REBUTTAL TESTIMONY OF ROBERT C. EDMUNDS, P.E.

BEFORE THE FLORIDA PUBLIC SERVICE COMMISSION

ON BEHALF OF

SOUTHERN STATES UTILITIES, INC.

DOCKET NO. 950495-WS

Q. PLEASE STATE YOUR NAME AND BUSINESS ADDRESS FOR THE RECORD.

A. My name is Robert C. Edmunds, P.E. My business address is Jones Edmunds & Associates, Inc., 730 N. Waldo Rd., Gainesville, Florida 32601.

Q. ARE YOU THE SAME ROBERT C. EDMUNDS WHO PREVIOUSLY PROVIDED DIRECT TESTIMONY?

A. Yes, I am.

Q. HAVE YOU REVIEWED THAT PORTION OF THE PREFILED DIRECT TESTIMONY OF OPC WITNESS TED BIDDY WHICH CONCERNS HYDRAULIC MODELING?

A. Yes, I have.

Q. DO YOU AGREE WITH MR. BIDDY'S TESTIMONY REGARDING HYDRAULIC MODELING?

A. No, I do not, and I would like to specifically address several aspects of Mr. Biddy's testimony regarding hydraulic modeling. First, it is inconceivable to me to suggest, as Mr. Biddy does, that the Commission ignore hydraulic modeling when, as I explained in my prefiled direct testimony, hydraulic modeling is the preferred and the most accurate way of quantifying the actual used capacity of water transmission and distribution facilities. Once the appropriate flow rate is selected to apply for used and useful determinations, it is indisputably true that no more valid technique exists for projecting the actual flow in each and every pipe than hydraulic modeling, short of installing devices to record simultaneous flow rate measurements in each and every pipe. This latter alternative would be so complicated and costly as to be impractical; consequently, hydraulic modeling is the only valid, realistic approach. The lot-count method cannot even be characterized as a method for evaluating used capacity and is absolutely and undeniably erroneous by comparison. I also disagree with Mr. Biddy's statements regarding calibration. Calibration is not, as he suggests, mandatory for hydraulic models in all cases. Additionally, I note that Mr. Biddy avoids entirely the importance of having used and useful considerations parallel design requirements.

Q. WOULD YOU ADDRESS MR. BIDDY'S ASSERTION THAT THE LOT-COUNT METHOD IS A BETTER METHOD THAN THE HYDRAULIC MODELING ANALYSIS TO EVALUATE USED AND USEFUL FOR DISTRIBUTION AND TRANSMISSION FACILITIES?

A. I disagree with Mr. Biddy in a very fundamental sense. Current connections utilize that portion of the transmission and distribution facilities which are required to meet the existing demand conditions placed on the facilities by those connections. The hydraulic modeling analysis will clearly quantify those demands. The hydraulic analysis is a flow-based approach similar to the flow-based approach utilized by the Commission in the past for evaluating used and useful for other components of water service facilities, and which Mr. Biddy himself recommends for those other water plant components. The lot-count method has no rational correlation whatsoever to the demand placed on transmission and distribution facilities by current customers and should be rejected on that basis alone.

Q. HAS YOUR FIRM PERFORMED A FIELD CALIBRATION OF THE TRANSMISSION AND DISTRIBUTION FACILITIES SERVING SSU'S PINE RIDGE SERVICE AREA?

A. Yes, we have.

Q. COULD YOU DESCRIBE THE RESULTS OF THAT CALIBRATION?

A. Yes. The calibration testing confirmed the validity of the hydraulic model for the east part of the Pine Ridge service area. In addition, test results clearly indicate that following installation of appropriately placed air release valves to purge entrapped air, the west part of the Pine Ridge model will achieve full calibration as well.

Q. **COULD YOU DESCRIBE HOW THE PINE RIDGE FACILITIES WERE CALIBRATED?**

A. Yes. A copy of the calibration report prepared under my supervision and control is identified as Exhibit _____ (RCE-1). To perform calibration, the Pine Ridge distribution facilities were hydraulically stressed at various locations by opening fire hydrants, with flows and pressures measured or computed at key locations. The field measured values then were compared with values predicted by the hydraulic model. The eastern part of the Pine Ridge model was immediately found to be satisfactorily calibrated, but the western part was found to be experiencing pressures as much as 13 psi lower than predicted by the model. As explained in the calibration report, experienced pressures within approximately 5 psi of modelled pressures are typically considered acceptable. Using the model as an investigative tool, a specific piping reach was found to be air bound. Upon air purging, a 12.5 psi measured versus modeled pressure disagreement was reduced to 5.3

psi. This indicates that, following installation of appropriate air release valves, the western part of the Pine Ridge model would be expected to achieve satisfactory calibration as well.

Q. **ON THE SUBJECT OF CALIBRATION, YOU SAID YOU DISAGREE WITH MR. BIDDY'S STATEMENT THAT CALIBRATION IS REQUIRED FOR HYDRAULIC MODELS THAT ARE UTILIZED TO EVALUATE USED AND USEFUL. COULD YOU EXPLAIN YOUR STATEMENT.**

A. Yes, I believe Mr. Biddy errs in stating an absolute regarding the need for calibration. Calibration is important in many cases; in other cases, it is less important. In designing new facilities, for example, modeling is relied on without the benefit of field calibration. Further, in certain cases, it is perfectly appropriate to undertake measures short of full calibration to confirm the reliability of a model's results. Whether a hydraulic model should be fully calibrated depends on a number of factors, particularly the cost-effectiveness of full calibration in light of the use being made of the model. Full calibration is a fairly expensive proposition. For the service areas the size of the four at issue in this case, complete calibration could cost anywhere in the approximate range of $25,000 to $60,000 for each service area, depending upon the difficulties encountered.

Q. COULD YOU ADDRESS THE NEED FOR FULL CALIBRATION ON THE SSU MODELS OTHER THAN PINE RIDGE?

A. There are several factors the Commission must keep in mind regarding the need for calibrating all of the models in this case. Considering all of these factors, I do not believe it necessary to require SSU to fully calibrate all four of the models submitted.

As I have stated, calibration, while always desirable, is not a mandatory industry practice in all cases. Hydraulic modeling is an important tool used regularly by practicing professional engineers to evaluate utility facilities for various purposes. In this case, the model is being used as a tool to compile flow ratios to arrive at a used and useful percentage. Considering this use to which the model is being put, I do not believe full calibration is particularly essential. However, I think it desirable to have adequate insurance that the ratios developed have a sufficient correlation to the facilities capabilities, and SSU has provided as much in this case through (1) the confirmation of the Pine Ridge model results as I have already explained and as stated in the calibration report and (2) Mr. Terrero's direct knowledge that all four of the distribution networks at issue were designed in the same way, constructed at about the same time, by the same firm, in accordance with those designs using the same materials. If deemed necessary, spot-testing of facility performance, rather than full calibration, may also be a useful verification mechanism to demonstrate that the model accurately reflects actual hydraulic performance. One additional consideration which carries somewhat more weight than those I just mentioned concerns how SSU's models were developed. In creating the steady state models for this filing, SSU made assumptions of a conservative nature, regarding peak demand per equivalent residential connection in particular, such that calibrated results would very likely reveal overall current flows throughout each distribution network higher than those in the models SSU filed. Thus, the used and useful computations should be relatively insensitive to minor variations in actual versus modeled flows.

Q. YOU MENTIONED EARLIER THAT MR. BIDDY IGNORES THE

IMPORTANCE OF HAVING USED AND USEFUL CONSIDERATIONS PARALLEL DESIGN REQUIREMENTS. COULD YOU EXPLAIN WHAT YOU MEAN?

A. Yes. Mr. Biddy acknowledges, at page 5 line 17 of his testimony, that mains must be sized to accommodate fireflow. He also seems to concede proper distribution network design requires system looping, for instance at page 18, line 6 of his testimony. He acknowledges, at page 15, line 8, that a hydraulic model is a reliable design tool. But he then concludes that design considerations should not be the same as used and useful considerations for distribution and transmission facilities. As I mentioned above, Mr. Biddy consistently invokes design considerations to support his views as to the used and useful percentages of all other water facility components, but eschews them as to transmission and distribution facilities.

Mr. Biddy does not address, and therefore seems wholly unconcerned with, the message the Commission sends utilities and design engineers through his proposed use of the lot-count method. As stated in my direct testimony, that message to utilities and engineers is basically two-fold: 1) design and construct transmission and distribution facilities properly at the utility's economic peril and 2) ignore available economies of scale.

Mr. Biddy states that the lot-count method recognizes an allowance for fireflow and looped lines in that current customers have allocated to them a portion of the total cost for all transmission and distribution lines throughout a service area or defined portion thereof. I believe Mr. Biddy glosses over several key points I made in my direct testimony.

Under the lot-count method, a utility's ability to recover investment associated with looping installations is entirely dependent upon the number of customers, if any, which connect directly to the loop lines. Thus, the utility's ability for meaningful recovery of investment associated with looping facilities is subject to an unknown variable. Contingent recovery of this sort, I maintain, poses little incentive to a utility to loop lines where installation of such facilities is required by design criteria to insure adequate and proper service to the customers. Mr. Biddy would put a utility in a position of being required to install looping facilities but being completely uncertain as to its ability to recover the costs therefor.

Another critical point Mr. Biddy glosses over is that the lot-count method attributes to current connections only a small fraction of that portion of the existing lines' capacity needed to meet the water service requirements of those current connections. As a result, the lot-count method provides little or no incentive to the utility to size its lines in accordance with the design standards and requirements mentioned in my direct testimony and basically penalizes the utility for proper design.

Mr. Biddy also apparently attempts to bolster his argument by stating that even under the lot-count method, current connections must bear a portion of the additional cost of a utility's sizing lines to accommodate a defined buildout condition. This, I believe, is an irrelevant consideration, primarily because a flow-based used and useful approach allocates these so-called additional costs to future customers anyway and also because current connections will benefit from the offsetting savings associated with a one-time facilities installation designed to meet a buildout condition (i.e., the economies of scale, avoided cost of facilities upgrading, and time value of money) when future connections come on line. Using Mr. Biddy's proposal, a utility would not be able to recover its full investment in transmission and distribution facilities even if the utility sized and structured such facilities to serve only current connections.

The more rational approach for measuring used and useful for transmission and distribution facilities is one which represents that portion of installed facilities utilized to meet the needs of current connections, incents a utility to follow design criteria, and incents a utility to take advantage of economies of scale. The hydraulic analysis approach fulfills all of these criteria infinitely better than the lot-count method.

Q. DO YOU HAVE ANYTHING FURTHER TO ADD?

A. No, not at this time.

STEADY-STATE MODEL CALIBRATION OF PINE RIDGE WATER TRANSMISSION AND DISTRIBUTION NETWORK

Presented to:

SOUTHERN STATES UTILITIES, INC.
Apopka, Florida

Presented by:

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1.0 INTRODUCTION

1.1 PURPOSE

A steady-state hydraulic model mathematically simulates the pressure and flow performance of a hydraulic network. Model calibration is performed for three purposes:

A. To verify the validity of the mathematical model in simulating network performance.

B. To identify and assist in resolving discrepancies in model versus network performance.

C. To "fine tune" model parameters for optimum model accuracy in the variety of expected demand conditions.

The purpose of this report is to evaluate the collected field data and the model calibration effort of the Pine Ridge water distribution network.

1.2 SCOPE

The scope of the work presented herein is focused on a general discussion of hydraulic modeling, collection and analysis of field data, air binding, localized model calibration, and circumstances associated with overall calibration of the Pine Ridge water distribution model.

2.0 HYDRAULIC MODELING

2.1 THEORY

Two basic principles are involved in steady-state modeling. These principles are the conservation of mass and the First Law of Thermodynamics. The conservation of mass principle states that the time rate of change of the system mass equals zero. The application of this principle leads to the continuity equation. The First Law of Thermodynamics states that the time rate of increase of the total stored energy of the system equals the net time rate of energy addition by heat transfer into the system plus the net time rate of energy addition by work transfer into the system. Steady-state application of this law leads to the energy equation. Energy dissipation due to wall shear stress (i.e., the energy lost due to friction at the pipe wall) is the most difficult term in the energy equation to accurately describe. The Hazen-Williams equation is an industry standard and is used herein to describe this energy dissipation.

Although manual solution to the energy and continuity equations is possible, it is very time consuming and prohibitive as a practical matter. Therefore, it is advantageous to solve the equations by use of a steady-state hydraulic computer program.

2.2 MODELING PROGRAM

The computer program used in this steady-state model calibration is Cybernet by Haestad Methods. Cybernet is basically a version of Kentucky Pipes with an AutoCAD graphical interface. Specifically, Cybernet solves the pressure network using the state-of-the-art KYPIPE2 computational algorithm. The program permits use of a variety of boundary conditions including constant head (given as elevation), pumps, constant demand, valves, and storage tanks. Pumps may be represented as useful power or by using head-discharge data from a pump curve.

2.3 MODEL DESCRIPTION

The first step in modeling a network of pipes is to describe the network as a series of nodes connected by pipe sections. This description results in a steady-state schematic representing pipe sections and nodes with a line-circle diagram.

A pipe section is described as constant diameter sections of pipe that may contain minor loss elements such as valves or bends. A complete pipe section description contains the section length, inside diameter, and pipe roughness. Pipe roughness is primarily a function of pipe material. Depending on pipe material and water chemistry, the pipe roughness may change with age. Pipe roughness is input in this model as the Hazen-Williams "C" coefficient. The Hazen-Williams "C" coefficient is a function of pipe roughness, pipe diameter, and the Reynold's number of flow in the pipe.

End points of pipe sections are connected by nodes which can be one of two types: junction nodes or fixed-grade nodes. Junction nodes are nodes located at the intersection of two or more pipes where flow is removed or added to the network. Fixed-grade nodes are nodes where both the elevation and pressure are known, such as at network discharge point.

Pumps used in the analysis are located in pipe sections and are described using a minimum of three points from the head-discharge curve. Other network components used in this analysis are pressure regulating valves (PRVs) and a check valve.

2.4 DEVELOPMENT OF PINE RIDGE WATER DISTRIBUTION MODEL

The two most important factors involved in the development of a representative model of a water distribution network are distribution of demand to nodes and accurate representation of the physical elements of the network. The Facilities Analysis Department of Southern States Utilities, Inc. (SSU) has assumed this responsibility.

SSU used water sales records from September 1994 to August 1995 to determine the current average daily flow (ADF) of each customer in Pine Ridge. The demand of each customer was then allocated to a hydraulically nearby node in the model. Customers that live in close hydraulic proximity to each other generally have their demands allocated to the same node.

SSU developed the model by use of construction plans, record drawings, well installation records, and accountant records. The current model is composed of 1,099 pipes, 989 junction nodes, 4 fixed-grade nodes, 3 well pumps, 2 booster pumps, 1 check valve, and 3 PRVs. Calibration of the model is dependent on the actual operational performance of the check valve, Field Booster Pump No. 1, and all three PRVs. Although only one booster pump (Model Booster Pump No. 2) was used during the calibration effort, operational performance of all pumps have been examined and the model adjusted accordingly. The PRVs act as control points in the model. For each simulation, the downstream set points of the PRVs in the model have been set to the actual hydraulic grade measured during each test event.

2.5 ADJUSTMENT OF DEMAND FOR SIMULATIONS

Network demand in the model may be adjusted to represent overall customer demand during any test by applying a multiplication factor to the nodal demands supplied by SSU. This effectively prorates the increase or decrease in overall network demand versus overall model ADF to all nodal demand locations equally.

2.6 MODELING AND THE NEED FOR MODEL CALIBRATION

The industry standard in modeling water distribution networks is to model required hydraulic elements (such as pumps, PRVs, check valves), ignore local losses, and apply a global Hazen-Williams "C" coefficient to the model for pipes of similar size, material, and internal condition. Some of the considerations associated with this type of modeling are as follows:

A. The Hazen-Williams "C" coefficient is a function of pipe inside diameter, pipe roughness, and the Reynold's number of flow in the pipe.

B. The Hazen-Williams equation is an empirical equation that describes the frictional energy loss in the pipe. However, the equation has to be adjusted to account for local energy losses. (i.e., fitting losses, etc.)

C. Depending on pipe material and water chemistry, the pipe roughness and inside diameter may change with age.

D. The hydraulic performance of certain elements in the water network and facilities may deteriorate.

E. Other factors, such as air binding, network blockages, installed utilities differing from those in utility records, etc., may affect network performance.

Therefore, it is sometimes difficult for a model to accurately predict pressure and flow distribution in real water transmission and distribution networks. Model calibration is performed for reliable prediction of field pressure and flow distribution. Typically, a model is considered calibrated if it can predict field pressures within 5 psi. However, if fluctuations are 10 psi or greater and occur at fairly short intervals, one must select a pressure level during a cycle (a high, medium, or low point) and attempt to calibrate the model for that condition, recognizing that there are some inherent inaccuracies in using a steady-state model to describe unsteady conditions (***Water Systems: Simulation and Sizing***, Walski, Gessler and Sjostrom).

3.0 FIELD CALIBRATION

3.1 PROGRAM

Prior to developing a field test program the following events occurred:

A. Production meter calibration.

B. Well pump capacity tests.

C. Week long data logging for development of diurnal curves.

D. Survey of test locations for elevations.

The field test program was developed by selecting specific hydrants to impose a demand that hydraulically stressed the facilities by dropping local pressures in the network to 20 psi. The number of supply sources was kept to the minimum number which could provide for current customer and test demands while maintaining adequate network pressure performance. The test configuration included a listing of the operating status of all supply wells, booster pumping station, PRVs, and locations of pressure and flow monitoring points.

Each field test configuration included the following items:

A. Monitor each operating well for flow, pressure, and hydropneumatic tank level.

B. Monitor each booster pump for suction and discharge pressure.

C. Monitor each PRV for pressure upstream and downstream of the valve.

D. Monitor each operating hydrant for flow and monitor residual pressure at a location nearby.

E. Monitor network pressure at selected residual monitoring points.

Pressure gauges were calibrated in the installed position by JEA prior to the test (previous day) using a dead weight calibrator.

Specifically, five tests were planned. In all tests, pumps and hydropneumatic tanks at Well Nos. 2 and 3 were valved off. This simplified the facilities by making Well No. 4 the only supply source. Pressures were recorded at all the monitoring points listed above at various times for each test scenario.

Test 1 consisted of stressing a hydrant on West Ranger Street at approximately 300 GPM and recording residual pressure on West Deputy Drive.

Test 2 consisted of stressing a hydrant on North Hatchet Circle at approximately 300 GPM and recording residual pressure on Tomahawk Drive.

Test 3 consisted of stressing a hydrant on West Pine Ridge Boulevard at approximately 300 GPM and recording residual pressure on West Cavalry Lane.

Test 4 consisted of stressing a hydrant on North Buffalo Drive at approximately 300 GPM and recording residual pressure on North Buffalo Drive.

Test 5 consisted of stressing a hydrant on North Red Ribbon Point at approximately 400 GPM and recording residual pressure on North Princewood Drive.

3.2 FIELD DATA

Two field efforts were performed for data acquisition necessary for model calibration. The field efforts were performed on November 17, 1995 and January 16, 1996. The information gathered during the second field effort is more detailed and is deemed more reliable. The January 16, 1996 collected field data is presented in Attachment 1. Comparison of measured to modeled pressures is presented in Attachment 2 (including subsequently determined closed and throttled valve status).

3.3 DATA ANALYSIS

Comparison of the field data to the model output data indicated that differences in field versus model pressures generally in excess of a 5 psi to 10 psi range were occurring in the western part of the network when that part of the network was hydraulically stressed by hydrant flow. The consistency of this modeled versus measured difference at the pressure monitoring points indicated that there was a physical explanation for the head loss. It was believed that the head loss was due to one or more of the following:

A. Air binding may be occurring in the network.

B. An obstruction may exist in the network. This may be a closed valve(s) or a physical obstruction in one or more pipes.

C. Installed pipe(s) may be different in size or connection from modeled pipe(s).

D. The roughness of a pipe(s) may have deteriorated to the point that it is responsible for the head loss.

A comparison of field and model pressures is presented in Attachment 2. Copies of input and output files for these simulations are available upon request.

The data analysis indicated that a field investigation of the operational status of all the valves in the pipeline that runs along Pine Ridge Boulevard would have to be performed.

3.4 FIRST FIELD INVESTIGATION

On February 2, 1996, SSU performed a field investigation in an attempt to locate the source of the head loss. The results of the field investigation are as follows:

A. A fully closed field valve (10 inch gate valve) was found on the eastern side of the tee that connects modeled pipe nos. 511, 516, and 3241.

B. A field valve (12 inch gate valve) 7/36th closed was found in model pipe no. 851.

C. A notable head loss was found at the northern connection between the eastern and western parts of the network.

D. The pressure at the hydrant closest to Pine Ridge Boulevard and North Perry Drive (Perry Hydrant) was not fluctuating as was the pressure at the hydropneumatic tank at Well No. 4.

E. Closing and opening of a valve on North Perry Drive appeared to remove the source of the head loss and pressures began fluctuating at the referenced hydrant in synchronization with the pressure at the hydropneumatic tank at Well No. 4.

3.5 SECOND FIELD INVESTIGATION

A second field investigation to evaluate the overall network performance was conducted by SSU and JEA on February 28, 1996 and February 29, 1996.

On the first day (2/28/96), Test 3 with pressure monitoring points at West Cavalry Lane, North Buffalo Drive, and Well No. 2 was repeated. The results of the test indicated that the western part of the network was again experiencing pressure losses in excess of the 5 to 10 psi range. As a consequence, the hydrant flow was allowed to continue and the network was investigated along the 8 inch main on Pine Ridge Boulevard. The results of this investigation indicated that a notable local pressure loss was accruing between Perry Hydrant and the hydrant closest to the intersection of Pine Ridge Boulevard and North Carnation Drive (Carnation Hydrant). This section is represented by model pipe nos. 631, 771, 776 and 781. The result of this investigation is herein referred to as Obstruction Test (2/28/96). Additional investigation found as follows:

A. A closed field valve (8 inch gate valve) was encountered in model pipe no. 2787.

B. The pressure at Perry Hydrant was not fluctuating with the hydropneumatic tank pressure at Well No. 4.

C. Manipulation of network operation to isolate and flow the 8 inch main on Pine Ridge Boulevard, and subsequent opening of the Carnation Hydrant, resulted in air being expelled from the network.

D. After air was expelled from the network, the pressure at Perry Hydrant began fluctuating by 5 psi in synchronization with the pressure at the Well No. 4 hydropneumatic tank.

On the second day (2/29/96), further manipulation of network operation to backflow the referenced 8 inch main and subsequent opening of the Carnation Hydrant resulted in a significant amount of air expulsion from the network. Repeating Test 3, which is herein referred to as Obstruction Test (2/29/96), and monitoring pressures at Perry and Carnation

Hydrants indicated that the network performance was significantly closer to the performance predicted by the hydraulic model.

A comparison of the field and model pressures for the data collected during the second field investigation is presented in Attachment 3. Copies of input and output files for these simulations are available upon request. As indicated, expulsion of the air from the pipeline resulted in the pressure at the Perry Hydrant agreeing within 5.3 psi with the model pressure versus a 12.5 psi disagreement before air purging.

3.6 AIR BINDING

When enough air accumulates in a pipe, the cross-sectional area available for flow can be reduced. Should the cross-sectional area available for flow in the pipe be less than the full pipe cross-sectional area, the laws governing the flow in the pipe change from pipeline hydraulics to open channel hydraulics. This phenomenon is called air binding. Some of the results of air binding are reduced capacity and an energy loss equal to the vertical length of the air pocket(s) plus the energy dissipated in the hydraulic jump, if present. An article from the ***Journal of American Water Works Association***, written by Robert C. Edmunds, is provided in Attachment 4. The article gives a more detailed explanation of air binding. Case studies involving air binding are presented on page 276 of the article. The case studies are very useful in understanding the effects of air binding.

The results of the field efforts and investigations indicate a high probability that air binding exists as an intermittent or chronic condition in the western part of the network. Although air binding is not currently indicated in the eastern part of the network, it might occur. A theoretical analysis of air binding in pipe no. 631 (the descending leg between the two parts of the network) indicates the following:

A. Under normal network demand, the pipes have a high probability of becoming air bound due to the rolling terrain and the lack of air release valves.

B. Under fire flow demand, the pipes are more likely to be in the incipient to clearing phase of air binding.

The theoretical analysis is presented in Attachment 5. The Gandenberger curve was used in the theoretical analysis. As shown by the graph in Attachment 5, if the plotted point is below the line, air binding will occur; if the point is far above the line, air binding will not occur; and if the point is near the line air binding may be in an incipient phase. Note that an incipient phase is not necessarily a clearing phase.

3.7 MODEL CALIBRATION

Calibration of the hydraulic model for the eastern part of the network is considered complete. This is indicated by examination of the measured versus the modeled pressure at the North Princewood Drive Hydrant for all tests reported in Attachment 2. As indicated, the measured versus the modeled pressure agrees within 5.6 psi for all tests. However, examination of the measured versus modeled pressures at West Deputy Drive, North Buffalo Drive, Well No. 2, booster station (suction side), and West Cavalry Lane indicates disagreement by as much as 13 psi, with the measured pressure almost always below the modeled pressure for all cases. The measured pressure is always below the modeled pressure for cases where the western part of the network is stressed by hydrant flow. Also, as indicated, the measured versus modeled pressure disagreement is relatively consistent from point to point in the western part of the network. All of these observations are consistent with the finding of air binding in the 8 inch main connecting the eastern and western parts of the network. As indicated in section 3.5, purging of trapped air from the

8 inch pipeline reduced the measured versus modeled pressure disagreement at the Perry Hydrant from 12.5 psi to 5.3 psi. Because pressure loss in the 8 inch main affects pressures throughout the western part of the network, a comparable reduction in pressure discrepancies would be expected at all pressure measuring locations in the western part of the network as well. Consequently, it is our opinion that the pressure discrepancies and model calibration in the western part of the network are being adversely effected by occasional or chronic air binding. Installation of properly placed air release valves to purge pockets of entrapped air would be expected to permit the western part of the network to function hydraulically as indicated by the hydraulic model.

4.0 CONCLUSION AND RECOMMENDATIONS

4.1 RESULTS

The hydraulic model accurately predicts pressure within 5 psi for the eastern part of the network. Therefore, the model can be considered calibrated with respect to the eastern part. A head loss is experienced in the western part, which we believe is due to air binding. The results of various field investigations have confirmed the presence of air in the network and expulsion of some of the air from the network has resulted in a decrease of head loss in the western part of Pine Ridge.

Expulsion of air from the network resulted in the following:

A. Field pressure recorded at Well No. 2 went from 13.2 psi below model prediction to 8.18 psi below model prediction for the same test configuration.

B. Field pressure recorded at Perry Hydrant went from 12.48 psi below model prediction to 5.27 psi below model prediction for the same test configuration.

Following installation of devices that will allow air to be continually purged from the network, we expect that the model will calibrate at a C-value of 145.

4.2 RECOMMENDATIONS

The following recommendations are provided for operation of the Pine Ridge water transmission and distribution network.

A. Air release valves should be installed at critical points throughout the water distribution network.

B. Following this, if air binding persists, air traps should be installed at specific locations around all wells.

ATTACHMENT 1

TEST #1

EVENT	TIME	PUMP STATUS	HYDRANT STATUS OPEN/CLOSE	DISCHARGE PRESSURE (PSIG)	"PUMP ON" TOTALIZER READING (GAL)	"PUMP OFF" TOTALIZER READING (GAL)	HYDROTANK LEVEL (INCHES)	PRV #1 UPSTREAM PRESSURE (PSIG)	PRV #2 UPSTREAM PRESSURE (PSIG)	PRV #2 DOWNSTREAM PRESSURE (PSIG)	PRV #3 UPSTREAM PRESSURE (PSIG)	PRV #3 DOWNSTREAM PRESSURE (PSIG)	BOOSTER STATION PUMP #1 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #1 DISC. PRES. (PSIG)	BOOSTER STATION PUMP #2 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #2 DISC. PRES. (PSIG)	WELL #2 (PSIG)	BUFFALO (PSIG)	PRINCEWOOD (PSIG)	RESIDUAL HYDRANT (PSIG)	STRESSED HYDRANT MICROMETER (GPM)
1																					
2																					
	1:20:00	ON	CLOSED																		
3	1:21:00		CLOSED				16.75	64	62	54	53	40	56	96	58	97	56	52	64	64	0
4	1:27:00		CLOSED				21	73	75	54	61	50	66	105	72	105	64	54	61	72	0
	1:27:00	OFF	CLOSED																		
	1:33:00	ON	CLOSED																		
5	1:33:00		CLOSED				16	64	67	53	52	40	56	97	56	97	56	54	65	64	0
6	1:36:00		CLOSED				21	71	74	55	60	50	70	105	70	105	64	52	61	72	0
	1:36:00	OFF																			
	1:56:00	ON	OPEN		266,000,160																
7	1:57:00		OPEN	60			17.5	64	64	53	49	32	51	96	52	96	46	44	64	36	260
8	1:58:00		OPEN	63			19.5	65	65	54	51	33	53	96	53	99	46	45	74	35	260
9	1:59:00		OPEN	69			20.5	69	70	54	55	36	56	103	56	103	52	45	76	36	260
	1:59:00	OFF	OPEN			266,002,410															
	2:06:00	ON	CLOSED		266,002,410																
10	2:06:00		CLOSED	57			16	63	65	54	51	40	56	96	56	97	54	52	65	64	0
11	2:07:00		CLOSED	70			21	72	74	54	60	48	66	105	66	105	60	52	75	66	0
	2:07:42	OFF	CLOSED			266,003,900															

EVENT	TIME	HYDRANT STATUS OPEN/CLOSE	DISCHARGE HGL (FT)	PRV #1 UPSTREAM HGL (FT)	PRV #2 UPSTREAM HGL (FT)	PRV #2 DOWNSTREAM HGL (FT)	PRV #3 UPSTREAM HGL (FT)	PRV #3 DOWNSTREAM HGL (FT)	BOOSTER STATION PUMP #1 SUC. HGL (FT)	BOOSTER STATION PUMP #1 DISC. HGL (FT)	BOOSTER STATION PUMP #2 SUC. HGL (FT)	BOOSTER STATION PUMP #2 DISC. HGL (FT)	WELL #2 HGL (FT)	BUFFALO HGL (FT)	PRINCEWOOD HGL (FT)	RESIDUAL HYDRANT HGL (FT)
		ELEVATIONS	81.93	137.59	130.32	130.32	153.92	153.92	72.45	72.45	72.45	72.45	65.94	101.59	76.87	83.84
1																
2																
3	1:21	CLOSED		285.282	273.397	254.935	278.228	246.228	201.681	288.604	206.296	296.296	215.171	221.590	224.562	231.532
4	1:27	CLOSED		306.052	303.397	254.935	294.689	269.305	229.373	314.758	236.604	314.758	233.632	226.205	263.793	249.994
5	1:33	CLOSED		285.282	284.935	252.628	273.920	246.228	206.296	296.296	206.296	296.296	215.171	226.205	226.870	231.532
6	1:36	CLOSED		301.436	301.089	257.243	292.382	269.305	233.988	314.758	233.988	314.758	233.632	221.590	263.793	249.994
7	1:57	OPEN	220.392	285.282	278.012	252.628	266.997	227.766	190.142	298.604	192.450	298.604	192.094	203.128	224.562	166.917
8	1:58	OPEN	227.315	287.590	280.320	254.935	271.612	230.074	194.758	298.604	194.758	300.912	196.709	205.436	247.639	164.609
9	1:59	OPEN	241.161	296.821	291.858	254.935	280.843	241.612	201.681	310.142	206.296	310.142	205.940	205.436	252.255	171.532
10	2:06	CLOSED	213.468	282.975	280.320	254.935	271.612	246.228	201.681	293.988	206.296	296.296	210.555	221.590	226.870	231.532
11	2:07	CLOSED	243.468	303.744	301.089	254.935	292.382	264.689	224.758	314.758	224.758	314.758	224.402	221.590	249.947	240.763

NOTE: ONLY USE EVENT 7 THROUGH EVENT 11.

EXHIBIT CACE-1
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TEST #2

EVENT	TIME	PUMP STATUS	HYDRANT STATUS (OPEN/CLOSE)	DISCHARGE PRESSURE (PSIG)	"PUMP ON" TOTALIZER READING (GAL)	"PUMP OFF" TOTALIZER READING (GAL)	HYDROTANK LEVEL (INCHES)	PRV #1 UPSTREAM PRESSURE (PSIG)	PRV #2 UPSTREAM PRESSURE (PSIG)	PRV #2 DOWNSTREAM PRESSURE (PSIG)	PRV #3 UPSTREAM PRESSURE (PSIG)	PRV #3 DOWNSTREAM PRESSURE (PSIG)	BOOSTER STATION PUMP #1 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #1 DISC. PRES. (PSIG)	BOOSTER STATION PUMP #2 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #2 DISC. PRES. (PSIG)	WELL #2 (PSIG)	BUFFALO (PSIG)	PRINCEWOOD (PSIG)	RESIDUAL HYDRANT (PSIG)	STRESSED HYDRANT MICROMETER (GPM)
	2:32:30	ON			266,007,100																
1	2:33:21		CLOSED	62			18	64	67	54	52	40	58	97	56	97	54	52	69	68	0
2	2:34:13		CLOSED	70			20.75	68	70	53	56	44	62	100	62	101	58	52	75	68	0
	2:34:22	OFF				266,008,770															
	2:36:30	ON			266,008,770																
3	2:38:44		OPEN	61			18	53	67	53	43	32	47	91	48	91	48	44	69	56	300
4	2:39:28		OPEN	66			19.5	55	67	53	44	34	50	94	50	93	49	44	71	56	320
5	2:40:05		OPEN	70			21	57	62	53	47	38	52	95	52	96	52	44	77	58	320
	2:40:40	OFF				266,011,150															
	2:45:40	ON			266,011,150																
6	2:46:00		CLOSED	63			18.5	66	69	53	53	42	60	96	60	96	54	50	67	68	0
7	2:47:00		CLOSED	72			22	73	76	53	62	51	71	105	70	106	64	52	77	74	0
	2:47:13	OFF				266,012,550															

EVENT	TIME	HYDRANT STATUS OPEN/CLOSE	DISCHARGE HGL (FT)	PRV #1 UPSTREAM HGL (FT)	PRV #2 UPSTREAM HGL (FT)	PRV #2 DOWNSTREAM HGL (FT)	PRV #3 UPSTREAM HGL (FT)	PRV #3 DOWNSTREAM HGL (FT)	BOOSTER STATION PUMP #1 SUC. HGL (FT)	BOOSTER STATION PUMP #1 DISC. HGL (FT)	BOOSTER STATION PUMP #2 SUC. HGL (FT)	BOOSTER STATION PUMP #2 DISC. HGL (FT)	WELL #2 HGL (FT)	BUFFALO HGL (FT)	PRINCEWOOD HGL (FT)	RESIDUAL HYDRANT HGL (FT)
		ELEVATIONS	81.93	137.59	130.32	130.32	153.92	153.92	72.45	72.45	72.45	72.45	85.94	101.59	78.87	139.4
1	2:33:21	CLOSED	225.007	285.282	284.935	254.935	273.920	246.228	206.296	296.296	206.296	296.296	210.555	221.590	233.793	291.706
2	2:34:13	CLOSED	243.468	294.513	291.858	252.626	283.151	255.456	215.527	303.219	215.527	305.527	219.786	221.590	249.947	296.323
3	2:38:44	OPEN	222.899	259.898	284.935	252.628	253.151	227.766	180.912	282.450	183.219	282.450	192.094	203.128	236.101	268.631
4	2:39:28	OPEN	234.238	264.513	284.935	252.628	255.458	232.382	187.835	289.373	187.835	287.065	199.017	203.128	240.718	268.631
5	2:40:05	OPEN	243.468	269.128	273.397	252.628	262.382	241.612	192.450	291.681	192.450	293.988	205.940	203.128	254.562	273.246
6	2:46:00	CLOSED	227.315	289.898	289.551	252.628	276.228	250.643	210.912	298.604	210.912	298.604	210.555	218.875	231.485	296.323
7	2:47:00	CLOSED	248.084	306.052	305.705	252.626	296.997	271.612	236.296	314.758	233.988	317.065	233.632	221.590	254.562	310.169

EXHIBIT (ACE-1)
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TEST #3

EVENT	TIME	PUMP STATUS	HYDRANT STATUS OPEN/CLOSE	DISCHARGE PRESSURE (PSIG)	"PUMP ON" TOTALIZER READING (GAL)	"PUMP OFF" TOTALIZER READING (GAL)	HYDROTANK LEVEL (INCHES)	PRV #1 UPSTREAM PRESSURE (PSIG)	PRV #2 UPSTREAM PRESSURE (PSIG)	PRV #2 DOWNSTREAM PRESSURE (PSIG)	PRV #3 UPSTREAM PRESSURE (PSIG)	PRV #3 DOWNSTREAM PRESSURE (PSIG)	BOOSTER STATION PUMP #1 SUC. PRES (PSIG)	BOOSTER STATION PUMP #1 DISC. PRES. (PSIG)	BOOSTER STATION PUMP #2 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #2 DISC. PRES. (PSIG)	WELL #2 (PSIG)	BUFFALO (PSIG)	PRINCEWOOD (PSIG)	RESIDUAL HYDRANT (PSIG)	STRESSED HYDRANT MICROMETER (GPM)
	3:44:05	ON			266,020,170																
1	3:44:34		CLOSED	61			18.25	64	67	54	52	40	59	97	59	97	56	55	64	85	0
2	3:45:20		CLOSED	69		266,021,630	21	71	72	54	58	46	68	103	67	103	66	55	76	90	0
	3:45:43	OFF																			
	3:49:22	ON			266,021,630																
3	3:49:57		OPEN	61			18	65	67	53	52	37	50	98	51	99	46	44	65	75	310
4	3:50:33		OPEN	64			19	68	70	53	54	38	52	101	53	101	48	44	71	75	300
5	3:51:51		OPEN	71.5			21.75	70	71	53	56	42	60	102	60	102	55	44	76	79	340
	3:52:10	OFF				266,024,160															
	3:56:00	ON			266,024,160																
6	3:56:22		CLOSED	62			18.5	67	70	53	55	42	59	100	59	100	54	51	65	85	0
7	3:57:21		CLOSED	70			21.25	72	72	54	60	48	68	104	68	105	62	52	74	90	0
	3:57:47	OFF				266,025,740															

EVENT	TIME	HYDRANT STATUS OPEN/CLOSE	DISCHARGE HGL (FT)	PRV #1 UPSTREAM HGL (FT)	PRV #2 UPSTREAM HGL (FT)	PRV #2 DOWNSTREAM HGL (FT)	PRV #3 UPSTREAM HGL (FT)	PRV #3 DOWNSTREAM HGL (FT)	BOOSTER STATION PUMP #1 SUC. HGL (FT)	BOOSTER STATION PUMP #1 DISC. HGL (FT)	BOOSTER STATION PUMP #2 SUC. HGL (FT)	BOOSTER STATION PUMP #2 DISC. HGL (FT)	WELL #2 HGL (FT)	BUFFALO HGL (FT)	PRINCEWOOD HGL (FT)	RESIDUAL HYDRANT HGL (FT)
		ELEVATIONS	81.93	137.59	130.32	130.32	153.92	153.92	72.45	72.45	72.45	72.45	85.94	101.59	76.87	25.01
1	3:44:34	CLOSED	222.699	285.282	284.935	254.935	273.920	246.228	206.604	296.296	206.604	296.296	215.171	228.513	224.562	221.164
2	3:45:20	CLOSED	241.161	301.436	296.474	254.935	287.766	260.074	229.373	310.142	227.065	310.142	238.248	228.513	252.255	232.702
3	3:49:57	OPEN	222.699	287.590	284.935	252.628	273.920	239.305	187.835	298.604	190.142	300.912	192.094	203.128	226.870	198.087
4	3:50:33	OPEN	229.622	294.513	291.858	252.628	278.535	241.612	192.450	305.527	194.758	305.527	196.709	203.128	240.716	198.087
5	3:51:51	OPEN	246.930	299.128	294.166	252.628	283.151	250.843	210.912	307.835	210.912	307.835	212.863	203.128	252.255	207.318
6	3:56:22	CLOSED	225.007	292.205	291.858	252.628	280.843	250.843	206.604	303.219	206.604	303.219	210.556	219.282	226.870	221.164
7	3:57:21	CLOSED	243.468	303.744	296.474	254.935	292.382	264.689	229.373	312.450	229.373	314.758	229.017	221.590	247.639	232.702

EXHIBIT (RCE-1)

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TEST #4

EVENT	TIME	PUMP STATUS	HYDRANT STATUS OPEN/CLOSE	DISCHARGE PRESSURE (PSIG)	"PUMP ON" TOTALIZER READING (GAL)	"PUMP OFF" TOTALIZER READING (GAL)	HYDROTANK LEVEL (INCHES)	PRV #1 UPSTREAM PRESSURE (PSIG)	PRV #2 UPSTREAM PRESSURE (PSIG)	PRV #2 DOWNSTREAM PRESSURE (PSIG)	PRV #3 UPSTREAM PRESSURE (PSIG)	PRV #3 DOWNSTREAM PRESSURE (PSIG)	BOOSTER STATION PUMP #1 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #1 DISC. PRES. (PSIG)	BOOSTER STATION PUMP #2 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #2 DISC. PRES. (PSIG)	WELL #2 (PSIG)	BUFFALO (PSIG)	PRINCEWOOD (PSIG)	STRESSED HYDRANT MICROMETER (GPM)
	4:24:47	ON			266,031,430															
1	4:25:15		CLOSED	61			16	64	67	58	51	39	56	97	58	97	52	54	64	0
2	4:26:12		CLOSED	69			21	70	72	58	57	46	66	104	65	103	59	54	74	0
	4:26:45	OFF				266,033,170														
	4:29:30	ON			266,033,170															
3	4:30:07		OPEN	60			18	62	64	63	50	36	52	96	52	95	46	46	60	270
4	4:31:05		OPEN	65			19.5	66	68	63	53	38	53	100	53	100	48	44	69	310
5	4:32:12		OPEN	70			20.25	69	72	63	56	41	56	102	58	102	51	44	75	320
	4:33:00	OFF				266,036,200														
	4:36:20	ON			266,036,200															
6	4:37:10		CLOSED	62.5			19	68	70	53	55	42	60	101	60	101	52	52	69	0
7	4:38:10		CLOSED	70.5			21.75	71	74	53	59	48	66	104	68	104	62	54	75	0
	4:38:20	OFF				266,037,970														

EVENT	TIME	HYDRANT STATUS OPEN/CLOSE	DISCHARGE HGL (FT)	PRV #1 UPSTREAM HGL (FT)	PRV #2 UPSTREAM HGL (FT)	PRV #2 DOWNSTREAM HGL (FT)	PRV #3 UPSTREAM HGL (FT)	PRV #3 DOWNSTREAM HGL (FT)	BOOSTER STATION PUMP #1 SUC. HGL (FT)	BOOSTER STATION PUMP #1 DISC. HGL (FT)	BOOSTER STATION PUMP #2 SUC. HGL (FT)	BOOSTER STATION PUMP #2 DISC. HGL (FT)	WELL #2 HGL (FT)	BUFFALO HGL (FT)	PRINCEWOOD HGL (FT)
		ELEVATIONS	61.93	137.59	130.32	130.32	153.92	153.92	72.45	72.45	72.45	72.45	85.94	101.59	78.87
1	4:25:15	CLOSED	222.699	285.282	284.936	264.166	271.612	243.920	206.296	296.296	206.296	286.296	205.940	226.205	224.562
2	4:26:12	CLOSED	241.161	299.128	296.474	264.166	285.458	260.074	224.758	312.450	222.450	310.142	222.094	226.205	247.639
3	4:30:07	OPEN	220.362	280.667	278.012	275.705	269.305	236.997	192.450	293.988	192.450	291.681	192.094	207.744	215.332
4	4:31:05	OPEN	231.930	289.898	287.243	275.705	276.228	241.612	194.758	303.219	194.758	303.219	196.709	203.128	236.101
5	4:32:12	OPEN	243.468	296.821	296.474	275.705	283.151	248.535	206.296	307.835	206.296	307.835	203.632	203.128	249.947
6	4:37:10	CLOSED	226.161	294.513	291.858	252.628	280.843	250.843	210.912	305.527	210.912	305.527	205.940	221.590	236.101
7	4:38:10	CLOSED	244.622	301.436	301.089	252.628	290.074	264.689	229.373	312.450	229.373	312.450	229.017	226.205	249.947

EXHIBIT (CCE-1)
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TEST #5

EVENT	TIME	PUMP STATUS	HYDRANT STATUS (OPEN/CLOSE)	DISCHARGE PRESSURE (PSIG)	"PUMP ON" TOTALIZER READING (GAL)	"PUMP OFF" TOTALIZER READING (GAL)	HYDROTANK LEVEL (INCHES)	PRV #1 UPSTREAM PRESSURE (PSIG)	PRV #2 UPSTREAM PRESSURE (PSIG)	PRV #2 DOWNSTREAM PRESSURE (PSIG)	PRV #3 UPSTREAM PRESSURE (PSIG)	PRV #3 DOWNSTREAM PRESSURE (PSIG)	BOOSTER STATION PUMP #1 SUC. PRES. (PSIG)	BOOSTER STATION PUMP #1 DISC. PRES. (PSIG)	BOOSTER STATION PUMP #2 SUC. PRES (PSIG)	BOOSTER STATION PUMP #2 DISC. PRES (PSIG)	WELL #2 (PSIG)	BUFFALO (PSIG)	PRINCEWOOD (PSIG)	STRESSED HYDRANT MICROMETER (GPM)
	5:25:06	ON	CLOSED		266,049,710															
1	5:26:00		CLOSED	64			19.5	66	69	53	54	40	58	98	58	97	53	54	73	0
2	5:27:00		CLOSED	69			21	67	69	53	55	42	60	100	60	100	58	54	73	0
	5:27:30	OFF	CLOSED			266,051,790														
	MISSED CYCLE																			
	5:38:38	ON	OPEN		266,056,670															
3	5:39:15		OPEN	61			18.75	64	70	51	53	40	58	98	58	98	52	54	54	410
4	5:40:20		OPEN	64			19.75	66	70	51	54	40	59	100	59	99	53	54	58	420
5	5:42:30		OPEN	66			20.5	65	70	53	54	44	64	98	63	99	58	54	61	400
	5:44:22	OFF	CLOSED			266,063,690														
	5:48:50	ON	CLOSED		266,063,890															
6	5:49:30		CLOSED	62			19	64	64	53	52	40	58	98	58	98	53	54	67	0
7	5:50:15		CLOSED	68			21.25	72	69	51	55	42	60	101	60	101	58	54	73	0
	5:51:06	OFF	CLOSED			266,065,900														

EVENT	TIME	HYDRANT STATUS OPEN/CLOSE	DISCHARGE HGL (FT)	PRV #1 UPSTREAM HGL (FT)	PRV #2 UPSTREAM HGL (FT)	PRV #2 DOWNSTREAM HGL (FT)	PRV #3 UPSTREAM HGL (FT)	PRV #3 DOWNSTREAM HGL (FT)	BOOSTER STATION PUMP #1 SUC. HGL (FT)	BOOSTER STATION PUMP #1 DISC. HGL (FT)	BOOSTER STATION PUMP #2 SUC. HGL (FT)	BOOSTER STATION PUMP #2 DISC. HGL (FT)	WELL #2 HGL (FT)	BUFFALO HGL (FT)	PRINCEWOOD HGL (FT)
		ELEVATIONS	81.83	137.59	130.32	130.32	153.92	153.92	72.45	72.45	72.45	72.45	85.94	101.59	76.87
1	5:25:15	CLOSED	229.622	289.896	289.551	252.628	278.535	246.228	206.296	296.604	206.296	296.296	208.248	226.205	245.332
2	5:26:12	CLOSED	241.161	292.205	289.551	252.628	280.843	250.843	210.912	303.219	210.912	303.219	215.171	226.205	245.332
3	5:30:07	OPEN	222.699	285.282	291.858	248.012	276.228	246.228	206.296	296.604	206.296	296.604	205.940	226.205	201.485
4	5:31:05	OPEN	229.622	289.896	291.858	248.012	278.535	246.228	206.604	303.219	206.604	300.912	208.248	226.205	210.716
5	5:32:12	OPEN	234.238	287.590	291.858	252.628	278.535	256.458	220.142	296.604	217.835	300.912	222.094	226.205	217.639
6	5:37:10	CLOSED	225.007	285.282	278.012	252.628	273.920	246.228	206.296	296.604	206.296	296.604	208.248	226.205	231.485
7	5:38:10	CLOSED	238.853	303.744	289.551	248.012	280.843	250.843	210.912	305.527	210.912	305.527	215.171	226.205	245.332

EXHIBIT ____ (RCE-1)
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ATTACHMENT 2

RESULTS OF MODEL CALIBRATION USING TEST #1, EVENT 9 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1402.25 rpm (it is operating at 79% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Residual (West Deputy Drive)	Western	38	47.33	-9.33
North Princewood Drive	Eastern	76	70.43	5.57
North Buffalo Drive	Western	45	55.66	-10.66
Well #2	Western	52	62.48	-10.48
PRV #1 (upstream)		69	68.714	0.286
PRV #2 (upstream)		70	69.667	0.333
PRV #2 (downstream)		54	54.002	-0.002
PRV #3 (upstream)		55	58.015	-3.015
PRV #3 (downstream)		38	37.301	0.699
Booster Station (suction side)	Western	56	67.13	-11.13
Booster Station (discharge side)		103	97.86	5.14

Hydrant Flow = 280 GPM

System Demand = 180 GPM

RESULTS OF MODEL CALIBRATION USING TEST #1, EVENT 11 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1349 rpm (it is operating at 76% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Residual (West Deputy Drive)	Western	68	72.39	-4.39
North Princewood Drive	Eastern	75	72.11	2.89
North Buffalo Drive	Western	52	61.24	-9.24
Well #2	Western	60	67.99	-7.99
PRV #1 (upstream)		72	71.933	0.067
PRV #2 (upstream)		74	72.887	1.113
PRV #2 (downstream)		54	54.002	-0.002
PRV #3 (upstream)		60	61.239	-1.239
PRV #3 (downstream)		48	47.307	0.693
Booster Station (suction side)	Western	66	72.6	-6.6
Booster Station (discharge side)		105	101.01	3.99

Hydrant Flow = 0 GPM

System Demand = 180 GPM

RESULTS OF MODEL CALIBRATION USING TEST #3, EVENT 2 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1331.25 rpm (it is operating at 75% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Residual (West Cavalry Lane)	Western	90	93.82	-3.82
North Princewood Drive	Eastern	76	71.05	4.95
North Buffalo Drive	Western	55	60.28	-5.28
Well #2	Western	66	67.03	-1.03
PRV #1 (upstream)		71	70.581	0.419
PRV #2 (upstream)		72	71.786	0.214
PRV #2 (downstream)		54	54.002	-0.002
PRV #3 (upstream)		58	60.303	-2.303
PRV #3 (downstream)		46	45.383	0.617
Booster Station (suction side)	Western	68	71.78	-3.78
Booster Station (discharge side)		103	99.56	3.44

Hydrant Flow = 0 GPM

System Demand = 155 GPM

RESULTS OF MODEL CALIBRATION USING TEST #3, EVENT 5 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1384.5 rpm (it is operating at 78% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Residual (West Cavalry Lane)	Western	79	87.97	-8.97
North Princewood Drive	Eastern	76	72.87	3.13
North Buffalo Drive	Western	44	57.03	-13.03
Well #2	Western	55	63.89	-8.89
PRV #1 (upstream)		70	69.97	0.03
PRV #2 (upstream)		71	71.201	-0.201
PRV #2 (downstream)		53	53.001	-0.001
PRV #3 (upstream)		56	59.735	-3.735
PRV #3 (downstream)		42	41.392	0.608
Booster Station (suction side)	Western	60	68.8	-8.8
Booster Station (discharge side)		102	98.94	3.06

Hydrant Flow = 340 GPM

System Demand = 155 GPM

RESULTS OF MODEL CALIBRATION USING TEST #4, EVENT 2 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1349 rpm (it is operating at 76% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
North Princewood Drive	Eastern	74	71.03	2.97
Residual (North Buffalo Drive)	Western	54	59.93	-5.93
Well #2	Western	59	67.1	-8.1
PRV #1 (upstream)		70	70.69	-0.69
PRV #2 (upstream)		72	71.73	0.27
PRV #2 (downstream)		58	58.002	-0.002
PRV #3 (upstream)		57	60.16	-3.16
PRV #3 (downstream)		46	45.34	0.66
Booster Station (suction side)	Western	66	71.32	-5.32
Booster Station (discharge side)		104	99.75	4.25

Hydrant Flow = 0 GPM

System Demand = 255 GPM

RESULTS OF MODEL CALIBRATION USING TEST #4, EVENT 5 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1437.75 rpm (it is operating at 81% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
North Princewood Drive	Eastern	75	70.99	4.01
Residual (North Buffalo Drive)	Western	44	52.08	-8.08
Well #2	Western	51	61.98	-10.98
PRV #1 (upstream)		69	69.65	-0.65
PRV #2 (upstream)		72	70.633	1.367
PRV #2 (downstream)		63	62.998	0.002
PRV #3 (upstream)		56	59.024	-3.024
PRV #3 (downstream)		41	40.326	0.674
Booster Station (suction side)	Western	58	66.32	-8.32
Booster Station (discharge side)		102	98.73	3.27

Hydrant Flow = 320 GPM

System Demand = 255 GPM

RESULTS OF MODEL CALIBRATION USING TEST #5, EVENT 5 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1775 rpm (it is operating at full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Residual (North Princewood Drive)	Eastern	61	56.69	4.31
North Buffalo Drive	Western	54	51.36	2.64
Well #2	Western	59	58.01	0.99
PRV #1 (upstream)		65	61.516	3.484
PRV #2 (upstream)		70	62.829	7.171
PRV #2 (downstream)		53	53.001	-0.001
PRV #3 (upstream)		54	50.28	3.72
PRV #3 (downstream)		44	42.865	1.135
Booster Station (suction side)	Western	64	56.48	7.52
Booster Station (discharge side)		98	96.97	1.03

Hydrant Flow = 400 GPM

System Demand = 279 GPM

RESULTS OF MODEL CALIBRATION USING TEST #5, EVENT 7 (1/16/96)

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Booster Pump Speed = 1455.5 rpm (it is operating at 82% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Residual (North Princewood Drive)	Eastern	73	69.74	3.26
North Buffalo Drive	Western	54	57.75	-3.75
Well #2	Western	56	64.52	-8.52
PRV #1 (upstream)		67	70.681	-3.681
PRV #2 (upstream)		69	72.332	-3.332
PRV #2 (downstream)		53	50.999	2.001
PRV #3 (upstream)		55	61.026	-6.026
PRV #3 (downstream)		42	41.47	0.53
Booster Station (suction side)	Western	60	68.29	-8.29
Booster Station (discharge side)		100	100.62	-0.62

Hydrant Flow = 0 GPM

System Demand = 279 GPM

ATTACHMENT 3

TEST #3 (2/28/96) - [Before Air Purging]

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Assumed Booster Pump Speed = 1384.5 rpm (it is operating at 78% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Residual (West Cavalry Lane)	Western	73	88.27	-15.27
North Buffalo Drive	Western	44	57.34	-13.34
Well #2	Western	51	64.2	-13.2

Stressed Hydrant @ West Pine Ridge Boulevard & West Cavalry Lane @ 340 GPM.

System Demand Without Fire Flow = 139 GPM

Total Demand = 479 GPM

EXHIBIT (RCE-1)

OBSTRUCTION TEST (2/28/96) - [Before Air Purging]

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Assumed Booster Pump Speed = 1384.5 rpm (it is operating at 78% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Carnation Hydrant	Eastern	56	56.85	-0.85
Perry Hydrant	Western	39	51.48	-12.48

Stressed Hydrant @ West Pine Ridge Boulevard & West Cavalry Lane @ 350 GPM.

System Demand Without Fire Flow = 278 GPM

Total Demand = 628 GPM

OBSTRUCTION TEST (2/29/96) - [Following 2nd Air Purging]

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Hazen-Williams C Factor = 145

Assumed Booster Pump Speed = 1384.5 rpm (it is operating at 78% of full speed)

Location	Sub-System Monitored	Field Pressure (psi)	Model Pressure (psi)	Difference (psi)
Carnation Hydrant	Eastern	60	58.07	1.93
Perry Hydrant	Western	48	53.27	-5.27
Well #2	Western	56	64.18	-8.18

Stressed Hydrant @ West Pine Ridge Boulevard & West Cavalry Lane @ 350 GPM.

System Demand Without Fire Flow = 138 GPM

Total Demand = 488 GPM

ATTACHMENT 4

EXHIBIT (RCE-1)

PAGE 38 OF 48

Air Binding in Pipes

Robert C. Edmunds

A survey of current research and recent case histories on the phenomenon of air binding suggests that while there is no generally agreed-upon solution to this problem, the adoption of some simple procedures can minimize its occurrence.

Air trapped in pipes can reduce pipeline carrying capacity, cause unexpected pressure surges, and produce objectionable "white water." This article summarizes state-of-the-art research and background data on the air binding phenomenon, compares case histories with theories developed to predict the occurrence of air binding, and describes a procedure that will assist pipeline designers in preventing air binding.

The Phenomenon

Two typical cases of air binding in pipelines demonstrate how this phenomenon occurs (Fig. 1). As flow begins in a pipe with mild slope, the normal depth—i.e., the depth associated with uniform flow—is greater than the critical depth for that flow and no hydraulic jump occurs. If the volume of the stagnant air pocket is not sufficient to fill the descending leg and if additional air reaches this zone in the pipeline, the air bubble grows in a downstream direction and maintains the same height at all points because of the fluid's uniform depth. The trapped air can be removed hydraulically either by generation of small air bubbles at the turbulent downstream end of the pocket, and entrainment into and transport by the fluid, or by sweeping the total air pocket down the pipeline. If an air pocket with low or no air velocity is assumed, the air pressure in the pocket must be everywhere the same. Calculating the general energy equation between the two sections of pipe (Fig. 1) will show that the head loss due to the trapped air pocket is equal to the vertical component of the length of the air pocket. Since in uniform flow the water surface is parallel to the channel invert, the energy loss is equal to the difference in invert elevation between the high and low points in the descending leg, assuming that the air pocket extends to the bottom of the slope. This point can be useful in locating unexplained head losses in pipelines by comparing the amount of unexplained head loss to the elevation differences in the pipeline profile.

In a pipe with steep slope (Fig. 2) the normal depth is less than the critical depth, and hydraulic jump is possible. (At mild slopes, special upstream control sections such as a partially opened gate or a rapid change in slope can also cause hydraulic jump to form.) The jump is the interface between upstream supercritical and downstream subcritical backwater curves or between upstream supercritical normal depth and the downstream subcritical backwater curve. If the hydraulic jump seals the line, air is pumped into the water downstream of the jump. At low flow the air hydraulically removed is a function of the flow conditions downstream of the jump. At some finite flow the entrained air is not carried downstream at all, but occasionally blows out through the jump, causing the jump to move temporarily downstream. At high flow the air, once entrained, is easily carried below the jump and the amount of air removed is a function of the hydraulic jump's ability to entrain air from the upstream pocket. As before, the entrapped air pocket can be hydraulically removed either by generation and entrainment of bubbles or by sweeping the air pocket down the pipeline.

To better demonstrate the hydraulic conditions within a closed pipeline

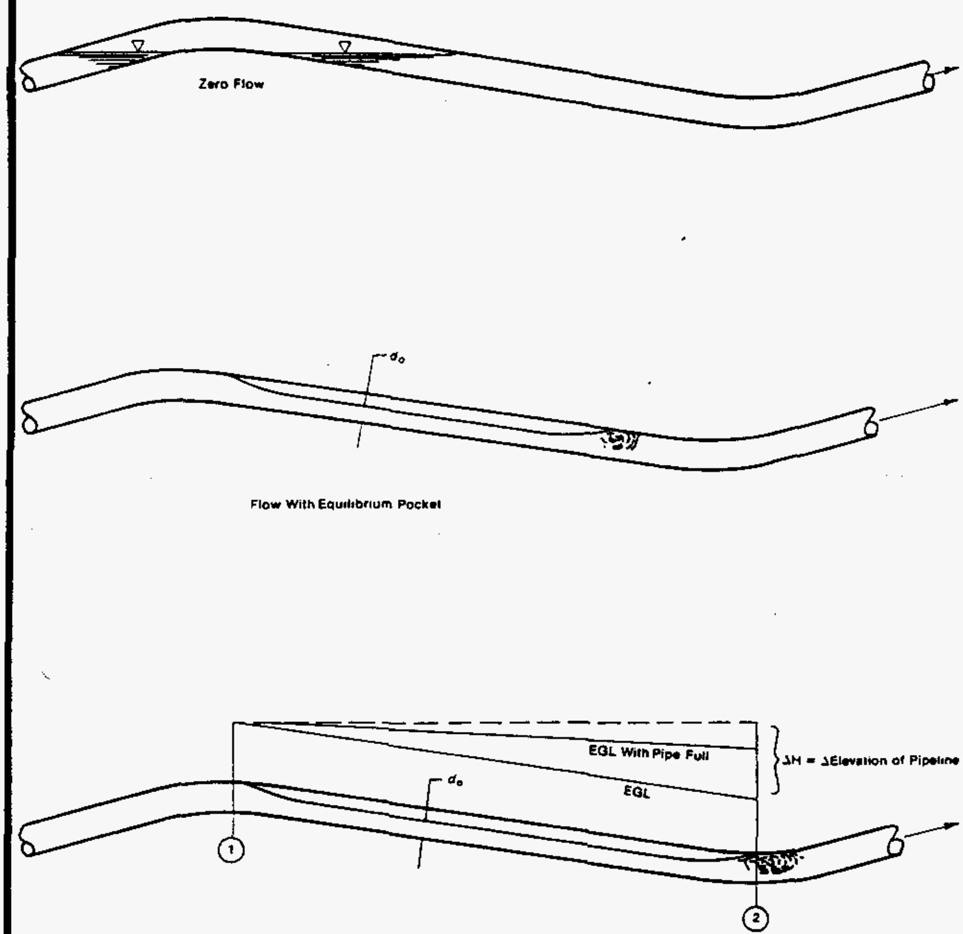

Fig. 1. Air Pocket in Pipe With Mild Slope

***Normal Depth* (d_o) > *Critical Depth* (d_{cr})**

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containing air, Kennison[1] developed a useful diagram that illustrates the following relationships (Fig. 3):

1. The critical discharge as a function of the depth of flow; that is, the depth at which the Froude number equals 1.0 (giving unstable water surfaces). The critical depth of flow can also be found if the discharge is given.
2. The normal discharge for any depth and slope. Introduction of additional air increases the bubble length, not the depth. (This relationship is plotted by assuming a C of 100 in the Chezy equation. It is useful because the units are the same as those for the critical discharge, thus permitting an immediate comparison of normal depth vs critical depth.)
3. Given the slope and the depth, the minimum flow required for the hydraulic jump to just fill the pipe and thus possibly pump air downstream. This was plotted using data developed by Kalinske and Robertson.[2]
4. Assuming uniform flow, the limit of the ability of the hydraulic jump to fill the pipe. These curves result from the intercepts of the curves for uniform flow and the curves giving the discharge necessary for the jump to fill the pipe.
5. The value of the Froude number for uniform flow at any depth and slope. If this number is greater than or equal to 1.0, hydraulic jump is possible.

Summary of Research on Air Removal by Hydraulic Means

Air pockets can be removed hydraulically by bubble generation and entrainment or by sweeping the pockets from the line. Should hydraulic jump occur within the line, the air removal capacity may be limited by hydraulic conditions downstream of the jump at low flows and by the air entraining limitations of the hydraulic jump at high flows. Kalinske and Robertson[2] correlated the air removal capacity resulting only from the air entraining limitations of the hydraulic jump and developed the relationship

$$Q_a = Q_w \cdot 0.0066\,(F - 1)^{1.4} \qquad (1)$$

where Q_a is the air removal capacity, Q_w is the water discharge, and F is the Froude number of the approaching flow, defined as $V/\sqrt{gY_e}$ (where V is the approach velocity, g is the acceleration due to gravity and Y_e the effective depth—i.e., the water cross-sectional area upstream of the jump divided by the surface width). This equation was found to be valid for conditions in which the fluid carried away all of the air the jump entrained. For any value of approach depth divided by pipe diameter there was a critical Froude number below which the pipeline would carry only part of the air pumped into the water by the jump (Fig. 4). The family of curves in Fig. 4 defines the point at which the air

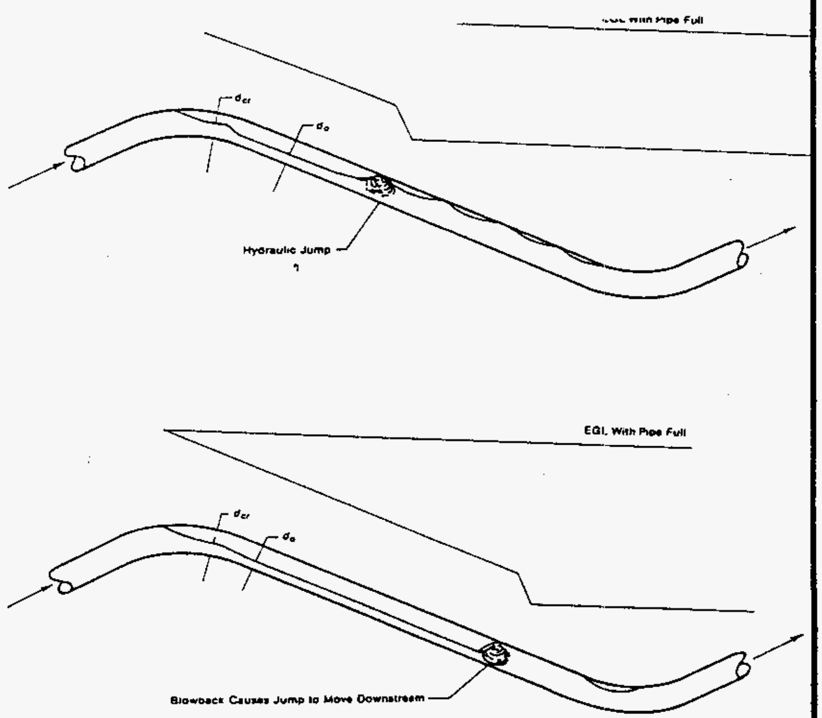

Fig. 2. Air Pocket in Pipe With Steep Slope

Normal Depth (d_o) < *Critical Depth* (d_{cr})

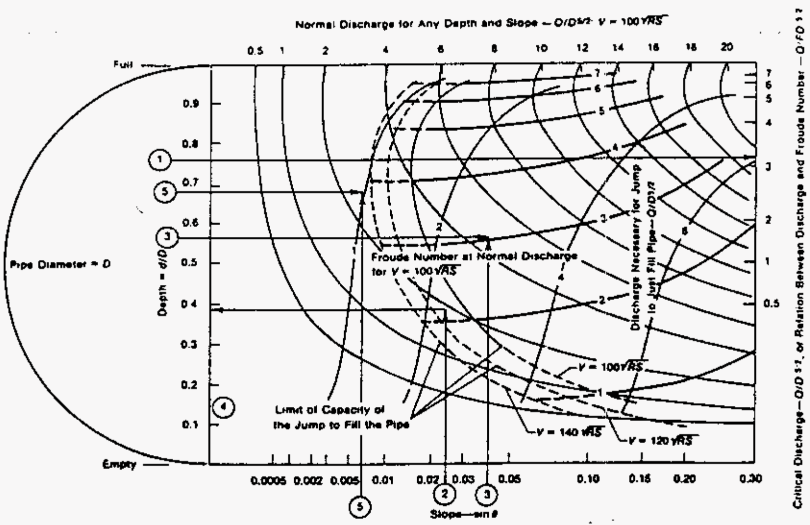

Fig. 3. Hydraulic Jump Inside a Pipe—F-P-S System (After Kennison[1])

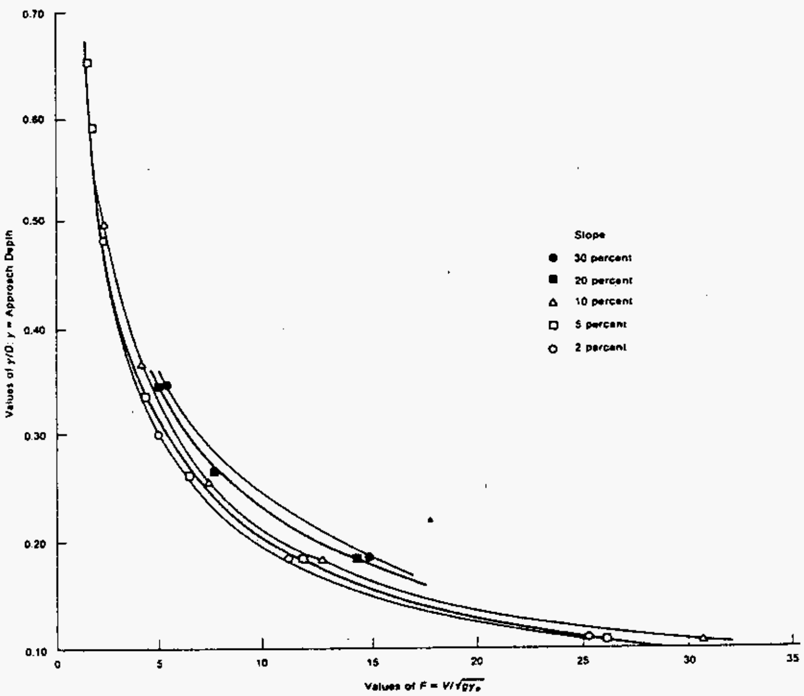

Fig. 4. Experimental Values of Critical Froude Numbers (After Kalinske & Robertson[2])

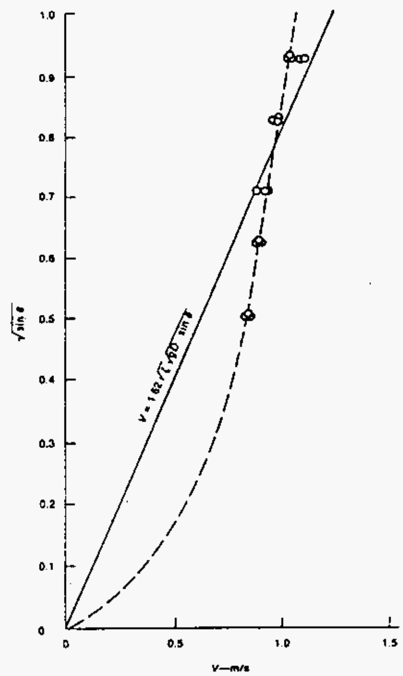

Fig. 6. Equilibrium Velocity vs Pipe Slope

4-in. Pipe; ζ ≐ 0.58
(After Kent[4])

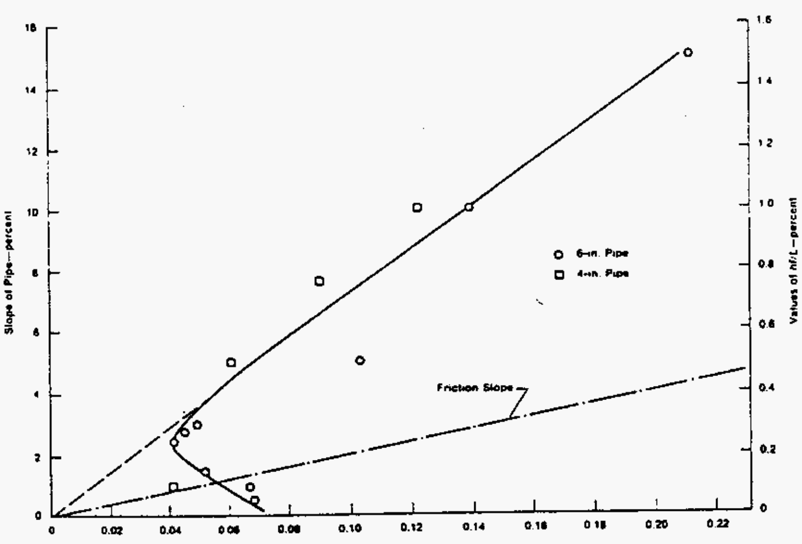

Fig. 5. Experimental Data Showing the Relation Between Pipe Slope, Pipe Diameter, Water Discharge, and Hydraulic Gradient When Air Removal Starts (After Kalinske & Bliss[3])

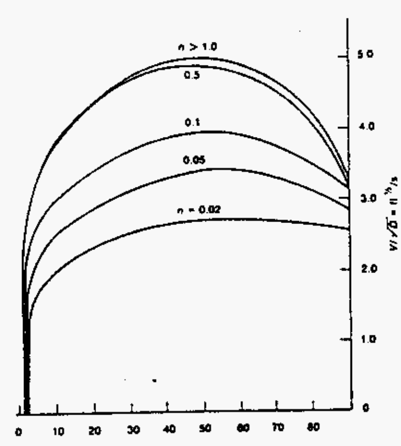

Fig. 7. Minimum Velocity to Move Air from High Point Pipe (After Gandenberger[5])

entraining capacity of the jump and air transporting capacity of the pipe downstream of the jump were found to be equal. These experiments were performed by inducing a hydraulic jump downstream of a partially open gate to easily manipulate the approach depth and effective depth. Experiments were performed in 4-in. and 6-in. acrylic pipes.

A number of researchers have examined the ability of the pipeline to transport discrete bubbles and pockets, where either no jump occurs or where the hydraulics downstream of the jump control air carrying capacity. Kalinske and Bliss[3] equated the theoretical drag and displacement forces on an air pocket in equilibrium and developed an expression relating the pipe slope and equilibrium flow, defined as the minimum discharge necessary to start air moving down the pipe downstream of the hydraulic jump (Fig. 5). The deviation in data at low slopes resulted from the hydraulic jump not completely sealing the line, thus requiring higher flows to entrain and transport the air. Also plotted is the friction slope of the full pipeline, indicating that air movement was obtained with energy grade line (EGL) slopes much milder than the pipe slopes. Experiments were performed in a 6-in. acrylic pipe.

Kent[4] also equated theoretical drag and displacement forces on an equilibrium air pocket. Experimental results were used to approximate the coefficient of drag, and the pocket equilibrium velocity was then correlated with pipeline slope as shown in Fig. 6. It was suggested that zeta (ζ), a shape factor, becomes constant for pockets whose length is greater than 1.5 times the pipe diameter. Kent also developed relationships for the loss-of-head vs percentage of air and pipe slope and the friction formula for flow with air pockets. Kent's experiments were performed in a 4-in. acrylic pipe.

Gandenberger[5] experimented on the movement of air bubbles and pockets from the peaks of 10.5-mm, 26-mm and 45-mm glass tubes and 100-mm steel pipe with slopes varying from zero to 90 degrees and water flowing upward and downward. Based on these experiments, a graph subsequently converted to English units by Mechler was developed that shows the minimum clearing velocity as a function of bubble volume (Fig. 7). The term n is defined as the bubble volume divided by $\pi D^3/4$ where D is pipe diameter. These relationships were considered to be valid for pipes with a diameter greater than 4 in. Both Kalinske and Gandenberger noted a tendency for bubbles to stop and adhere at irregularities in the pipeline.

Wisner et al[6] applied previous theories to several case histories and, noting

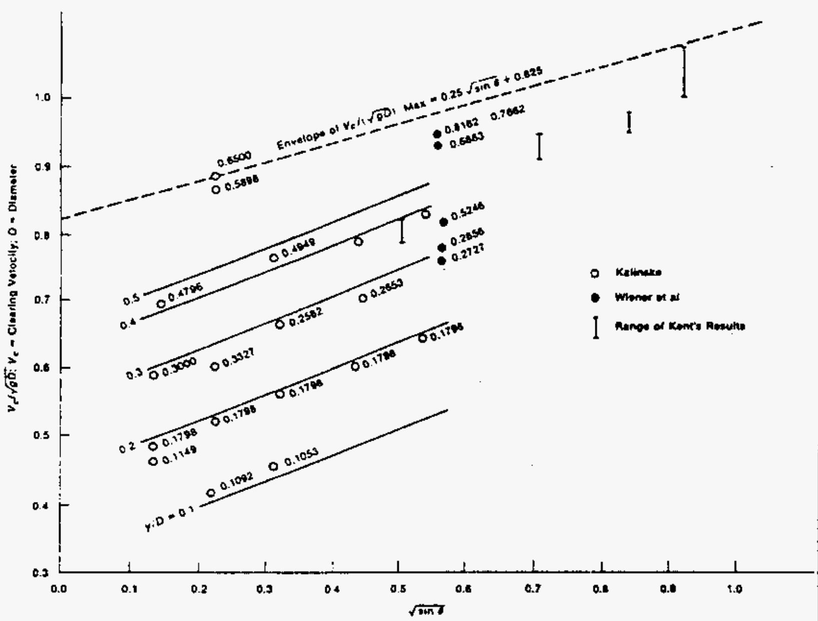

Fig. 8. Recommended Envelope Curve for Clearing of Aerated Pockets—F-P-S System

Figures Beside Each Point Indicate Corresponding y/D: y = Approach Depth
(After Wisner, Mohsen, & Kouwen[6])

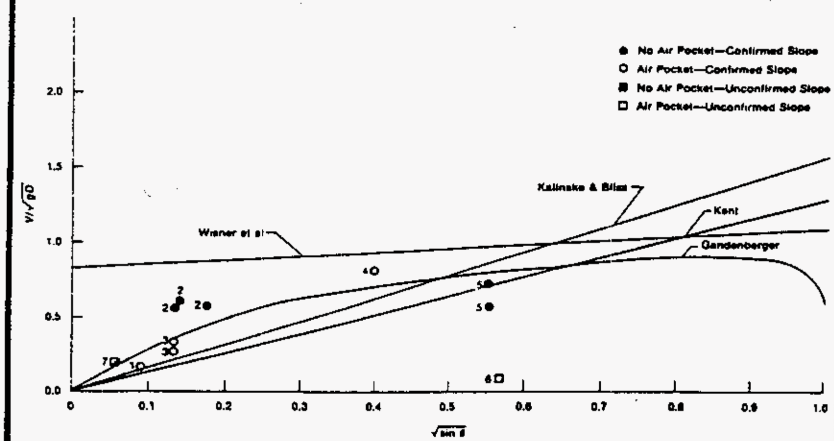

Fig. 9. Minimum Velocity vs Pipe Slope Recommended by Various Researchers—F-P-S System

serious air binding in one case, experimented with the rise velocity of bubbles in still water and with equilibrium pockets in a 10-in. diameter clear pipe at 18.5-deg slope. Adding their data to the data of Kalinske and Robertson[2] and Kent,[4] they recommended a lower bound clearing velocity (Fig. 8), defined as the minimum velocity necessary to clear a pocket out of the line—without specific reference to sweeping or generation and entrainment removal methods. These authors replotted data from the chart of Kalinske and Robertson (Fig. 4). Kalinske and Robertson's data defined the points at which the pipeline would carry only a part of the air pumped into the water by the jump but where some air transport was taking place; Kent's data defined the velocities required for air pocket equilibrium. This inconsistent definition of the data points could cause Wisner's envelope to predict conservatively high velocities at low slopes.

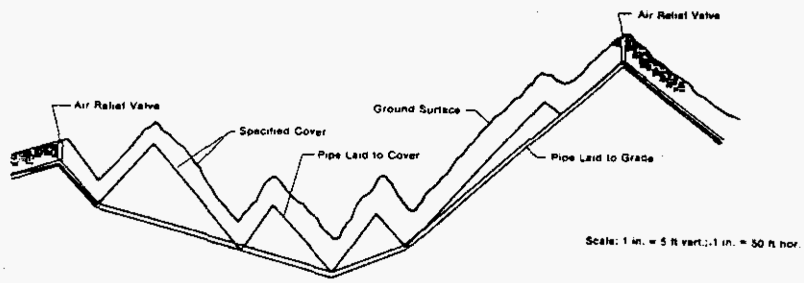

Fig. 10. Pipe Laid to Cover vs Pipe Laid to Grade

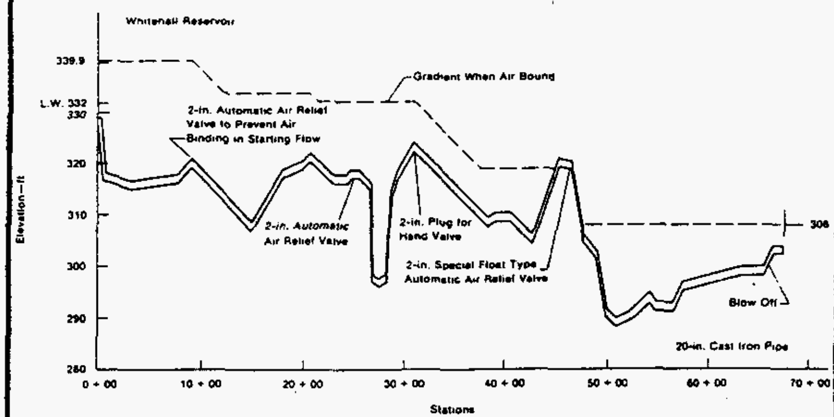

Fig. 11. Whitehall Pipeline Profile (After Kennison[1])

Correlation of Research and Field Data

If the recommendations of these researchers are reduced to consistent units and plotted to the same scale (Fig. 9), areas of agreement and divergence are evident. It should be noted that Kent[4] and Gandenberger[5] both defined velocities at which clearing was incipient but not necessarily in progress. Therefore air pockets could normally occur at and below velocities defined by their relationships. Divergences between these relationships may occur because of variations in the definition of terms, scale effects, or variation in the conditions adopted by each investigator.

Data taken from case histories of existing pipelines from both the literature and from the author's experience have also been plotted.

Case 1 is a 48-in. raw water collection line in south Florida fed by vertical turbine pumps which inject the air that bleeds into the pump discharge columns into the pipeline. The pipeline was erroneously suspected of air binding because of unexplained head loss in the line, which was actually caused by a partially closed valve. At the portion of the line that was investigated, the slope was 0.452 deg and the average flow 55.8 ML/day (14.4 mgd). An air pocket was found but was not large enough to produce serious loss of head.

The data points for Case 2 are reported by Kennison[1] and are taken from the 20-in. Whitehall and 24-in. Ashland lines in Massachusetts. No apparent air pockets were found.

Case 3 is reported by Richards[7] to be a 78-in. power plant discharge line flowing under partial vacuum. Air binding was found in the full length of the pipe slope; the existing vacuum priming system was insufficient to remove the air pocket.

Case 4 is reported by Richards[7] to be a 66-in. power plant condensor discharge line flowing under partial vacuum. The vacuum release tap was located upstream of the remaining air pocket, which extended part of the way down the downstream slope.

Case 5 is reported by Babb and Johnson[8] to be a 12-ft diameter discharge line siphon outlet structure at Grand Coulee dam. The line has a horizontal bend at the vertical knee. At the lower flow all air was cleared and vacuum established in 17 min. At the higher flow all air was cleared and vacuum established in 4.5 min.

Case 6 is a 16-in. D.I.P. force main in south Florida. A clogged air release valve upstream of a subaqueous canal crossing was unplugged and blew for several minutes, whereupon the 6-in. drain and blowoff valve was opened at the bottom of the descending leg at an elevation 6 m (20 ft) below that of the knee. This valve vented air for 10–15 min; the remainder of the air was vented through the air release valve.

Case 7 is a 36-in. D.I.P. outfall line in south Florida. Taps were made in the existing line to confirm friction coefficients with flows from 17.4–32.5 ML/day (4.6–8.6 mgd). A 1-in. tap just upstream of a 36-in. side outlet tee and 24 × 36-in. reducer vented air for 2–5 min each day it was opened. A 1-in. tap 146 m (480 ft) upstream vented no air although the flow and slope were identical. Slope of the EGL was 0.07 deg at 23.4 ML/day (6.2 mgd) or approximately 0.14 deg at 32.5 ML/day (8.6 mgd). Pipe slope is 0.20 deg.

Design of Pipelines to Prevent Air Binding

The following suggested design procedure incorporates other published recommendations along with the author's experience.

Step 1. Many unanticipated air pockets seem to be caused by the uncontrolled laying-to-cover of a pipeline. Typically the pipeline right of way is surveyed along a line offset from the centerline location. This profile is plotted on cross section sheets and air release valve locations determined by its use. A simple lay-to-cover specification permits the contractor to lay the pipeline at any depth so long as it is below the specified cover. Also, ground surface elevation differences may exist between the offset profile and the ground profile over the pipe centerline. It is suggested that if a lay-to-cover specification is preferred, the contract specify that the installed pipeline be profiled by the contractor as part of his work; as an alternative, cost permitting, the pipeline could be laid to a predetermined grade, particularly in hilly areas. This may permit the elimination of air release valves at intermediate high points (Fig. 10).

Step 2. Depending on the approach, the pipeline should be laid out to a trial profile. The design flow is then imposed on the pipeline to determine where air release valves are required for proper flow after the design flow is achieved.

Kennison[1] reported that where the energy grade line of a pipe during flow has a slope steeper than the pipe slope, bubbles move along easily because of the decreasing pressure gradient. In other words, the reference for air propagation is not necessarily a level line, but rather the energy grade line.

Alternatively, or at higher flows, one of the previously discussed criteria for pipe slope vs clearing velocity may be used. Because of air binding occurrences which conflict with some researchers' recommendations, conservative judgment is urged. For example, Kennison[1] placed air release valves at two obvious high points preceding steep descending legs—stations 25 + 50 and 46 + 64 (Fig. 11). Where air release valves are not yet placed but air binding is predicted, an energy loss equal to the vertical component of the descending leg should be included in the calculations.

Step 3. The pipeline should be analyzed for starting the flow. (With enough air-bound legs, the available head may not be able to start flow.) Assuming the worst case, the designer should total the vertical components of the remaining unvented descending legs and compare that figure with the available head. If the available head is less than or equal to the sum of these energy losses, the flow may not start. Therefore, additional release valves must be added until the energy grade line permits a flow that will clear all remaining flow pockets. Note that in the Fig. 11 profile, even with the aforementioned air valves, the starting head was not sufficient to overcome the remaining air-bound descending legs. Therefore, additional air release valves were added at stations 9 + 20 and 31 + 00.

Where it is difficult to obtain a sufficiently flat downgrade, it is better to have the steepest part of the slope near the upstream end and the flattest part near the downstream end. If the water flow cannot remove the air pocket, the loss of head will then be confined to a relatively short length of pipe. If the steepest invert grade were located near the downstream end of the slope, the air pocket would extend back to the top of the descending leg, causing a much greater head loss. Furthermore, the shorter the descending leg, the steeper the slope that can safely be designed, since the worst that might happen would be binding over a short section.

Investigators have found that a positive pipe slope in the direction of flow can be installed at any slope without encountering air problems in the ascending line.

Whitsett and Christiansen[9] report that the Metropolitan Water Dist. of Southern California experienced air problems caused by cascading; their experience indicates that the most severe problems occur with hydraulic jumps at vertical or horizontal bends in the pipeline. They recommend keeping the line and grade straight from the peak of the line to below the static water surface if cascading is necessary. Also, they have found that venting downstream of the hydraulic jump controls pressure surging but does not relieve white water.

In some circumstances it is desirable to obtain a sub-atmospheric siphon condition at knees above the operating energy grade line. Kennison has been successful in installing a combination air release and vacuum priming valve at such a point (station 47 + 00 of the Whitehall pipeline profile shown in Fig. 11). This valve releases air until the line approaches the normal depth for the flow resulting from the energy grade line with unprimed siphon. At this point it closes and remains closed as the water sweeps air pockets from the siphon knee. Kennison's data indicate that upon release of vacuum at this and other points, vacuum recovery occurs rapidly. Of course, the valve should always be installed below the minimum water surface of the upstream reservoir so that in case of air leakage into the pipe upstream of this valve some flow would still be maintained.

Conclusions

Additional field data will confirm one or more of these recommendations for minimum velocity to clear air pockets. A simple technique is to close existing air release valves on lines known to receive air from vertical turbine pumps or gases from septic sewage. In each case studied, the following data should be reliably noted:

1. Pipe slope—preferably expressed as the sine of the descending angle
2. Type of pipe material, its age, and, if possible, roughness coefficient. This will permit future evaluation of the effect of wall roughness on air removal.
3. Pipe inside diameter
4. Maximum sustained flow or, if little variation, average flow
5. Whether or not air pockets were discovered downstream of the knee.

These data can be organized and plotted as shown in Fig. 9. (The author would appreciate receiving any such data.)

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ATTACHMENT 5

AIR BINDING WITHOUT HYDRANT FLOW

Purpose: Determine if air binding is likely to occur in pipe #631under normal system demand. (Use Test 3, Event 2)

Given:	Elevation of node J3300 =	92.62	ft
	Elevation of node J92080 =	107.32	ft
	Length of pipe between nodes =	383	ft
	Pipe inside diameter =	7.96	inches
	Velocity in pipe #631 =	0.45	ft/sec

Solution:

1. Determine $(\sin\theta)^{0.5}$

   $\sin\theta = (107.32-92.62)/383 =$ 0.03838

   $(\sin\theta)^{0.5} = 0.196$

2. Determine $V/(gD)^{0.5}$

   $V/(\text{gravity}xD)^{0.5} = 0.45/(32.174x7.96/12)^{0.5} =$ 0.0974

3. Plot $V/(gD)^{0.5}$ vs. $(\sin\theta)^{0.5}$

   See FIGURE 1.

4. Conclusion

   The potential for the occurence of air binding is high.

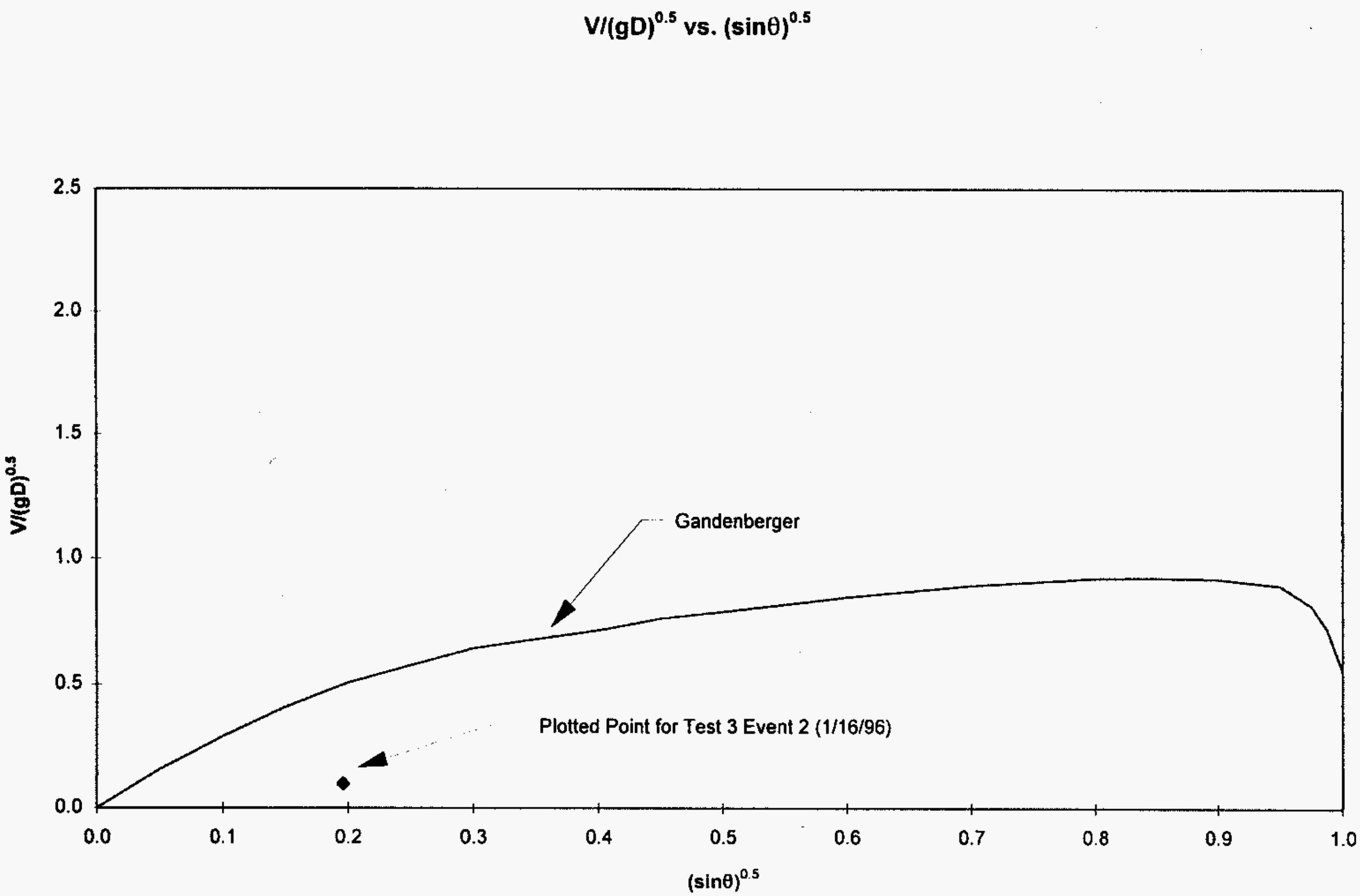

FIGURE 1. Indication of potential air binding under normal network demand (Test 3, Event 2).

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AIR BINDING WITH HYDRANT FLOW

Project No.: 19540-489-01-09
Project Name: SSU Model Calibration

Purpose: Determine if air binding is likely to occur in pipe #631under fire flow demand.

Given:	Elevation of node J3300 =	92.62 ft
	Elevation of node J92080 =	107.32 ft
	Length of pipe between nodes =	383 ft
	Pipe inside diameter =	7.96 inches

Solution:

1. Determine $(\sin\theta)^{0.5}$

   $\sin\theta =$ (107.32-92.62)/383 = 0.03838

   $(\sin\theta)^{0.5} = 0.196$

2. Determine $V/(gD)^{0.5}$

   For Test 1 Event 9

   $V/(gD)^{0.5} = 2.36/(32.174\text{x}7.96/12)^{0.5} =$ 0.51085

   For Test 3 Event 5

   $V/(gD)^{0.5} = 2.64/(32.174\text{x}7.96/12)^{0.5} =$ 0.57146

   For Test 4 Event 5

   $V/(gD)^{0.5} = 2.80/(32.174\text{x}7.96/12)^{0.5} =$ 0.60609

   For Test 5 Event 5

   $V/(gD)^{0.5} = 2.62/(32.174\text{x}7.96/12)^{0.5} =$ 0.56713

3. Plot $V/(gD)^{0.5}$ vs. $(\sin\theta)^{0.5}$

   See FIGURE 2.

4. Conclusion

   Air binding is likely to be in the incipient to clearing phase.

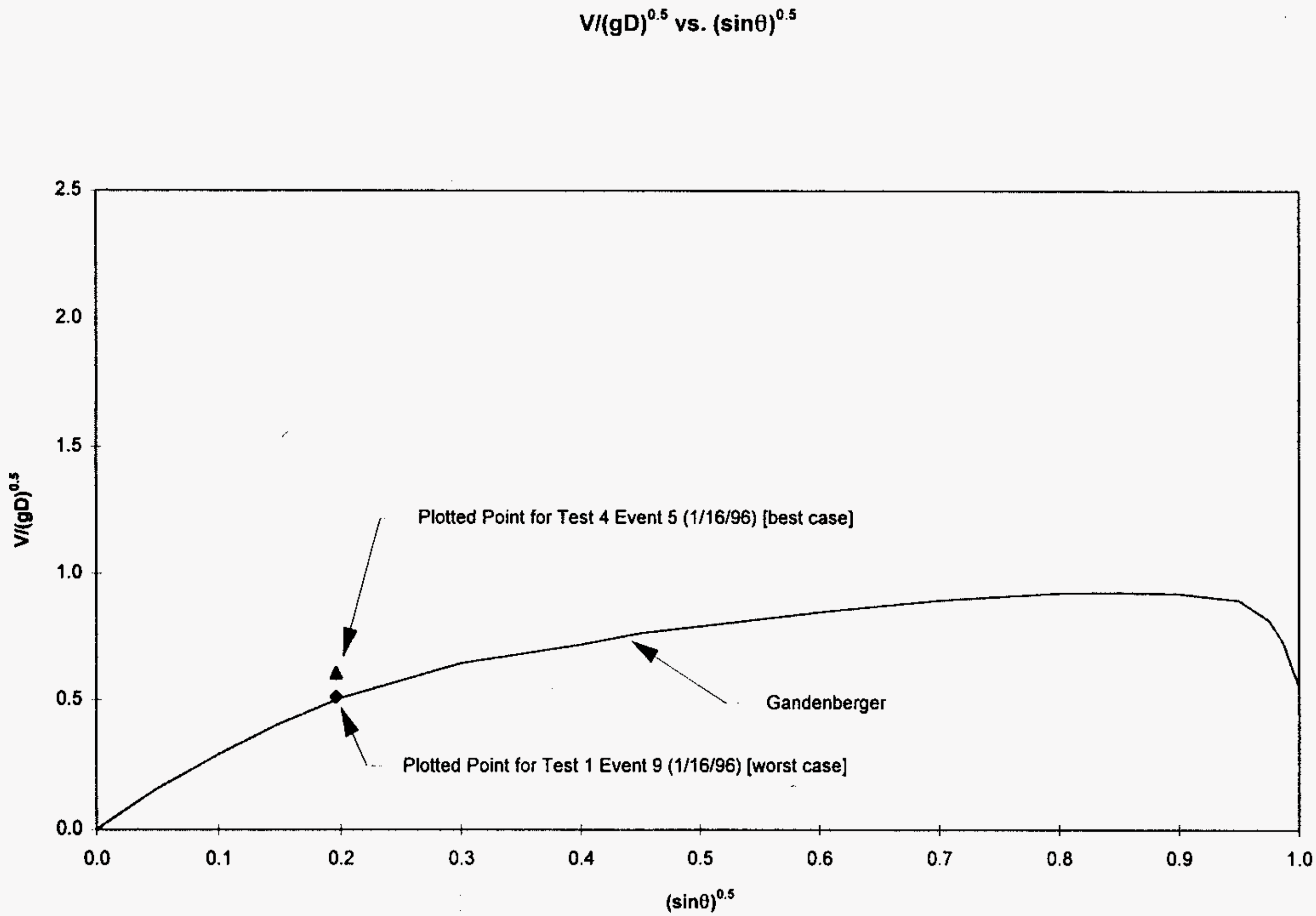

FIGURE 2. Indication of potential for air binding with hydrant flow.

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