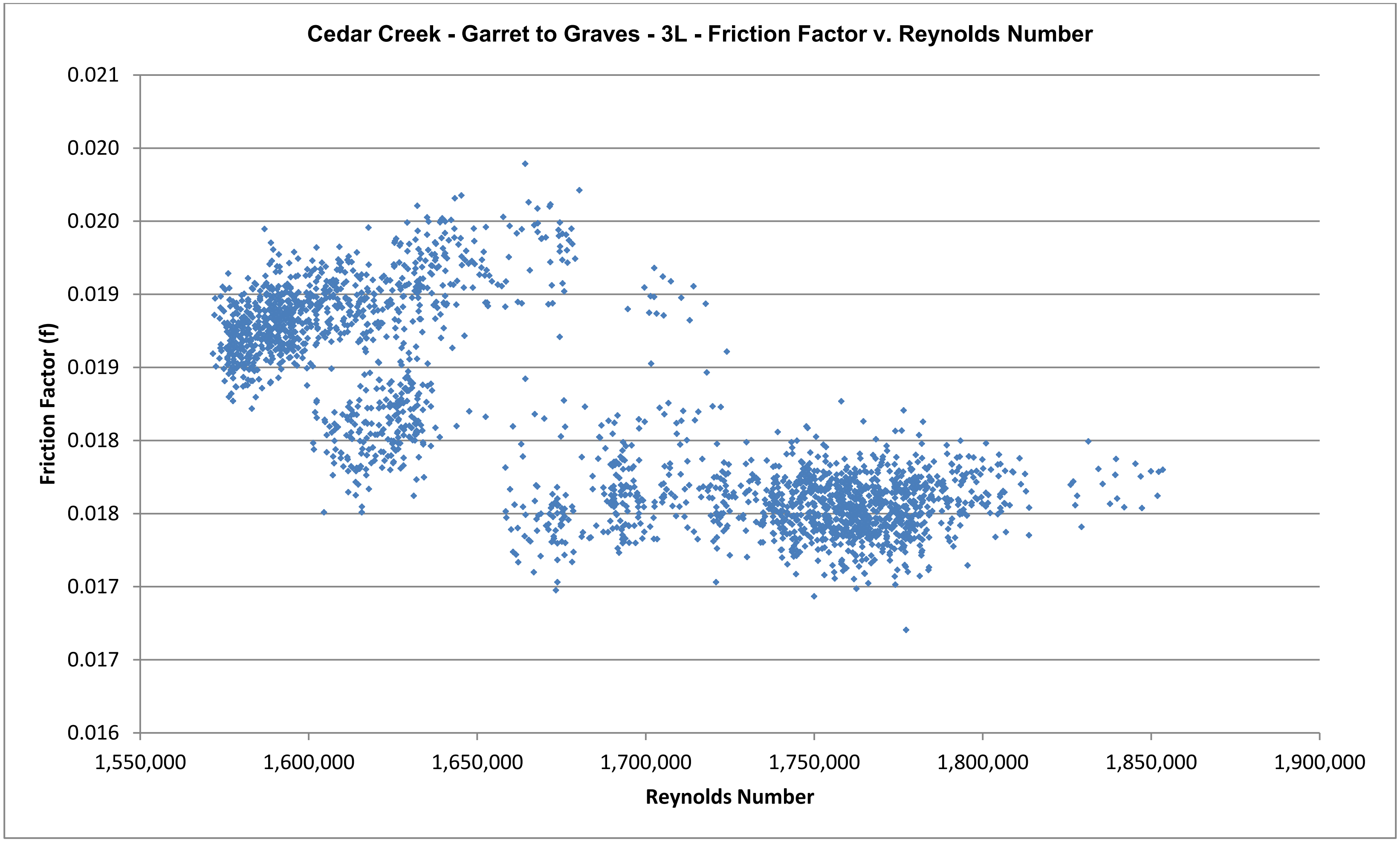
Crisp Pike to Garrett



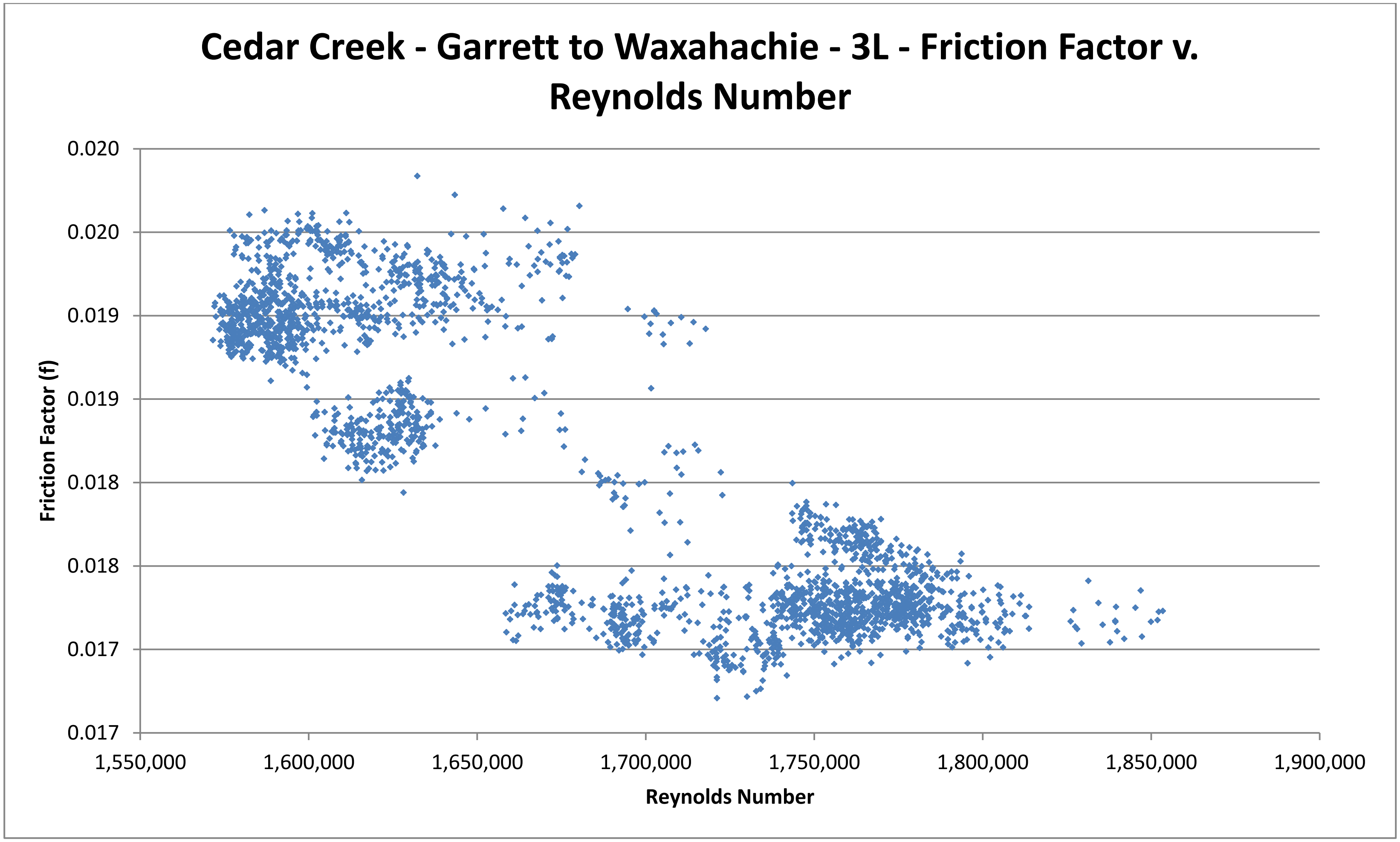
Garrett to Ebenezer



Garrett to Graves



Garrett to Waxahachie



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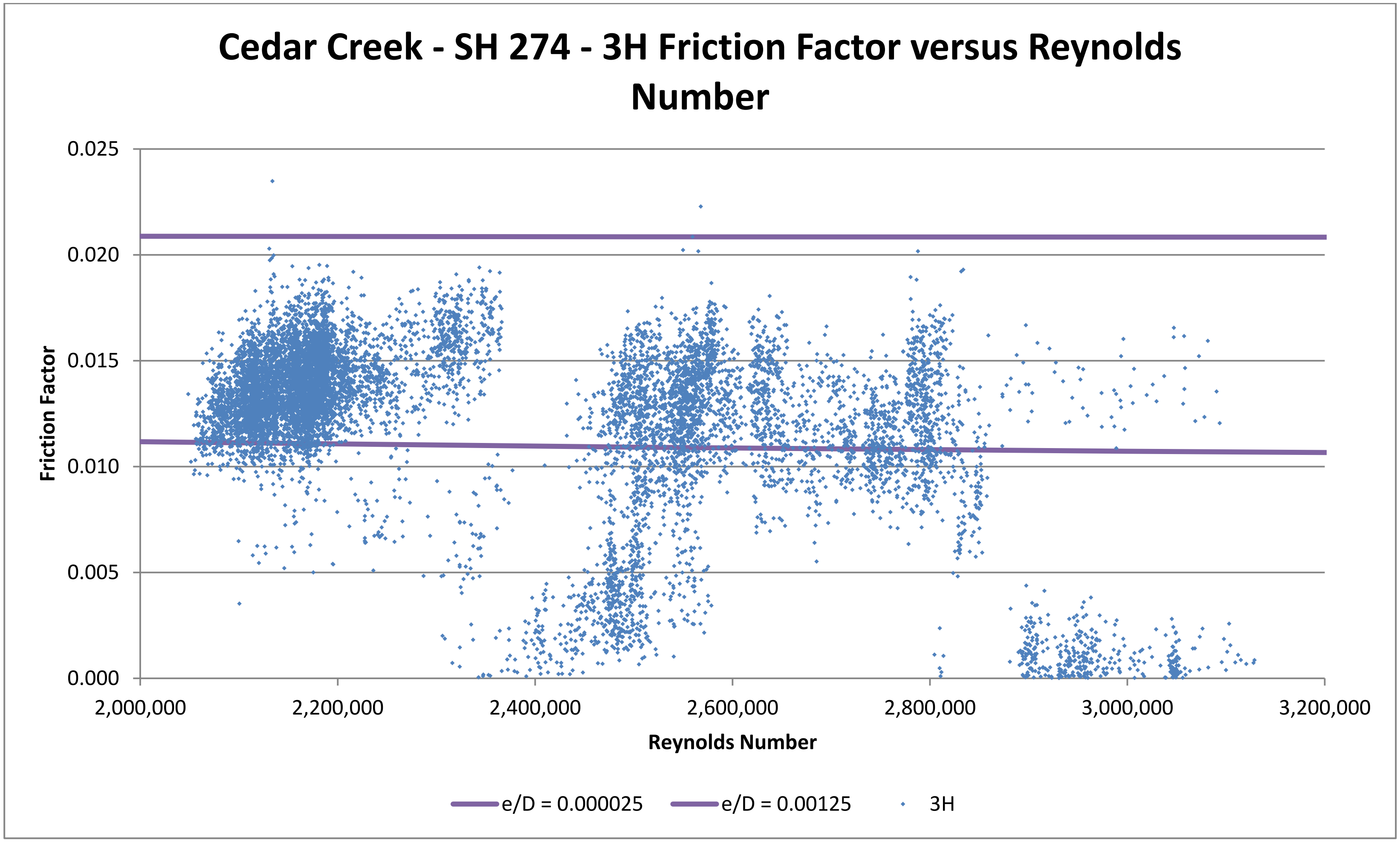
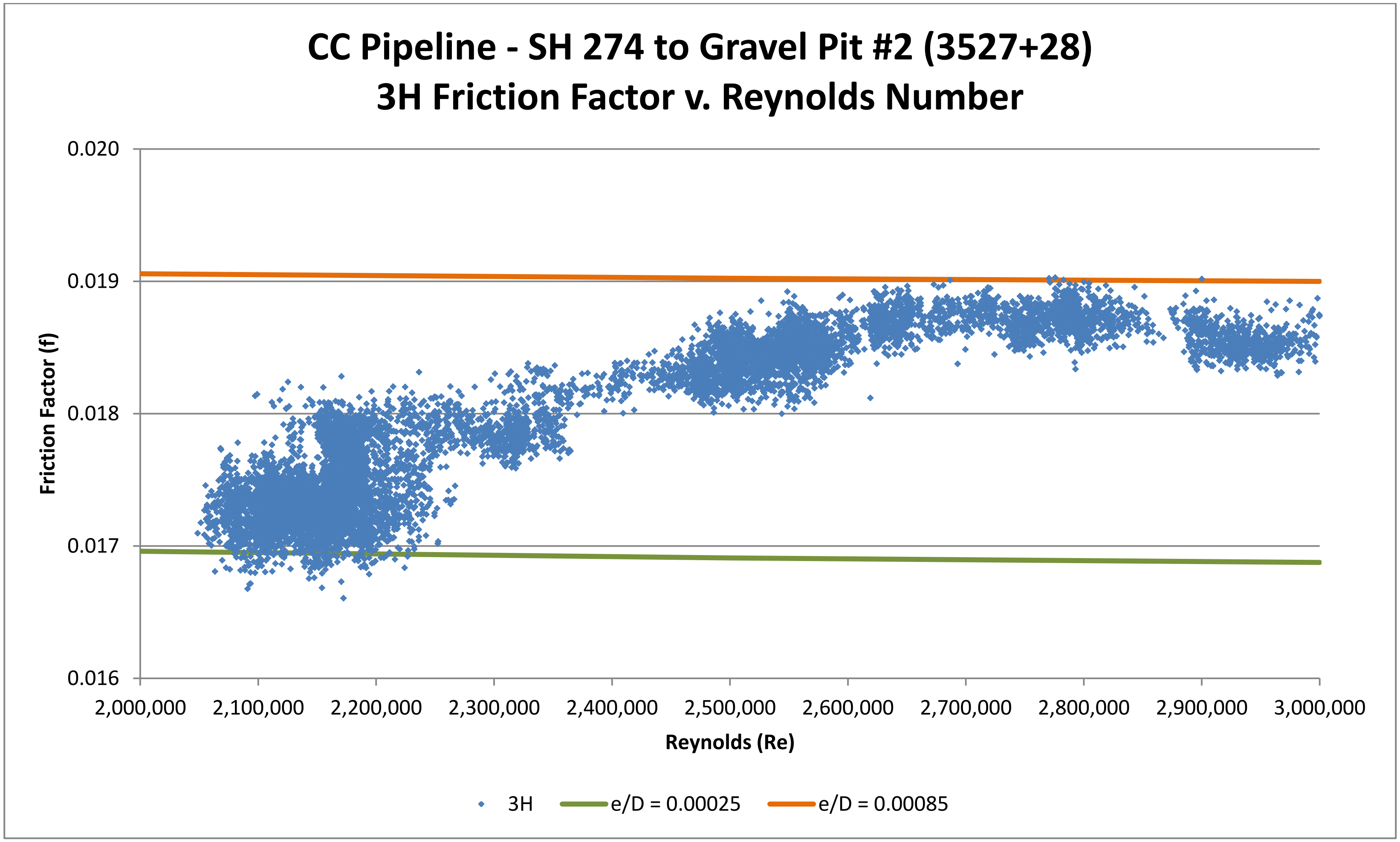
Cedar Creek 3H Pump Combination

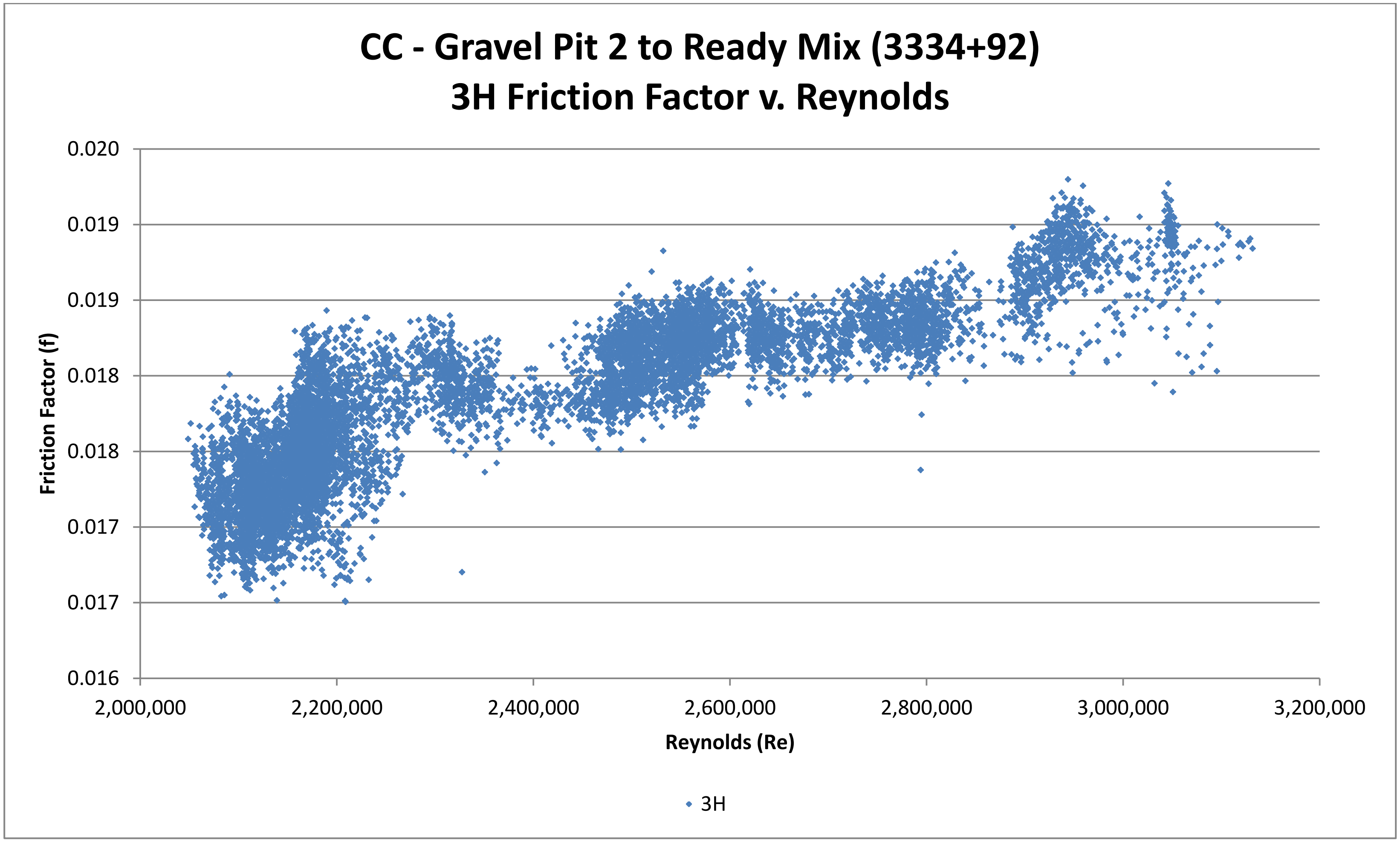
Cedar Creek Lake Pump Station to State Highway 271



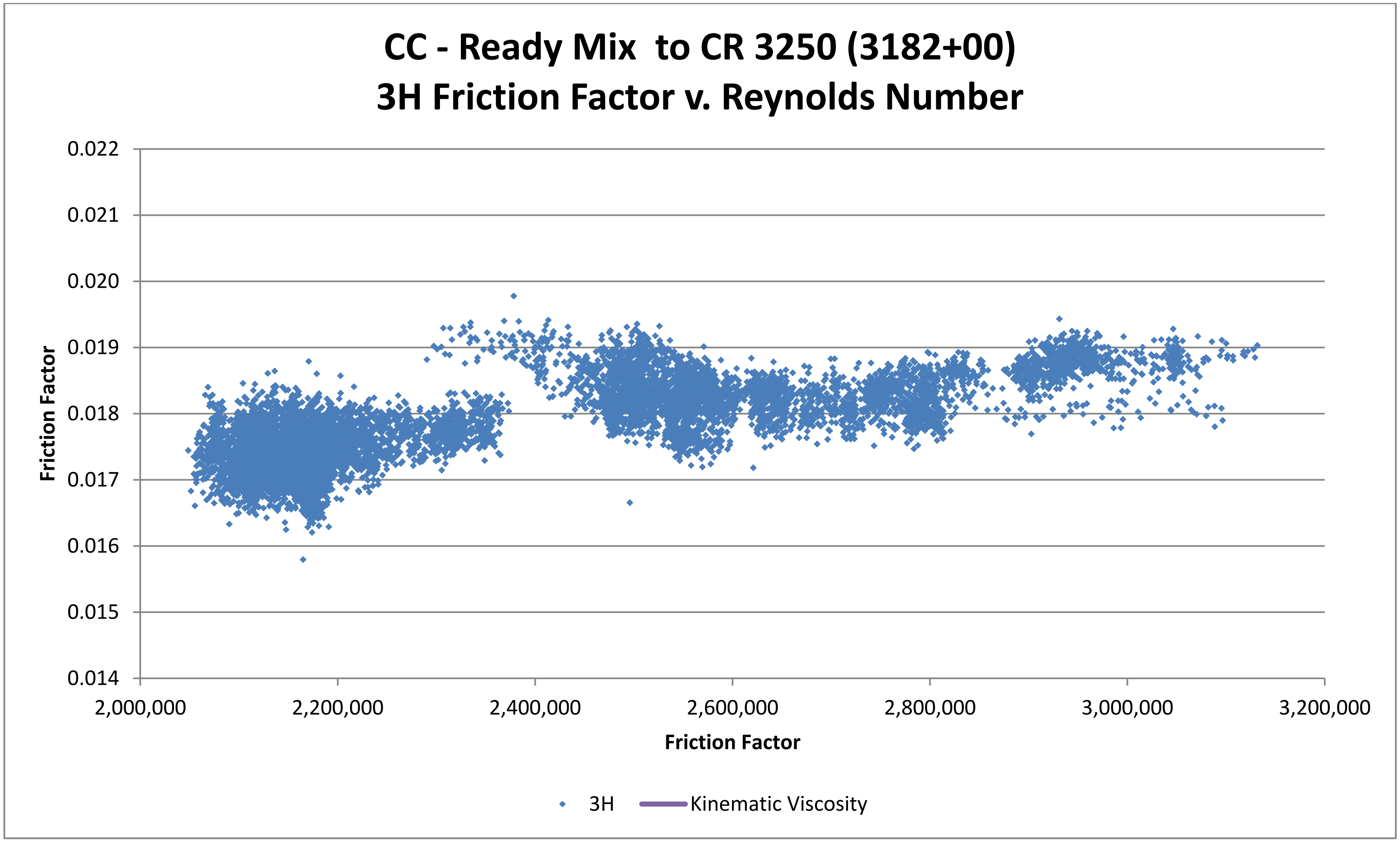
SH 274 to Gravel Pit #2



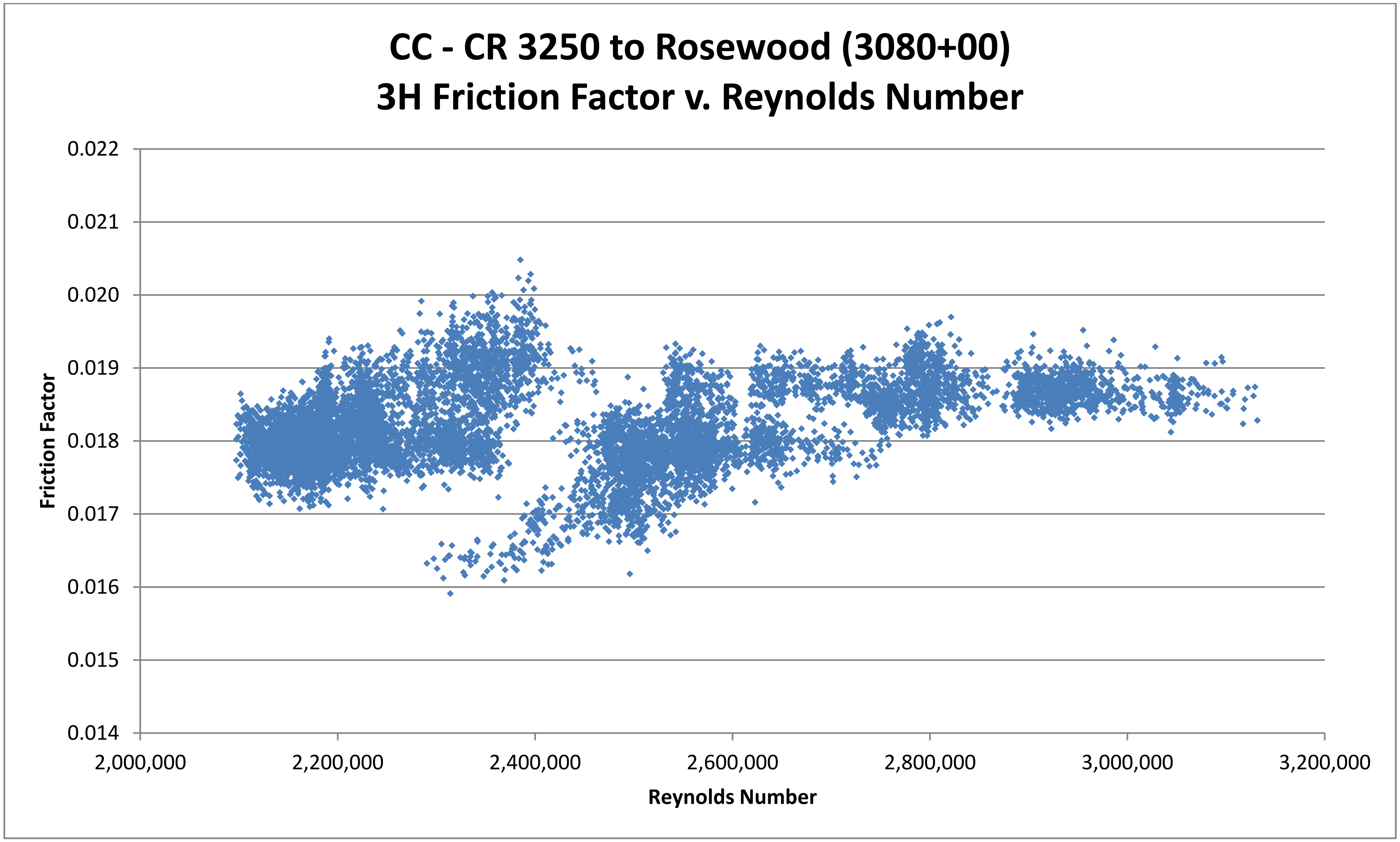
Gravel Pit #2 to Ready Mix



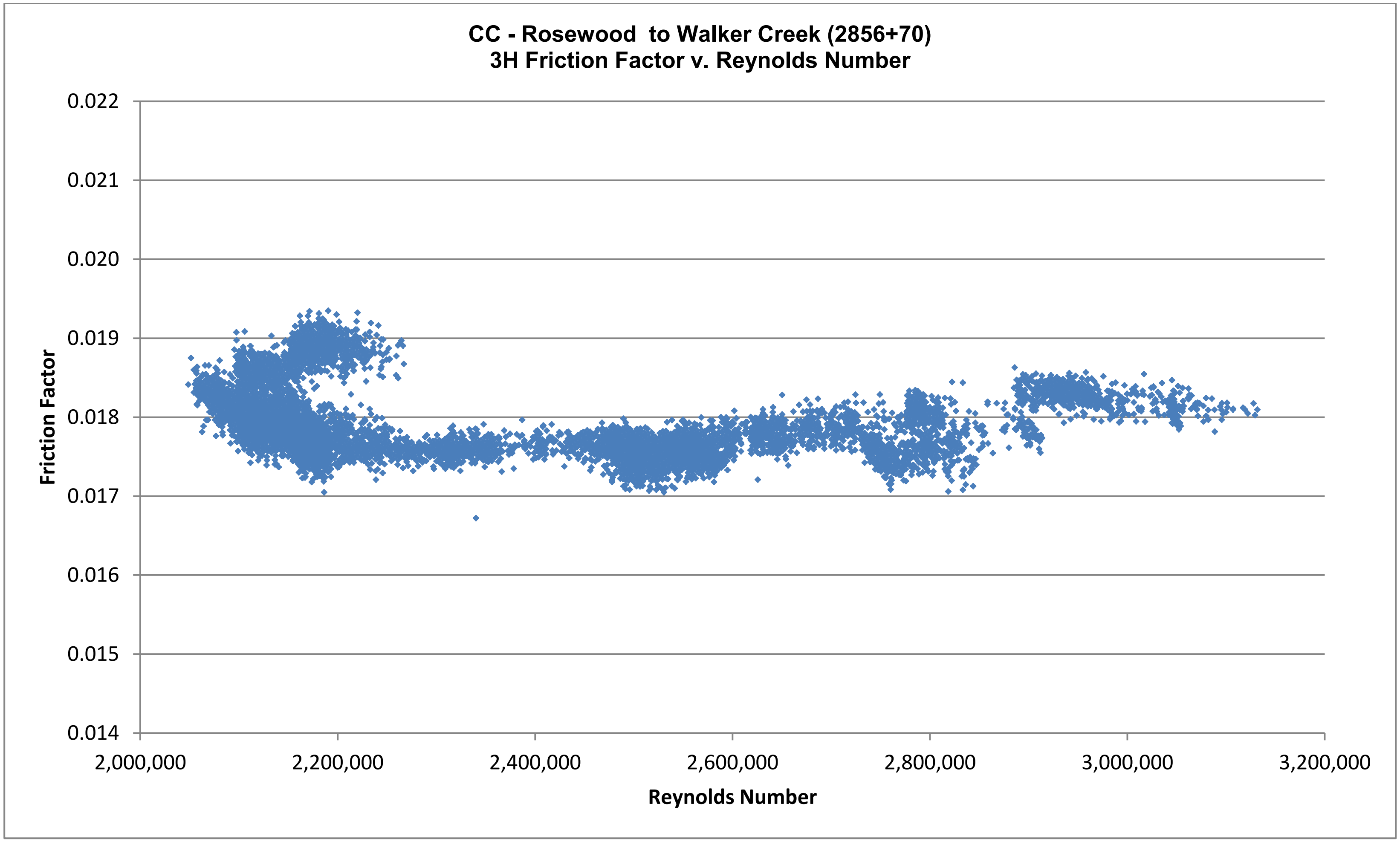
Ready Mix to CR 3250



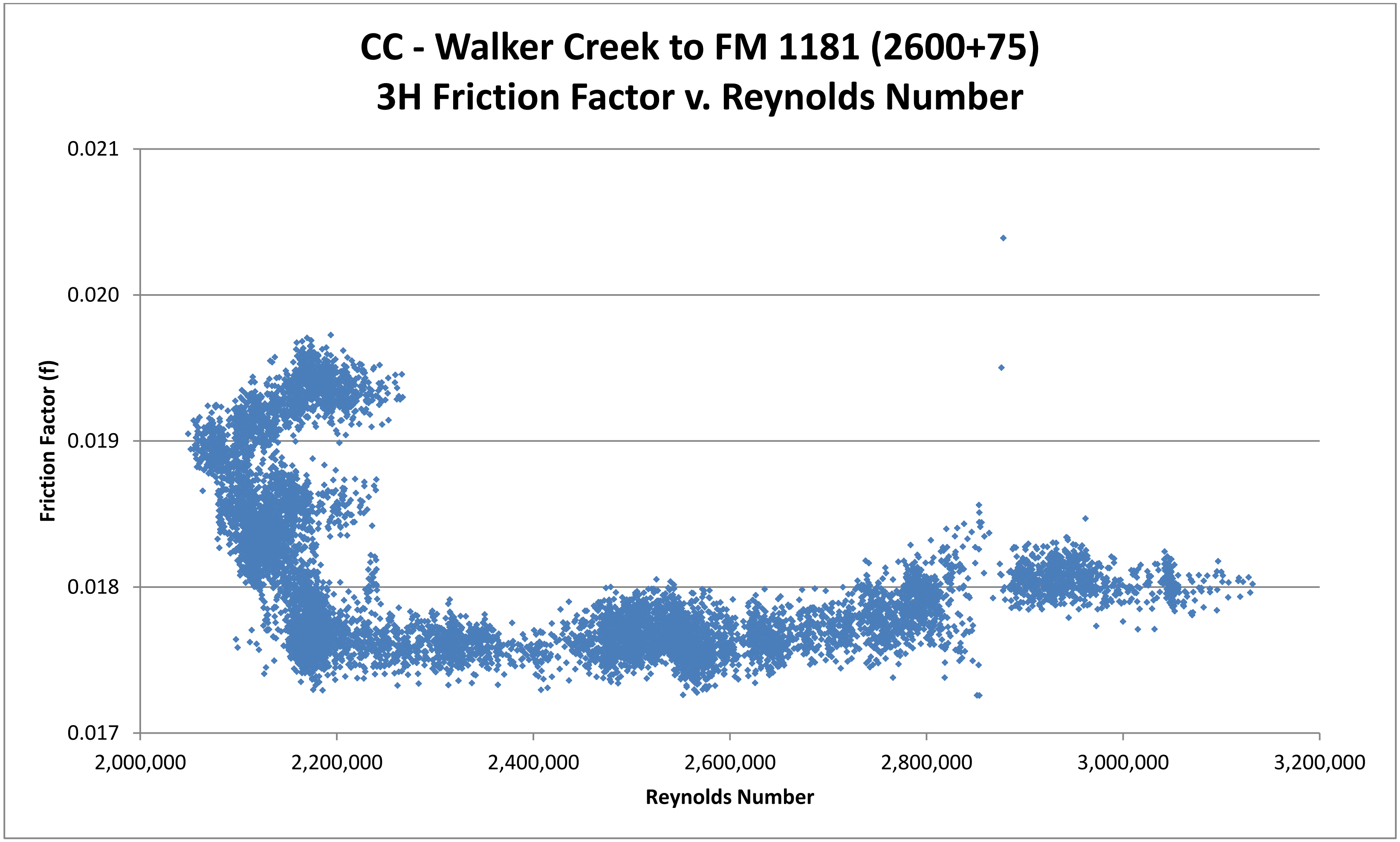
CR 3250 to Rosewood



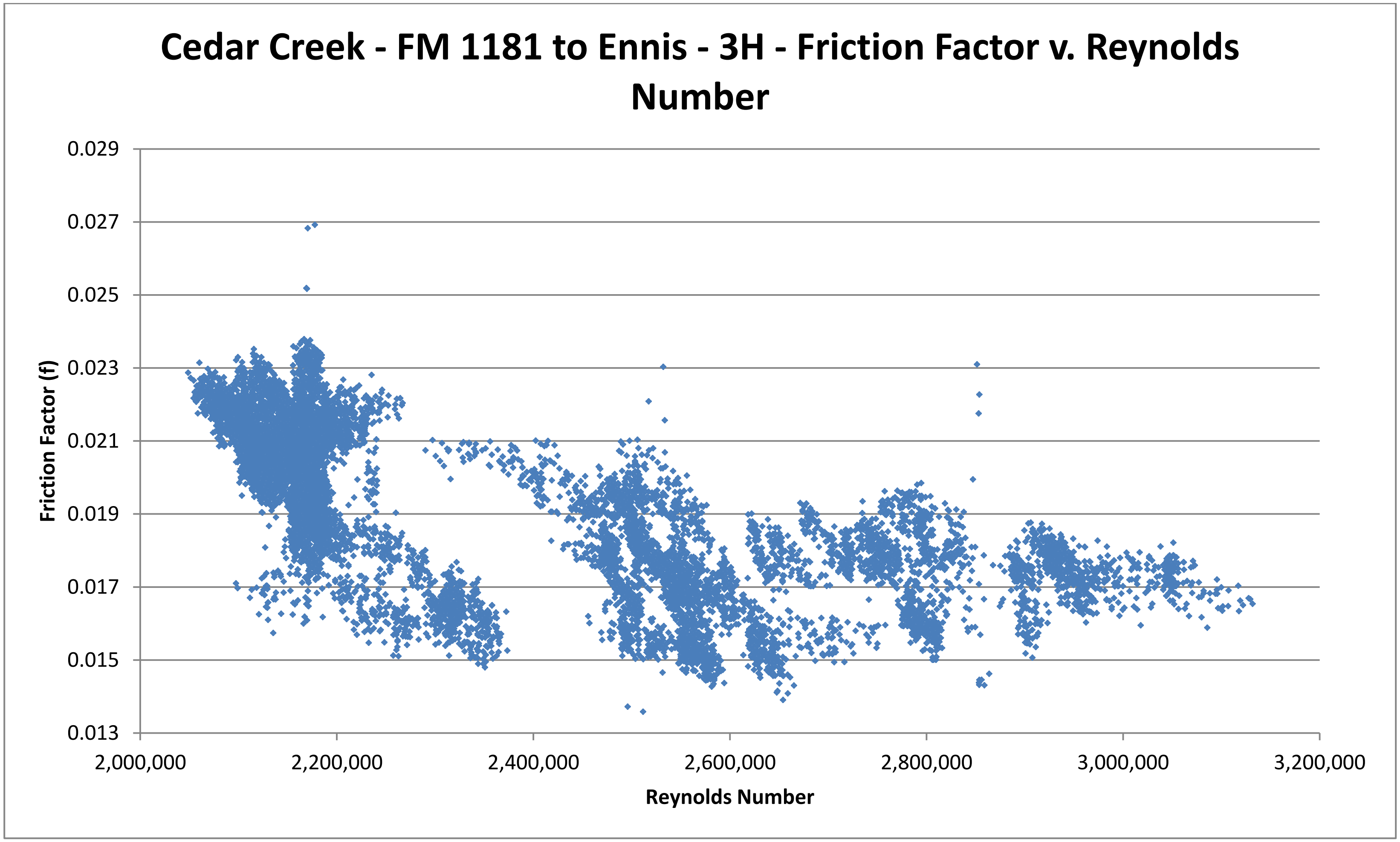
Rosewood to Walker Creek



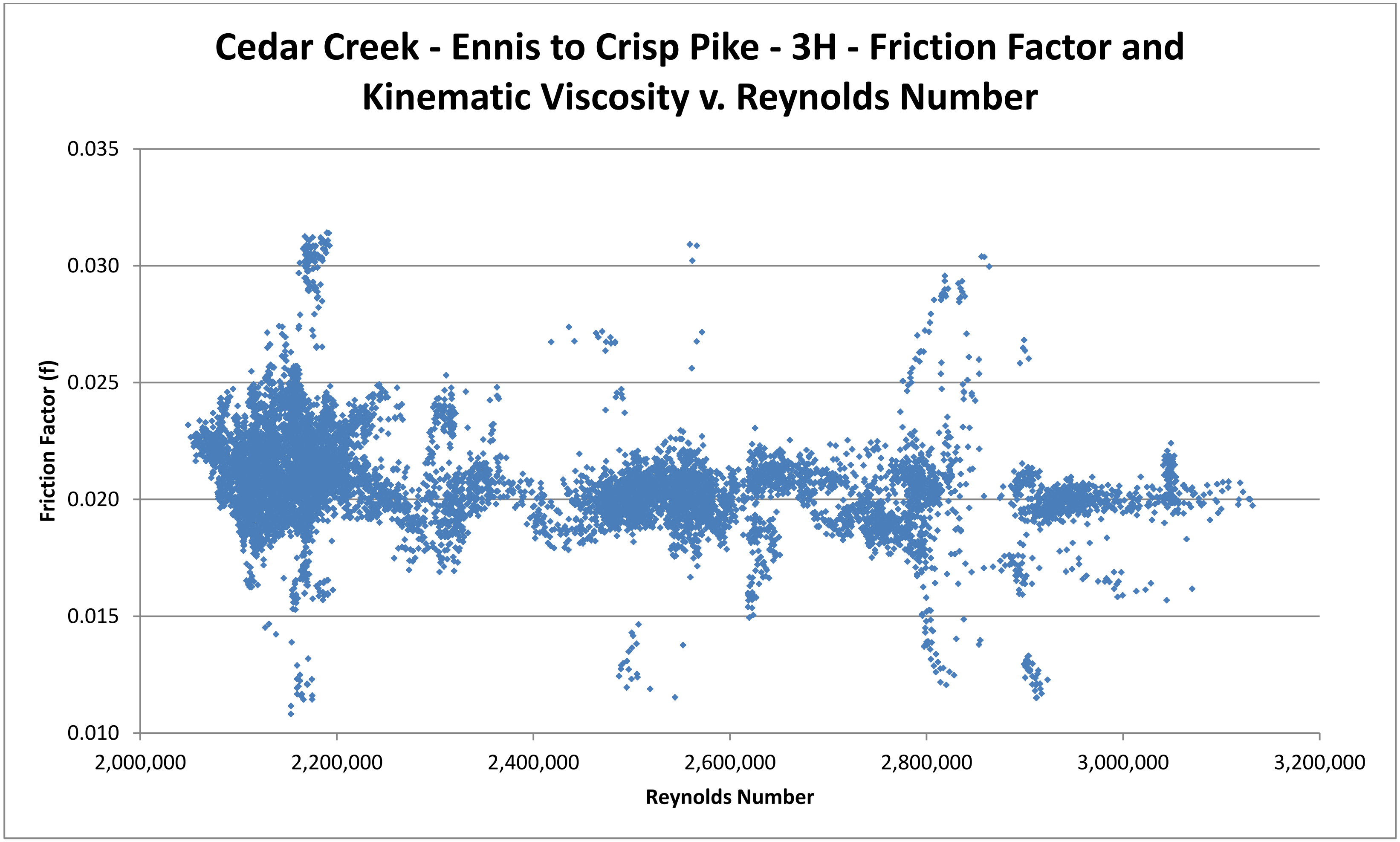
Walker Creek to FM 1181



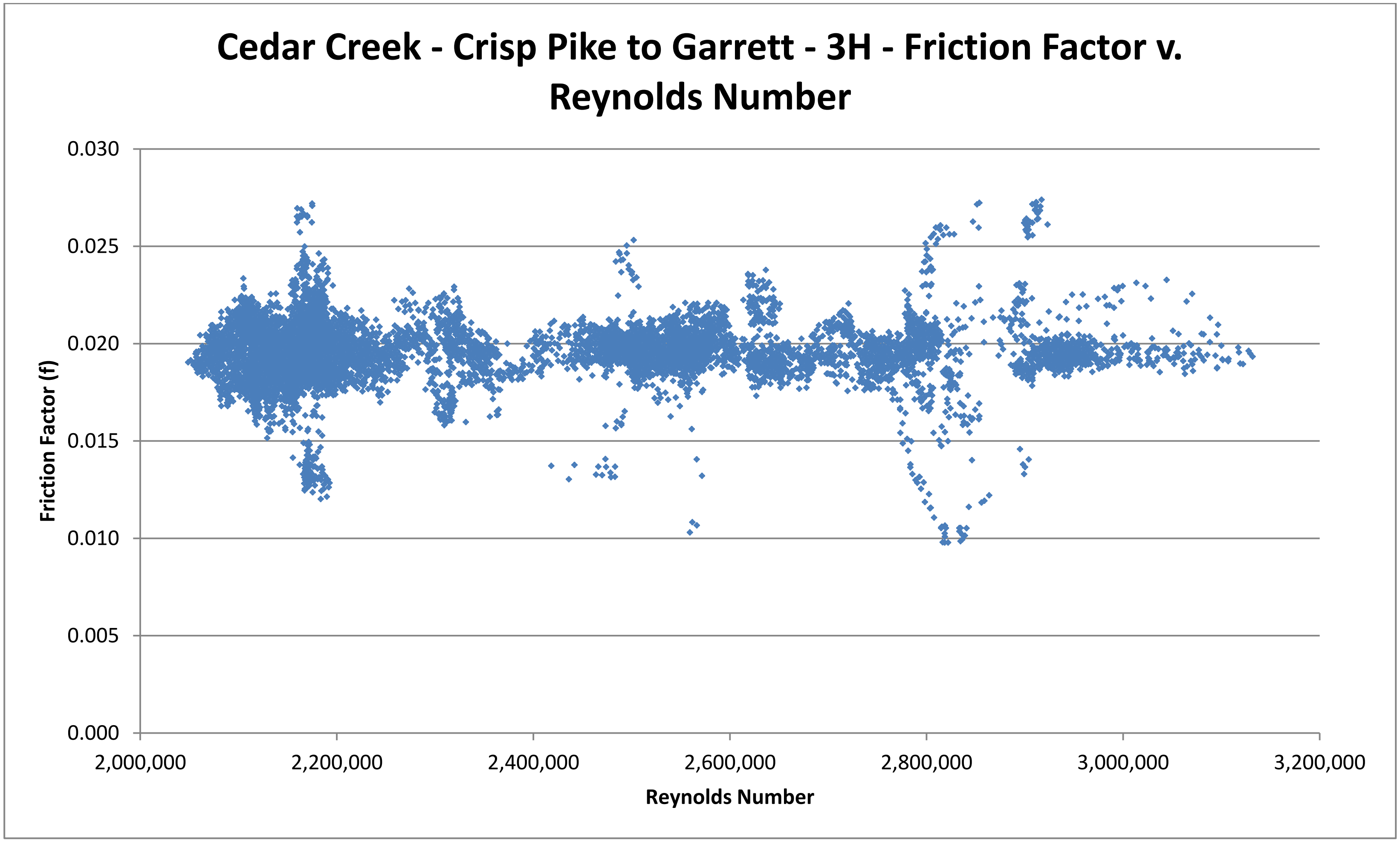
FM 1181 to Ennis



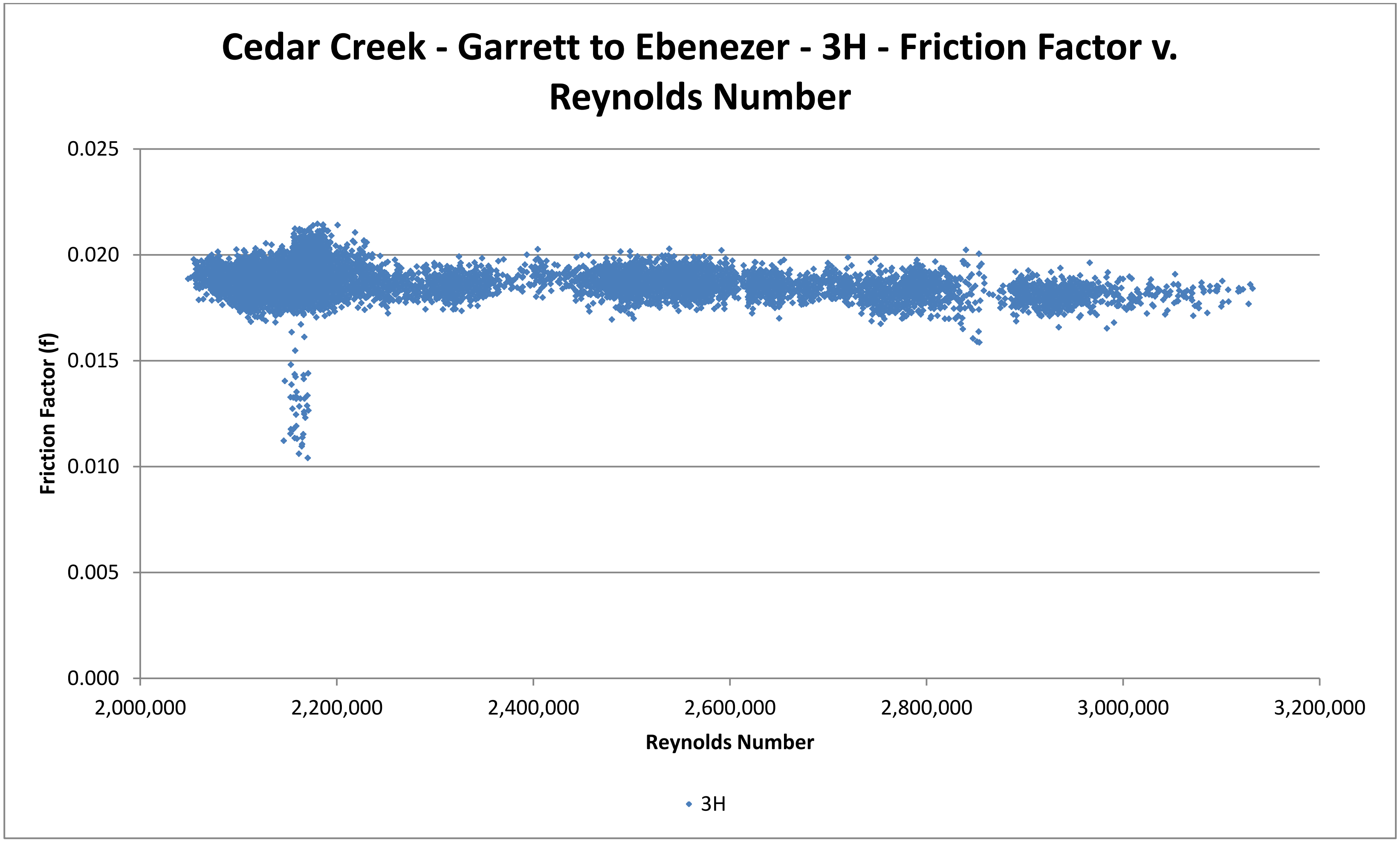
Ennis to Crisp Pike



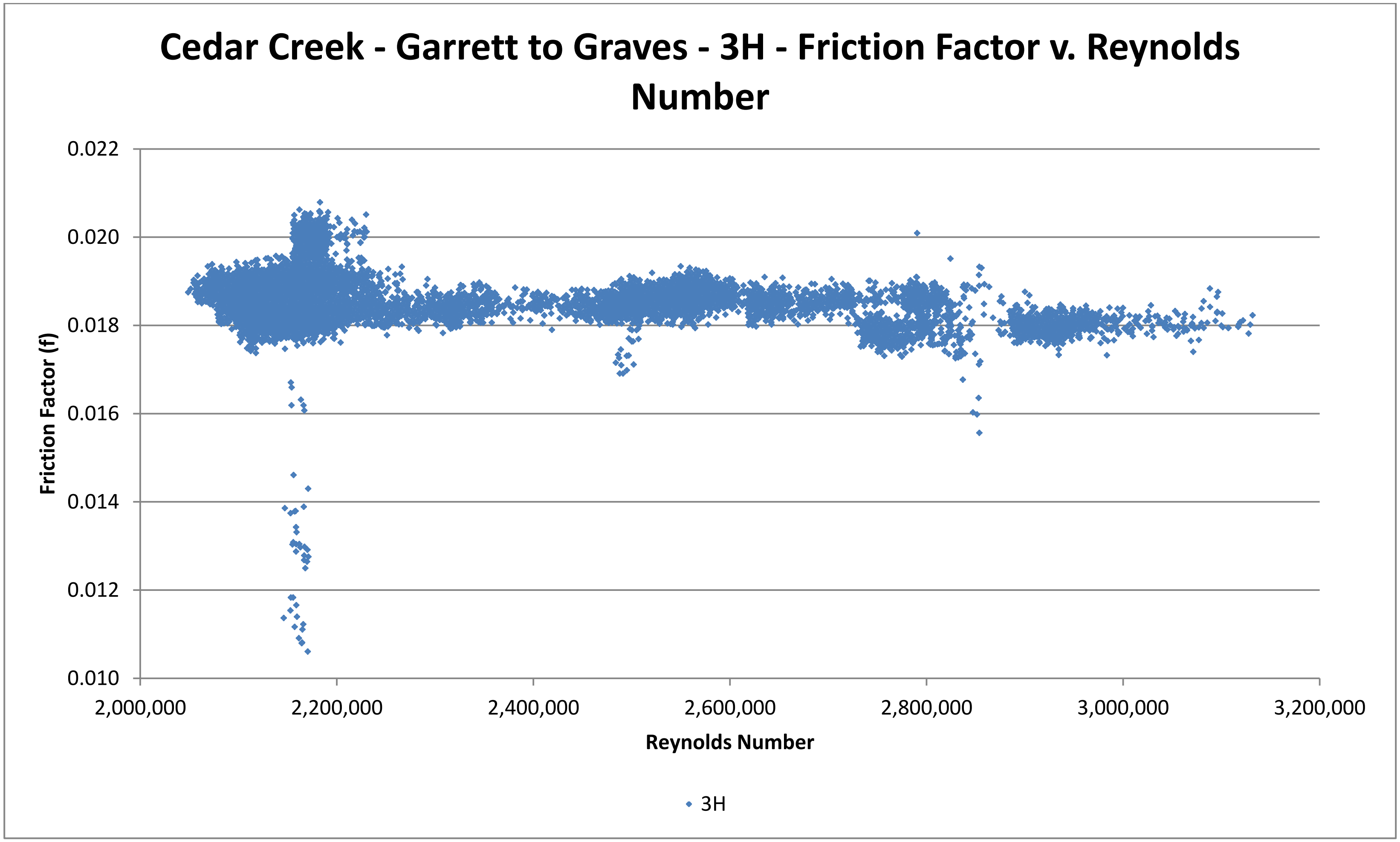
Crisp Pike to Garrett



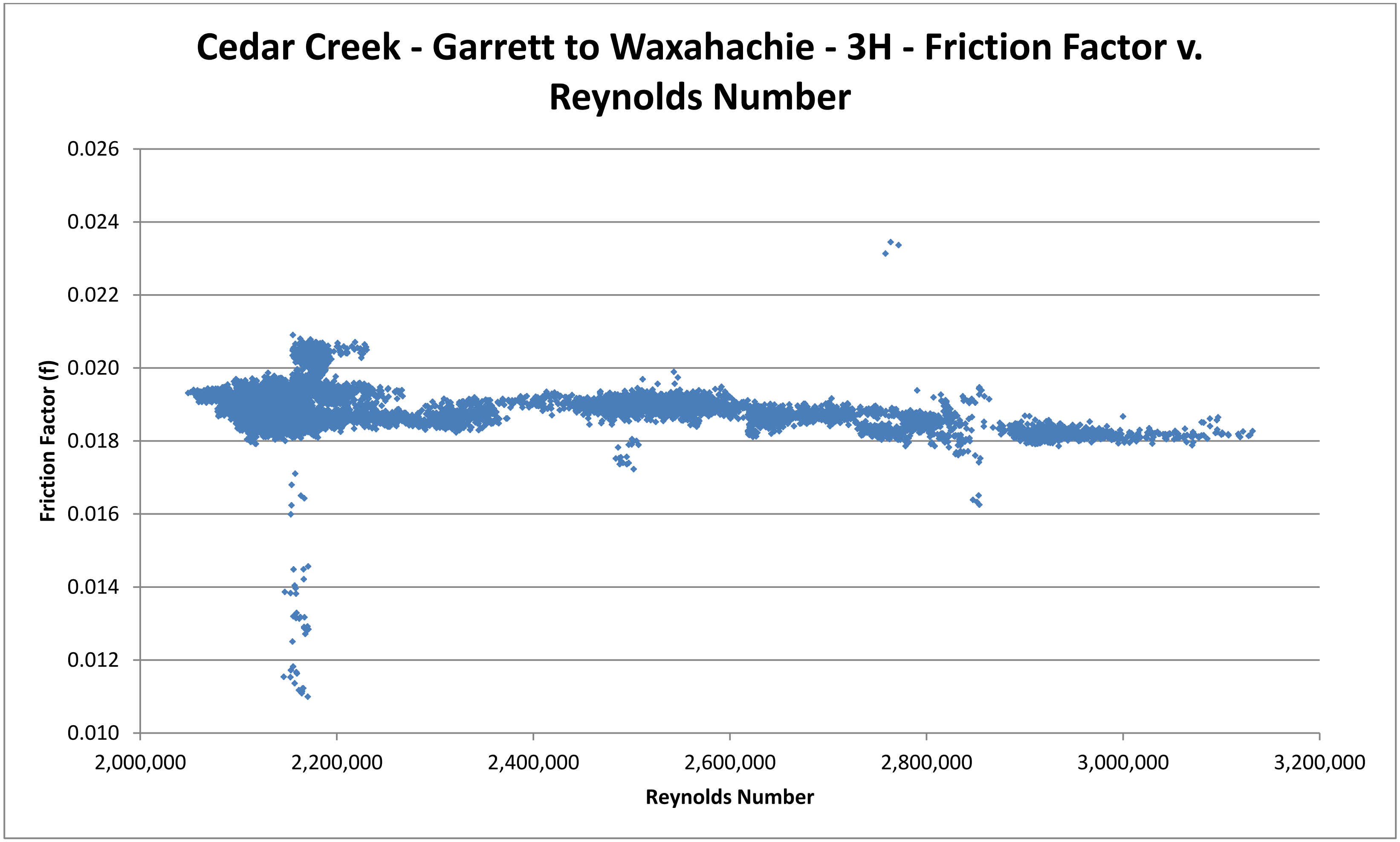
Garrett to Ebenezer



Garrett to Graves



Garrett to Waxahachie



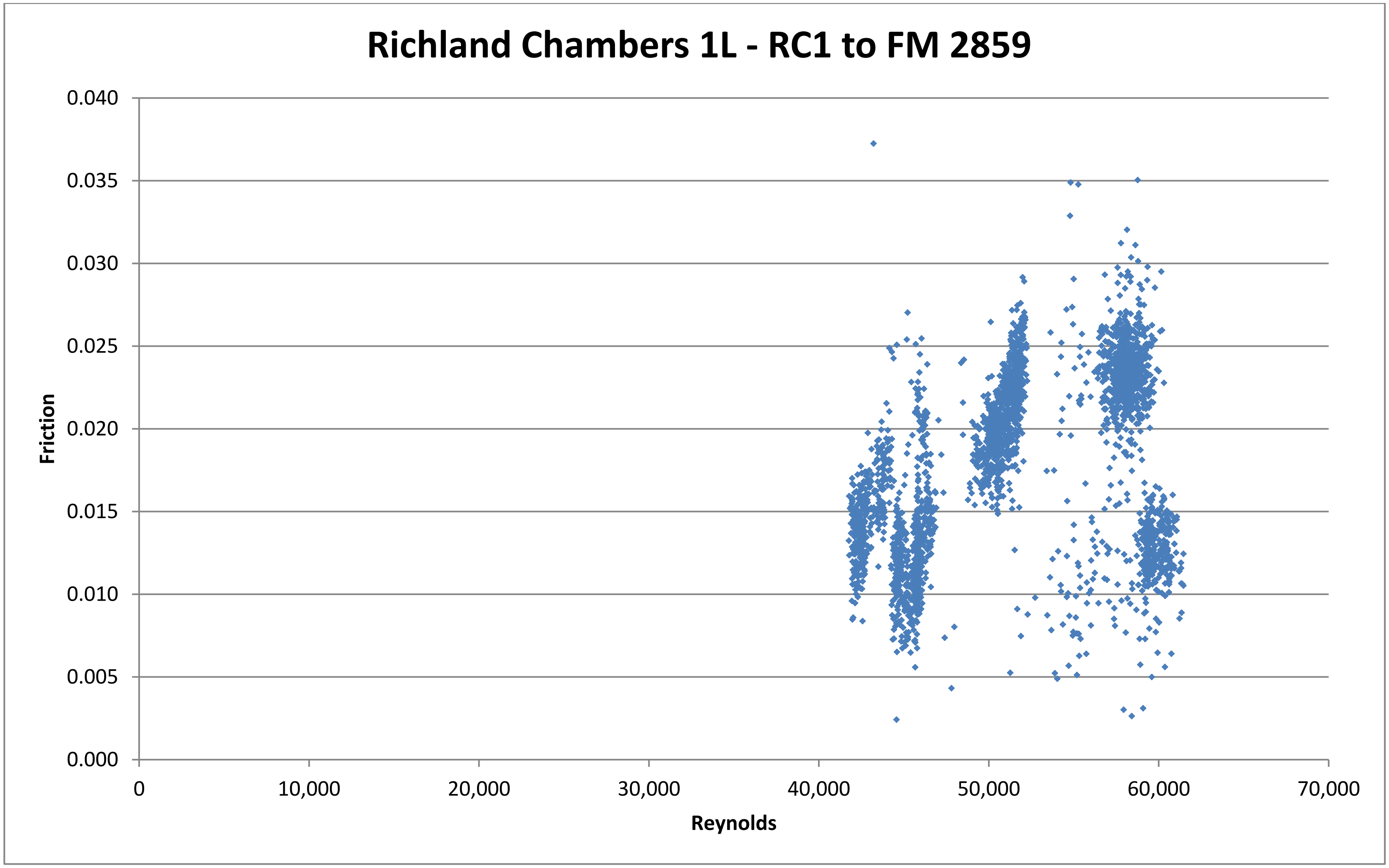
APPENDIX D

RICHLAND CHAMBERS FIELD STUDY

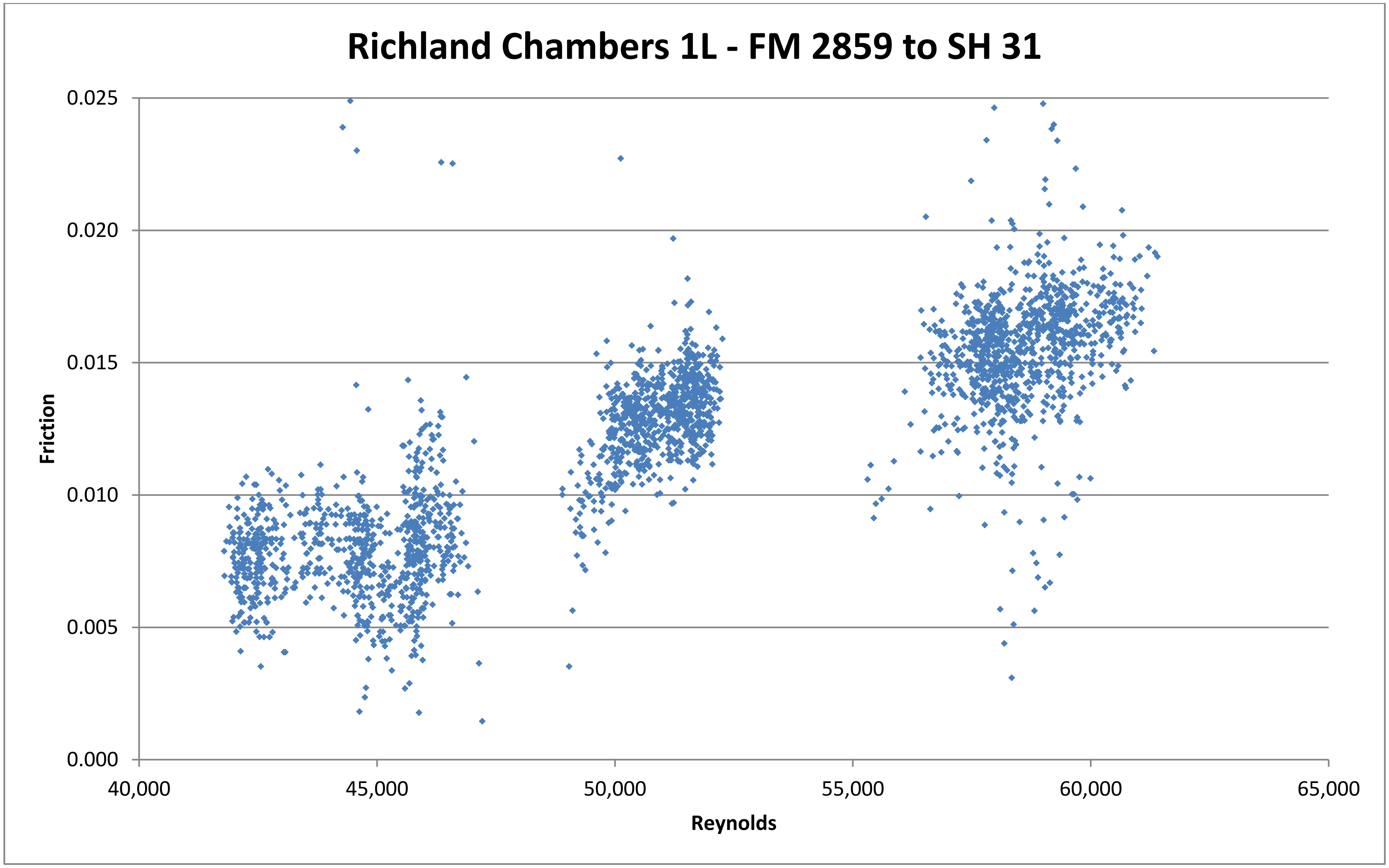
FRICTION FACTOR AND MOODY DIAGRAM RESULTS

Richland Chambers 1L Pump Combination

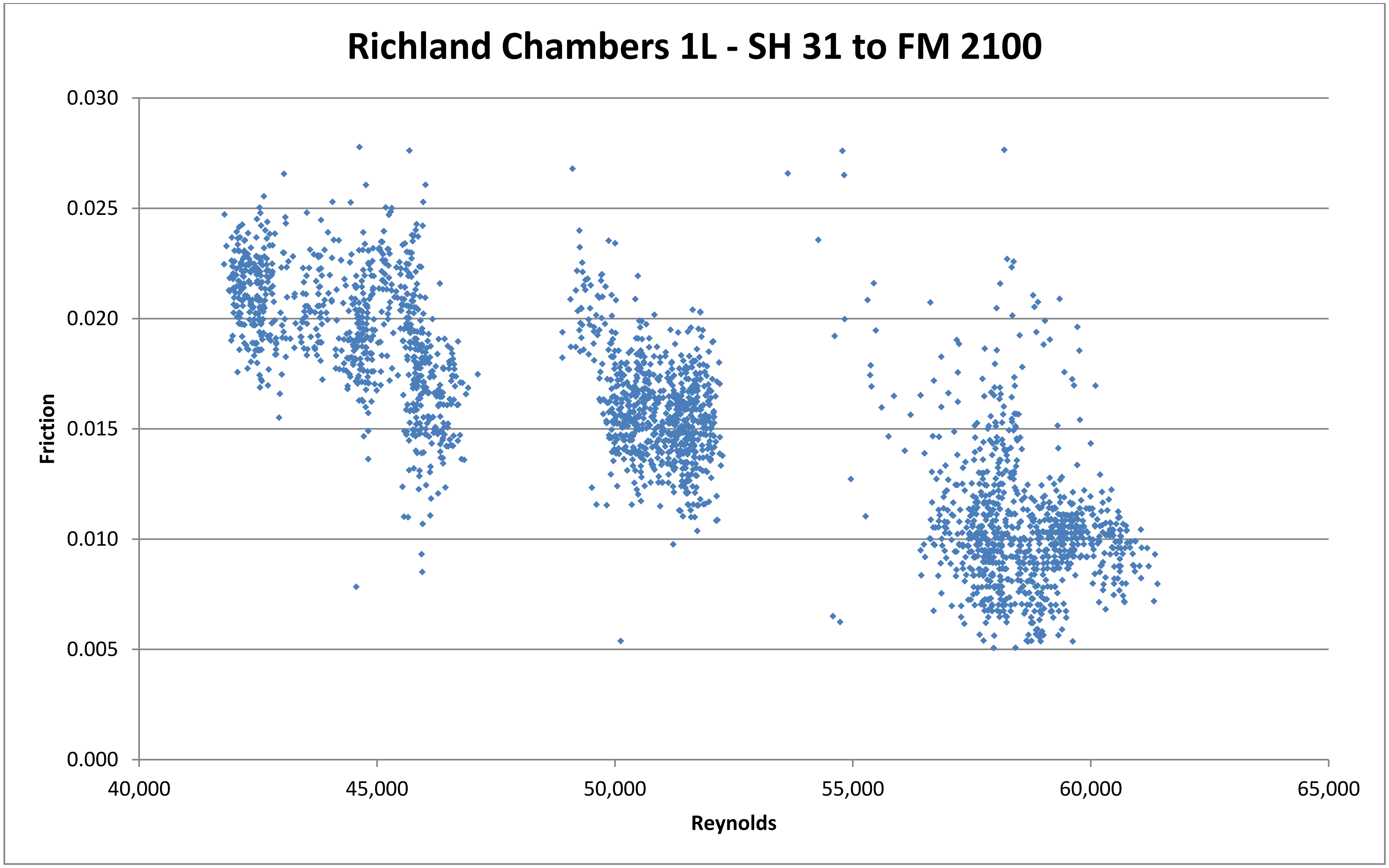
Richland Chambers Lake Pump Station to Farm-to-Market 2859



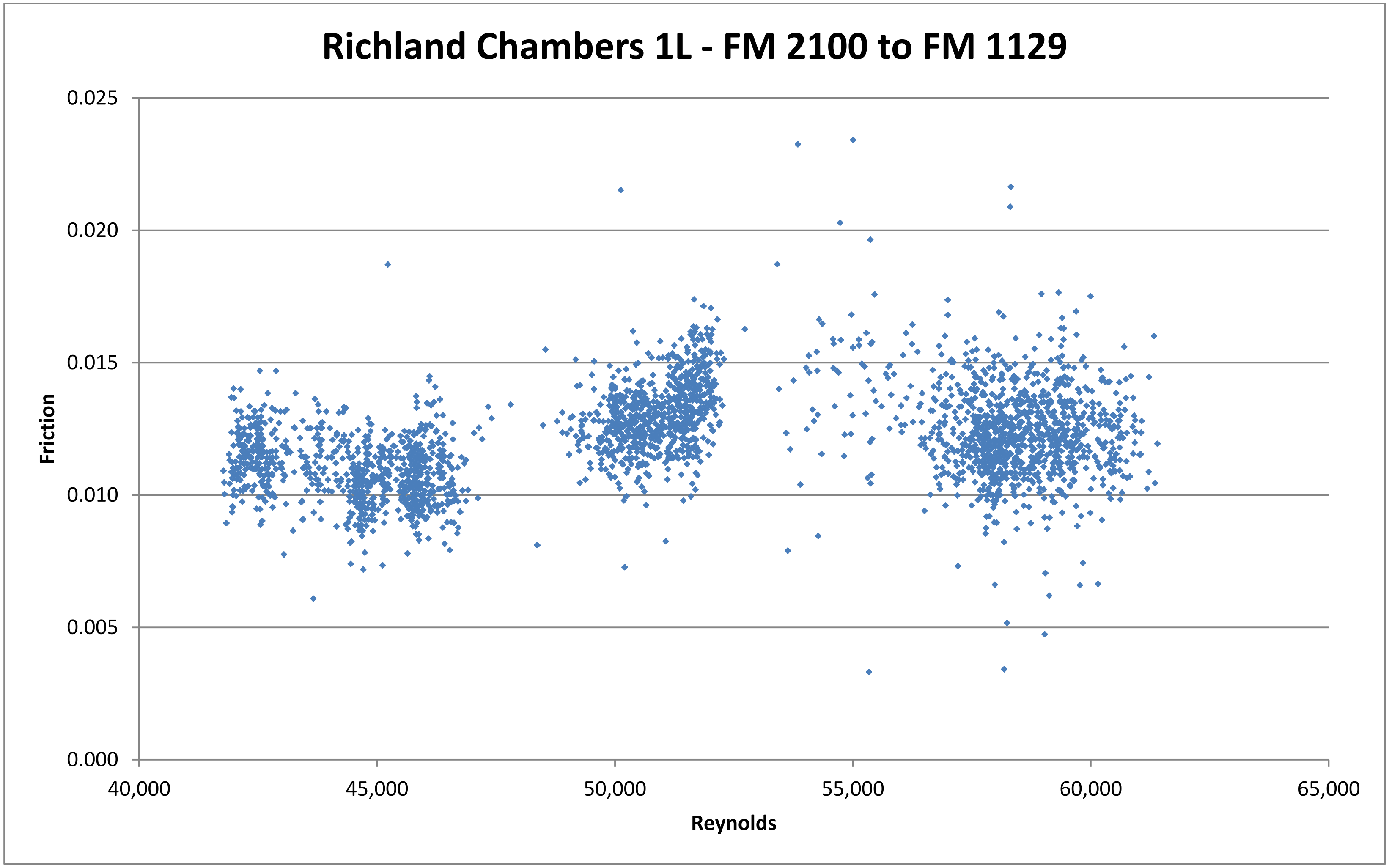
Farm-to-Market 2859 to State Highway 31



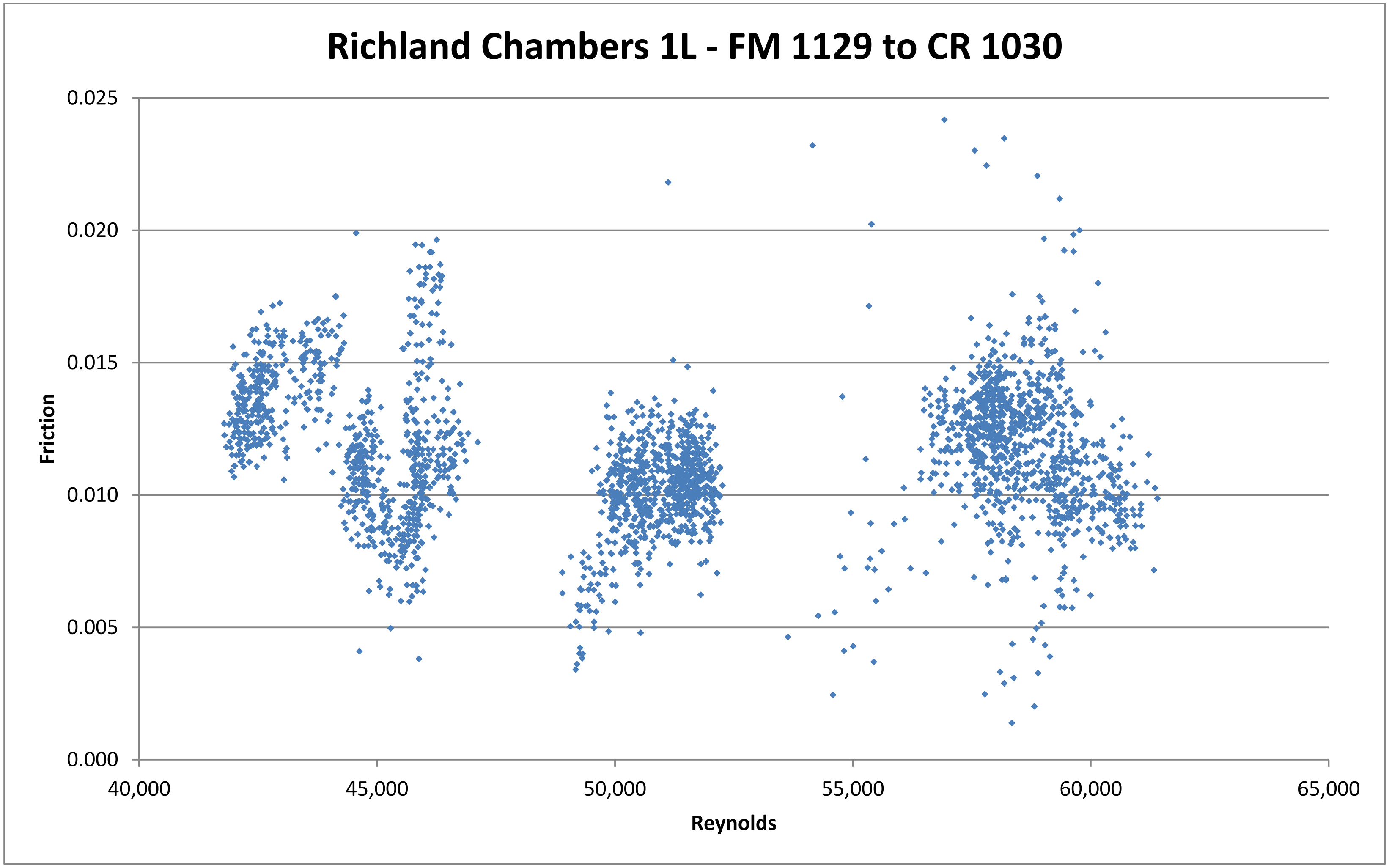
State Highway 31 to Farm-to-Market 2100



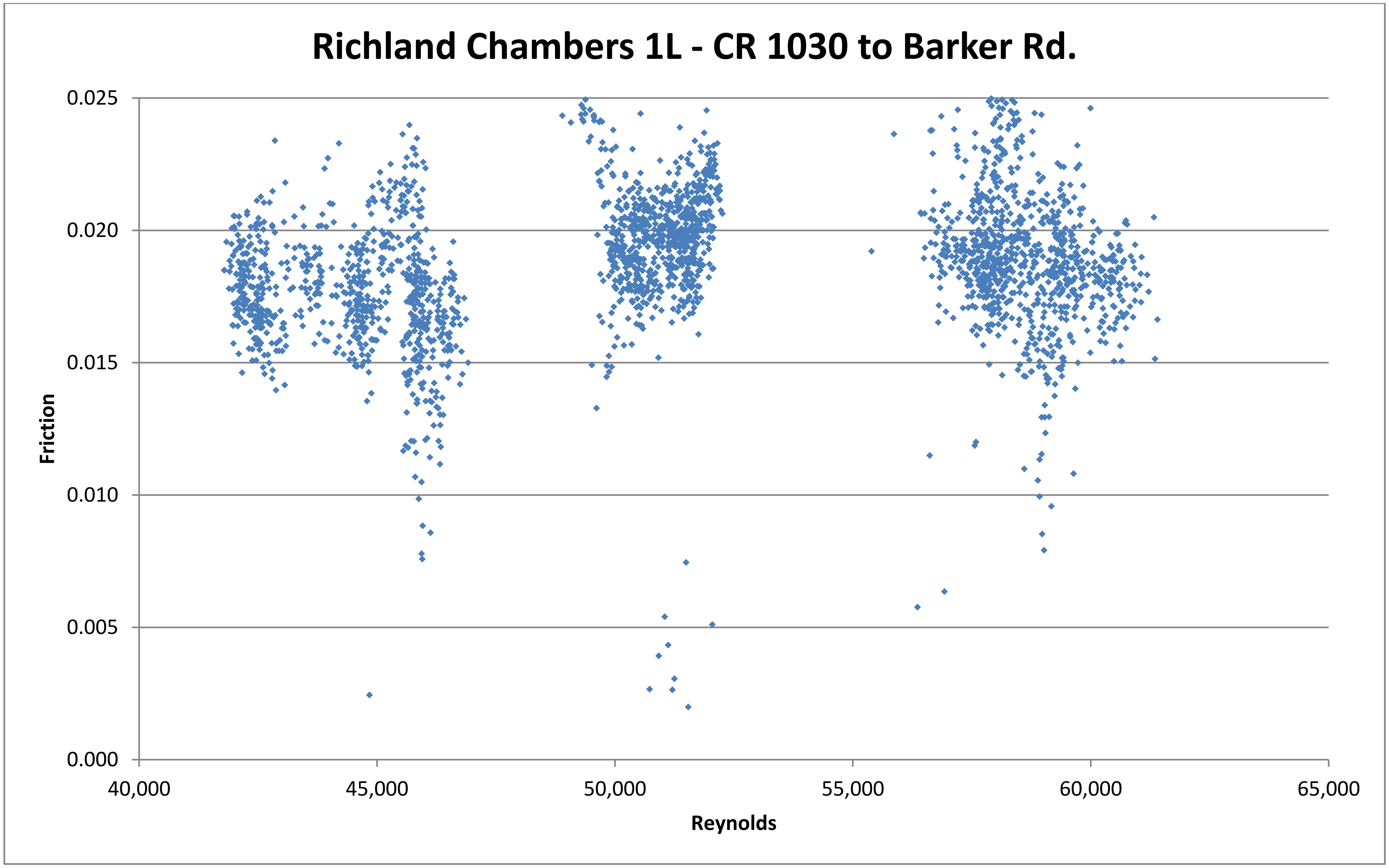
Farm-to-Market 2100 to Farm-to-Market 1129



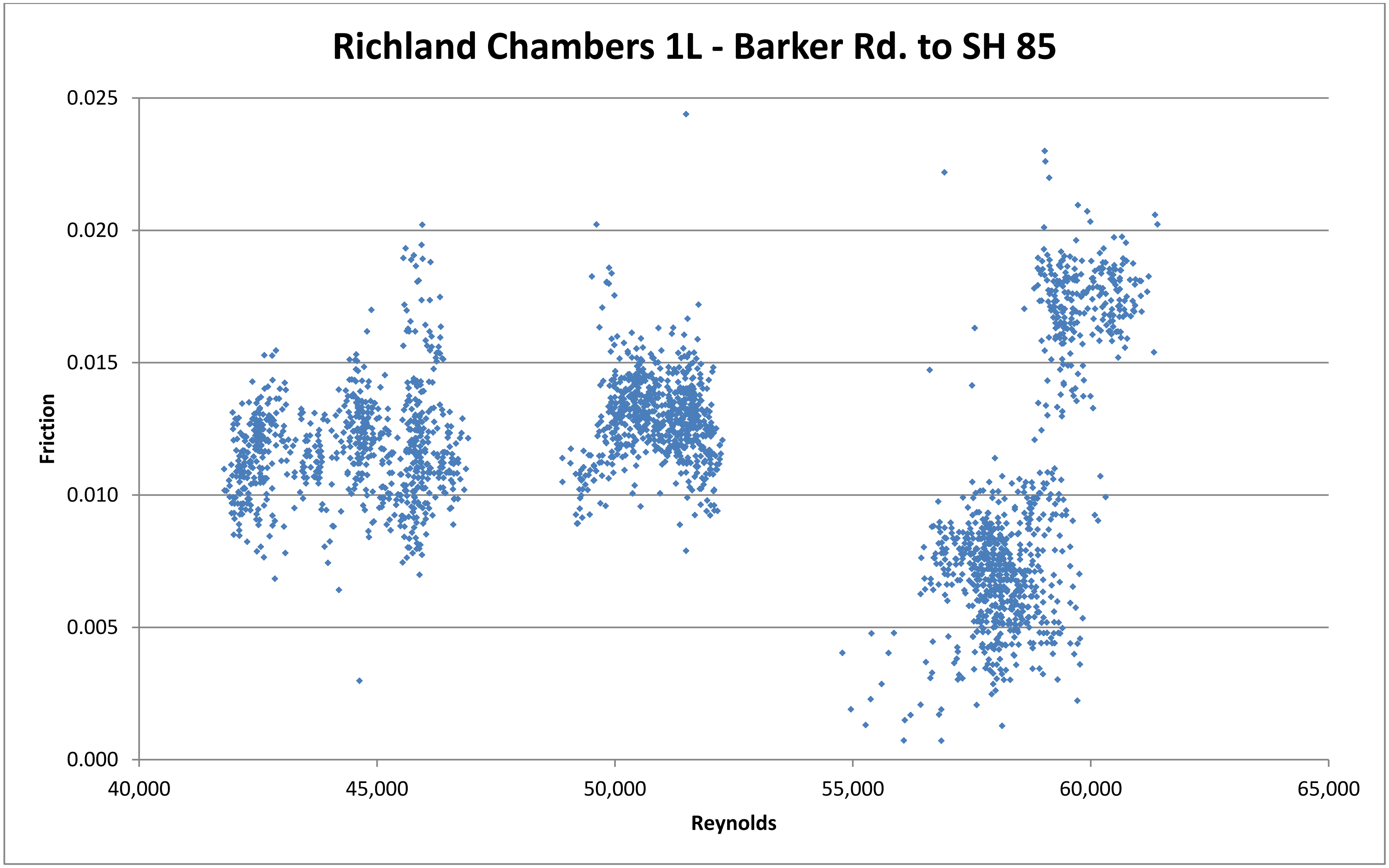
Farm-to-Market 1129 to County Road 1030



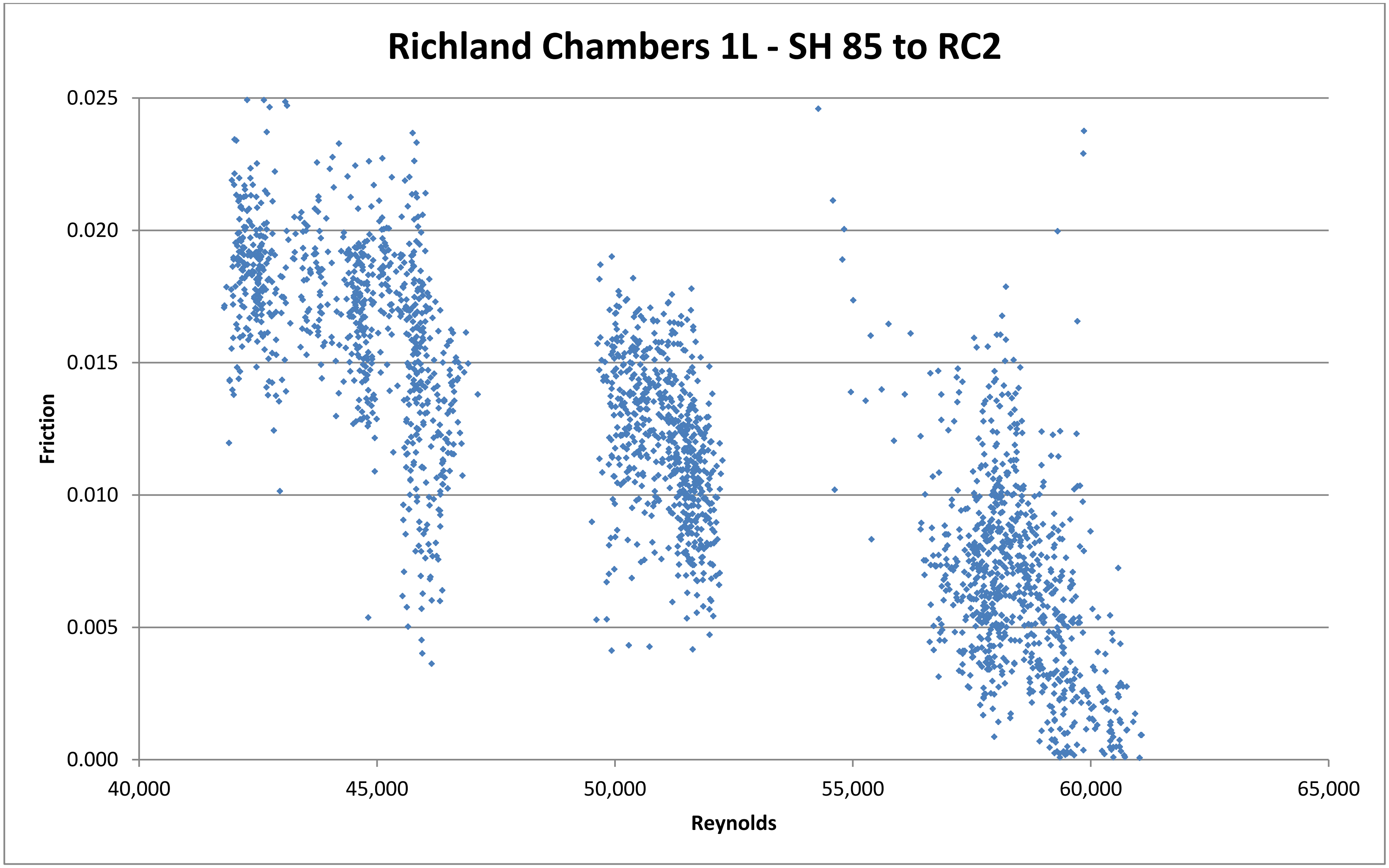
County Road 1030 to Barker Road



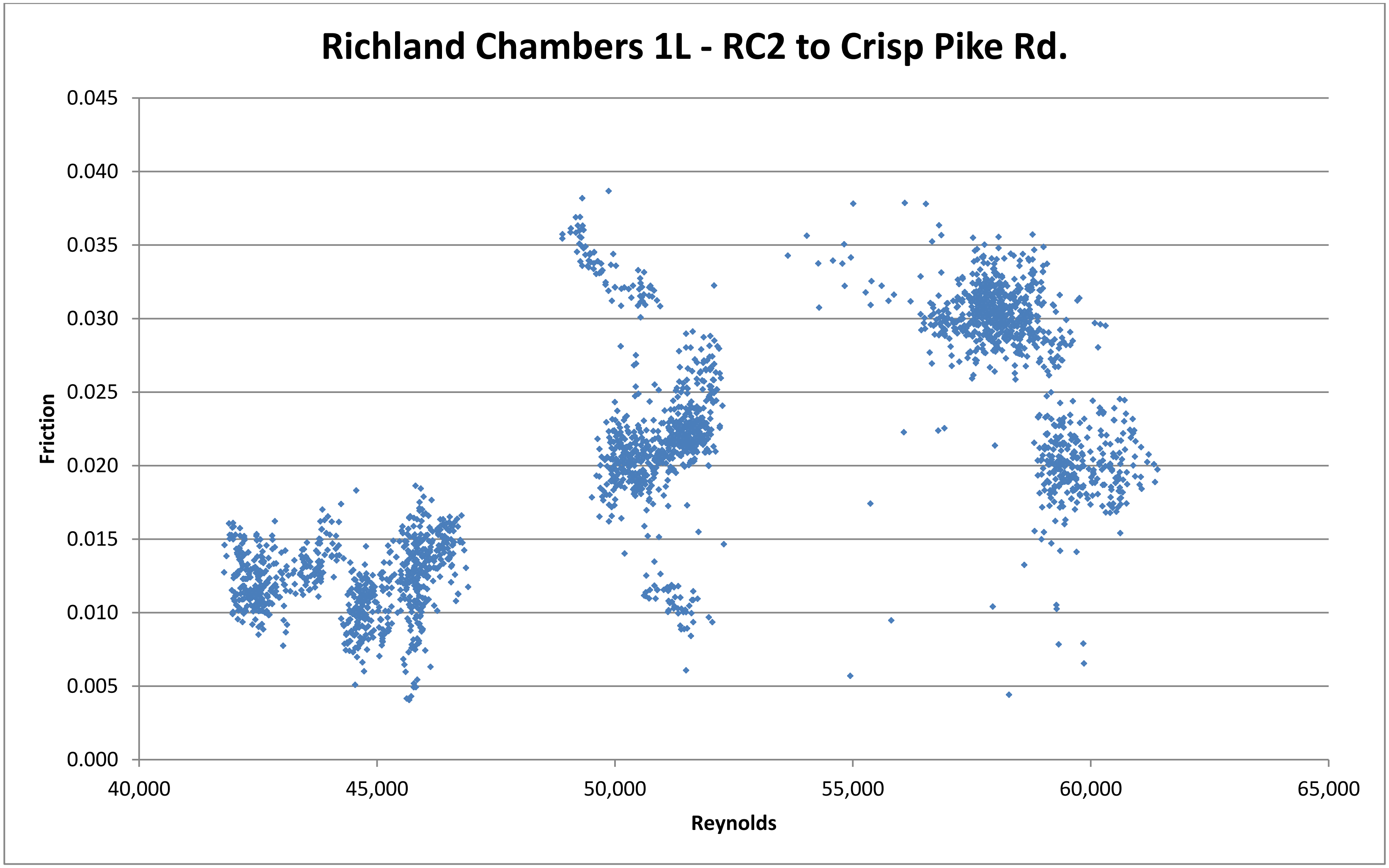
Barker Road to State Highway 85



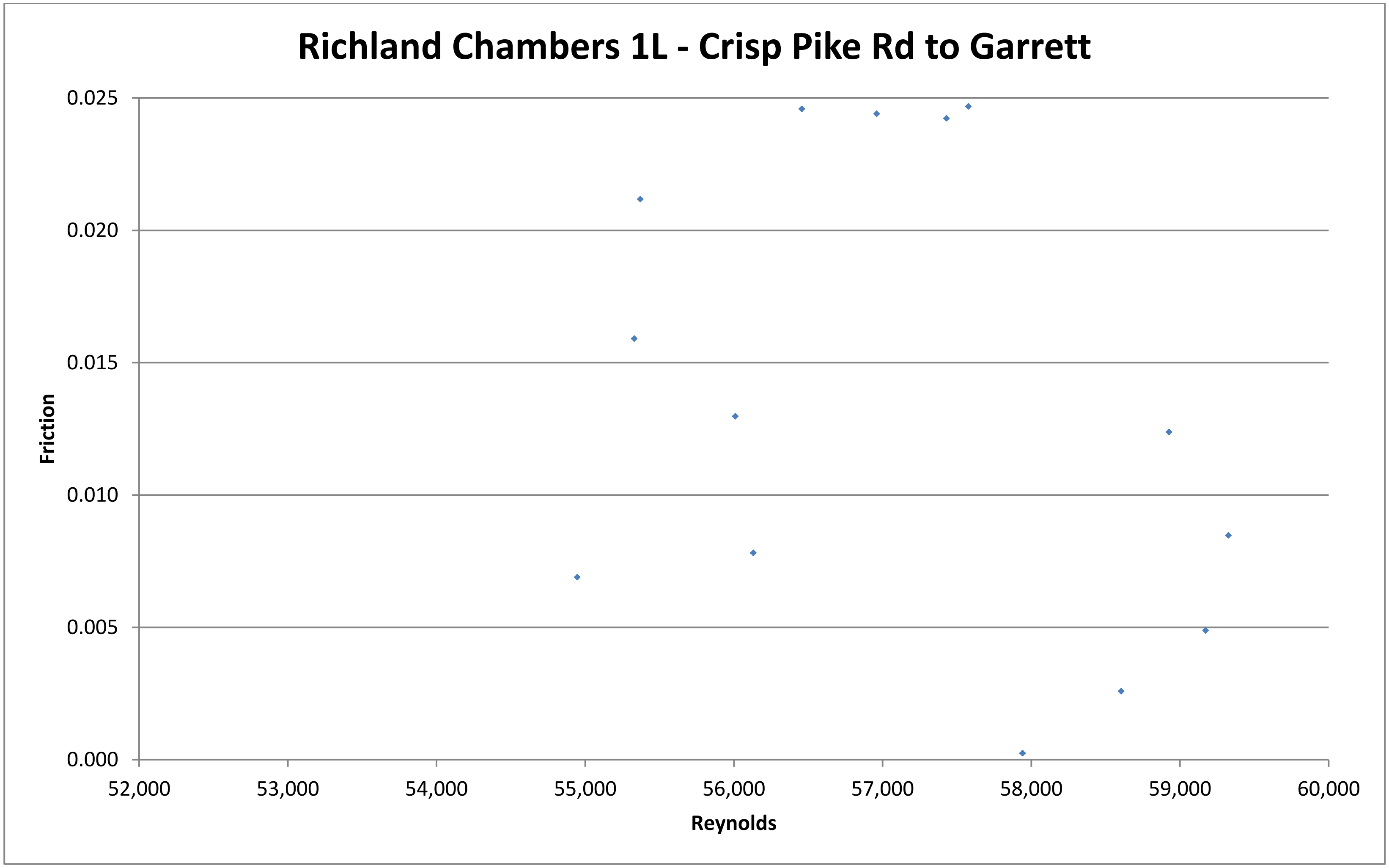
State Highway 85 to Richland Chambers Ennis Pump Station



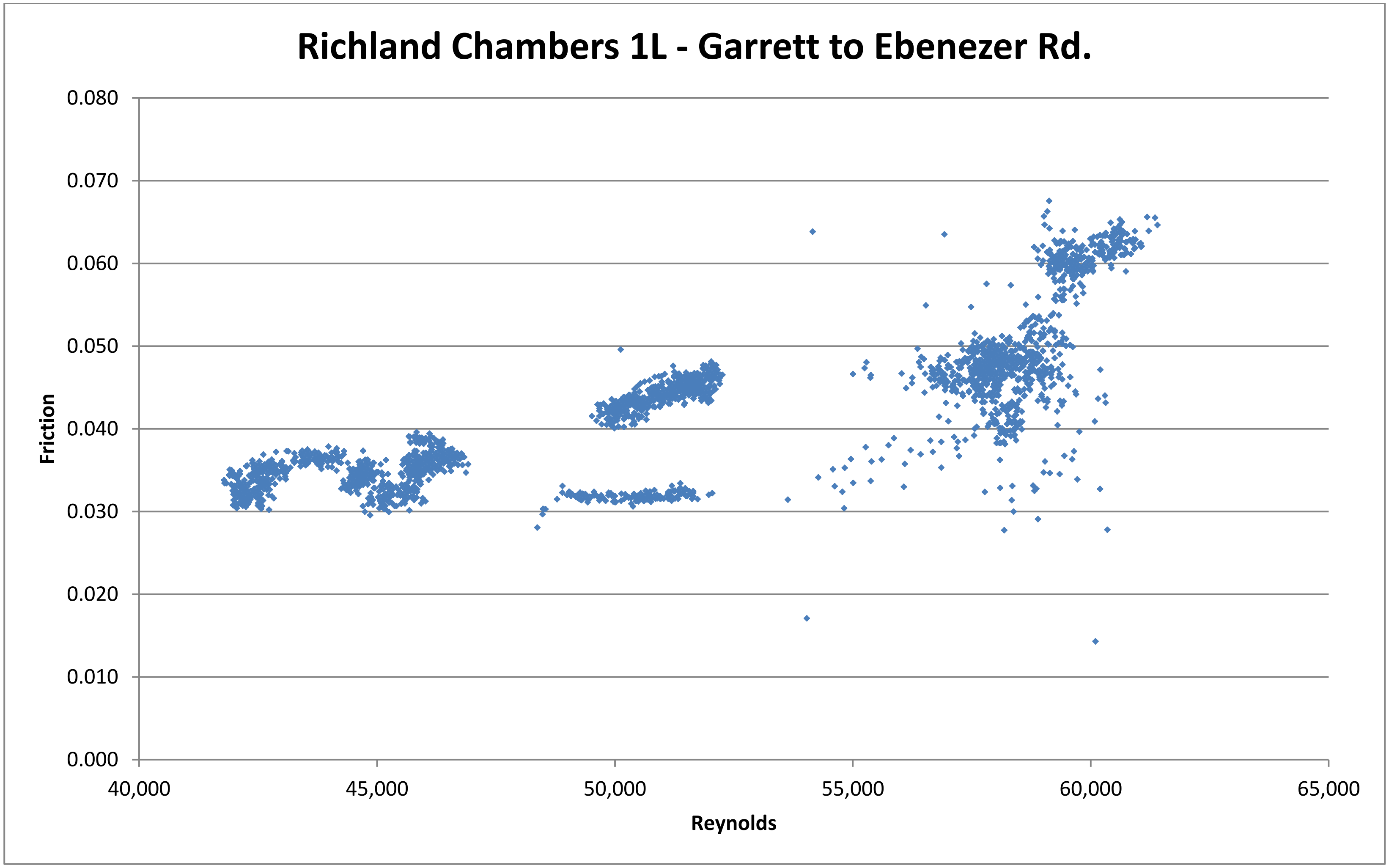
Richland Chambers Ennis Pump Station to Crisp Pike Road



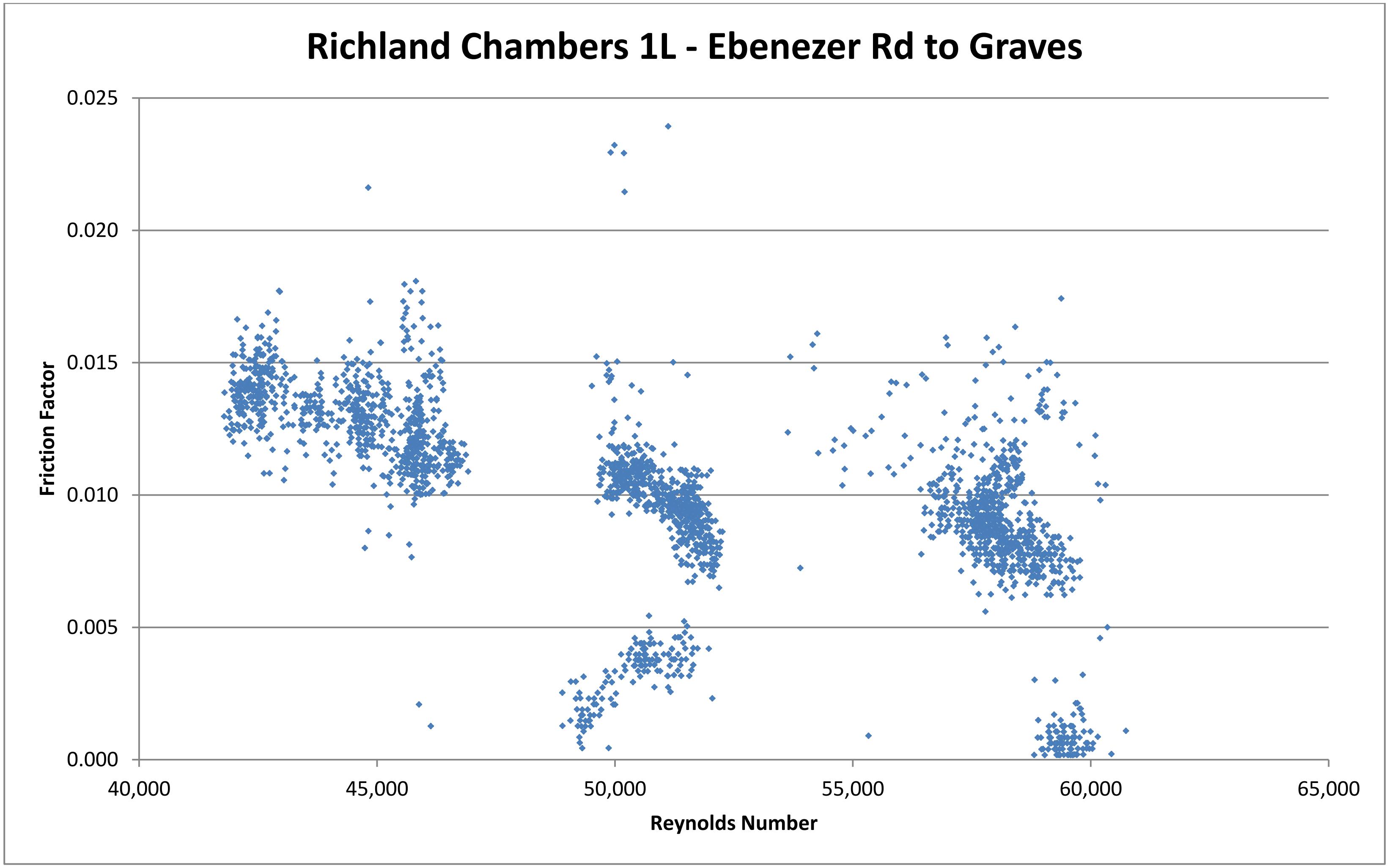
Crisp Pike Road to Garrett Road



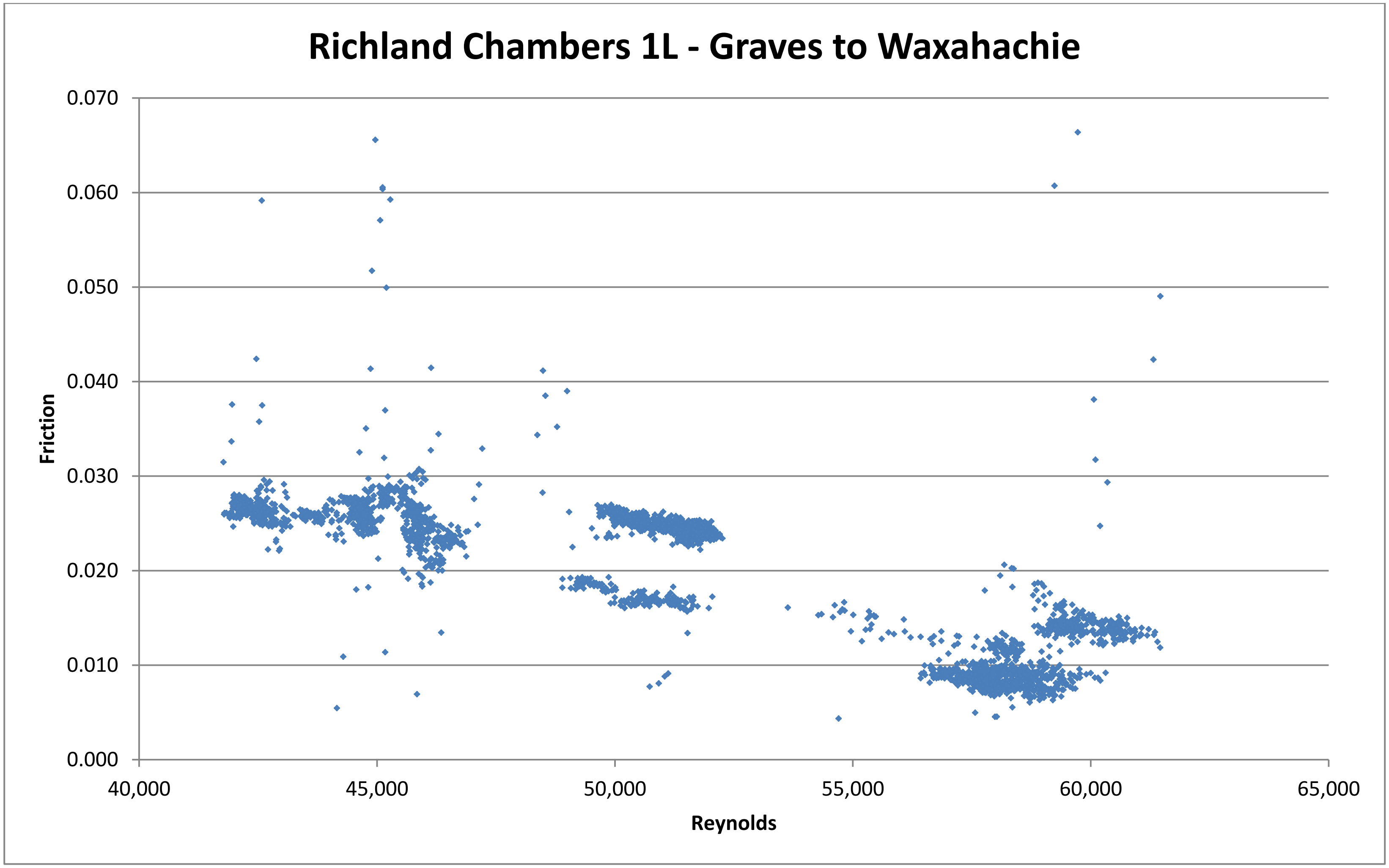
Garrett Road to Ebenezer Road



Ebenezer Road to Graves Road

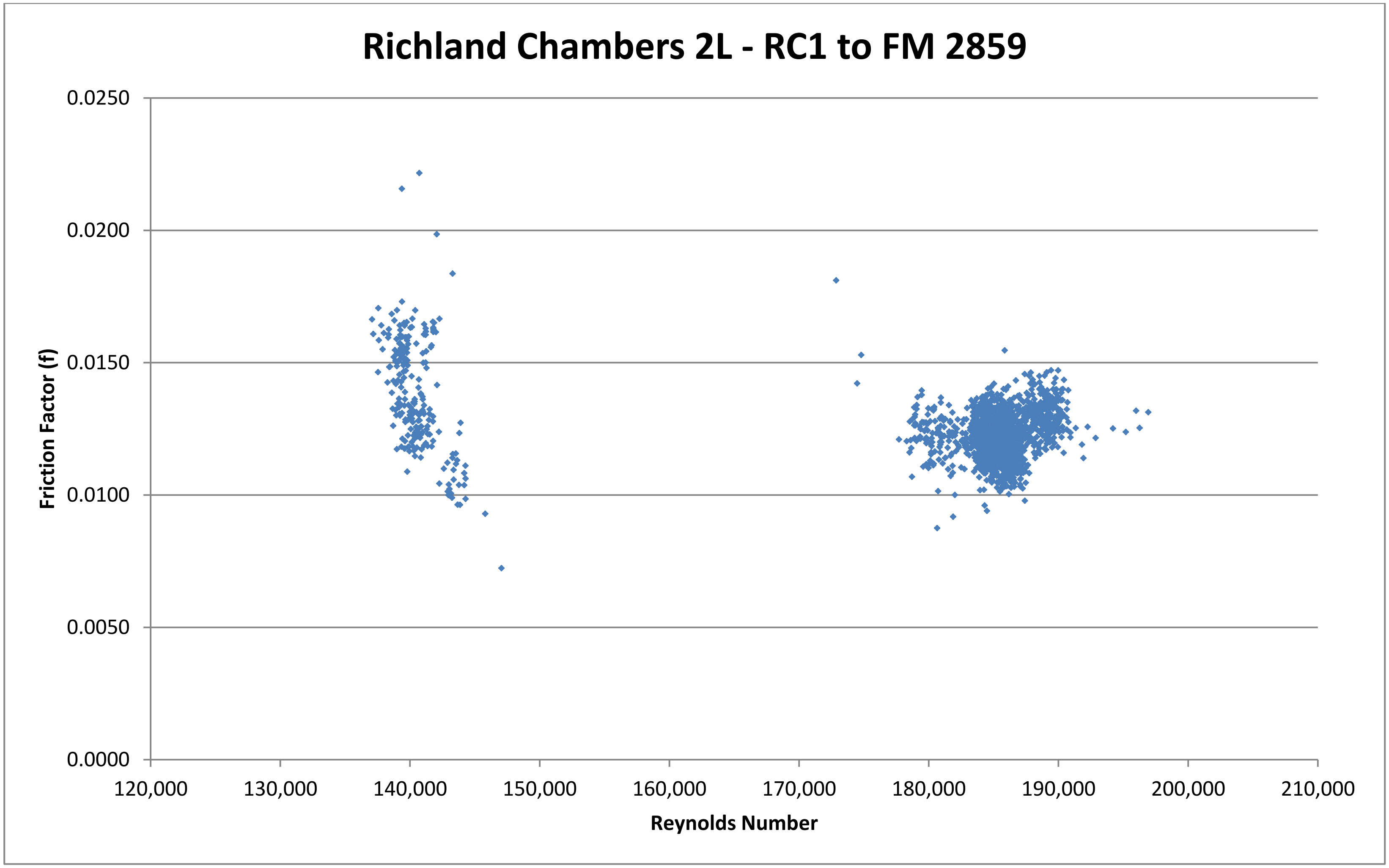


Graves Road to Richland Chambers Waxahachie Pump Station

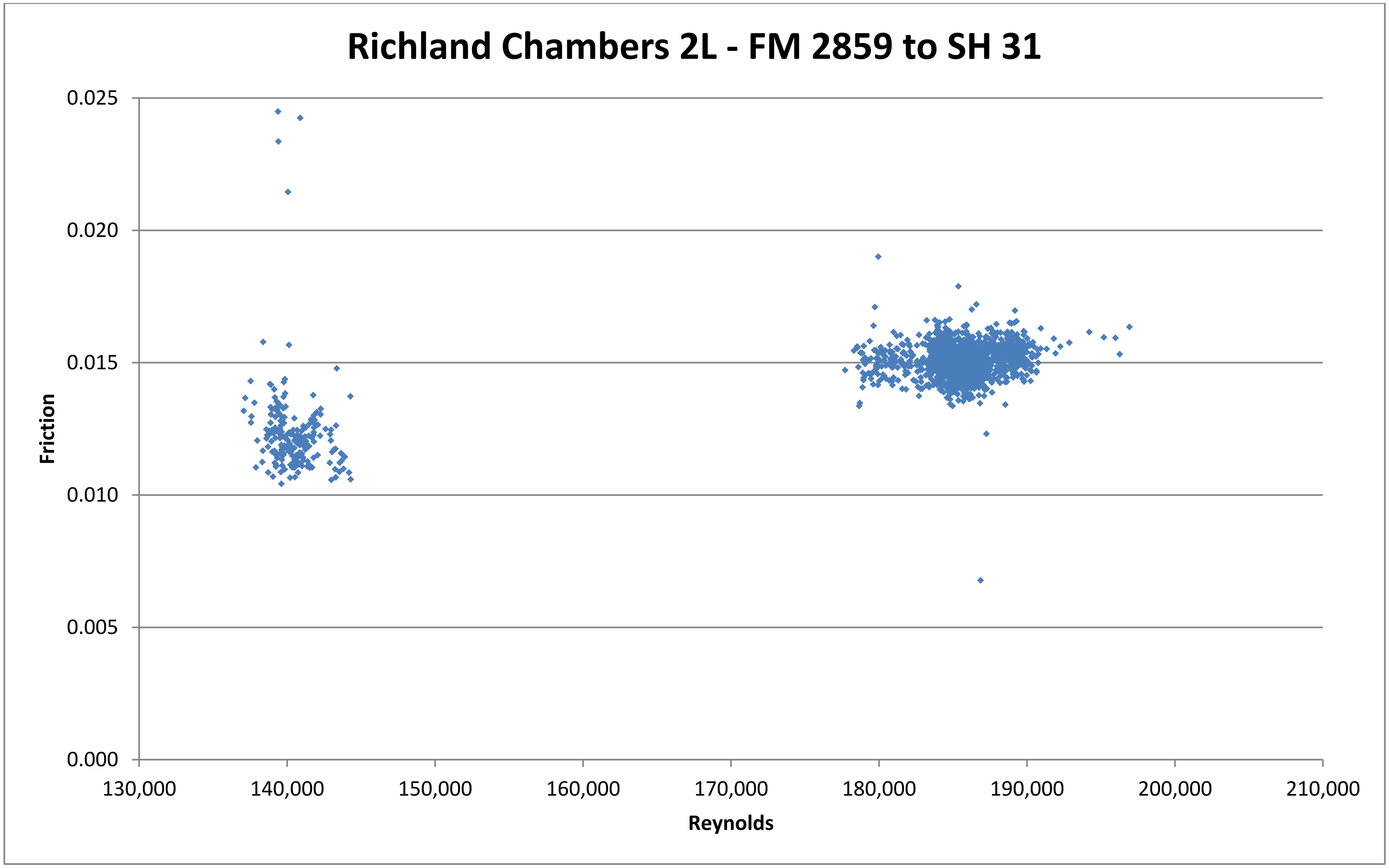


Richland Chambers 2L Pump Combination

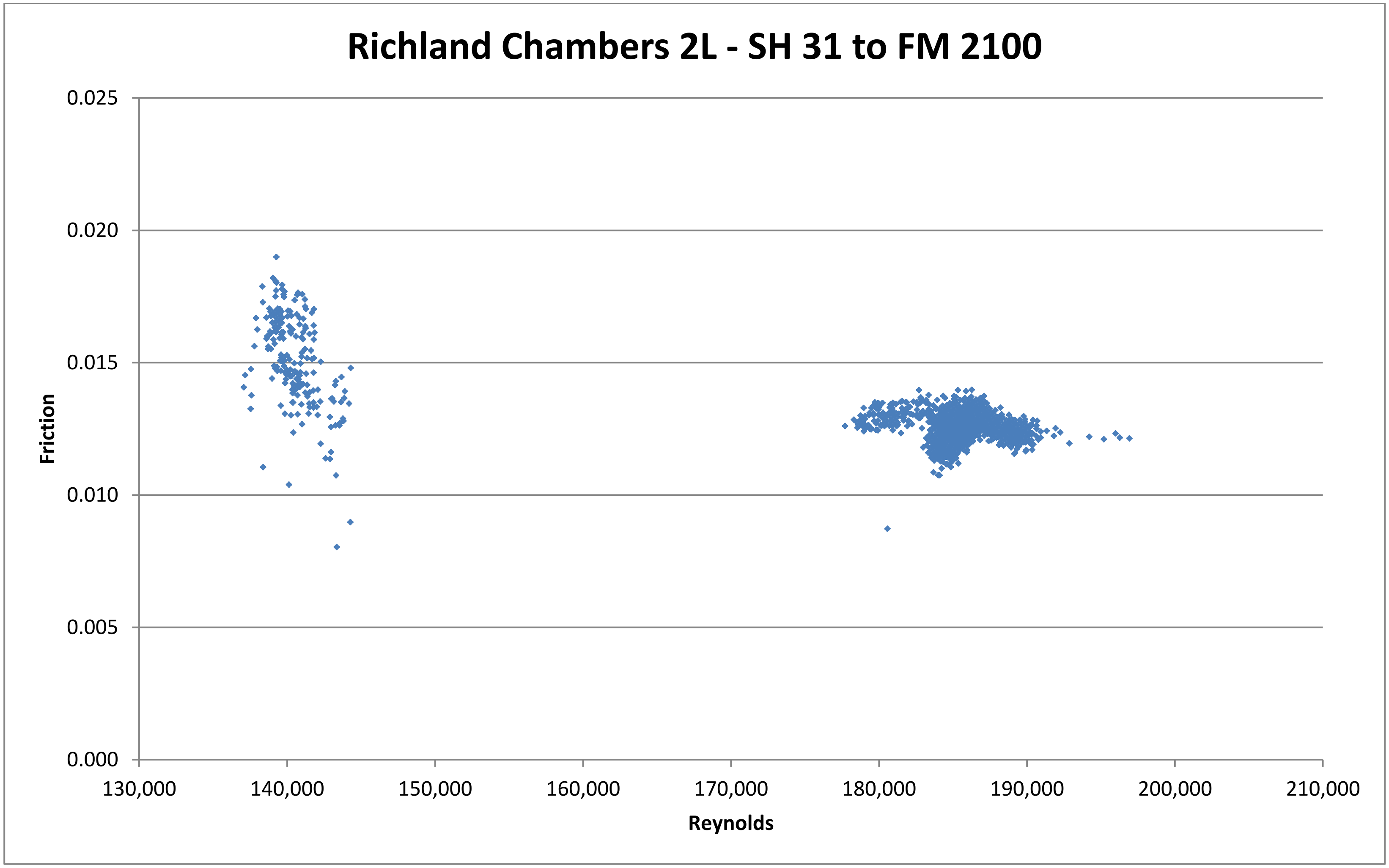
Richland Chambers Lake Pump Station to Farm-to-Market 2859



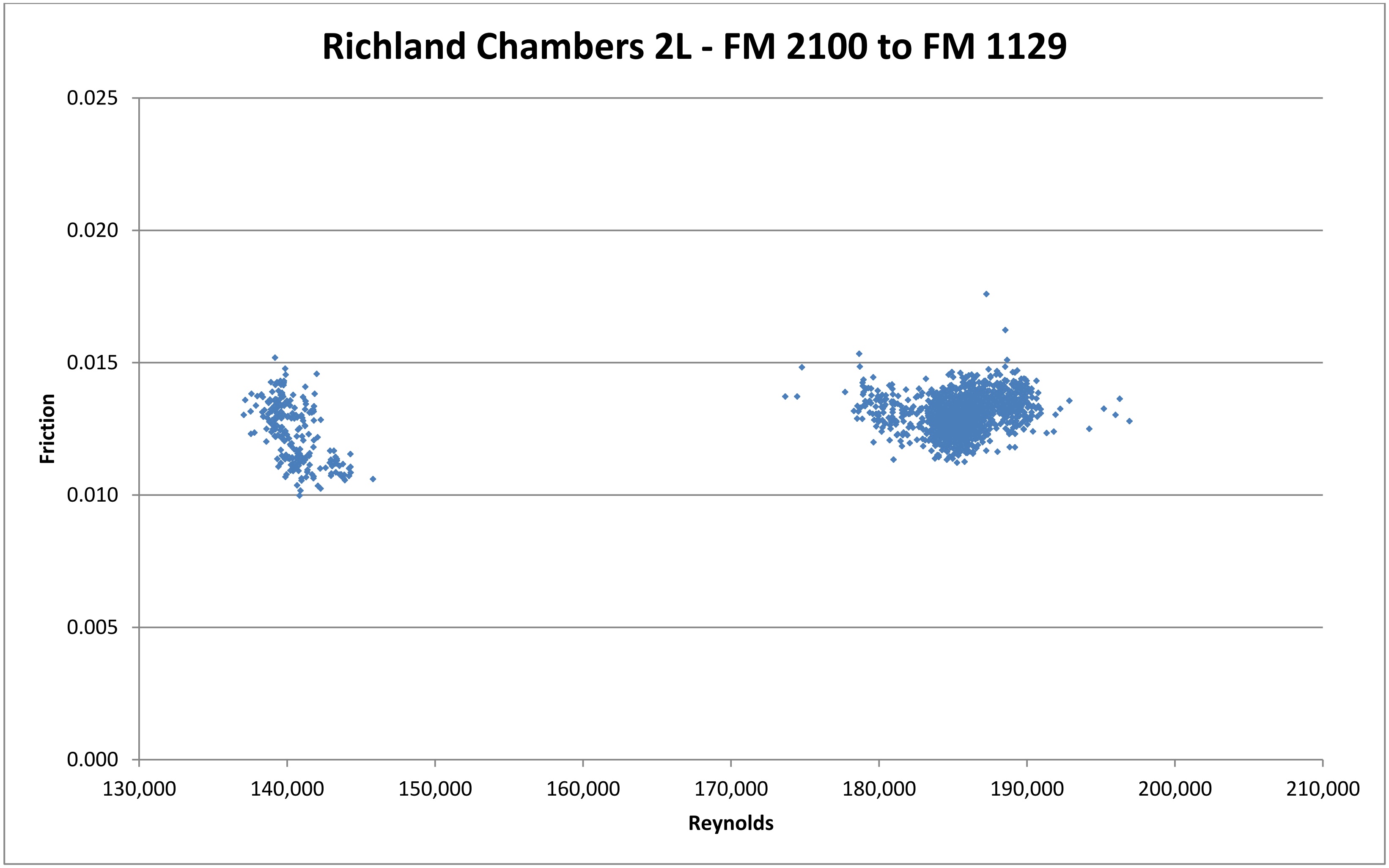
Farm-to-Market 2859 to State Highway 31



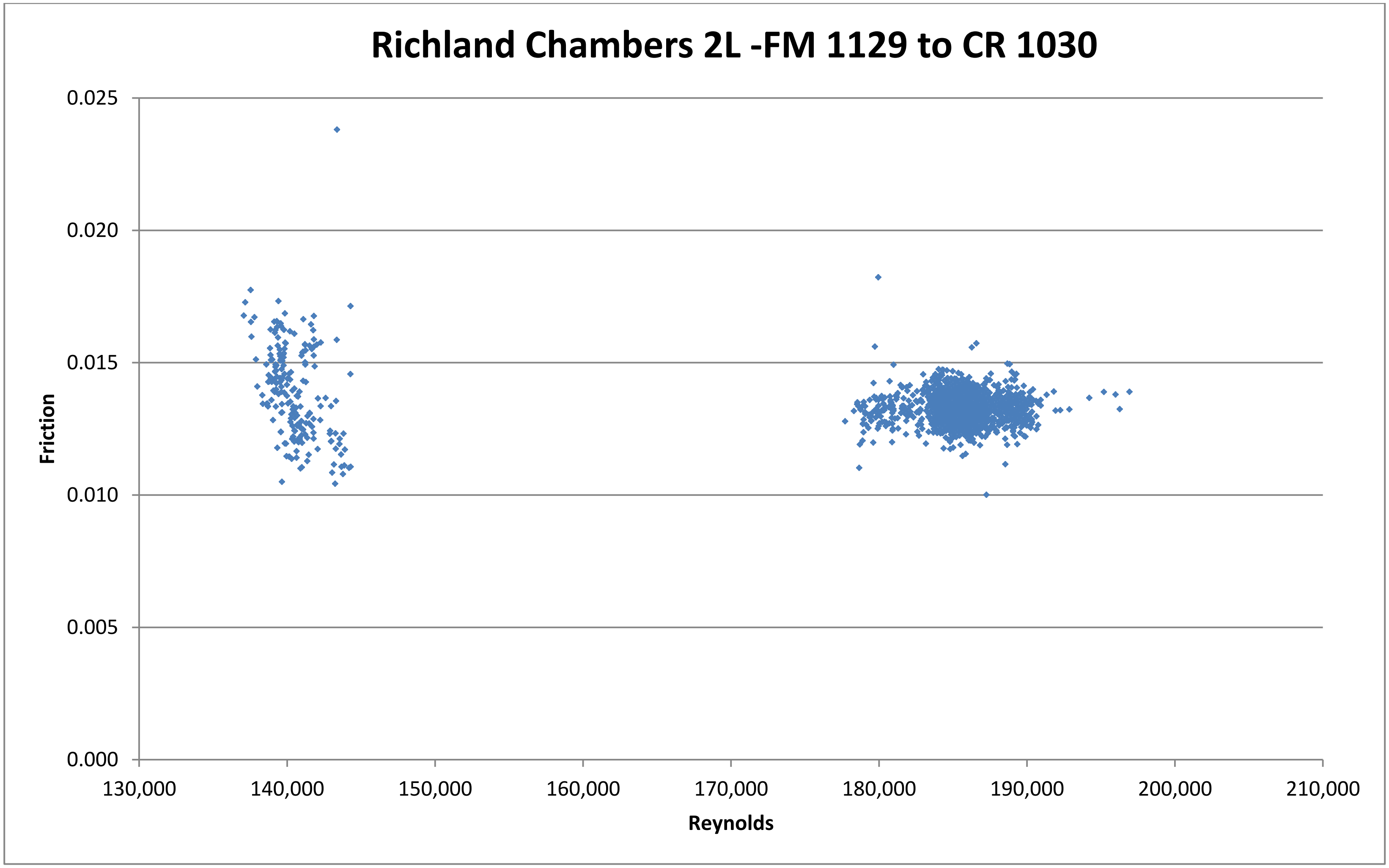
State Highway 31 to Farm-to-Market 2100



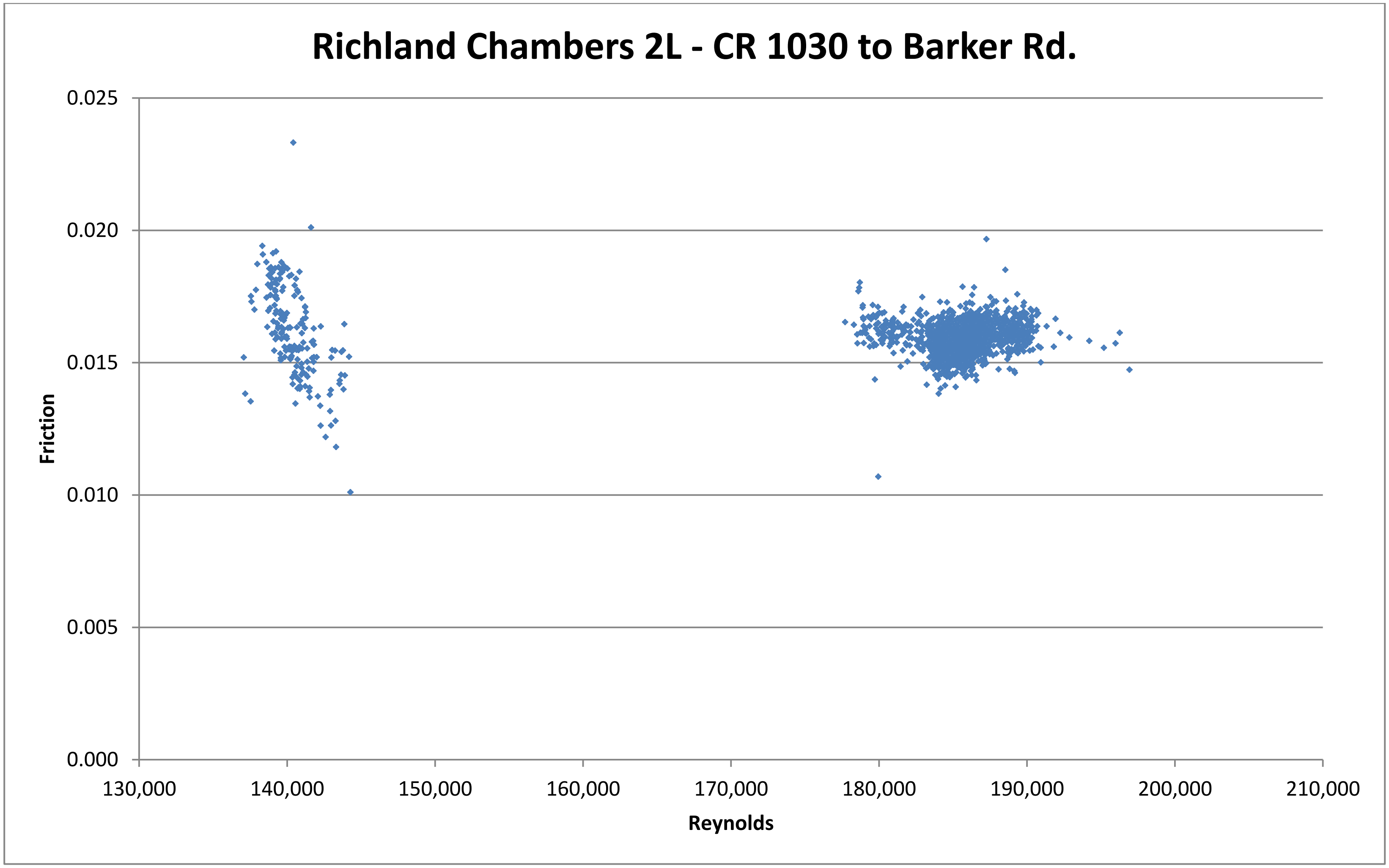
Farm-to-Market 2100 to Farm-to-Market 1129



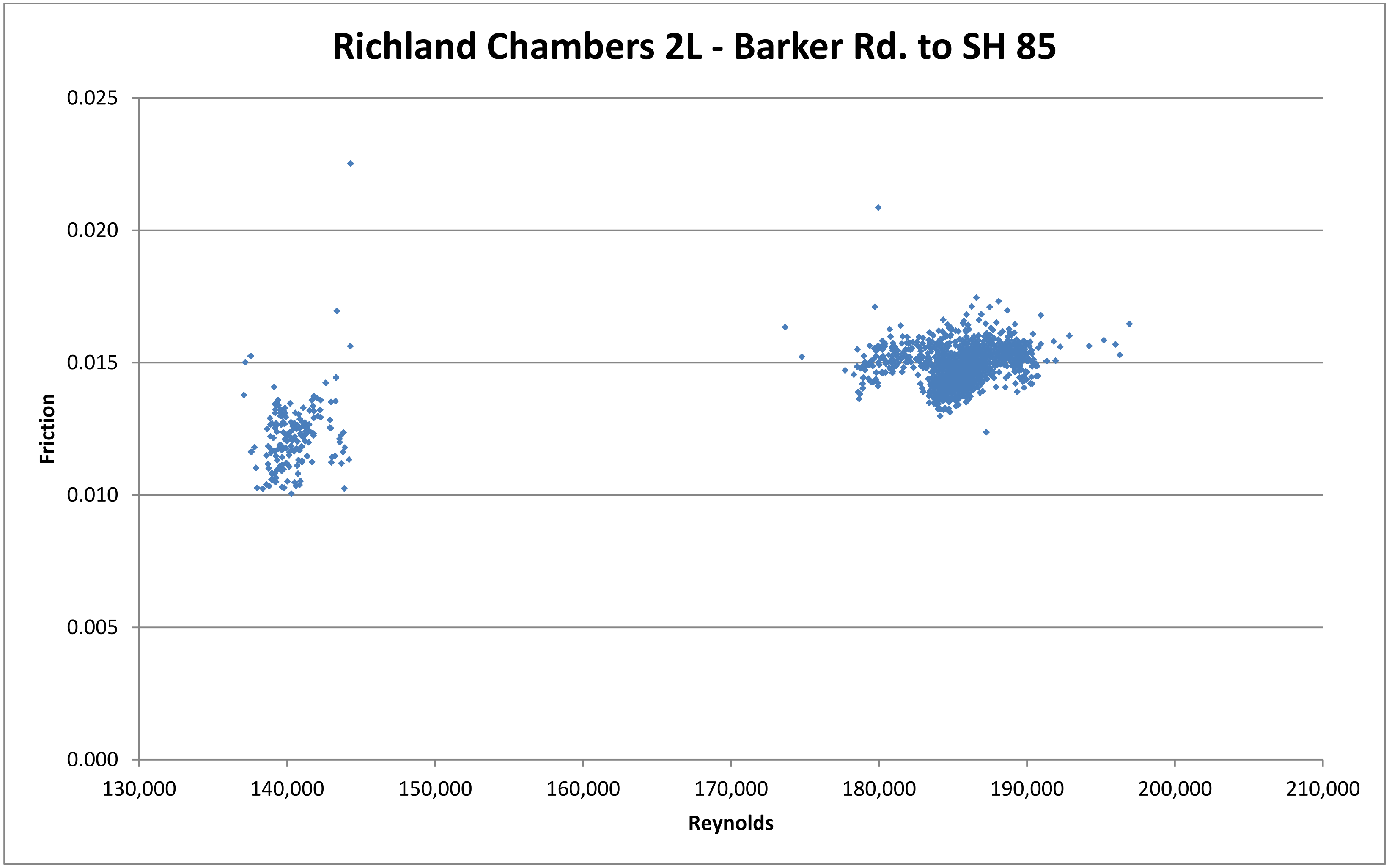
Farm-to-Market 1129 to County Road 1030



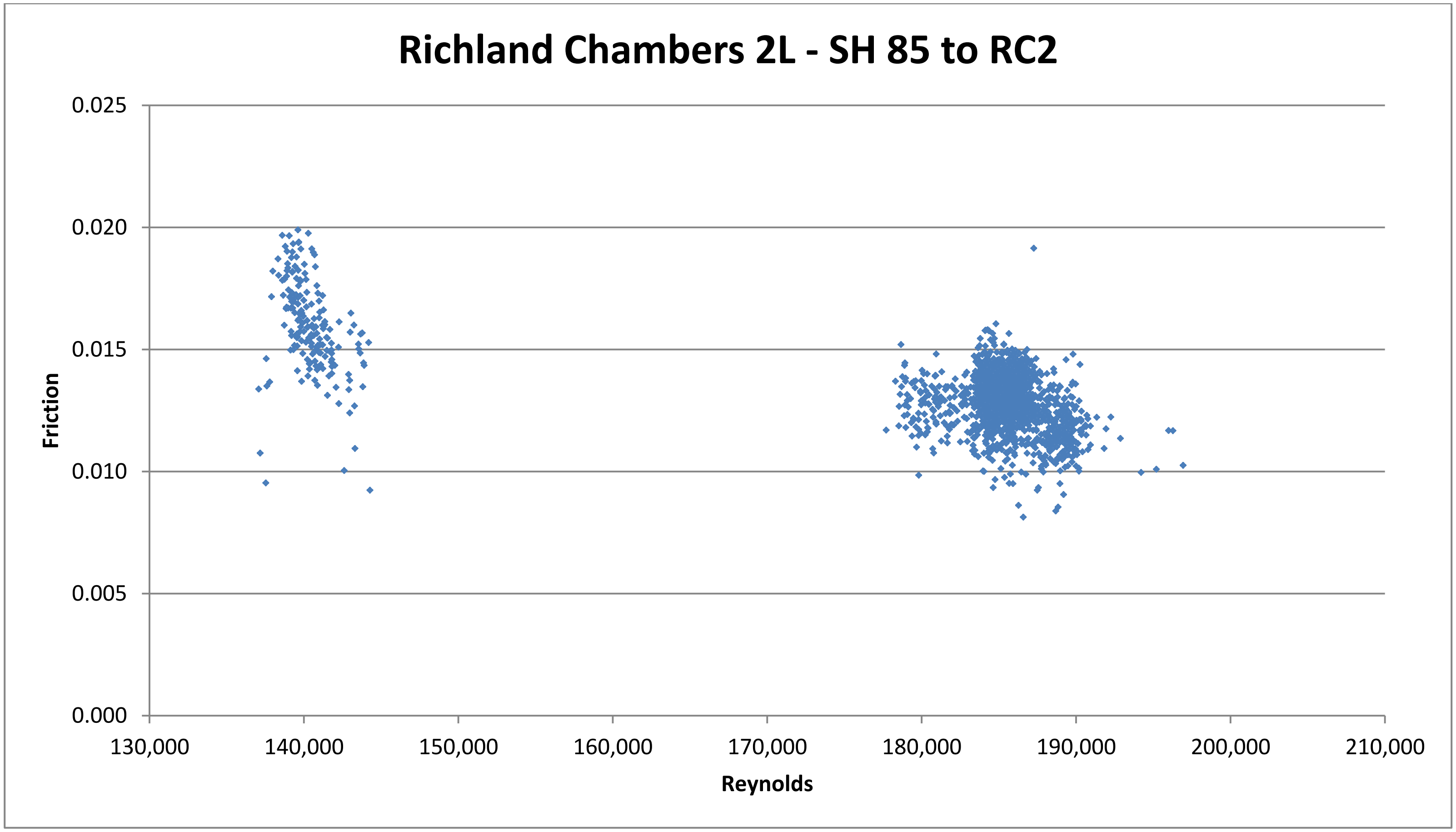
County Road 1030 to Barker Road



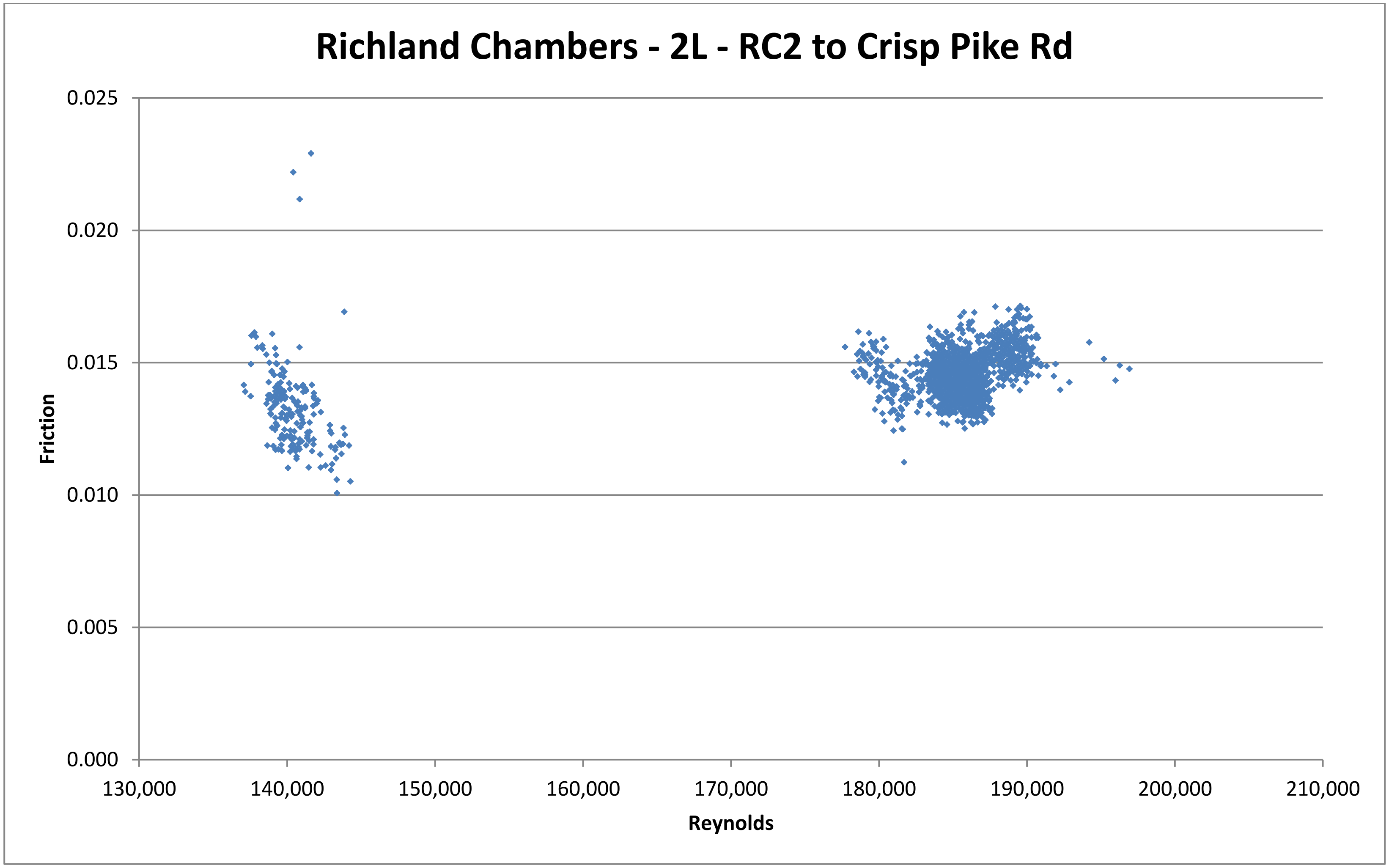
Barker Road to State Highway 85



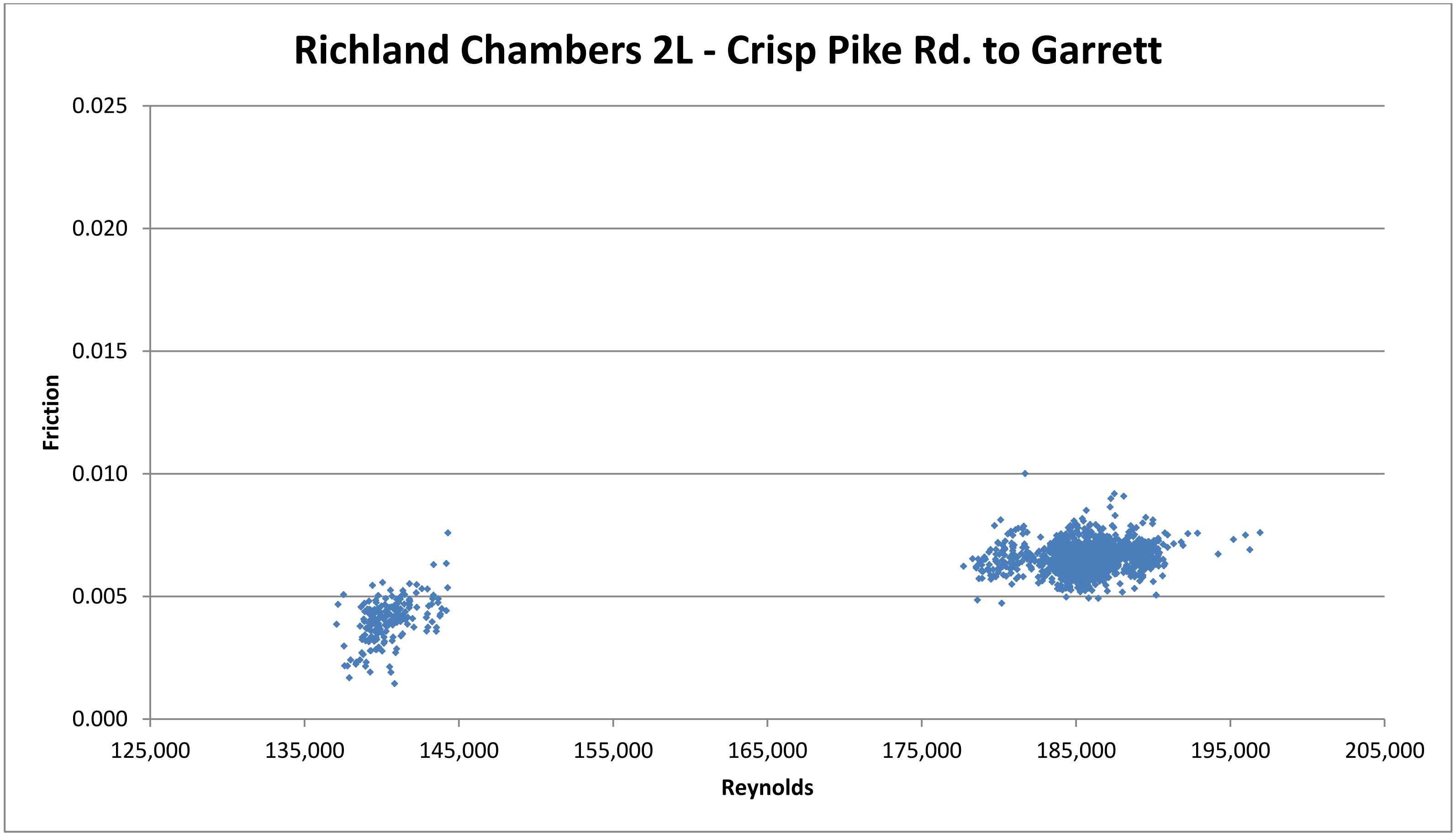
State Highway 85 to Richland Chambers Ennis Pump Station



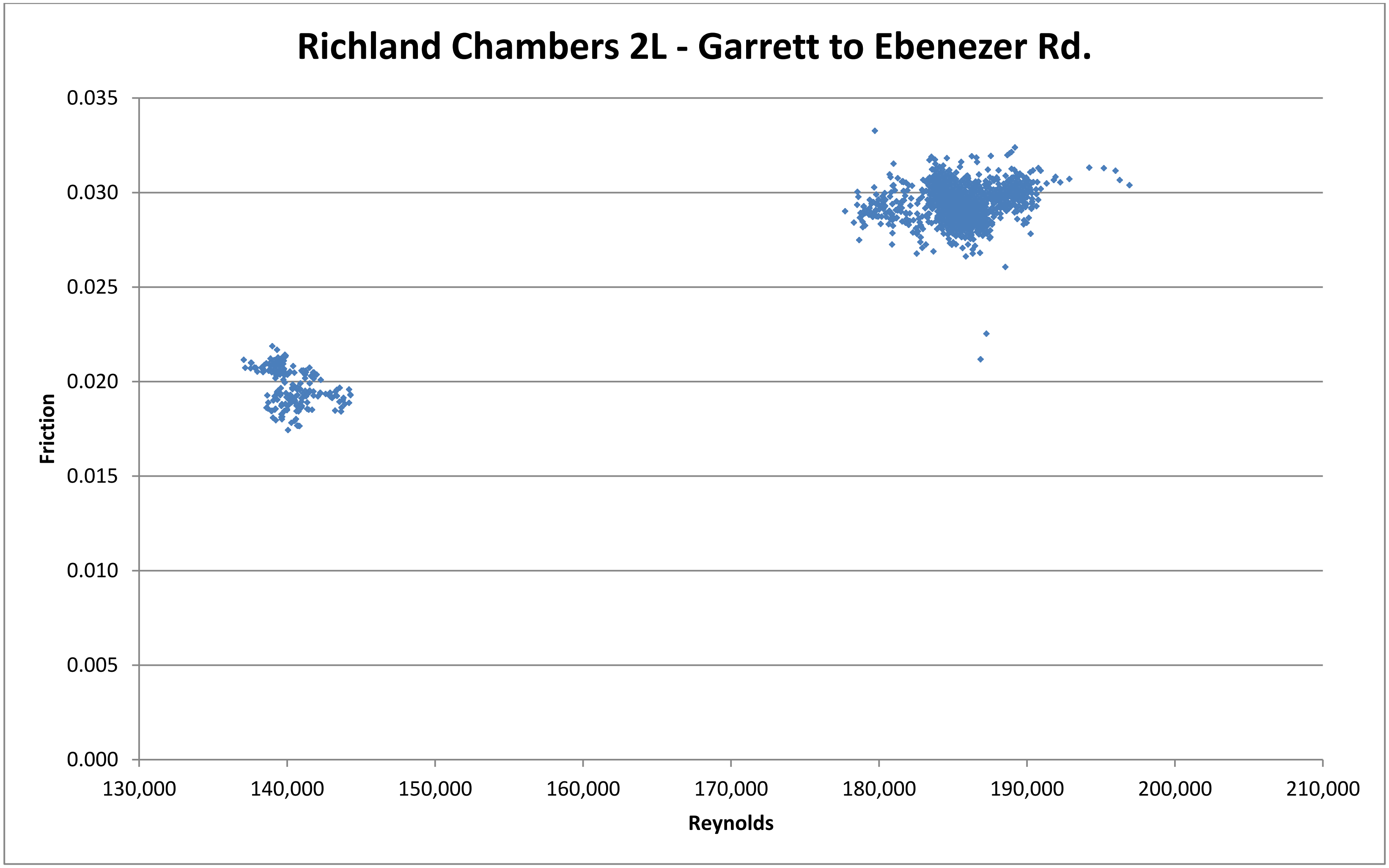
Richland Chambers Ennis Pump Station to Crisp Pike Road



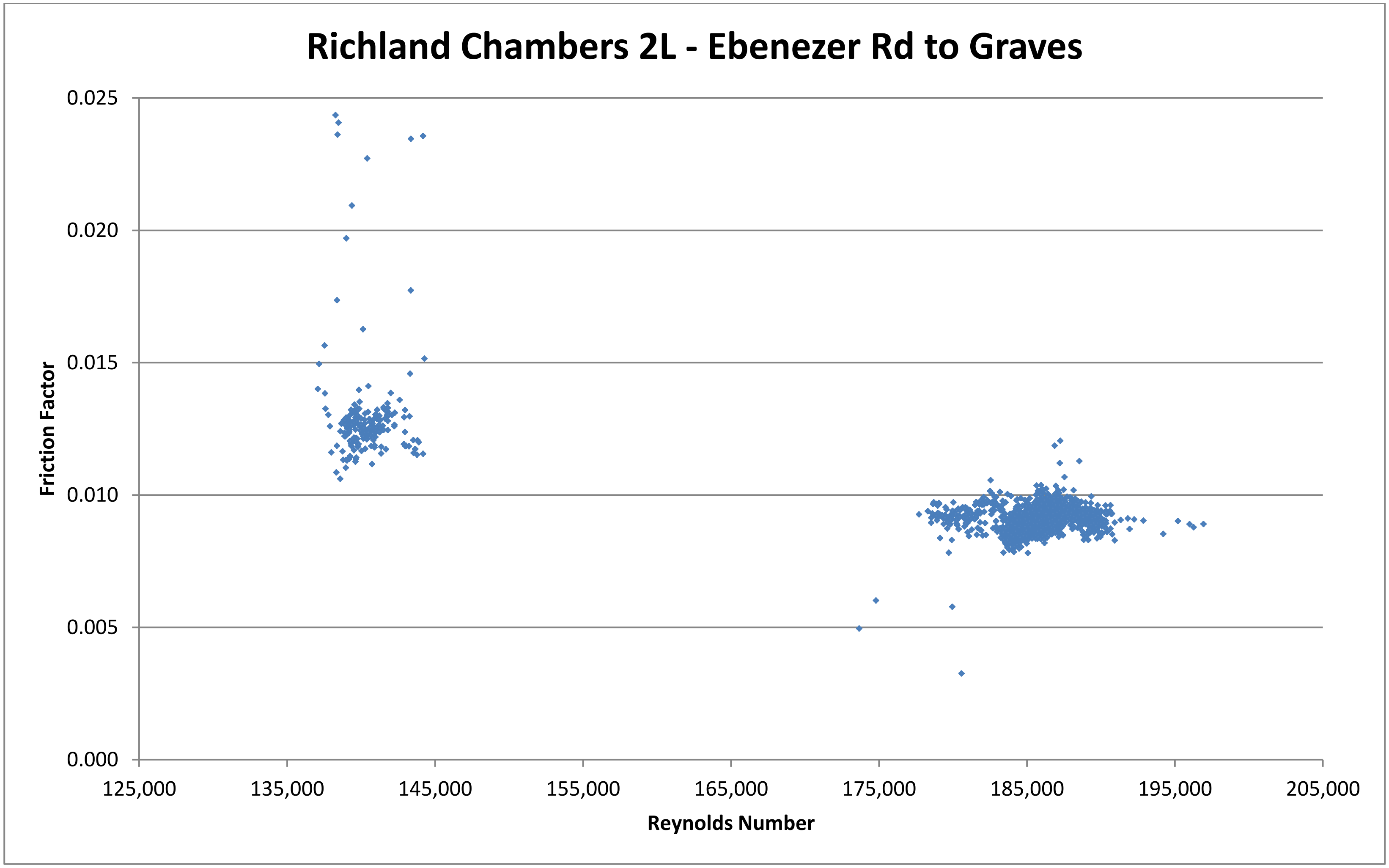
Crisp Pike Road to Garrett Road



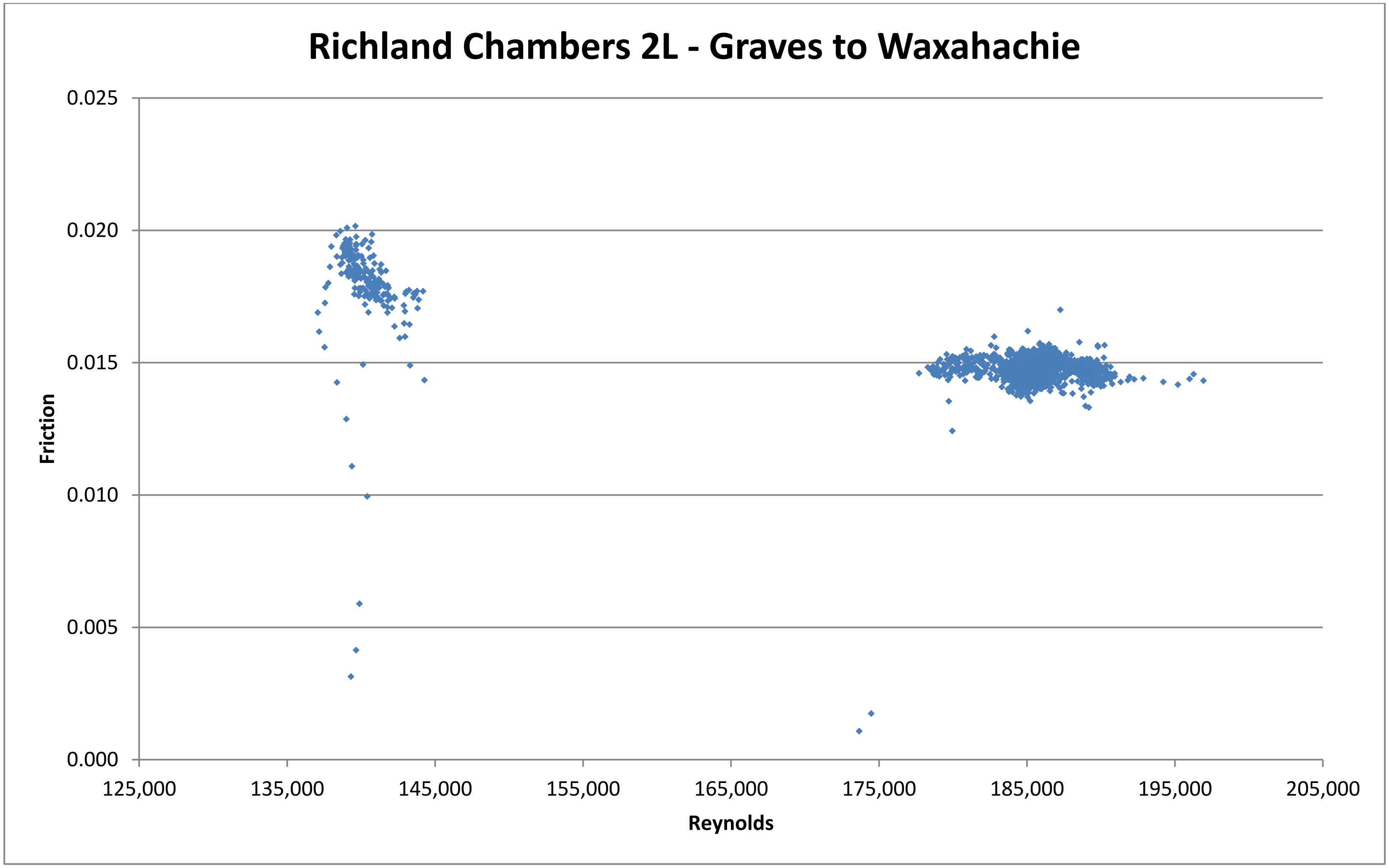
Garrett Road to Ebenezer Road



Ebenezer Road to Graves Road

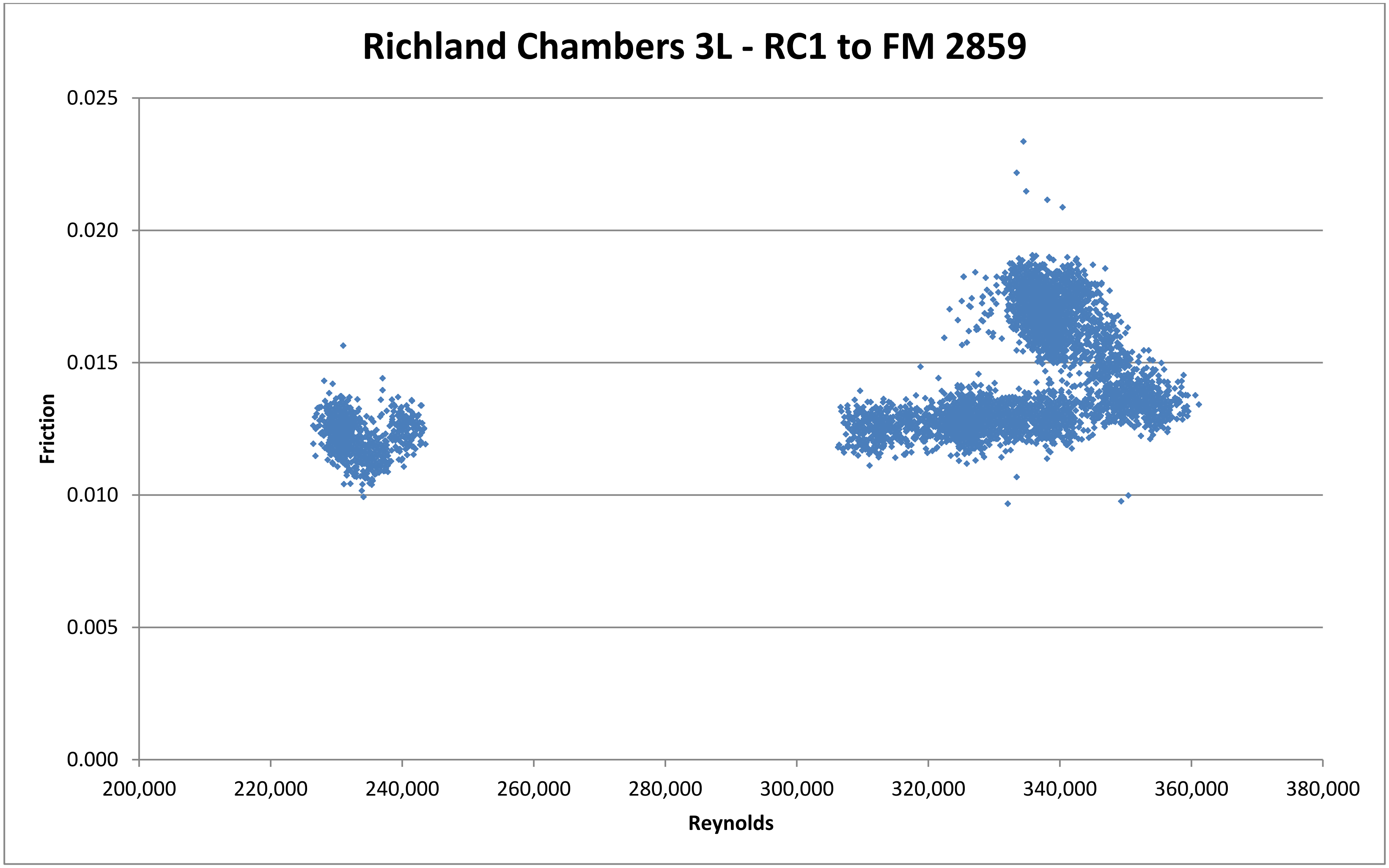


Graves Road to Richland Chambers Waxahachie Pump Station

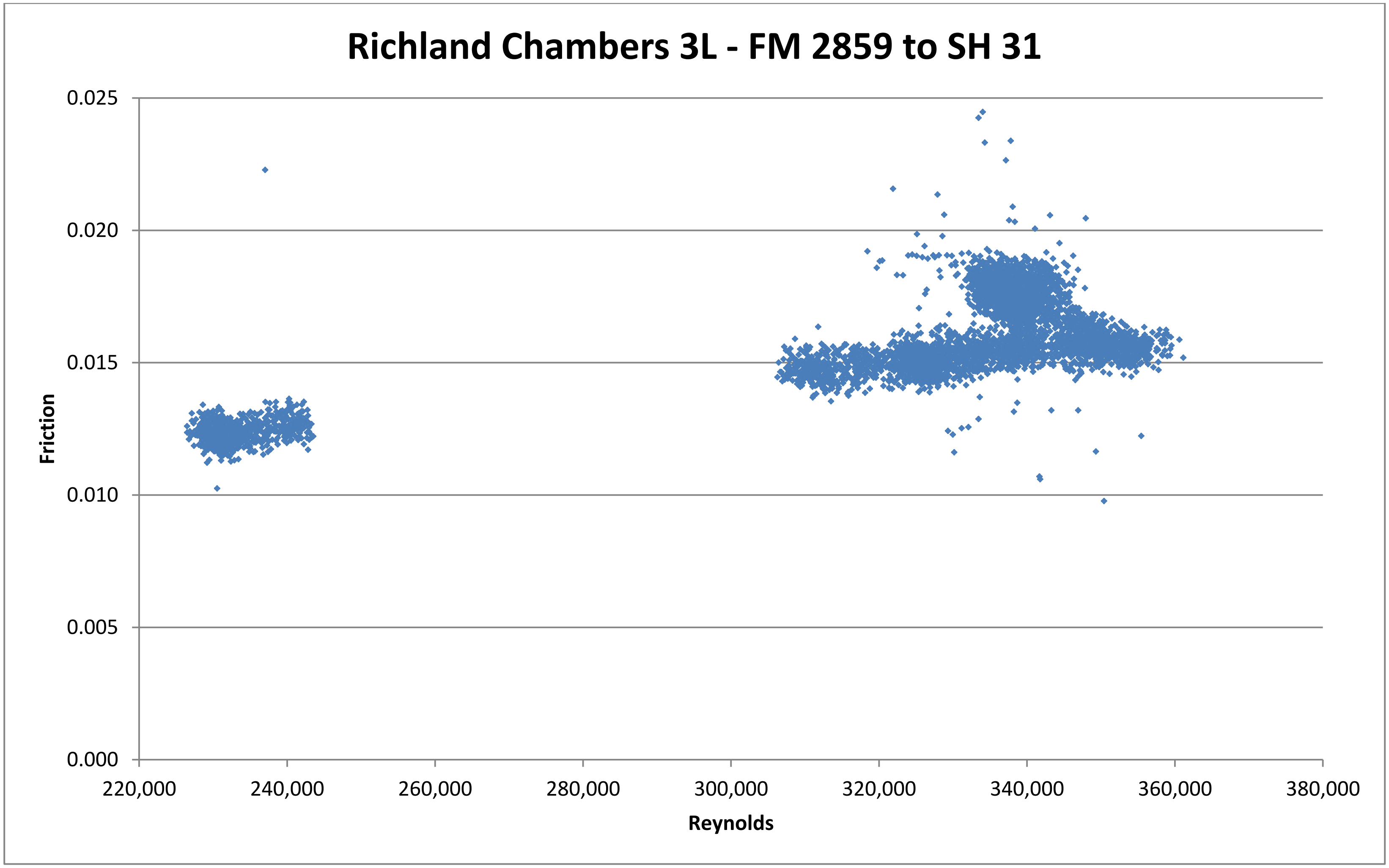


Richland Chambers 3L Pump Combination

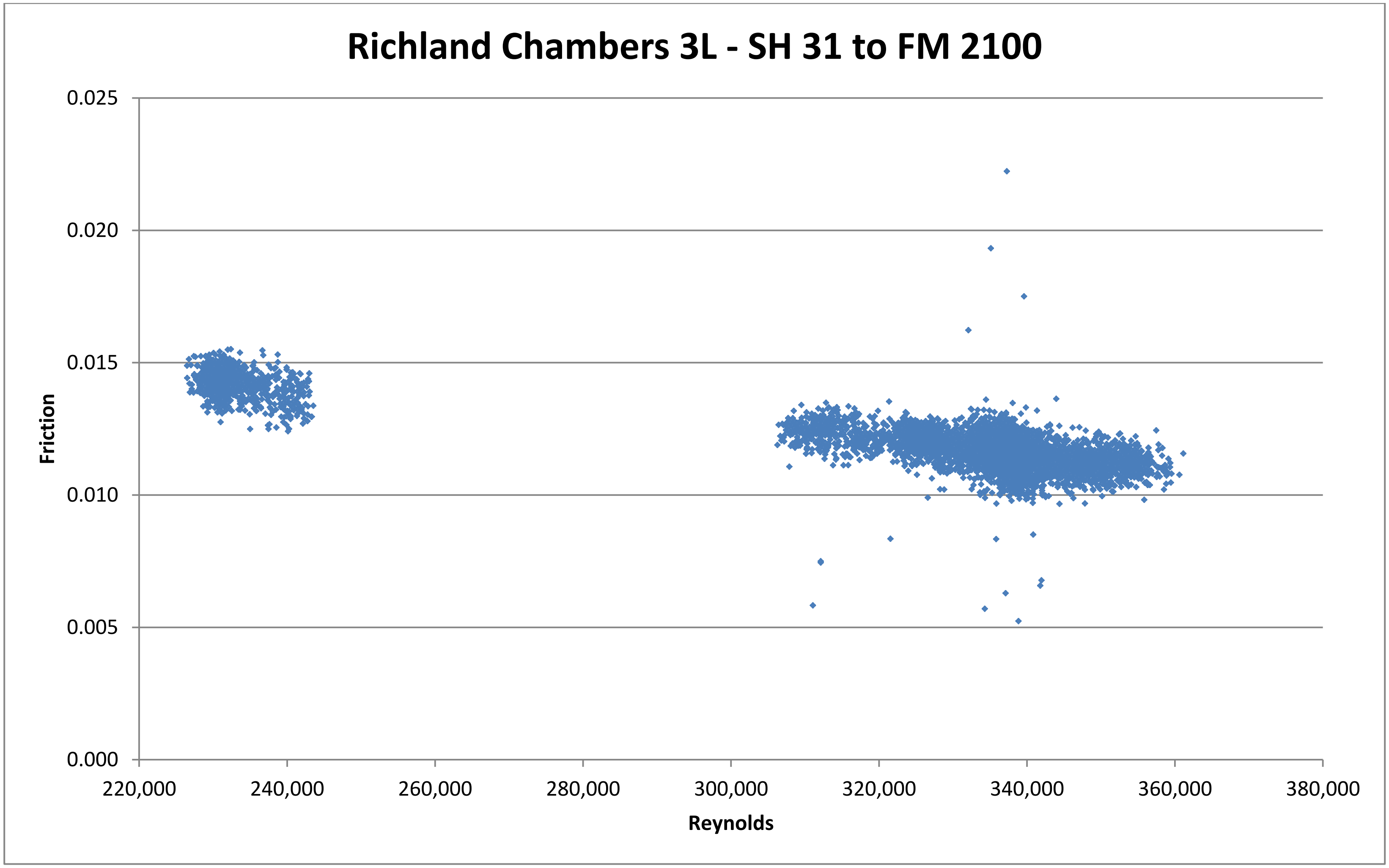
Richland Chambers Lake Pump Station to Farm-to-Market 2859



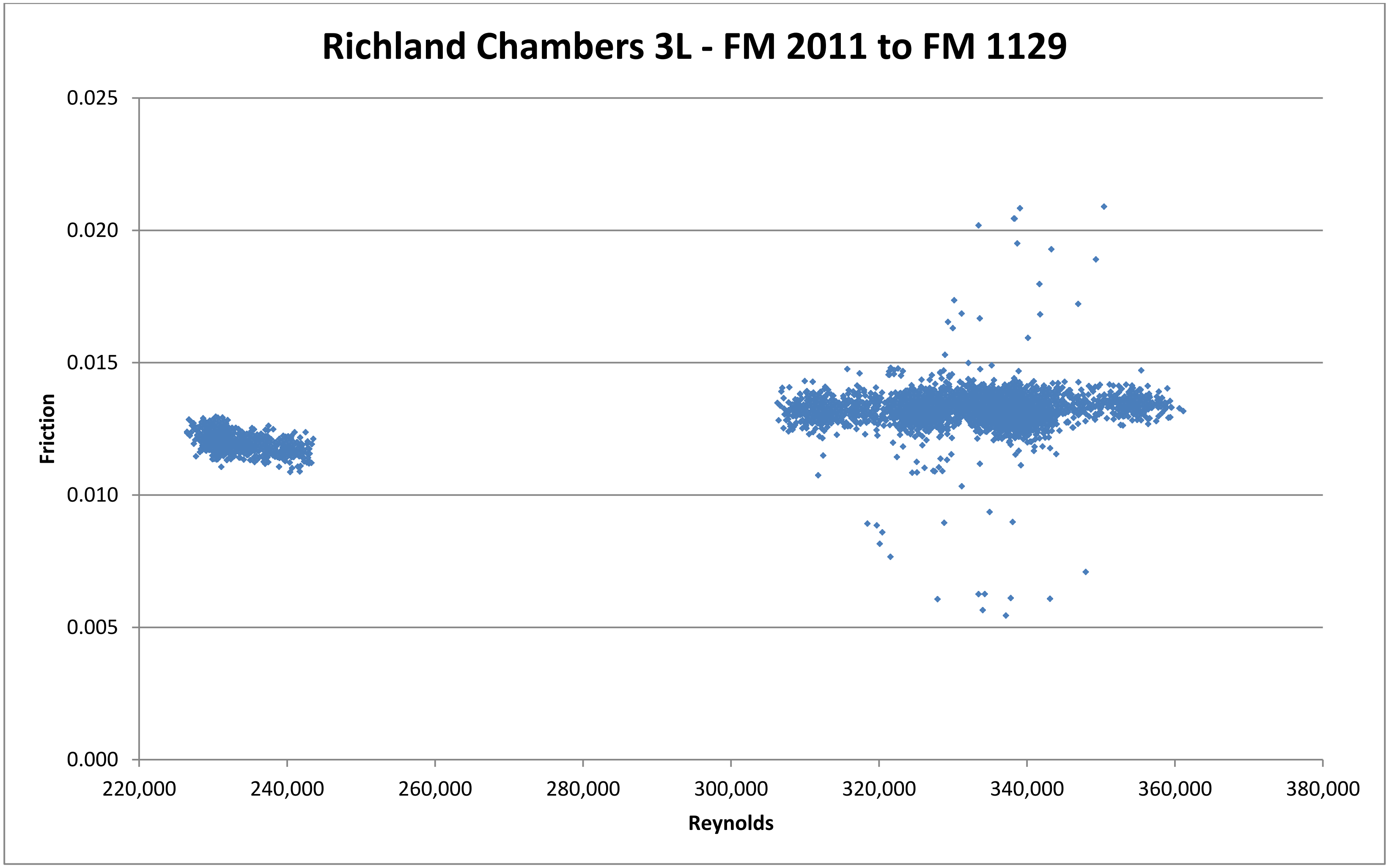
Farm-to-Market 2859 to State Highway 31



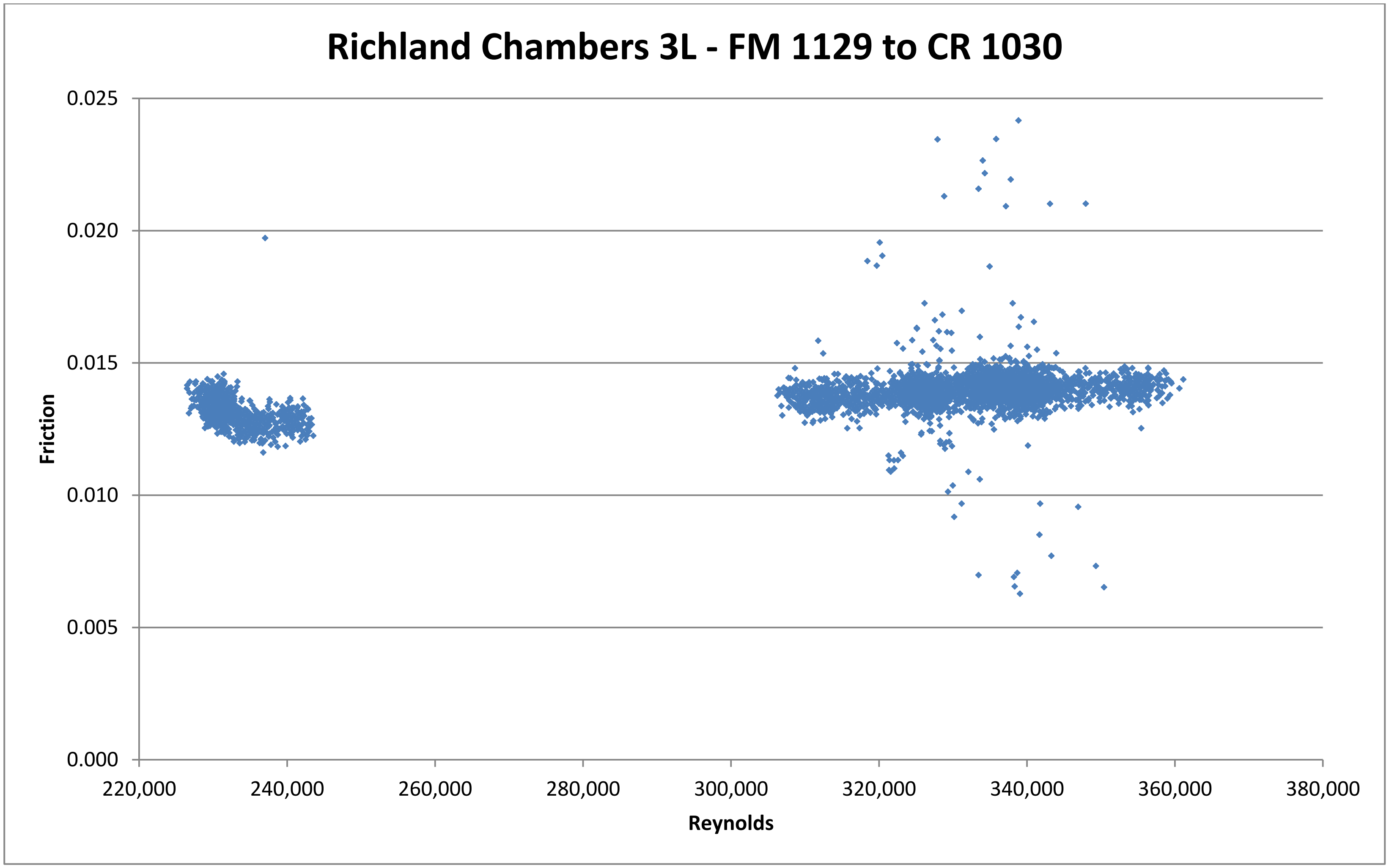
State Highway 31 to Farm-to-Market 2100



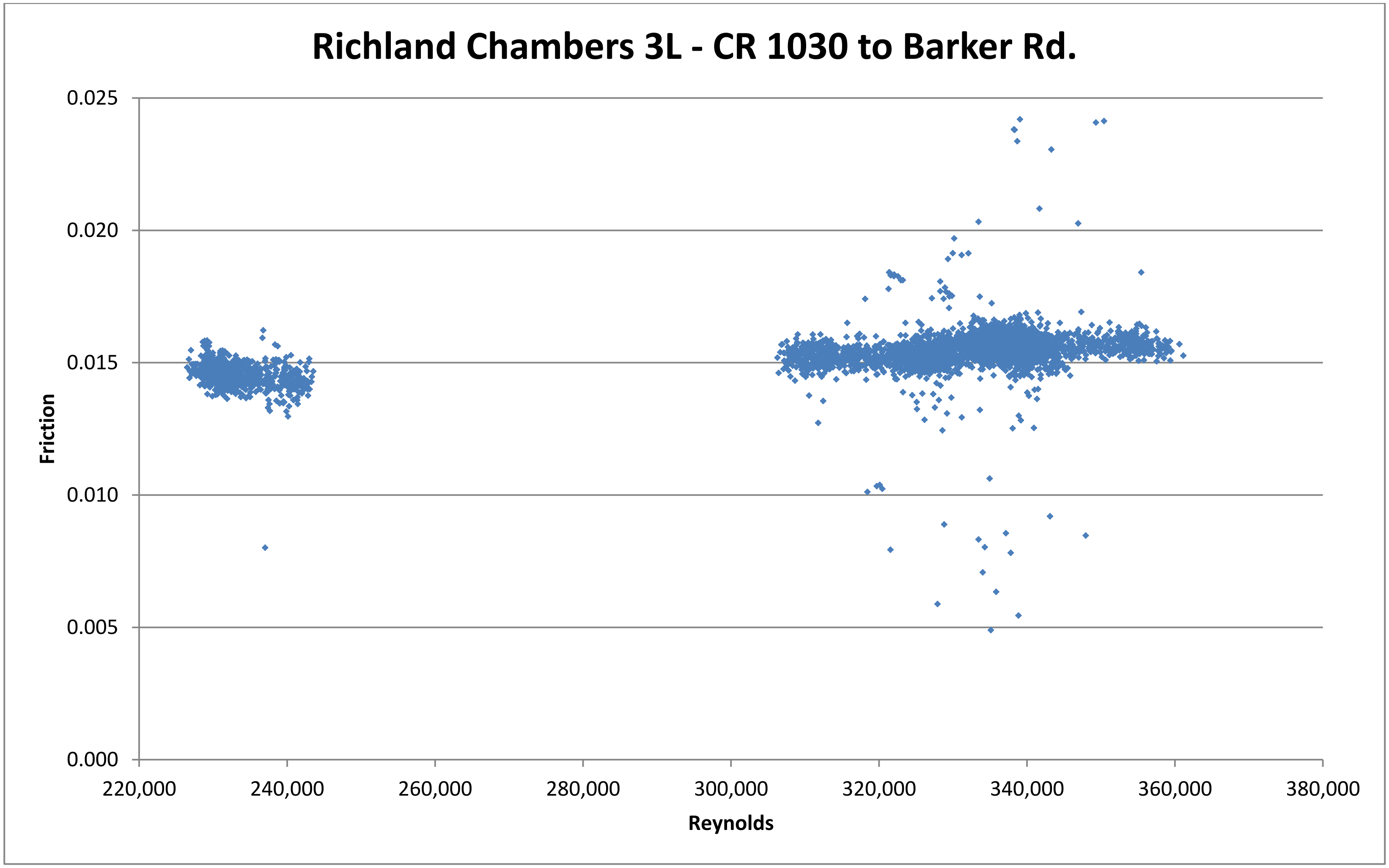
Farm-to-Market 2100 to Farm-to-Market 1129



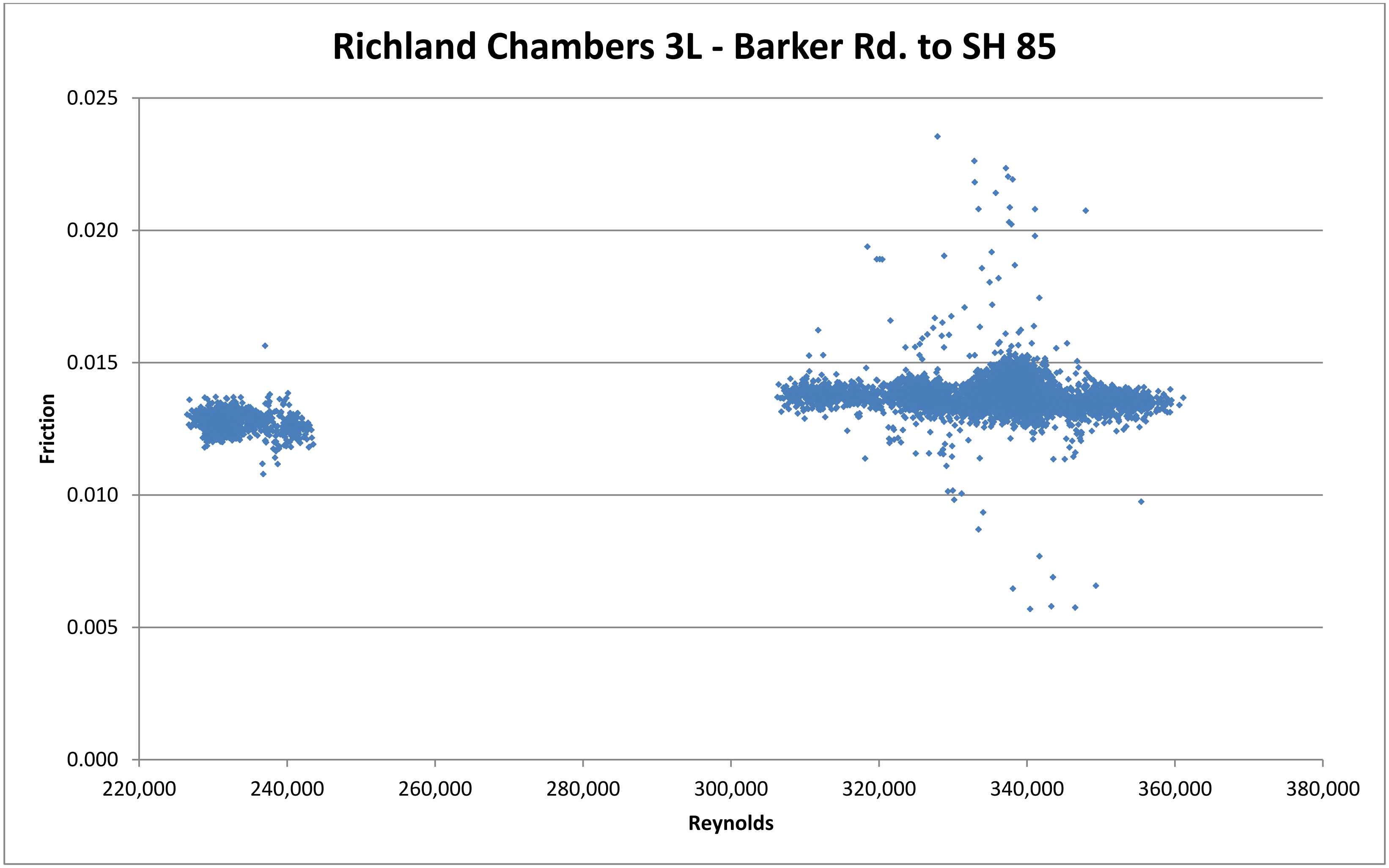
Farm-to-Market 1129 to County Road 1030



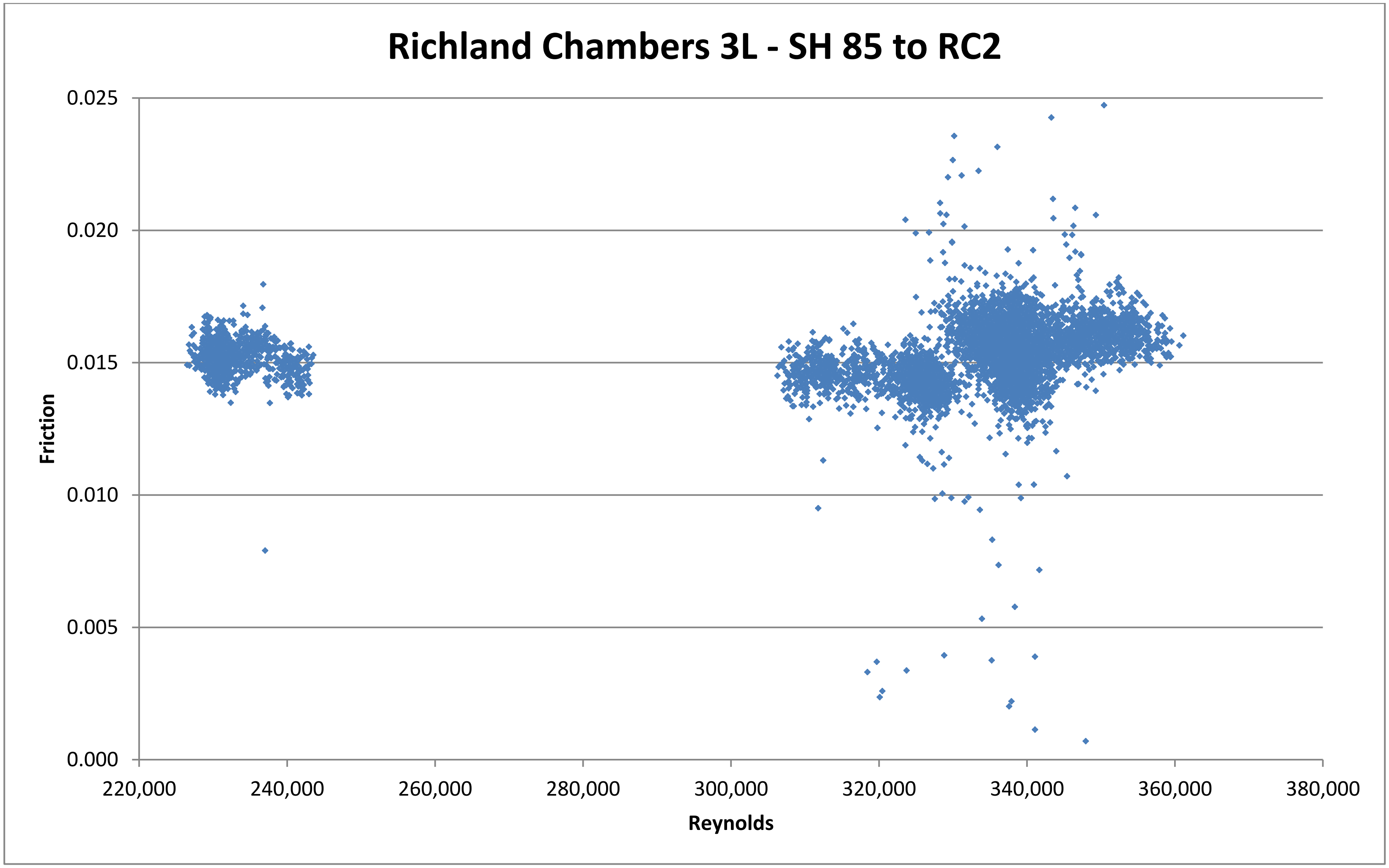
County Road 1030 to Barker Road



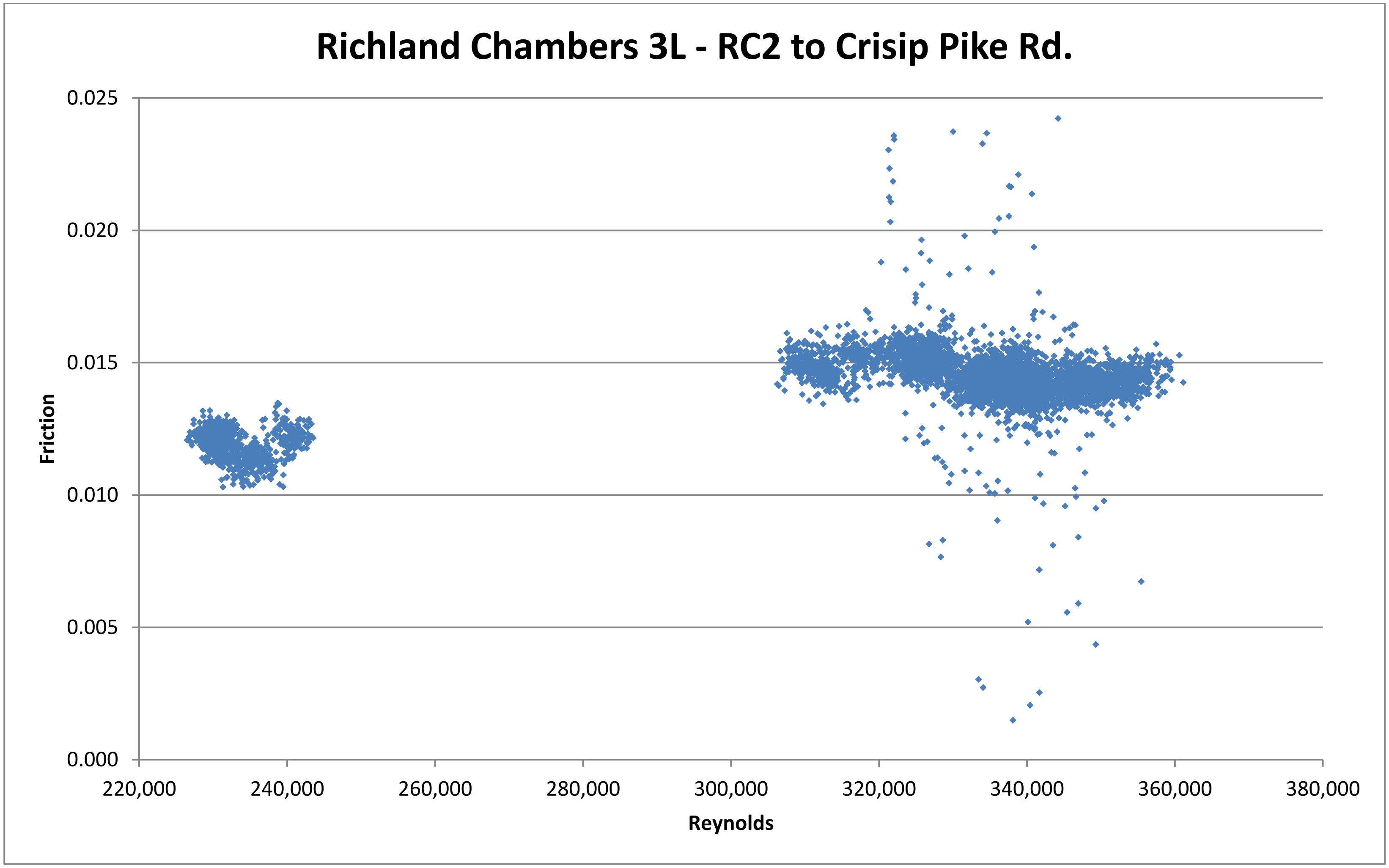
Barker Road to State Highway 85



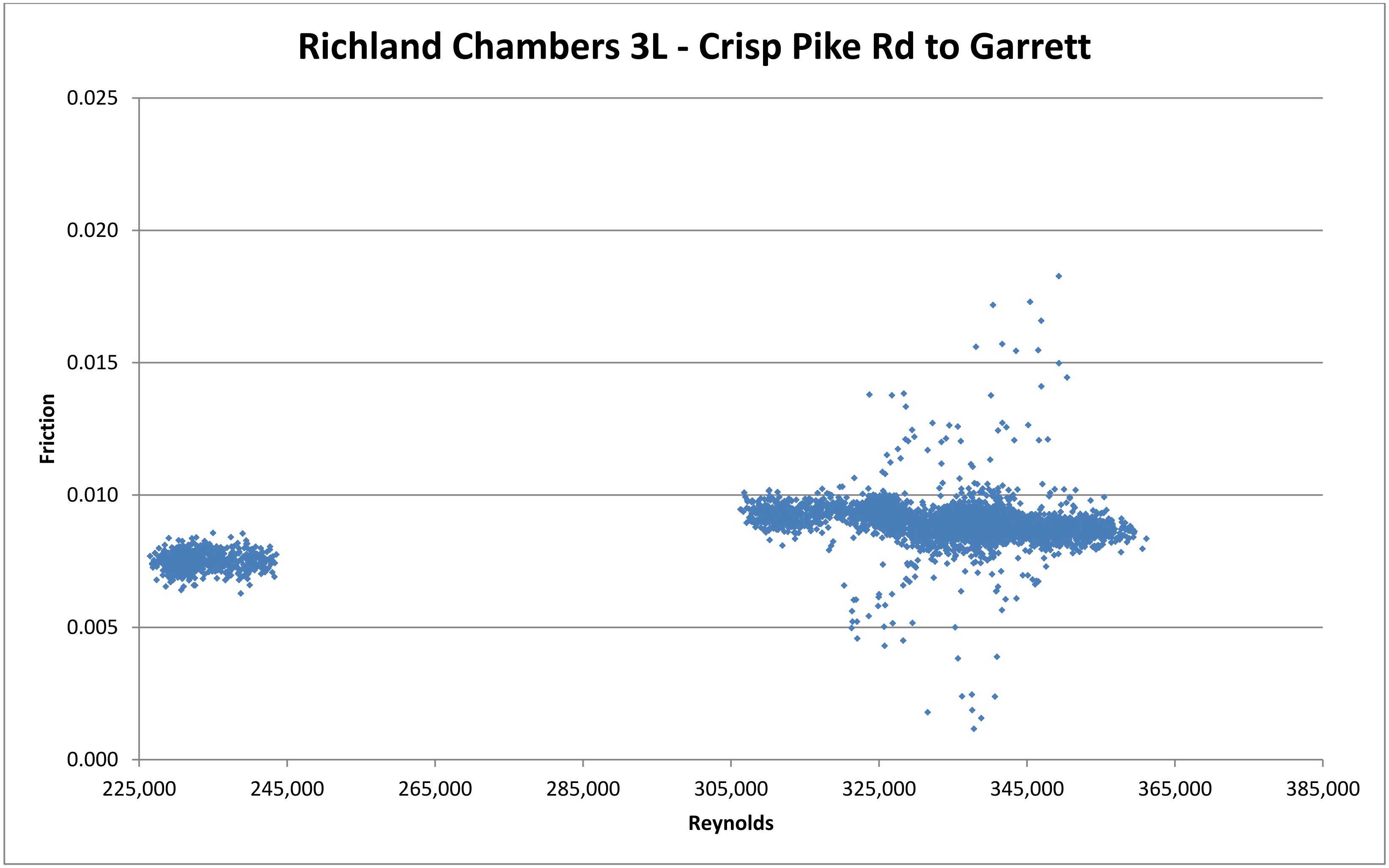
State Highway 85 to Richland Chambers Ennis Pump Station



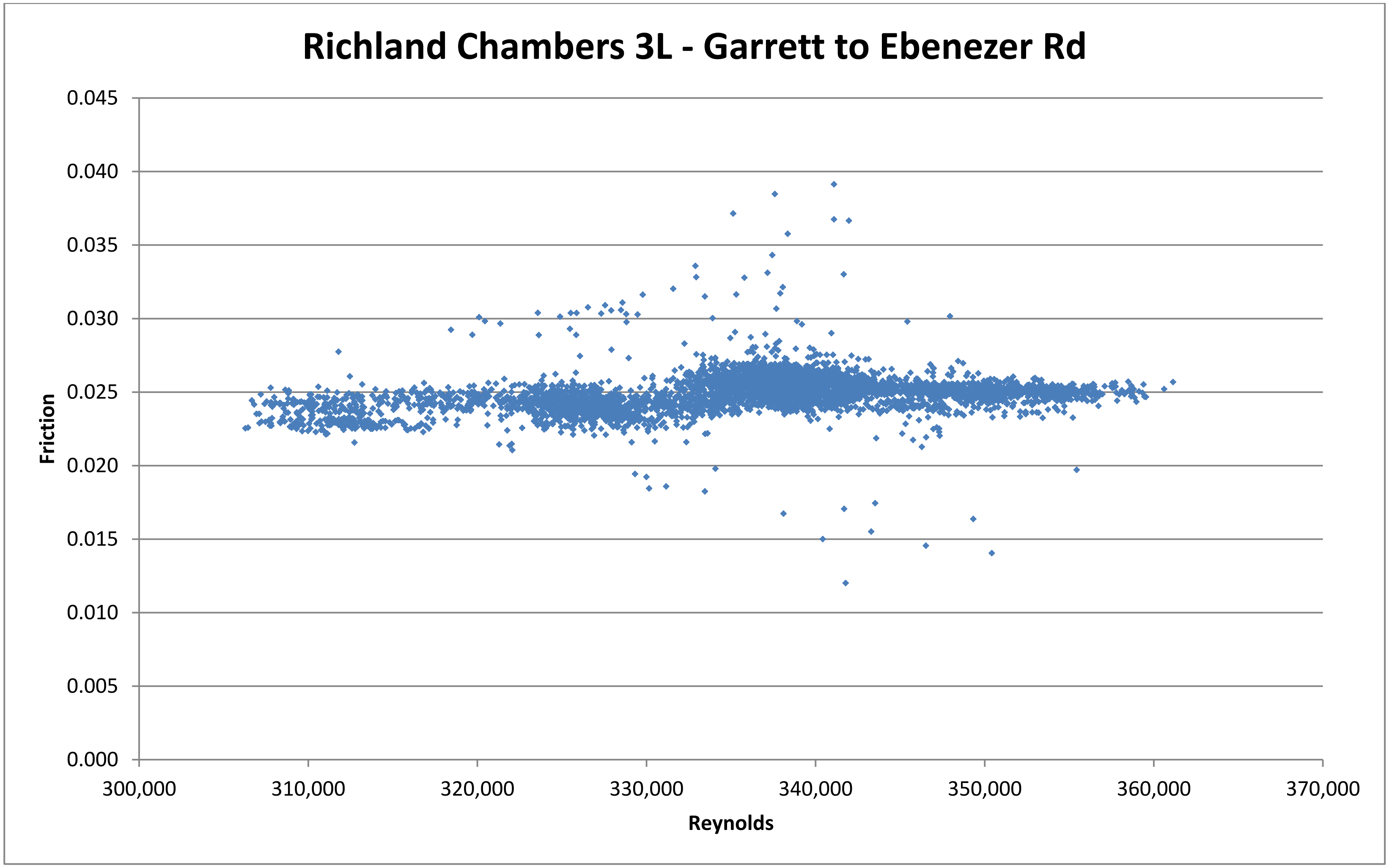
Richland Chambers Ennis Pump Station to Crisp Pike Road



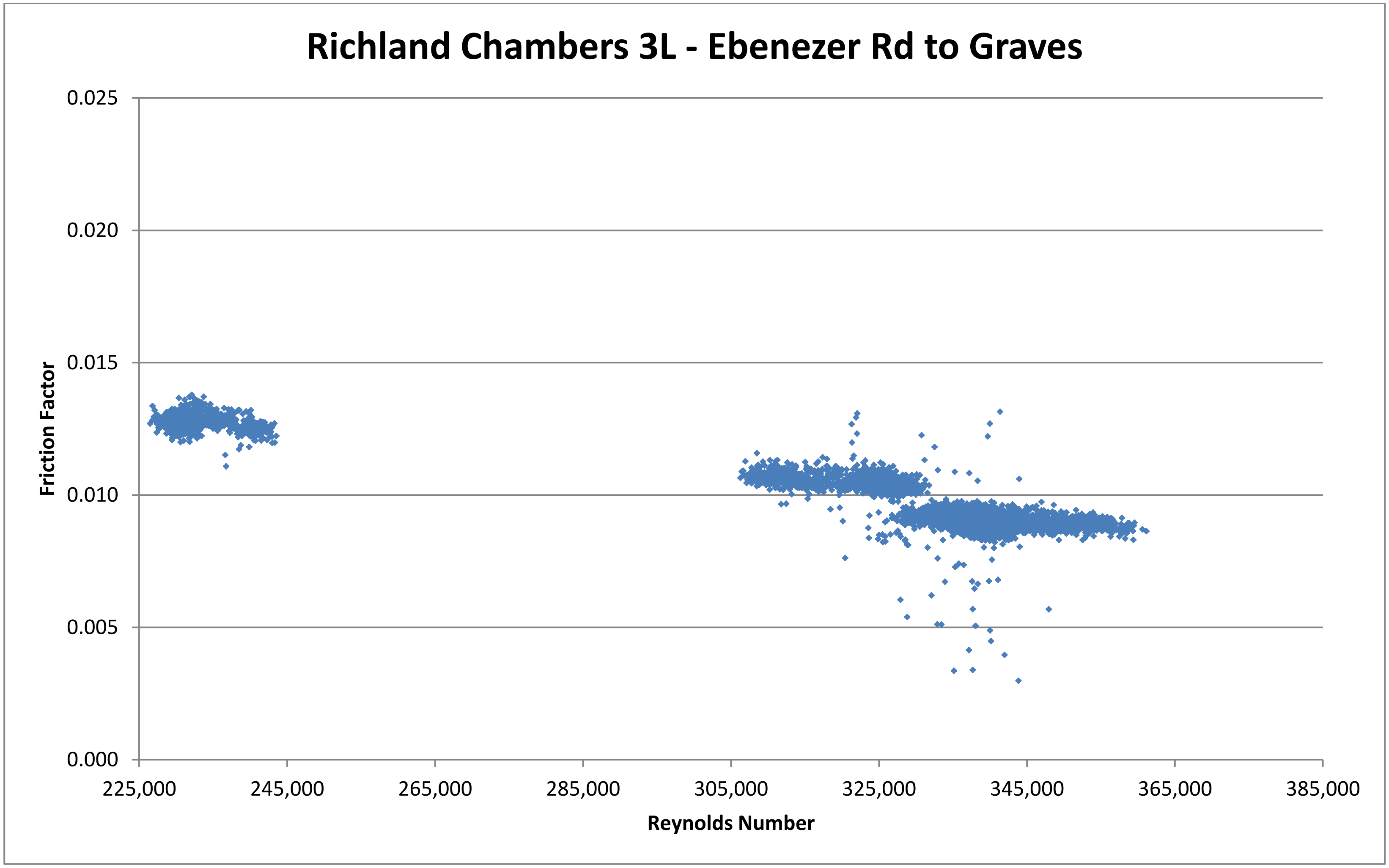
Crisp Pike Road to Garrett Road



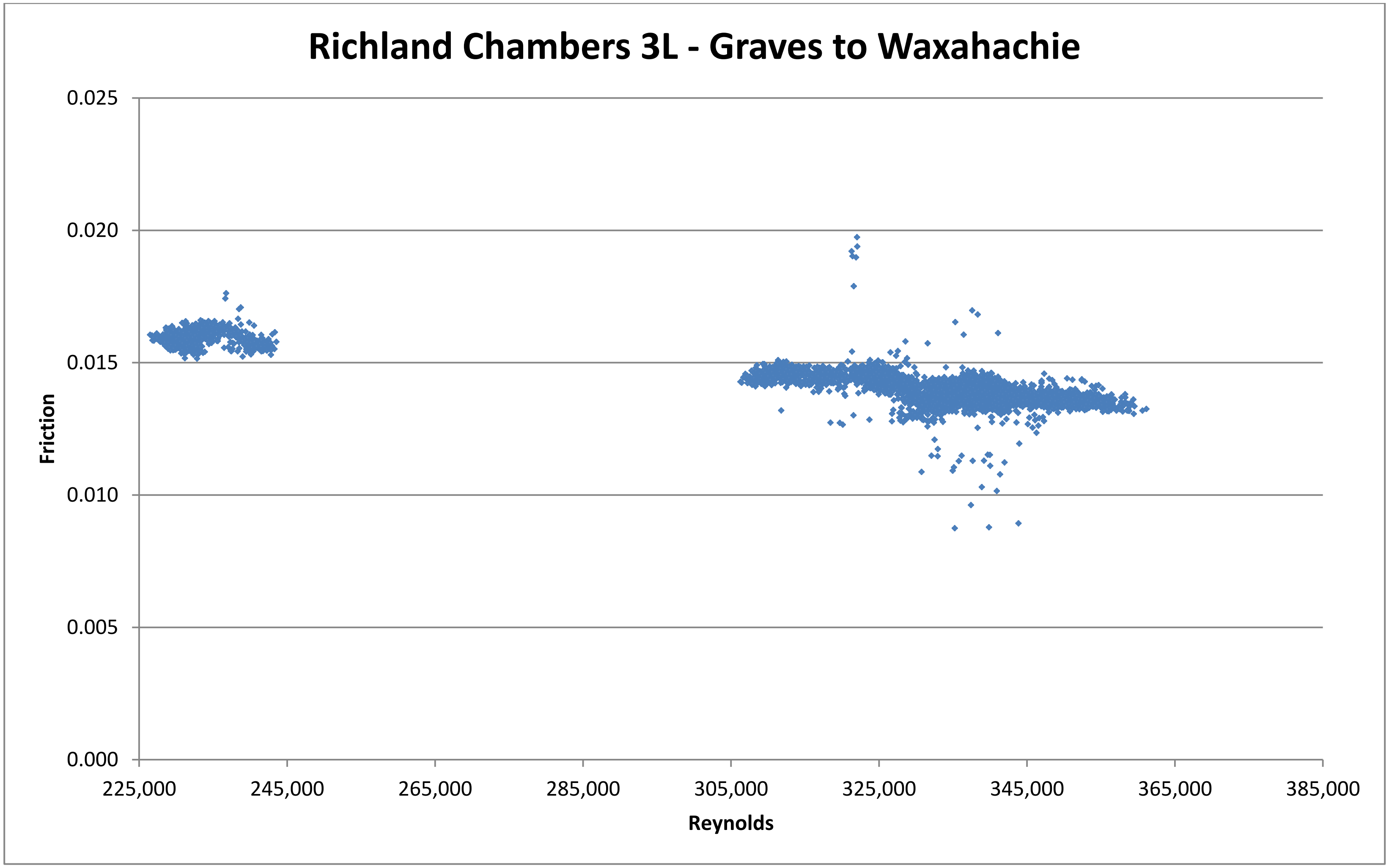
Garrett Road to Ebenezer Road



Ebenezer Road to Graves Road

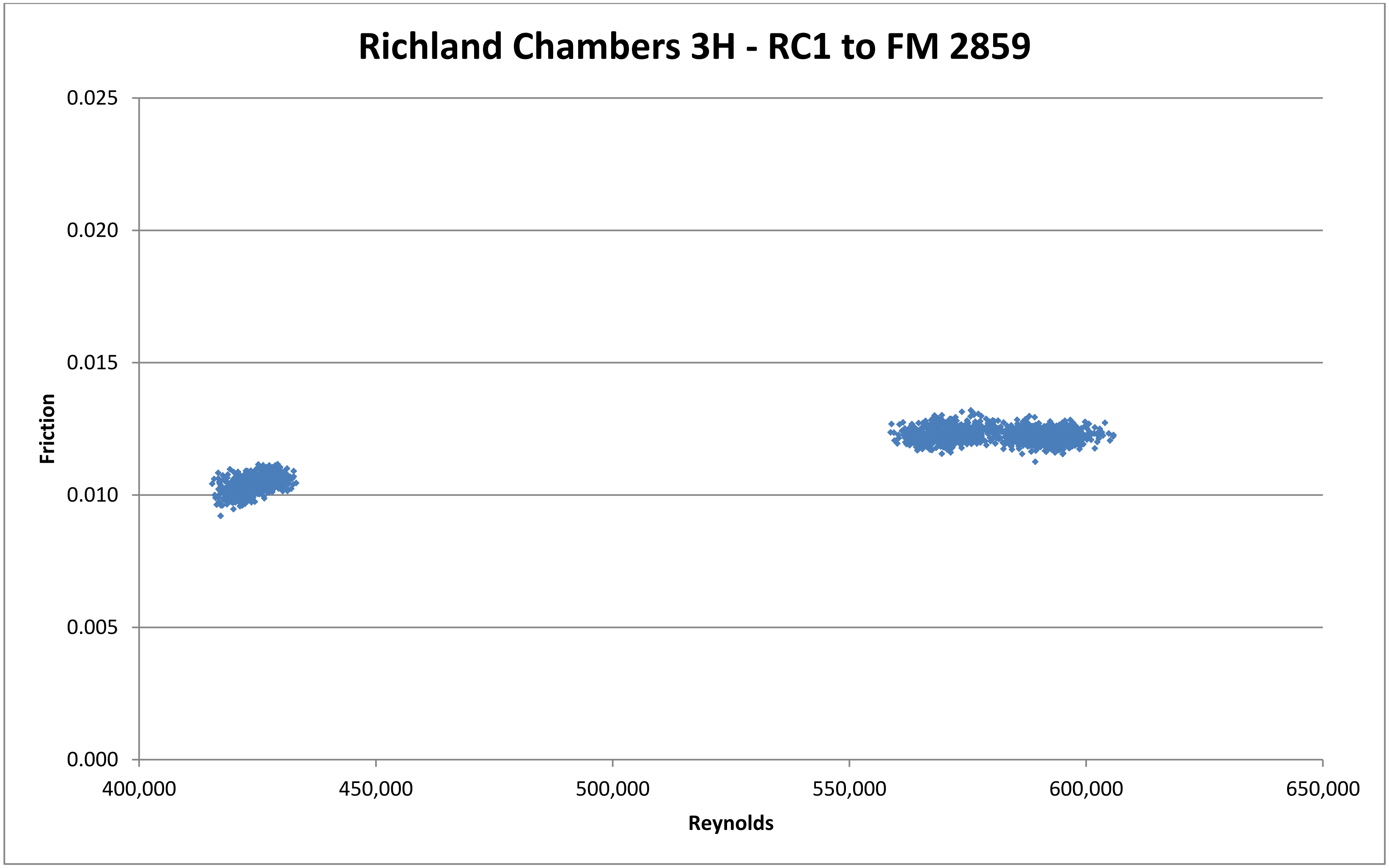


Graves Road to Richland Chambers Waxahachie Pump Station

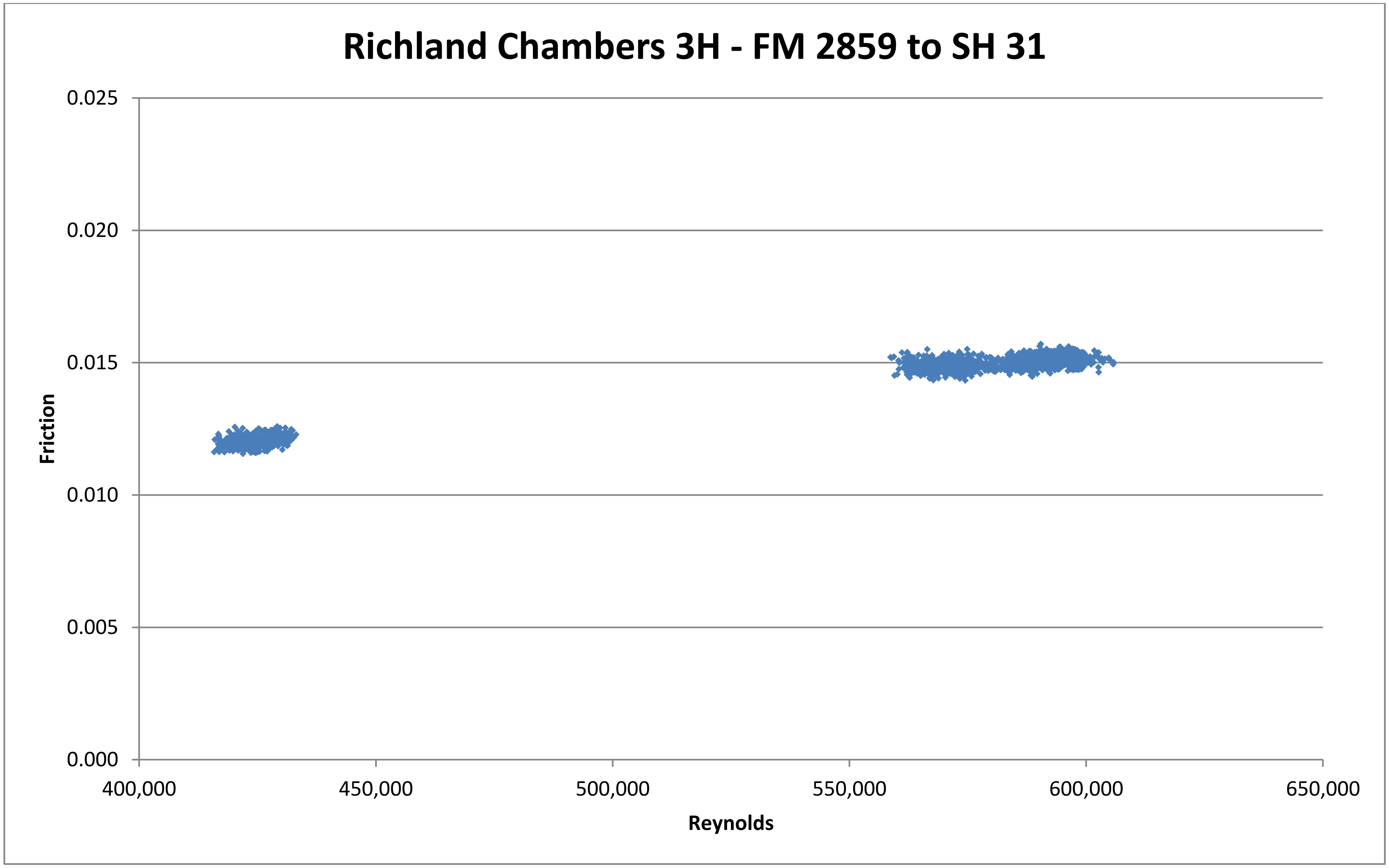


Richland Chambers 3H Pump Combination

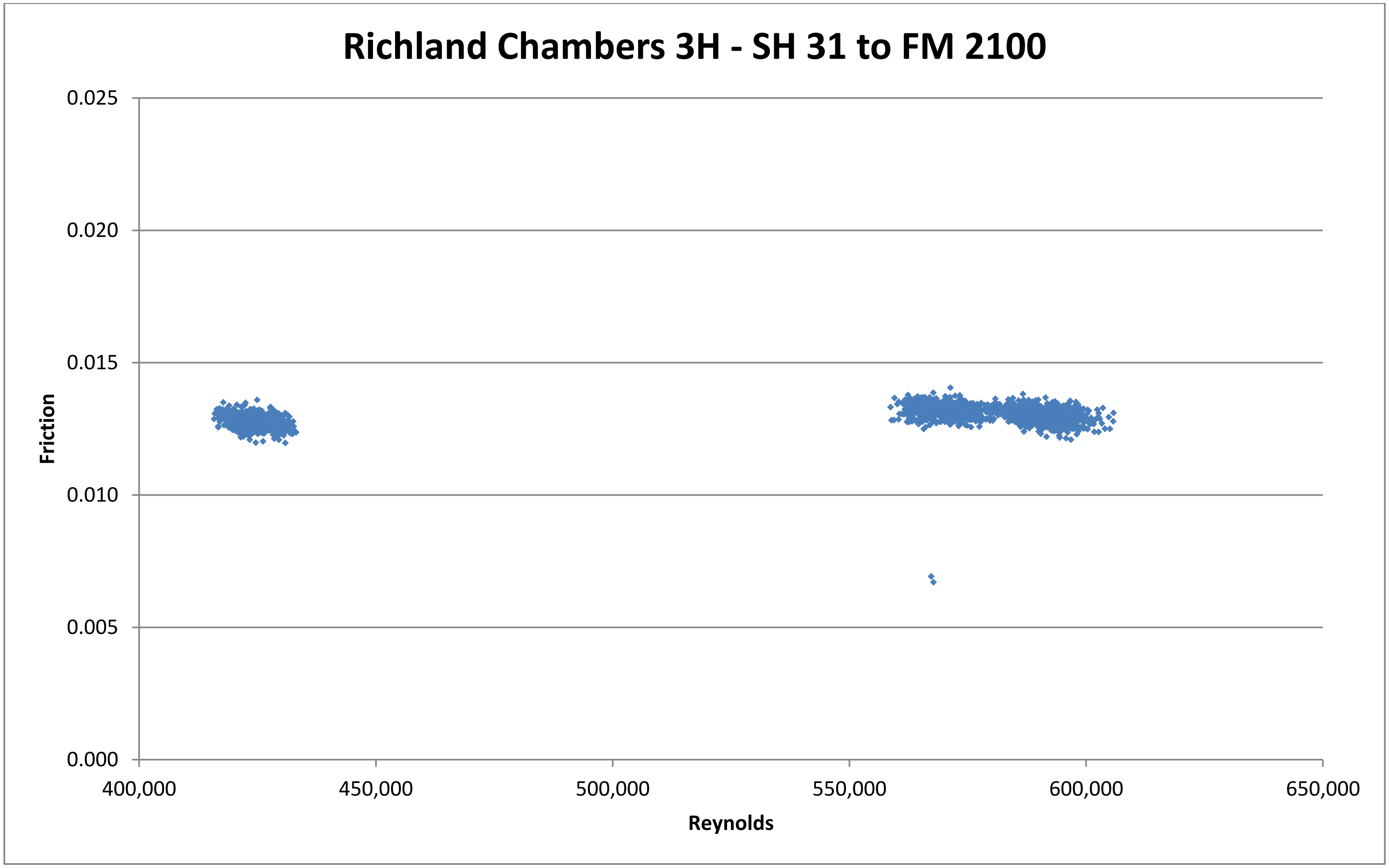
Richland Chambers Lake Pump Station to Farm-to-Market 2859



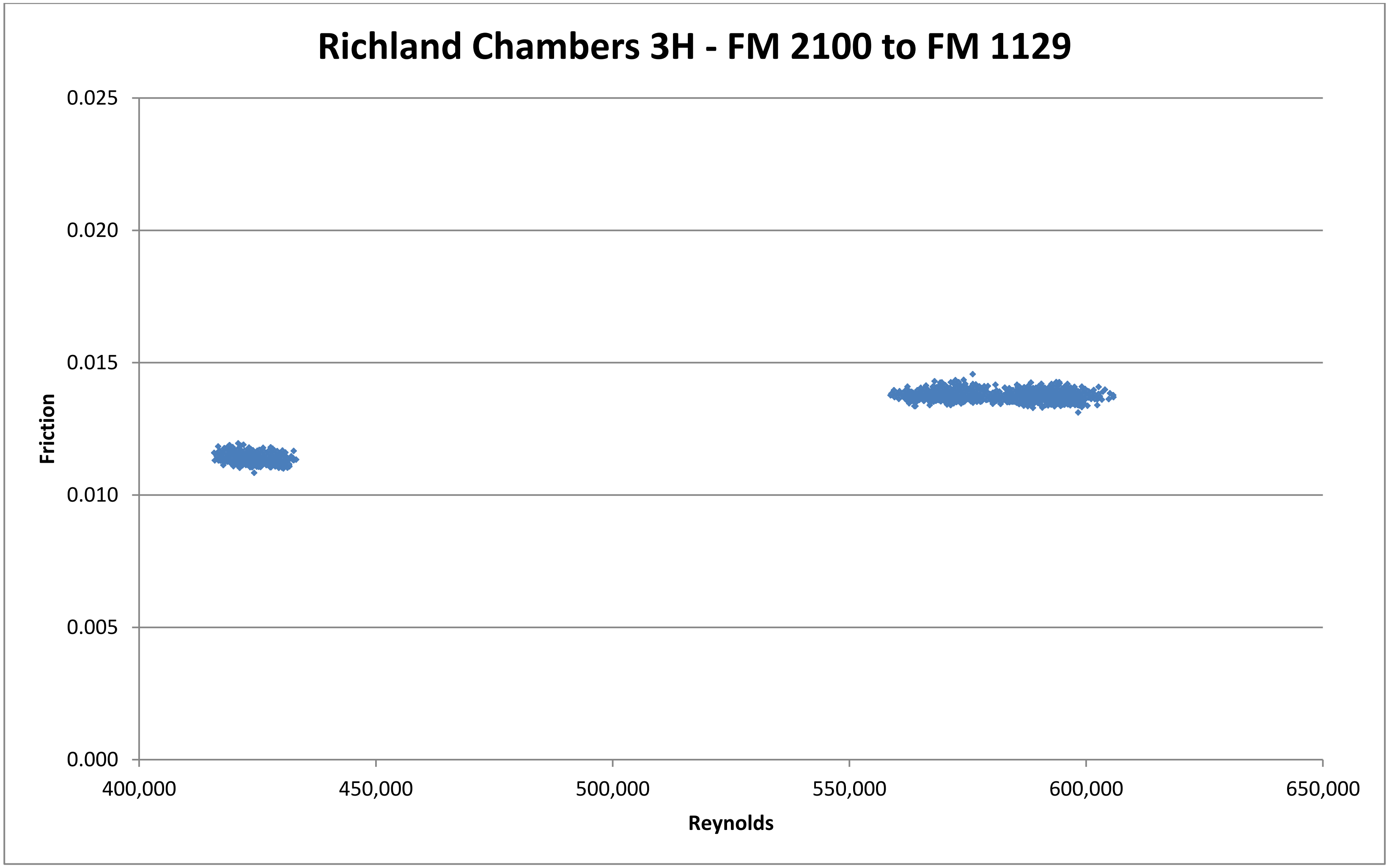
Farm-to-Market 2859 to State Highway 31



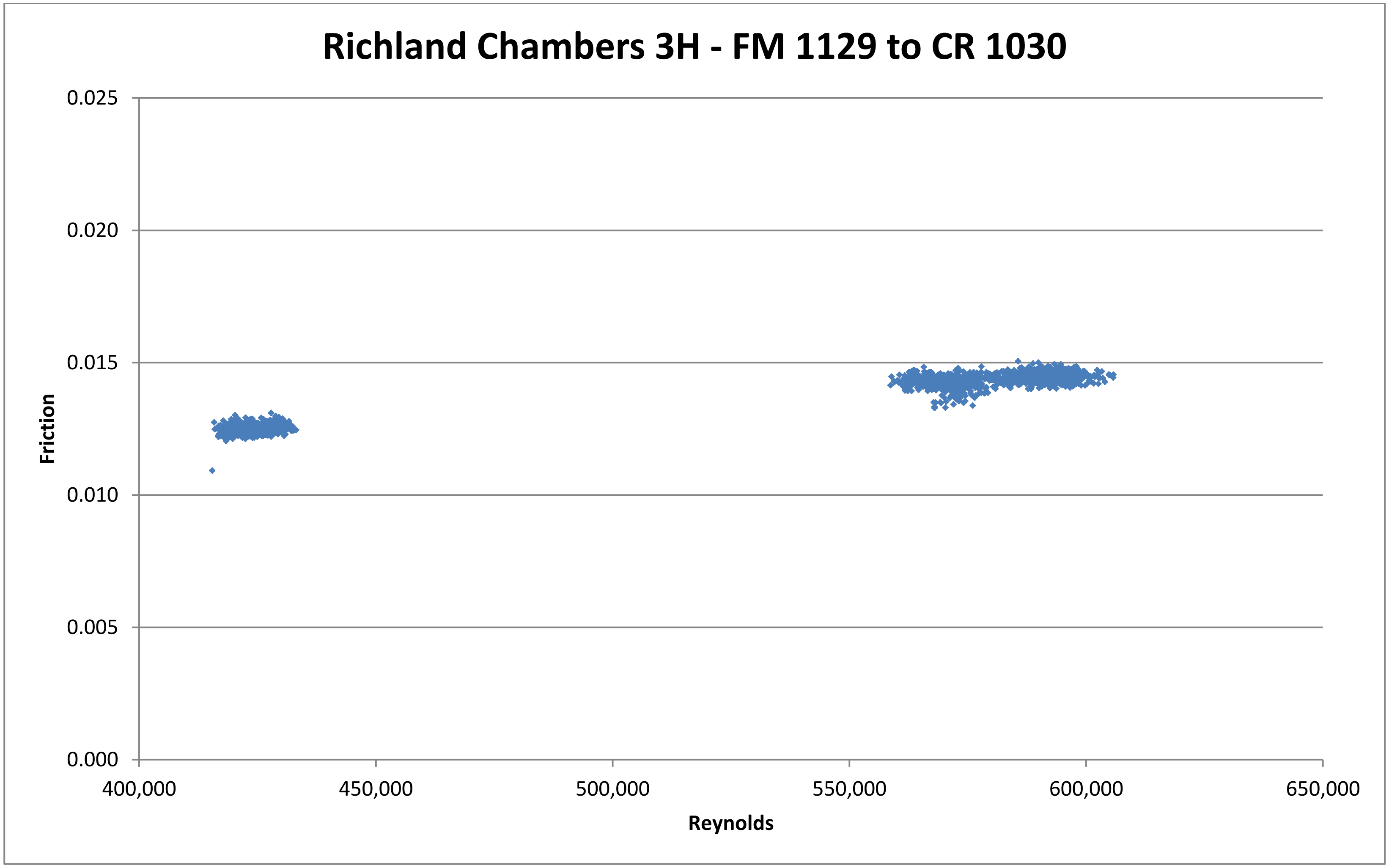
State Highway 31 to Farm-to-Market 2100



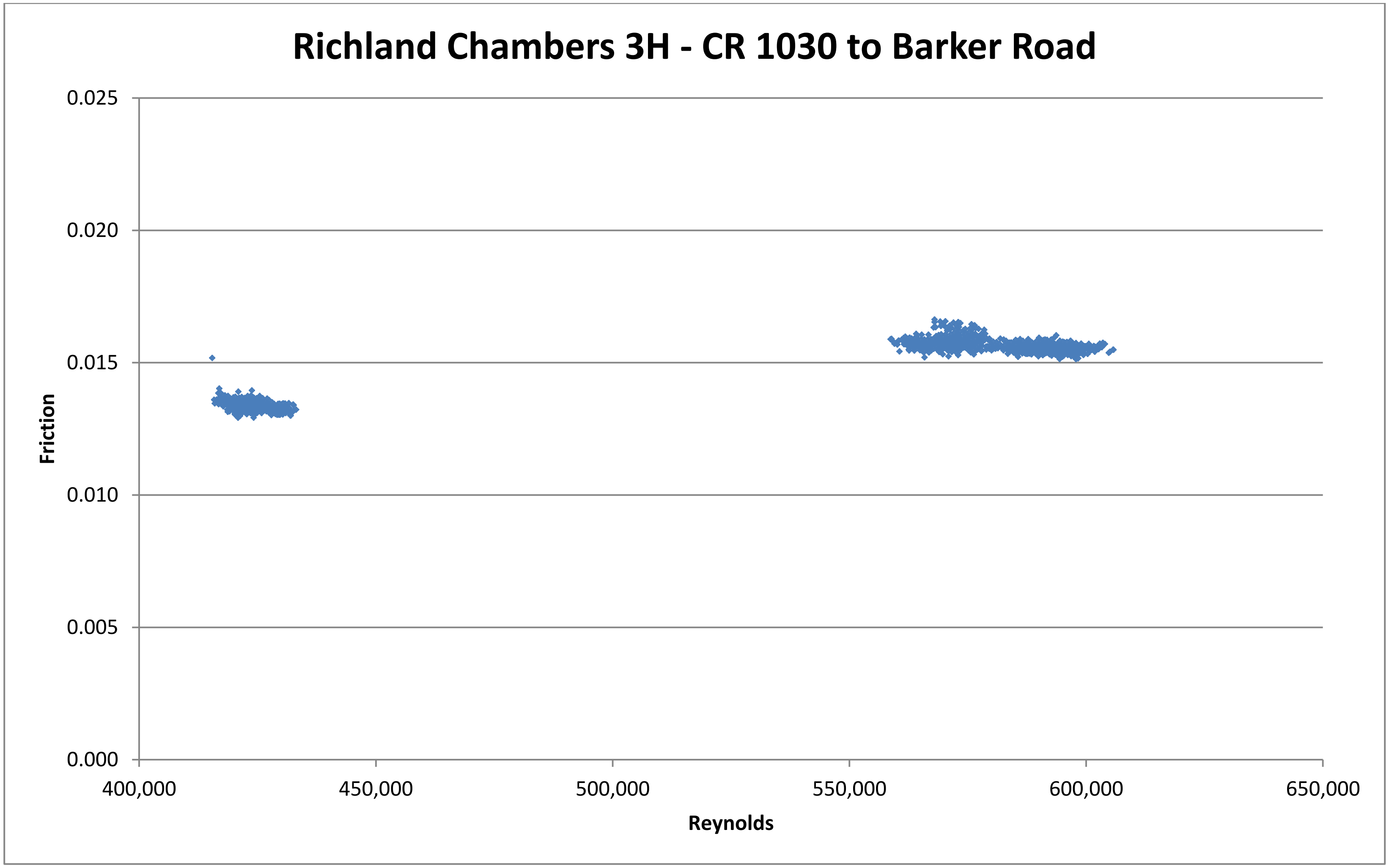
Farm-to-Market 2100 to Farm-to-Market 1129



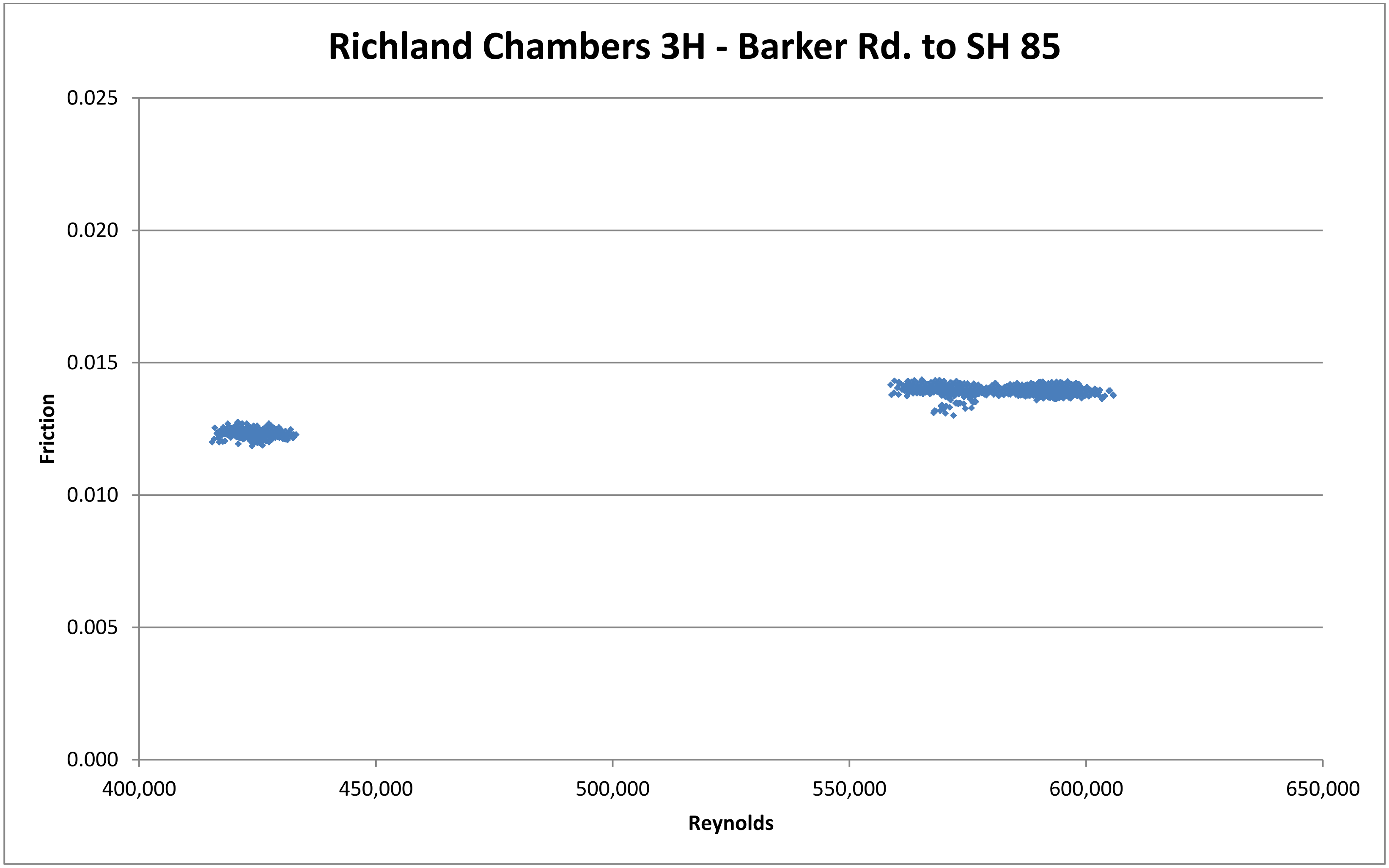
Farm-to-Market 1129 to County Road 1030



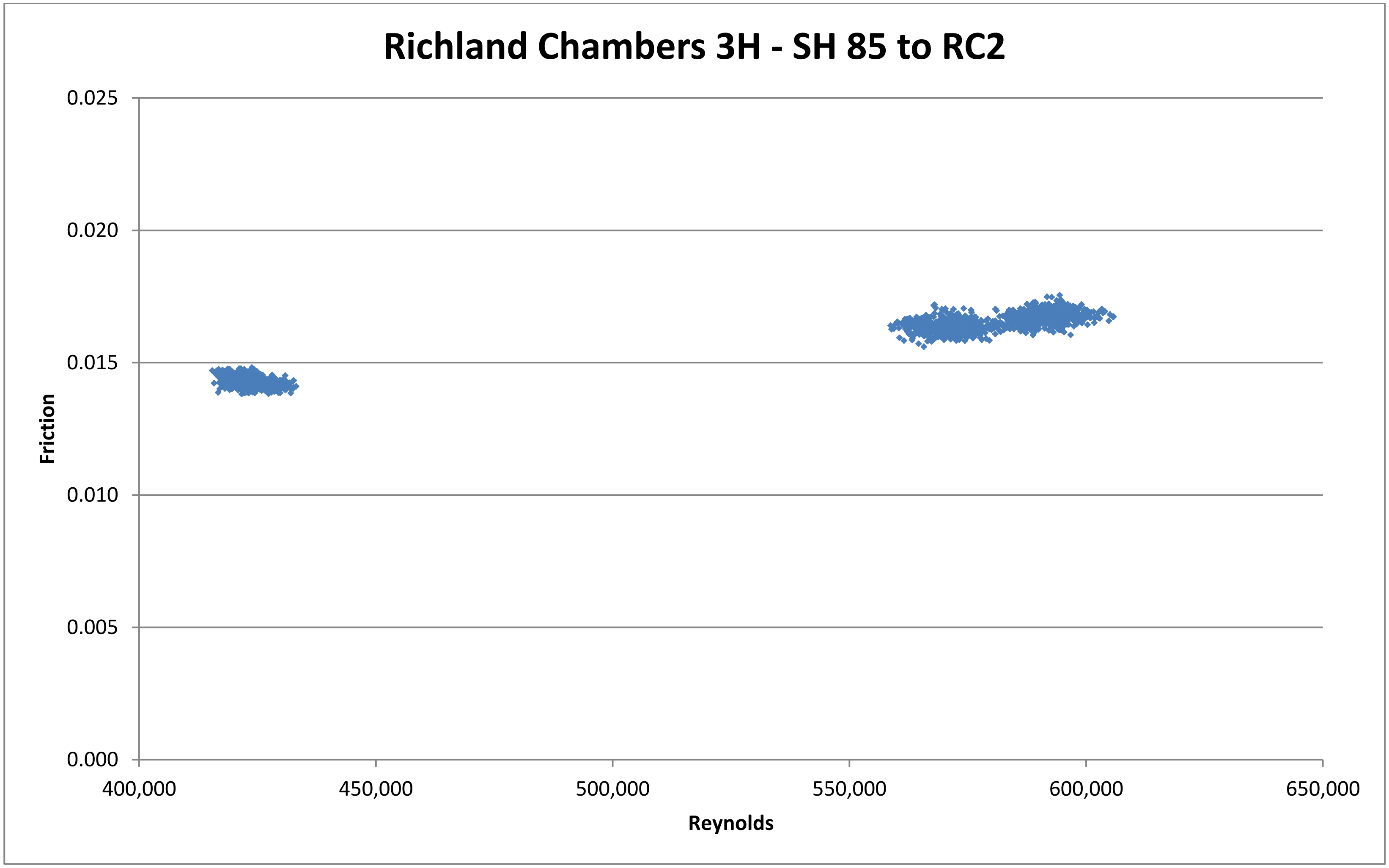
County Road 1030 to Barker Road



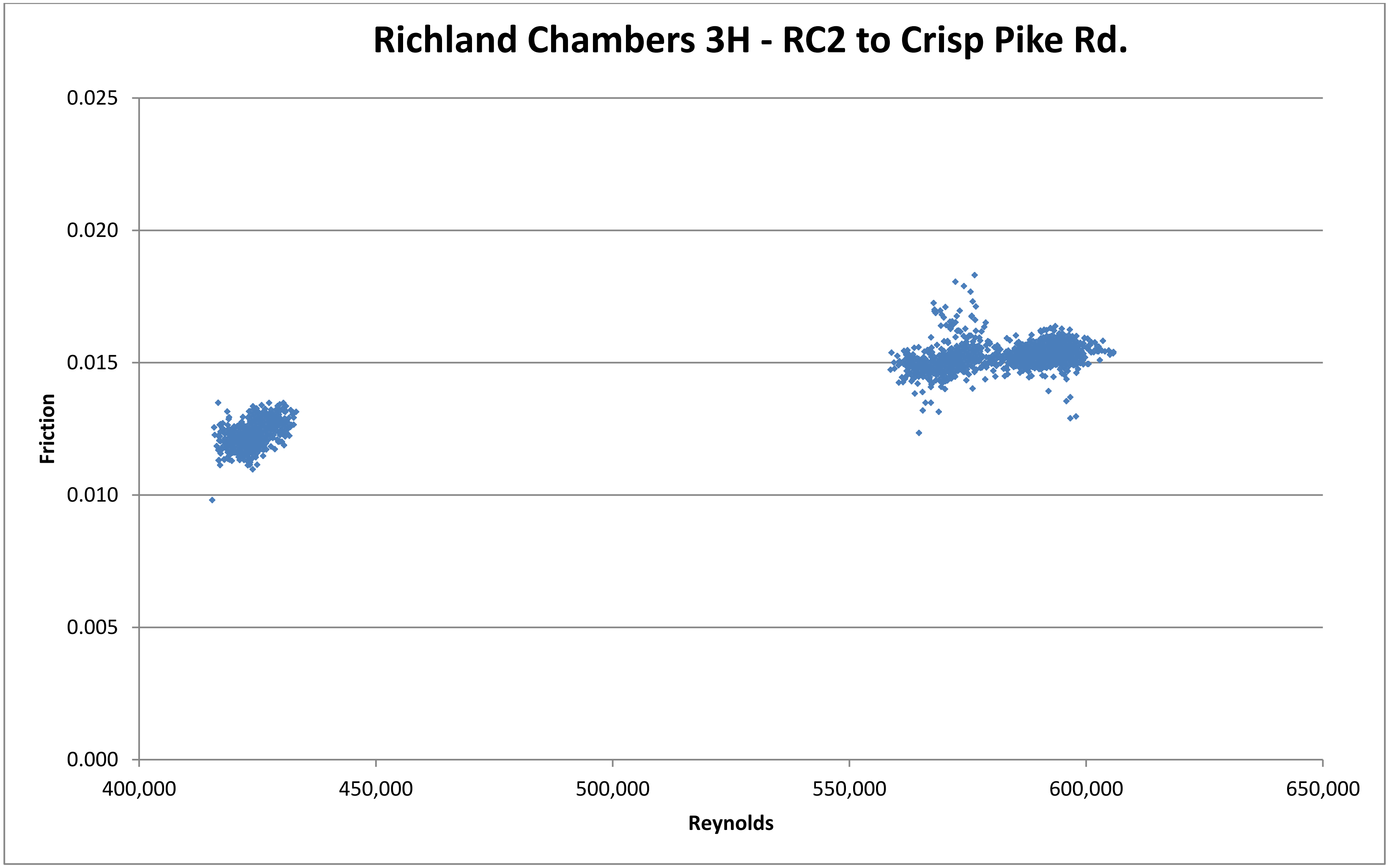
Barker Road to State Highway 85



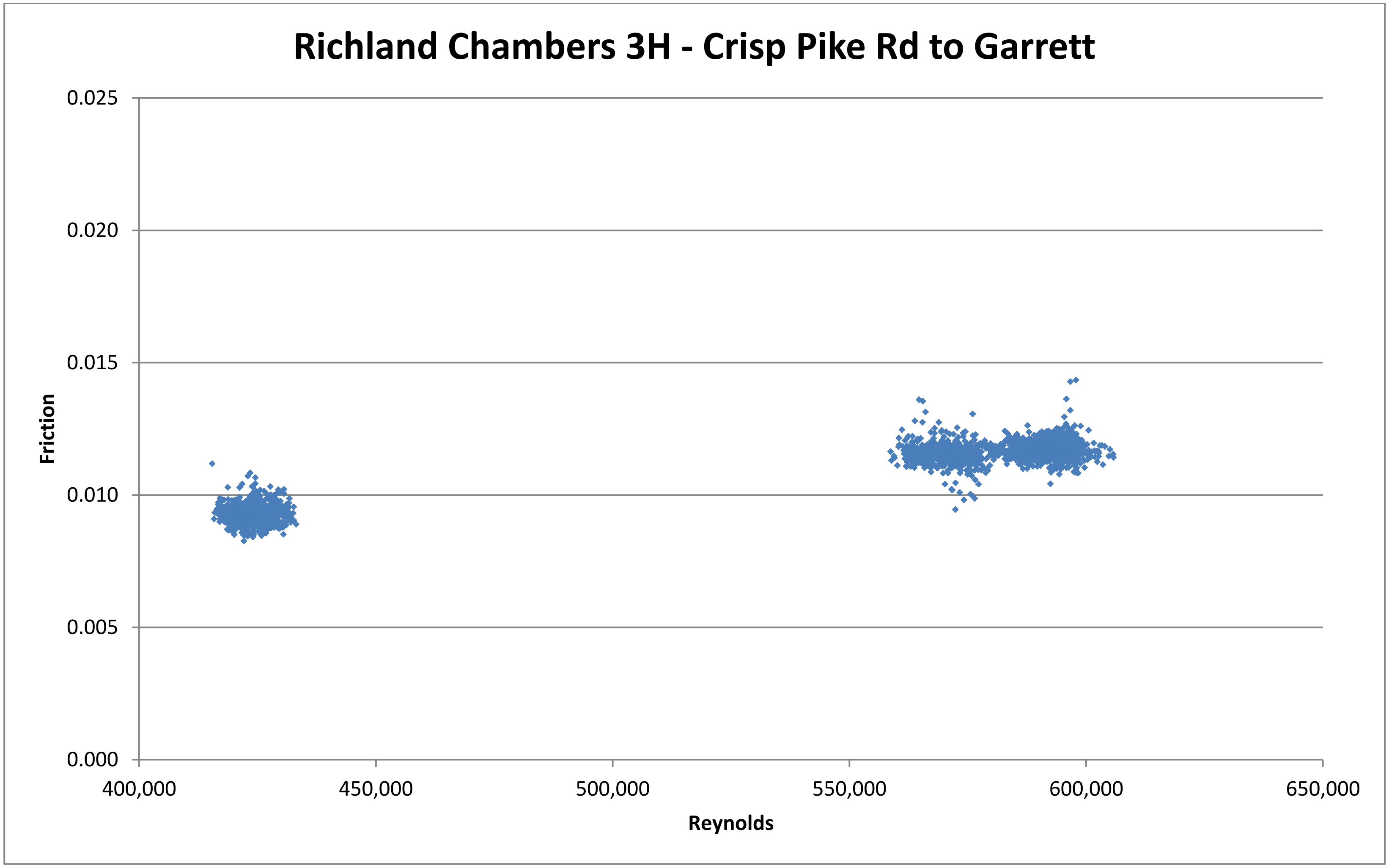
State Highway 85 to Richland Chambers Ennis Pump Station



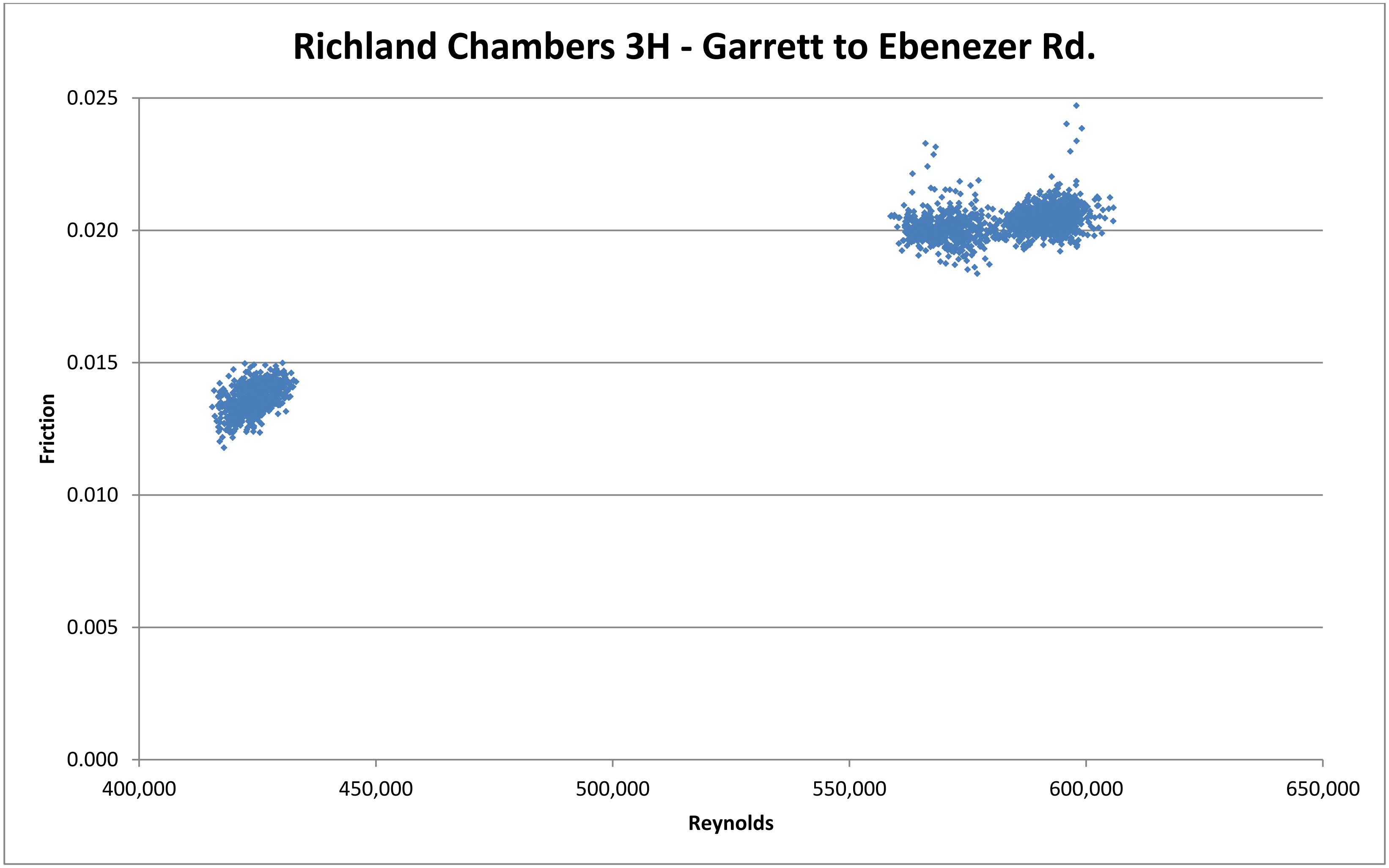
Richland Chambers Ennis Pump Station to Crisp Pike Road



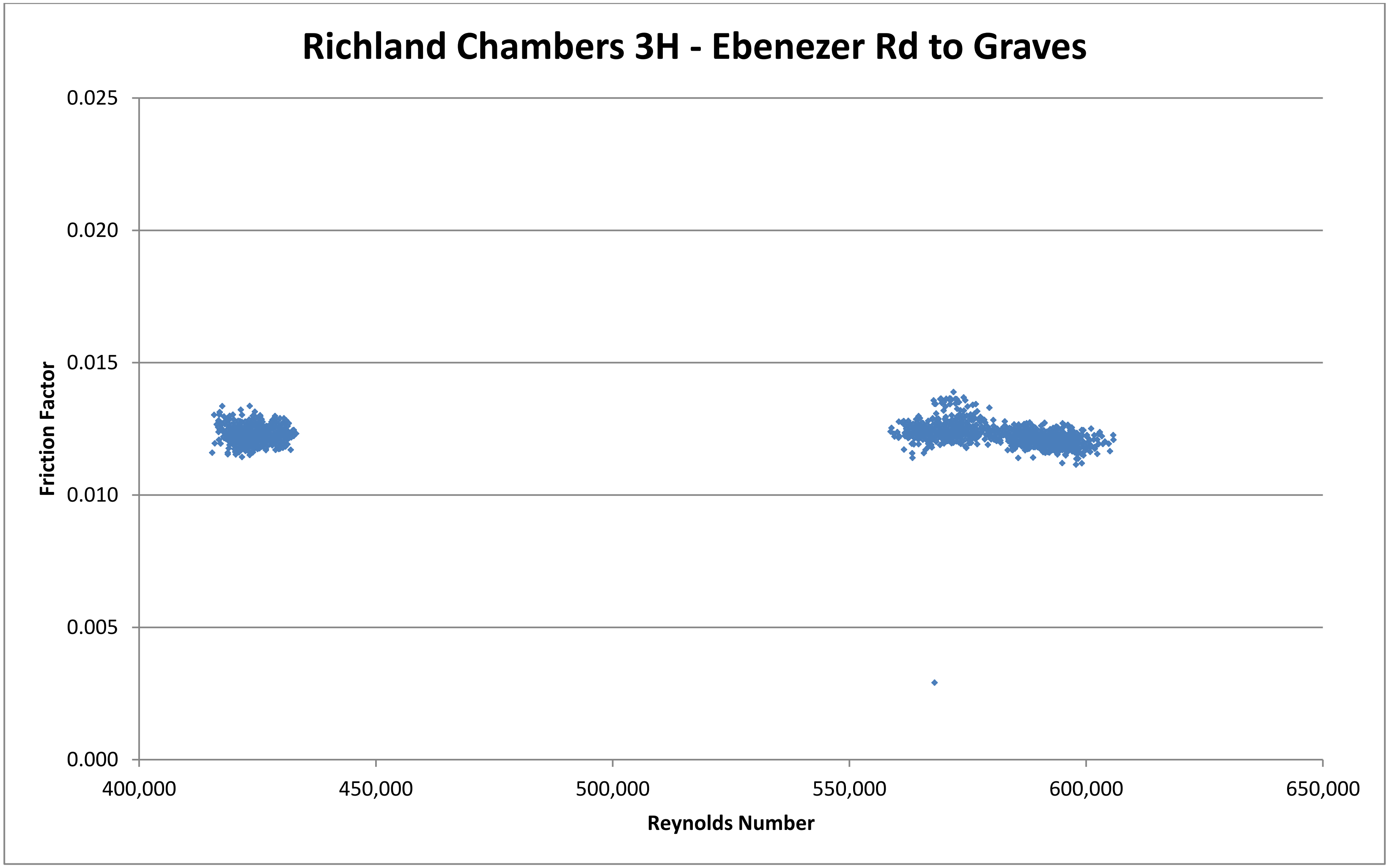
Crisp Pike Road to Garrett Road



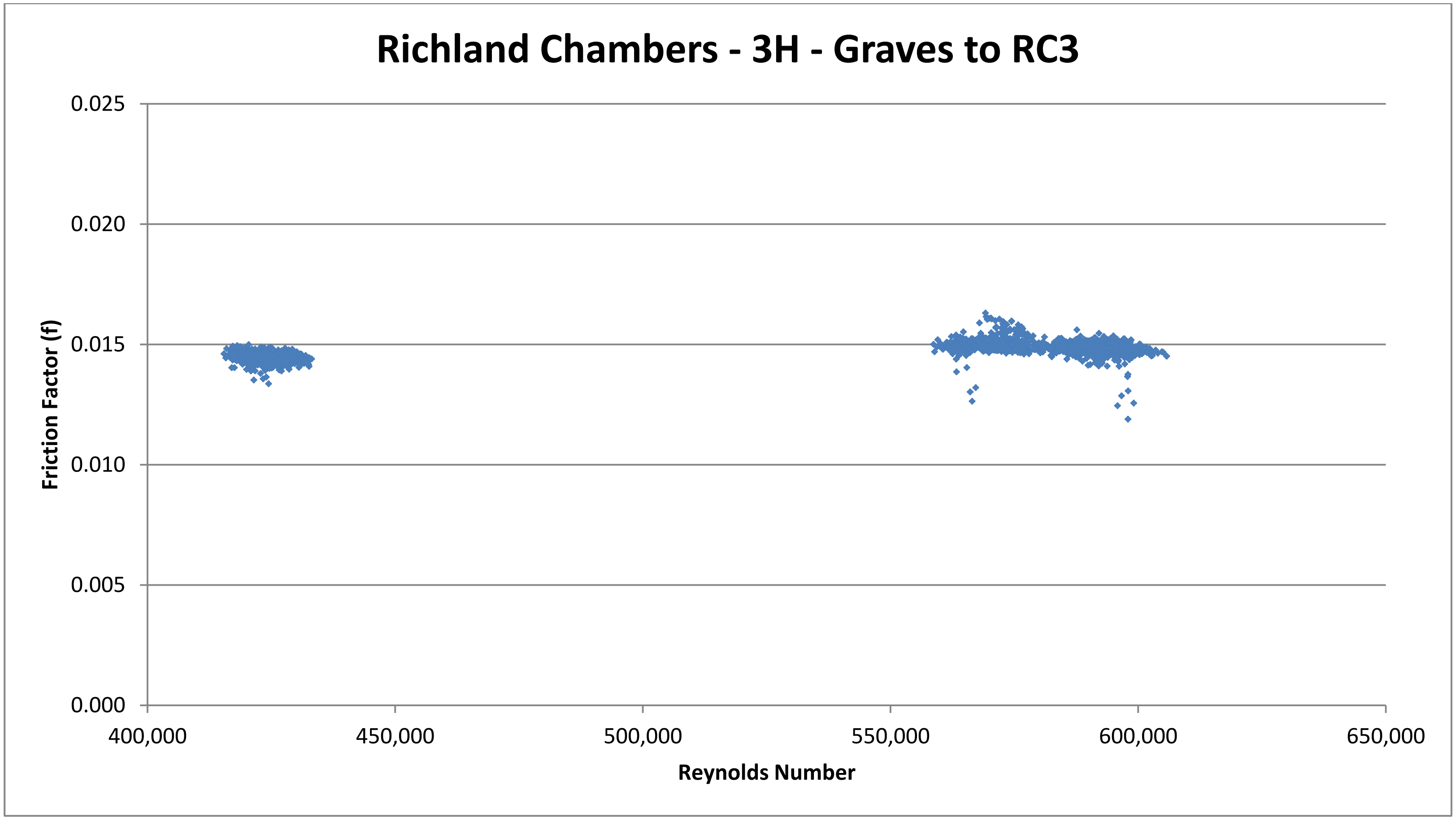
Garrett Road to Ebenezer Road



Ebenezer Road to Graves Road

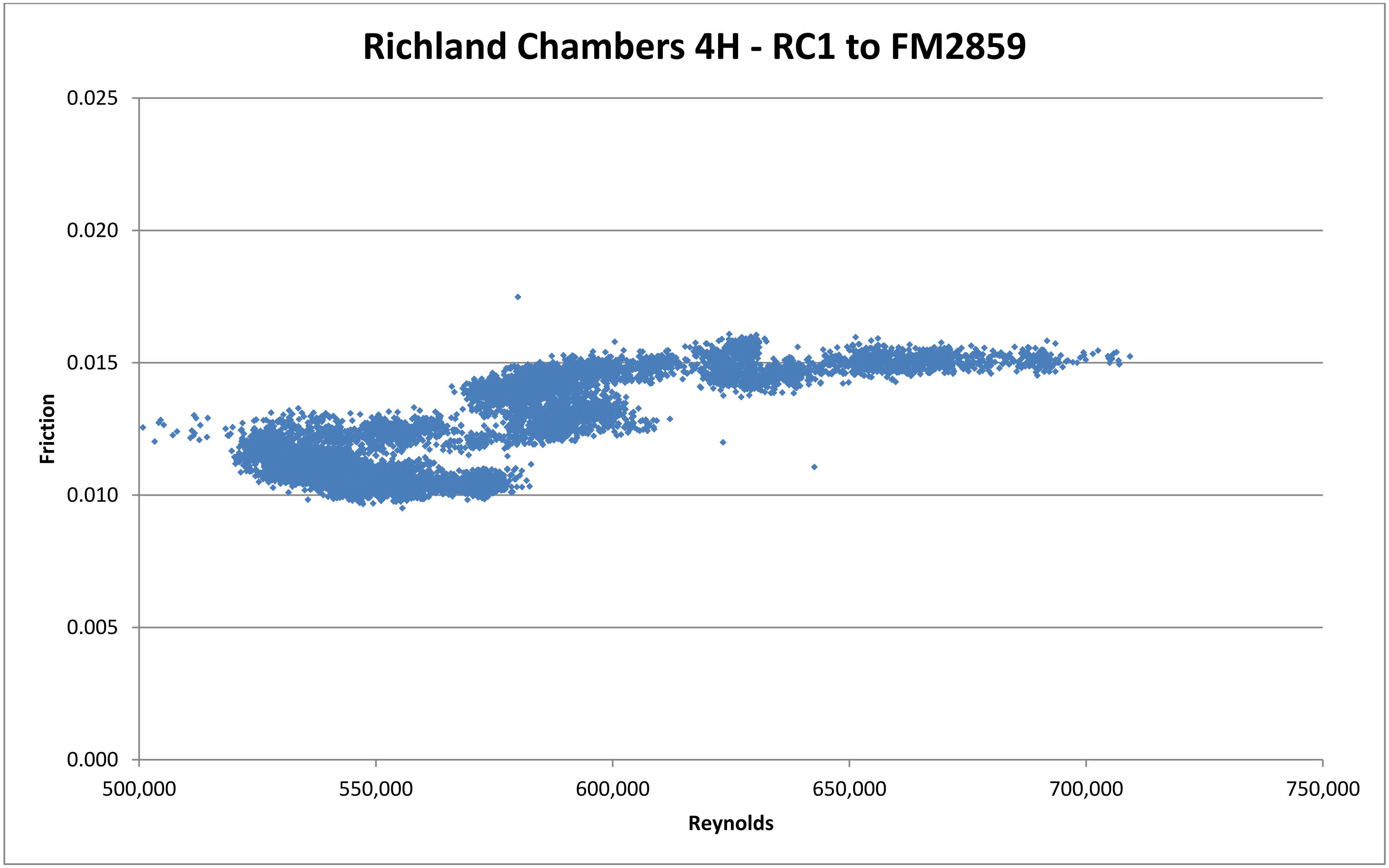


Graves Road to Richland Chambers Waxahachie Pump Station

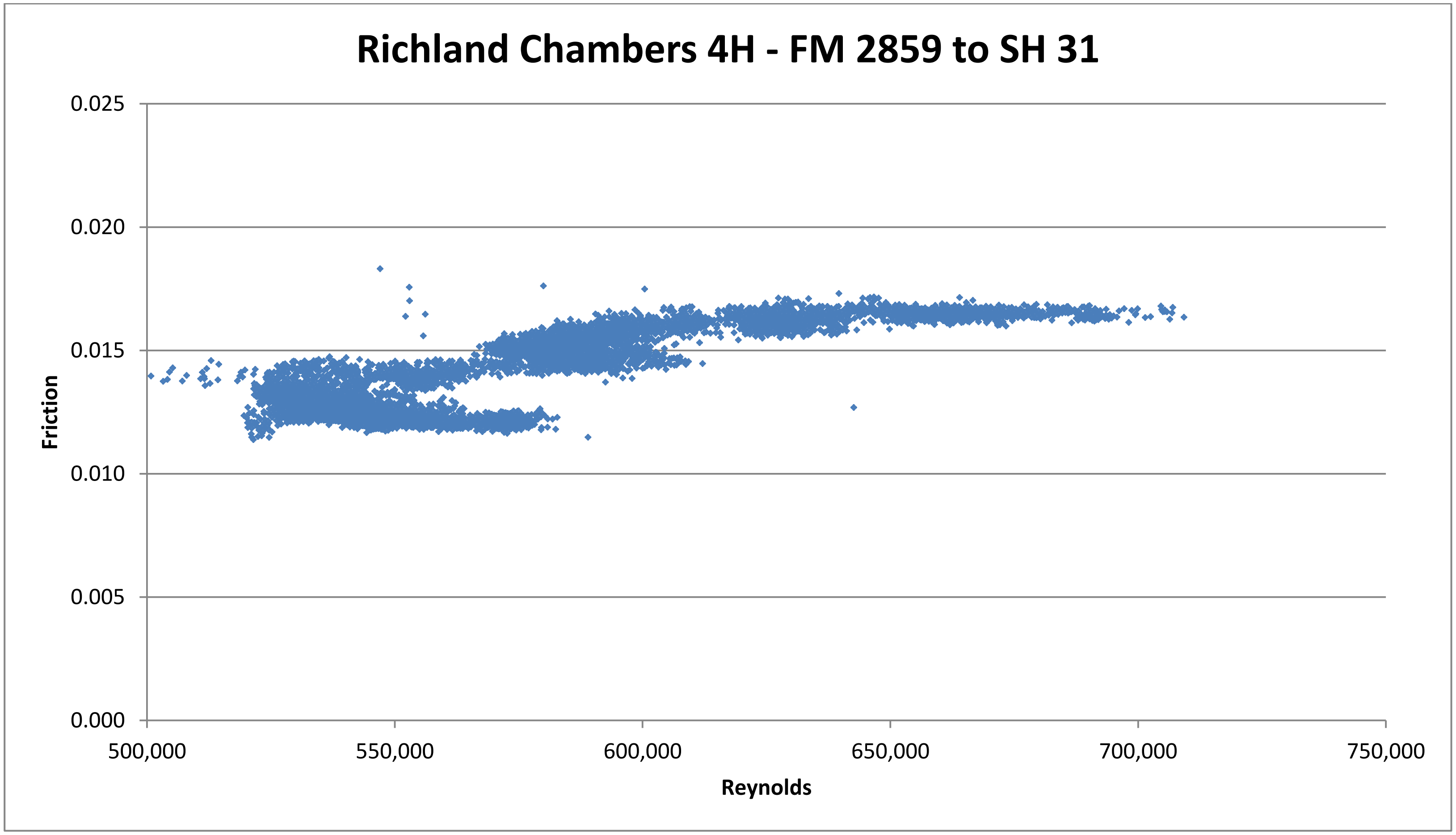


Richland Chambers 4H Pump Combination

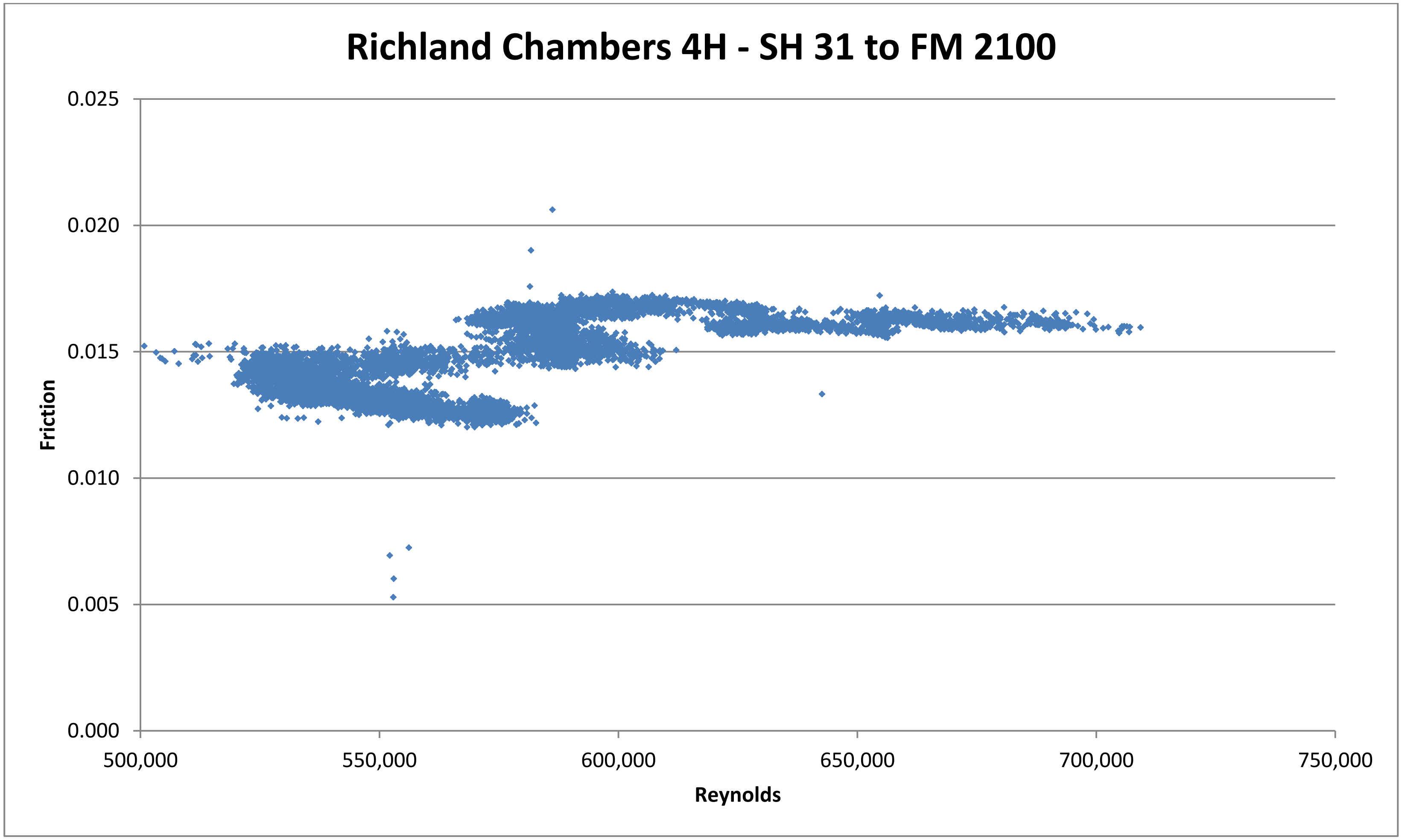
Richland Chambers Lake Pump Station to Farm-to-Market 2859


Richland Chambers 4H - RC1 to FM2859
0.025
0.020
0.015
0.010
0.005
0.000
Friction
500,000
550,000
600,000
650,000
700,000
750,000
Reynolds

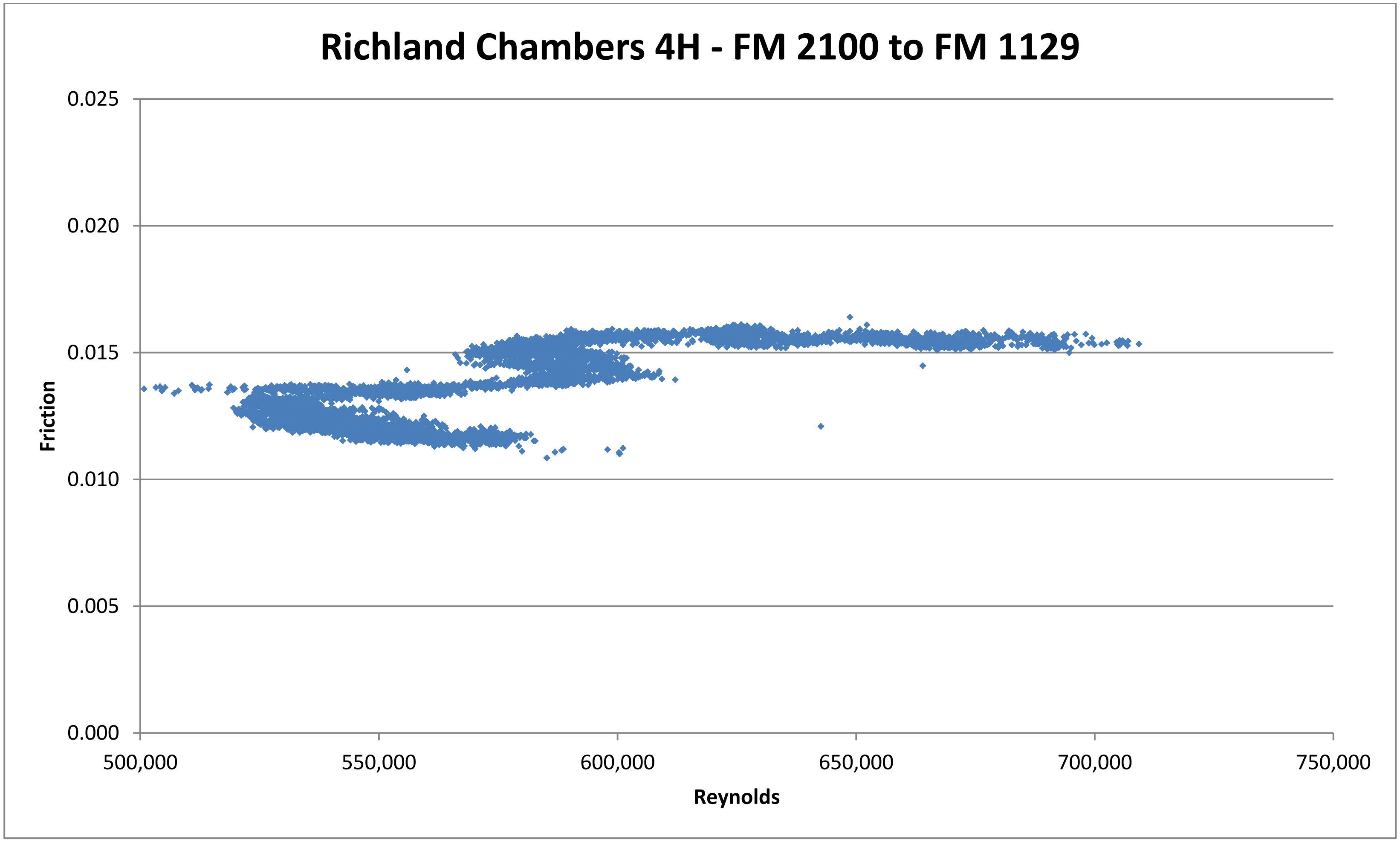
Farm-to-Market 2859 to State Highway 31



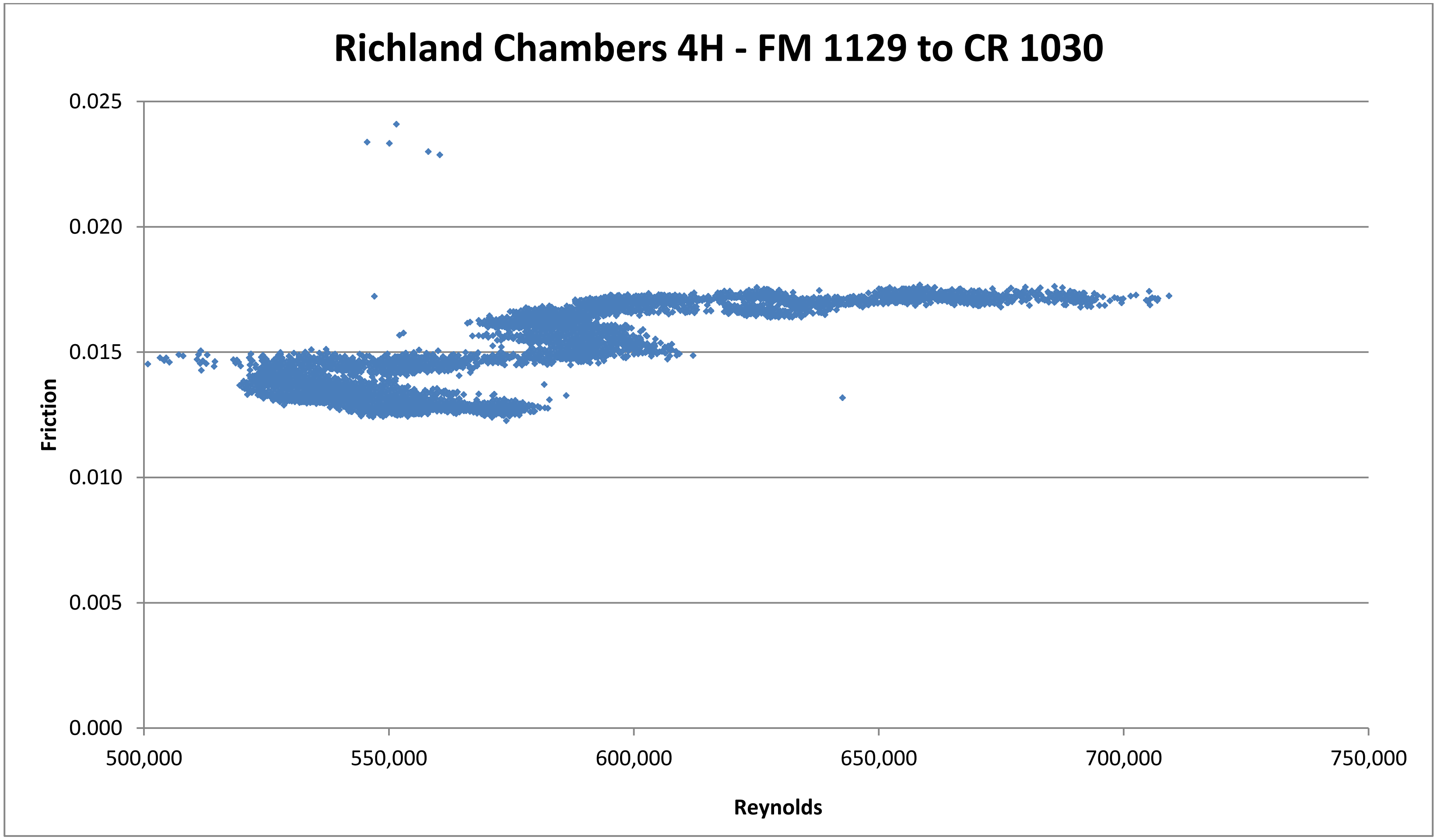
State Highway 31 to Farm-to-Market 2100



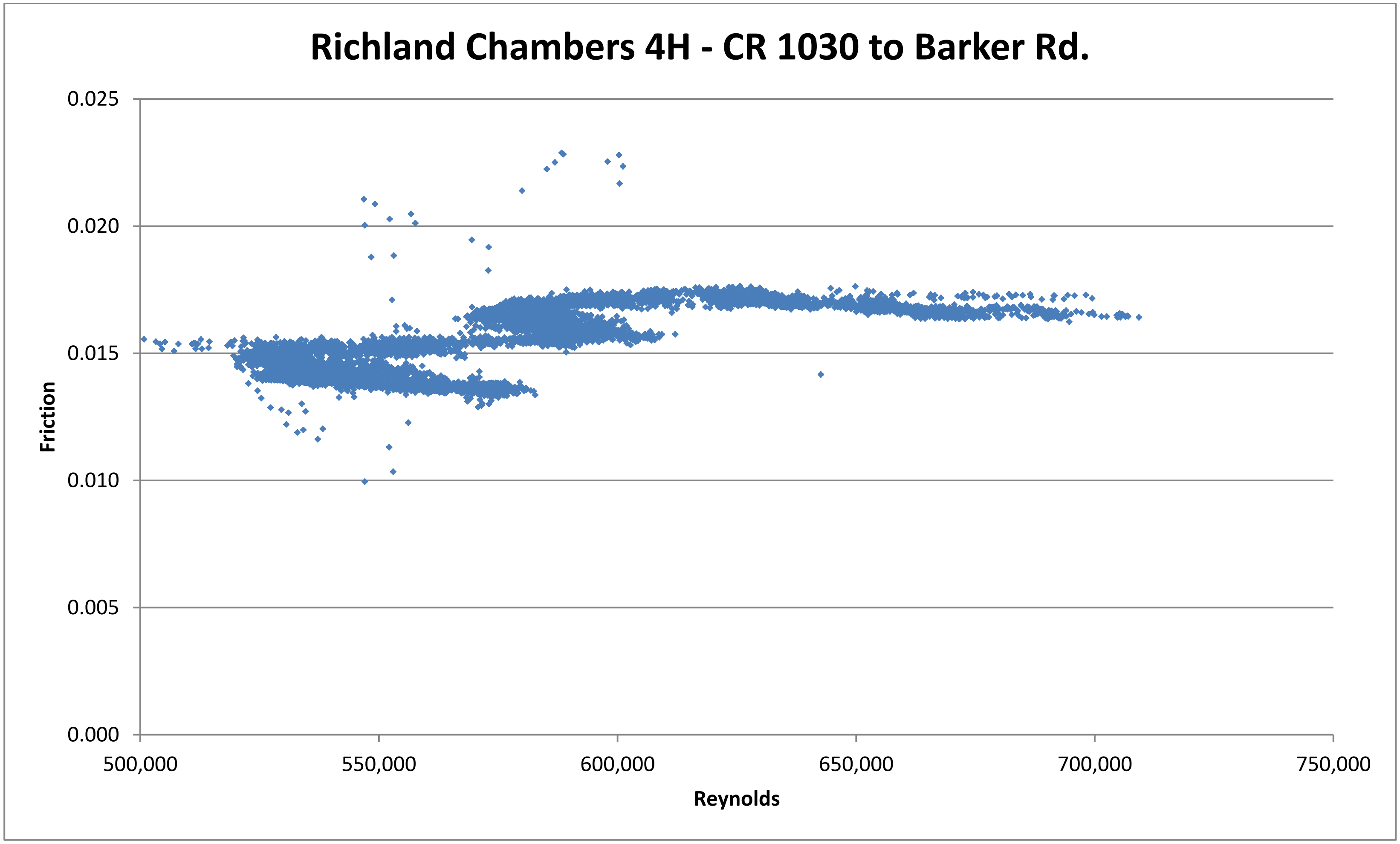
Farm-to-Market 2100 to Farm-to-Market 1129



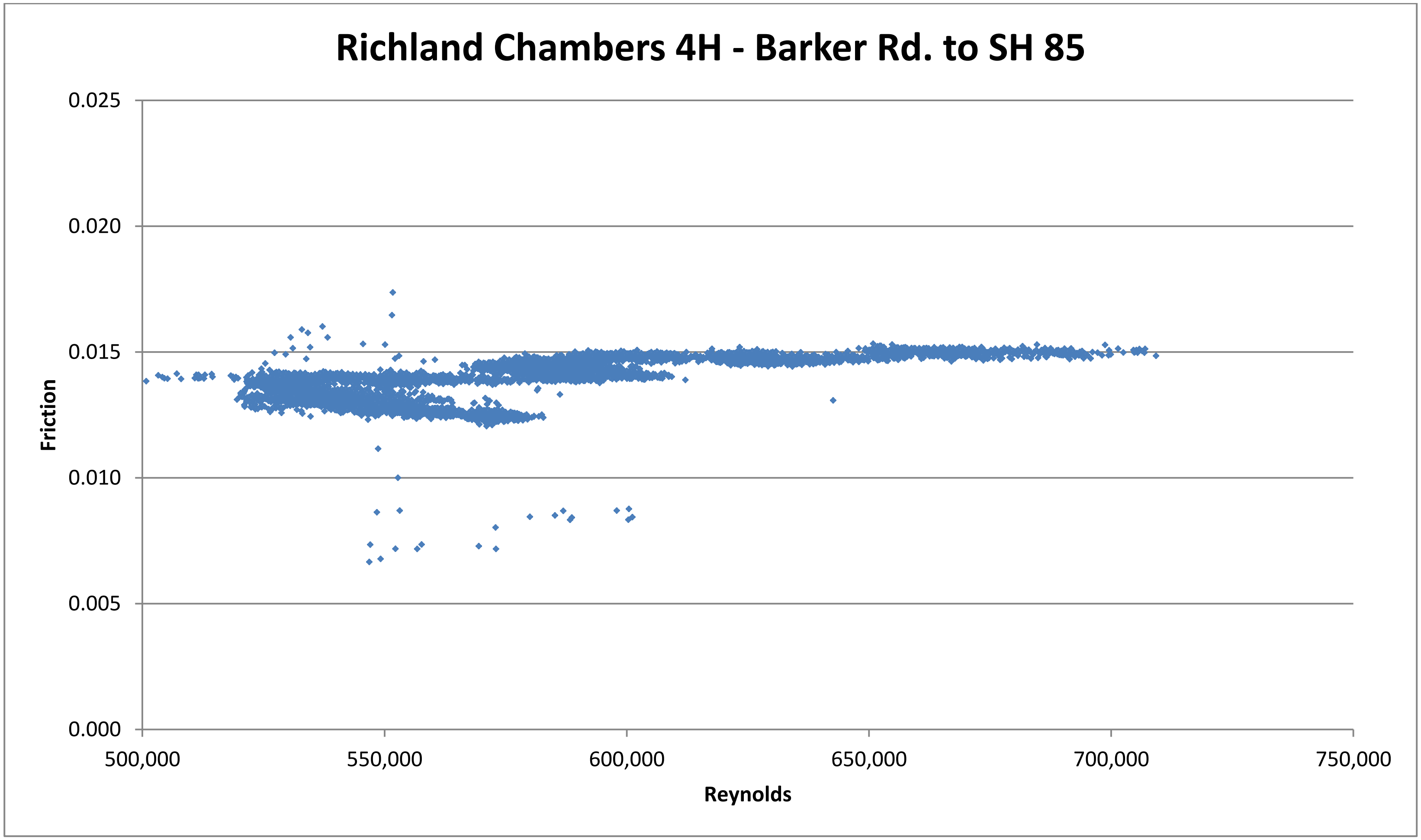
Farm-to-Market 1129 to County Road 1030



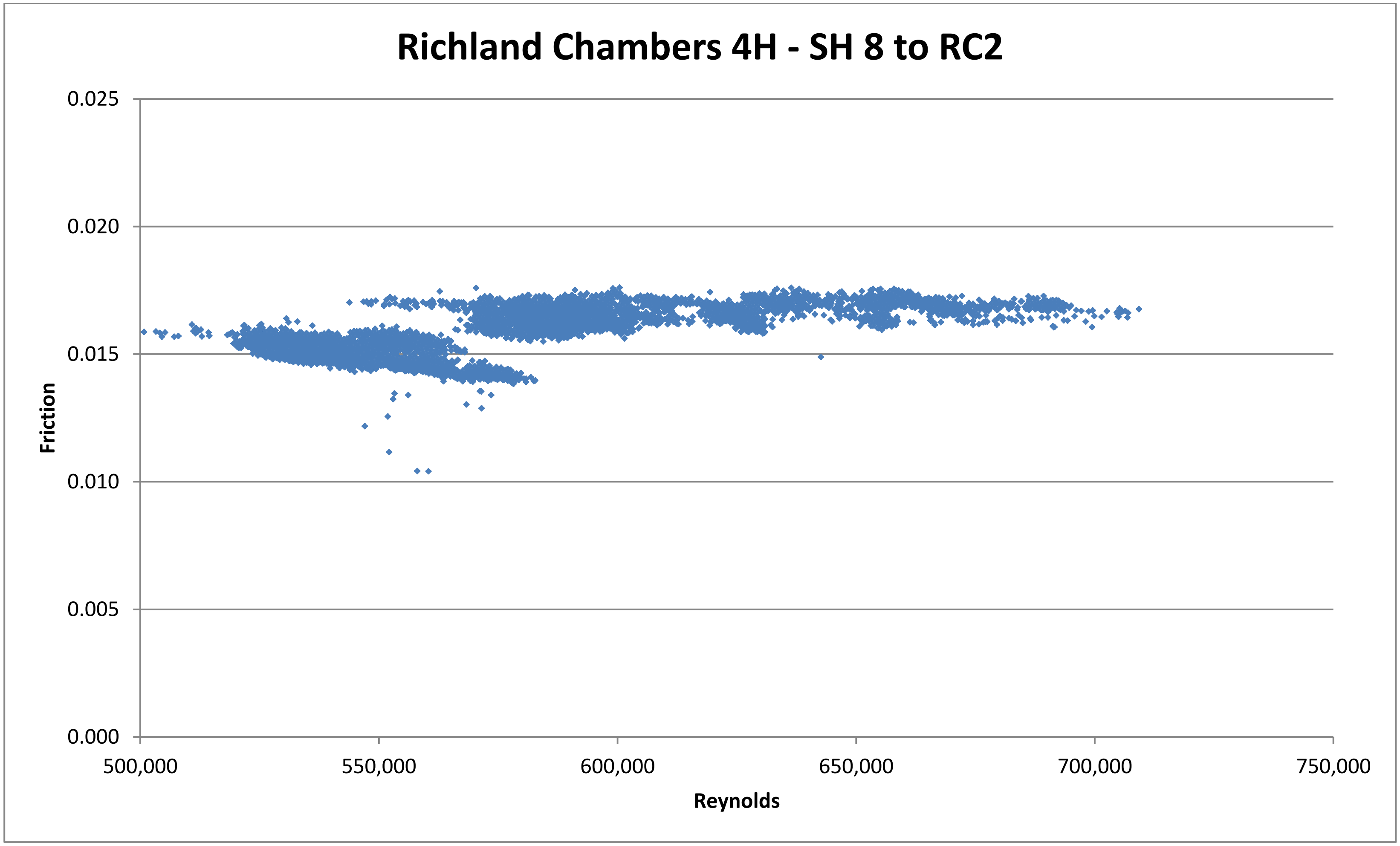
County Road 1030 to Barker Road



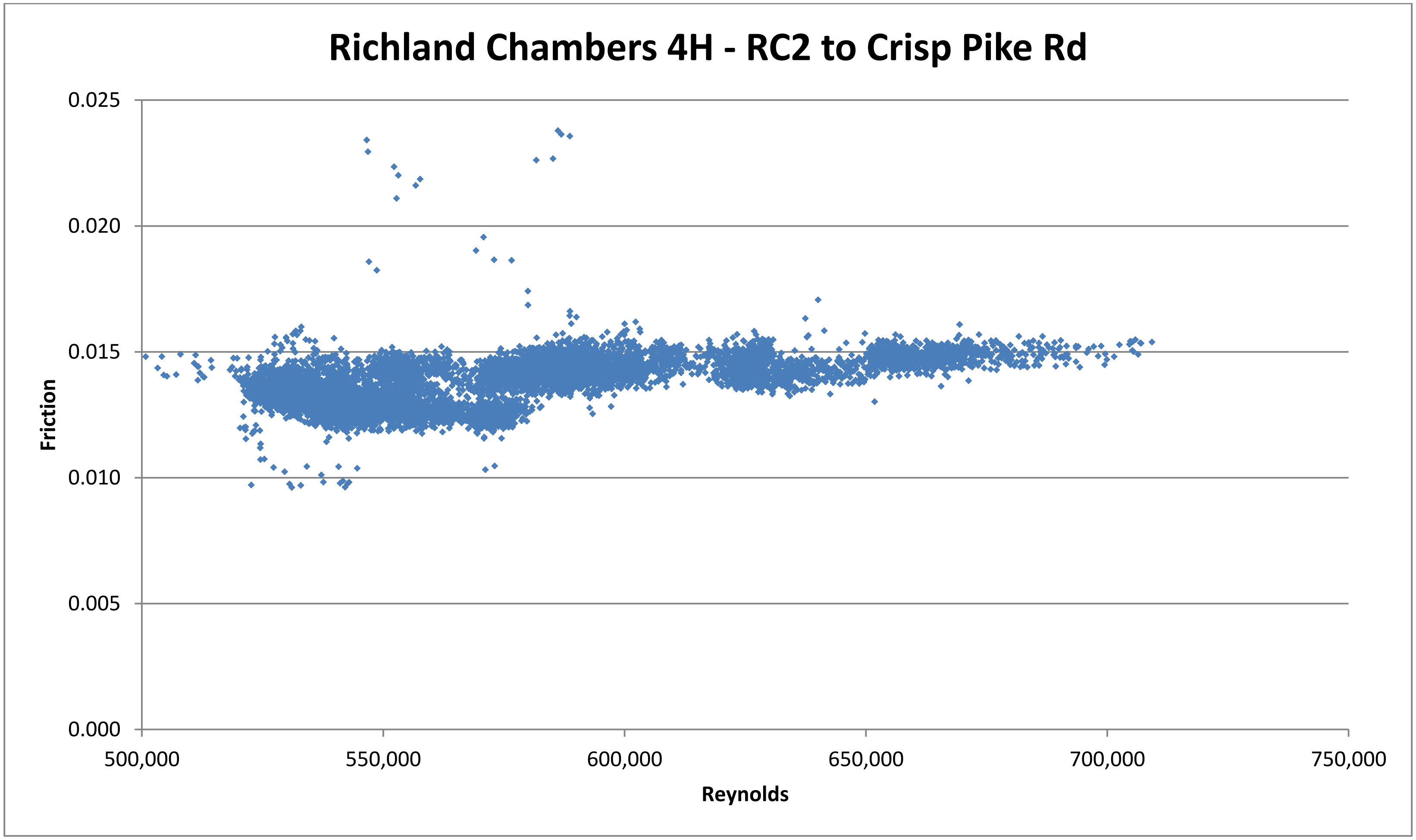
Barker Road to State Highway 85



State Highway 85 to Richland Chambers Ennis Pump Station

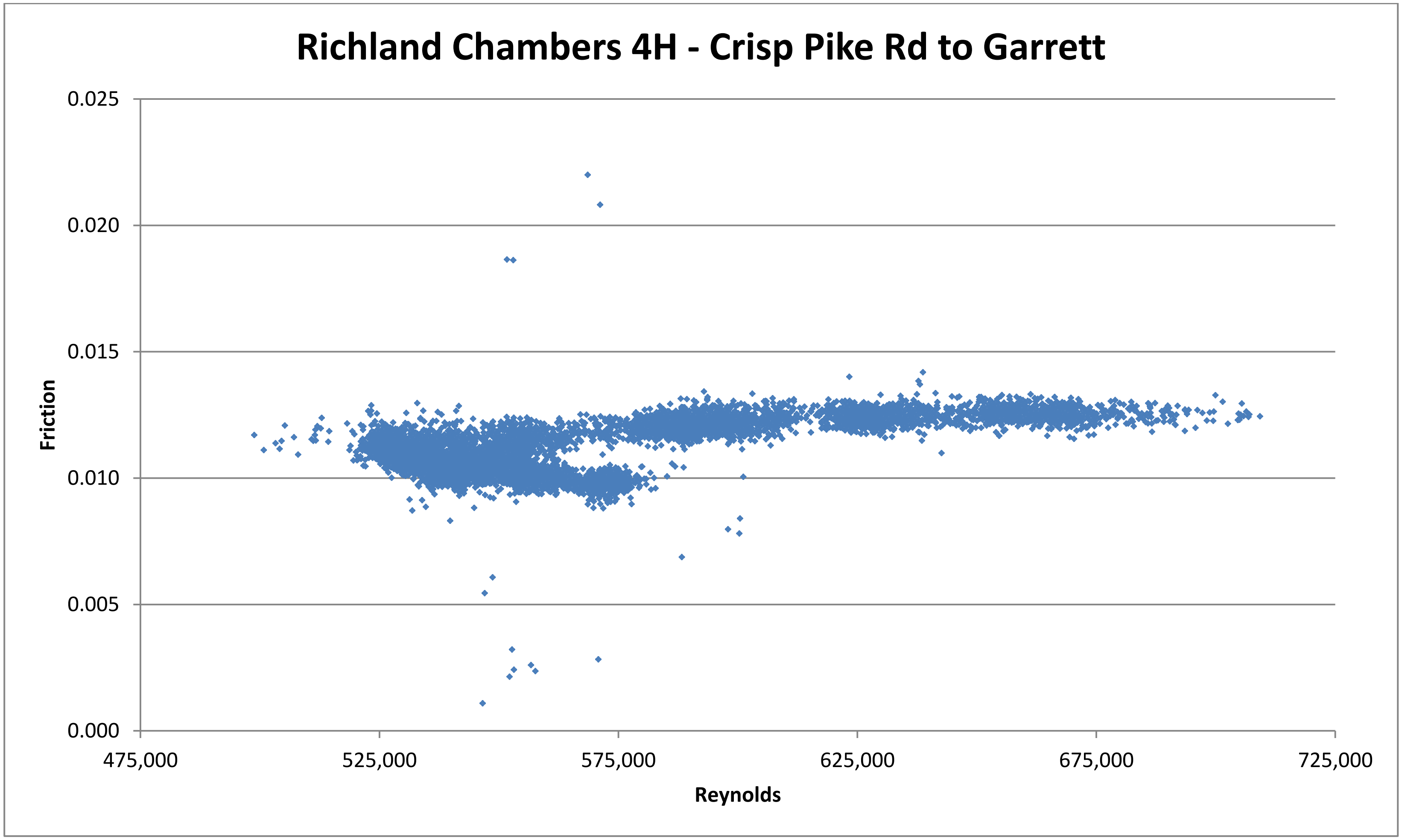


Richland Chambers Ennis Pump Station to Crisp Pike Road

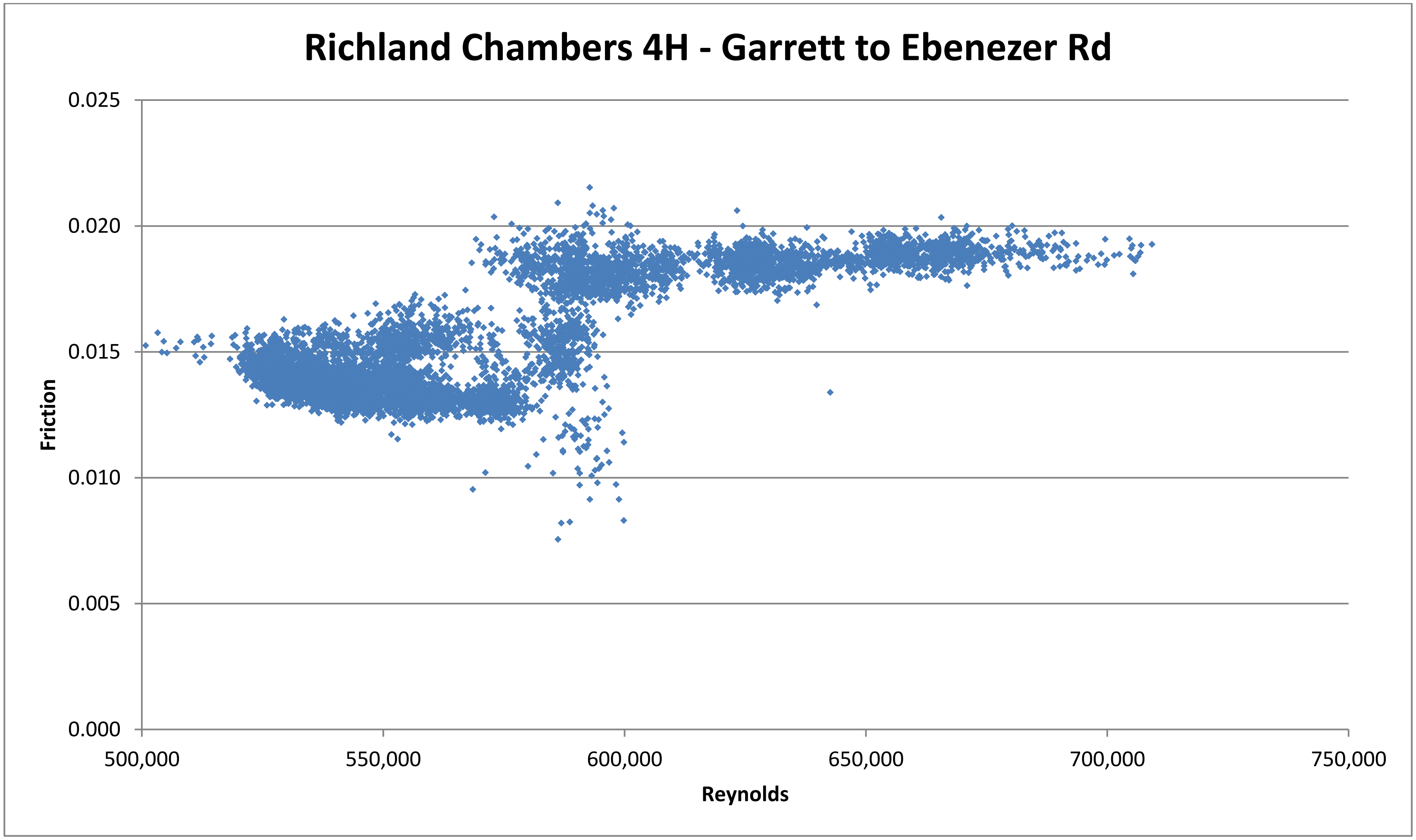


229

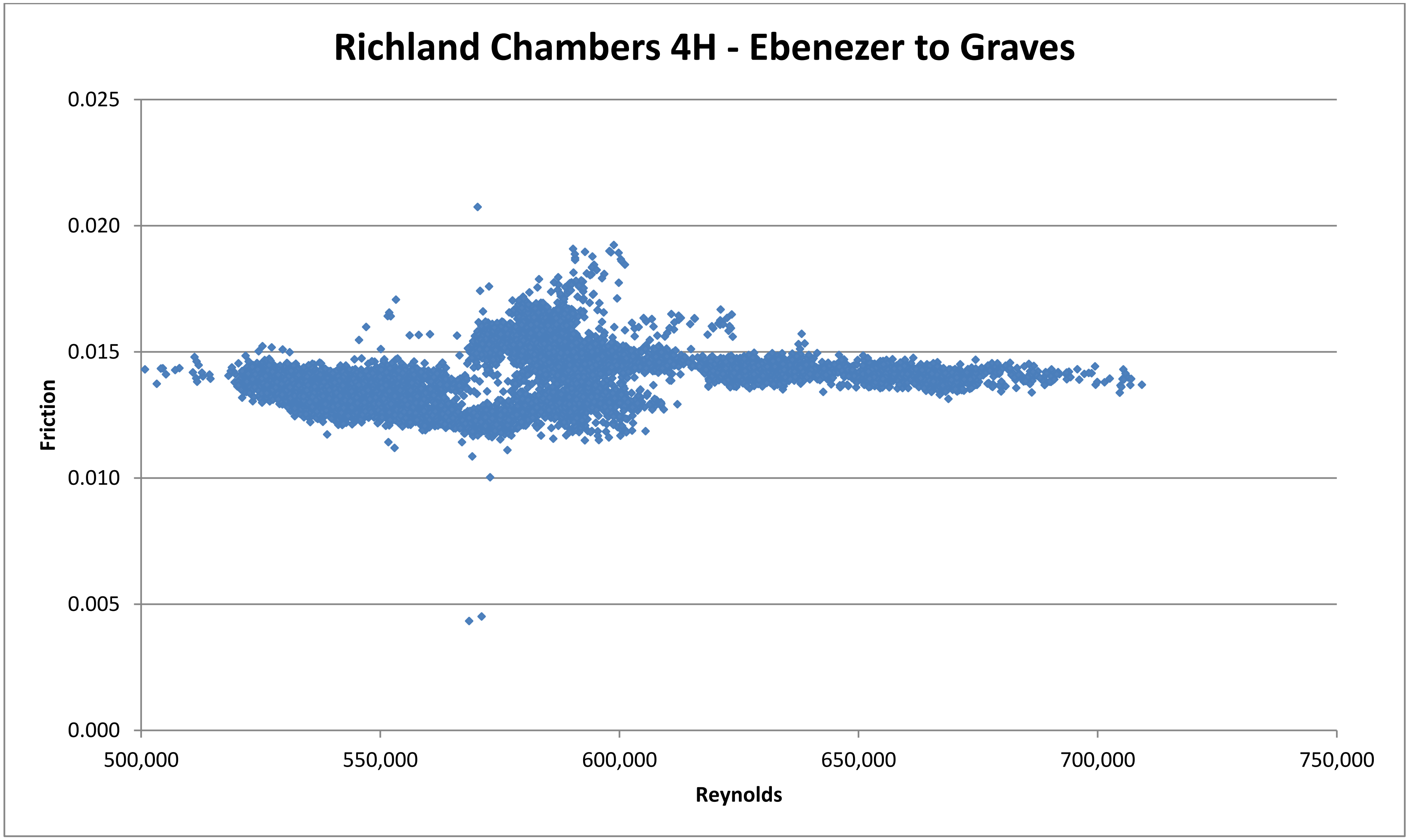
Crisp Pike Road to Garrett Road



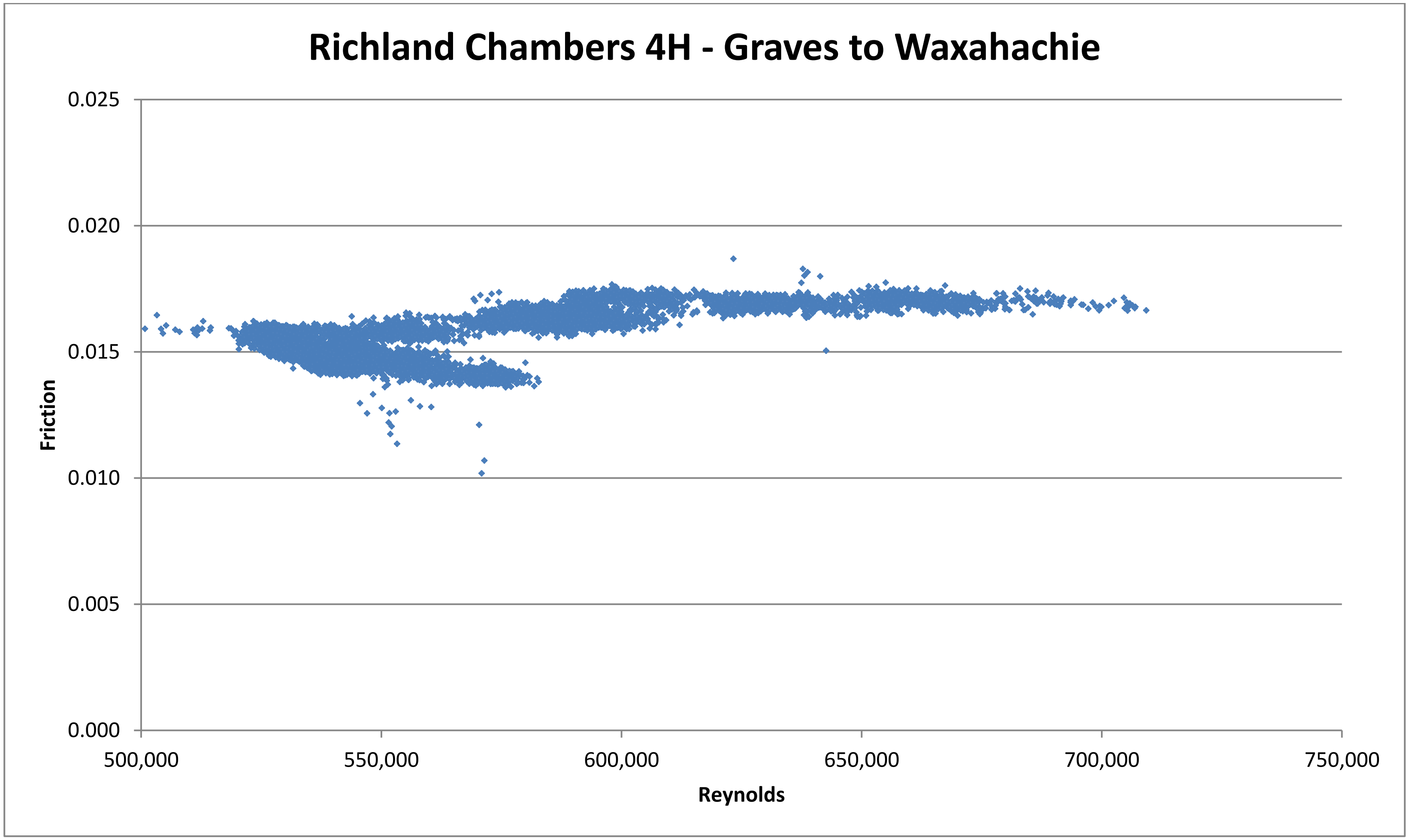
Garrett Road to Ebenezer Road



Ebenezer Road to Graves Road



Graves Road to Richland Chambers Waxahachie Pump Station

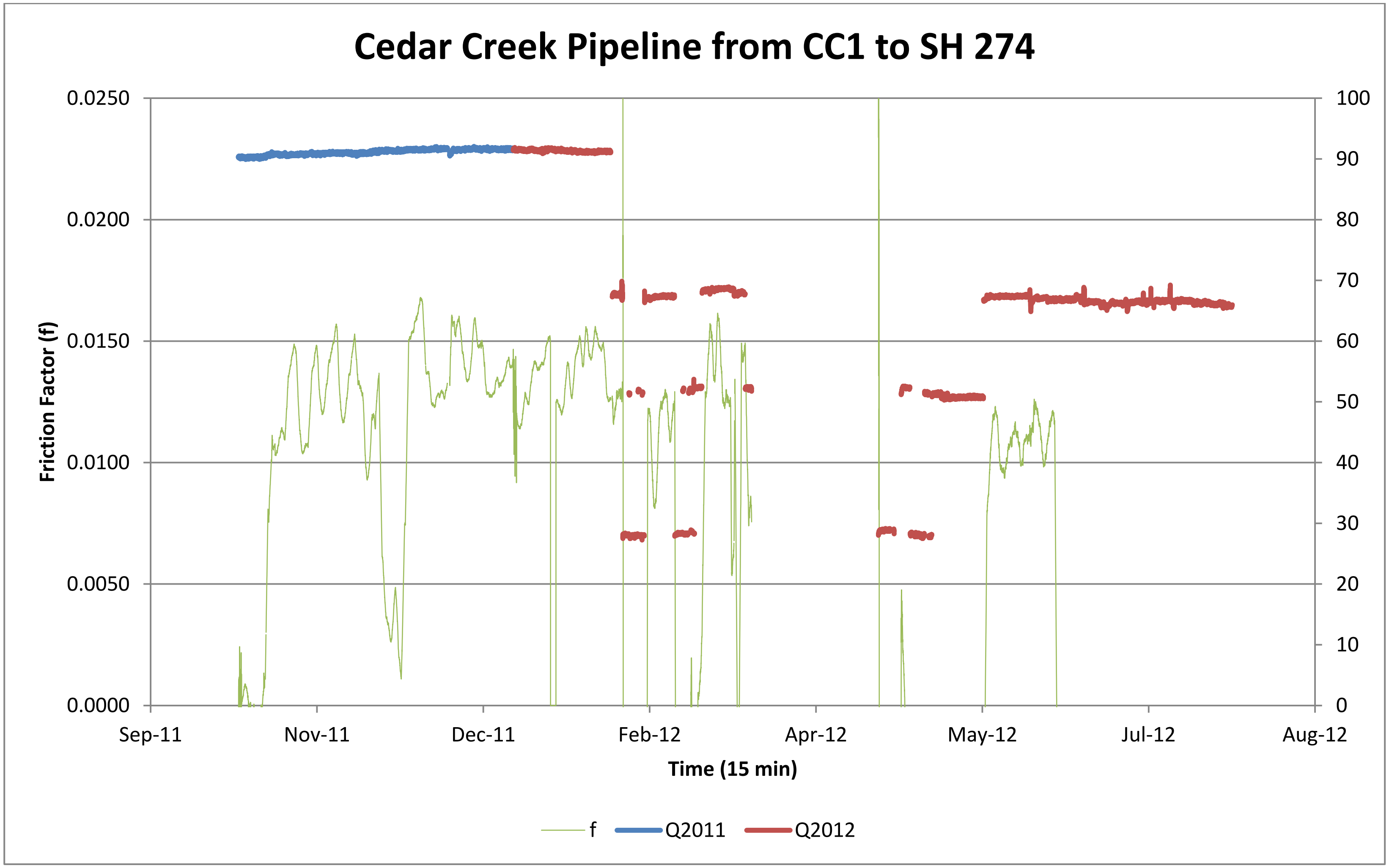


APPENDIX E

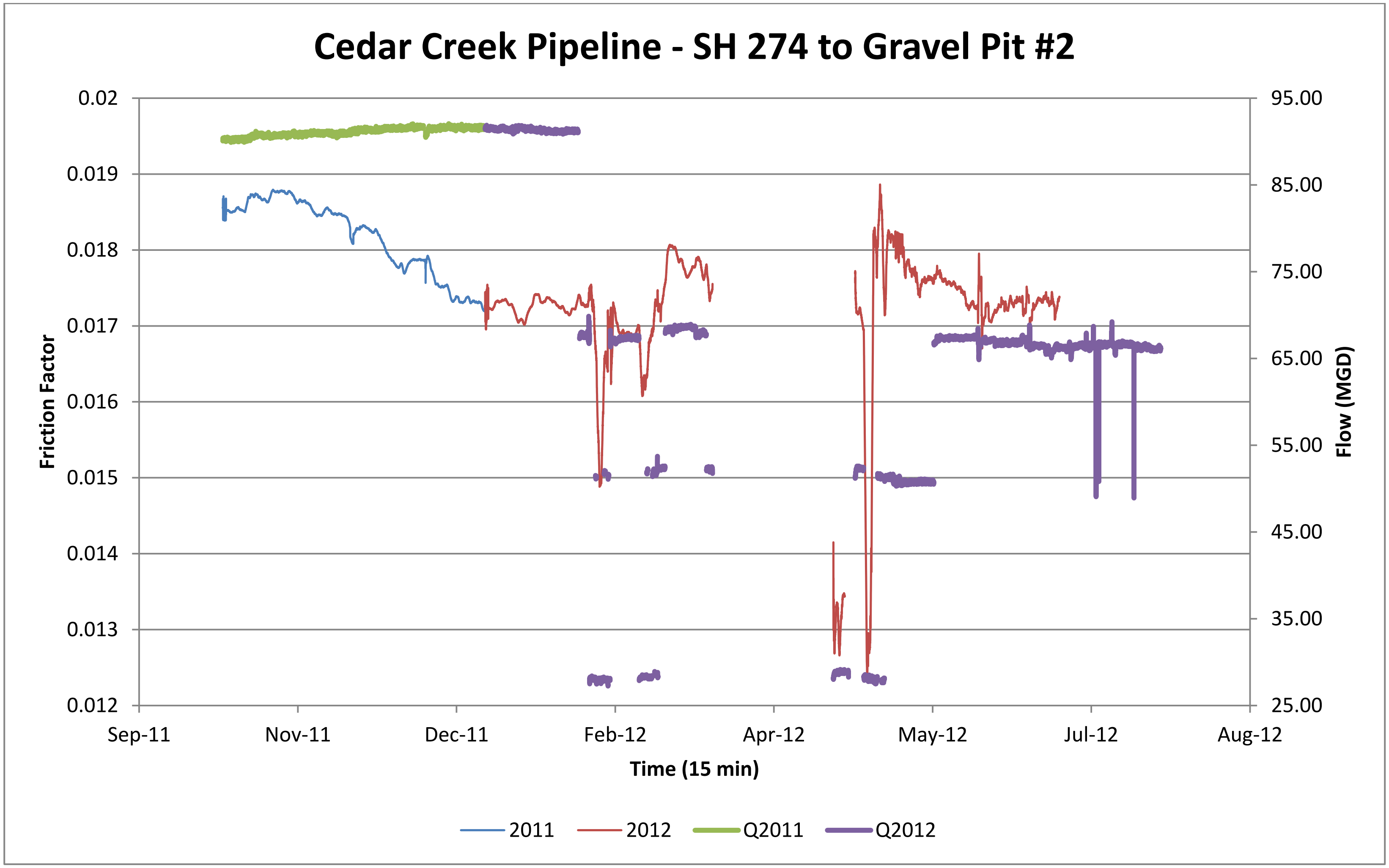
CEDAR CREEK FIELD STUDY

FRICTION FACTOR VERSUS TIME RESULTS.

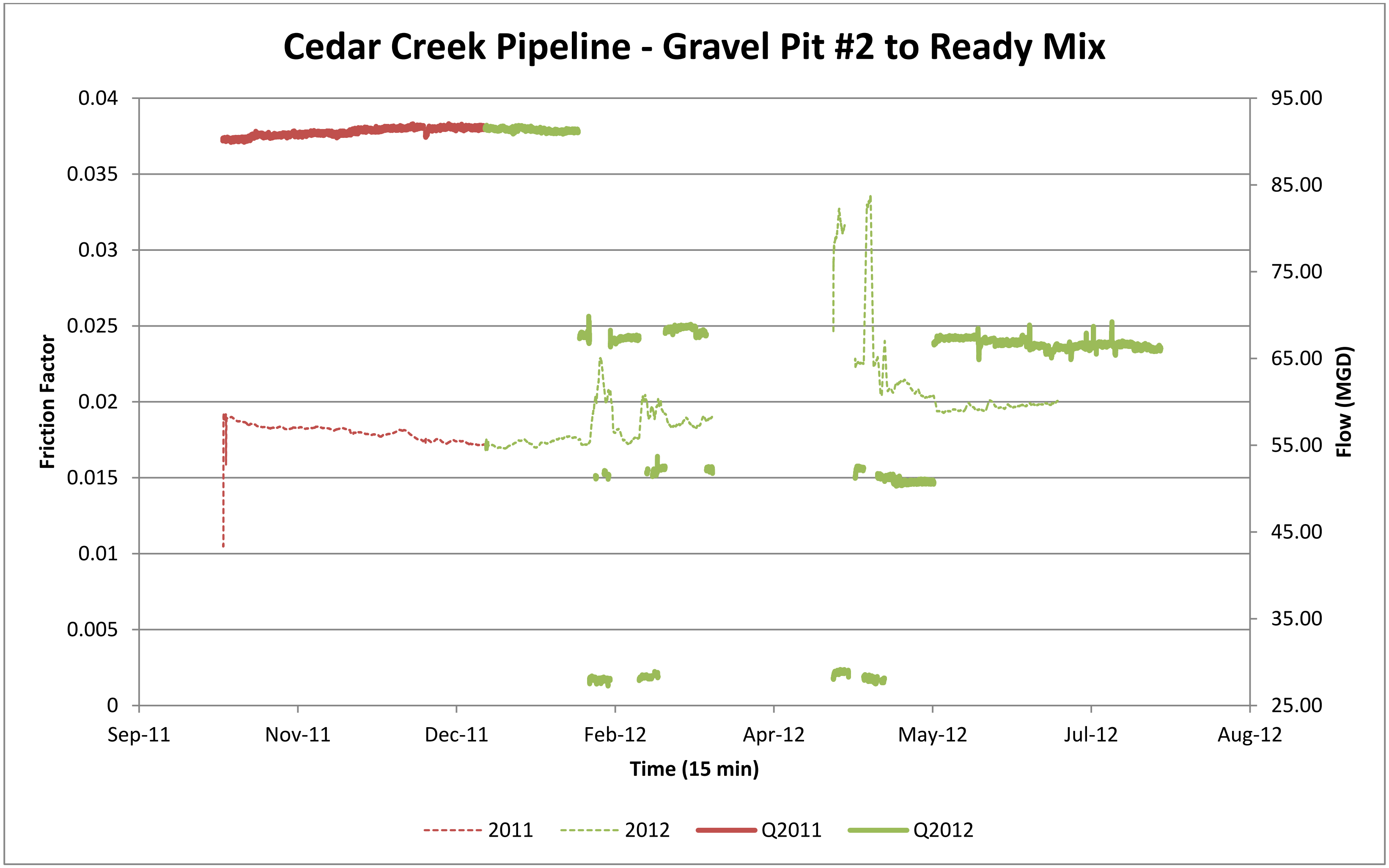
Cedar Creek Lake Pump Station to State Highway 274



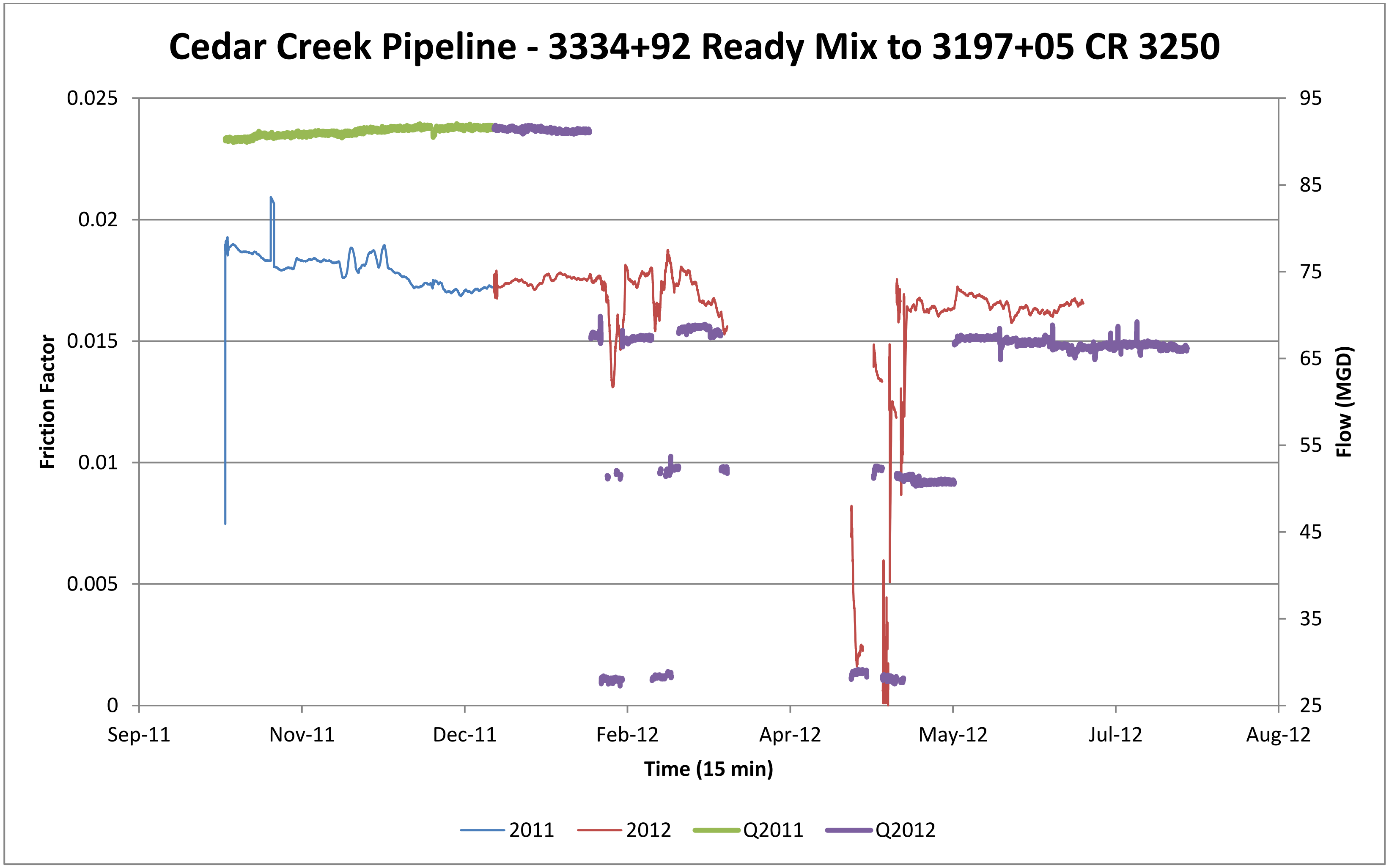
SH 274 to Gravel Pit #2



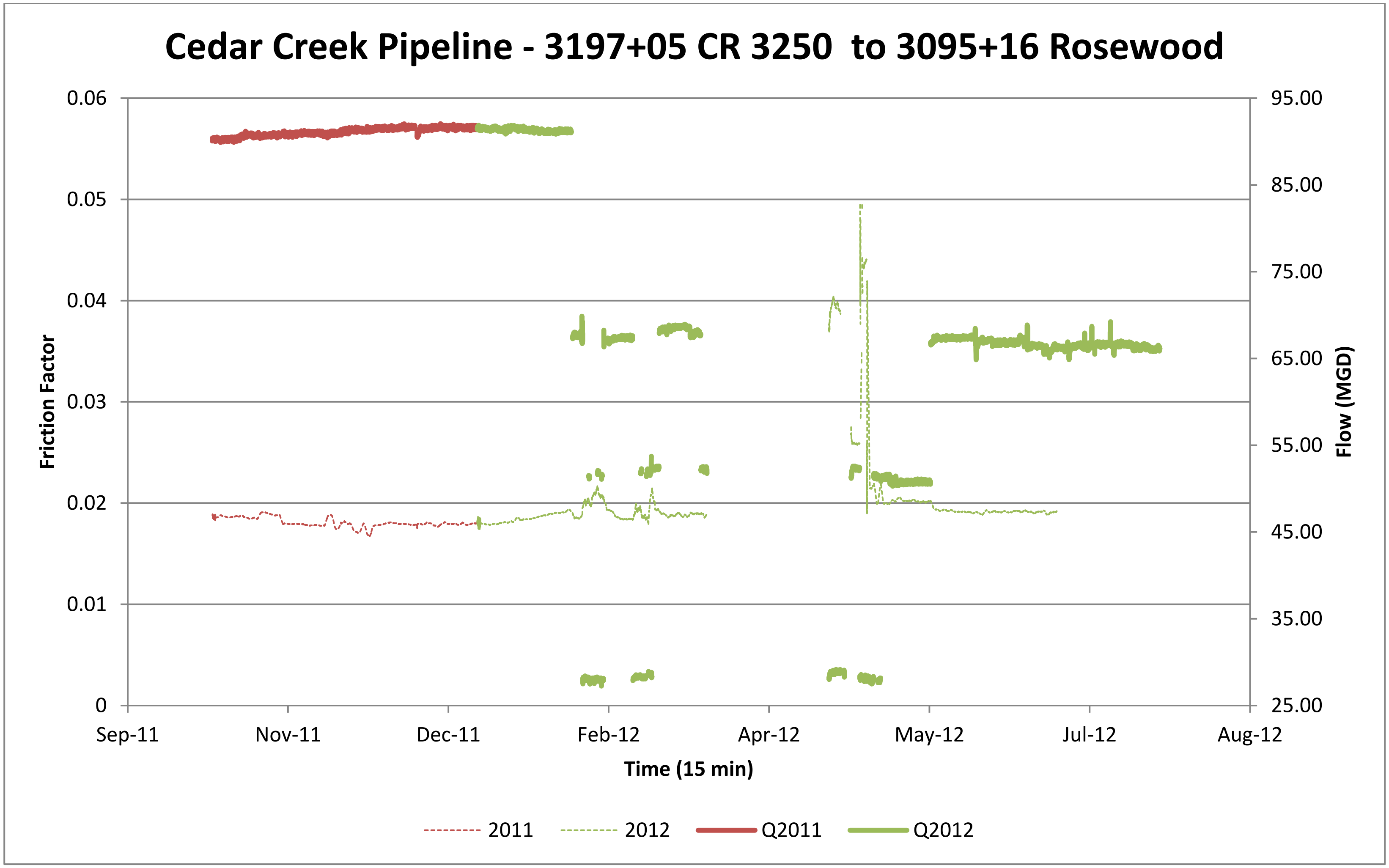
Gravel Pit #2 to Ready Mix



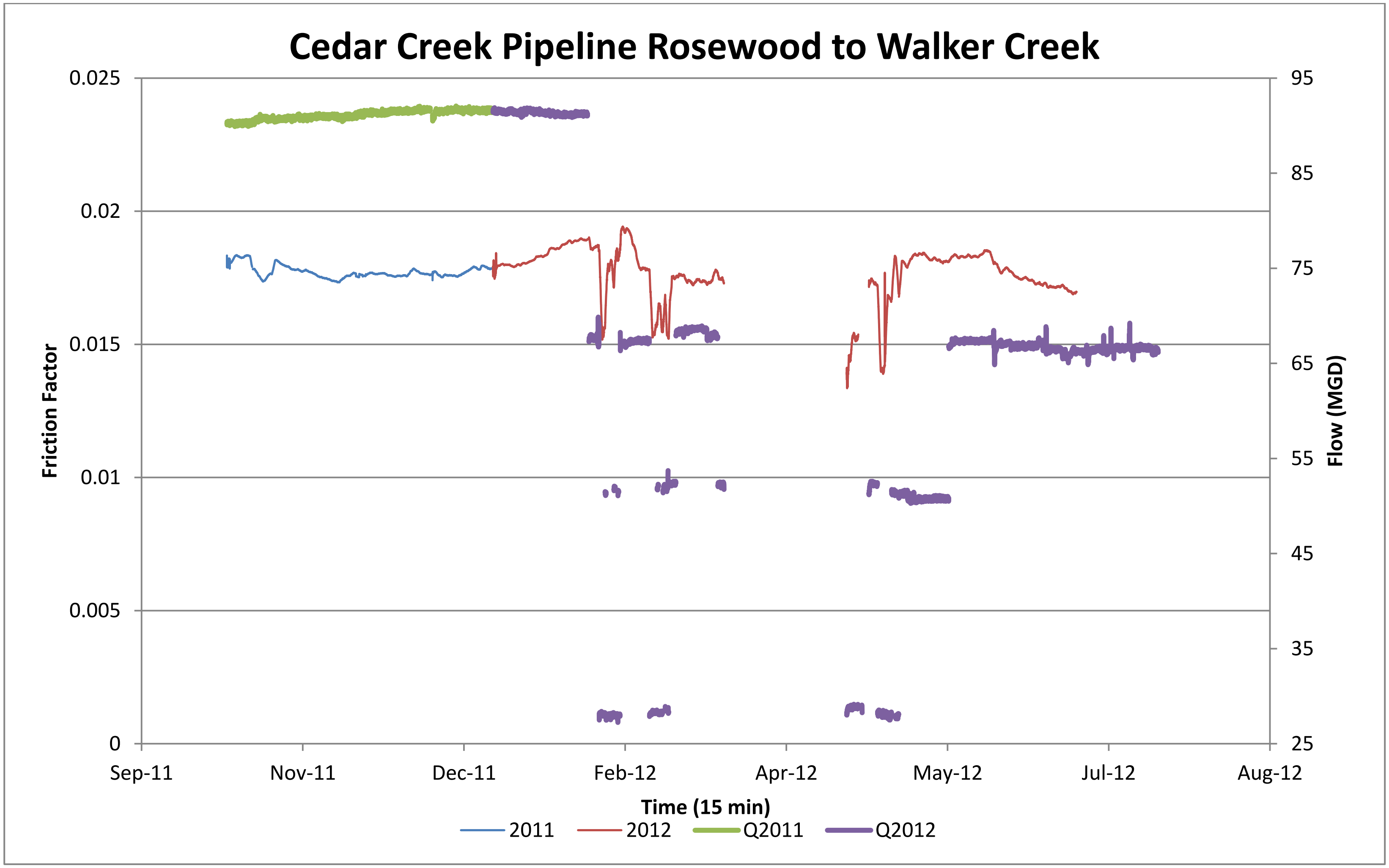
Ready Mix to CR 3250



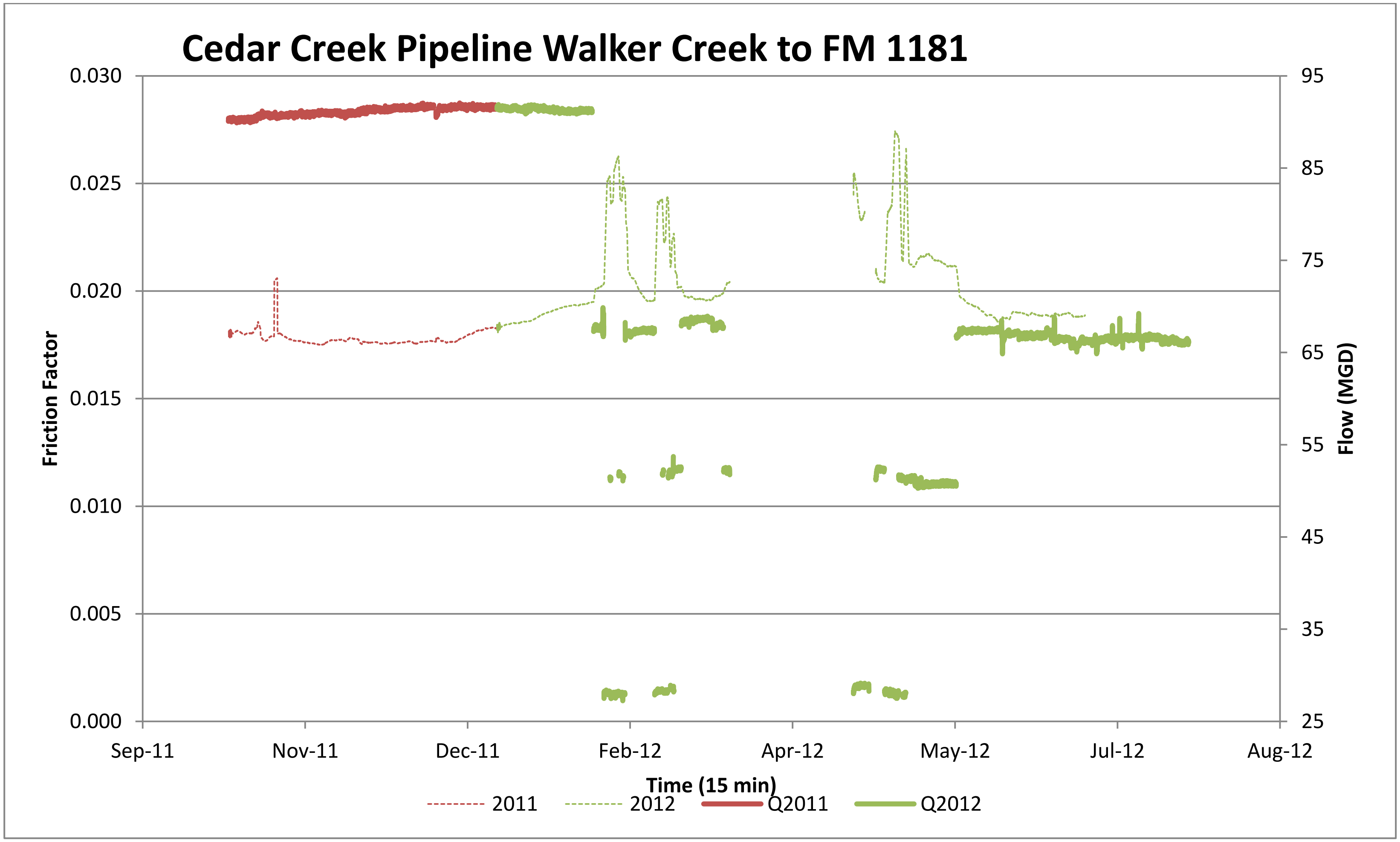
CR 3250 to Rosewood



Rosewood to Walker Creek

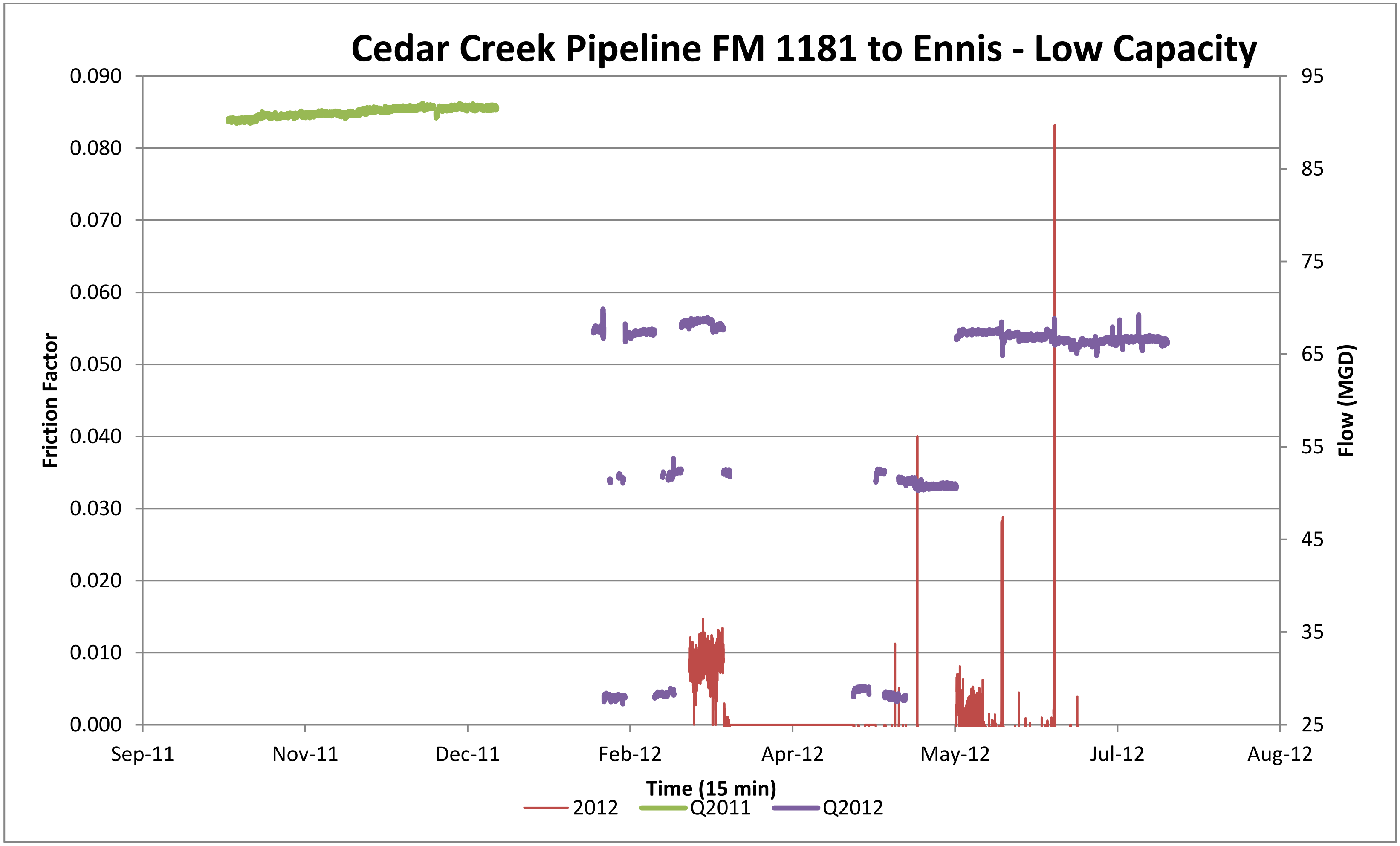


Walker Creek to FM 1181



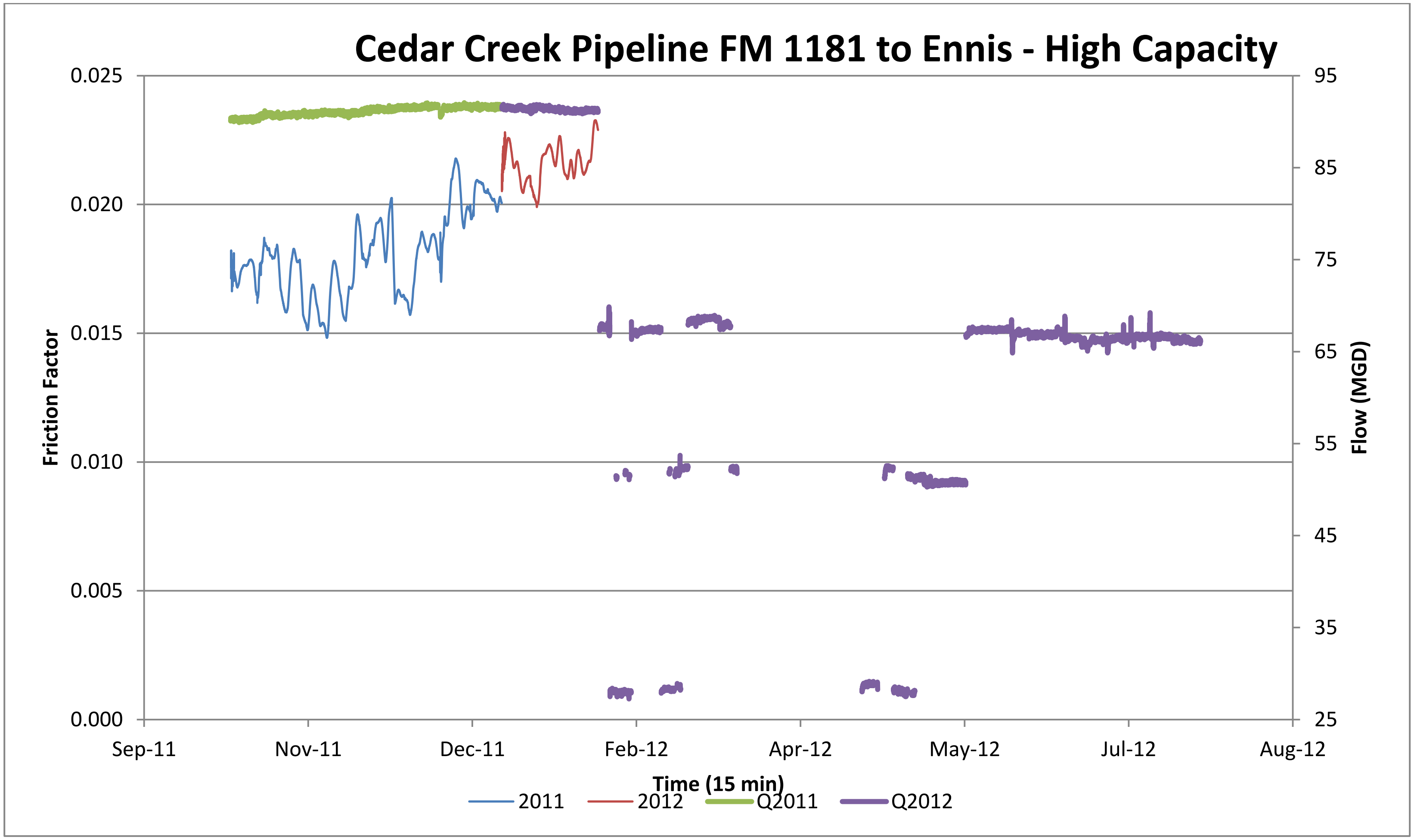
241

FM 1181 to Ennis – Low Capacity

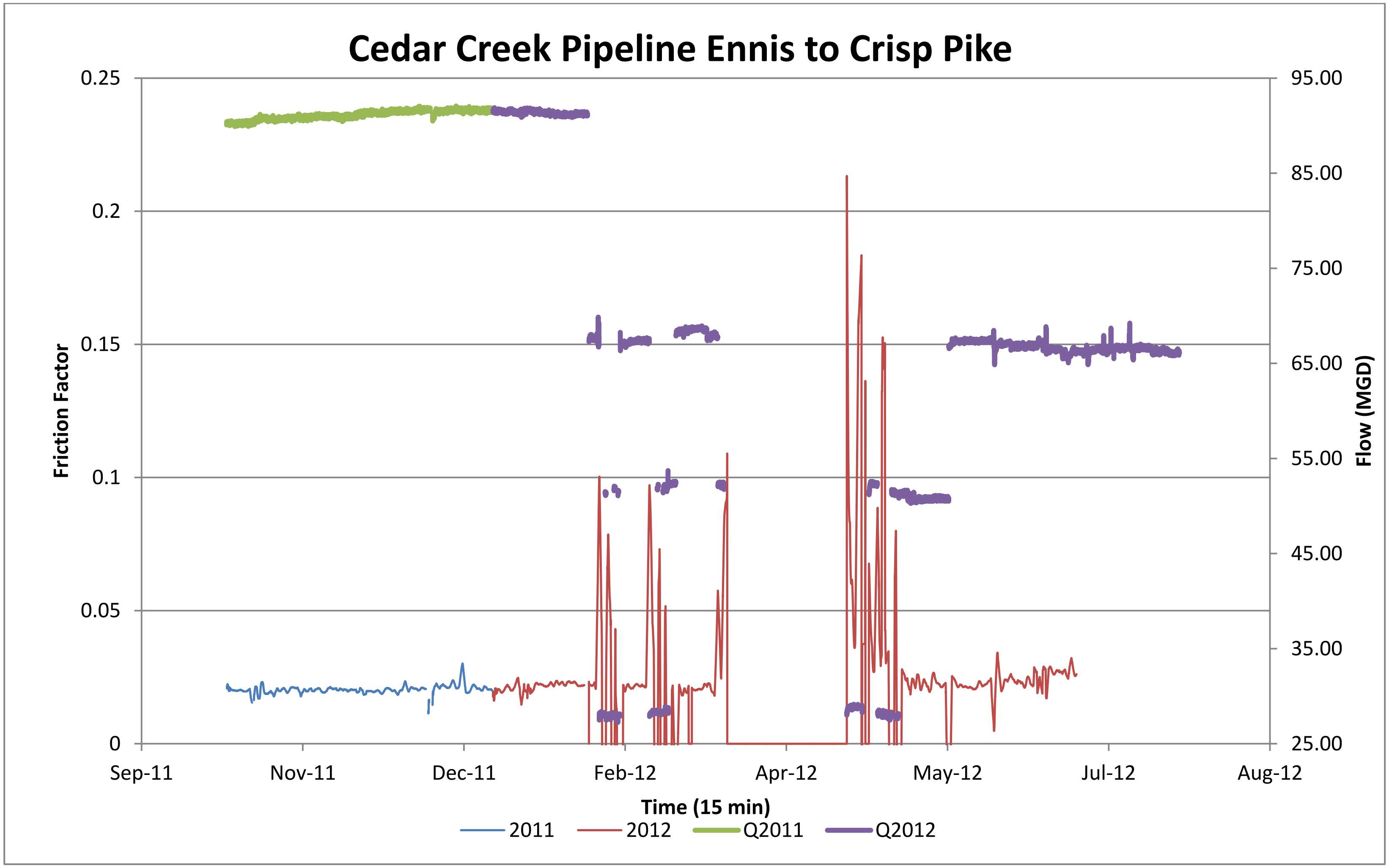


242

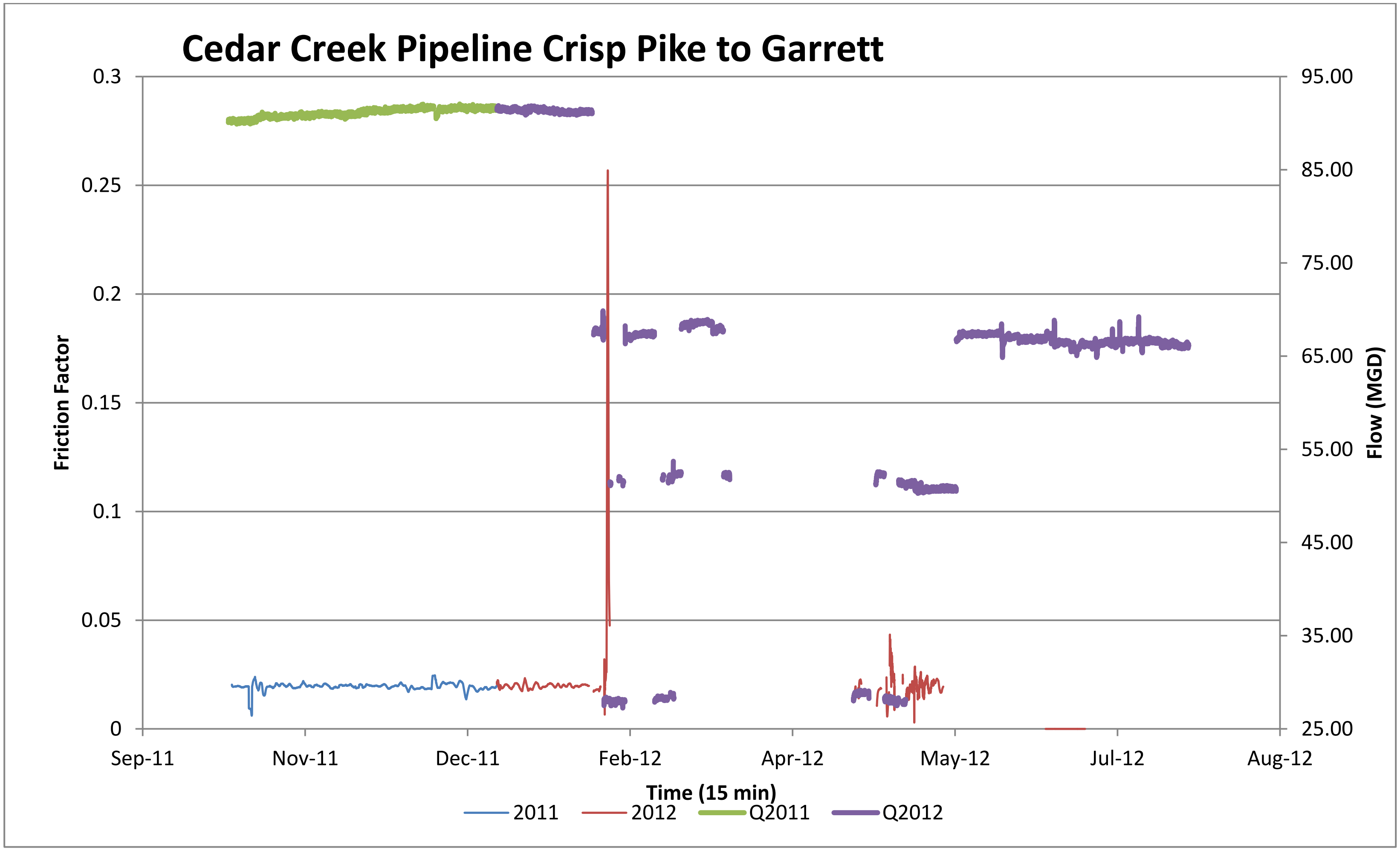
FM 1181 to Ennis – High Capacity



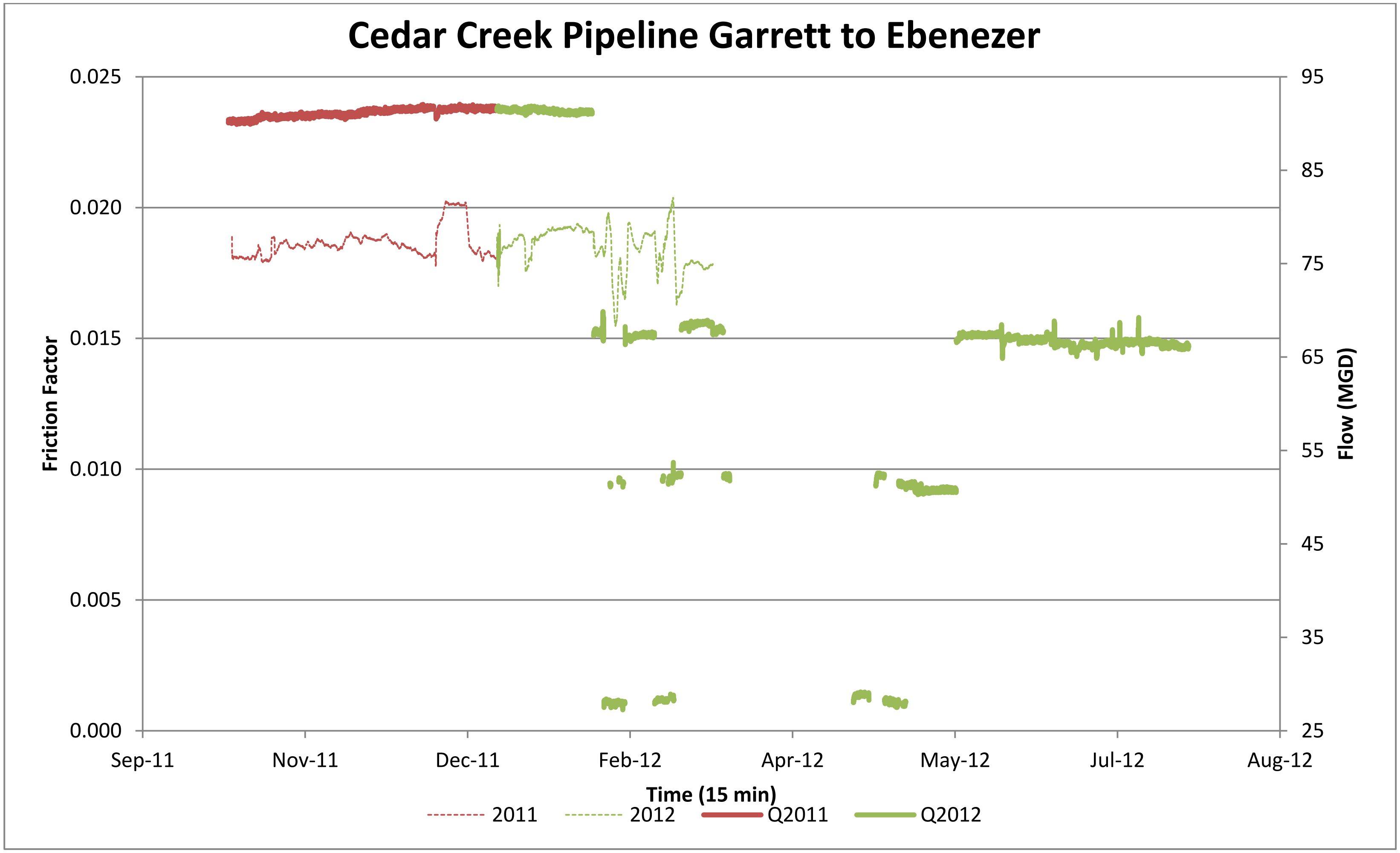
Ennis to Crisp Pike



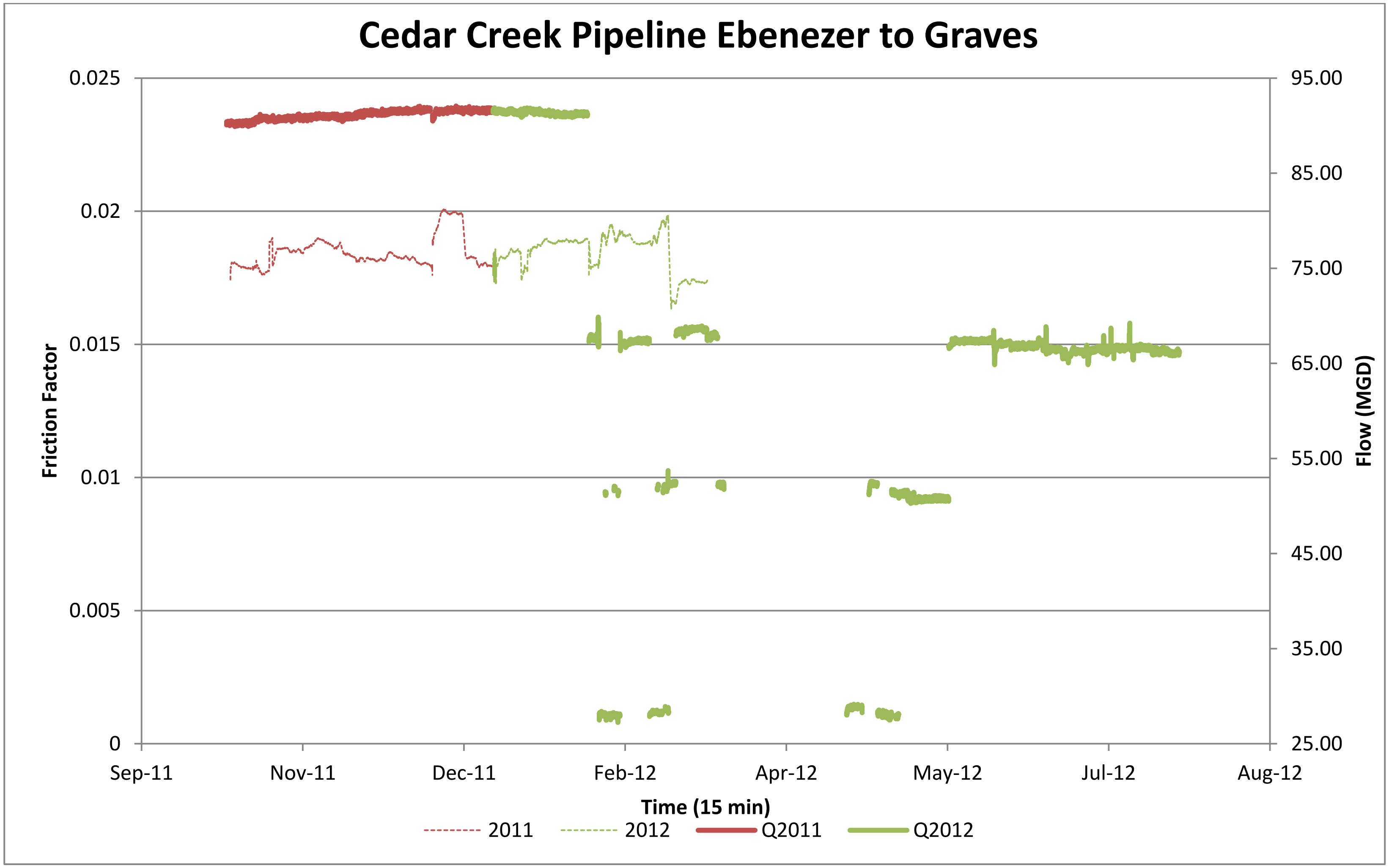
Crisp Pike to Garrett



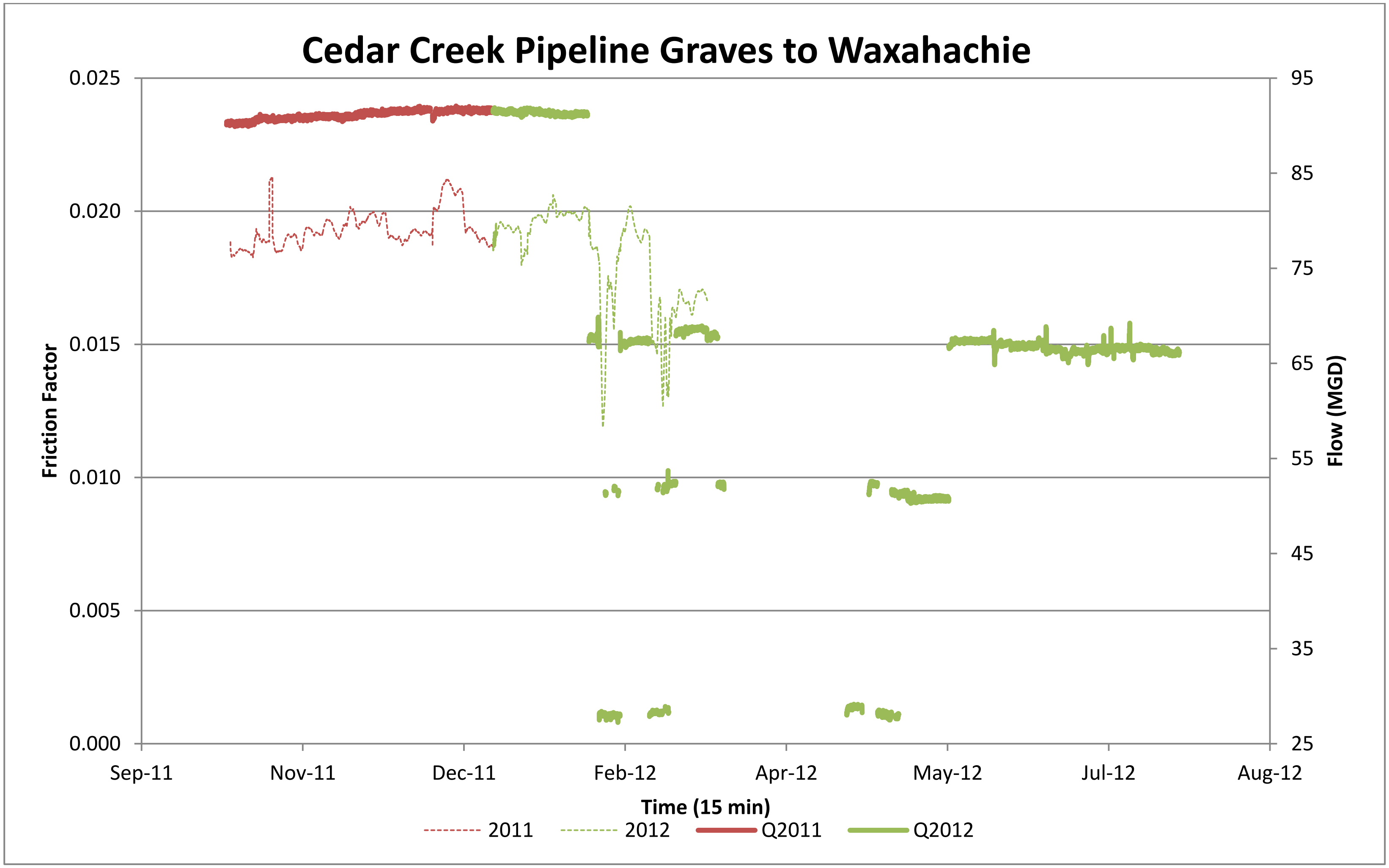
Garrett to Ebenezer



Ebenezer to Graves



Graves to Waxahachie

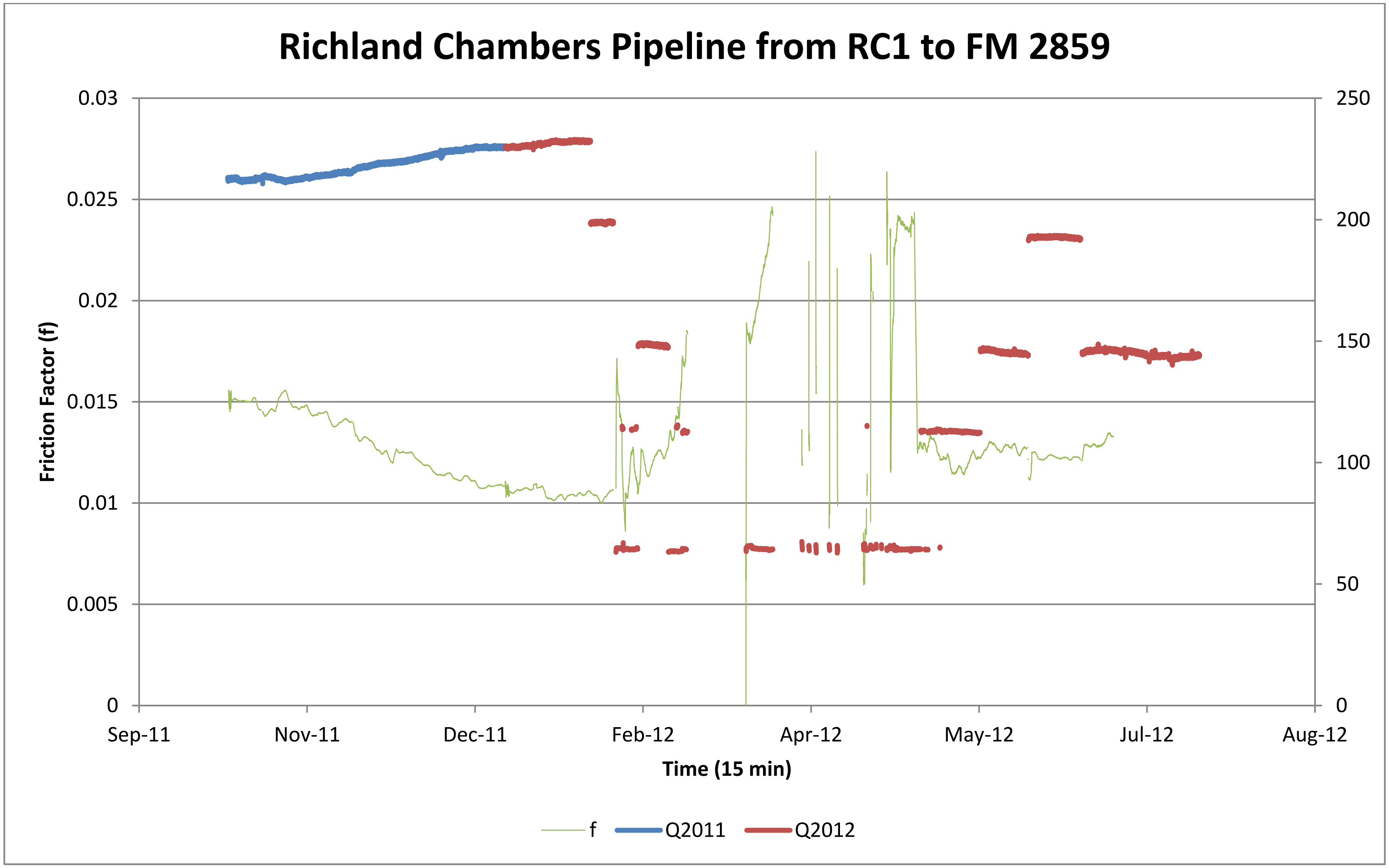


APPENDIX F

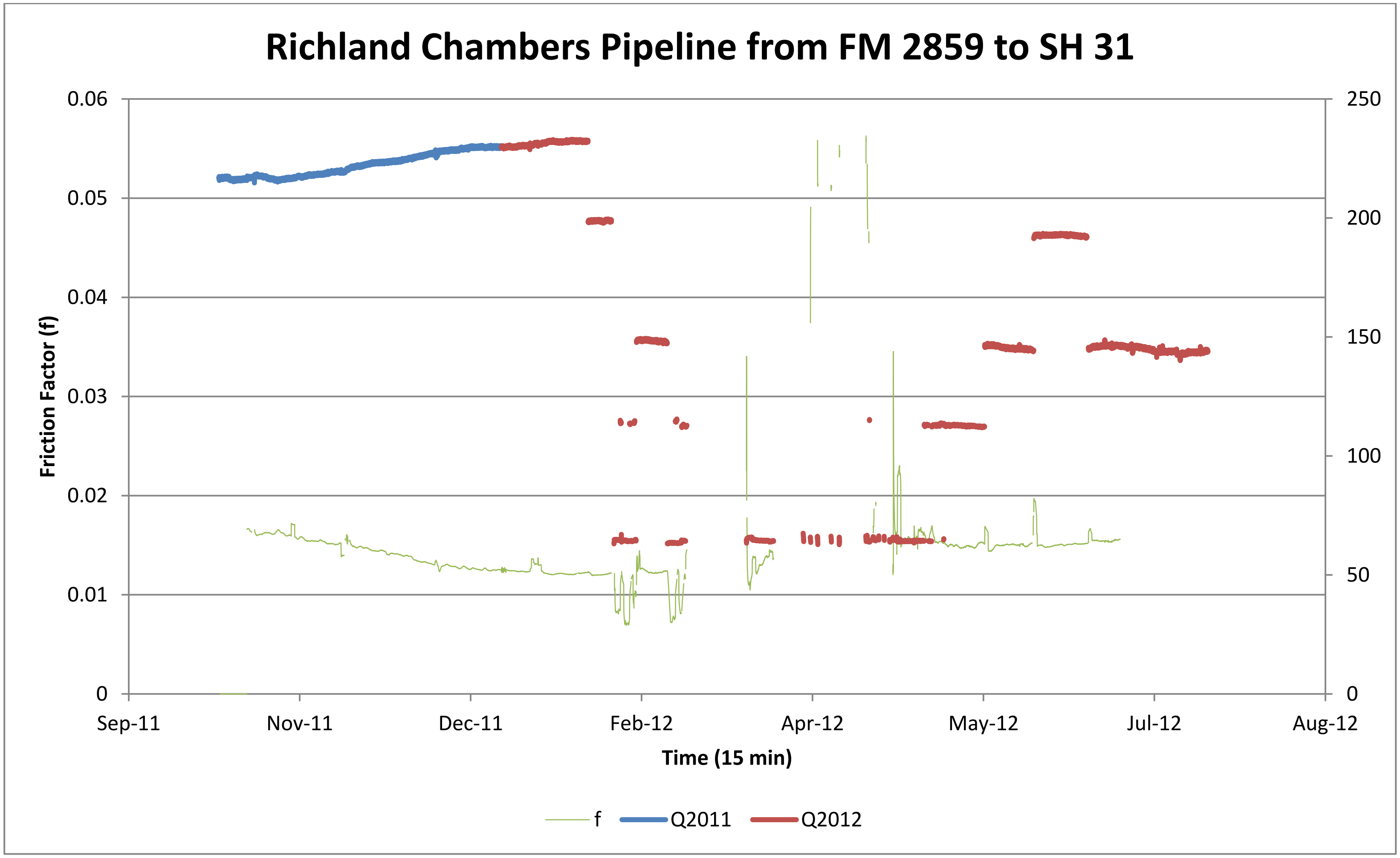
RICHLAND CHAMBERS FIELD STUDY

FRICTION FACTOR VERSUS TIME RESULTS

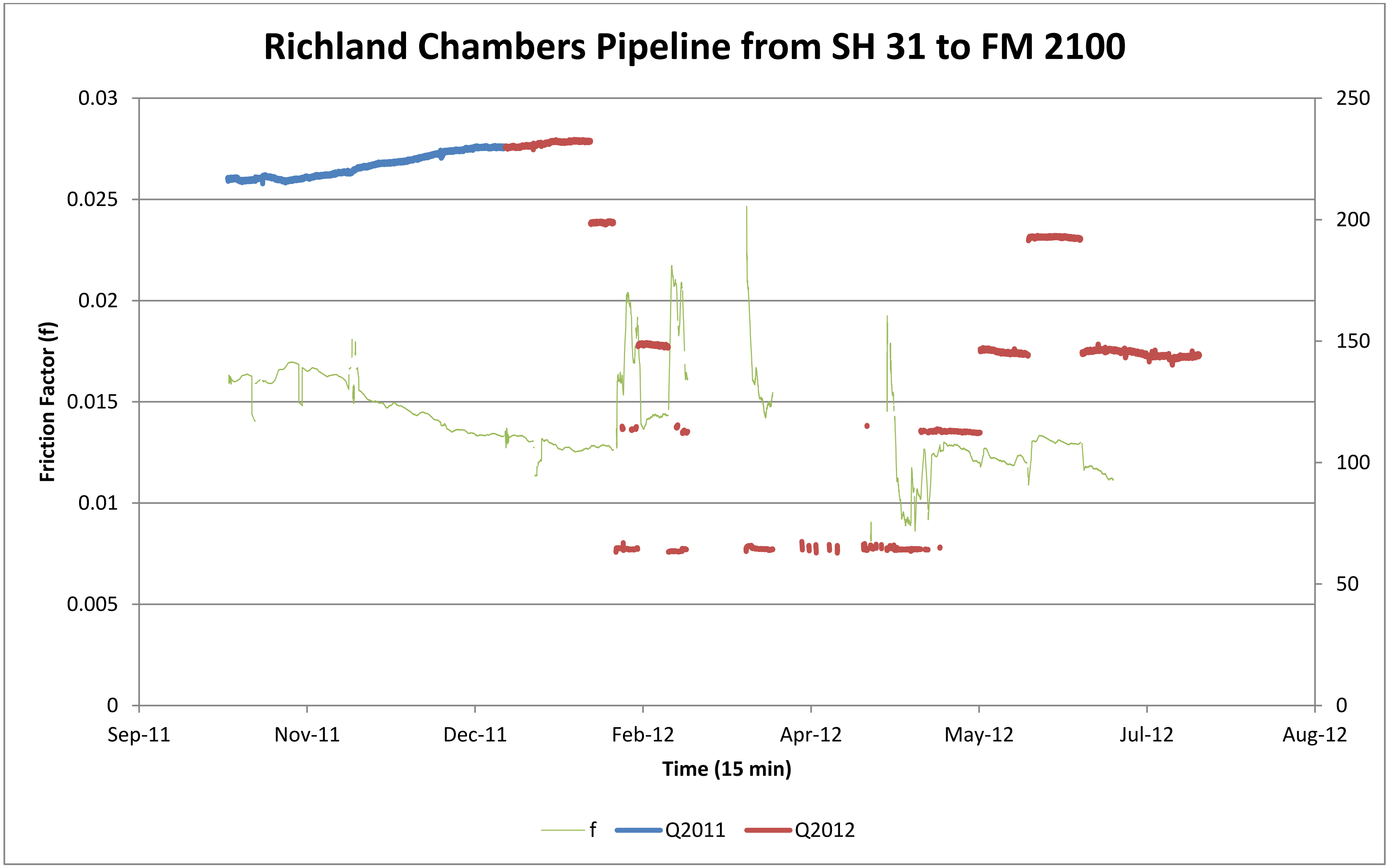
Richland Chambers Lake Pump Station to Farm-to-Market 2859



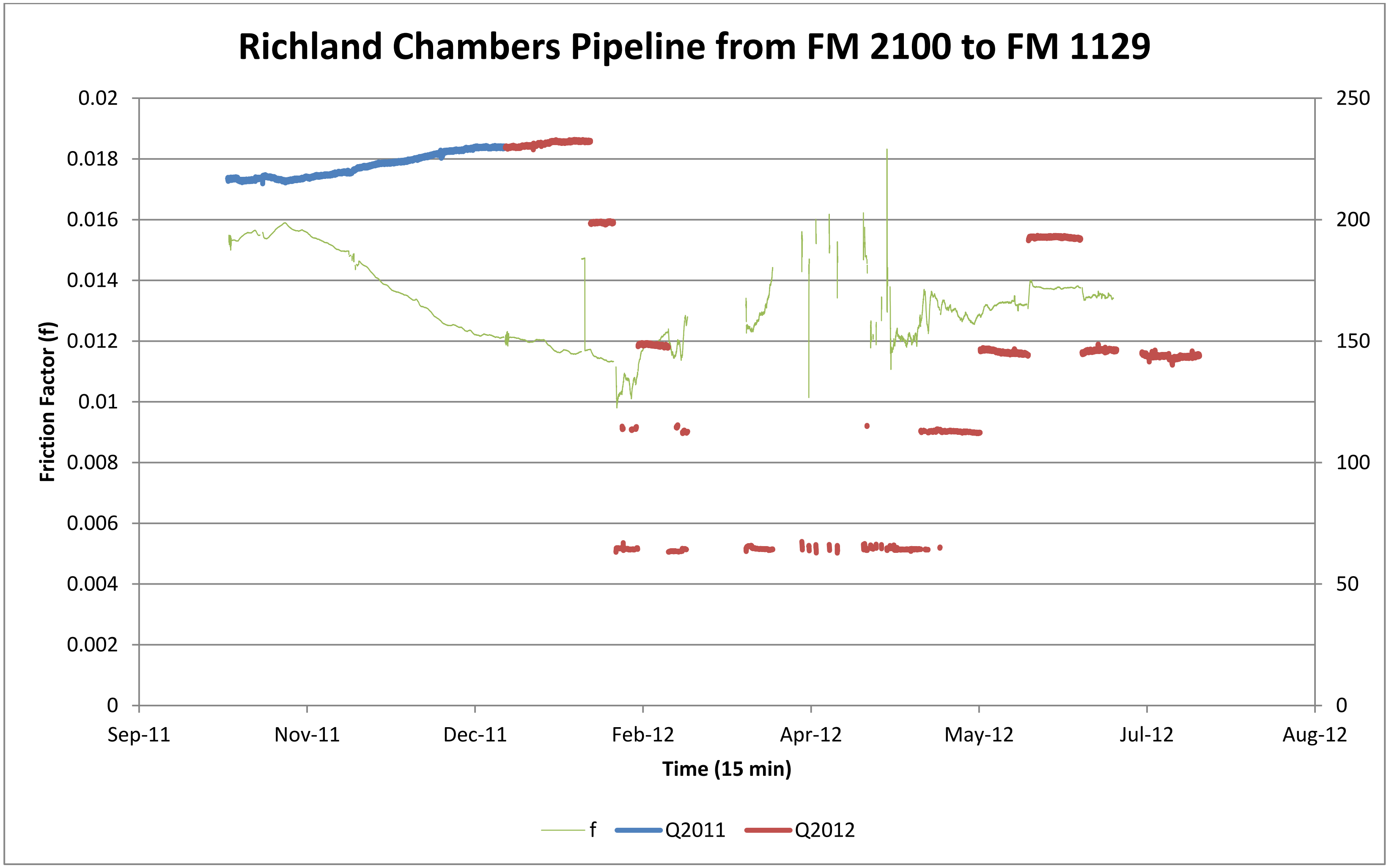
Farm-to-Market 2859 to State Highway 31



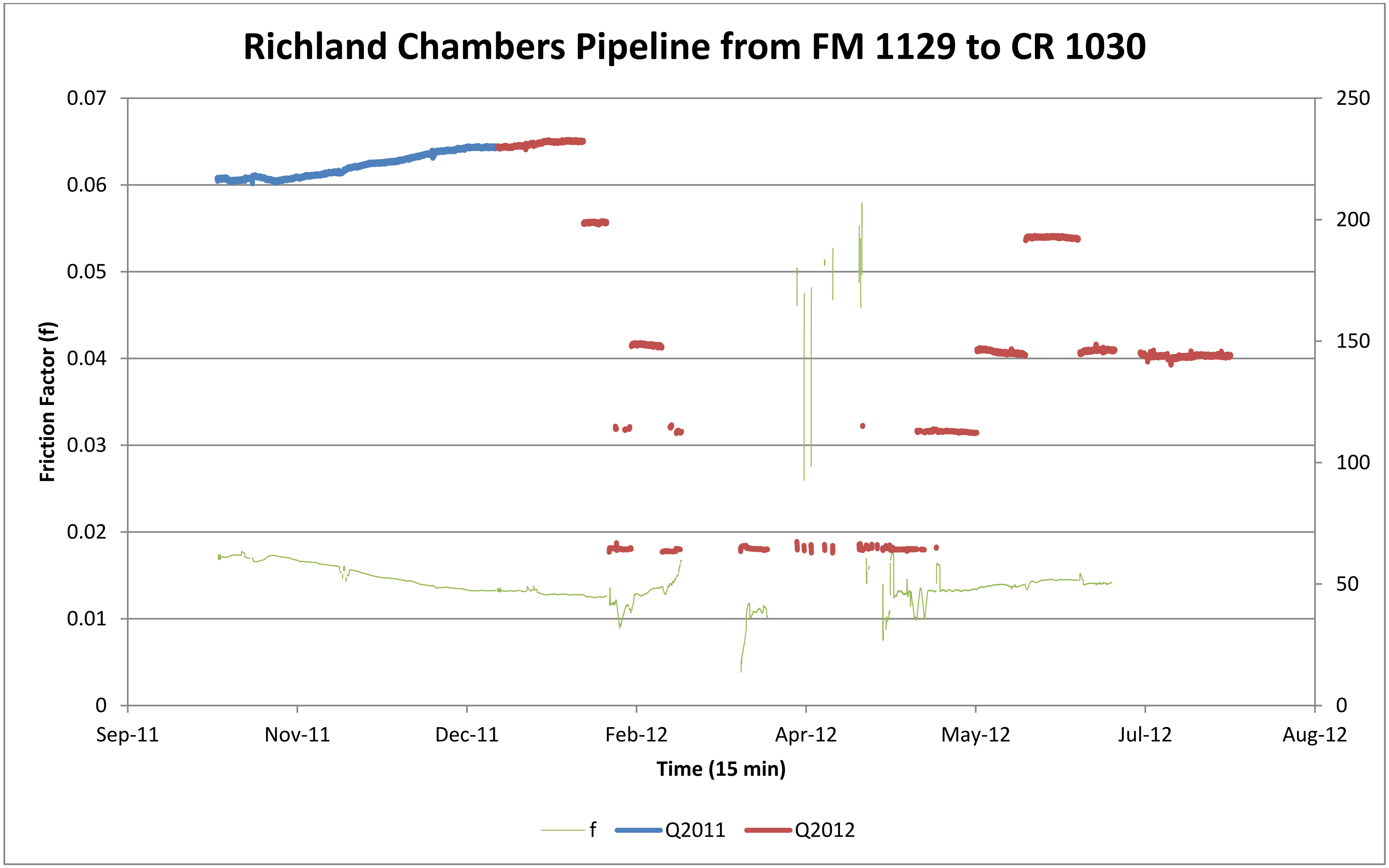
State Highway 31 to Farm-to-Market 2100



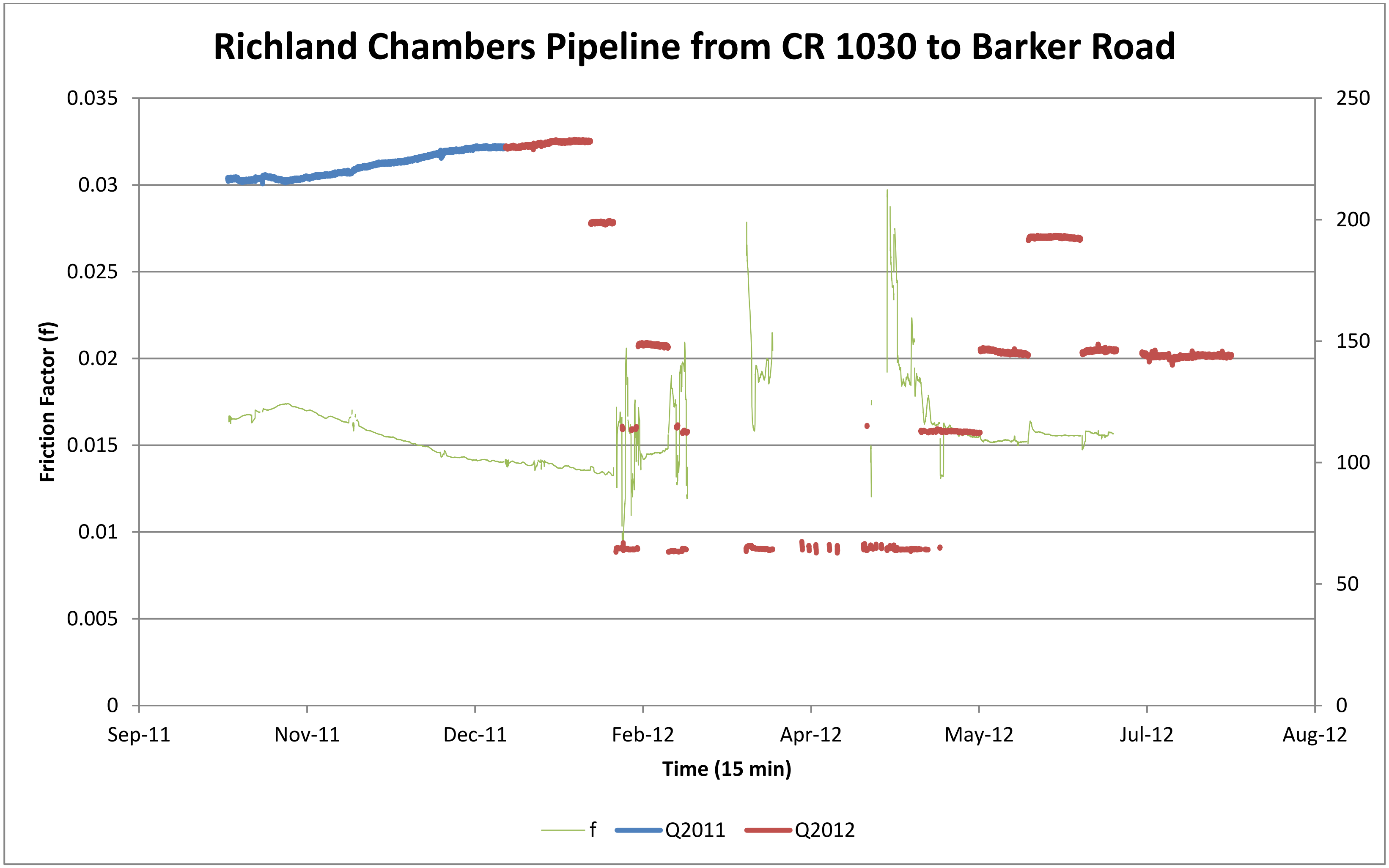
Farm-to-Market 2100 to Farm-to-Market 1129



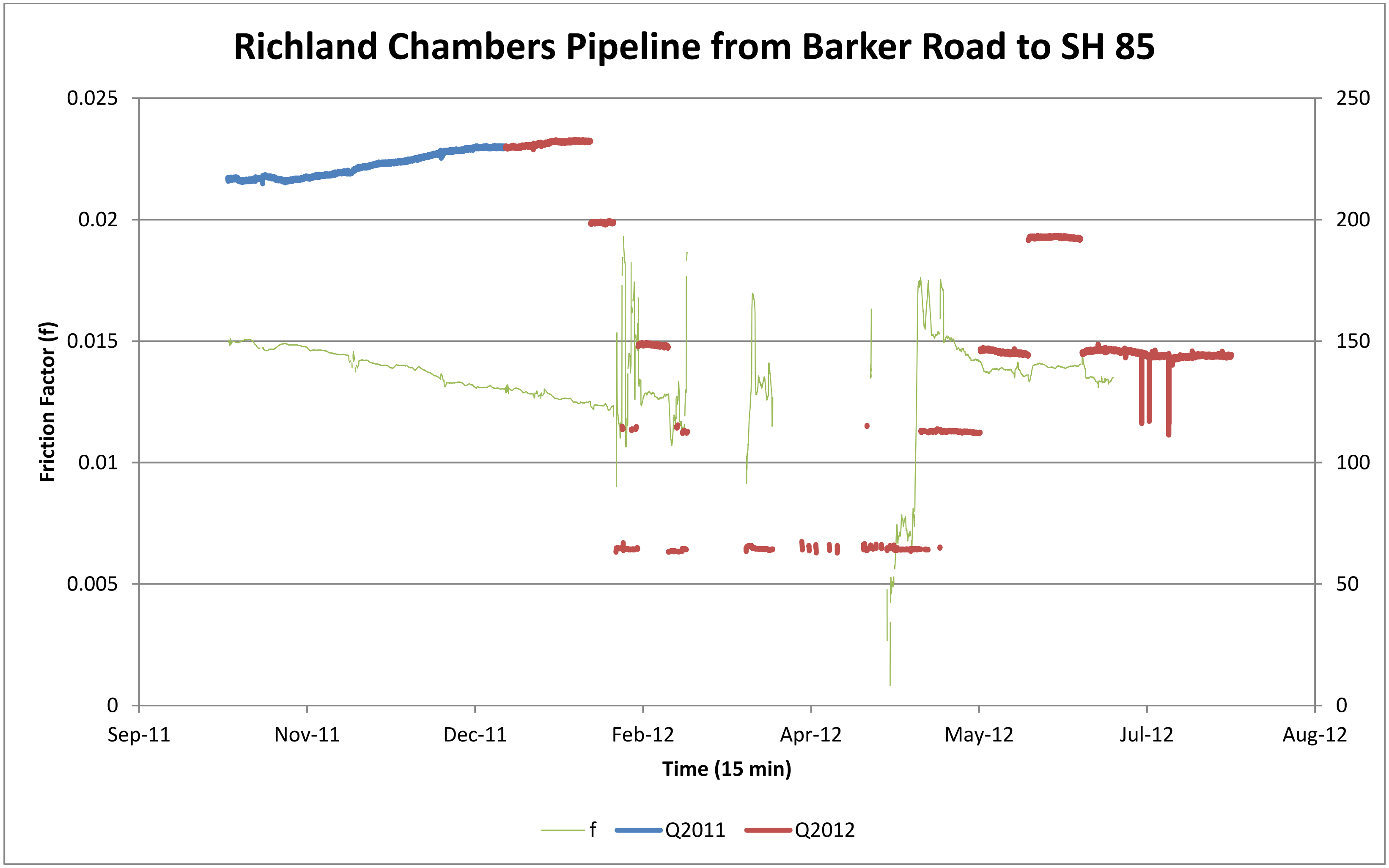
Farm-to-Market 1129 to County Road 1030



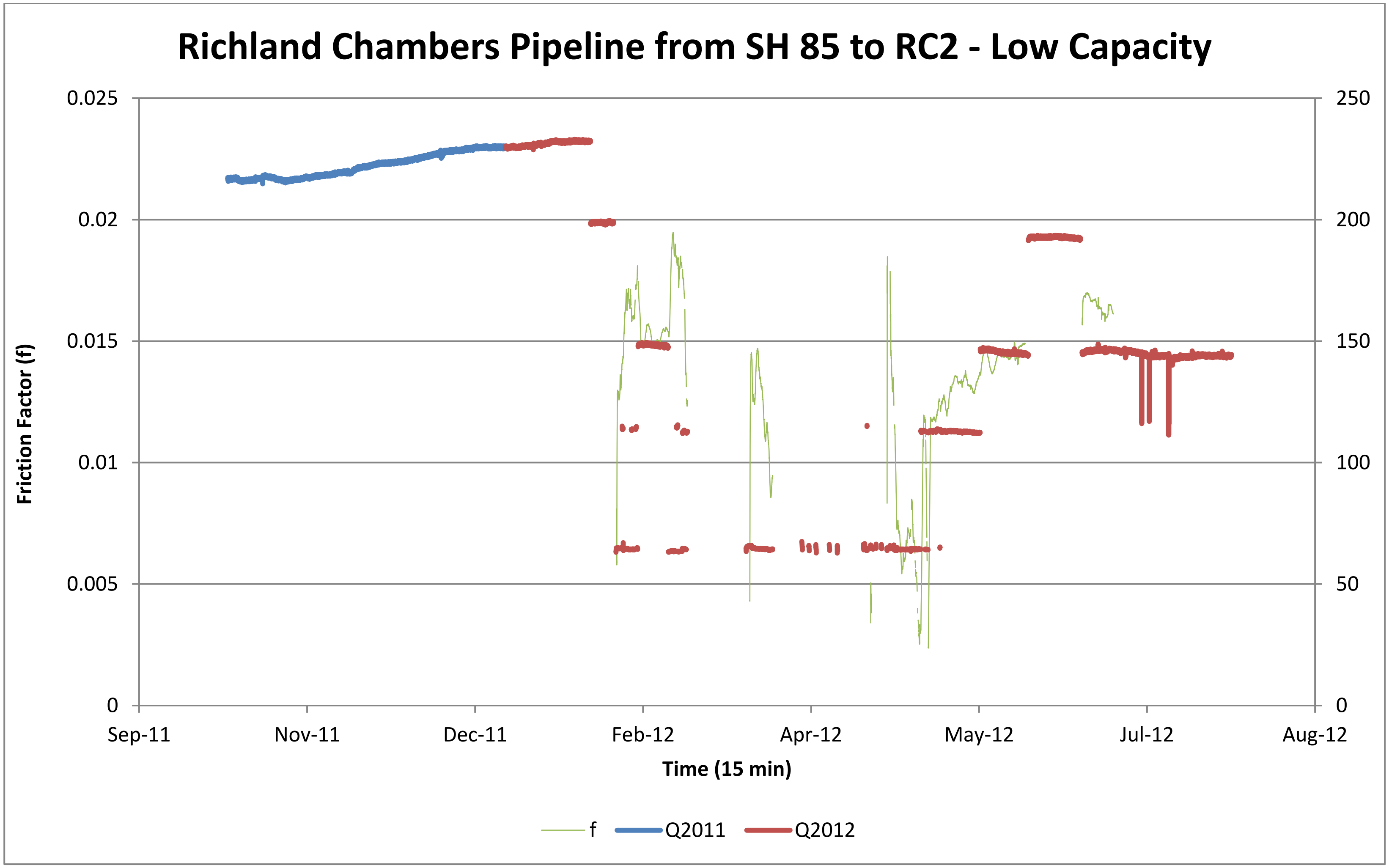
County Road 1030 to Barker Road



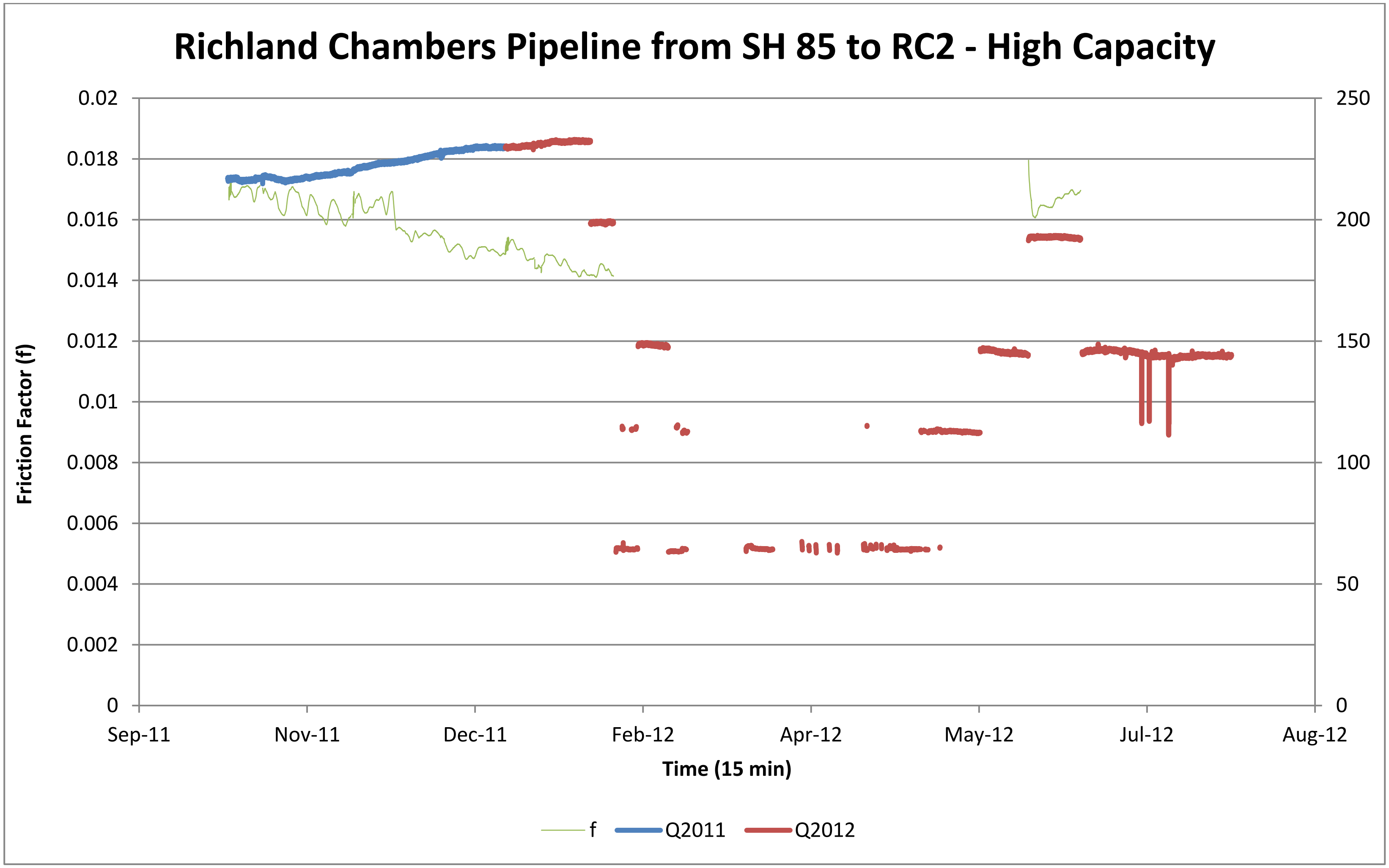
Barker Road to State Highway 85



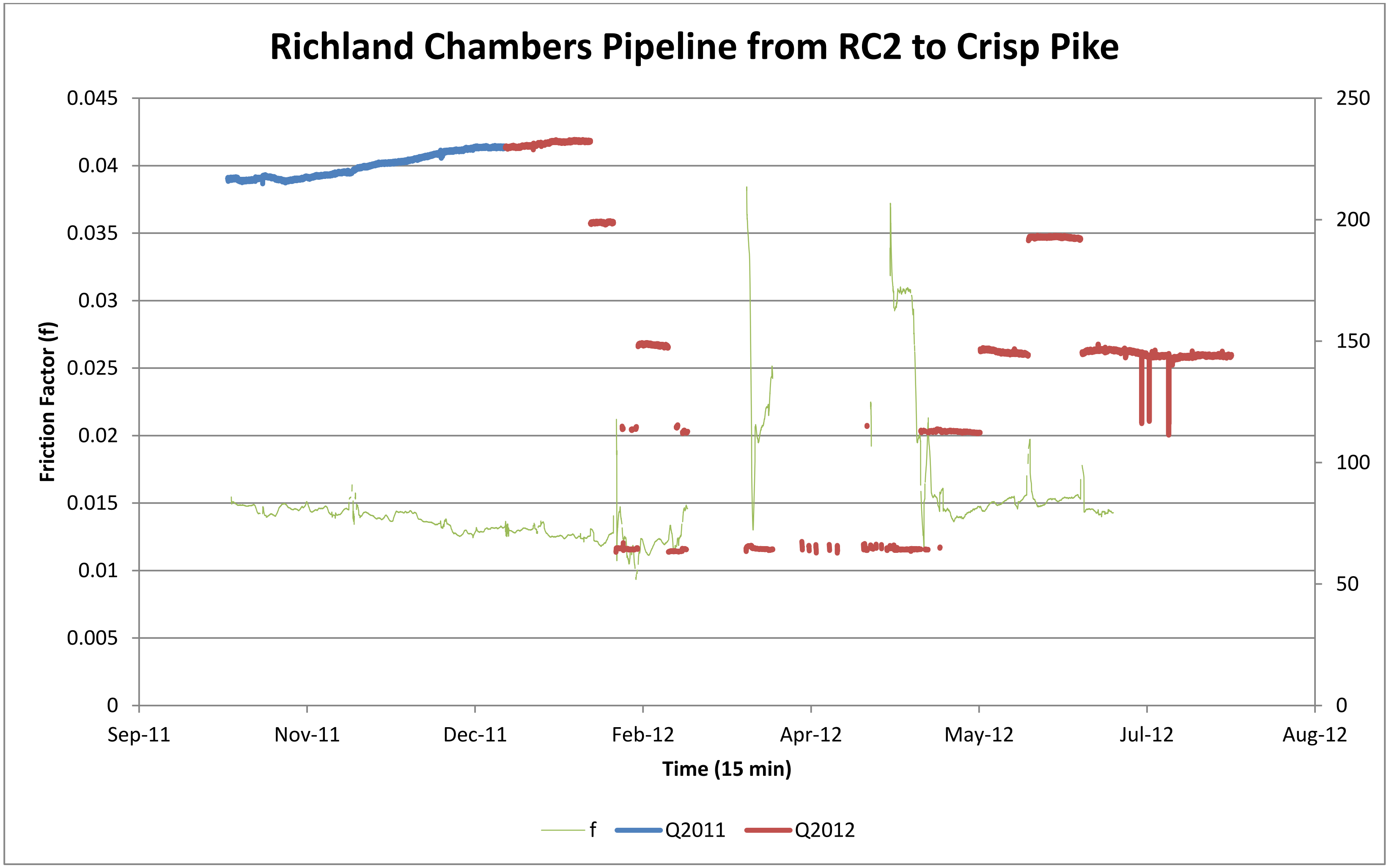
State Highway 85 to Richland Chambers Ennis Pump Station – Low Capacity



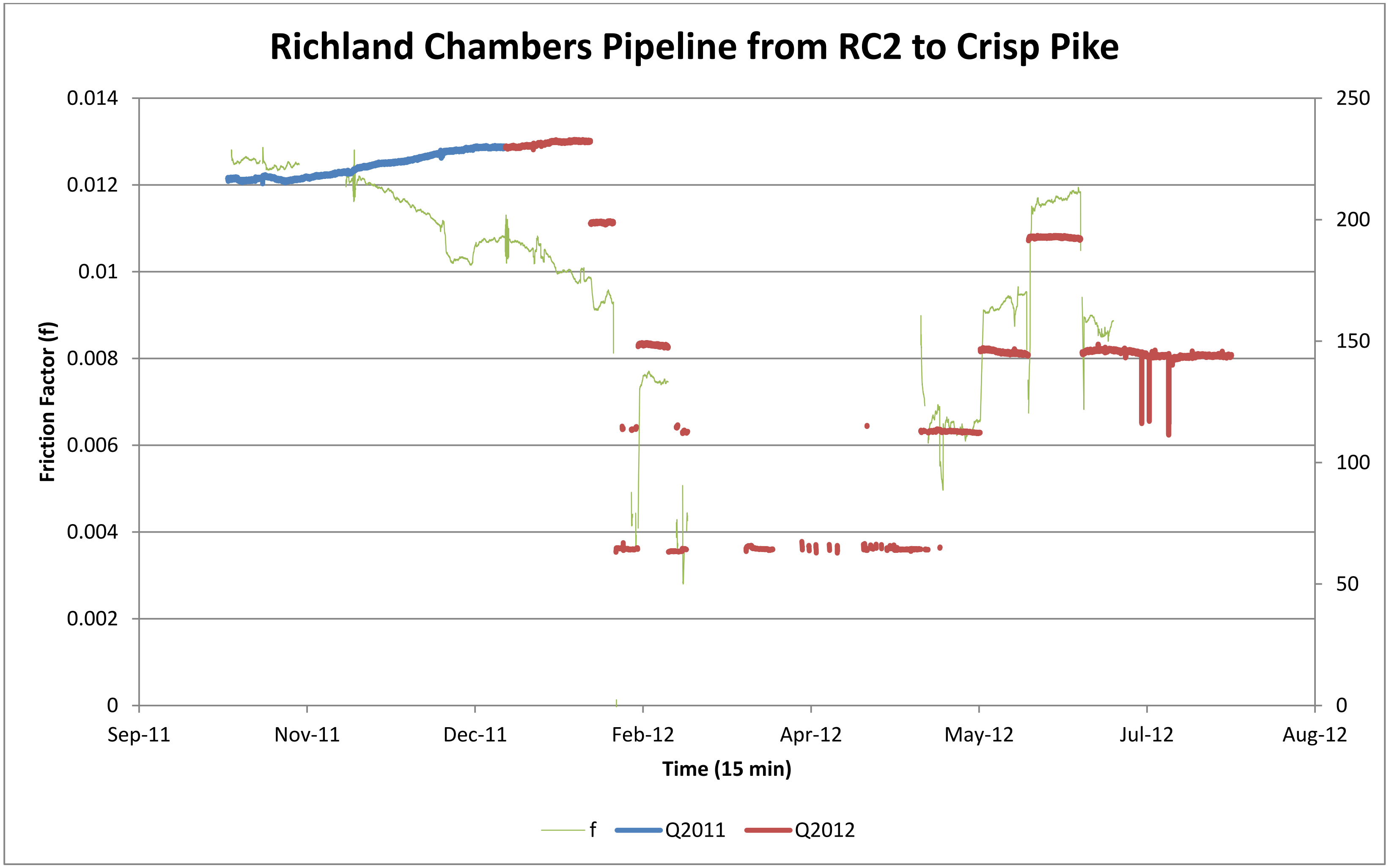
State Highway 85 to Richland Chambers Ennis Pump Station – High Capacity



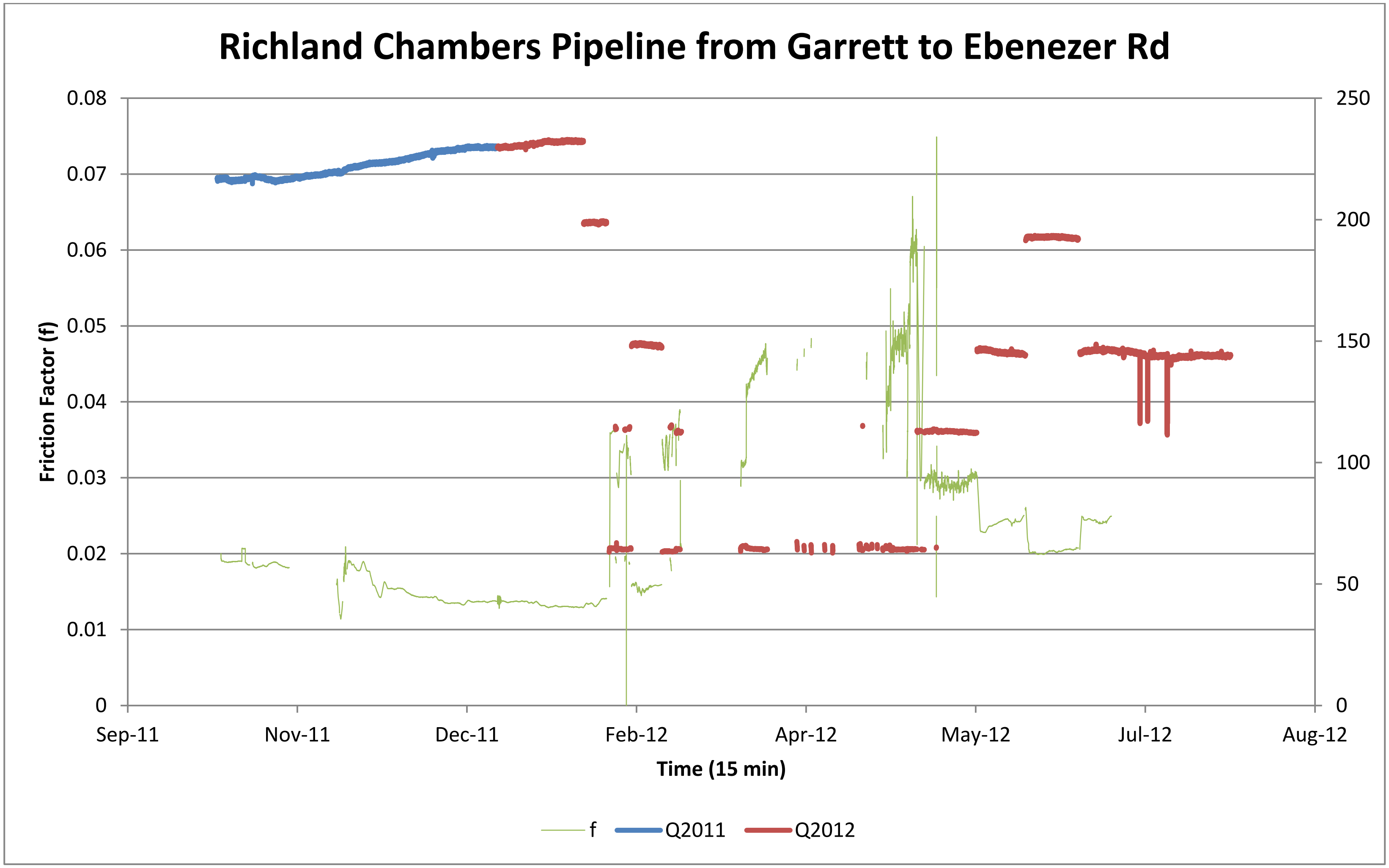
Richland Chambers Ennis Pump Station to Crisp Pike Road



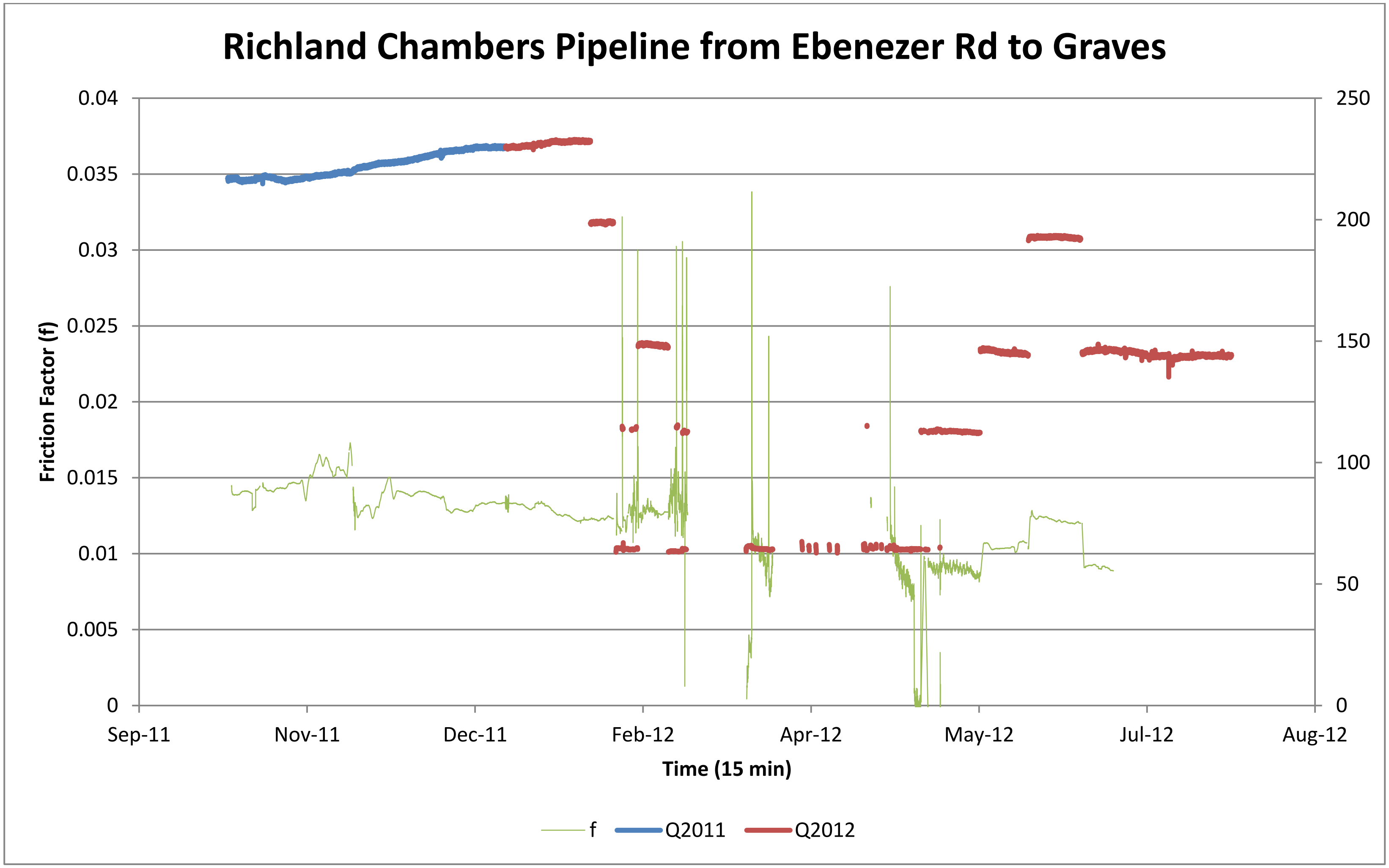
Crisp Pike Road to Garrett Road



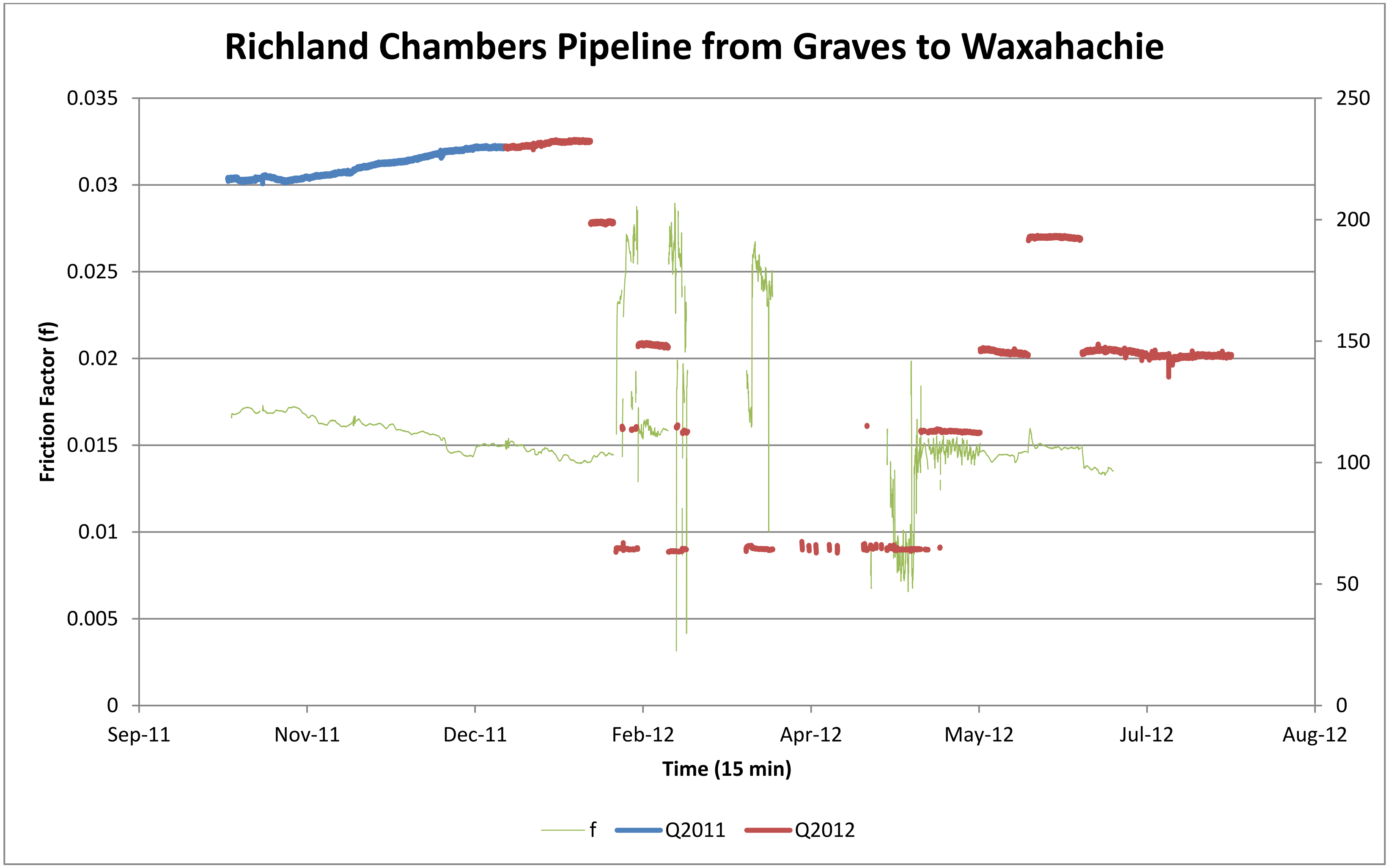
Garrett Road to Ebenezer Road



Ebenezer Road to Graves Road



Graves Road to Richland Chambers Waxahachie Pump Station



APPENDIX G

SURVEY

265

Project information		Coordinate System	
Name:	F:\Projects\Ennis\20111021 Elevations for Shelly at EPS.vce	Name:	US State Plane 1983
Size:	132 KB	Datum:	NAD 1983 (Conus)
Modified:	10/24/2011 9:34:08 AM (UTC:-5)	Zone:	Texas North Central 4202
Reference number:		Geoid:	GEOID09 (Conus)
Description:		Vertical datum:	

Additional Coordinate System Details

Local Site Settings			
Project latitude:	?	Ground scale factor:	1
Project longitude:	?	False northing offset:	0.000 ft
Project height:	300.000 ft	False easting offset:	0.000 ft

Point List

ID	Easting (US survey foot)	Northing (US survey foot)	Elevation (US survey foot)	Feature Code
0001	2567968.414	6814866.628	453.936	base
0002	2567996.844	6814807.201	453.546	backsite
0004	2567954.327	6814711.598	453.670	rc2 flow
0005	2568048.941	6814710.449	453.496	rc2 station pressure
0006	2568045.017	6814900.983	453.014	cc tank south
0008	2568090.668	6815024.212	453.068	cc tank north
DPKC0811	2647929.836	6893824.404	459.055	

Project information		Coordinate System	
Name:	F:\Projects\Waxahachie\20111021 Elevations for Shelly at WPS.vce	Name:	US State Plane 1983
Size:	135 KB	Datum:	NAD 1983 (Conus)
Modified:	10/26/2011 7:51:31 AM (UTC:-5)	Zone:	Texas North Central 4202
Reference number:		Geoid:	GEOID09 (Conus)
Description:		Vertical datum:	

Additional Coordinate System Details

Local Site Settings			
Project latitude:	?	Ground scale factor:	1
Project longitude:	?	False northing offset:	0.000 ft
Project height:	300.000 ft	False easting offset:	0.000 ft

Point List

ID	Easting (US survey foot)	Northing (US survey foot)	Elevation (US survey foot)	Feature Code
0001	2481787.396	6847580.272	617.820	rc3h flow
0002	2481756.543	6847713.313	617.033	rc3h station discharge pressure
0003	2481806.180	6847608.384	617.648	rc3h finish floor
0004	2481790.602	6847608.304	617.573	rc3h finish floor 2
0005	2481943.162	6847712.190	618.066	rc3l finish floor 2
0006	2481918.445	6847835.280	616.449	cc3 finish floor 2
0007	2482120.586	6847884.379	616.343	cc tank west 2
0008	2482114.945	6847886.805	616.571	cc tank west 2
ID	Easting	Northing	Elevation	Feature Code

	(US survey foot)	(US survey foot)	(US survey foot)	
0009	2482308.800	6847919.345	616.101	cc tank east
0010	2481926.894	6847503.634	618.139	rc3 tank
0001	2679004.002	6705858.698	338.749	natural ground
0002	2679000.660	6705819.763	339.459	finished floor rc1
0006	2697866.004	6779583.620	343.949	finished floor cc1
0007	2679153.418	6705840.003	340.161	top of pipe 1 @ rc1
0008	2679159.597	6705843.621	340.133	top of pipe 2 @ rc1
206	6814935.066	2568040.384	453.690	cc tank top curb
207	6814880.507	2568060.978	453.536	cc tank top curb
208	6814870.551	2568101.908	453.664	cc tank top curb
209	6815025.871	2568211.771	453.748	cc north tank
210	6815072.654	2568187.138	453.696	cc north tank
211	6815075.790	2568144.608	453.819	cc north tank
036	6814696.403	2567917.189	453.854	rc ennis pmp st flr

Tape Down Measurements:		Elevation:
Pipe 1	12.3	327.861
Pipe 2	10.3	329.833

Hole 1: the one closest to the pump station

Hole 2: the hole farthest from the pump station

Ennis Survey Shots

Waxahachie Survey Shots

REFERENCES

Andrewartha, J., K. Perkins, J. Sargison, J. Osborn, G. Walker, A. Henderson, and G. Hallegraeff. "Drag Force and Surface Roughness Measurements on Freshwater Biofouled Surfaces." *BIOFOULING* 26, no. 4 (2010): 487–496.

Barton, Andrew F, Michael R Wallis, Jane E Sargison, Alexandru Buia, and Gregory J Walker. "Hydraulic Roughness of Biofouled Pipes, Biofilm Character, and Measured Improvements from Cleaning." *JOURNAL OF HYDRAULIC ENGINEERING-ASCE* 134, no. 6 (June 2008): 852–857.

BRETT, TM. "HEAD-LOSS MEASUREMENTS ON HYDROELECTRIC CONDUITS." *JOURNAL OF THE HYDRAULICS DIVISION-ASCE* 106, no. 1 (1980): 173–190.

Characklis, W.G. "Biofilm Development and Destruction in Turbulent Flow" presented at the Annual Cooling Tower Institute Meeting, Houston, Texas, January 1979.

Lewandowski, Z., and P. Stoodley. "Flow Induced Vibrations, Drag Force, and Pressure Drop in Conduits Covered with Biofilm." *WATER SCIENCE AND TECHNOLOGY* 32, no. 8 (1995): 19–26.

PICOLOGLOU, BF, N. ZELVER, and WG CHARACKLIS. "BIOFILM GROWTH AND HYDRAULIC PERFORMANCE." *JOURNAL OF THE HYDRAULICS DIVISION-ASCE* 106, no. 5 (1980): 733–746.

Stoodley, P., J. Boyle, A. B. Cunningham, I. Dodds, H. M. Lappin-Scott, and Z. Lewandowski. “Biofilm Structure and Influence on Biofouling under Laminar and Turbulent Flows.” In *BIOFILMS IN THE AQUATIC ENVIRONMENT*, edited by C. W. Keevil, A. Godfree, D. Holt, and C. Dow, 13–24. ROYAL SOCIETY OF CHEMISTRY SPECIAL PUBLICATIONS. THOMAS GRAHAM HOUSE, SCIENCE PARK, CAMBRIDGE CB4 4WF, CAMBS, ENGLAND: ROYAL SOC CHEMISTRY, 1999.

Stoodley, P., I. Dodds, J. D. Boyle, and H. M. Lappin-Scott. “Influence of Hydrodynamics and Nutrients on Biofilm Structure.” *JOURNAL OF APPLIED MICROBIOLOGY* 85, no. Suppl. S (1999): 19S–28S.

Stoodley, P., Z. Lewandowski, J. D. Boyle, and H. M. Lappin-Scott. “Oscillation Characteristics of Biofilm Streamers in Turbulent Flowing Water as Related to Drag and Pressure Drop.” *BIOTECHNOLOGY AND BIOENGINEERING* 57, no. 5 (March 5, 1998): 536–544.

Vo, Garret Dan, Eric Brindle, and Jeffrey Heys. “An Experimentally Validated Immersed Boundary Model of Fluid-biofilm Interaction.” *WATER SCIENCE AND TECHNOLOGY* 61, no. 12 (2010): 3033–3040.

BIOGRAPHICAL INFORMATION

Shelly Hattan is a professional engineer who works in the water resources industry. She earned her Bachelors of Science from the University of Texas at Arlington, Texas in December of 1994. Her research interests are geared toward pipeline transmission from the construction through operations. She is currently assigned to the Integrated Pipeline Project which is a regional partnership between the Tarrant Regional Water District and the City of Dallas. Her future plans include possibly continuing her biofilm research to earn a Doctorate of Science.