A REVIEW OF THE PRESSURE TRANSIENT EFFECTS ON WATER DISTRIBUTION MAIN FAILURES

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Abstract

This paper presents a review of pressure transient effects on water pipeline failures. Water distribution mains in the world are becoming older and hence are experience more frequent failures. Prediction of pipe failures become very important as the failure of large diameter pipes could lead to high consequences of failure (economic as well as social) such as property damage, interruption to traffic and the loss of confidence in the water utility. The most challenging task is to predict such failures which can depend on several factors that are hard to estimate accurately. Among such factors, internal water pressures, pipe corrosion and traffic loads and can be paramount. Most of the failures of large water mains reported in the literature occurred as a result of high internal water pressure acting on corroded sections of the pipes. Hence, evaluation of steady state as well as transient pressures in the water network is particularly important in the process of pipe failure prediction. This paper reviews the state of the art of pressure transient modelling to determine the failures of water mains. Relevant failure prediction methods have also been identified to predict the failures of corroded as well as non-corroded water pipes associated with high transient events. The use of pressure transient modelling has been elaborated with the aid of a case study to determine the failures of corroded water pipelines.

Keywords: *Pressure transients, pipe failure, water networks, corrosion, failure prediction*

1. Introduction

Urban water distribution mains around the world are becoming older and hence are more prone to failure. Failure of large diameter mains is critical mainly because that can disrupt the water supply to the consumers. Furthermore, it may result in very high capital, social, and environmental costs to the public as well as to the water utilities. This is a global issue that respective water utilities need to effectively for the replacement and rehabilitation of water pipe assets. Current knowledge of how, where and when these failures occur is limited as a result of lack of available data on failures of larger diameter (>300mm) pipes. However, operational requirements that exist today are very different from the requirements of early days when pipes were initially installed. The standards of pipe material, pipe lay and design were also different during the construction of these older pipe lines. There are many factors need careful estimation in order to evaluate pipe failures accurately. Although understanding of such factors is important, it is hard to obtain sufficiently precise data that represent the failure condition of the pipe mainly because pipes are buried in the soil and condition of the pipes at each location is unknown for most of the pipes in a large water supply network.

Generally, failure of water pipes mainly depends on several factors such as pipe structural properties, material type, pipe-soil interaction, and quality of installation, internal loads due to water pressure and external loads due to overburden soil, traffic loads, frost loads and third party interference, and material deterioration due largely to the external and internal chemical, bio-chemical and electro-chemical environment [1]. Most of the networks installed before 1960's used the pipe material as cast iron that behaves mostly in brittle manner [25]. Although the external loads due to soil above the pipe can be estimated, it is hard to judge the loads coming from the live traffic. Corrosion rate is another key factor to determine pipe failures, but it is a highly troublesome to estimate as of the dependence on several factors such as soil moisture content, type of soil, pH etc. [2]. In addition, interntal water pressure can be quite unpredictable during a transient event which can lead to fail the pipe as evidenced in many previous case studies [26].

Internal water pressure can be subdivided into two main categories: namely, static operational water pressure and transient water pressure. The static water pressure can be obtained without much difficulty if there is utility owned measurements and hydraulic model, but separate hydraulic analysis and field pressure monitoring to validate the model are needed to obtain magnitudes of transient pressures. Pressure transient can be introduced as a transitional phase of the system from one steady state to another. This phenomenon can arise during sudden start up or closure of pump or valve, sudden change in demand condition such as fire fighting, during main break and due to action of a check valve. Once generated, these pressure waves will propagate throughout the distribution network causing significant pressures in certain locations of the networks. Pipes installed more recently can easily resist such loads as new design standards include provisions for transient pressure, but pipes installed long time ago can be susceptible to failures during transient events due to substantial reduction in wall thickness induced by corrosion. Hence, it is important to investigate the effects of pressure transients in order to facilitate the failure prediction process of operating water mains. This paper presents a review of the methods available to estimate the magnitudes of transient pressures along with a summary of field monitored pressures reported in literature. The knowledge of such methods can be helpful in determining the magnitudes of transients that can most probably be the major factor contributing to the water main failures.

2. Methods Available to Estimate the Magnitudes of Pressure Transients

Historical records indicate that the pipes failures due to water hammer or pressure transients happened since the 19th century. However, the amount of published literature is limited on failures induced by pressure transients due to the fact that the information about such failures was not shared easily in scientific and engineering community. The published paper by Bonin [27] in 1960 has reported damages to the water turbine in the Oigawa power station, Japan in 1950. This failure was caused by water hammer due to sudden closure of a butterfly valve.

In 1898, Joukowski introduced well known equation (1) to obtain the maximum possible pressure change upon pressure transient event. This equation is treated as the first quantitative assessment to obtain pressure rise during a transient event.

$$\Delta H = \pm \frac{a}{g} . \Delta V \qquad \text{------------------------------(1)}$$

Where a= speed of pressure transient wave, g =gravitational acceleration ΔH =change in pressure (given as head) and ΔV = change in flow velocity. This equation provides maximum pressure rise when the velocity in the pipe is changed, t_c is less than $2L/a$, where, L is the distance between the point of disturbance and the closest wave reflection point, and a is the speed of the pressure transient wave and t_c is the time to change the mean flow condition. Standard tables are available [3] to find approximate values of pressure increase for wave speed which primarily depends on pipe material and geometry. According to this equation, every 1m/s velocity change in the system can cause approximately 100m pressure rise in the pipes when the pipe material is steel, cast iron or ductile iron. However, this equation is not entirely applicable to field conditions due to theoretical limitations. As such, general equations have been proposed in literature [4] to describe the actions of a pressure wave during transient events as given in equations (2) and (3) for continuity and momentum conditions respectively. These equations, which are also called as classical equations of water hammer, are being widely used in several computer programs such as Surge 2000, InfoSurge, etc., to analysis the pressure transients in water distribution networks.

$$\frac{dH}{dt} + \frac{a^2}{gA} . \frac{dQ}{dx} = 0 \qquad \text{-------------------------(2)}$$

$$\frac{dH}{dx} + \frac{1}{gA} . \frac{dQ}{dt} - f(Q) = 0 \qquad \text{------------------(3)}$$

Where H is the pressure head, Q is the volumetric flow rate, A is the pipe cross sectional area and $f(Q)$ represents nonlinear pipe friction term which is a function of the flow rate. It is impossible to solve these two equations simultaneously except for networks with only few pipe segments. Numerical solutions have been available in computer software packages to solve such equations, especially catering for large distribution networks. The most commonly used methods in computer software to analyse pressure transients are the Characteristic method and Wave plan method (subsequently named as wave characteristic method). These two methods will be discussed briefly in this paper and descriptive information can be found in publications [4-6].

Method of Characteristics (Eulerian Method)

The continuity and momentum equations shown in equations (2) and (3) are pair of quasi-linear hyperbolic partial differential equations, which consist of two independent variables, time and distance along the pipe, and two dependant variables, velocity and hydraulic grade line. These equations can be transferred into four ordinary differential equations using the characteristic method [4, 5, 7]. Subsequently, finite difference techniques can be used to solve these equations with respect to time and space for obtaining pressures and flows for selected segments of the pipe.

Wave Characteristic Method (Lagrangian Method)

The main difference between the wave characteristic method (WCM) and the method of characteristic (MOC) is that WCM updates the hydraulic state of the system only when changes occur due to the exiting condition, whereas MOC updates the system as time advances in uniform increments. The WCM method tracks the propagation of transient pressure wave in the system and it calculates new conditions only at times when changes actually occur. This method is well documented in [6, 8-10] and a brief description of the WCM is given here.

A transient pressure wave can be generated in a pipe as a result of the changes in flow condition through pump start up, shutdown or valve operation etc., where incremental change in flow rate introduced to the system could result in substantial transient pressure waves. As the wave propagates along the system, necessary modifications are required to apply to cater for the junctions, pipe friction and other system components. This temporal variation can be summed up to obtain the final hydraulic grade line and the flow rate at the end of each time step. One of the very important characteristics of the WCM is the selection of time the step increment (Δt). This value should be sufficiently small to capture all disturbances ($\Delta t < L_{min}$ /a where, L_{min} = length of the shortest pipe in the network), and all modifications to the pressures and flows should be an integer number of this time step. Simply put, this method represents pipe system as a connected graph of finite number of discontinuities connected by frictionless pipe segments. The method develops a schedule of events that will occur as the pressure wave propagates through the network and solves new state of the system immediately after the discontinuity or event.

The features of commonly available computer packages to analyse pressure transients using the WCM and MOC are given in Table 1 highlighting their advantages and disadvantages.

Table 1: Summary of available computer packages for pressure transient analysis and their respective merits

Method of calculation	**Name of the computer package**	**Advantages**	**Disadvantages**
Wave Characteristic method	Surge 2000	➢ Computational efficiency and stability of calculations (suitable for very large networks). ➢ Suitable for pressure sensitive demand calculation. ➢ Dynamic friction option. ➢ Fully integrated with GIS software and steady state modelling software package, PIPE2000. ➢ Intrusion calculation.	➢ Single step friction calculation per pipe leads to the limitations on length of single pipe section.
	H20Surge/ InfoSurge	➢ Computational efficiency and stability of calculations (suitable for very large networks). ➢ Fully integrated with GIS and CAD software. ➢ User driven real time functionality that allows running EPS simulations and surging run for desired critical time with automatic calculation of boundary conditions. ➢ Intrusion calculation.	➢ Single step friction calculation per pipe leads to the limitations on length of single pipe section.
Method of Characteristics	Hammer	➢ Distributed pipe friction can be included. ➢ HAMMER can run on any of four supported platforms - GIS, CAD, MicroStation, and stand-alone offering true interoperability.	➢ Computationally inefficient and instability, depending on the selection of computational time.

3. Distribution Modelling Considerations to Estimate Magnitudes of Pressure Transients

Distribution networks which serve drinking water to the public are massive in size and complex in nature. Generally, it is quite hard to model entire network due to excessive model development time and inherent issues with computational aspects. Alternatively, full network can be modelled by excluding pipes below a certain diameter, but keeping hydraulically important connections or need to select a section from the network where, there is high risk of pressure transients. However, skeletonizing pipe connecting two pressure zone can distort actual results, and hence in cases where the effect of skeletonization is unknown, it is recommended to keep all the pipes in the model [11]. It is to be noted that due to the uncertainty of the effects of skeletonization, the best practice is to use previous knowledge of modelling to remove less important pipes segments.

Additional parameters are required to perform pressure transient analysis than that demanded in conventional steady state hydraulic models. In conventional steady state hydraulic analysis, pump start-up event can be simulated by simply providing the start-up time and pump head loss characteristic curve. However, performing a transient analysis requires data such as rated conditions of the pump, the curve of the pump impeller rotational speed vs. time during start-up and information of the associated check valves.

The calculation of the wave speed of the pressure wave also plays high significance in the process of pressure transient analysis. Don et al. [6] proposed the following equation (4) to determine the wave speed of a pressure wave:

$$c = \sqrt{\frac{E_f}{\rho(1 + K_R E_f D / E_c t\ )}} \quad \text{-------------------------}(4)$$

Where E_f and E_c are the elastic modulus of the fluid and conduit, K_R is the coefficient of restraint for the longitudinal pipe movement, ρ is the density of fluid within the conduit, D is the pipe diameter and t is the pipe wall thickness. Recent studies [12,13] showed that for a network which consists of majority of its pipe material as metal pipes such as cast iron, mild steel and ductile iron, c can be assumed as 915m/s (3000ft/s) for obtaining realistic pressure transient magnitudes. This conclusion has been verified against field measured pressures elsewhere [12, 13].

The other important characteristic of transient modelling is to maintain proper communication among field crew, asset management and modelling team. Nowadays, most of the water utilities operate using Geographical Information System (GIS) based approaches, integrated with hydraulic models to upgrade the latest changes to their network such as pipe replacement or extension of the network. While such approach can be helpful for effective internal communication, they can also be beneficial to interpret the outputs of pressure transient modelling analyses. However, such outputs should be validated first against actual field measured data prior to use in any failure assessments. In order to avoid the problems due to mismatch between models and field measurements, it is recommended [12] to compare the results from existing steady state hydraulic models with actual field measurements which are readily available through Statutory Control and Data Acquisition System (SCADA). Most of the large water networks are integrated with SCADA to monitor live data of the network. Extended Period Simulation (EPS) can be performed to obtain steady state pressures in absence of calibrated steady state hydraulic models.

4. Field Pressure Monitoring to Capture Pressure Transients

The reliability of any pressure transient hydraulic model depends on how accurately they can predict the real field events. To measure magnitudes of transients, conventional pressure measuring instruments are not adequate as the wave propagation speed of pressure transients is close to the speed of sound.

Table 2.Summery of field pressure monitoring

Authors	Network	Network/ pipe length (km)	Average daily demand (m^3/d)	Operating pressure (kPa)	Minimum pressure recorded (kPa)	Maximum pressure recorded (kPa)	Pressure difference from operation (kPa)	Remarks
Ebacher et al. **[17]**	Large size surface water system	1,590	210,000	433.6	35.3	-	rise=N/A fall= 398.3	Pressure monitoring for 17 months in the network. Down surge events resulting in negative pressure are recorded. These events were caused by power failure causing pump shutdowns.
Fleming et al**.** **[18]**	Medium sized ground water system	96	11,583	348	8	587	rise=239 fall=340	Pressure monitoring was conducted for 5 days. These events were caused by pump operation.
Friedman et al. **[19]**	Large size surface water system with 2 pressure zones and 12 storage tanks	-	154,806	860-1,205	-	-	rise=N/A fall=903	This event corresponds to sudden pump stoppage due to power failure at main pump station. This measuring site is located immediately downstream of main pump station.
Friedman et al. **[19]**	Same system above	-	154,806	965	173	1,170	rise=205 fall=792	These pressure measurements were taken during annual pump shutdown event of same system above. Pressure monitoring was conducted during five different days. This measuring site is same as previous.
Friedman et al. **[19]**	Same system above	-	154,806	620	0	1,130	Rise=510 fail=620	This pressure measurement was taken at near service line during event of fire fighting when fire department closes and then opening the hydrant under normal fire fighting condition using a pumper truck.
Friedman et al. **[19]**	Large size surface water system with seven pressure zones and 6 storage tanks	-	96,518	380	103	-	Rise= N/A fall= 267	This pressure measurement is correspond to one main break occurred in site close to high service zone
McInnis et al. **[20]**	Moderate size surface water system with majority of pipe are 300mm	90	-	1300	-	-	±250-300	This pressure measurement is corresponding to shut down and start-up of single centrifugal pump. Pump was shut down by pushing the emergency stop button, which immediately cuts power to the pump motor.

N/A=value was not found in the reference

High speed pressure transducers should be used to take measurement for pressures during transients in order to obtain reliable results. In addition, measuring equipment should have high internal memory to record readings for a reasonable period of time. One such instrument which is used extensively during previous research work is RADCOM pressure transient data logger [12, 13, 15, 16]. The instrument and the way it can be attached to a fire hydrant (most common place where instrument attach to the network) are shown in figure 1 and 2 respectively. A summary of previous pressure monitoring work conducted using RADCOM has been presented in Table 2.

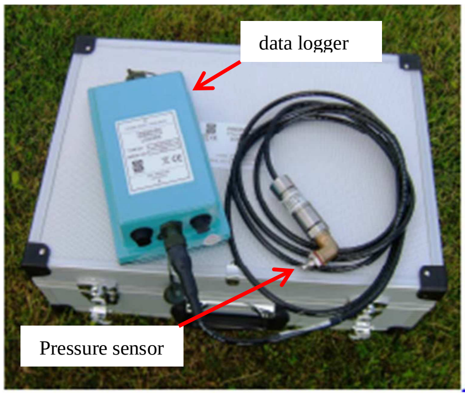

Figure1. Radcom pressure sensor and data logger

Figure2. Instrument installed at a fire hydrant

Preliminary objective of the work summarised in Table 2 was mainly to investigate the susceptibility of distribution networks to negative pressures which may bring contaminated water into the network through a possible system leak. It can be seen from the previous studies that the water pressure rise due to a transient event in a given system can be as high as 600kPa. Such pressure rise, coupling with the static water pressure would generate massive water pressures within the system, leading to catastrophic failures or dramatic reduction in the factor of safety of corroded pipes. Therefore, careful assessments are needed in this regard to estimate precise rise in water pressures.

5. Pipe Failure Prediction

There are several methods available in literature to predict pipe failures. In 1930, Schlick proposed a failure criterion (Fig 3) which is being extensively used at present to predict failure loads of new pipes and factor of safety of existing pipes [21]. The application of Schlick diagram for pressure transient is straightforward for newly installed pipes or for pipes with uniform corrosion as it only demands to add the increment of pressure on top of the existing static head. The determination of acceptability depends on the total internal (mainly water pressure) and external (traffic and earth loads) loading to the pipe as shown in Fig. 3.

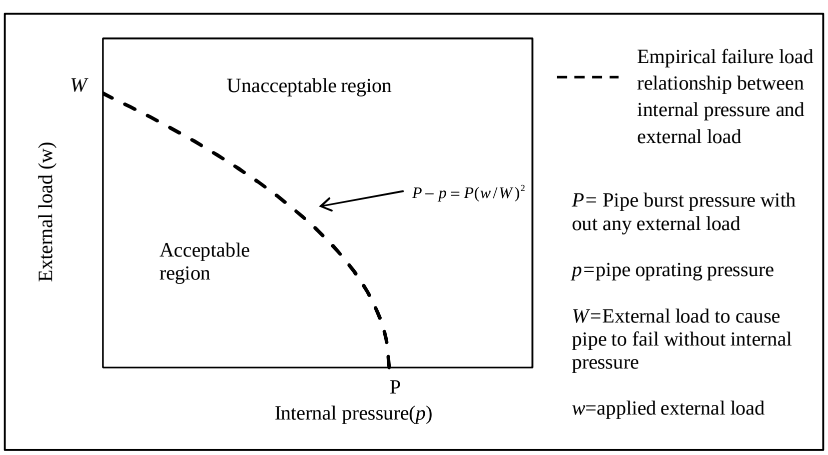

Figure3. Schlick failure criterion for water pipes

It should be noted that the Schlick diagram approach is only applicable for new pipes or pipes with uniform corrosion. The majority of pipes in water networks contain corrosion defects which have different shapes and sizes, and hence violating the assumptions of Schlick failure criterion. Alternative methods are available in literature to analyse such out-classed scenarios. Most of such methods are from the studies based on oil and gas pipelines (mainly made of ductile materials) of which the structural capacity is far above than that of cast iron water pipes. Hence further studies are required to validate and/or to propose modifications for available failure methods to apply in water pipes.

A failure assessment method that commonly uses to accommodate the failures of corroded pipes is based on Failure Assessment Diagrams (FAD) [22-24]. In FAD diagrams (Fig. 4), horizontal axis (L_r) is ratio of the applied stress to the stress to cause plastic yielding of the structure containing a flaw, and vertical axis (K_r) is the ratio of the applied linear elastic stress intensity factor to the material fracture toughness. All relevant references provide different levels of assessments depending on the level of conservativeness required. Higher assessment levels need more inputs and involve complex calculations in contrast to lower level assessments. Figure 4 shows typical failure assessment diagram given in BS 7910 assessment level 2. Failure is described by limiting line which is a nonlinear function of L_r. It is to be noted herein that the full diagram has been slightly modified in order to use it for a brittle material such as cast iron, but complete curve should be used when assessing materials that have significant yielding prior to failure. The method can be used to asses fully brittle failure (K_r=1) as well as plastic collapse of material (L_r=Lr_{max}).

Use of failure assessment diagram to predict failures is not as straightforward as the use of Schlick diagram due to the fact that both L_r and K_r depend on applied loads (pressure loading due to transient). Appropriate stress concentration factors, stress intensity factors and flaw classification methods should be used in all relevant calculations. Though most of the pre-required factors can be assessed in simple calculations, obtaining an appropriate stress intensity factor can be troublesome especially for water pipes. In such cases, they can be obtained either though finite element analysis or experiments for a given material loaded at a particular flaw geometry.

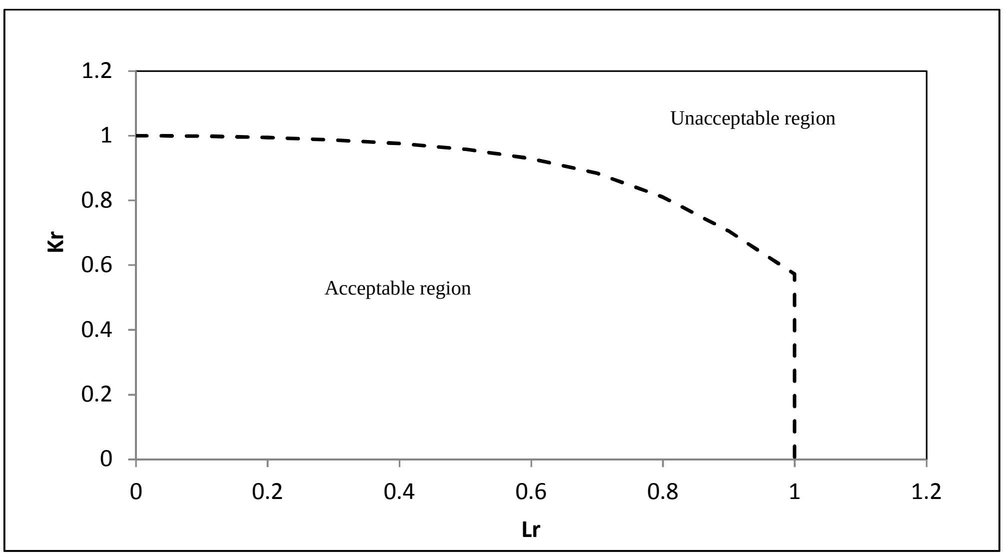

Figure 4. Typical failure assessment diagram (FAD)

6. Case Study

A case study has been conducted on the basis of an imaginary network to elaborate the use of pressure transient analysis in determining the failure of corroded cast iron pipelines. The details of the selected network are given in Table 3 and schematic of the network is shown in figure 5. The network is modelled in Surge2000 pressure transient analysis program for two different transient events which were induced by different pump start-up times (7s and 25s). Having obtained the transient pressures using the developed hydraulic models, failure assessments were conducted on two pipes having different defect geometries based on FAD – Level 2 given in API 579 (22) (volumetric effect of corrosion defect is not considered herein). The schematic view of corrosion defects are given in figure 6a and 6b for 3-D and sectional view of the corrosion defect respectively. Table 4 shows the results of pressure transient analyses along with the properties of the pipe (including defect geometry) used for the failure assessments.

Table3. Network data

Pipe ID	Length/(m)	Diameter/(mm)	Roughness	Node ID	Elevation/(m)	Demand(L/s)
P-1	725	600	100	1	05	0.0
P-2	227	600	100	2	30	0.5
P-3	337	203	100	3	18	0.9
P-4	357	203	100	4	18	0.5
P-5	265	600	100	5	30	0.5
P-6	405	600	100	6	20	0.3
P-7	796	600	100	7	42	0.3
P-8	415	203	100	8	34	0.6
P-9	140	600	100	9	55	0.9
P-10	312	203	100	10	40	0.3
P-11	320	600	100	11	45	2.2
P-12	504	600	100	12	45	1.0
P-13	177	600	100	13	64	0.1
P-14	143	600	100	14	48	0.1
P-15	88	600	100	15	50	0.1
P-16	427	203	100	16	46	1.3
P-17	490	203	100	17	50	1.3
P-18	186	203	100	18	30	1.3
P-19	324	203	100	19	46	0.3
P-20	201	203	100	20	49	1.2
P-21	191	203	100	21	49	1.0
P-22	181	203	100	22	04	0.6
P-23	379	203	100	23	52	0.5
P-24	396	203	100	24	51	0.7
P-25	342	203	100	25	54	0.4
P-26	375	203	100	27	40	0.5
P-27	88	600	100	28	34	0.0
P-28	93	600	100	29	34	0.4
P-29	123	305	100	30	40	0.2
P-30	160	600	100	31	55	1.1
P-31	239	203	100	32	34	1.1
P-32	182	203	100	33	48	0.1
P-33	199	600	100	34	45	0.1
P-34	333	203	100	35	34	0.0
P-35	132	600	100	36	34	0.1
P-36	25	203	100	26	72	Tank
P-37	233	203	100	37	15	Reservoir
P-38	325	203	100	38	15	pump
P-39	315	203	100	-	-	-
P-40	190	203	100	-	-	-
P-41	180	203	100	-	-	-
P-42	25	600	100	-	-	-

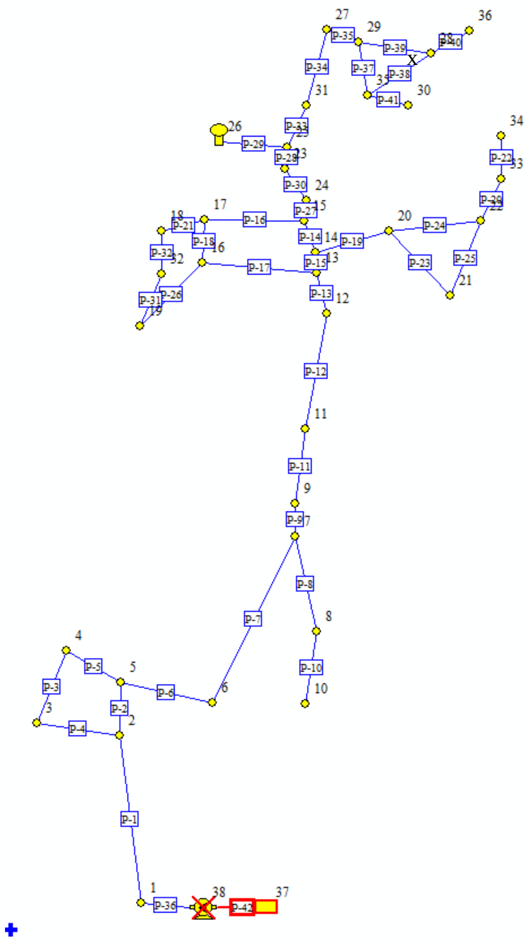

Figure 5: Schematic view of pipe network

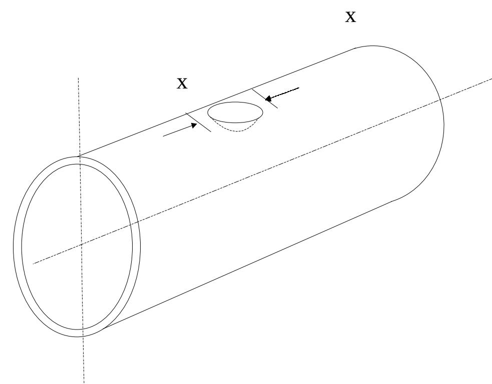

Figure 6a: Schematic view of pipe with spherical corrosion defect

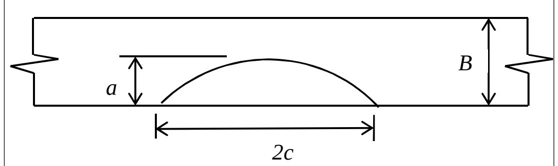

Figure 6b: Section x-x; 2D projection of corrosion defect

Table4. Details of pressure and assumed corrosion defects on selected pipes

Pump start-up time/(s)	Pipe ID	Pipe diameter/(mm)	Nominal wall thickness/ (mm)	Steady state pressure/(kPa)	Maximum pressure/(kPa)	a/mm	c/mm
07	P-1	600	25	502	985	10	25
	P-7	600	25	305	705	18	50
25	P-1	600	25	502	690	10	25
	P-7	600	25	305	492	18	50

The resulted wave pulses during pressure transient events are shown in figure 7. The maximum pressure rises of 483 kPa and 400 kPa are observed in the pipes of P-1 and P-7 respectively, during 7s pump start-up event. The difference in the pressures can be attributed with the different damping actions acted on pipes which are located approximately 1.4 km apart. The increase in pump start-up time to 25s resulted less pressure rises (~180kPa) in both the pipes analysed in this study. This is because once the pump start-up time is increased, the rate of change of mean flow rate becomes less than the previous value.

Failure assessments were conducted to investigate the conditions of the two corroded cast iron pipes under different pump start-up times using FAD – Level 2 assessment given in API 579 [22]. The investigations have been performed on the basis of assumed material properties as shown in Table 5. Having obtained the fracture ratio (*Kr*) and stress ratio (*Lr*), the condition of the pipe can be dictated from the FAD diagram as shown in Figure 8. For example, the factor of safety (FOS) of the pipe P-1 for 25s pump start-up event can be defined as the ratio of OA/OB. It can be seen from the results that the lowest FOS was resulted in pipe P-7 even though the maximum pressure in pipe P-1 is the highest. This reveals that the severe corrosion coupling with a lower pressure can trigger the pipe to failure rather than higher pressure acting on less corroded pipe. Therefore it is highly significant to obtain the accurate corrosion details of the pipe along with the precise transient pressures to determine the failure condition of water pipes.

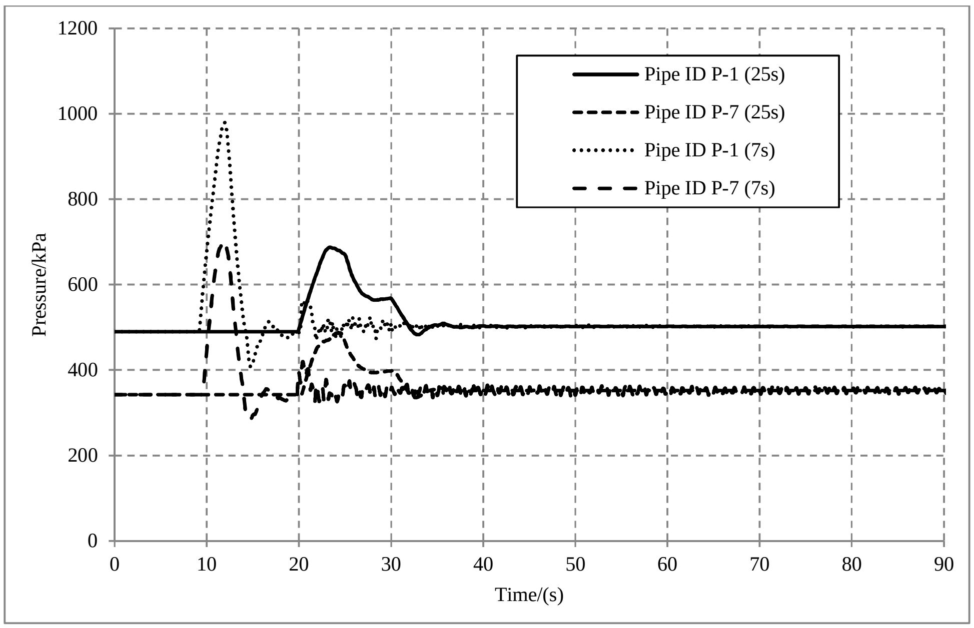

Figure 7: Resultant pressure variation during simulated transient events

Table5. Parameters used for FAD analysis

Parameter	Load case			
	P-1 (7s)	P-1 (25s)	P-7 (7s)	P-7 (25s)
Maximum pressure[a]/(kPa)	985	690	705	492
Nominal stress[b]/(MPa)	11.8	8.28	8.46	5.90
Tensile(σ_t) strength/(MPa)	130			
yield (σ_{ys}) strength(0.2% proof)/(MPa)	100			
Fracture toughness(K_{mat})/(MPa√m)	10			
Reference stress[c](σ_{ref}) /(MPa)	70.5	48.3	67.1	46.8
Stress ratio ($L_r = L_{ref}/L_{ys}$)	0.94	0.64	0.89	0.62
Stress intensity factor [c] (K_I)/(MPa√m)	1.19	0.85	2.23	1.57
Fracture ratio (K_r=K_I/K_{mat})	0.17	0.12	0.32	0.23
Factor of safety(FOS) based on FAD	2.0	3.2	1.64	2.41

a - Loads due to external loads were assumed zero and the maximum pressure is based on the maximum water pressure
b - Nominal stress was calculated using the equation, σ_{nom}=pR/t where, p is the maximum pressure, R is the pipe mean diameter ,and t is the nominal pipe wall thickness
c – The values were calculated based the procedure given in API 579

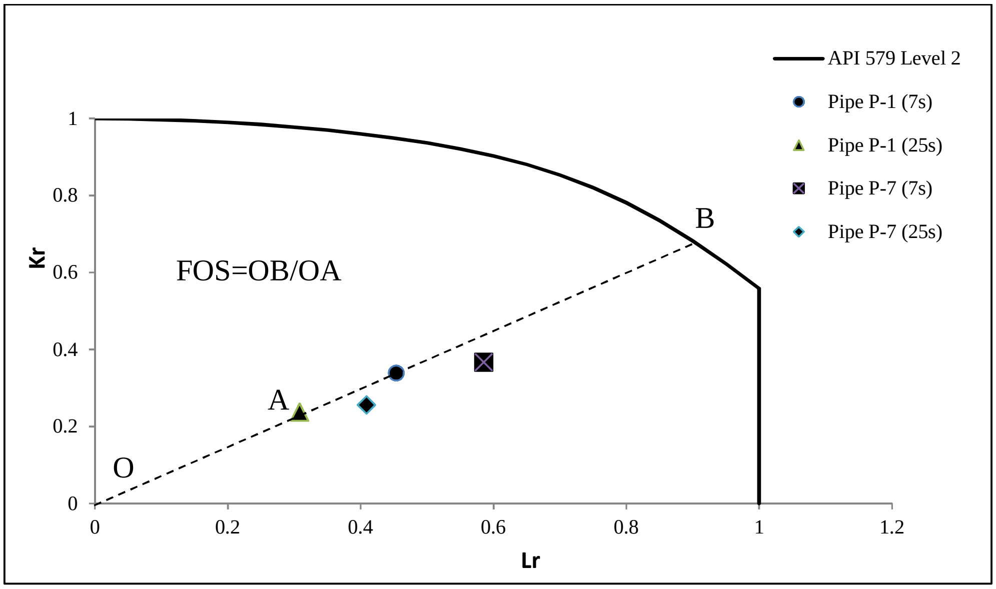

Figure 8: API 579 Level 2 FAD analysis for selected pipes

7. Conclusion

This paper reviews the state of the art of pressure transient modelling to determine the failures of water mains. There is a high possibility of pressure fluctuations in water distribution networks due to the events such as pump start up, valve closure etc., as can be evidenced from past studies. Such fluctuations induce waves that can be transmitted to the whole network causing substantial pressures at certain locations. These pressures can be assessed either using the WCM or the MOC depending on the accuracy required or resources availability. Pipe failures will occur when such pressures reach the degraded structural capacity of the pipe by corrosion. In the absence of corrosion induced defects, failure assessments can be conducted simply by using Schlick failure criterion. However, more rigorous analysis such as FAD needs to be performed to assess the failures, if there is high level of corrosion in the water main. However, these methods need to be further examined for more brittle cast iron water pipes undertaking specific research on water pipe failures and corrosion patterns. Such research is currently underway at Monash University through a major collaborative research program in partnership with a number of organizations (see acknowledgement).

8. Acknowledgement

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