

**Renewal of Potable Water Systems Using Cement
Mortar Lining; an Investigation into Corrosion
Reduction and Water-loss Prevention**

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners. I understand that my thesis may be made electronically available to the public.

Keith B. Moggach

Abstract

Many Canadian cities are faced with the problem of an aging and deteriorating iron water distribution network - pipe breaks, leakage, and/or aesthetic water quality problems. Public confidence in municipalities' ability to deliver safe, clean drinking water to customers has been eroded, especially in areas of water distribution networks receiving coloured water events that result from the internal corrosion of aging iron watermains. Cement mortar lining is one of the most widely used non-structural watermain rehabilitation methods for the prevention of coloured water events due to internal iron pipe corrosion; however, it is also thought/claimed to be a means of controlling corrosion pin-hole leakage.

This thesis presents the results of a laboratory testing program designed to investigate the renewal of potable watermains via the use of cement mortar lining. The specific focus of this thesis is the ability of the cement mortar lining to bridge corrosion pin-holes and prevent water loss from the watermain, and the effects of mortar application on the corrosion protection provided to the iron watermain by cement mortar lining. The results of this study are based on short term testing and do not consider fatigue.

The ability to bridge corrosion pin-holes / water loss prevention laboratory testing program found that pressure should not be returned to a newly lined watermain until the lining has cured for a period of at least four days to prevent failures from occurring prior to the lining achieving sufficient strength characteristics if the lining is to be used as a structural rehabilitation technique. The cure time corrected normalized thickness at failure data was found to be a Gumbel distributed data set. The Gumbel distribution can be used to predict

the lining thickness required to bridge a known corrosion pin-hole diameter with a set degree of confidence that failure will not occur. A 3 mm thick cement mortar lining can bridge a pin-hole 12.0 mm in diameter while a 5mm thick cement mortar lining can bridge a pin-hole 19.9 mm in diameter with a 95% probability that failure will not occur

Through the corrosion prevention testing program it was determined that the thickness of the cement mortar lining does not affect the ability of the lining to prevent corrosion from occurring. This was determined for cast iron pipes which have been lined for a period of one year. It is recommended that corrosion potential testing be performed on cement mortar lined watermains that have been in service for a longer period of time to determine if this consistent over the life cycle of the cement mortar lined watermain.

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Dedication

To my family and friends.

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1. Introduction

1.1 General

Essential services delivered by Civil Infrastructure Systems (CIS) in North America provide the building blocks upon which healthy, prosperous and safe communities are constructed. Transportation and environmental systems are fundamental to the prosperity that North American cities have enjoyed in the past 60 to 100 years, a period of extraordinary growth. The deterioration of these North American infrastructure systems, highways, roads and airports, water supply, storm water and wastewater treatment systems is now reported and recognized as a national problem (TD Bank Financial Group, 2002). Failure to address the renewal of this aging infrastructure will result in significantly increased maintenance, operation, and rehabilitation costs. Any loss of service would threaten public health, the environment and the economic prosperity of our communities.

Many Canadian cities are faced with the problem of an aging and deteriorating iron water distribution network - pipe breaks, leakage, and/or aesthetic water quality problems (*dirty*,

red, brown or black water referred to as coloured water). Recent events such as, the Walkerton, Ontario E-coli breakout, have eroded public confidence in municipalities ability to deliver safe, clean drinking water to customers. This is especially true in areas of water distribution networks that are receiving coloured water events that result from the internal corrosion of aging iron watermain.

Since the early 1930s cement mortar lining has been used as an iron pipeline rehabilitation method and is currently one of the most commonly employed watermain lining practices in North America. Cement mortar lining consists of applying a cement coating to the interior of pipe. This coating raises the pH at the iron surface and moves the iron pipe into a state of passivation. Thus, the cement mortar lining results in a very thin, practically invisible, stable oxide layer on the iron pipe, which inhibits corrosion.

The City of Toronto Water and Waste Water Services is the industrial partner for this project. The Water and Waste Water Department is the public water utility that operates, maintains and delivers potable water to the residents of the City of Toronto.

1.2 Problem Statement

Recent cement mortar watermain rehabilitation lining studies completed at the University of Waterloo and the Centre for the Advancement of Trenchless Technologies (CATT), also located at the University of Waterloo, have noted that few known published studies are available to substantiate industry claims with respect to:

- Ability of the cement mortar to bridge corrosion pinholes in the pipeline

- The effect of cement mortar thickness and percentage of cement mortar coverage required in order to prevent internal iron pipe corrosion from occurring

1.3 Goals and Objectives

The objective of the study is to complete field and laboratory studies that will determine if the industry claims outlined in Section 1.2 can be substantiated. This data will also help the City of Toronto and other Municipalities justify their annual cement mortar lining water pipeline rehabilitation budget.

An improved understanding of these key cement mortar lining properties will also aide the City of Toronto in updating and revising their current cement mortar lining specifications.

2. Literature Review

2.1 The State of Urban Water Distribution Networks

Unlike other municipal infrastructure resources that our economy relies upon to function properly, water distribution networks suffer from a case of a public misperception of need. This is due to the facts that water distribution networks are buried assets which the public cannot easily see and that their performance over the past 50 to 100 years has been remarkable. Often, the deterioration of watermains is not apparent until a catastrophic failure, causing major disruptions, occurs or the water flowing from a tap is coloured and odoriferous. The public may not perceive the deterioration of North America's water distribution networks but the deterioration is occurring. There are 54 000 community drinking water systems in the United States of America which supply potable water to over 250 million Americans (ASCE, 2006). The provision of clean water is an essential part of the North American economic and public health system. According to the Water Infrastructure Network "not meeting the investment needs of the next 20 years risks

reversing the environmental, public health, and economic gains of the last three decades (WIN, 2000).”

In 1997 the Canadian Water and Wastewater Association estimated that \$11.5 billion CDN would be required for watermain upgrading over the next 15 years (CWWA, 1997). These Canadian estimates from 1997 are most likely lower than the actual totals currently needed due to under-funding in the past decade. Due to their similar infrastructure ages and use of construction techniques the United States of America’s water infrastructure needs are a good analogue to Canadian needs and they have been studied in more detail.

In 2000 the Water Infrastructure Network estimated that \$11 billion per year would need to be invested in American water distribution systems above the current funding levels (WIN, 2000). This funding gap of \$11 billion per year does not account for any increases in demand and thus is a low cost estimate of the differences between projected funding and required capital investment (ASCE, 2006). These estimations also neglect the impact of increased operational and maintenance costs of expanded networks. In 2002 the United States Environmental Protection Agency estimated that the drinking water capital, operational and maintenance funding gap for the next 20 years will range from \$45 billion to \$263 billion depending on the degree of future increases in spending (EPA, 2002). When looked at in terms of purely capital investment, the speculated needs rose \$10 billion from the speculated needs released in similar report by the US EPA in 2001 (EPA, 2002). These predictions were made early this century and show evidence of increasing every time they are recalculated, however, the United State’s federal budget for drinking water infrastructure has

remain static at \$850 million for the past three years (ASCE, 2006). This trend can only lead to further increases in the gap between the funding available for drinking water infrastructure and the cost associated with providing North American's with safe drinking water.

The figures quoted above refer to entire water distribution systems including water treatment facilities, finished water storage, finished water distribution systems, source water development, water supply management, source water protection, demand management, and rehabilitation of raw water transmission and storage infrastructure. Thus they can be expected to be lower for the linear (watermain) component of the drinking water system. However, the National Guide to Sustainable Municipal Infrastructure claims that watermains and pumping devices account for between 50-80% of the expenses incurred in the operation of an overall potable water system (NRC, 2003). Thus finding lower cost means of extending the service life of existing watermains is an essential aspect of overcoming the drinking water system funding gap.

The Water Infrastructure Network surmised the reason for the funding gap as follows:

“Over the next several decades, many cities will need to replace water and wastewater facilities and pipes that were installed in response to population growth and demographic shifts in the late 1800s and early 1900s. The next wave of infrastructure investments responded to post-war demographic changes in the 1920s and 1950s. Since the economic lives of materials shortened with each new investment cycle, many local utilities will face unprecedented funding hurdles as multiple generations

of infrastructure wear out, more or less at the same time, over the next two decades (2001).”

The timing of these waves of infrastructure investment is an important driver behind the need to know more about the cement mortar lining process. As shown in Section 2.5.1 these periods of infrastructure investment correspond to periods where iron was the primary material used for watermain construction. Thus it is these pipes which are in need of rehabilitation and are primary candidates for rehabilitation using cement mortar lining.

In 1993 Rajani and McDonald surveyed 21 Canadian cities which included 11% of the population of Canada. The survey indicated that 50% of the water distribution pipes in use in Canada were grey cast iron (McDonald and Rajani, 1995). Kirmever et al. provided similar results (48% cast iron) for the United States in 1994. These numbers have decreased in the past 13 years due to population growth coupled with the addition of new watermains constructed of alternative materials to the water distribution system. However, they still indicate that cast iron composes a significant portion of the water distribution systems.

The percentage of cast iron watermains in the drinking water system makes research into means of prolonging the service life of these watermains an important aspect in minimizing the capital costs associated with replacing these watermains as well as minimizing operational and maintenance costs associated with cleaning and flushing the watermains. In order for North America to deal with the funding gap in our drinking water system cost savings must be found in all aspects of the drinking water system. Prolonging the service life

of the iron watermain in the drinking water system through the use of cement mortar lining is a step in the direction of reducing the funding gap.

Prolonging the service life of the iron watermain through rehabilitation techniques is necessary because it is not feasible to replace the existing aging watermain. This is due to:

- the high cost and lack of funding available for replacement
- social impacts to the public and the economy due to replacement techniques
- the small portion of existing pipes that are in a state of deterioration which requires replacement

For network owners to effectively manage these networks in an efficient and cost effective manner they require a variety of tools. Improvement of water quality due to rehabilitation by cement mortar lining is one of these useful tools.

2.2 Watermain Rehabilitation Techniques

There are two main classifications of watermain rehabilitation techniques: structural and non-structural. Both the structural and non-structural classifications have numerous specific techniques which are tailored towards specific rehabilitation goals and site specific conditions.

Due to the buried asset nature of watermain it is hard to concretely determine the condition of the watermain. There have been some recent trials completed where small video cameras have been inserted in hydrants to access watermain but the technology is still in its infancy and is not an industry accepted practice (Bajor, 2006). There is not an accepted standard

methodology for assessing the condition of in-service watermain. Condition assessment of in-service watermain is challenging and not commonly attempted due to the following reasons:

- watermain are a pressurized system
- internal entry often requires that the line be taken out of service and utilizes specialized equipment and time consuming disinfection procedures
- there are limited and costly non-destructive testing techniques available to perform external condition assessment
- there is a limited amount of historical performance data available for the majority of water distribution networks

As a result most decisions regarding the need for structural versus non-structural replacement decisions need to be made based on customer complaints regarding water aesthetics, monitoring of pipe hydraulics, and watermain breakage frequency reports from the project vicinity.

Structural watermain rehabilitation is required in situations where there is a high watermain break occurrence rate or where leakage is severe. Non-structural watermain rehabilitation is required in situations where the hydraulic capacity of the watermain is limited by tuberculation and or drinking water aesthetics are compromised (NRC, 2003).

The National Guide to Sustainable Municipal Infrastructure has outlined a schematic flow-chart to aide municipalities in choosing the proper rehabilitation technology for a specific

project. This flow chart is represented in Figure 2-1. The technologies outlined by the National Guide to Sustainable Municipal Infrastructure Best Practice Manual deal only with general construction technologies and there is a wide variety of industry techniques which fit in each technology heading. Based on this Best Practice Manual cement mortar linings are recommend for structurally sound pipes where internal corrosion is the main problem (NRC, 2003). The United Kingdom Water Industry Research outlined a similar rationale for determining the usefulness of cement mortar lining on specific watermain rehabilitation projects (UKWIR, 2000).

In the best practice model cement mortar lining is primarily listed as a non-structural rehabilitation method, however, it is also listed as a means of controlling pin-hole leakage. This complies with industry claims that cement mortar lining is an effective means of preventing leakage from corroded iron watermains. Cement Lining Corporation International and Spiniello Companies, two of the larger North American cement mortar lining contractors, both claim that cement mortar lining stops leaks from corroded iron watermains (Spiniello Companies, 2006, Cement Lining Corp Int'l, 2006).

A literature review has been complete and no significant amount of scholarly research was found to substantiate the claim that cement mortar lining can prevent leaking due to corrosion. These industry claims are mainly based upon a study performed by the City of Detroit in 1940 (City of Detroit, 1940) which were also reported in Water and Sewage in 1947 (Dorrance, 1947). These test results are explained in Section 2.3.3.

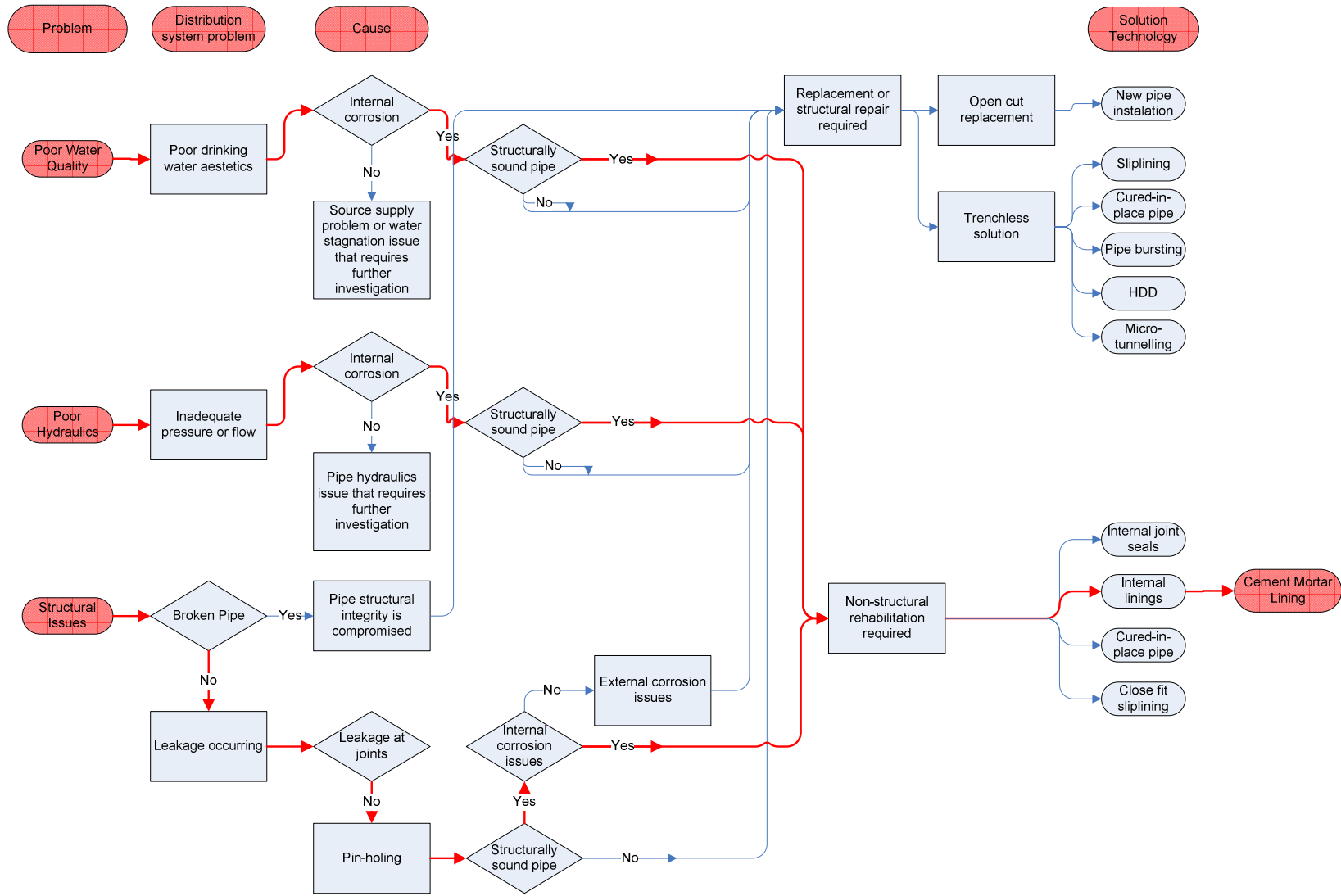


Figure 2-1: Rehabilitation technology selection flow chart (adapted from NRC, 2003)

2.3 Cement Mortar Lining

2.3.1 History

The ability to protect pipes against internal corrosion via the use of a cement lining has been known since the early 1840s and was first used for cast iron pipes in 1845 (Dorrance, 1947, Wood, 1933). However, it was not possible to line pipes already in service at that time and in-situ cement mortar lining was not commonly practiced until the 1940s.

There is not an agreed upon date for the first successful in-situ lining of iron pipes using cement mortar lining. Dorrance claims the first in-situ installation of cement mortar lining occurred in Akron Ohio in 1940 while Wilson claims the first installation occurred in 1921 and Matheny claims that the first in-situ internal lining was performed in Australia in the early 1930s (Dorrance, 1947, Wilson, 1971, Matheny, 1961).

In 1939 the first North American standard for in-place cement mortar lining of water pipe was published as part of American Water Works Association (AWWA) standard C205-41 (formerly 7A-7-41), Standard for Cement-Mortar Protective Lining for Steel Water Pipe (AWWA C602-00, 2001). Cement mortar lining is currently one of the most commonly used watermain rehabilitation methods in North America.

2.3.2 Cement Mortar Lining Process

The process of Cement Mortar Lining can vary in detail from contractor to contractor but all cement mortar lining jobs require that the same basic steps be performed. The process outlined below complies with the Corporation of the City of Toronto’s specifications for cement mortar lining (City of Toronto, 2001). The method specified by the City of Toronto complies with the AWWA standard C602-00 for cement mortar lining pipes larger than 100mm in diameter (AWWA, 2001). An overview of the cement mortar lining process is presented in Figure 2-2.

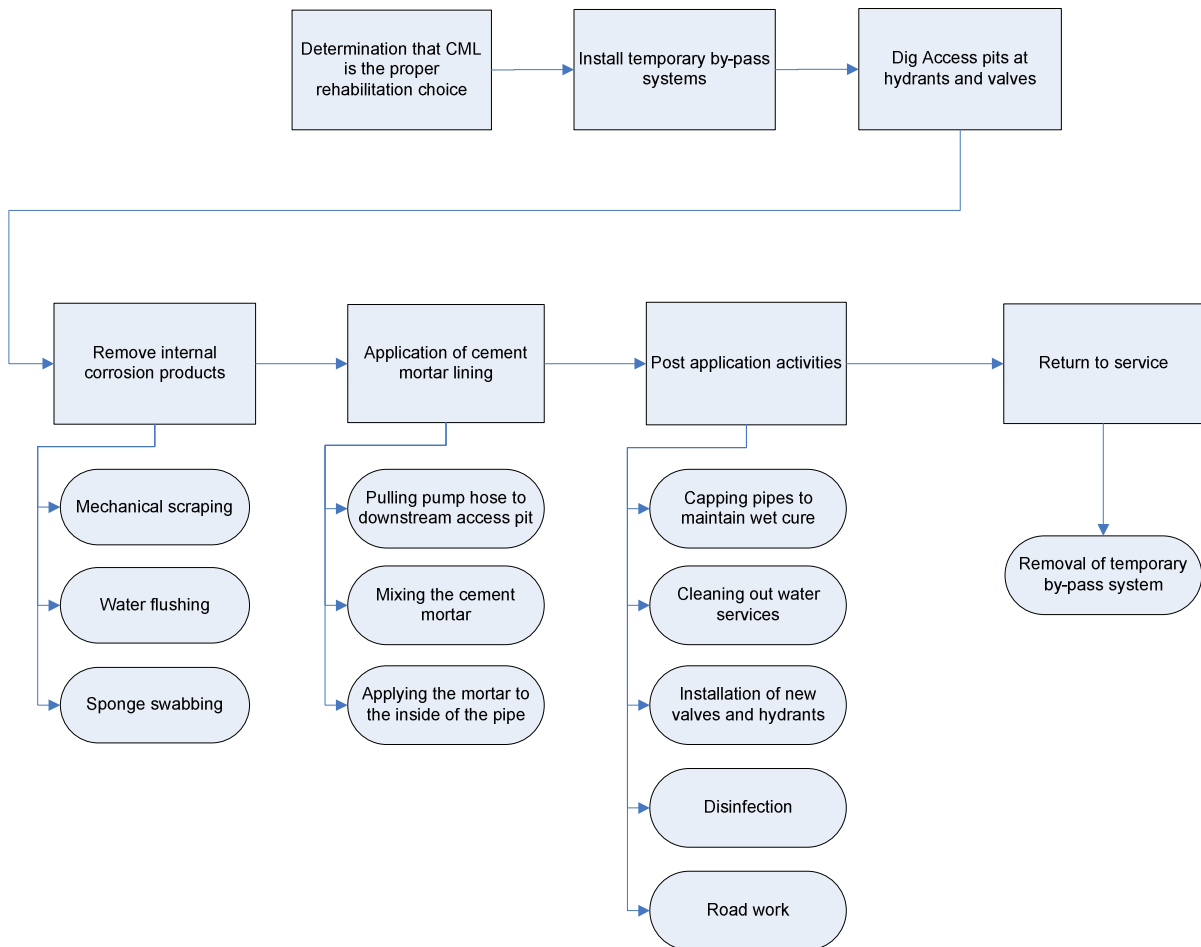


Figure 2-2: Cement mortar lining (CML) process flow chart.

The first task that must be completed during a cement mortar lining project is the installation of temporary by-pass watermains. This aspect is the most costly and time consuming part of the entire cement mortar lining project. The temporary by-pass watermains must be disinfected and meet all drinking water requirements before they can be used to supply water to the residents and businesses effected by the cement mortar lining project. The supply, operation and maintenance of temporary by-pass watermains has been estimated to be 40 to 60 percent of the total cement mortar lining contract cost (Knight, 2006).

Once the temporary by-pass watermains are installed access pits are dug at fire hydrant and or water valve locations. The maximum length of a single lining run is limited by the locations of access pits. An access pit must be dug at each end of the lining run. After the access pits have been dug the valve box or the hydrant is removed so that there is an open access to the watermain to be lined.

When access to the watermain has been obtained the process of cleaning the interior of the watermain begins. This is normally done by mechanical scraping of the interior of the pipe. This mechanical scraping is done using the tool shown in Figure 2-3. The mechanical scraping can also coincided with a process of water-jetting to remove the larger tuberculation build up inside of the pipe. After the mechanical scraping has been completed a hard sponge, referred to as a swab, is pulled or pushed through the pipe to remove any remaining tuberculation products as well as standing water in the pipe.



Figure 2-3: Mechanical scraping tool used to clean internal corrosion products.

Once the pipe has been cleaned and excess water in the pipe has been removed, the process of applying the cement mortar lining begins. The cement mortar is mixed on the pumping rig (Figure 2-4) where water is added to bring the cement mortar to the desired consistency. The pumping tube is pulled to the receptor pit where lining will begin. Cement mortar is pumped to the receptor pit and once it reaches the required consistency the tube with the trowelling device shown in Figure 2-5 is pulled back through the watermain to the pumping rig. This process of pumping mortar into the pipe just ahead of the mechanical trowel which places it on the pipe wall in the required thickness is the core of the cement mortar lining process. Once the main has been lined and has undergone a significant curing process (usually about 24 hrs) water or air is blown backwards from the house connections to remove the lining from the service taps on the watermain. This must be done while the lining is in the initial stages of curing.

Disinfection of the watermain can be commenced as soon as 24 hours after the lining has been applied to the interior of the watermain (Sarrami, 2006). The disinfection procedure

involves flushing the watermain with pressurized water. Therefore, although the watermain is not usually returned to service for a minimum four days, pressure can be applied to the cement mortar lining after a 24 hour cure time.



Figure 2-4: Pumping machine where mortar is mixed and pumped into the watermain.



Figure 2-5: Mechanical trowelling device used to apply mortar to the inside of watermains.

Once the required disinfection procedures have been undertaken and the watermain has passed the stipulated pressure tests it can be restored to service. However, it is common practice for all of the watermains in the construction area to be returned to service at once. This results in very different curing times for different sections of the water distribution network under construction. The watermains which were lined first can wait several weeks to be returned to service while the watermains which were lined last can be returned to service within several days of being lined.

2.3.3 Previous Cement Mortar Lining Tests

Compared to most other commonly used construction technologies there has been very little physical testing done to validate the effectiveness of cement mortar lining. In many cases ‘rules of thumb’ are used to estimate the physical properties based on individual contractors and consultants own personal experiences.

The majority of industry claims regarding the ability of cement mortar linings to stop leaks and bridge corrosion pin-holes are based upon two testing programs performed in 1940; one by the City of Akron, Ohio, and one by the City of Detroit, Michigan. Both of these tests require some revision to assess their applicability to the small diameter pipes commonly lined at the present time.

In 1940 the City of Akron Ohio decided to cement mortar line a substantial amount of large diameter, 914 mm (36”) and 1219 mm (48”), steel watermain (Dorrance, 1947). Akron chose cement mortar lining as a means of “preventing the escape of water when small areas

(of the watermain) became so thin that the (corrosion) scale would not hold the pressure” (Dorrance, 1947). Due to the large diameter of the pipe it can be assumed that flow decreases due to the corrosion scales were not a problem and it is unlikely that water aesthetics were a major concern either. Thus the pipes were only cement mortar lined to prevent water loss in areas where pin-holes resulted from corrosion of the steel watermain. Due to the experimental nature of the Akron cement mortar lining project a series of tests were completed to determine the strength of the cement mortar lining and the ability of the lining to bridge corrosion pin-holes (Dorrance, 1947). The test apparatus was a pressure chamber where water pressure could be applied to a cement mortar lined steel plate with a hole drilled in the plate beneath the cement mortar lining. The holes drilled varied in size from 6.35 mm (¼”) to 31.75 mm (1 ¼”) in diameter (Dorrance, 1947). It was found that “pressure up to 1379 kPa (200 psi)” could be maintained without causing damage to the cement mortar lining. After the pipe was installed it was determined that standard operating pressures of 276 to 345 kPa (40 to 50 psi) could be maintained without any seepage of water through the cement mortar lining (Dorrance, 1947). However, the information available regarding this testing program is lacking in detail. There is no record available with regards to the thickness of the cement mortar lining that was tested or to the range of pressures that the cement mortar lining withstood throughout the testing program.

The testing undertaken by the City of Detroit is the most widely referenced study regarding the physical properties of cement mortar lining. That is not to say that it is referenced in published literature but rather in industry promotional material. The Detroit study had the following stated goals which it sought to determine physical values for (Detroit, 1940):

- The quality of the bond between the lining and the pipe
- The ability of the lining to withstand deflection of the iron host pipe (from both external earth movement loading and internal pressure fluctuations)
- The thickness of the cement mortar lining that is required to prevent blow-outs due to internal water pressure in case of corrosion pin-holing.

One of the primary characteristics of cement mortar lining is the ability of the cement to remain in-place without relying on a bond between the pipe and the cement mortar. The arch-action effect holds the lining firmly against the iron pipe without the need for a bond. Thus, the relative strength of the bond is irrelevant.

Since cement mortar lining is no-longer considered a potential structural lining the ability of the cement mortar lining to withstand pipe deflections is not relevant. Also, the pipes that are currently commonly lined are of much smaller diameter than the large diameter pipes lined and tested by the City of Detroit and are not subject to large degrees of deflection.

The data collected by the City of Detroit regarding to the prevention of water loss due to blow-outs of the cement mortar lining in areas of localized corrosion that have resulted in the formation of pin-holes is relevant to this study. To study this the City of Detroit took an 11.6 metre (38 foot) length of 1219 mm (48") diameter 12.7 mm (1/2") thick steel plate pipe and divided it into five sections with lining thickness of 6.35 mm, 9.53 mm, 12.7 mm, 19.1 mm, and 25.4 mm (1/4", 3/8", 1/2", 3/4" and 1") (Detroit 1940). Holes varying in size from 25.4 mm (1") to 160 mm (6.28") were drilled in the steel pipe prior to lining and plugged so that the

plugs could be removed individually and thus individual pin holes could be pressure tested to failure.

The City of Detroit started the test with only the plug from the 160 mm (6.28") diameter hole in the 25.4 mm (1/2") thick lined section removed. It was found that at normal operating pressures no catastrophic failure or leakage occurred at this hole. At 655 kPa (95 psi) a crack was heard. Upon later inspection a crack was discovered running longitudinally along the pipe. This crack was attributed to a previous loading, performed prior to proper setting of the mortar lining, and resulted in slight leakage occurring from both along the crack and from between the lining and the steel pipe. At 1207 kPa (175 psi) significant leakage began. When the pressure reached 1448 kPa (210 psi) cracking occurred and the pressure was lowered to normal operating pressures. At normal operating pressures the plugs from the 63.5 mm (2 1/2") holes in the 6.35 mm (1/4") and 9.35mm (3/8") thick linings were removed, the 160 mm (6.28") diameter hole was re-plugged and pressure was reapplied. No cracks were found in the 63.5 mm (2 1/2") diameter holes at pressures reaching 1827 kPa (265 psi). The pressure test was repeated on these holes at a later date with a more powerful pump and failure was deemed to occur at 1848 kPa (268 psi) (Detroit, 1940).

Based on these reports the City of Detroit concluded that "a lining of 1/4 inch nominal thickness will provide ample protection for ordinary pressures in new mains subject to mild corrosion, and in reconditioning old mains where it is definitely known that the steel pipe has been only slightly affected. For reconditioning old mains where corrosion is known to be

severe it appears that linings from 9.53 mm ($\frac{3}{8}$ "") to 12.7 mm ($\frac{1}{2}$ "") will present a means of preserving large investments at very nominal cost" (Detroit, 1940).

There are several problems regarding the applicability of the Detroit study to the cement mortar lining work currently being performed in the City of Toronto. Of primary concern is the size of the pipe being lined. The majority of pipe lined in the City of Toronto is smaller diameter residential transmission pipe (typically 254 mm or less). Thus the thickness of the lining in these pipes is commonly in the range of 3 to 5 mm, significantly thinner than that studied by the City of Detroit. The Detroit study also gradually increased the pressure to the maximum failure pressure of 1848 kPa (268 psi). It is common for watermains to be subject to sudden increases in water pressure, the water hammer effect, where a wave of increased pressure is transmitted down the pipe (Crowe and Roberson, 1993)). Thus the loading rates encountered in operation would be much greater than those studied by the City of Detroit. The Detroit study also focussed heavily on large diameter pin-holes. The majority of the study dealt with pin-hole diameters of 160 mm (6.28") and 63.5 mm (2.5"). These are outside of the range of pin-hole diameter that the City of Toronto would consider acceptable for rehabilitation with cement mortar lining. Based on the above concerns it is necessary to establish a pressure testing program which is more applicable to the City of Toronto cement mortar lining program.

2.4 The Coloured Water Problem

Being a non-structural watermain rehabilitation technique, cement mortar lining is primarily required to prevent the 'coloured water' problem. Coloured water is one of the most visible

aesthetic drinking water problems encountered in water distribution systems. Coloured water results from both improperly treated source water that contains iron and the internal corrosion of iron watermains (Clement et al., 2002). Although source water is pointed to as a potential culprit, “the majority of coloured water problems arise as a result of the corrosion of iron-containing pipes combined with the dissolution of corrosion scales, tubercles, present in the interior of the pipes” (Lin et al., 2001).

Discoloured water is most commonly formed when iron enters the bulk water as ferric particles or as ferrous particles which are quickly oxidized to ferric particles (Sarin et al., 2004). During normal system operation the corroded material on the interior of the watermains generally remains intact. However, pressure surges and flow change can cause hydraulic scouring that results in large quantities of ferric and ferrous particles being removed in short time spans resulting in coloured water occurrences at homeowners taps (Sarin et al., 2004). Customers located at the ends of distribution systems generally experience more discoloured water problems because the flow rates are lower and the pipes have a smaller diameter creating conditions where more flow blockage due to corrosion scales can occur (Gummow, 1984).

Currently coloured water is regarded only as an aesthetic water quality problem. However, there is also the potential for more serious water quality issues to arise from the presence of the coloured water causing corrosion scales inside of cast iron watermains. Scales have been linked to high demand for dissolved oxygen and chlorine in the water distribution system, an increase in internal biofilm growth, and the adsorption of toxic chemicals, specifically

arsenic and radium, which can be released from the scales if the water quality (i.e. pH, dissolved oxygen levels) changes (Sarin, et al. 2004).

Due to the water quality issues resulting from the formation of corrosion scales on the inside of iron based watermains it is necessary to create an environment where these corrosion scales do not form. Cement mortar lining is the most commonly used technique for the mitigation of the corrosion scale problem and thus the effectiveness of cement mortar lining requires research. The process of iron corrosion and the means by which cement mortar lining prevents internal watermain corrosion are outlined in Sections 2.5.2 and 2.5.4.

2.5 Iron Watermains

2.5.1 Introduction

Iron is the most common material in the majority of water distribution networks in Canada. A 1995 survey showed that 50% of water distribution pipes in Canada were grey cast iron pipes (Rajani and McDonald, 1995). Although the present day percentage of cast iron pipes is less than 50%, due to the use of plastic pipe materials in most new developments, the amount of cast iron in Canada's water distribution system is still significant.

The first recorded use of cast iron piping was a water supply pipe for the Dillenberg Castle in Germany which was installed in 1455 (Wilson, 1970). However, cast iron was not commonly used in North America until the 1850s when vertical casting was introduced and the quality of the pipes became more consistent (Matheny, 1961, Wilson, 1970). Cast iron

was then the main watermain pipe material used in Canada until the late 1960s (Rajani and Kleiner, 2001).

Iron pipes are comprised of two distinct types of pipe materials: cast iron and ductile iron. Cast iron is a thick walled pipe due to the low strength characteristics of cast iron. Ductile iron is a higher strength material and therefore was designed with decreased wall thicknesses. The use of ductile iron for the construction of watermains in the City of Toronto began in the 1960s (Toronto Water, 2005). Due to the decreased pipe wall thickness of ductile iron, corrosion problems became apparent earlier in the life cycle of ductile iron pipes. For this reason linings were applied to ductile iron pipes at the manufacturing facility. The initial internal linings were asphaltic cement based. Asphaltic cement gave way to factory applied cement mortar linings in the 1960s (Knight, 2006).

The major problem encountered with old iron pipes is the effect of corrosion on the pipes. Internal corrosion can create water quality and flow transmission problems and the combination of external and internal corrosion can create pin-holes in the pipe which can lead to water-loss.

Corrosion can be generally defined as the destruction of a metal as a result of the chemical interaction of the metal and its surrounding environment (Jones, 1996). The corrosion of iron in water is a complex process involving the breakdown of iron into various iron oxide compounds. The iron oxide compound formed is determined by the electrical potential of the system as well as the pH of the solution (Pourbaix, 1973).

The following sections examine the processes by which internal and external corrosion occur, the problems that arise as a result of iron pipe corrosion, and how cement mortar lining protects iron pipes against internal corrosion.

2.5.2 Internal Iron Pipe Corrosion

The corrosion of iron is an electrochemical process that requires the metallic iron to be oxidized to form ferrous and or ferric ions. Electrochemical cells that promote corrosion by electron transfer through an electrical circuit containing an anode, cathode and an electrolyte are the principles behind iron corrosion (Jones, 1996). The key anodic and cathodic reactions in the iron corrosion cell are presented in Table 2-1.

Table 2-1: Key anodic and cathodic reactions involved in the corrosion of iron.

Anodic Reactions	
$\text{Fe} \rightarrow \text{Fe}^{2+} + 2\text{e}^-$	} oxidation of iron
$\text{Fe}^{2+} \rightarrow \text{Fe}^{3+} + \text{e}^-$	
Cathodic Reactions	
$\text{H}_2\text{O} + \frac{1}{2}\text{O}_2 + 2\text{e}^- \rightarrow 2\text{OH}^-$	Reduction of oxygen
$2\text{H}^+ + 2\text{e}^- \rightarrow \text{H}_2$	Hydrogen evolution

Corrosion of the inside of cast iron watermains is mainly caused by bacterial aided under-deposit corrosion. Over time aerobic bacteria will form colonies on the inside of the pipes.

These aerobic bacteria colonies form differential aeration cells which cause the under-deposit corrosion. The formation of the differential aeration cells is a result of the retardation of oxygen transport through the biofilm, as well as, the remaining oxygen being consumed by bacterial metabolism. When the concentration of oxygen is limited under the bacteriological growth the primary cathodic reaction in iron corrosion, the reduction of oxygen, can no longer take place and hence the area under the bacteriological growth becomes anodic to the undisturbed area. As a result of the large size difference between the cathode (uncovered area) and the anode (under-deposit area) the corrosion rate at the anode is greatly increased. Due to this increased corrosion rate the effects of under-deposit corrosion can be catastrophic and can occur much faster than expected (Jones, 1996, Videla, 1996). Figure 2-6 through Figure 2-8 outlines a simplified process of under deposit corrosion.

One of the main problems caused by bacterial aided under-deposit corrosion is the formation of tubercles. These tubercles can aid the pipe in preventing water loss by plugging pinholes developed under the bacteria deposits, however, the problems caused by tuberculation are of much greater concern. The tubercles restrict the flow of water through the pipes which can prevent the necessary fire flows from being achieved (Klein and Rancombe, 1985). The tubercles also tend to be broken off during pressure surges leading to coloured water being delivered to households. The composition of a tubercle is shown in Figure 2-9. The tubercles form as a result of the interaction of bacteria and the under-deposit corrosion products and further aid in limiting the amount of oxygen transported to the metal surface, enhancing the differential aeration cells (Videla, 1996).

2.5.3 External Iron Pipe Corrosion

Cement mortar lining as a pipe rehabilitation method is not designed to prevent external iron pipe corrosion (NRC, 2003). However, it is often claimed that cement mortar lining is a means of preventing water loss through pin-holing of the watermain. Thus, it is necessary to know whether or not external iron pipe corrosion is occurring as most pin-holing is the result of external corrosion (Rajani and Kleiner, 2001). External iron pipe corrosion is generally attributed to the following four basic forms of corrosion illustrated in Figure 2-10 (Klein and Rancombe, 1985, Rajani and Kleiner, 2001):

- Corrosive soils
- Soil conditions which can cause differential aeration cells
- Stray current electrolysis
- Galvanic cells created by dissimilar metals

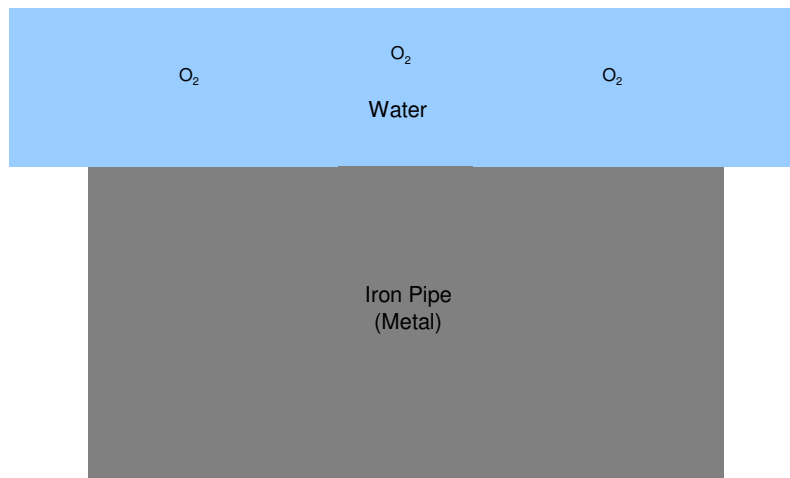


Figure 2-6: Under Deposit Corrosion Process Stage 1. Oxygen content is the same along the pipe. No bacterial growth has occurred. Under-deposit corrosion is not occurring, however, general corrosion is occurring.

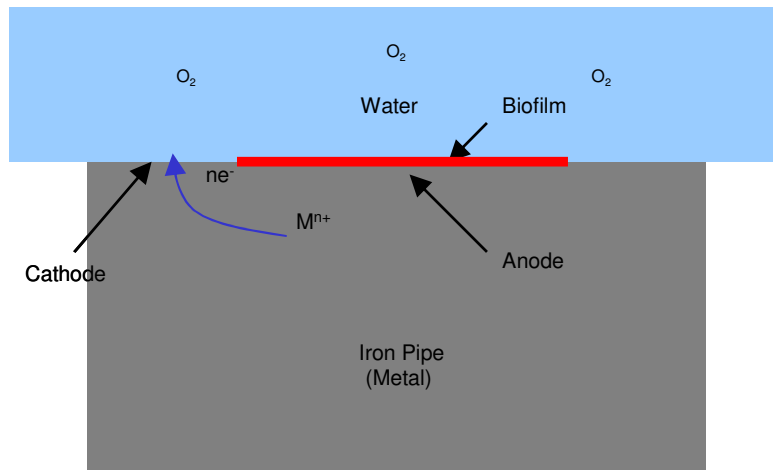


Figure 2-7: Under Deposit Corrosion Process Stage 2: Biofilm of aerobic bacteria forms over portions of the pipe surface. The area under the biofilm becomes deficient in oxygen compared to the surrounding metal. Hence, the area under the biofilm becomes anodic with respect to the uncovered areas. General corrosion is still occurring.

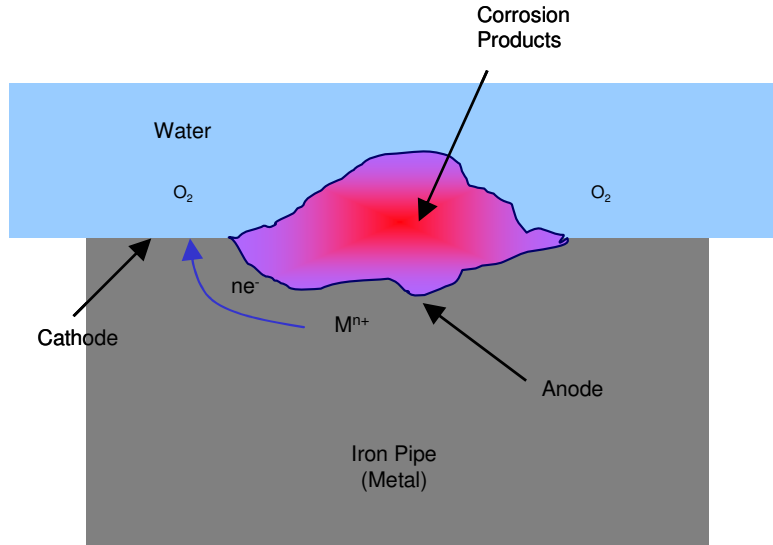


Figure 2-8: Under Deposit Corrosion Process Stage 3: The corrosion products combine with bacteria to create tubercles, further limiting oxygen movement into the area of metal loss. (Under deposit corrosion is an *Autocatalytic* process)

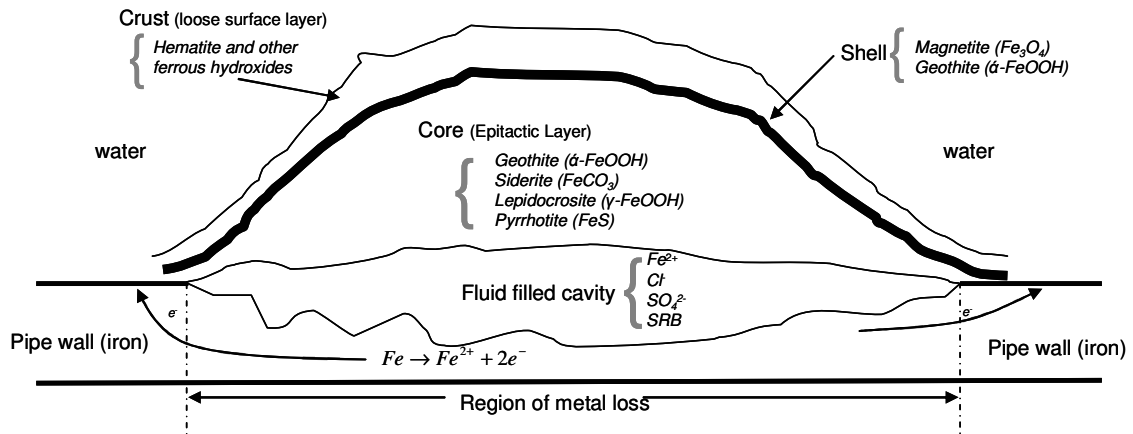


Figure 2-9: Tubercle composition. (Adapted from Knight and Amyot, 2006 and Jones, 1996).

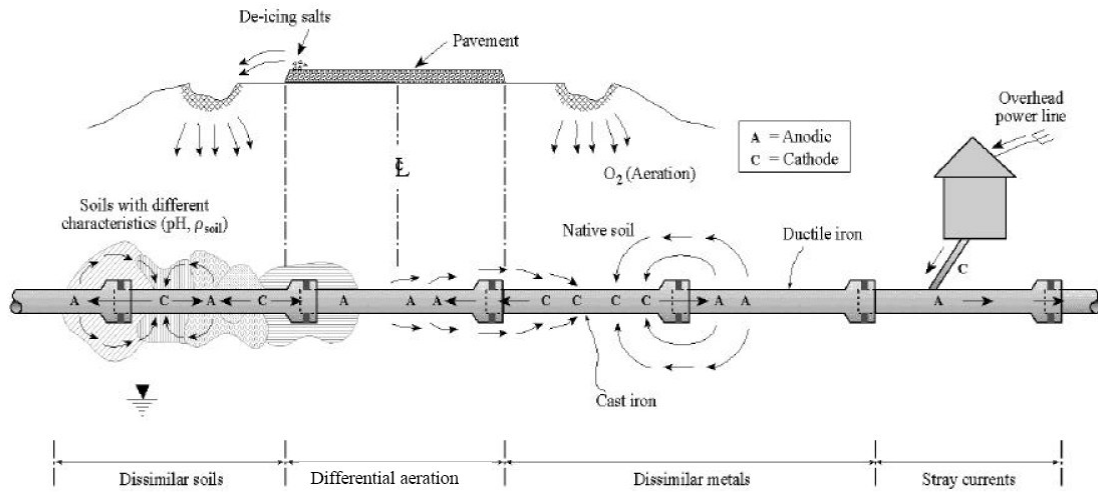


Figure 2-10: Conditions leading to external iron pipe corrosion (Rajani and Kleiner, 2001).

The corrosivity of a given soil is affected by the soil type, pH, electrical resistivity, moisture content, and amount of sulphate reducing bacteria (SRB) in the soil. Clays and silts are often more corrosive than sandy soils as a result of their higher degree of saturation and lower resistivity (Doyle et al., 2003). However, this is not always the case; dry clay can be no more corrosive than dry sand. The City of Toronto is located in an area where the surficial geology is dominated by clayey-silt tills and sand to silty-sand lake deposits (Sharpe et al., 1997). Although these soil types are considered to be inductive to corrosion Doyle et al. found that the correlation between soil resistivity and corrosion was much greater than the correlation between soil type and corrosion in the City of Toronto (2003).

Differential aeration cells similar to those outlined in Section 2.5.2 can form in soils. They are usually the result of differing soil conditions along the pipe (Jones, 1996). The combination of a clayey soil with a poor hydraulic conductivity layered along a pipe with a sandy soil with a high hydraulic conductivity can lead to differences in the amount of dissolved oxygen which reaches the pipe. As a result, the primary cathodic reaction in iron corrosion, the reduction of oxygen, is limited in the clayey soil compared to the sandy soil and a differential aeration cell can form (Jones, 1996).

External iron pipe corrosion can be greatly accelerated by the presence of stray electric currents. Stray currents, as they relate to underground pipe corrosion, are direct currents which flow through the earth to the pipe (Bonds, 1997). These stray currents commonly arise from electrical grounding systems, direct current powered street cars and subway systems, and improperly configured cathodic protection systems (Bonds, 1997). For stray

currents to cause corrosion the stray current must enter the pipe and flow along the pipe for a period of time before discharging from the pipe (Bianchetti, 2001). It is at the point of current discharge that the pin-holes indicative of stray current electrolysis are located (Bonds, 1997). In the case of cast iron pipes these pin-holes may not be readily visible. The graphitisation products which remain after the ferrous material has been leached out of the cast iron pipe block these pin-holes and make them appear as if they are not there (Rajani and Kleiner, 2001, Kuhn, 1930). However, upon mechanical scrapping of the watermain some of the graphitisation products are removed causing the pin-holes to appear.

External pipe corrosion due to galvanic cells created by dissimilar metals is a result of coupling different metallic pipeline materials and coupling metallic water services to pipelines. When two different alloys are coupled in a conductive media one of them is preferentially corroded (Jones, 1996). Table 2-2 shows the galvanic series for common alloys in seawater. The table is ordered from most anodic to most cathodic, thus any metal located below cast iron in the table will be cathodic with respect to cast iron. This will cause the cast iron to preferentially corrode. Although dissimilar metals can easily create galvanic cells they are easily prevented by proper metal combination and insulation. The cast iron pipe anode in the galvanic cell is also usually very large which results in the corrosion being spread out over a larger area which dramatically reduces the occurrence and rate of localized corrosion (Bonds, 1997).

2.5.4 Corrosion Prevention by Cement Mortar Lining

Cement mortar lining is used as a method for rehabilitating watermain suffering from internal corrosion due to its ability to prevent the corrosion from reoccurring over the life span of the original pipe. The following section outlines the process by which cement mortar lining passivates the iron and hence prevents internal corrosion from occurring.

Table 2-2: Galvanic series in seawater (Jones, 1996).

<i>Anodic (active)</i>
↑
magnesium
zinc
aluminium alloys
steel or iron
cast iron
stainless steel (active)
lead
tin
naval brass
nickel (active)
brasses
copper
bronzes
nickel (passive)
stainless steel (passive)
silver
titanium
gold
platinum
↓
<i>cathodic (noble)</i>

The cement mortar used in cement mortar lining is specified to contain one part of either type I or II portland cement to 1-1½ parts clean sand (ASTM, 2000). The sand acts as a structural member of the cement mortar and the portland cement acts as the binder which holds the sand together. The powdered portland cement is composed mainly of calcium silicates and

calcium aluminates which upon the addition of water react to form hydrated calcium silicates, hydrated calcium aluminates and calcium hydroxide (AWWA RF, 1996). When applied the cement mortar is a porous material with the pore spaces being filled with water (Legrand and Leroy, 1990). As a result of this porosity the cement mortar lining does not act as a barrier preventing reaction between the water and the iron surface. Figure 2-11 illustrates the composition of cement mortar as it is applied to an iron surface.

The formation of calcium hydroxide and hydrated calcium silicates and aluminates is the key to maintaining the passivity of the iron pipe. The presence of the Ca^{2+} and OH^- ions causes the pH of the water in the cement mortar pores to be increased to pHs greater than 12 (Legrand and Leroy, 1990, ACI, 1985). As early as 1927 Carson reported that it was the increase in pH that resulted in the corrosion protection properties of cement mortar lining (Carson, 1927). Carson assumed that the dissolved iron present in the transmitted water would precipitate at the outer edge of the cement lining producing a protective barrier which prevented further corrosion (1927). This assumption has since been revised, mainly as a result of increased awareness of the electrochemical properties of corrosion due to the work of Marcel Pourbaix.

Pourbaix developed a series of potential versus pH diagrams which outline the various ranges of pH and potential that phases of metals are stable in an aqueous electrochemical system (Jones, 1996). The Pourbaix diagram for iron in pure water is presented in Figure 2-12. The boundary lines which delineate the differing areas of stability are based in the Nernst equation (2-1) assuming the general equation for a half cell reaction (2-2). Pourbaix diagrams show both the reactions and reaction products that will be present when a given

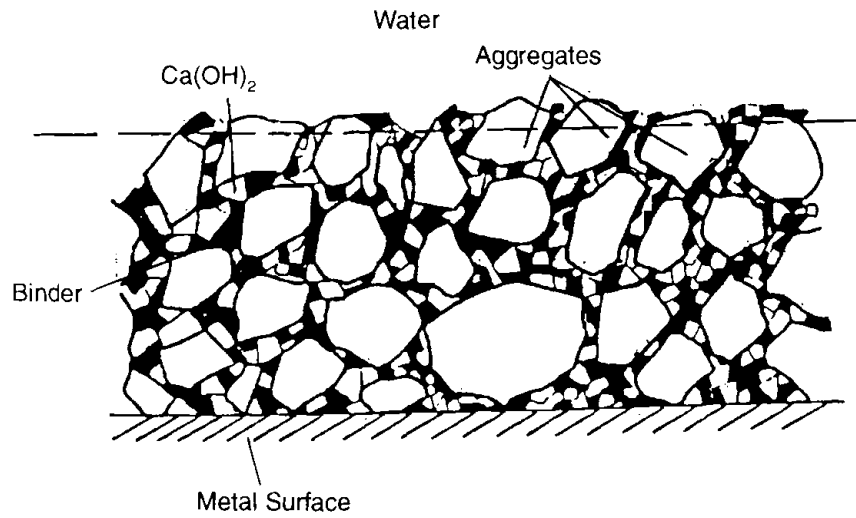


Figure 2-11: Structure of cement based material.

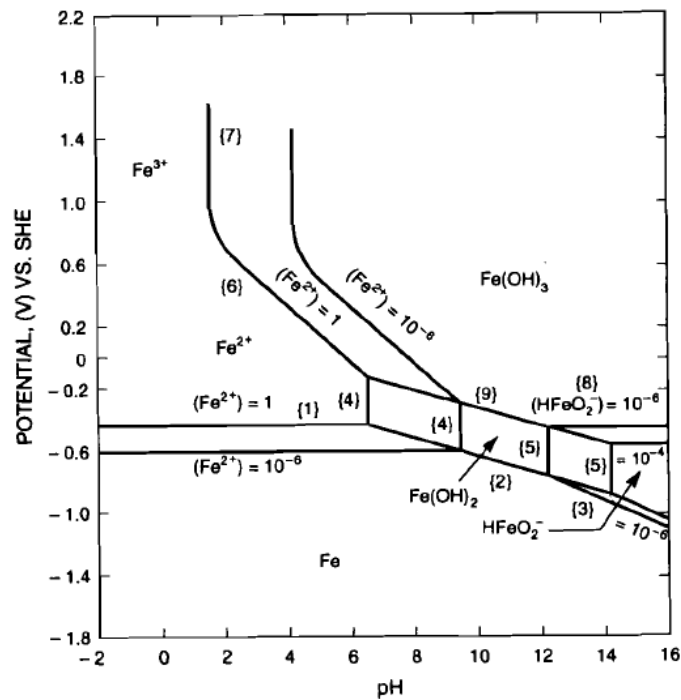


Figure 2-12: Pourbaix diagram for Fe/H₂O system at 25°C.

metal/water system has reached equilibrium (Jones, 1996). Pourbaix presented an extensive collection of Pourbaix diagrams in 1974 (Pourbaix, 1974).

Modified Nernst equation:

$$e = e^{\circ} + \frac{0.059}{n} \log \frac{A^a}{B^b} - \frac{m}{n} 0.059 pH \dots\dots\dots (2-1)$$

General half-cell reaction:

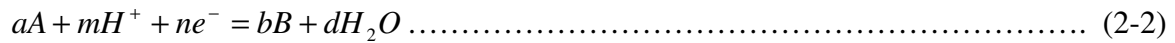


Figure 2-12 represents the theoretical ideal of what happens in a Fe/H₂O system at 25°C. In practice the concentration of other ions in the water supply, specifically chloride and dissolved oxygen, change the equilibrium conditions (AWWA RF, 1995). As a result it is more useful to view the Pourbaix diagram for Fe/H₂O as presented in Figure 2-13. From Figure 2-13 it can be seen that iron in contact with water with a pH in the range present in the pores of cement mortar lining is in the passivation zone.

Pourbaix claims that the iron is passivated in the high pH ranges due to the formation of insoluble iron oxide or hydroxide, predominantly Fe₂O₃ and or Fe₃O₄, which adhere sufficiently to the iron and are impermeable such that they “stifle” the corrosion of the underlying metal (1973).

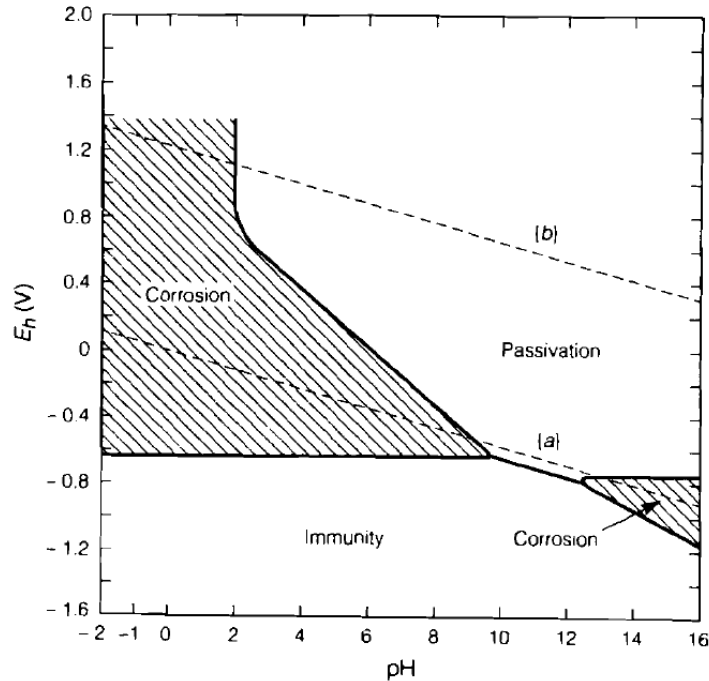


Figure 2-13: Simplified Pourbaix diagram for Fe/H₂O system.

Bloom and Goldenberg attribute passivity to a thin layer of a conductive modified-spinel-structure-magnetite (Fe₃O₄) atop of unoxidized iron (1965). The thin Fe₃O₄ layer is purported to be covered by an electrically insulating layer of γ -Fe₂O₃.

The American Concrete Institute (ACI) describes the protective film which provides passivity as a “thin and tightly adherent oxide film on the metal surface ... of which the exact composition has been difficult to determine (1985).”

Legrand and Leroy claim that the elevated pH prevents corrosion of the iron pipes for the following two reasons (1990):

- Increased pH correlates to lowered cathodic electrode potential.

- The formation of a protective magnetic iron oxide film on the metal wall.

Legrand and Leroy also note that this is a temporary means of protection if the water in the distribution network is aggressive to the calcium hydroxide in the binder.

It should be noted that there is not a specific iron oxide film which has been conclusively proven to be the means by which the state of passivity is induced. The Fe/H₂O system in which the iron oxide formation is tested is also an idealized version of the Fe/H₂O system found in municipal water distribution systems. Thus, other ions such as dissolved oxygen and chloride are likely to affect the nature of the protective iron oxide which is formed.

However, it is reasonable to agree with ACI means of describing the protective film which forms as a result of the increased pH at the cement mortar / iron interface.

3. City of Toronto

3.1 History

The City of Toronto is Canada's largest urban centre. The Greater Toronto Area has a population of 5,304,100 people of whom about half live in the City of Toronto (Statistics Canada, 2006).

Toronto is located in Southern Ontario on the northern shore of Lake Ontario. It was originally founded as a French trading post in 1720 and proceeded to be abandoned and rebuilt by the French until the end of the Seven Years War in 1763 when the French ceded control of Upper Canada to the English (Benn, 2006). However, Toronto was not officially settled by the English until 1787 when the land was purchased from the Mississauga Natives. In 1793 the Capital of Upper Canada was moved to Toronto and in 1834 Toronto was officially classified as a City with a population of 9,250 (Benn, 2006).

In 1843 Toronto's first public water supply company, Furniss Works, was founded. Furniss Works continued to be Toronto's only public water supply company until 1873 when the public water supply was taken over by the City of Toronto (City of Toronto, 2006a).

3.2 Water Distribution System

The current Toronto water distribution system is divided into six operating zones with 12 separate pressure districts. These pressure districts were selected to maintain water pressures between 275 kPa and 793 kPa during normal pumping conditions. On average there are 1,404 mega litres of water consumed in Toronto on a daily basis (City of Toronto, 2006b). Key features of the City of Toronto's distribution network are shown in Figure 3-1.

Toronto's water distribution system contains approximately 5015 kilometres of watermain in addition to 510 km of larger diameter water transmission mains (Toronto Water, 2005). The majority of the older watermains are cast iron pipes (City of Toronto, 2006c, 2000; Toronto Water, 2005). Figure 3-2 and Figure 3-3 show the composition of the City of Toronto's water distribution system by both age and material. From this it can be seen that 41% have been in service for more than 50 years. Figure 3-3 also indicates that 81% are cast iron watermains. Cast iron was the material of choice for new watermain installations until the early 1970s (Rajani and Kleiner, 2001) and the prevalence of cast iron in Toronto's watermain network mirrors the population growth trends in Toronto (Toronto Water, 2005).

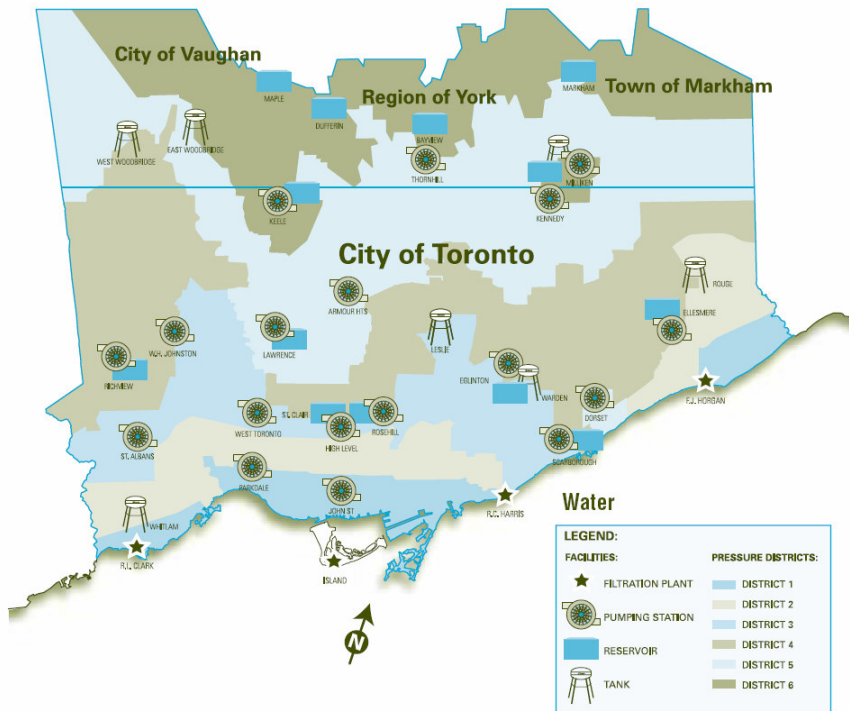


Figure 3-1: Key features of the City of Toronto Water Distribution System (Toronto Water, 2004).

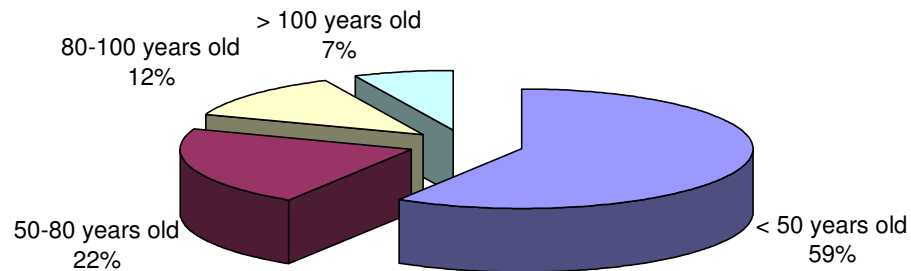


Figure 3-2: City of Toronto watermains by age (adapted from Toronto Water, 2005).

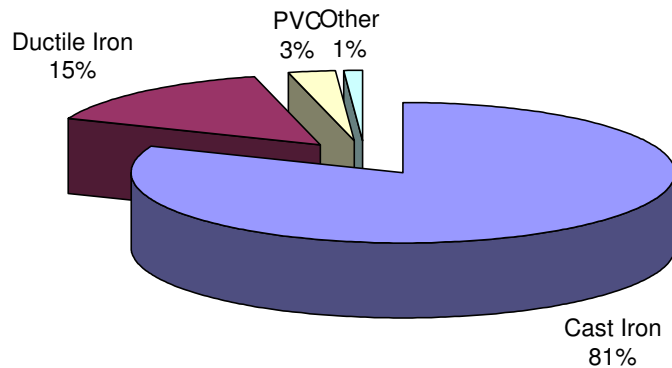


Figure 3-3: City of Toronto watermains by material (adapted from Toronto Water, 2005).

A significant portion of Toronto’s distribution system has lasted longer than the expected 60 to 100 year life expectancy of the individual pipes. As a result of these pipes nearing the end of their service lives they are more likely to suffer breaks. In 2003 Toronto suffered 30.46 breaks per 100 km of water pipe (Toronto Water, 2005). This is significantly higher than the average break rate of 14.9 breaks per 100 km of water pipe reported by the Ontario Municipal CAO’s Benchmarking Initiative (OMBI) municipalities (Toronto Water, 2005). Water loss due to main breaks and leakage leads to increased distribution and water treatments costs and therefore must be minimized. The age and composition of Toronto’s water distribution network can also cause water quality and hydraulic issues due to the build up of corrosion products on the inside of the watermains.

3.3 Cement Mortar Lining Program

Due to the prevalence and age of unlined cast iron watermain in the City of Toronto's water distribution network the City of Toronto has implemented a substantial cement mortar lining program. The City of Toronto rehabilitates an average of 100 km of watermain per year, the majority of which is cement mortar lined (City of Toronto, 2006d).

Toronto has been cement mortar lining smaller diameter watermain since the late 1960s. However, most of this lining has been done without a complete understanding of the physical properties of the cement mortar lining. The City of Toronto has relied upon manufactures' data and previous experience to set their specifications and inspection rates. The results of this thesis investigation are intended to aide the City of Toronto in decisions regarding future cement mortar lining specifications.

The City of Toronto has performed investigations regarding the long-term performance of cement mortar lining. These investigations, a sample of which is presented in Figure 3-4, show that cement mortar lining withstands the build up of internal corrosion products over a significant time period.



Figure 3-4: Cement mortar lined pipes from the City of Toronto. The pipes were lined in 1969 (top) and 1979. The pictures were taken in 1998. They show excellent long-term performance of the cement mortar lining (pictures courtesy of Kamran Sarami).

4. Methodology

4.1 Determination of Parameters to be Investigated

The following sections outline the methodology used to select the parameters investigated in this thesis.

The City of Toronto Water and Waste Water Services department has been utilizing cement mortar lining as a means of rehabilitating corroded iron water mains since the late 1960s.

However, it was felt that the cement mortar lining specifications they use could be improved and or clarified. The City of Toronto was also interested in understanding the ‘rules of thumb’ espoused by the cement mortar lining industry.

Early on it was felt that this study would be of greater value if it investigated parameters of interest to the City of Toronto and the Southern Ontario cement mortar lining industry. To accomplish this, an on-line survey was developed and distributed to the Centre for Advancement of Trenchless Technologies (CATT) membership.

4.1.1 Selection of Survey Parameters

To develop the survey a list of potential cement mortar lining concerns were generated (Table 4-1). The survey topics were generated with the input of Kamran Sarami of the City of Toronto, Ian Doherty of Trenchless Design Engineering, and Dr. Mark Knight of the University of Waterloo; all of whom are experts in the field of watermain rehabilitation.

4.1.2 Selection of Study Parameters

The survey was distributed to the CATT membership which includes municipalities, consultants, contractors, and academics; providing a good cross-section of interests in the field of watermain rehabilitation. The participants were required to rank each of the potential topics, on a scale of one to five with five being the highest ranking, with regards to the need for more information on the given topic. Responses were received from 26 interested parties. Figure 4-1 shows the summarized rankings as a percentage of the highest ranking achieved. This format is used to highlight the topics which received the greatest interest. From Figure 4-1 it can be seen that the corrosion resistance parameters were of the most interest followed closely by two topics related to rehabilitated pipes which have been in service for a number of years: the diameter of lined pipe versus the longevity of lining life and cement analysis of older cement mortar linings. The topic of least interest was pipe structural strength increase due to cement mortar lining.

Table 4-1: Survey topics to be investigated.

Item of Concern
Ability to bridge corrosion pinholes Size of pinhole Length of time the pinhole can be bridged Dynamic vs static pressures
Durability of CML Low Alkalinity High pH Other chemical effects ie; chlorine levels, soft vs hard water
Ability of self repair when cracked/chipped
Percentage of CML coverage required for state of passivation to be effective at preventing pipeline corrosion
Pipe structural strength increase due to CML Blowout prevention Ring deflection
Influence of the components making up the CML on the lifespan Type of sand/sand content Amount of cement/cement type Water chemistry of CML mixture
Are Chlorine residuals increased in CML pipe?
Is the pH temporarily or permanently raised in CML pipe?
What is the optimum CML design thickness?
Consistency of the mix cement analysis of older liners durability of CML vs mix type (test what conditions/original components combine to make for best long lasting pipe)
Is there a correlation between diameter of lined pipe and longevity of life?
How clean does the pipe need to be in order to have good lining application?
C-factor testing before and after - measure improvements

However, when the survey results are looked at as a percentage of the total votes possible, the difference between interests in individual parameters is much less pronounced. Looked at this way, a minimum interest score is 26 and a maximum interest score is 130; all of the potential topics received scores between 77 and 88. This shows that all of the potential topics garnered roughly the same interest level.

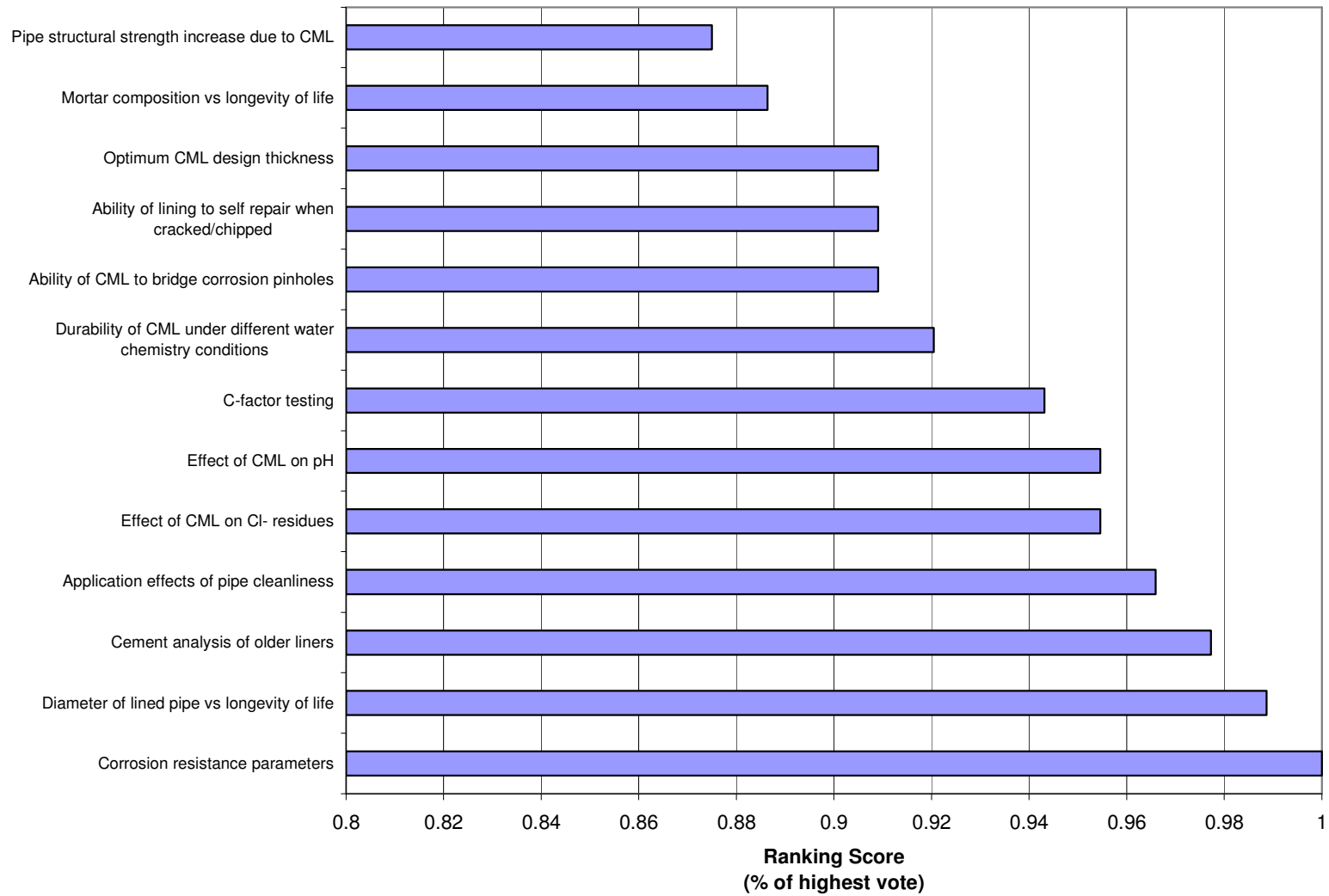


Figure 4-1: Summary of survey results.

As a result of common interest levels it was decided that topics with limited feasibility and topics which have already been significantly investigated would be removed from the study parameters. Thus, topics which involved excavation of old mains were removed due to cost issues. There has also been a significant amount of work done relating to water quality effects of cement mortar lining (see Amyot, 2004, Douglas et al., 1996) and Hazen-Williams coefficient (C-factor) testing (Luk, 2001 presents a summary of results from the City of Toronto).

As a result of this the following topics were chosen as being feasible and of the greatest interest and relevance:

- Ability of the cement mortar to bridge corrosion pin-holes in the pipeline
- The effect of cement mortar thickness and percentage of cement mortar coverage required to prevent internal iron pipe corrosion

4.2 Ability to Bridge Corrosion Pin-holes / Water Loss

Prevention

There are several problems regarding the applicability of the previous studies on water loss prevention. Of primary concern is the size of the pipe being lined. The majority of pipe lined in the City of Toronto is small diameter residential transmission pipe (less than 250 mm). Thus the thickness of the lining in these pipes is commonly in the range of 3 to 5 mm, significantly thinner than that studied previously. The Detroit study also gradually increased the pressure to the maximum failure pressure of 1848 kPa (268 psi). It is common for

watermains to be subject to sudden increases in water pressure, the water hammer effect, where a wave of increased pressure is transmitted down the pipe (Crowe and Roberson, 1993). Thus, the loading rates encountered in operation would be much greater than those studied by the City of Detroit. The Detroit study also focussed heavily on large diameter pin-holes. The majority of the study dealt with pin-hole diameters of 160 mm (6.28”) and 63.5 mm (2.5”). These are outside of the range of pin-hole diameter that the City of Toronto would consider acceptable for rehabilitation with cement mortar lining.

Based on the above concerns it was considered necessary to establish a pressure testing program which was more applicable to the City of Toronto cement mortar lining program. The following sections outline the water loss prevention testing undertaken in this study.

4.2.1 Physical Characteristics of Cement Mortar

To ensure that the cement mortar used in the laboratory was consistent with the cement mortar used in the City of Toronto a testing program was implemented. Cement mortar samples were collected from each cement mortar lining contractor working in the City of Toronto and tested for maximum compressive strength. These values were then compared to the compressive strength values obtained from specimens created in the laboratory.

Compressive strength testing was carried out in accordance to ASTM C109, Standard Test Method for Compressive Strength of Hydraulic Cement Mortars on field and laboratory specimens which were cured longer than 28 days (ASTM, 2002). The compressive strength testing was performed at the University of Waterloo using a Forney QC-50-DR compressive strength testing machine which is shown in Figure 4-2. The strength characteristics of

cement mortar are dependant on the water-cement ratio of the cement mortar, with lower ratios generally providing higher strength (Luk, 2001). During the cement mortar lining process water is added by the contractor with no record of the water content recorded. This made it necessary to collect multiple samples from all of the cement mortar lining contractors working in the City of Toronto in order to ensure that contractor water addition did not have a significant impact on the cement mortar strength characteristics. The on-site addition of water is done in accordance with City of Toronto specifications and AWWA standards (City of Toronto, 2001, AWWA, 2001).



Figure 4-2: Forney QC-50-DR compressive strength testing machine.

Cement mortar samples were also collected for the purpose of obtaining compressive strength versus time information. Fifteen mortar cubes were cast on October 2, 2005 at Dittmer Crescent in Etobicoke. Samples were tested at in accordance with ASTM C109 using a Forney QC-50-DR compressive strength testing machine time intervals of 24, 50, 72,

99 and 120 hours after collection. Three samples were tested at each time interval. The compressive strength versus time testing was performed after the compressive strengths were found for all of the contractors working in the City of Toronto. Based on the previous cement mortar compressive strength testing it was determined that one supply of cement mortar was sufficient for the compressive strength versus time testing.

Table 4-2 shows the locations where samples were collected and the contractor completing the cement mortar lining work.

Table 4-2: Cement mortar sampling locations.

Street	Borough	Contractor	Collection Date	# of Samples
Little Boulevard	York	FerPal	20/6/05	9
Corundum Crescent	Scarborough	Spiniello	11/07/05	9
Lloyd Manor Road	Etobicoke	FerPal	12/07/05	9
Arkona Drive	Scarborough	Spiniello	18/07/05	9
Homewood Avenue	North York	New Tide	25/07/05	9
Tower Drive	Scarborough	Spiniello	18/08/05	9
Dittmer Crescent	Etobicoke	FerPal	3/10/05	15

4.2.2 Water Loss Prevention Testing Program

The following section outlines the physical testing program implemented to determine the ability of cement mortar linings to bridge corrosion pin-holes and hence prevent water loss from rehabilitated watermain.

As outlined in Section 2.3.3 there are several applicability issues regarding previous testing of the ability of cement mortar linings to bridge corrosion pin-holes. Specifically the thickness of the mortar lining tested and the diameter of the pin-hole bridged, as well as, the rate of pressure loading. Thus, the testing program was tailored to provide data regarding these concerns.

4.2.2.1 Test Apparatus

4.2.2.1.1 Pressure Intensifier

The standard operating pressures encountered in the City of Toronto are designed to be between 275 kPa and 793 kPa (City of Toronto, 2006b). However, if a water-hammer effect is induced the pressure can be much greater than this. In Ottawa transient pressures of up to 1035 kPa have been recorded (Zhao et al., 1999). Thus a pressure vessel capable of handling pressures of 1000 kPa was required. A factor of safety of two was incorporated and the pressure testing apparatus was designed to handle pressures of 2000 kPa.

To maintain pressures of 2000 kPa in the pressure vessel a pressure intensifier belonging to the University of Waterloo Department of Earth Sciences was used. The pressure intensifier,

shown in Figure 4-3, uses the University of Waterloo pressurized air supply and converts it to water pressure at a 1:16 ratio. The University of Waterloo maintains a maximum air pressure of 675 kPa in the pressurized air supply. Thus, upon intensification water pressures in excess of 2000 kPa could be easily maintained.

The air supply regulation was performed using a two solenoid valves; an input regulator valve and a bleed valve. The solenoid valves were controlled by a voltage regulator built by the University of Waterloo. The solenoid valves limited the maximum pressure that could be applied to the pressure vessel. The rate at which water pressure was increased within the pressure vessel was dependant on a manual input by the test operator. As a result of this manual operator input the rate of pressure increase could not be easily maintained in all tests and loading rate variation was introduced to the testing scheme.

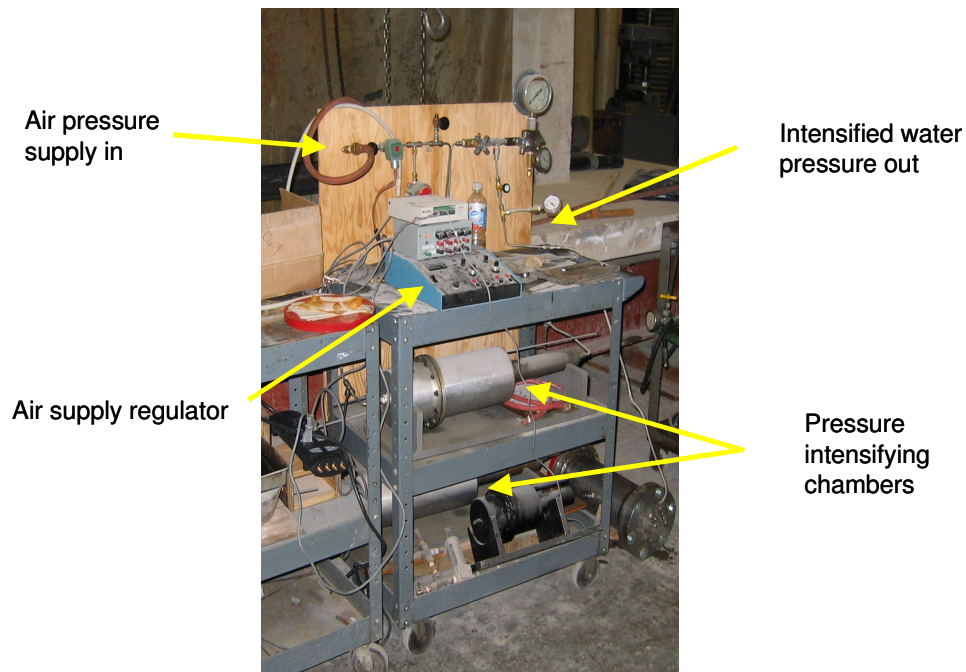


Figure 4-3: Air to water pressure intensifier.

The pressure intensifier also had a fixed volume of water that it could pressurize. If water-loss through the cement mortar lining or through improperly tightened gaskets exceeded the volume of the intensifier then the test had to be stopped so that the pressure intensifier could be re-filled.

4.2.2.1.2 Pressure Vessel Design

The pressure vessel, shown in Figure 4-4, consists of a 30 cm long steel pipe with an internal diameter of 10.795cm. Both ends of the pressure vessel are capped with bolted on 2.54 cm thick steel top and bottom plates which use an E300-70-354 EPDM o-ring to create a seal. The top plate is equipped with a bleed valve and is the location where the water pressure supply is connected to the pressure vessel. The bottom plate has a 7.62cm diameter machined area 8mm in depth where cement mortar is placed over varying diameter pin-holes. Figure 4-5 shows the bottom plate filled with cement mortar. Following completion of repetitive testing the pin-hole diameter was incrementally increased.

A side profile of the bottom plate showing the means in which the cement mortar was applied is shown in Figure 4-6. As illustrated in Figure 4-6 cement mortar was prevented from entering the pin-hole by a wax barrier placed in the pin-hole prior to application of cement mortar. Prior to the application of pressure to the cement mortar the wax was removed from the pin-hole. This was done so that cement mortar would not adhere to the interior of the pin-hole.

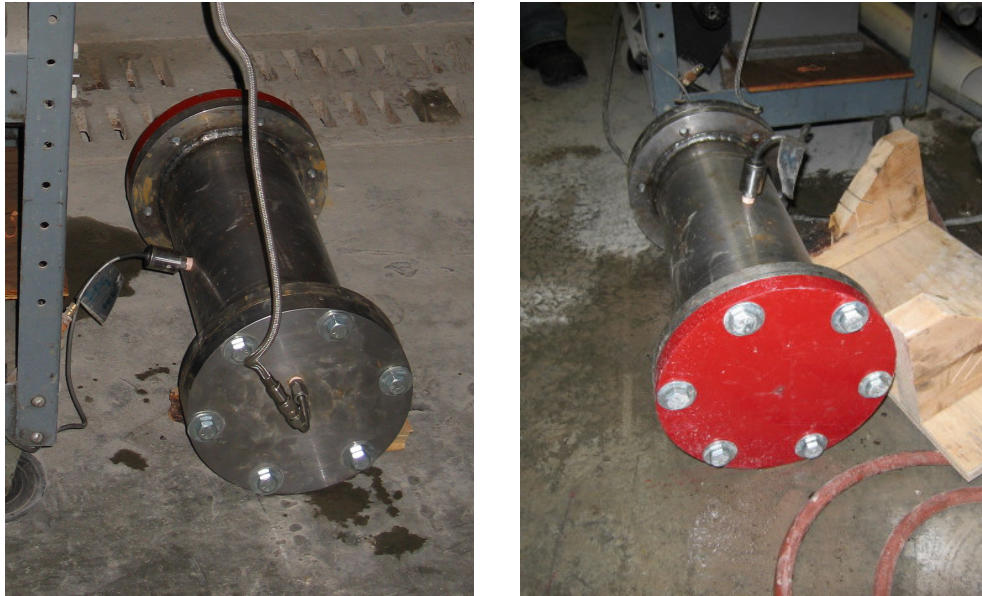


Figure 4-4: Pressure vessel showing top plate (left) and bottom plate (right).



Figure 4-5: Lined post test bottom plate where blow-out has occurred.

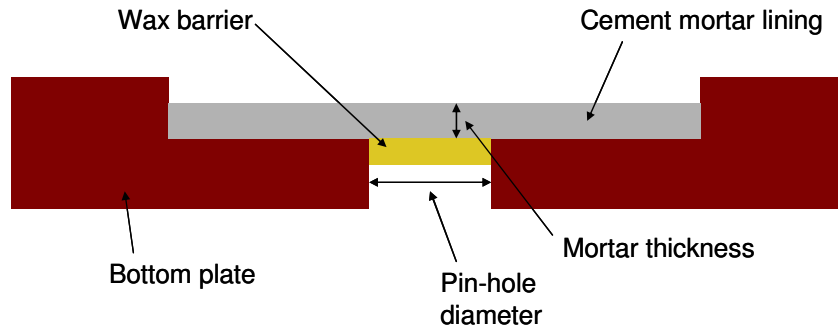


Figure 4-6: Cross sectional view of bottom plate showing cement mortar and wax barrier.

4.2.2.1.3 Data Acquisition System

The data acquisition system used to record the pressures exerted on the cement mortar lining was created using Lab View 7.1. The following two 2070 kPa gauge pressure transducers were used to record the water pressure acting on the cement mortar lining:

- Honeywell Sensotec model FP2000 pressure transducer with a 4-20 mA output and,
- MicroCell model P105 pressure transducer with a 0-0.1 V output.

Calibration data for both pressure transducers is located in Appendix A.

The pressure transducer readings were transferred to the data acquisition card using a National Instruments SCB-68 circuit board. A 12-bit PCMIA National Instruments DAQCard-AI-16-E-4 data acquisition card was used to record pressure data to the notebook computer used for data acquisition.

4.2.2.2 Testing Program

The ability to bridge corrosion pin-holes / water-loss prevention testing program consisted of a series of 82 successful tests. These tests were designed to simulate typical conditions

inside of a cement mortar lined pipe and to cover a range of corrosion pin-hole sizes which are commonly encountered in corroded cast iron watermains.

Pin-hole diameters ranging from 2.39 mm to 25.6 mm were used in combination with cement mortar thicknesses ranging from 1.28 mm to 5.65 mm. The curing time of the cement mortar was also varied to account for the variation in time from lining to re-pressurization of rehabilitated watermains as outlined in Section 2.3.2.

The standard test consisted of slowly increasing the water pressure to a maximum of 2000 kPa and then maintaining that pressure for 20 to 30 minutes. This was then followed by a series of cyclic pressure variations designed to rapidly increase the water pressure in the pipe to simulate the rapid increase from standard operating pressures of approximately 700 kPa to transient water pressures up to 2000 kPa. The cyclic pressure variations were undertaken with the intention of simulating the water hammer effect.

A long-term test was performed to determine the effects of loading the system after an operating pressure was maintained for several hours. The long-term test consisted of an initial base test followed by three pressure surges at intervals of 24 hours. A residual pressure was maintained in the pressure vessel between the pressure surges to determine the pressure that the cement mortar lining maintains when no energy is added to the system. The long-term test was also designed to determine the water loss potential of the cement mortar lining when only residual pressures were applied to the lining.

4.3 Corrosion Prevention

The following section outlines the physical testing program implemented to determine the impact that cement mortar thickness and quality of mortar application have on the ability of cement mortar to prevent internal iron pipe corrosion.

To measure the potential that corrosion was occurring in the cement mortar lined cast iron pipe the corrosion potential technique was used. This method requires the use of relatively simple laboratory techniques and the experimental results are easily interpreted. The corrosion potential technique is also commonly used to map corrosion activity in steel reinforced concrete structures. Thus, an applicable ASTM standard test procedure has been developed (ASTM C876-91).

4.3.1 Cement Mortar Lined Pipe Specimens

Two sections of cement mortar lined pipe were collected from the City of Toronto for corrosion potential testing. The pipe sections were collected seven days after lining had been performed from the Lloyd Manor Road rehabilitation site in Etobicoke. Pipe section #1 was 40 cm in length and pipe section #2 was 105 cm in length. Both pipe sections were 15 cm internal diameter. The lining thickness in both pipes ranged from 0.88mm to 7.22mm.

The lined pipe sections were stored in the University of Waterloo humidity room in optimal humidity conditions from the date of collection, August 10 2005, until August 15 2006 when they were removed and the corrosion testing program was initiated. The lined pipe sections

were stored in the humidity room to ensure that shrinkage cracks did not develop in the cement mortar lining, as well as, to initiate corrosion and ensure that the cement mortar did not dry to the point that it became dielectric.

Figure 4-7 shows pipe section #1 before and after pipe section #1 was placed in the humidity room. The variations in lining diameter as well as the effects of corrosion can be seen in Figure 4-7.



Figure 4-7: Pipe section #1 at collection (left) and prior to testing.

4.3.2 Corrosion Potential Testing Program

The corrosion potential testing program was undertaken in accordance with ASTM C876-91, Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete (ASTM, 1999). This test method was designed to determine the corrosion activity of steel embedded in concrete which has not been dried to the point that it is dielectric. ASTM C876-91 states that the test method is applicable “regardless of the depth of concrete cover”

over the iron being tested (1999). Corrosion potential testing was designed for testing steel reinforcing bars surrounded by concrete but upon investigation and conversation with Dr. Carolyn Hansson, Professor of Corrosion Engineering at the University of Waterloo, it was determined that it is applicable to testing cement mortar lined iron pipes (Hansson, 2005).

The test apparatus consisted of a copper-copper sulphate half cell electrode (CSE), a high impedance voltmeter, an electrical junction device, and an electrical contact solution. The half cell electrode used for testing, shown in Figure 4-8, was a Tinker and Rasor copper-copper sulphate model 2A half cell electrode. The voltmeter used to record the corrosion potentials, shown in Figure 4-9, was a Fluke 87 III true rms multimeter with an end of scale accuracy of $\pm 0.2\%$ at $40M\Omega$. The electrical junction device and contact solution consisted of a sponge wetted with liquid soap diluted with potable water. This sponge was placed on



Figure 4-8: Tinker and Rasor copper-copper sulphate half cell electrode.



Figure 4-9: Fluke 87 III voltmeter.

the surface until the voltmeter reading of the half cell potential was stable for a period of 5 minutes. This was done to ensure that the half cell reading accurately represented the corrosion activity.

Pipe section #1 was used for the corrosion potential testing due to the difficulties inherent in consistently measuring the corrosion potential in the same location in a longer pipe with accessibility issues. The exterior of the pipe was cleaned of corrosion products and a 5cm square grid pattern was drawn on the pipe as shown in Figure 4-10. The corrosion potential measurement locations correspond to the locations directly beneath the grid intersections on the interior of the pipe. The grid was aligned so that measurements would be taken at locations where lining thickness variations occurred.



Figure 4-10: Pipe section #1 showing grid pattern for corrosion potential measurements.

Corrosion potential testing provides information about the probability that corrosion is occurring, not the rate or amount of corrosion that has occurred. ASTM C876-91 results are to be interpreted using the following guidelines (ASTM, 1999):

- if potentials over an area are more positive than -0.20V CSE there is a greater than 90% probability that corrosion is NOT occurring
- if potentials over an area are b/w -0.2 and -0.35 V CSE then the results are ambiguous
- if potentials over an area are more negative than -0.35 V CSE then there is greater

5. Results and Discussion

5.1 Ability to Bridge Corrosion Pin-holes / Water Loss Prevention

The following sections outline the results of the investigation of the ability of cement mortar linings to bridge corrosion pin-holes and to prevent water loss from the watermains. It contains results relating to the physical characteristics of the cement mortar used for cement mortar lining projects and the results of the cement mortar lining pressure tests.

5.1.1 Physical characteristics of Cement Mortar Lining

A testing program was implemented to ensure that the cement mortar used in the laboratory was consistent with the cement mortar used in the City of Toronto. Cement mortar samples were collected from each cement mortar lining contractor working in the City of Toronto and tested for maximum compressive strength. These values were then compared to the

compressive strength values obtained from laboratory mixed specimens. Compressive strength testing was carried out in accordance with ASTM C109, Standard Test Method for Compressive Strength of Hydraulic Cement Mortars on samples cured for time periods longer than 28 days (ASTM, 2002).

The results of compressive strength testing on individual samples are presented in Appendix B and are summarized in Figure 5-1. The mean long-term (>28 day) maximum compressive strength of the field cast mortar cube specimens was 48.7 MPa with a standard deviation of 6.14 MPa. The mean maximum compressive strength of the laboratory cast mortar cube specimens was 50.1 MPa with a standard deviation of 3.71 MPa.

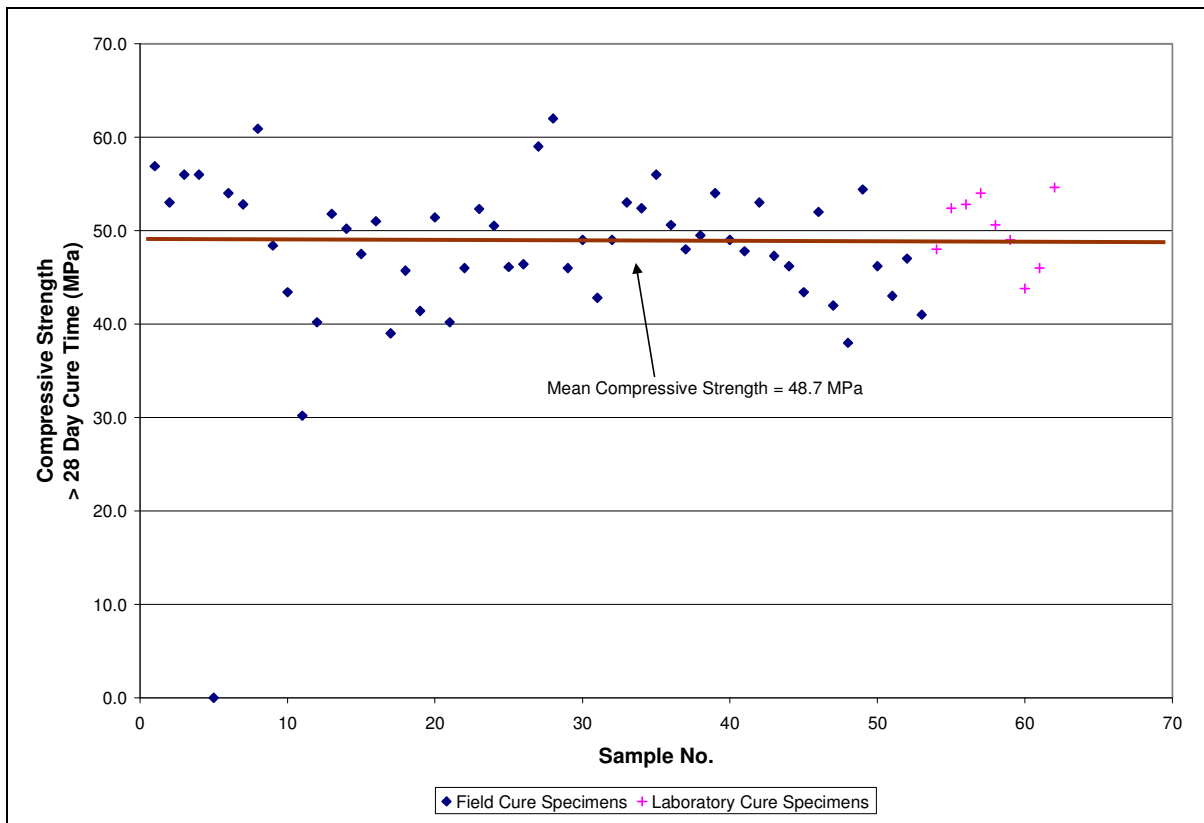


Figure 5-1: Compressive strength of field and laboratory cast mortar cube specimens.

The compressive strength test results presented in Figure 5-1 show a high degree of similarity between laboratory and field cast specimens. As outlined in Section 4.2.1, the strength characteristics of cement mortar are dependant on the water-cement ratio, with lower ratios generally providing higher stress resistance. Thus, it was important that the range of water-cement ratio used in laboratory testing have compressive strength values similar to field cast specimens. Due to the high degree of similarity between the laboratory and field compressive strength values it was determined that the water-cement ratios used in the laboratory replicated field applied cement mortar.

Cement mortar samples were also collected for the purpose of obtaining compressive strength versus time information. Fifteen mortar cubes were cast on October 2, 2005 at Dittmer Crescent in Etobicoke. Three of these samples were tested, in accordance with ASTM C109 using a Forney QC-50-DR compressive strength testing machine, at intervals of 24, 50, 72, 99 and 120 hours after collection. The results of the compressive strength versus time testing are shown in Figure 5-2.

The compressive strength versus time test results, Figure 5-2, show that the cement mortar used for cement mortar lining in the City of Toronto reaches the mean maximum compressive strength in a period of at least four days. The compressive strength reached in a 24 hour time period is significantly less than the average long-term (>28 day) maximum compressive strength. Therefore, pressures that a cement mortar lined watermain can withstand after a 24 hour cure period are expected to be significantly less than pressures that a watermain which has been cured for at least four days can withstand. Based on

compressive strength testing, a newly cement mortar lined pipe should not be returned to service until the cement mortar has cured for a time period of at least four days if it is to have maximum strength. As outlined in Section 2.3.2, pressure can be returned to the watermain during the disinfection process as soon as 24 hours after lining has been completed. The compressive strength testing results indicate that the time period between lining and disinfection should be extended from a minimum period of 24 hours to a period of at least 96 hours if the lining is expected to bridge pin-holes in the watermain.

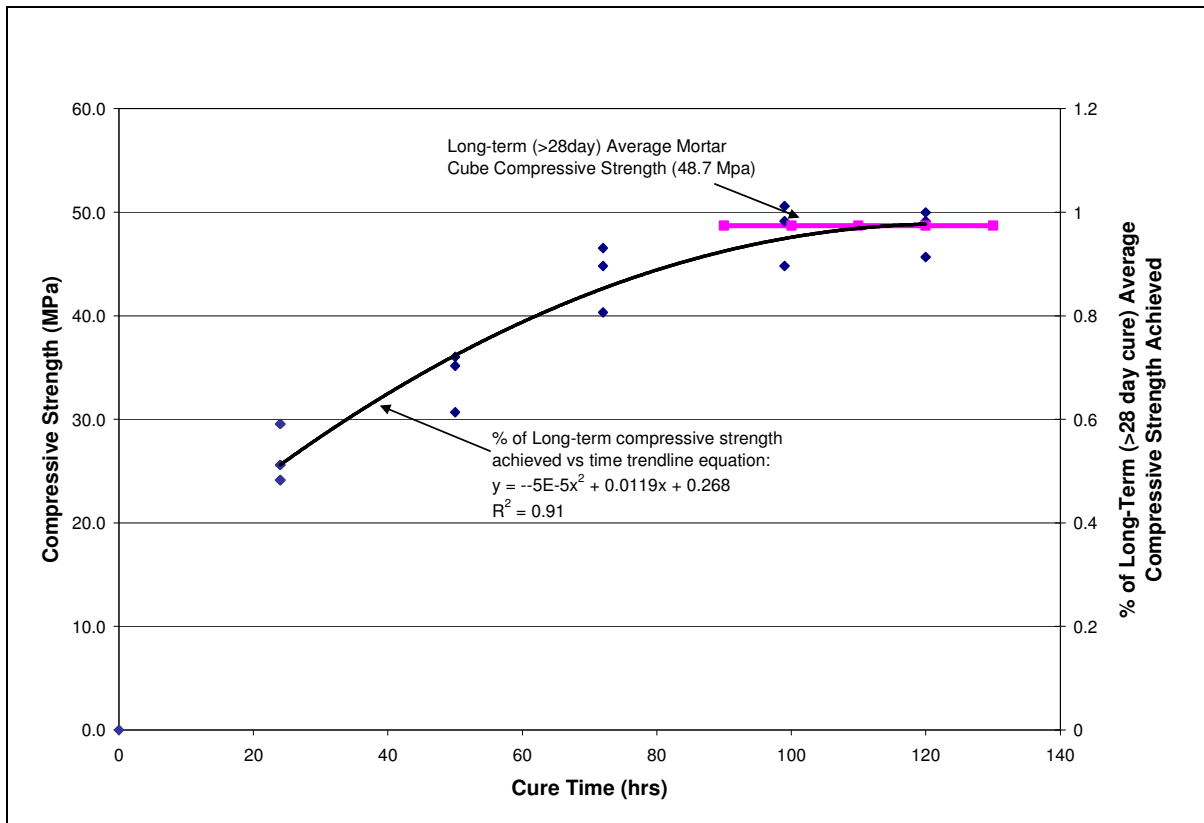


Figure 5-2: Compressive strength versus time for field cast mortar cube specimens.

A parabolic equation was fit to the compressive strength versus time test data to determine the percentage of long-term (>28 day) average compressive strength. Using Excel equation 5-1 was found to have an R² value of 0.91.

$$Cf = -0.00005(t_c)^2 + 0.0119(t_c) + 0.268 \dots\dots\dots(5-1)$$

where: Cf = c% long-term average compressive strength achieved (cure time correction factor)
t_c = cure time in hours

Using this equation, a 24 hour cure period corresponds to 52% of the average (>28 day) compressive strength; a 48 hour cure period corresponds to 72% of the average (>28 day) compressive strength; a 72 hour cure period corresponds to 86% of the average (>28 day) compressive strength; a 120 hour cure period corresponds to 100% of the average (>28 day) compressive strength.

5.1.2 Water Loss Prevention Test Results

The ability to bridge corrosion pin-holes / water-loss prevention testing program consisted of a series of 82 successful tests. These tests were designed to simulate typical conditions inside of a cement mortar lined pipe and to cover a range of corrosion pin-hole sizes which are commonly encountered in corroded iron watermains. Pin-hole diameters ranging from 2.39 mm to 25.6 mm were used in combination with cement mortar thicknesses ranging from 1.28 mm to 5.65 mm. The curing time of the cement mortar was also varied to account for

the variation in time from lining to re-pressurization of rehabilitated water mains as outlined in Section 2.3.2. The tests followed either the standard format or the long-term format, both of which are outlined in Section 4.2.2.2.

For the remainder of this Section failure will be defined as loss of pressure due to catastrophic blow-out of the cement mortar lining covering the corrosion pin-hole. Figure 5-3 contains an example of the cement mortar lined bottom plate after failure had occurred. In all tests where failure occurred all of the mortar covering the pin-hole was removed in a circular pattern after a catastrophic failure of the cement mortar lining.

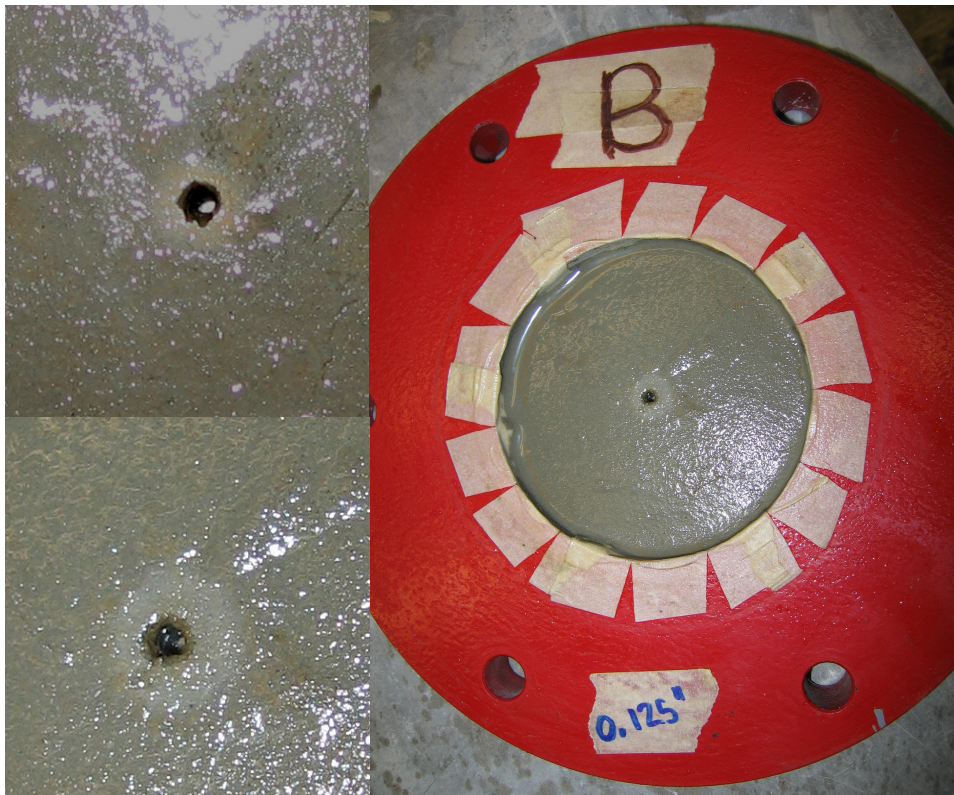


Figure 5-3: Cement mortar lined bottom plate after failure occurred.

Of the 82 successful tests performed, failure of the cement mortar lining occurred in only 27 tests. Appendix C contains graphical representations of all 82 successful tests with accompanying test method descriptions. Summarized test results are presented in Table 5-1 and Table 5-2.

Seven tests were performed using cement mortar that was cured for a time period between 22 hours and 24.5 hours to simulate pressure being returned to the watermain during disinfection as outlined in Section 2.3.2. The results of these short cure tests are shown in Table 5-3.

Failure of the cement mortar lining occurred in four of the seven short cure tests performed. For three of the four short cure tests where failure occurred, the pin-hole diameter/lining thickness combinations which resulted in failure were retested using a longer cure time. Retesting found that the lining did not fail when the cure period was at least 48 hours. The results of the repeat tests are shown in red in Table 5-3.

One short cure test was performed where a retest using the same pin-hole diameter/lining thickness combination was not performed. This test was the 3.18 mm diameter pin-hole with a normalized thickness of 0.72 shown in Table 5-3. However, seven day testing was performed on two samples of 12% larger diameter with lining thickness not exceeding 0.01mm greater than the 3.18 mm diameter sample. In both of these seven day tests failure did not occur, indicating that the failure was not repeatable for longer-cure specimens. The seven day test results are located in Table 5-1.

Table 5-1: Water loss prevention test results, 2.39-8.38mm pin-hole diameter.

Diameter (mm)	Cure Time (hrs)	Thickness (mm)	Blow out	Normalized Thickness (mm/mm)	Maximum Applied Pressure / Pressure at Failure (kPa)
2.39	96	1.9	No	0.80	2000
	96	1.85	No	0.77	>2000
3.18	24.5	2.29	Yes	0.72	1960
	144	2.54	No	0.80	2000
	144	3.11	No	0.98	2000
3.56	22	2.3	No	0.65	2000
	144	2.3	No	0.65	2000
	144	2.16	No	0.61	2000
3.96	23	2.63	No	0.66	2000
	47.5	1.3	Yes	0.33	1496
	144	3.32	No	0.84	2000
	144	1.48	No	0.37	2000
4.80	75	3.08	No	0.64	2000
	1200	2.32	No	0.48	2000
5.00	75	3.16	No	0.63	2000
5.18	24	1.28	Yes	0.25	1887
	24.5	3.09	No	0.60	2000
	43.5	3.53	No	0.68	1925
	96	1.74	No	0.34	1925
	1200	2.68	No	0.52	1925
6.35	45.5	1.66	Yes	0.26	1925
	96	1.8	No	0.28	1925
	1200	2.74	No	0.43	1925
6.60	24	1.77	Yes	0.27	862
	41.5	1.58	Yes	0.24	1925
	46.5	3.79	No	0.57	1925
	96	2.02	No	0.31	1925
	120	2.18	No	0.33	2000
	1500	2.1	No	0.32	2000
7.47	24	2.5	Yes	0.33	1795
	41.5	1.97	No	0.26	1925
	46	3.14	No	0.42	1925
	67.5	1.98	No	0.27	2000
	96	1.65	Yes	0.22	1925
	1500	1.4	No	0.19	2000
8.38	43.5	2.21	No	0.26	1925
	45	1.55	Yes	0.18	2000
	47.5	1.81	Yes	0.22	1670
	120	1.67	Yes	0.20	1900
	120	1.51	Yes	0.18	1690
	120	2.31	No	0.28	2000
	1500	1.56	No	0.19	2000

Table 5-2: Water loss prevention test results, 9.91-25.6mm pin-hole diameter.

Diameter (mm)	Cure Time (hrs)	Thickness (mm)	Blow out	Normalized Thickness (mm/mm)	Maximum Applied Pressure / Pressure at Failure (kPa)
9.91	44	2.22	Yes	0.22	2000
	44.5	2.38	No	0.24	2000
	46	2.58	No	0.26	2000
	120	1.95	Yes	0.20	1635
	120	2.32	Yes	0.23	2000
	144	2.64	No	0.27	2000
11.2	44.5	2.15	Yes	0.19	1705
	47	3.22	No	0.29	2000
	120	2.02	Yes	0.18	2000
	120	2.57	No	0.23	2000
12.8	43	4.87	No	0.38	1925
	44	2.47	Yes	0.19	2000
	46.5	3.57	No	0.28	2000
	46.5	2.13	Yes	0.17	1740
	120	2.05	Yes	0.16	1076
	120	2.87	Yes	0.22	2000
	120	3.67	No	0.29	2000
	144	3.5	No	0.27	2000
	144	3.3	No	0.26	2000
	168	2.44	Yes	0.19	2050
	168	2.82	No	0.22	2000
	168	2.8	No	0.22	2000
1500	3.09	No	0.24	2000	
15.0	47	3.37	Yes	0.22	2000
	144	4.05	No	0.27	2000
	144	4.19	No	0.28	2000
	168	3.53	No	0.24	2000
	168	3.03	No	0.20	2000
	168	2.91	No	0.19	2000
	168	3.36	No	0.22	2000
19.2	47	3.8	Yes	0.20	1640
	144	4.55	Yes	0.24	2000
	144	4.24	No	0.22	2000
	168	4.97	No	0.26	2000
	168	4.1	No	0.21	2000
	168	4.15	No	0.22	2000
	168	5.2	No	0.27	2000
25.6	168	3.77	Yes	0.15	1565
	168	3.94	Yes	0.15	1975
	168	5.65	No	0.22	2000
	168	5.2	No	0.20	2000

The inability to repeat the short cure failures in specimens which had undergone curing periods of at least 48 hours showed that cement mortar lining specimens that are cured for a period of 24.5 hours or less are significantly more likely to fail than specimens which receive an adequate amount of time to properly cure. Short cure tests were limited to pin-hole diameters of 7.47mm and less where inducing a failure with fully cured specimens was less likely.

Table 5-3: Water loss prevention results, short cure specimens including >48 hour retest results.

Diameter (mm)	Cure Time (hrs)	Thickness (mm)	Blow out	Normalized Thickness (mm/mm)	Maximum Applied Pressure / Pressure at Failure (kPa)
3.18	24.5	2.29	Yes	0.72	1960
3.56	22	2.3	No	0.65	2000
3.96	23	2.63	No	0.66	2000
5.18	24	1.28	Yes	0.25	1887
	24.5	3.09	No	0.60	2000
	96	1.74	No	0.34	1925
6.60	24	1.77	Yes	0.27	862
	1500	2.1	No	0.32	2000
7.47	24	2.5	Yes	0.33	1795
	67.5	1.98	No	0.27	2000

To determine the distribution of cement mortar lining failure events the normalized thickness parameter was introduced. Normalized thickness is defined as follows:

$$\text{Normalized Thickness } (nT) = \frac{\text{lining thickness (mm)}}{\text{pin-hole diameter (mm)}} \dots\dots\dots(5-2)$$

Figure 5-4 shows all failure data presented as normalized thickness versus pin-hole diameter.

Figure 5-4 presents the failure data without taking into account the strength variations caused

by cure time. From Figure 5-4 it can be seen that as pin-hole diameter was increased the normalized thickness at failure decreased. The normalized thickness at failure sample mean was 0.234 with a standard deviation of 0.108. In Figure 5-4 the data point which has a normalized thickness of 0.72 corresponds to the 24.5 hour cure time test with a 3.18 mm diameter and lining thickness of 2.29 mm. Failure occurred in this test at a normalized thickness 218% greater than any other failure. Table 5-4 represents the test parameter combinations that were similar to the failed test with a normalized thickness of 0.72. From Table 5-4 it can be seen that failure did not occur in any test parameter combinations that were similar to the failed test. This indicates that the failure at a normalized thickness of 0.72 was an outlier.

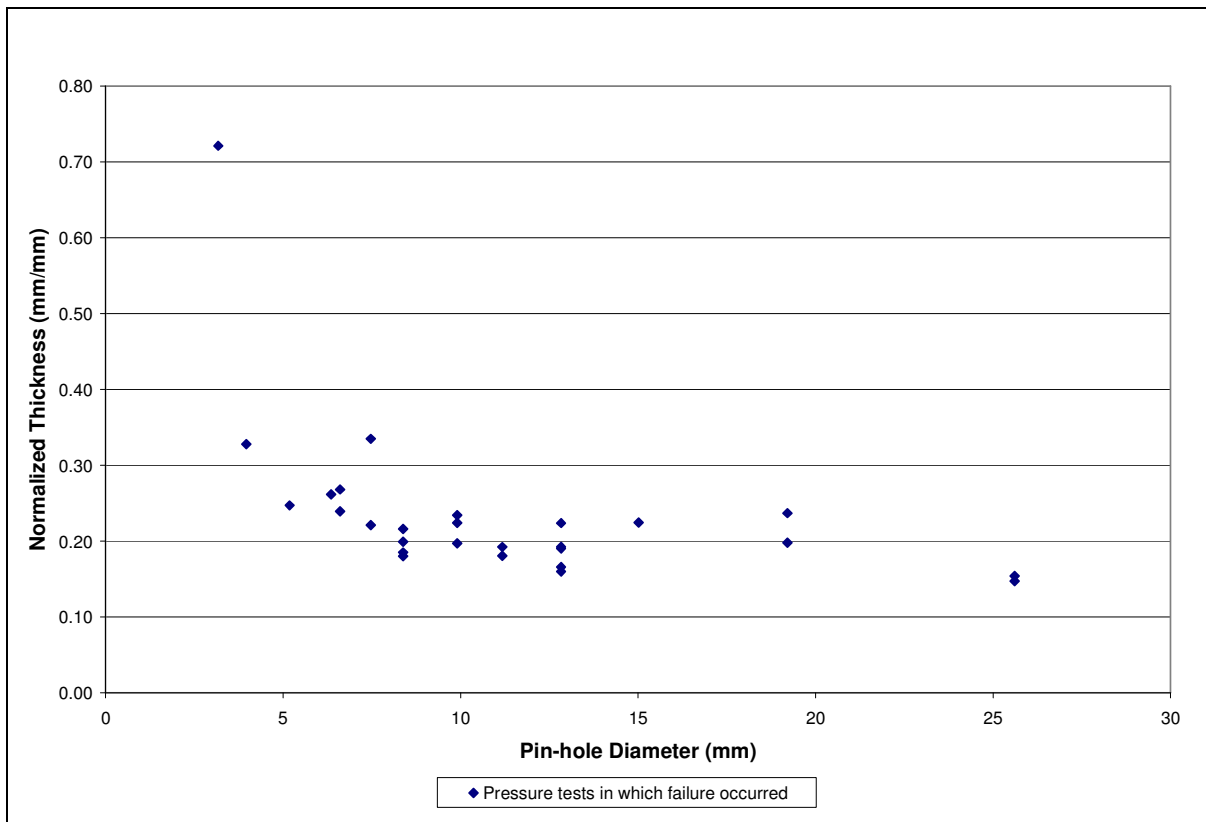


Figure 5-4: Water loss prevention failure data; normalized thickness at failure vs. pin-hole diameter.

Table 5-4: Tests with similar parameters to high normalized thickness failure sample.

Diameter (mm)	Cure Time (hrs)	Thickness (mm)	Blow out	Normalized Thickness (mm/mm)	Maximum Applied Pressure / Pressure at Failure (kPa)
2.39	96	1.9	No	0.80	2000
	96	1.85	No	0.77	>2000
3.18	24.5	2.29	Yes	0.72	1960
	144	2.54	No	0.80	2000
3.56	22	2.3	No	0.65	2000
	144	2.3	No	0.65	2000
	144	2.16	No	0.61	2000
3.96	23	2.63	No	0.66	2000

The data presented in Figure 5-4 does not account for the variation in cement mortar strength due to differing cure times. Therefore, it was necessary to correct normalized thickness values to account for cement mortar strength variations due to curing time. Results from the mortar cube compressive strength versus curing time testing, presented in Section 5.1.1, were used as the basis for this correction factor.

Figure 5-2 in Section 5.1.1 was used as the basis for equation 5-1 which can be used to determine the percentage of long-term (>28 day) average compressive strength achieved after a designated time period:

$$Cf = -0.00005(t_c)^2 + 0.0119(t_c) + 0.268 \dots\dots\dots(5-1)$$

The cure time corrected (CTC) normalized thickness was calculated as follows:

$$CTC \text{ normalized thickness} = (\text{normalized thickness}) * Cf \dots\dots\dots(5-3)$$

Equation 5-2 is valid for curing times between 24 hours and 96 hours. As outlined in Section 5.1.1, once the cement mortar has cured for a period of at least four days it reaches the long-term (>28 day) average compressive strength and a correction factor is no longer required.

Figure 5-5 represents the cure time corrected normalized thickness. The cure time corrected normalized thickness at failure sample mean was 0.177 with a standard deviation of 0.054. From Figure 5-5 it can be seen that once the failure data has been corrected to account for strength variations due to differing cure times, the cure time corrected normalized thickness no longer increases as the pin-hole diameter decreases.

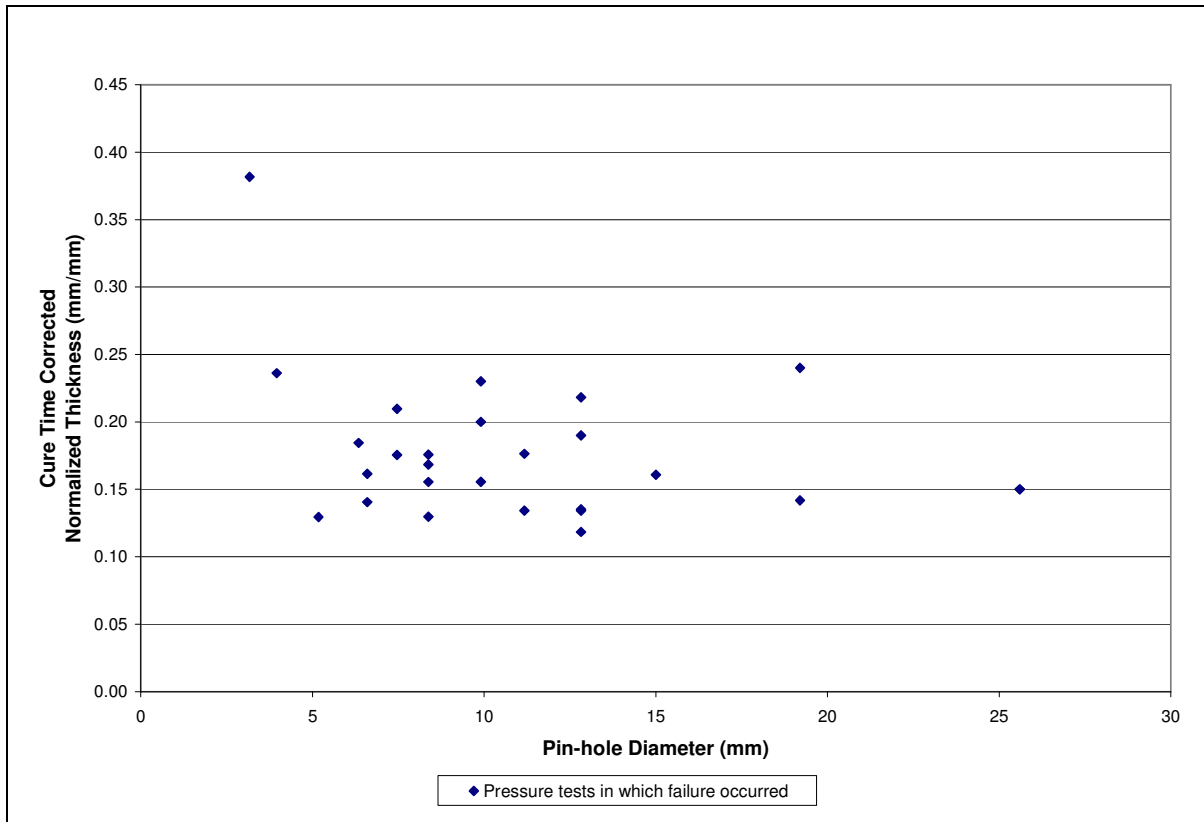


Figure 5-5: Water loss prevention failure data; cure time corrected normalized thickness at failure vs. pin-hole diameter.

To compile a more accurate view of the cure time corrected normalized thickness at failure a distribution fitting was performed. Probability paper plots were created for Normal, Exponential, Weibull, and Gumbel distributions. The probability paper plots are located in Appendix D. The Gumbel distribution was found to be the best fit distribution with an R^2 value of 0.9879. Both the Gumbel and Exponential distributions provided high R^2 values, however, for design purposes the upper bound of the cure time corrected normalized thickness was of the most interest. The Gumbel distribution was used because it is an extreme value distribution designed to focus on the upper bound of the data (Pandey, 2004). Figure 5-6 shows the Gumbel probability paper plot and Figure 5-7 shows the probability density function for the cure time corrected normalized thickness at failure. From the Gumbel distribution probability paper plot, Figure 5-6, the following distribution parameters were found:

- a location parameter (α) of 0.1525
- a scale parameter (β) of 0.0333

From the above distribution parameters the following mean and standard deviation were obtained:

- a distribution mean (μ) of 0.172
- a distribution standard deviation (σ) of 0.043

To determine the size of pin-hole that can be bridged by a set mortar thickness or the thickness of mortar required to bridge a set pin-hole size it is necessary to analyse the upper bound of the probability density function illustrated in Figure 5-7. The Gumbel probability density function illustrated in Figure 5-7 is based on the following equation:

$$f(x) = \frac{1}{\beta} e^{-\left(\frac{x-\alpha}{\beta}\right)} e^{-e^{-\left(\frac{x-\alpha}{\beta}\right)}} dx \dots\dots\dots (5-4)$$

where:

- the location parameter (α) is 0.1525
- the scale parameter (β) is 0.0333

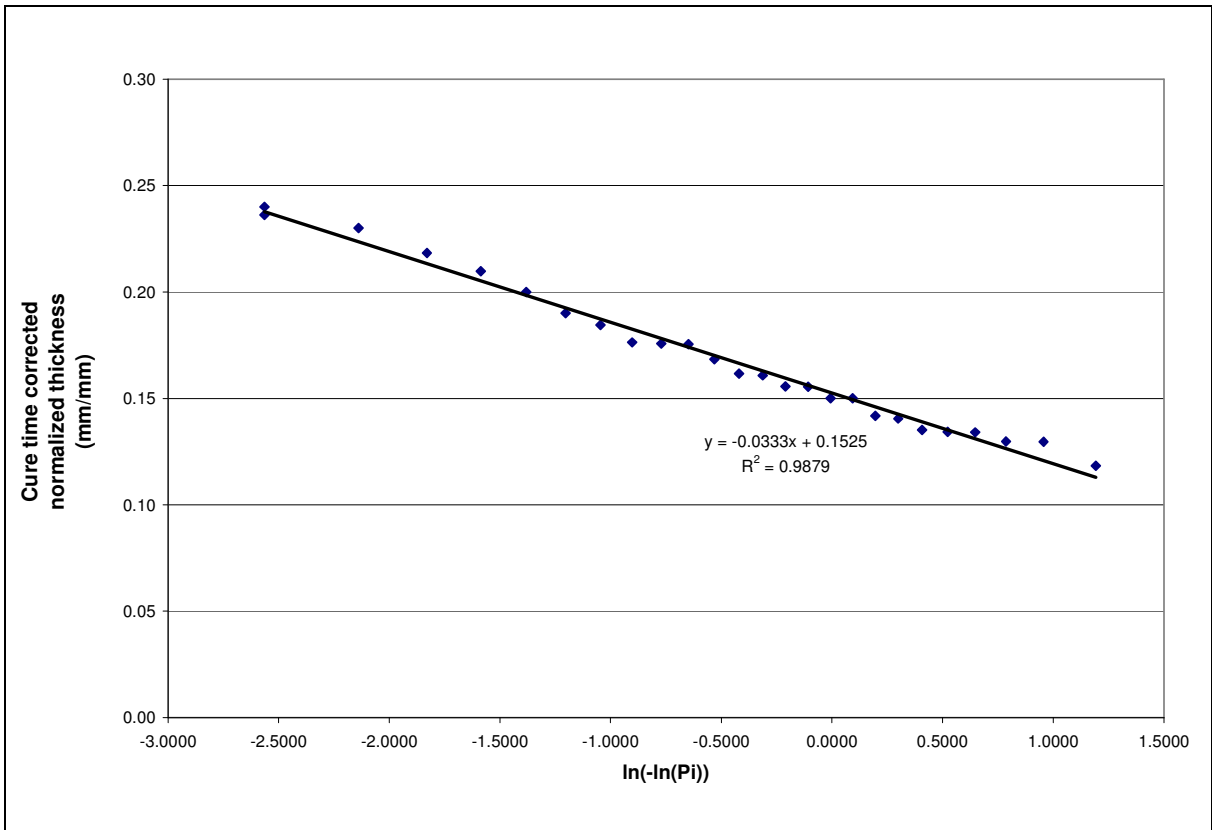


Figure 5-6: Gumbel distribution probability paper plot.

The total sum of the area under any probability density function curve is one. Thus the area under the curve that corresponds to 95% confidence level that failure will not occur can be found by integrating the area under the curve according to the following formula:

$$0.95 = \int_0^b \frac{1}{\beta} e^{-\left(\frac{x-\alpha}{\beta}\right)} e^{-e^{-\left(\frac{x-\alpha}{\beta}\right)}} dx \dots\dots\dots (5-5)$$

By solving this integral for b a value of $b = 0.251$ was obtained. This value corresponds to the cure time corrected normalized thickness at which there is a 95% chance that failure will not occur. Solving for the 99% probability that failure will not occur provides a value of $b = 0.304$

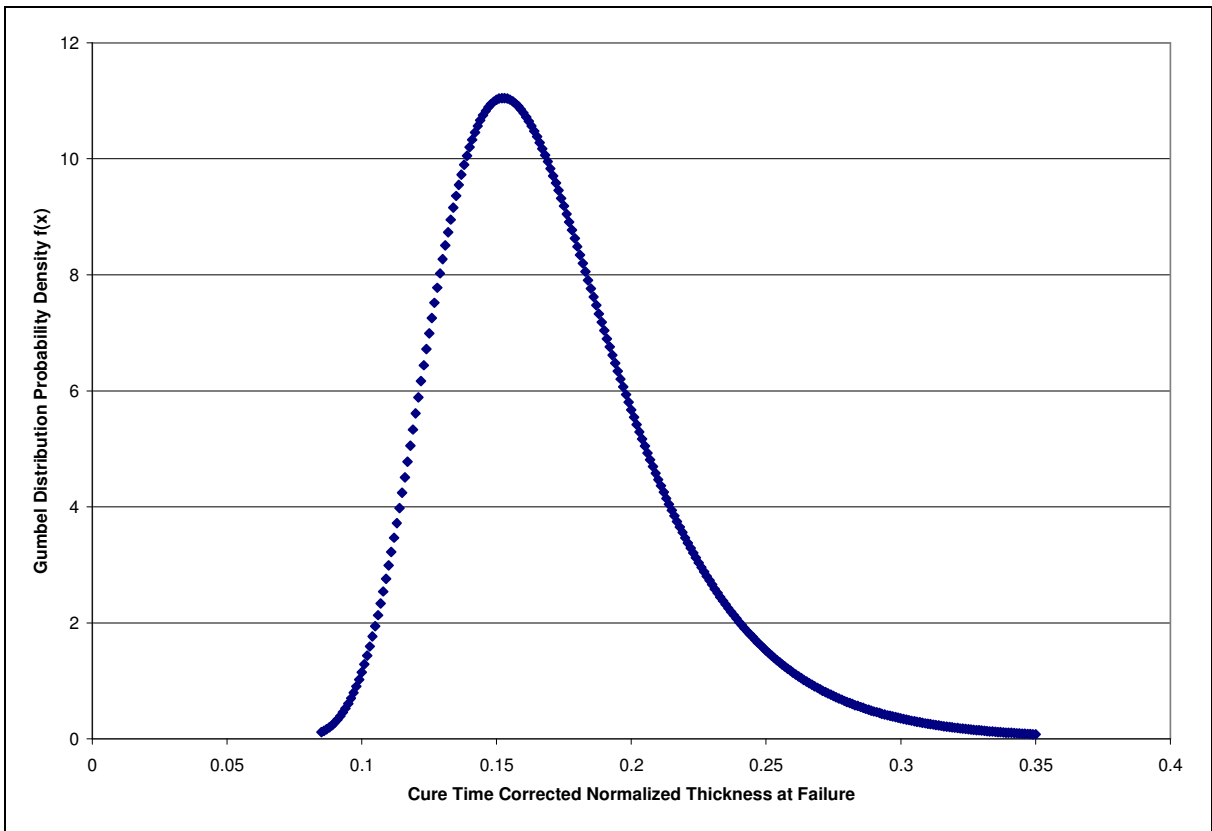


Figure 5-7: Gumbel probability density function of cure time corrected normalized thickness at failure.

Cure time corrected normalized thickness values corresponding to the 95% and 99% probability that failure will not occur can be used to aide the cement mortar lining design process. Assuming that pressure will not be applied to the main until the cement mortar lining has been cured for a period of at least four days Equation 5-6 can be used to determine the thickness required to bridge corrosion pin-holes with a set degree of confidence that failure will not occur.

$$\text{Required Lining Thickness} = (\text{Pin-hole Diameter}) * b \dots\dots\dots(5-6)$$

Where b is the cure time corrected normalized thickness corresponding to the chosen probability that failure will not occur.

The following example is used to illustrate how the cure time corrected normalized thickness can aide the design process. If a pipeline in need of rehabilitation is known to have corrosion pin-holes with an upper bound of 20mm, then a lining can be designed to have a 95% probability that failure will not occur by using the cure time corrected normalized thickness value of $b = 0.251$. Using Equation 5-6 a required lining thickness of 5.02mm was obtained.

Thus this laboratory testing program has developed a means that watermain rehabilitation planners can use to tailor the cement mortar lining thickness required to a specific project. However, it should be noted that the results presented do not prove that a failure will not occur at a given normalized thickness, rather, they provide a probability that failure will not occur. Site specific conditions such as pressure surges, graphitization remaining after pipe

cleaning as well as variations in cement mortar characteristics due to contractor added water content will affect the probability that failure will not occur. The results of this testing program are based on short term testing, thus, the long-term fatigue properties of the cement mortar lining are not known. Thus it is recommended that an appropriate factor of safety be added to the cement mortar lining thickness by the watermain rehabilitation planner.

5.1.2.1 Cement Mortar Shear Strength Analysis

The shear strength at failure of the cement mortar lining was analysed to determine if the cement mortar lining shear strength was consistent through out the testing program. The shear strength of the mortar cubes was analysed using the Mohr-Coulomb failure criterion and the shear strength of the cement mortar lining was analysed using a force-balance method which assumed a vertical shear failure of the lining.

Figure 5-8 outlines the system used to perform the force balance analysis. The force balance method for determining the shear strength of the cement mortar lining at failure was based upon the assumption that the summation of forces acting in the vertical (y) direction was equal to zero at equilibrium.

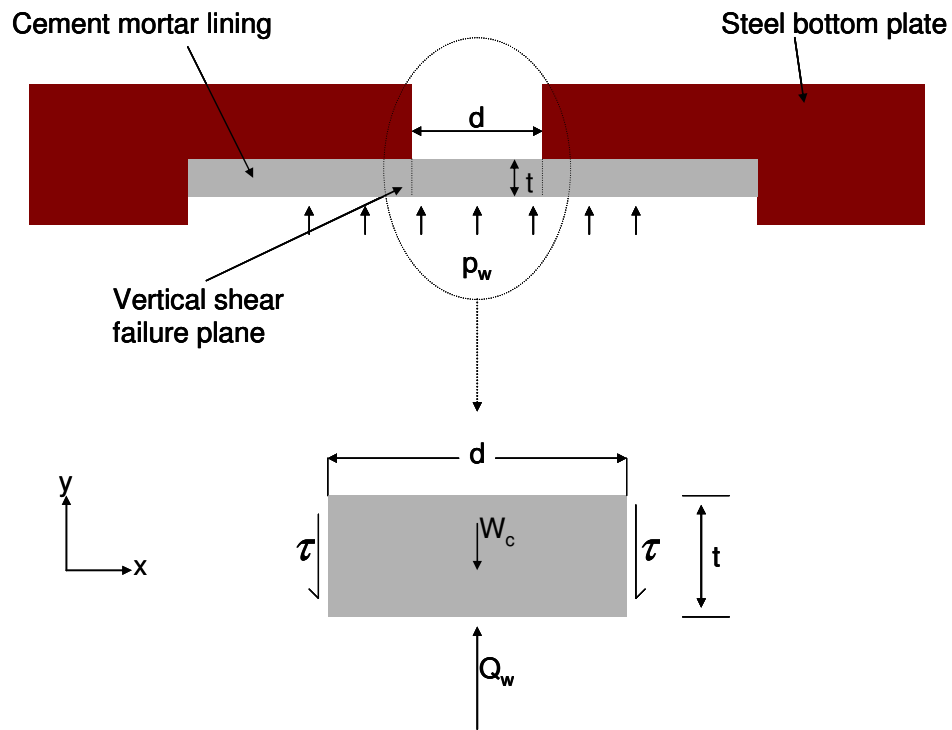


Figure 5-8: System used for force balance analysis of cement mortar lining shear strength.

The symbols used in Figure 5-8 are defined as follows:

- p_w = the water pressure inside of the pressure vessel at failure
- t = the thickness of the cement mortar lining
- d = the pin-hole diameter
- W_c = the weight of the cement mortar section
- Q_w = the force the water acting on the cement mortar section
- τ = the shear strength of the cement mortar (T = the shear force)

Based on the system outlined in Figure 5-8 the force balance in the y direction was performed as follows:

$$\sum F_y = 0 = Q_w - W_c - T \dots\dots\dots(5-7)$$

$$Q_w = W_c + T \dots\dots\dots(5-8)$$

$$p_w \frac{\pi d^2}{4} = \gamma_c \frac{\pi d^2}{4} t + \tau \pi d t \dots\dots\dots(5-9)$$

$$\therefore \tau = p_w \frac{d}{4t} \dots\dots\dots(5-10)$$

Since the weight of the cement mortar was negligible with respect to the pressure applied to the cement mortar lining it was removed from the final calculation of shear strength.

Equation 5-8 was then used to determine the maximum shear stress in each test that failure occurred.

Figure 5-9 presents the cement mortar shear strength at failure plotted against the cure time of the specimen. Figure 5-9 shows that the shear strength of the cement mortar increased as the cure time increased. This agrees with the trend of increased compressive strength with increased cure time shown in Section 4.2.1. The average shear strength at failure calculated using the force balance method was 2.10 MPa with a standard deviation of 0.59 MPa.

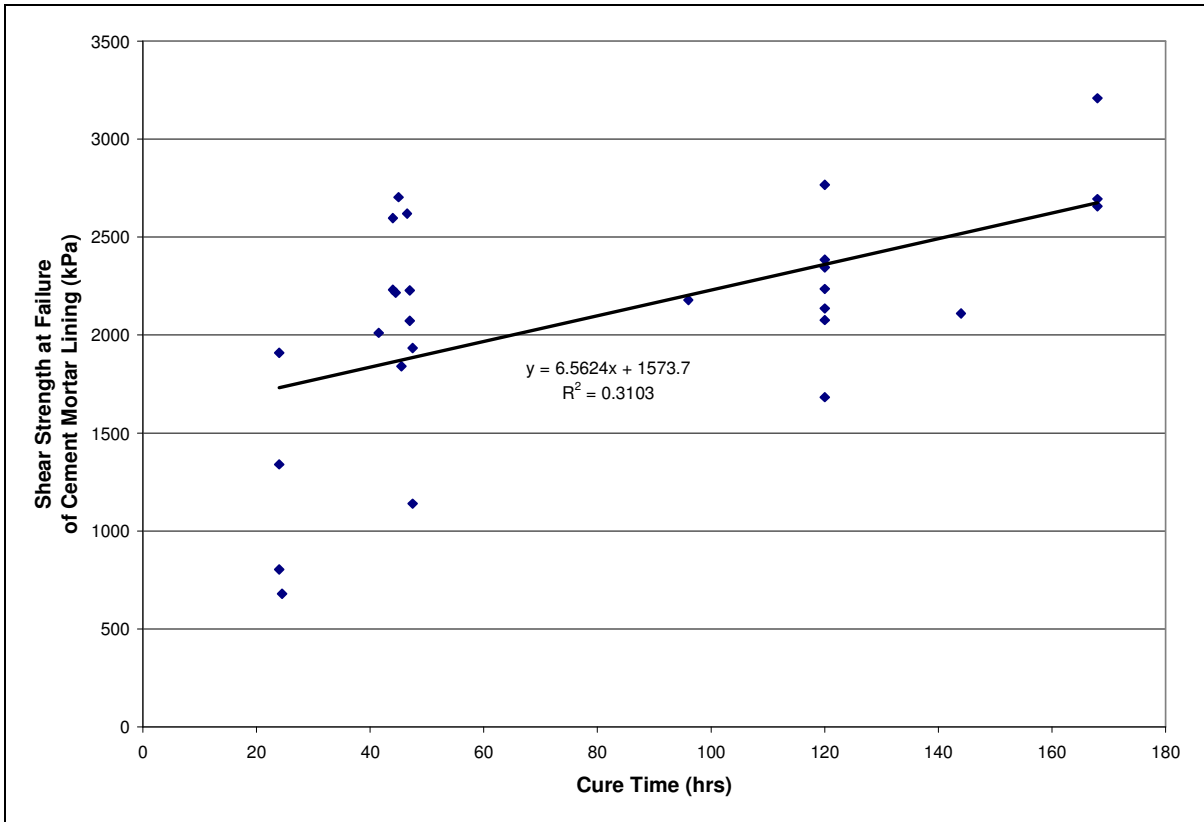


Figure 5-9: Shear strength at failure of cement mortar lining versus cure time.

Shear strength of the cement mortar was also calculated using the mortar cube unconfined compressive strength test data presented in Section 5.1.1. The shear strength of the mortar cubes was calculated using the Mohr-Coulomb failure criterion (equation 5-9) which is illustrated in Figure 5-10 :

$$\tau_f = c + \sigma_n \tan \phi \dots\dots\dots(5-9)$$

where: τ_f = shear stress at failure

c = cohesion

σ_n = normal stress on the failure plane

ϕ = angle of internal friction

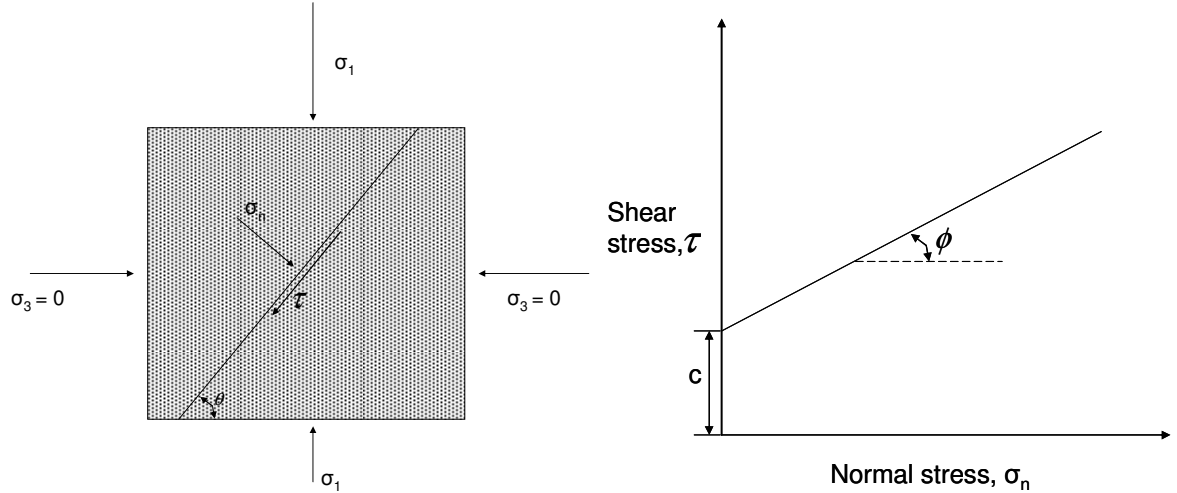


Figure 5-10: Mohr-Coulomb failure criterion.

To determine the shear strength at failure the following assumption was made:

- $\Phi = 45^\circ$

Using the assumed angle of internal friction ($\Phi = 45^\circ$) and the mean long-term (>28 day) maximum compressive strength of the field cast mortar cube specimens outlined in Figure 5-1 in Section 5.1.1 ($\sigma_1 = 48.7$ MPa) the Mohr-Coulomb failure criterion was analysed.

Figure 5-11 presents the results of the Mohr-Coulomb failure analysis. Figure 5-11 illustrates that the shear strength of the cement mortar at failure (τ_f) was 17.2 MPa.

The average shear strength at failure of the cement mortar lining obtained using the force balance method was 2.10 MPa. The shear strength at failure of the field cast cement mortar cubes obtained using the Mohr-Coulomb failure criteria was 17.2 MPa. The idealized vertical shear failure is a reasonable method for determining the design shear strength of the

mortar for pin-hole blow-out applications. The discrepancy between these two results is likely due to the application of the Mohr-Coulomb failure criterion to this application.

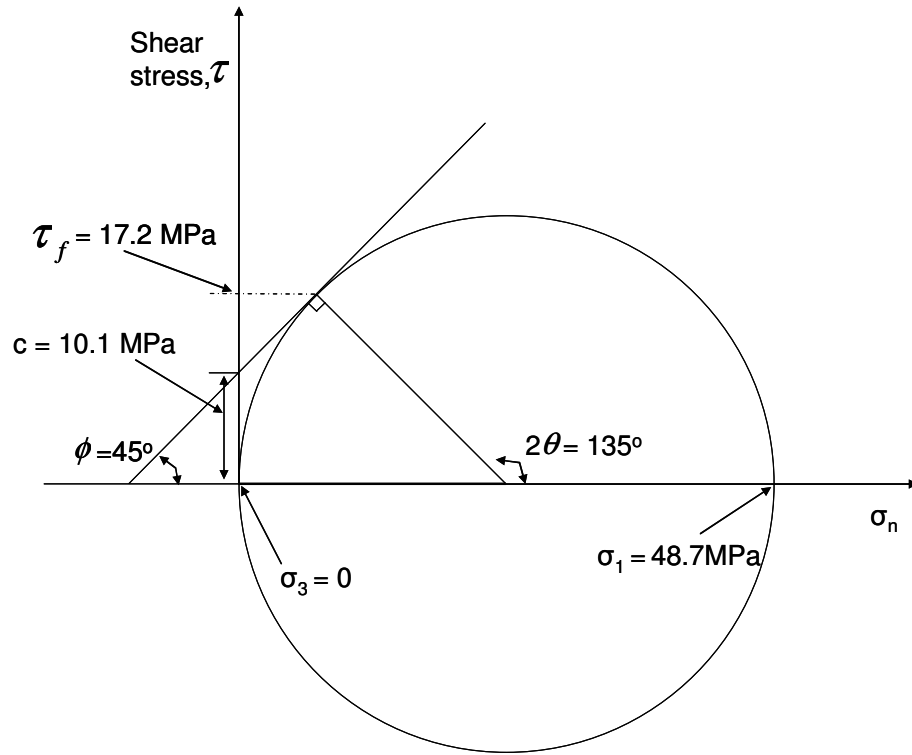


Figure 5-11: Mohr-Coulomb failure criterion analysis for field cast cement mortar cubes.

5.2 Corrosion Prevention

The following section outlines the results of the physical testing program implemented to determine the effects that cement mortar thickness and quality of mortar application have on the ability of cement mortar to prevent internal iron pipe corrosion.

To measure the magnitude of the corrosion occurring in the cement mortar lined cast iron pipe the corrosion potential technique was used. The corrosion potential testing program was undertaken in accordance with ASTM C876-91, Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete (ASTM, 1999).

ASTM C876-91 results may be interpreted using the following guidelines (ASTM, 1999):

- if potentials over an area are more positive than -0.20V CSE there is a greater than 90% probability that corrosion is NOT occurring
- if potentials over an area are b/w -0.2 and -0.35 V CSE then the results are ambiguous
- if potentials over an area are more negative than -0.35 V CSE then there is greater than a 90% chance that corrosion is occurring

5.2.1 Corrosion Prevention Testing Results

Pipe section #1 was used for the corrosion potential testing due to the difficulties inherent in consistently measuring the corrosion potential in the same location in a longer pipe with accessibility issues. The exterior of the pipe was cleaned of corrosion products and a 5cm grid pattern was drawn on the pipe as shown in Figure 4-10. The corrosion potential measurement locations correspond to the locations directly beneath the grid intersections on the interior of the pipe. The grid was aligned so that measurements would be taken at locations where lining thickness variations occurred.

The cement mortar lining thickness was measured at the north end and south end of the pipe (the pipe is shown in Figure 4-10 in Section 4.3.2). The convention for reporting cement

mortar thickness as well as corrosion potential measurements is outlined in Figure 5-12.

Measurements were recorded at 5 centimetre intervals around the circumference of the pipe.

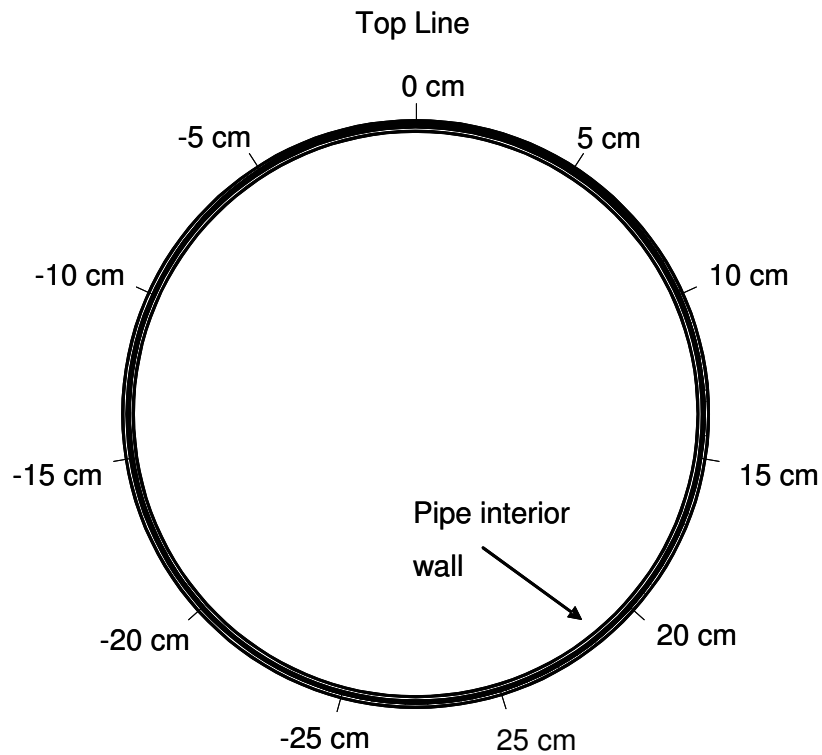


Figure 5-12: Pipe section #1 measurement reading convention. View from north end of pipe.

Figure 5-13 illustrates the distribution of thickness measurements at both the north and south end of pipe section #1. Figure 5-13 shows that 1/3 of the thickness measurements were in the range of 2-3 mm and 70% of the thickness measurements were between 1 mm and 4 mm.

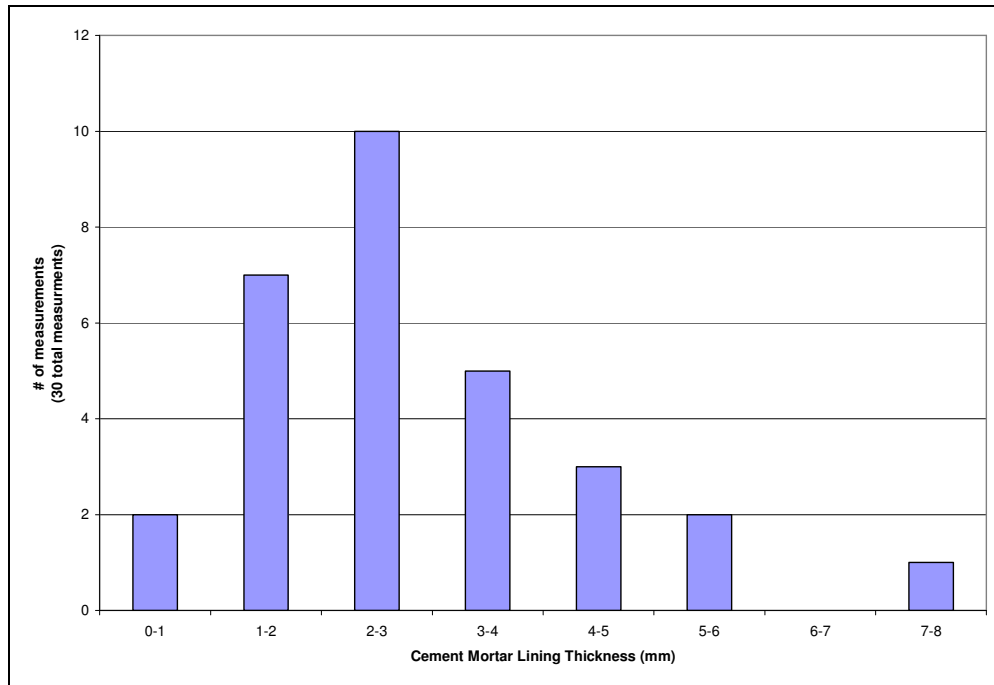


Figure 5-13: Distribution of cement mortar lining thicknesses in pipe section #1.

From Figure 5-13 it can be seen that the cement mortar lining is not of a uniform thickness over the entire interior surface of the pipe. The thickness of the cement mortar lining at the north end of pipe section #1 varied from a maximum of 7.22mm to a minimum of 0.89mm. The thickness of the cement mortar lining at the south end of pipe section #1 varied from a maximum of 5.61mm to a minimum of 0.88mm. The cement mortar lining thickness profiles and both ends of the pipe showed ridges where abrupt changes in cement mortar thickness occurred. The most pronounced ridge occurred at the -11cm coordinate where the cement mortar changed abruptly from a thickness of 1.05mm to 2.79mm at the north end of pipe section #1. The variations in cement mortar lining thickness are most likely due to the mechanical trowelling device used to place the cement mortar on the inside of the pipe. The mechanical trowelling device that would most likely be the cause of these thickness variations is shown in Figure 2-5 in Section 2.3.2.

The corrosion potential testing program was performed according to the methodology outlined in Section 4.3.2. Figure 5-14, Figure 5-15, and Figure 5-16 show the results of the three corrosion potential contour maps produced. The corrosion potential contour maps were created by measuring the corrosion potential using a Cu/CuSO₄ half cell electrode (CSE) at points directly under the grid points shown Figure 4-10 in Section 4.3.2. The corrosion potential contour plots are plotted according to the grid location rather than the actual location of the internal corrosion potential measurement. This was done in order to simplify data collection process and make the data collection process more consistent.

The three corrosion potential contour maps show similar corrosion potential patterns. Pipe section #1 showed the most negative potential measurements at the bottom of the pipe near the midpoint of the length of the pipe during the three testing periods. In all three tests the most negative potential recorded was between -0.86 V CSE and -0.76 V CSE. According to ASTM C876-91 potentials more negative than -0.35 V CSE indicate that there is greater than a 90% chance that corrosion is occurring. In all three tests the least negative potentials recorded were located near the topline of the pipe at both the north and south end. The potentials recorded at these points ranged from -0.27 V CSE to -0.17 V CSE. According to ASTM C876-91 potentials less negative than -0.20 V CSE indicate that there is a greater than

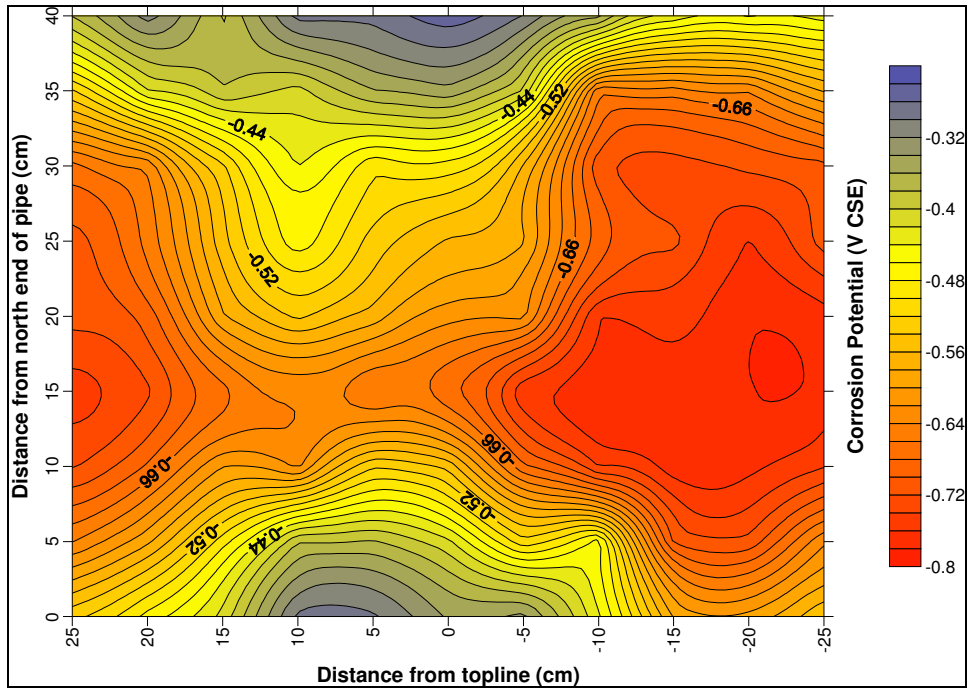


Figure 5-14: Corrosion potential contour map pipe section #1, Aug 15, 2006.

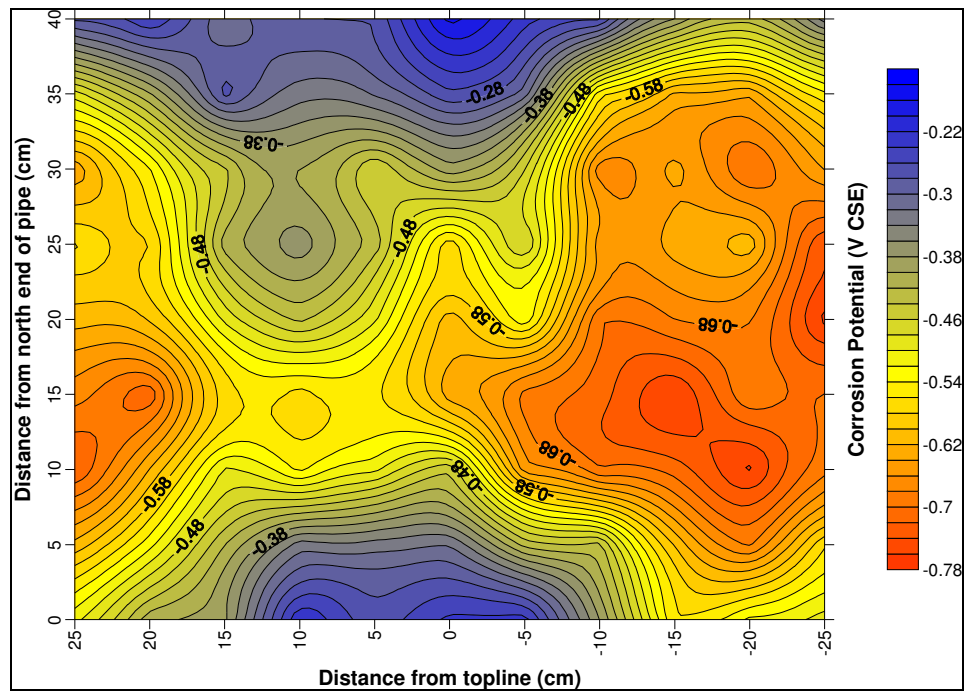


Figure 5-15: Corrosion potential contour map pipe section #1, Aug 16, 2006.

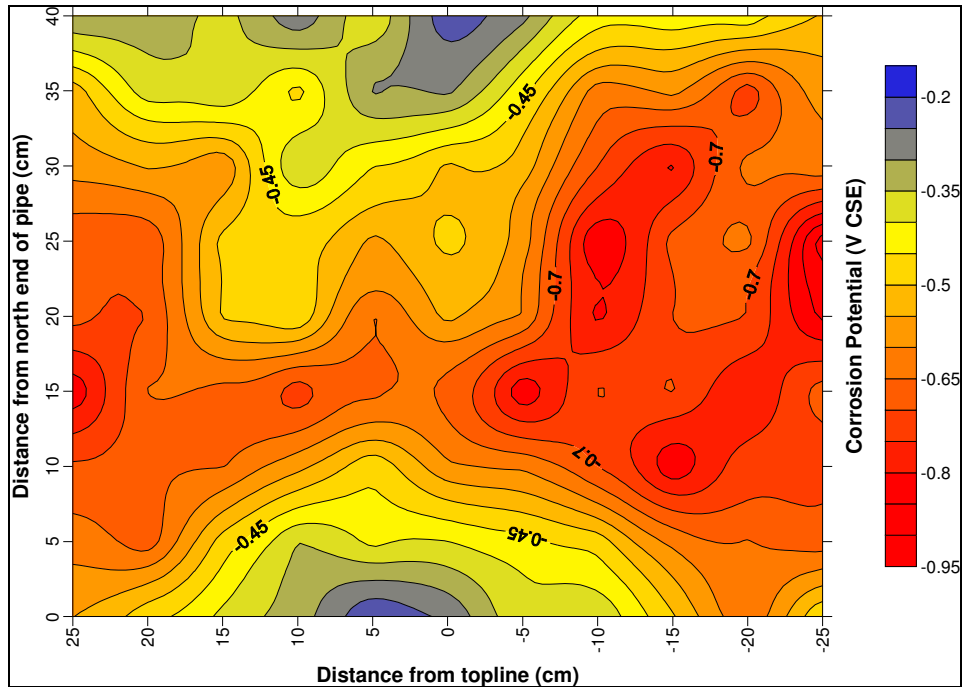


Figure 5-16: Corrosion potential contour map pipe section #1, Aug 17, 2006.

90% probability that corrosion is NOT occurring. The corrosion potential contour maps illustrate that the majority of corrosion potentials measured in pipe section #1 are more negative than -0.35 V CSE. This indicates that there is a greater than 90% probability that corrosion is occurring on majority of the interior of pipe section #1. This is in contrast to field observations, presented in Section 3.3, that show that cement mortar lining prevents internal iron watermain corrosion. However, the voltage ranges provided in ASTM C876-99 are guidelines which need to be applied with caution.

The most likely reason for the distribution of increased corrosion potentials at the bottom of pipe section #1 near the midpoint of the pipe's length is the position in which the pipe was stored prior to testing, and the position in which the pipe was stored during testing. Pipe section #1 was stored leaned against a wall in a humidity room for over one year prior to

testing. The location of the pipe closest to the wall corresponds to the bottom of pipe section #1. During the testing period the pipe was stored horizontally with a wet sponge inside the pipe and both ends of the pipe sealed with plastic to prevent the cement mortar lining from drying. Both of these storage positions could have lead to a greater amount of moisture exposure of the bottom of pipe section #1 than the topline section of pipe section #1.

The goal of this testing program was to determine whether or not the thickness and application quality of cement mortar lining influenced the ability of the cement mortar lining to prevent corrosion from occurring. In Figure 5-17 the pipe section #1 corrosion potential contour map shown in Figure 5-16 has been overlaid by the cement mortar lining thickness contours of pipe section #1. Pipe section #1 lining thickness contours presented in Figure 5-17 are based upon a linear interpolation of the cement mortar lining thickness measured at each end of the pipe. To create this interpolation it was assumed that lining thickness varied in a linear pattern from the north to the south end of pipe section #1 at each offset from the topline.

From Figure 5-17 it is apparent that there is not a correlation between corrosion potential and cement mortar lining thickness. Thus, from the results of this testing program it can be concluded that the thickness of cement mortar lining does not affect the ability of the cement mortar lining to prevent corrosion. However, these tests were performed using a pipe which had been lined for a period only slightly longer than one year. It would be useful to perform tests on samples which have been in service for several years to determine the long-term

effects of cement mortar lining thickness on the ability of the cement mortar lining to prevent corrosion.

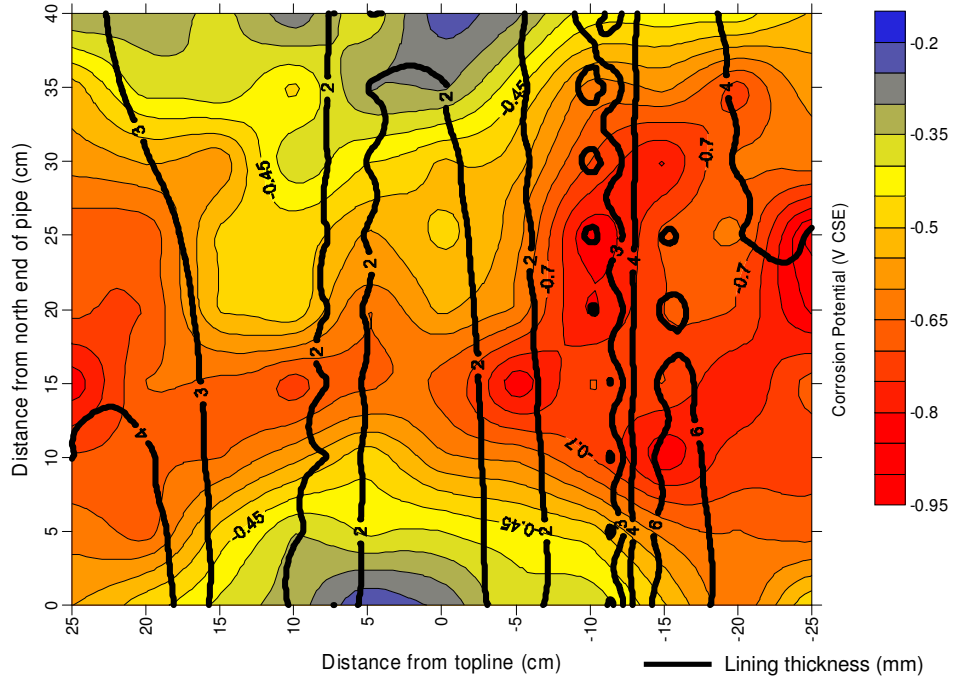


Figure 5-17: Corrosion potential contour map pipe section #1, Aug 17 2006, overlaid by pipe section #1 interpolated lining thickness contours.

6. Conclusions

The main goals of this research were to determine the ability of cement mortar lining to bridge corrosion pin-holes and the influence of cement mortar lining thickness on the passivation of cast iron watermain. This Chapter summarizes the main conclusions that can be drawn from the research program.

6.1 Ability to Bridge Corrosion Pin-holes / Water Loss

Prevention

Results from the physical testing of cement mortar cubes collected from the City of Toronto provided the following observations and conclusions:

- The compressive strength of the cement mortar was consistent between all contractors sampled in the City of Toronto in 2005.
- The average long-term (>28 day) compressive strength of field cast cement mortar cubes was 48.7 MPa. The average long-term (>28 day) compressive strength of

laboratory cast cement mortar cubes was 50.1 MPa. Since the measured strengths are similar it is assumed that the field and laboratory samples had similar water-content ratios and mechanical properties.

- The compressive strength of the cement mortar cubes increased with time until a curing period of at least four days at which point the compressive strength stabilized at the average long-term (>28 day) compressive strength.
- If the lining is used to bridge pin-holes water pressure should not be applied to the watermain until the cement mortar has cured for a period of time such that the mortar strength has reached the average long-term (>28 day) compressive strength.

Results from the water loss prevention testing program provided the following observations and conclusions:

- Failures occurring in short cure specimens were not repeatable in specimens which had undergone curing periods of at least 48 hours. This shows that cement mortar lining specimens that are cured for a period of 24.5 hours or less are significantly more likely to fail than specimens that receive an adequate amount of time to properly cure. This assumes that the mortar in question has a 28 day compressive strength comparable to the average long-term (>28 day) compressive strength.
- The cure time corrected normalized thickness at failure values followed a Gumbel distribution.
- Using the Gumbel distribution, probabilities that failure will not occur for set cure time corrected normalized thickness values were obtained. These values can be used

to determine the lining thickness required to bridge a set pin-hole diameter at a specified confidence level that the lining will not fail over the pin-hole.

- A 3 mm thick cement mortar lining can bridge a pin-hole 12.0 mm in diameter while a 5mm thick cement mortar lining can bridge a pin-hole 19.9 mm in diameter with a 95% probability that failure will not occur.
- Using the force balance method and assuming a vertical shear failure mechanism the cement mortar lining was found to have a shear strength at failure of 2.1 MPa. This value is appropriate for design purposes.

6.2 Corrosion Prevention

Results from the corrosion prevention testing program provided the following observations and conclusions:

- The cement mortar lining applied to pipe section #1 ranged in thickness from 0.88 mm to 7.22 mm. The variations in cement mortar lining thickness are most likely due to the mechanical trowelling device used to place the cement mortar on the inside of the pipe.
- The majority of the corrosion potentials measured in pipe section #1 are more negative than -0.35 V CSE. According to ASTM C876-99 guidelines, this indicates that there is a greater than 90% probability that corrosion is occurring on majority of the interior of pipe section #1. This is in contrast to field observations that show cement mortar lining prevents internal iron pipe corrosion. The applicability of ASTM C876-99 guidelines to this application requires further investigation.

- The corrosion potentials were not correlated with the thickness of the cement mortar lining.

7. Recommendations

This chapter summarizes the main recommendations that can be drawn from this research program. This chapter also outlines recommendations for future research.

If the cement mortar lining is to be used to bridge pin-holes in watermains, pressure should not be applied to the watermain until the cement mortar has cured for a period of time such that the mortar strength has reached the average long-term (>28 day) compressive strength. For this research this strength was achieved after four days of curing. Longer cure times may be required.

If cement mortar lining is to be used to bridge small pin-holes the liner must be applied with a uniform thickness. Drag trowelling methods, used in this study, did not produce linings with uniform thickness.

Long-term testing should be performed to determine the fatigue properties of cement mortar linings. Long-term fatigue testing requires the development of a pressure intensifier capable of pressurizing a larger volume of water than used in this study.

Further testing is required to determine:

- The applicability of the corrosion potential ranges recommended in ASTM C876-99 to thin cement mortar linings.
- If other electrochemical measurement techniques are better suited to determine the potential for pipe wall corrosion under the cement mortar lining.

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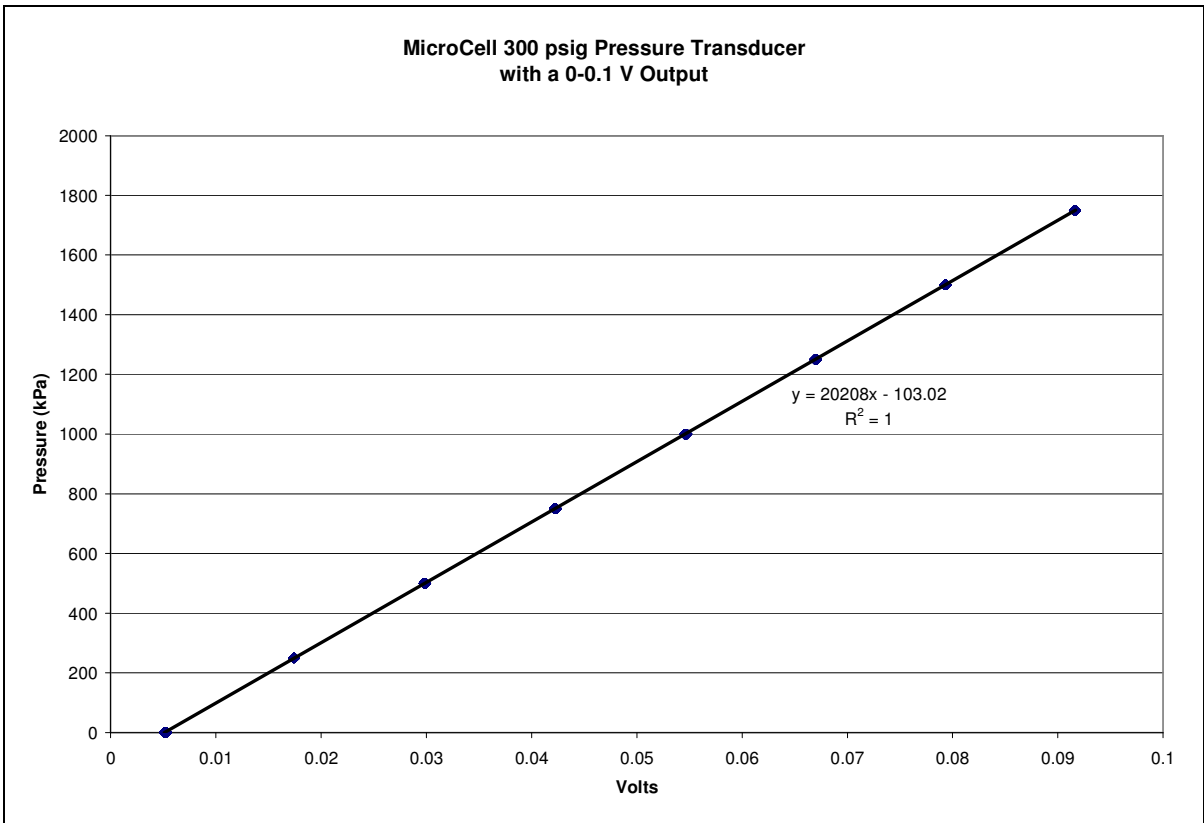
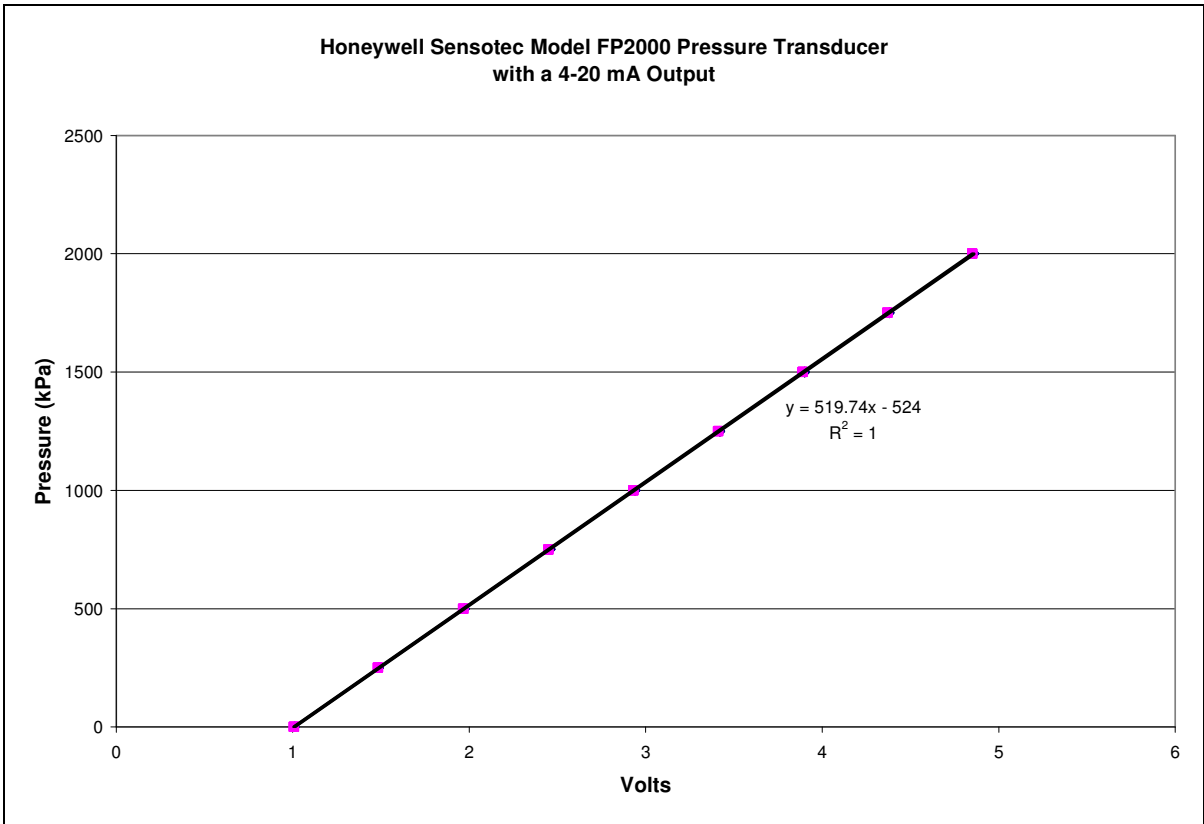
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Appendix A
Pressure Transducer Calibration Data



Appendix B
Mortar Cube Compressive Strength Test Results

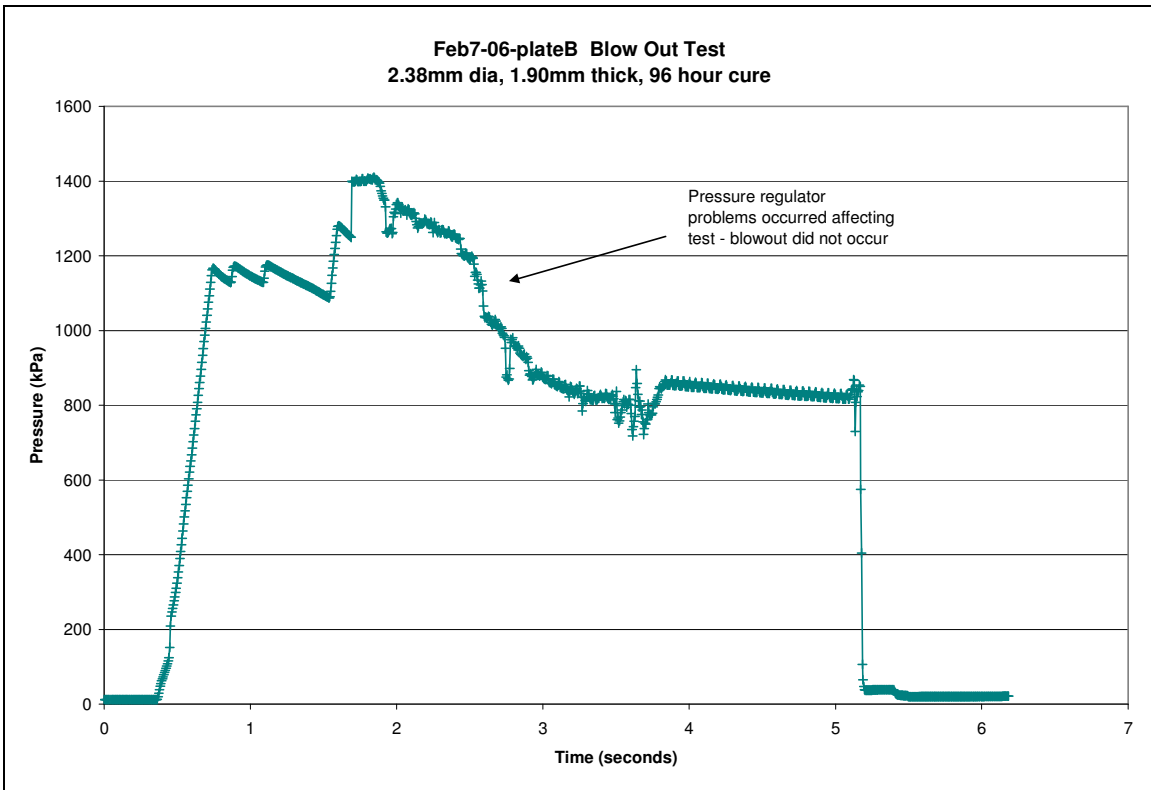
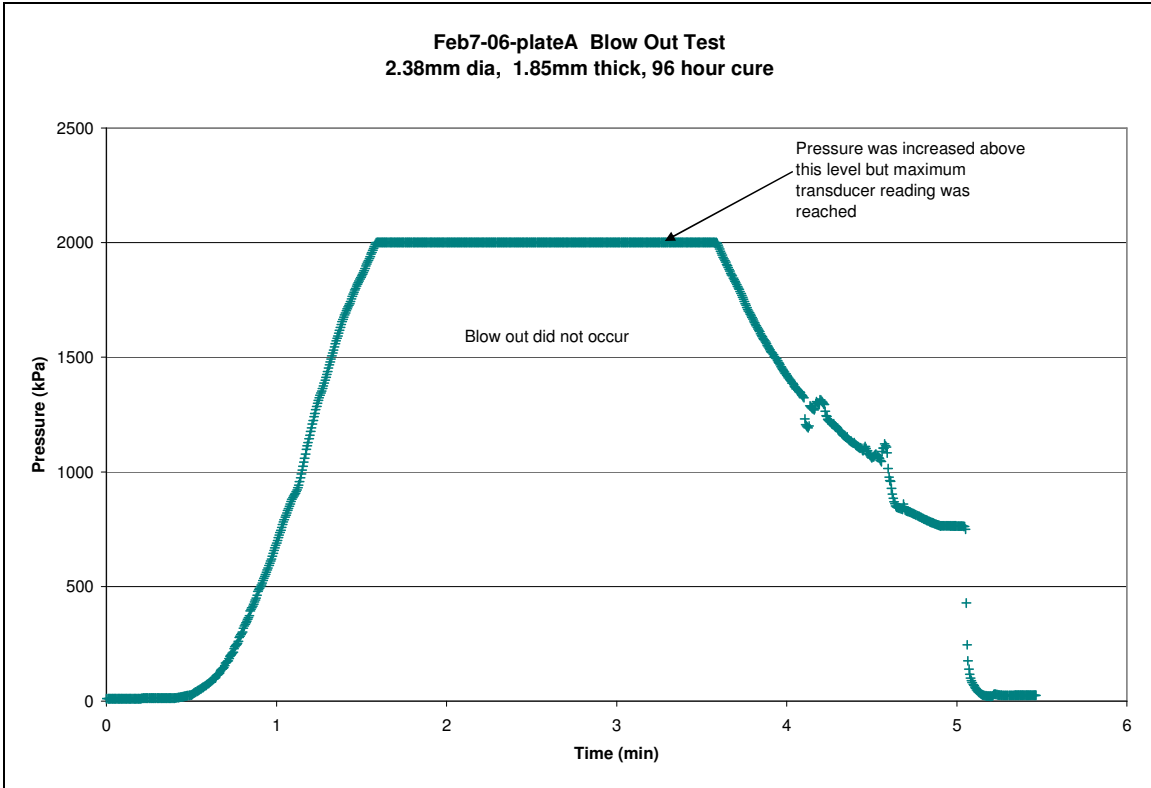
Compressive Strength Test Results for Field and Laboratory Cast Specimens

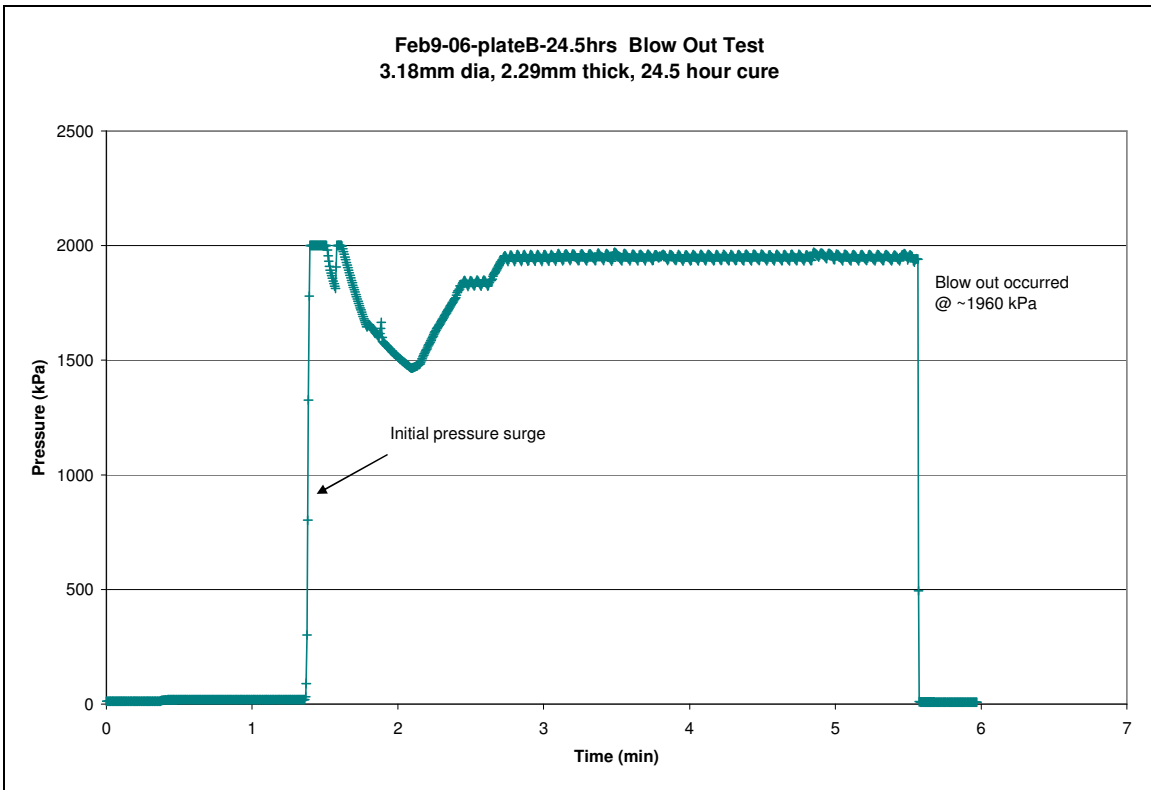
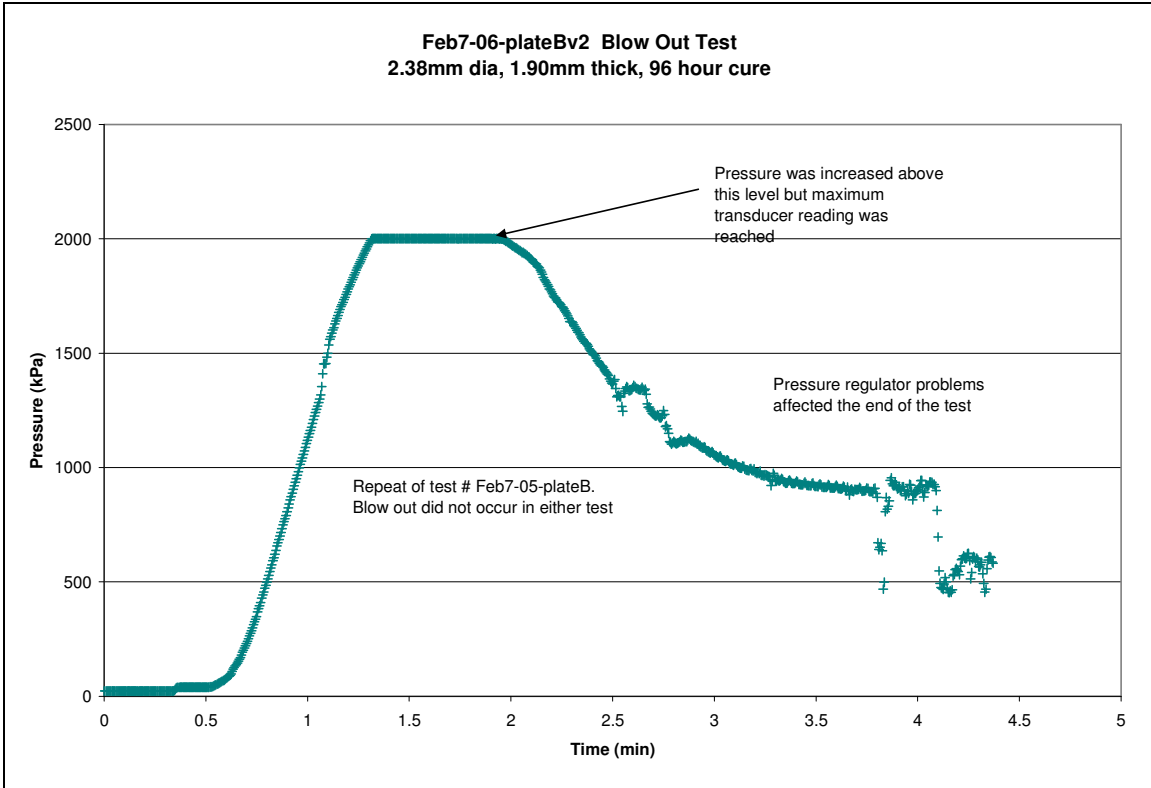
Project Location:	Contractor:	Sample ID:	Date Collected:	Date Tested:	Days Cured:	Peak Load (kN):	Max. Compressive Strength (Mpa):
Little Blvd	FerPal	20/6/5-1	20-Jun-05	29-Aug-05	70	142.25	56.9
		20/6/5-2			70	132.50	53.0
		20/6/5-3			70	140.00	56.0
		20/6/5-5			70	140.00	56.0
		20/6/5-6			70	103.20	no value
		20/6/5-7			70	135.00	54.0
		20/6/5-8			70	132.00	52.8
		20/6/5-9			70	152.25	60.9
Corrundum Crescent	Spinello	11/7/5-1	11-Jul-05	8-Sep-05	59	121.00	48.4
		11/7/5-2			59	108.50	43.4
		11/7/5-3			59	75.50	30.2
		11/7/5-4			59	100.50	40.2
		11/7/5-5			59	129.50	51.8
		11/7/5-6			59	125.50	50.2
		11/7/5-7			59	118.75	47.5
		11/7/5-8			59	127.50	51.0
Lloyd Manor Drive	FerPal	12/7/5-1	12-Jul-05	8-Sep-05	58	114.30	45.7
		12/7/5-2			58	103.50	41.4
		12/7/5-3			58	128.50	51.4
		12/7/5-4			58	100.50	40.2
		12/7/5-5			58	115.00	46.0
		12/7/5-6			58	130.75	52.3
		12/7/5-7			58	126.25	50.5
		12/7/5-8			58	115.25	46.1
Arkona Drive	Spinello	18/7/5-1	18-Jul-05	26-Sep-05	70	147.50	59.0
		18/7/5-2			70	155.00	62.0
		18/7/5-3			70	115.00	46.0
		18/7/5-4			70	122.50	49.0
		18/7/5-5			70	107.00	42.8
		18/7/5-6			70	122.50	49.0
		18/7/5-7			70	132.50	53.0
		18/7/5-8			70	131.00	52.4
Homewood Avenue	New Tide	25/7/5-1	25-Jul-05	28-Sep-05	65	126.50	50.6
		25/7/5-2			65	120.00	48.0
		25/7/5-3			65	123.75	49.5
		25/7/5-4			65	135.00	54.0
		25/7/5-5			65	122.50	49.0
		25/7/5-6			65	119.50	47.8
		25/7/5-7			65	132.50	53.0
		25/7/5-8			65	118.25	47.3
Tower Drive	Spinello	18/8/5-1	18-Aug-05	28-Sep-05	35	108.50	43.4
		18/8/5-2			35	130.00	52.0
		18/8/5-3			35	105.00	42.0
		18/8/5-4			35	95.00	38.0
		18/8/5-5			35	136.00	54.4
		18/8/5-6			35	115.50	46.2
		18/8/5-7			35	107.50	43.0
		18/8/5-8			35	117.50	47.0
Laboratory Samples	N/A	4/8/5-1	4-Aug-05	28-Sep-05	56	120.00	48.0
		4/8/5-2			56	131.00	52.4
		4/8/5-3			56	107.00	52.8
		4/8/5-4			56	135.00	54.0
		4/8/5-5			56	126.50	50.6
		4/8/5-6			56	122.50	49.0
		4/8/5-7			56	109.50	43.8
		4/8/5-8			56	115.00	46.0
4/8/5-9	56	136.50	54.6				

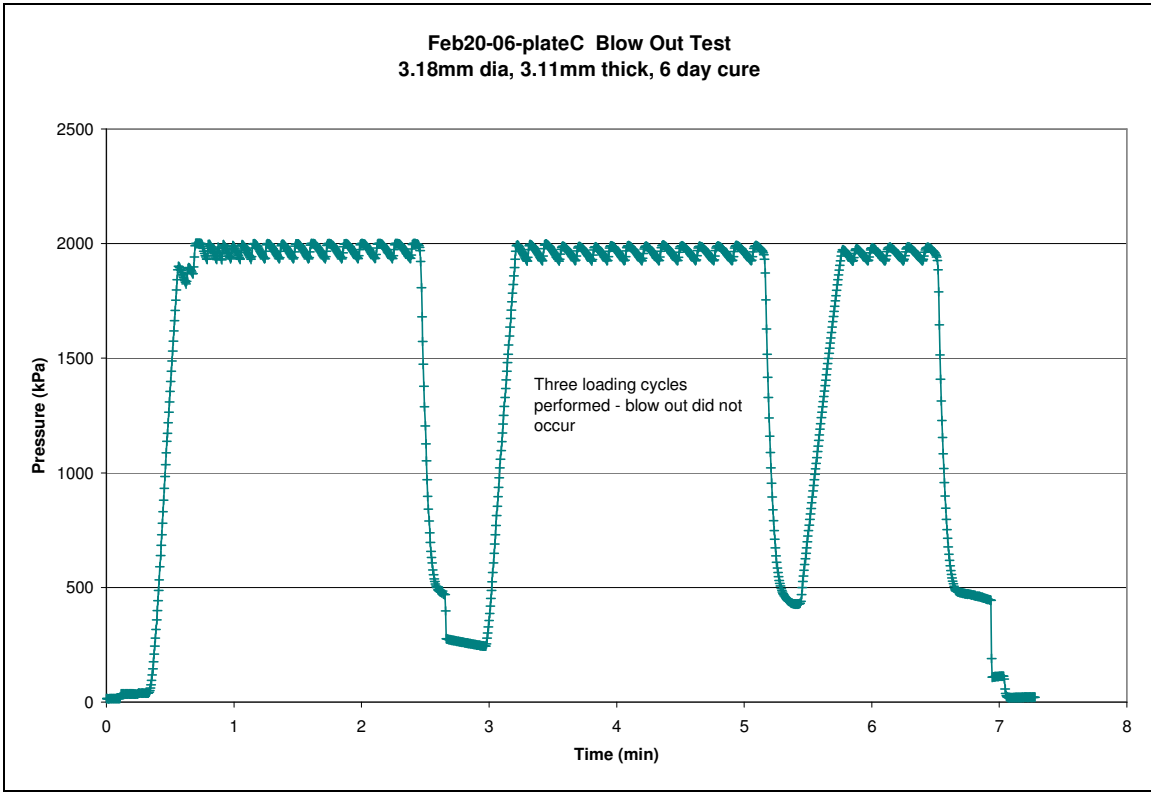
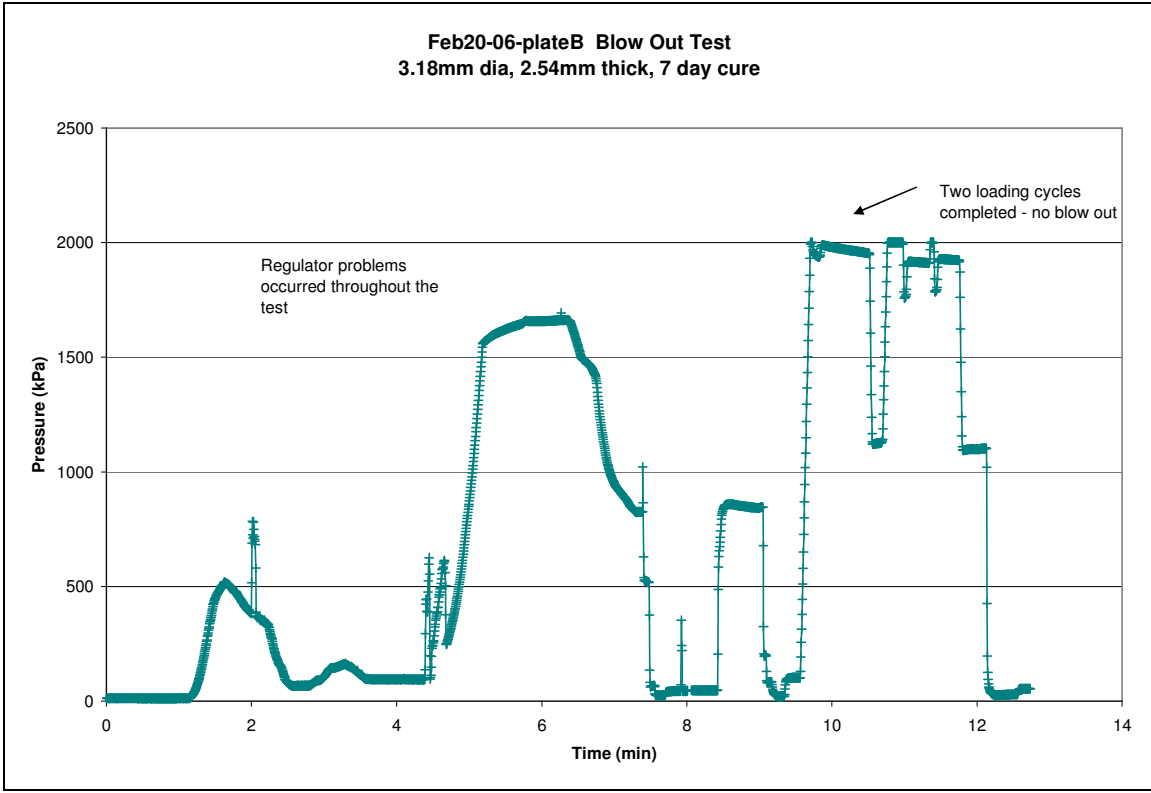
Mortar Cube Testing - Compressive Strength vs Time

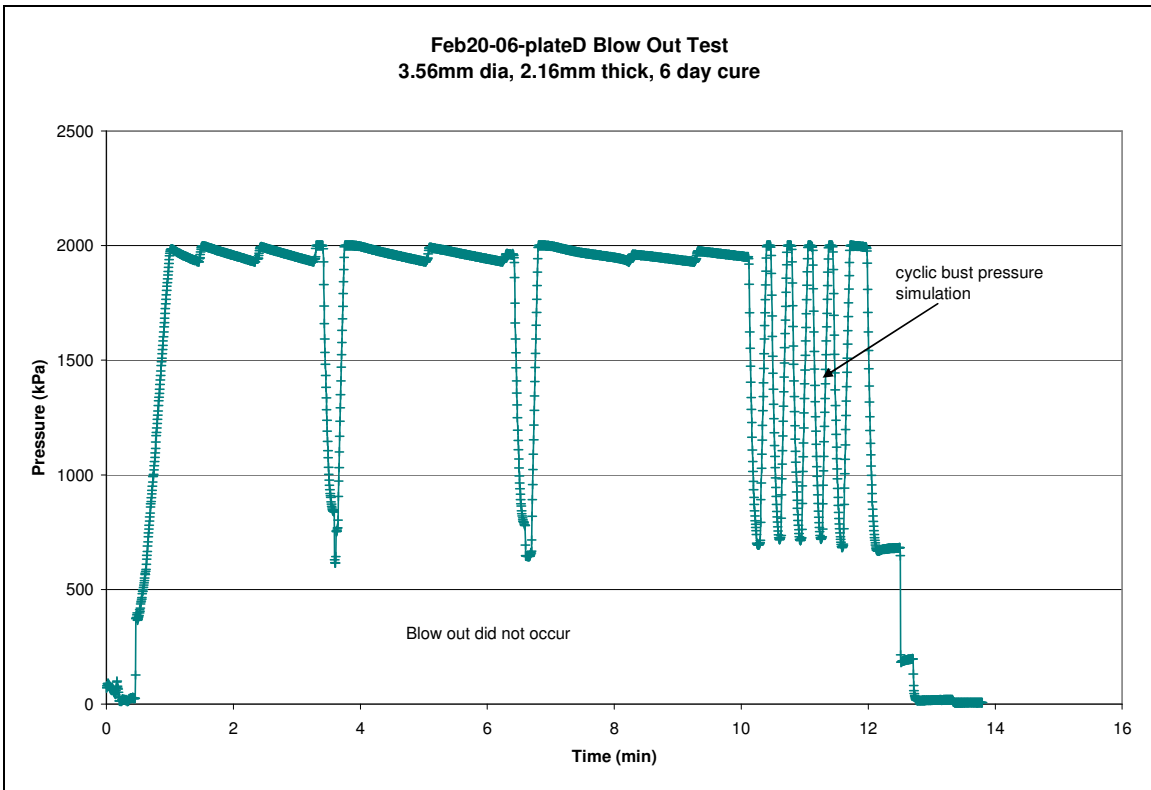
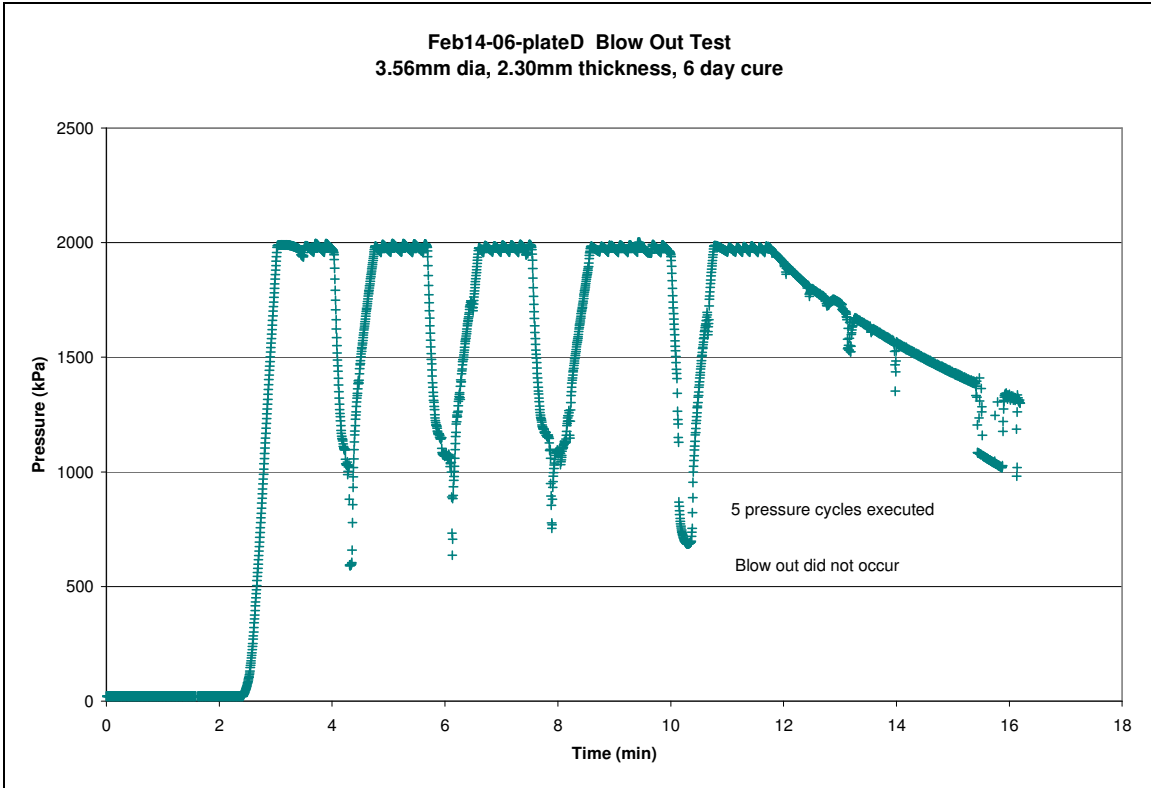
Collection Date	Test Date	Curing Time (hrs)	Curing Time (days)	Sample ID	Max Load (kN)	Compressive Strength (MPa)
3-Oct-05	4-Oct-05	24	1	3/10/5-1	62.00	24.1
				3/10/5-2	66.00	25.6
				3/10/5-3	76.25	29.6
	5-Oct-05	50	2.1	3/10/5-4	90.65	35.2
				3/10/5-5	92.75	36.0
				3/10/5-6	79.00	30.7
	6-Oct-05	72	3	3/10/5-7	120.10	46.5
				3/10/5-8	104.09	40.3
				3/10/5-9	115.65	44.8
	7-Oct-05	99	4.1	3/10/5-10	126.77	49.1
				3/10/5-11	115.65	44.8
				3/10/5-12	130.56	50.6
	8-Oct-05	120	5	3/10/5-13	117.88	45.7
				3/10/5-14	126.77	49.1
				3/10/5-15	129.00	50.0

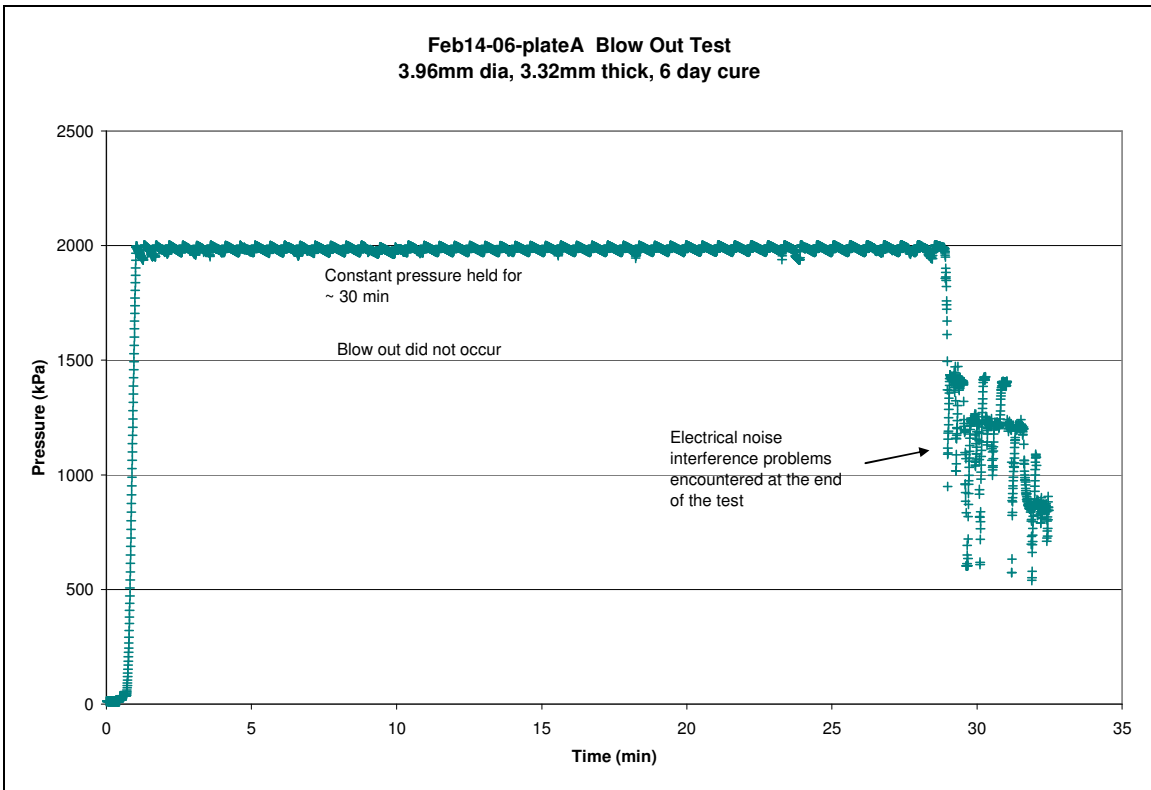
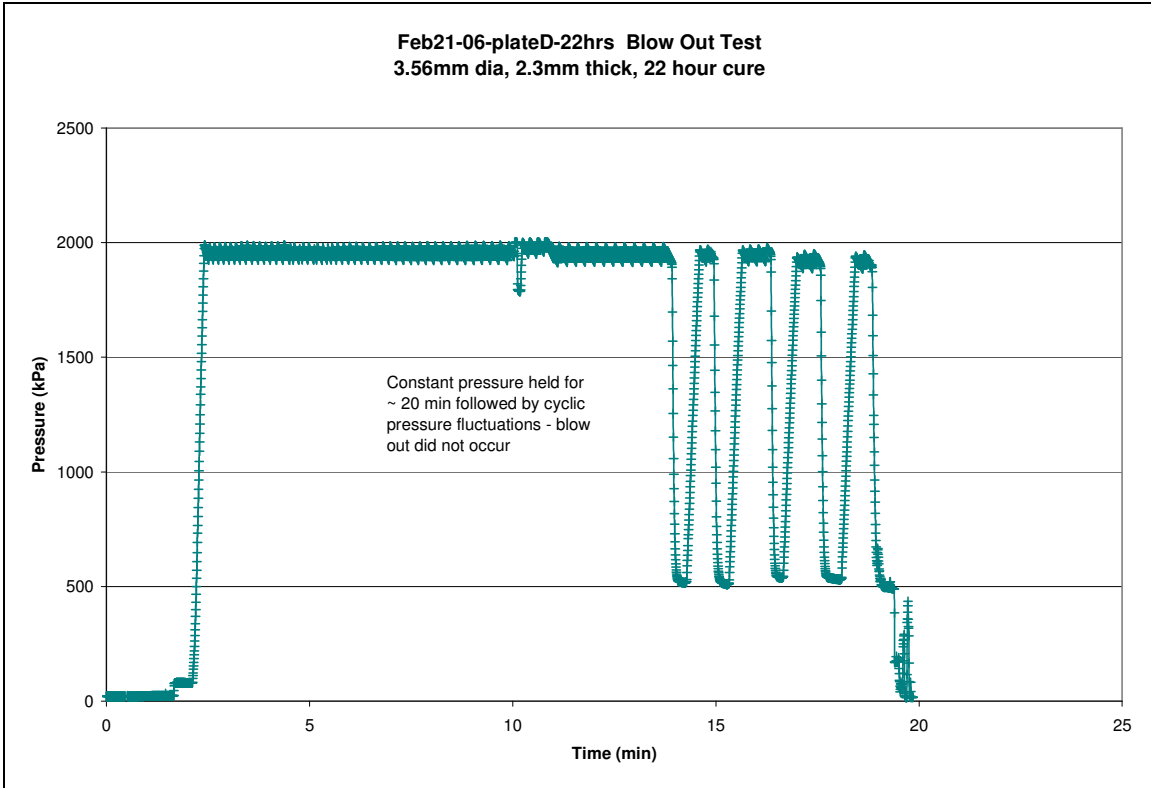
Appendix C
Ability to Bridge Corrosion Pin-holes / Water Loss
Prevention Testing Program Results

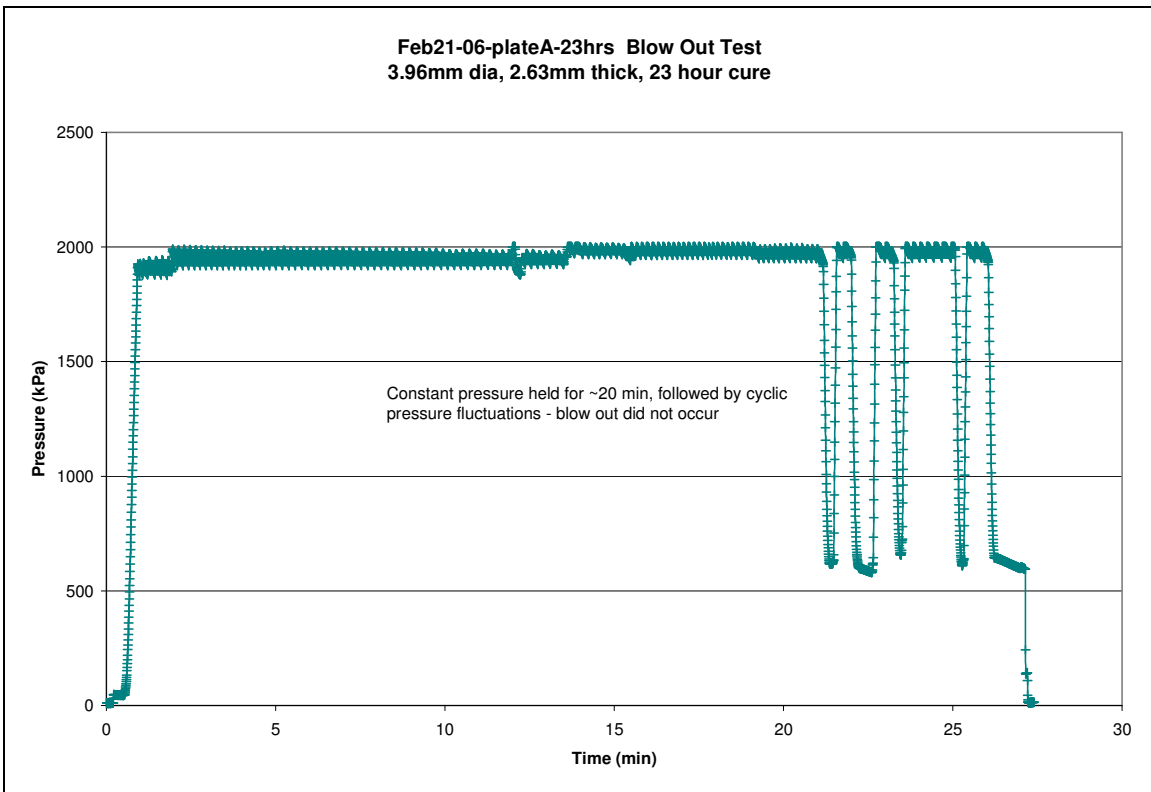
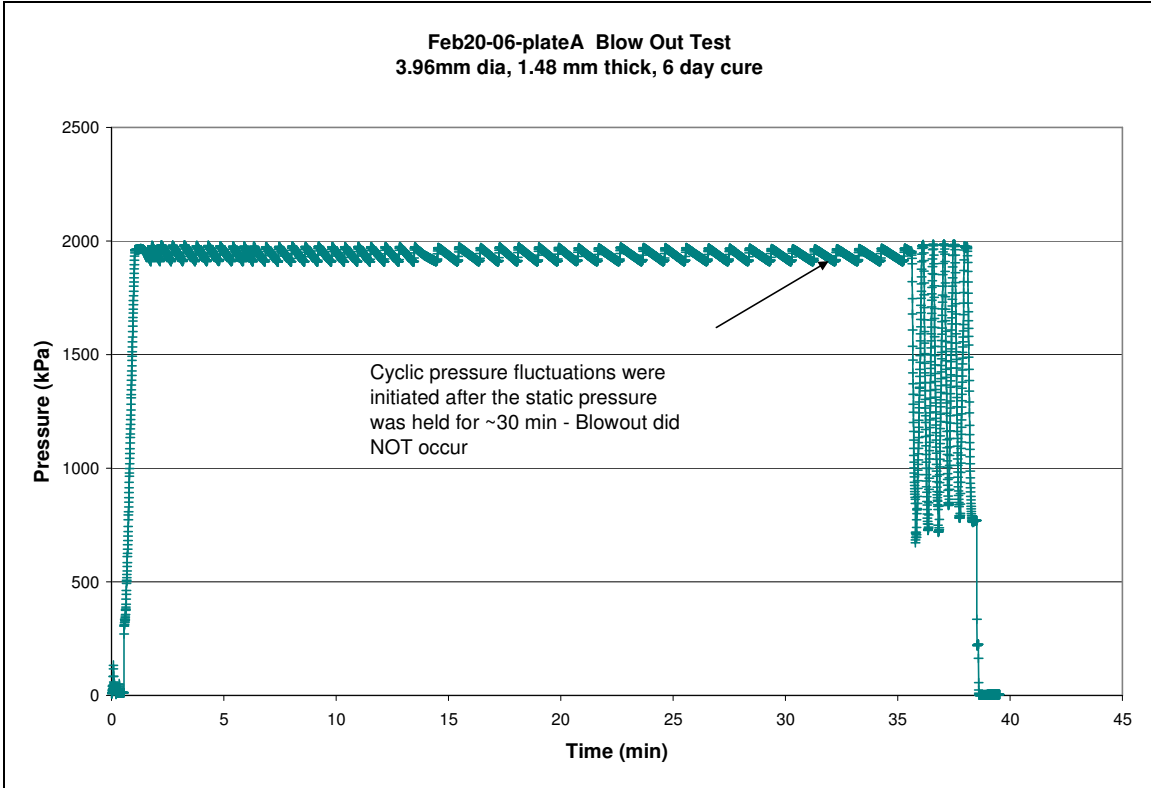


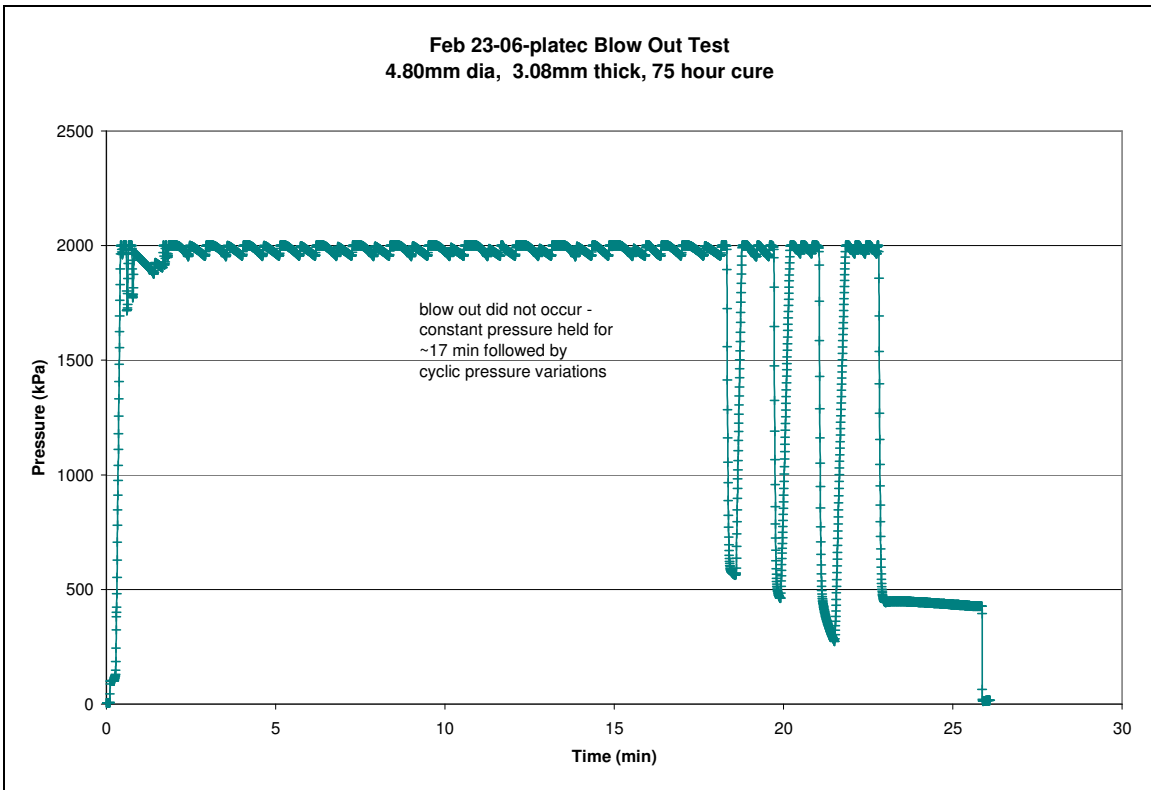
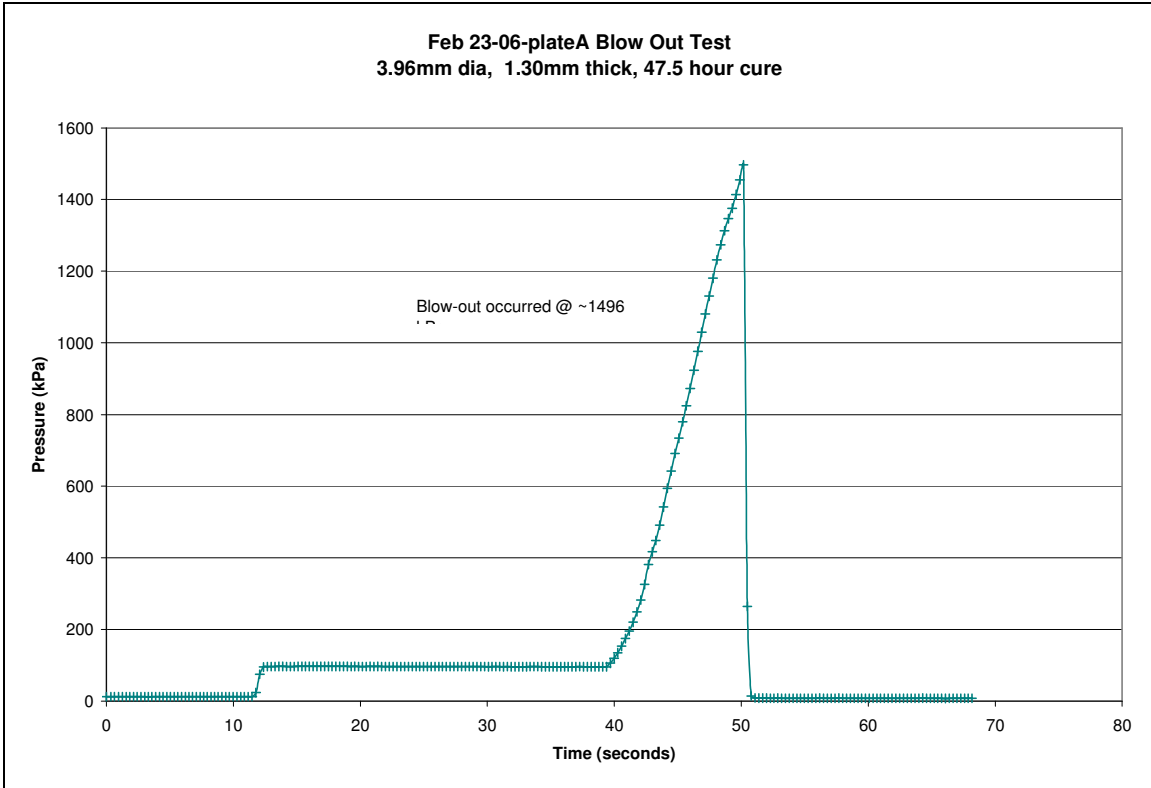


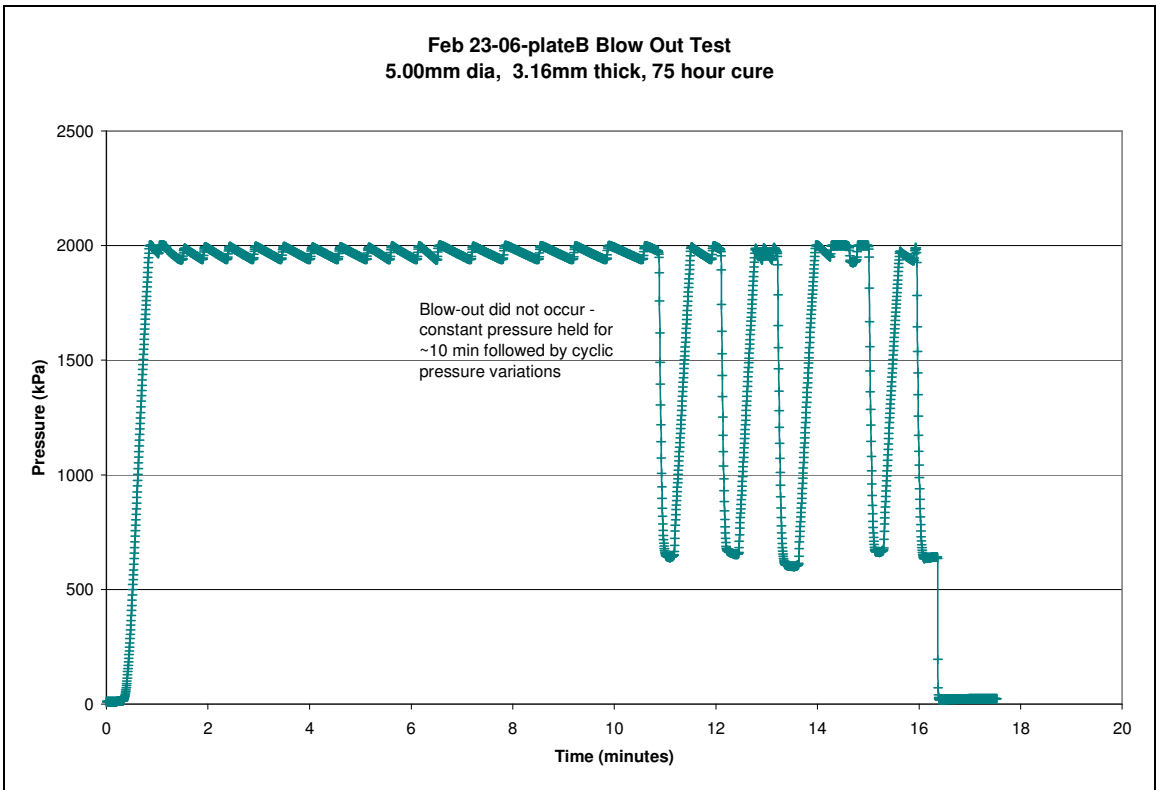
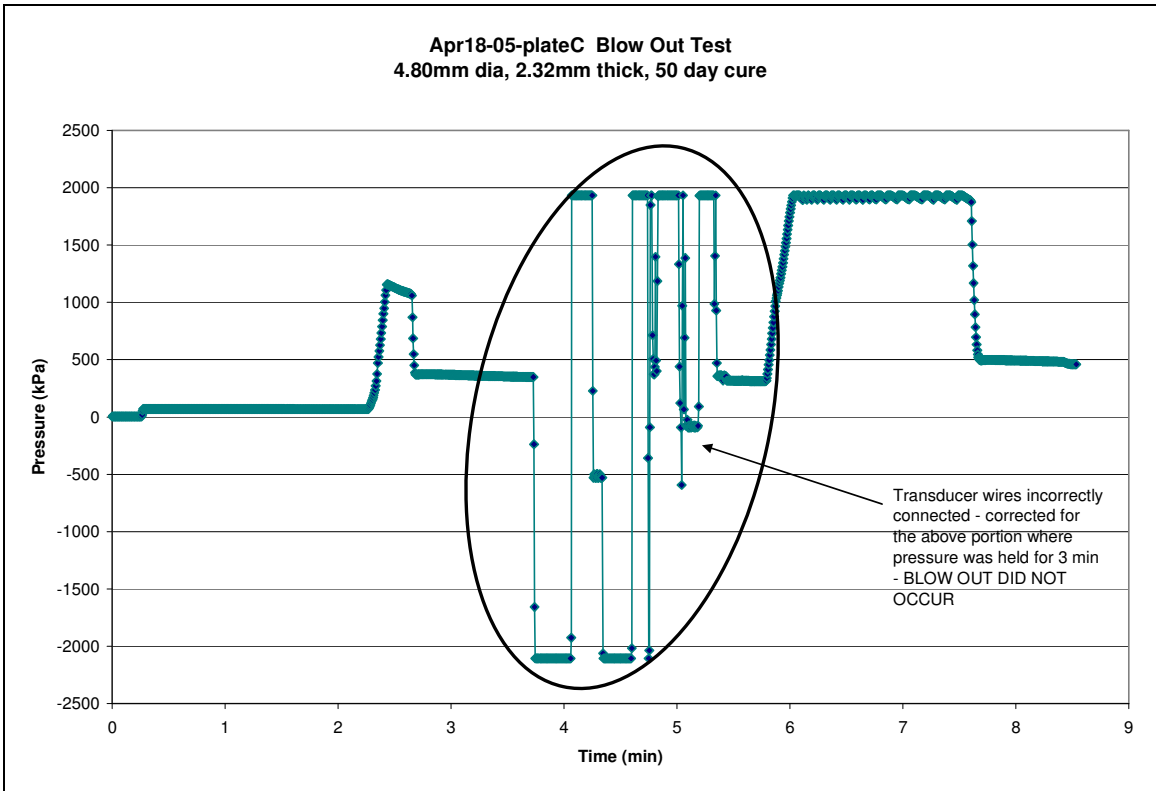


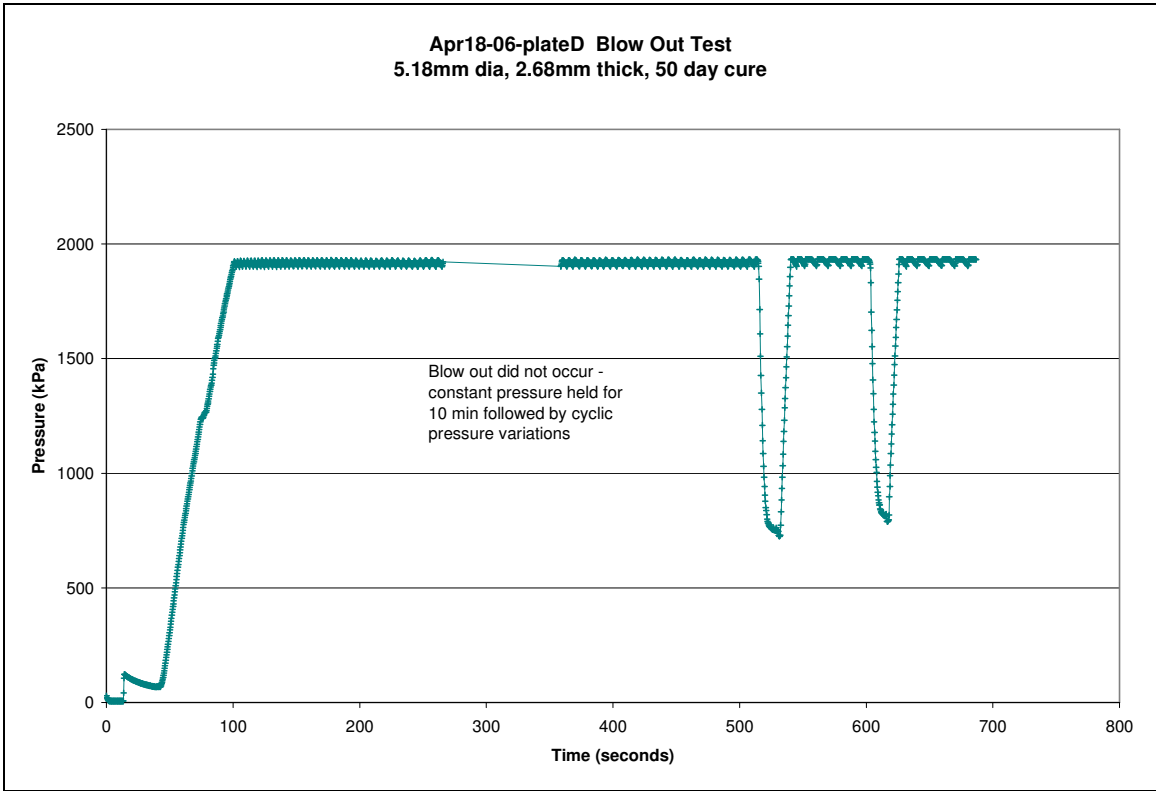
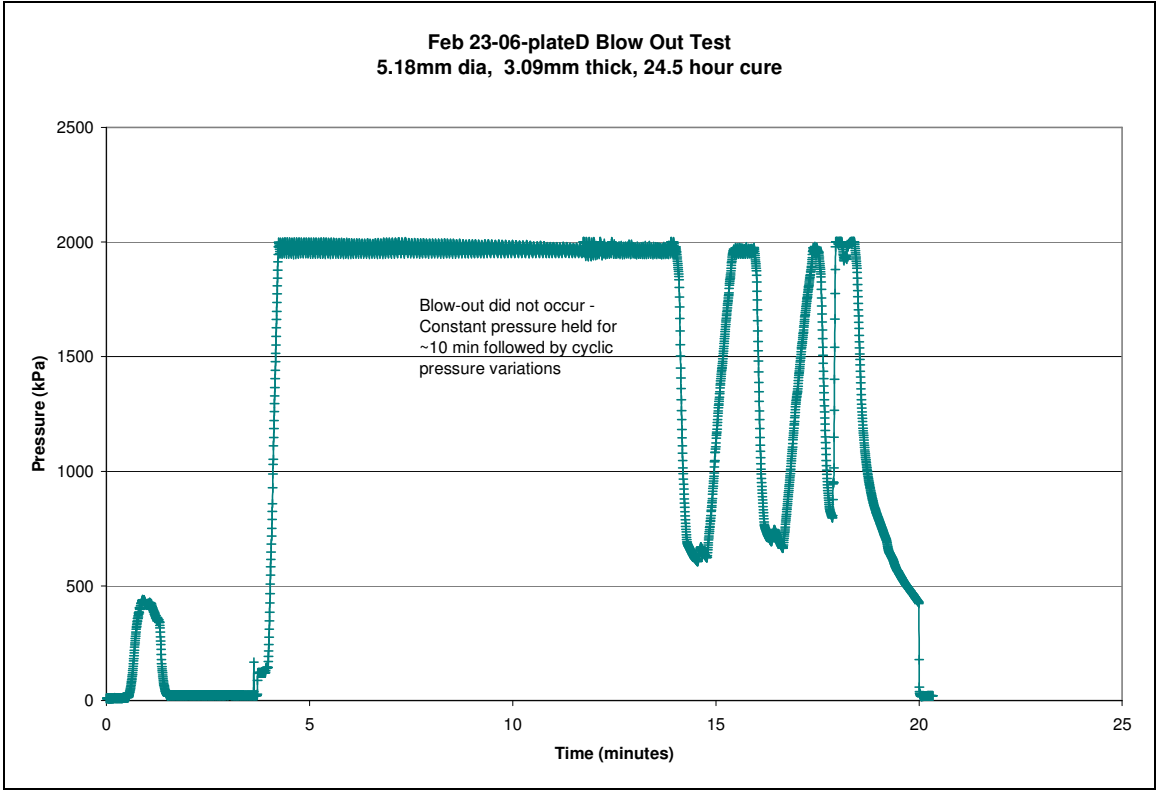




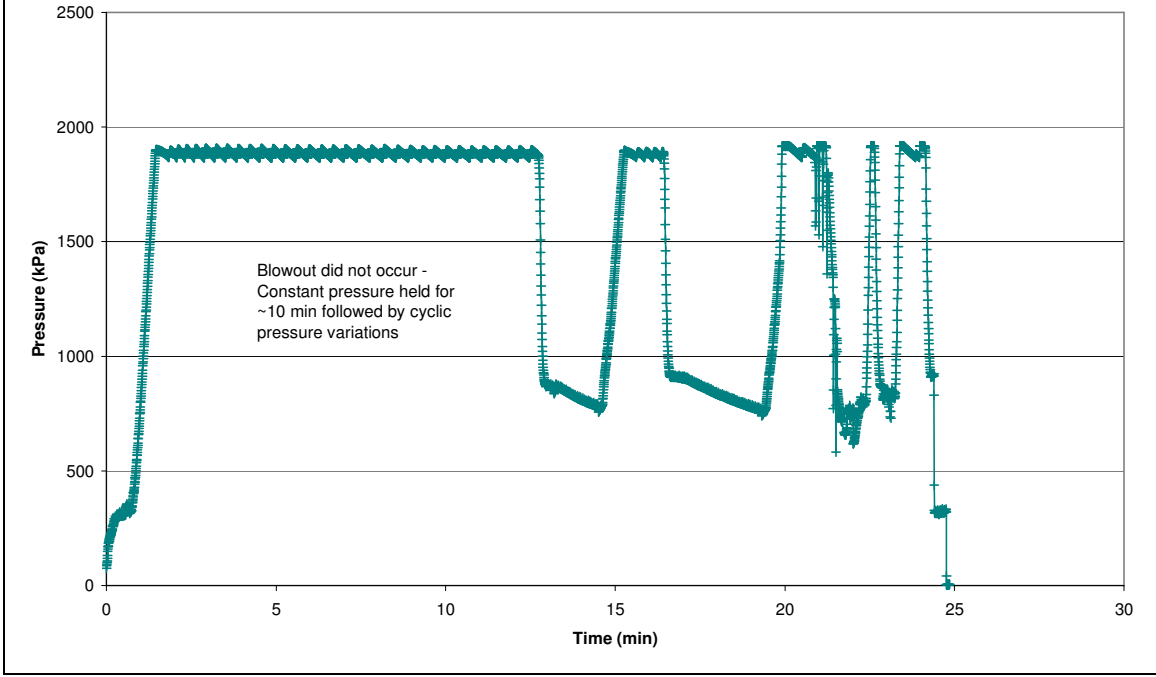




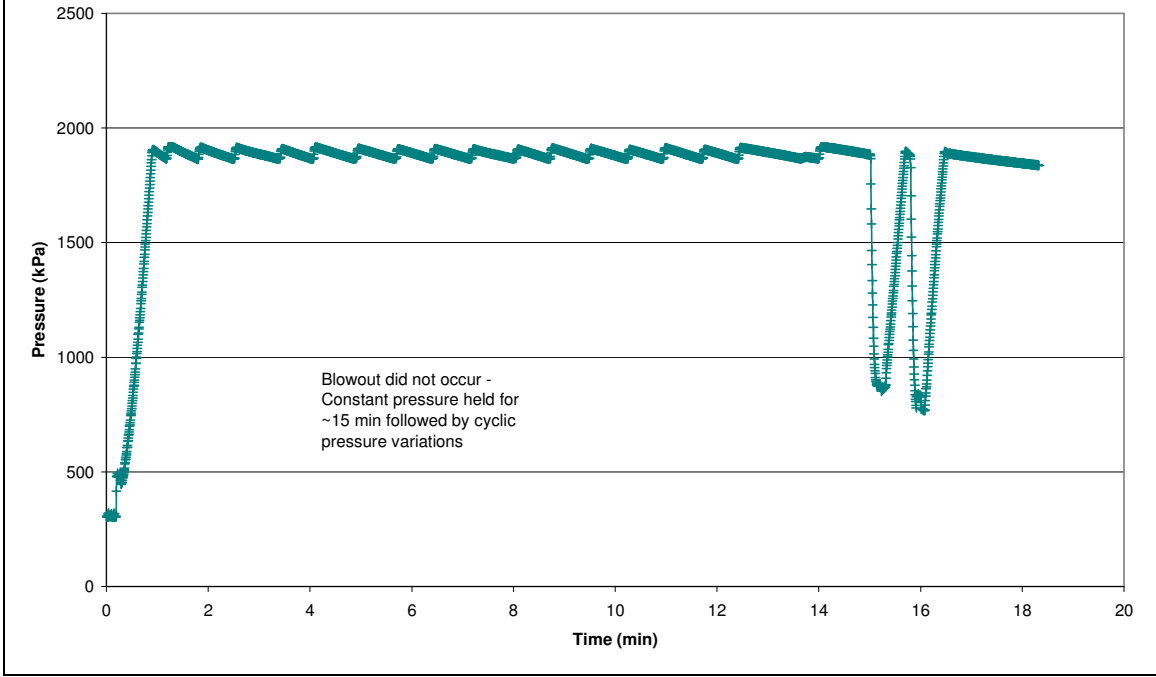


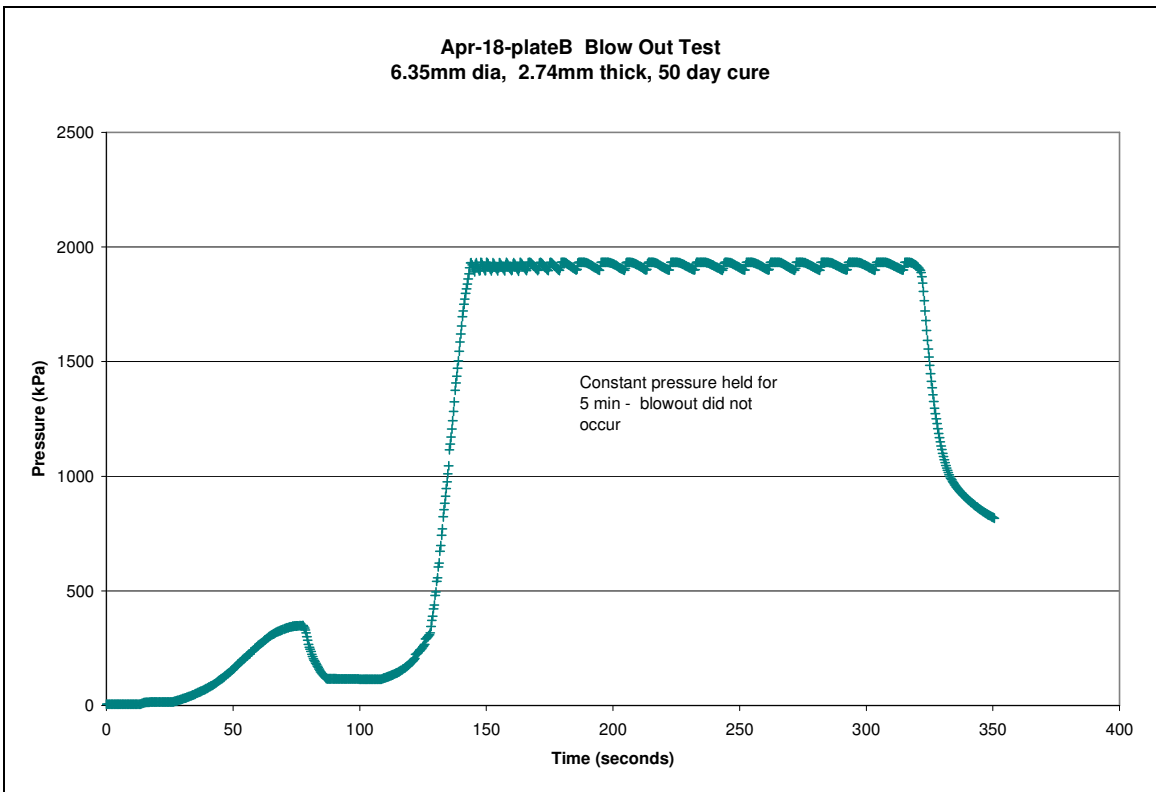
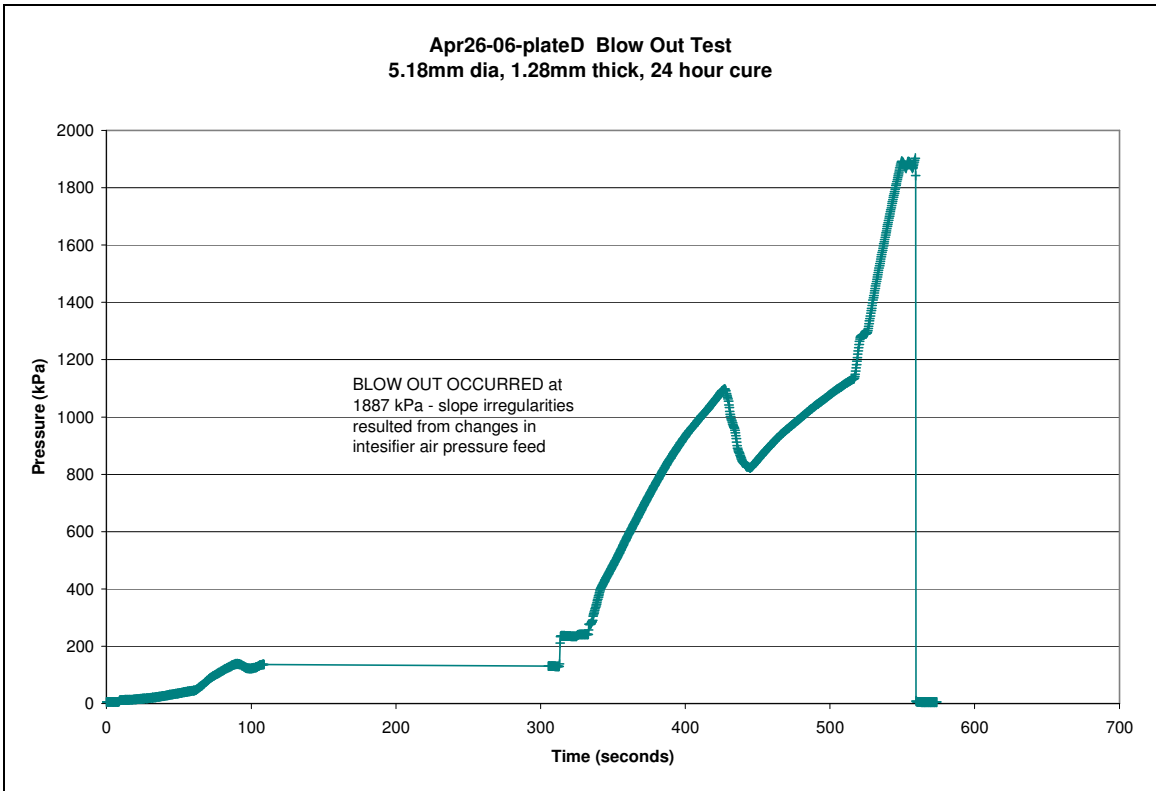


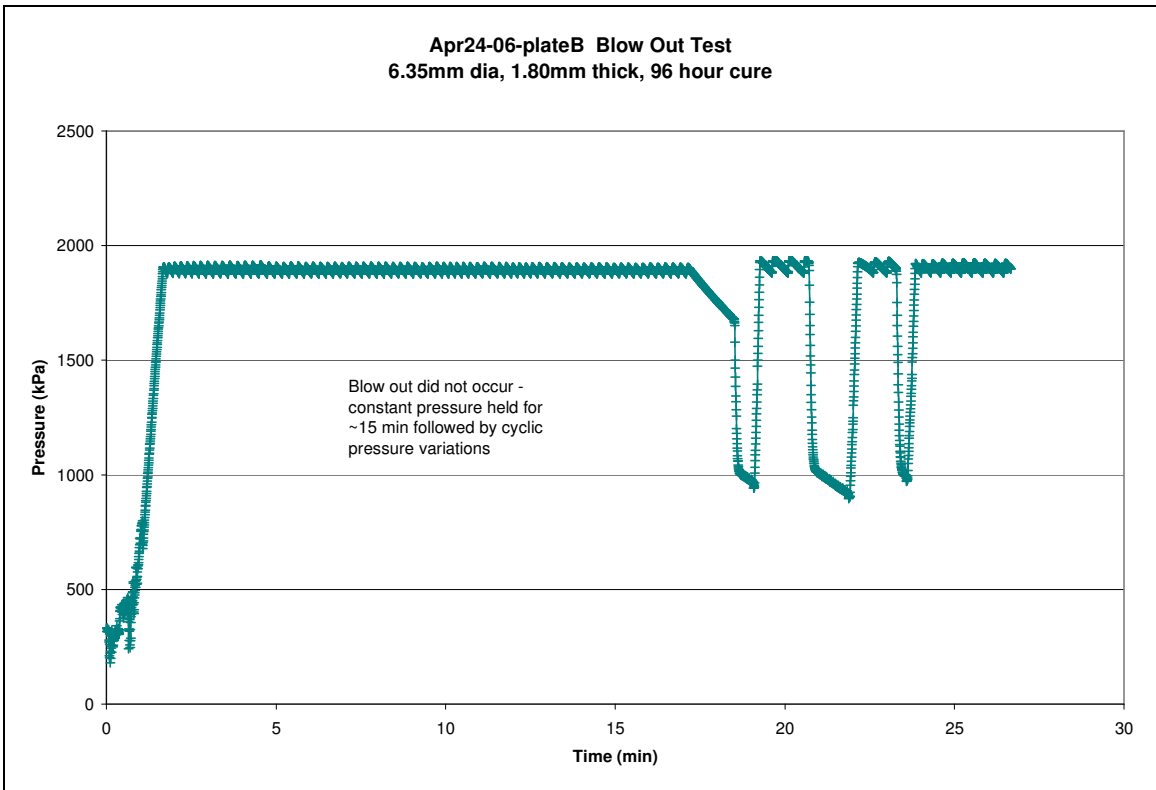
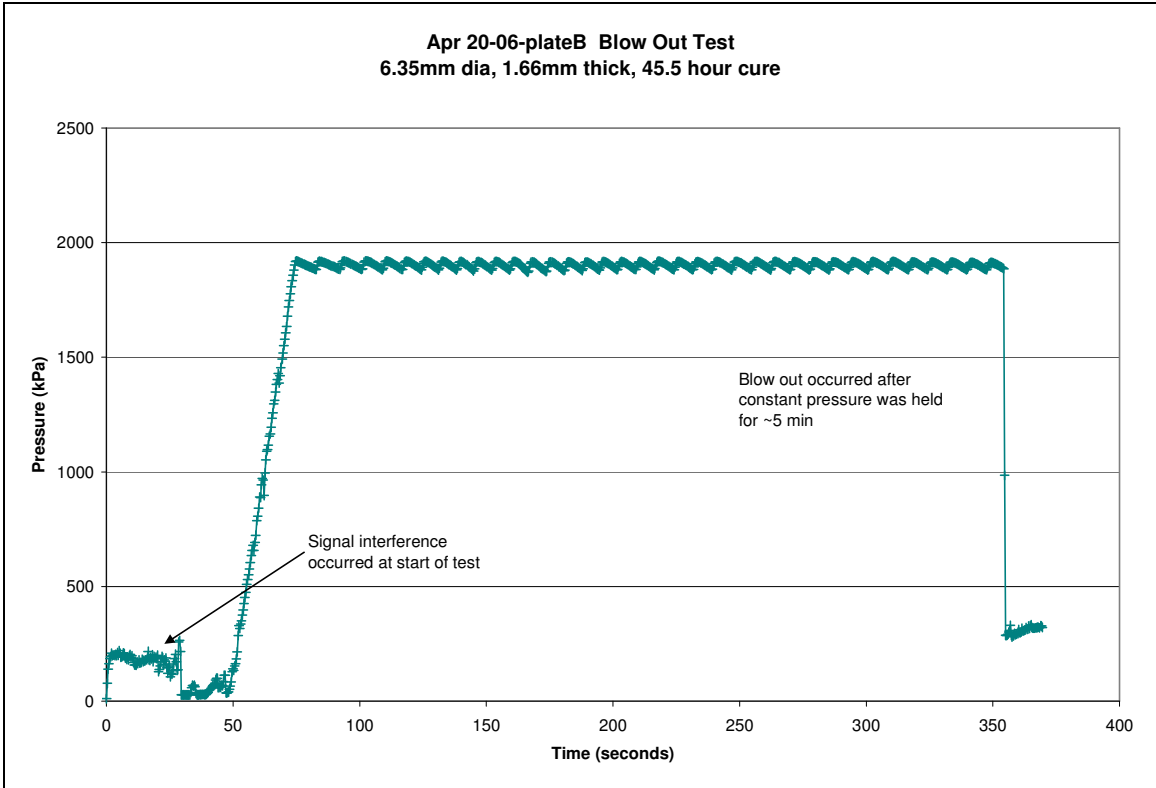
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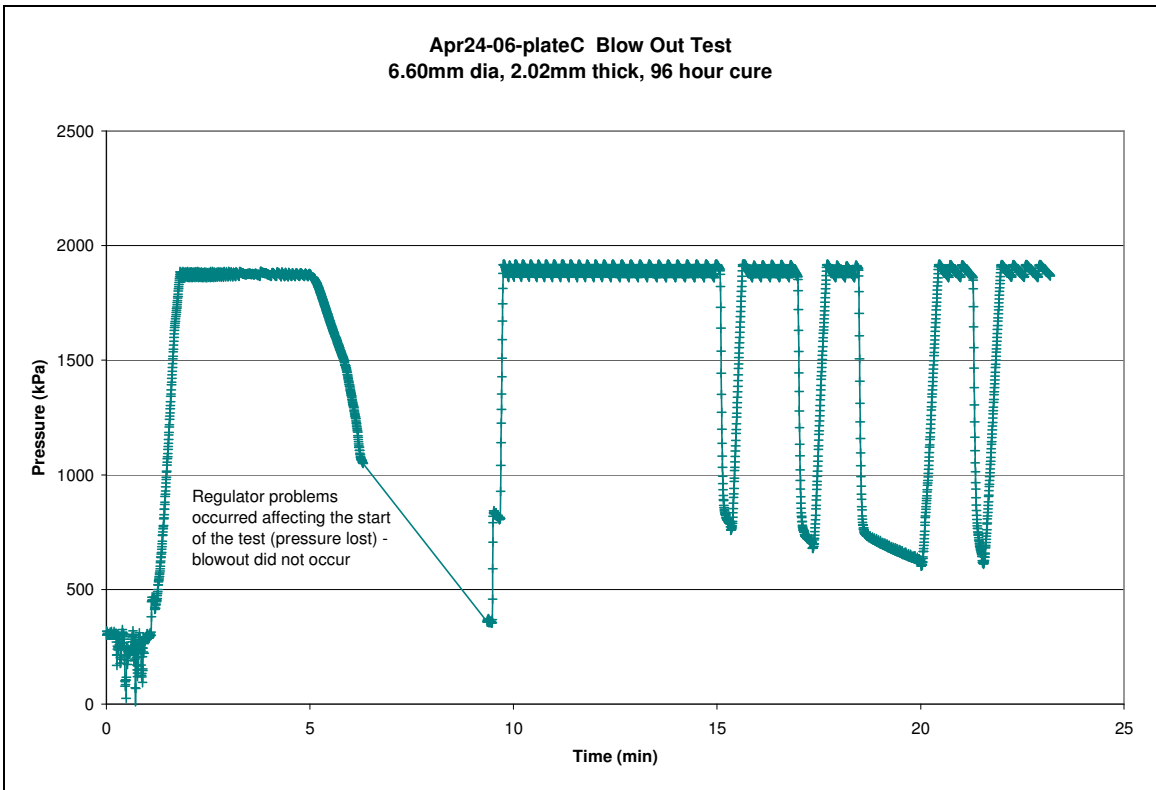
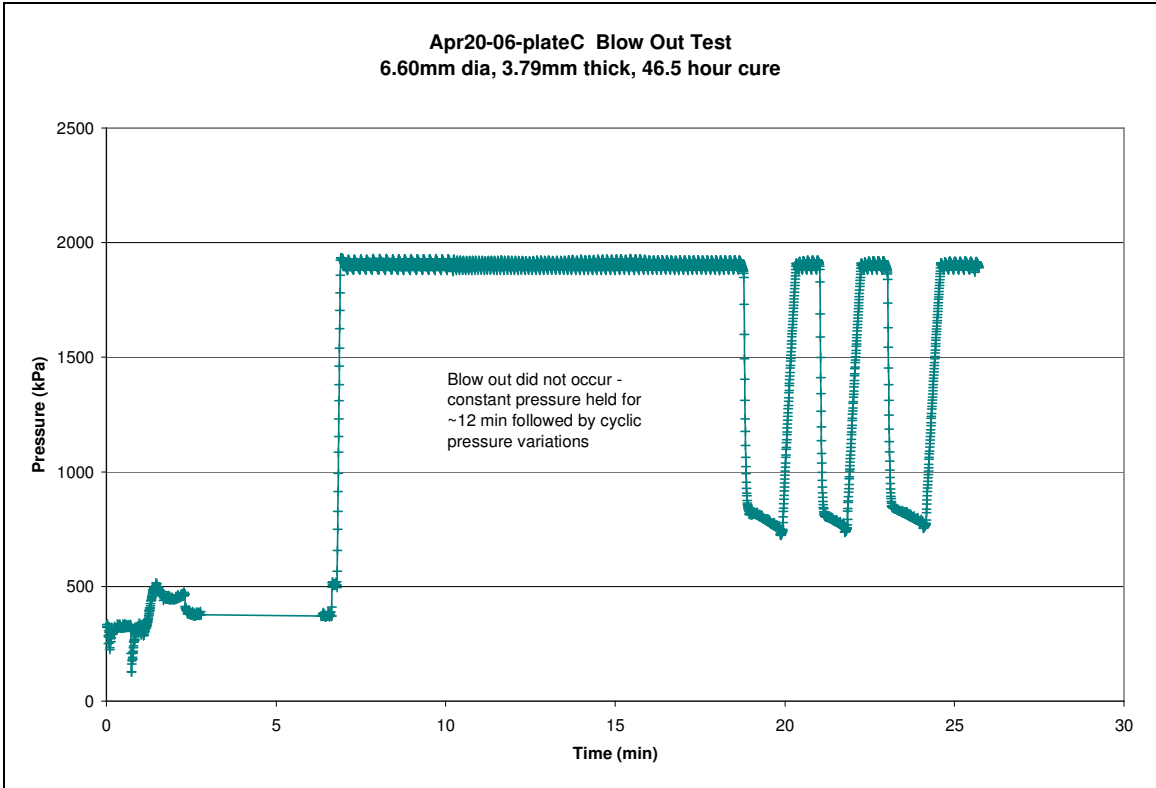


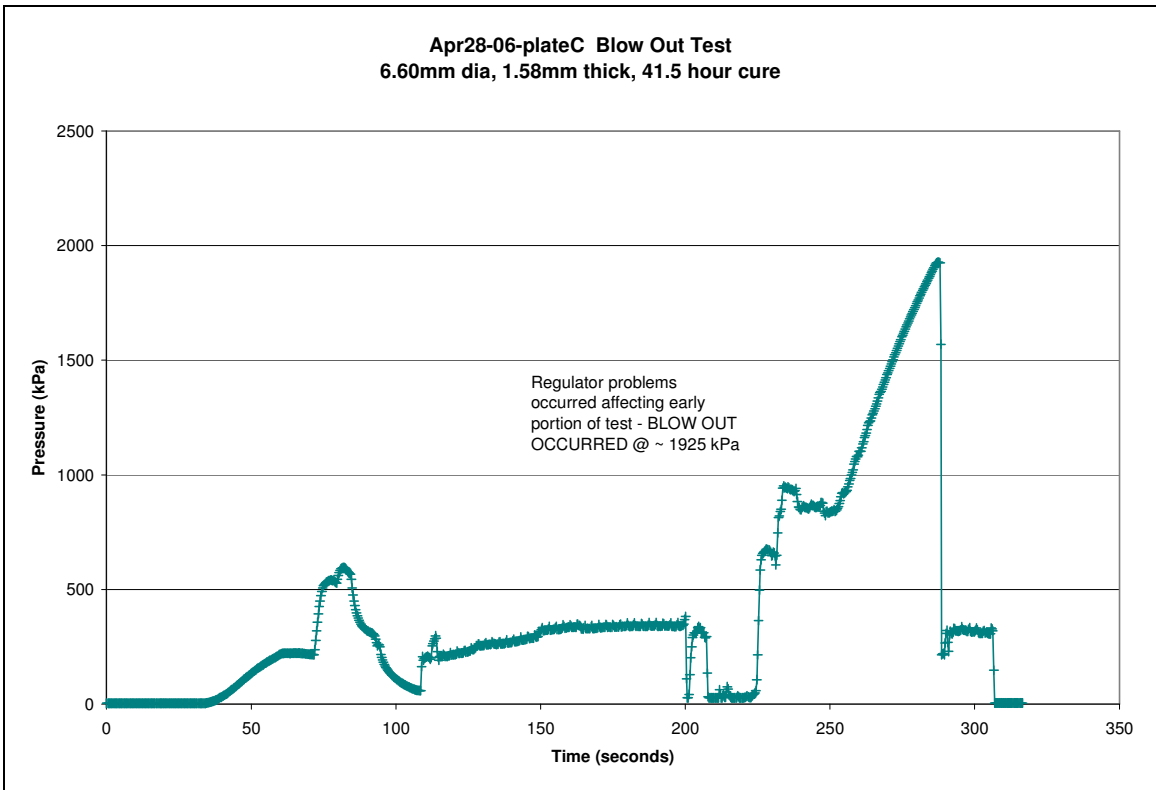
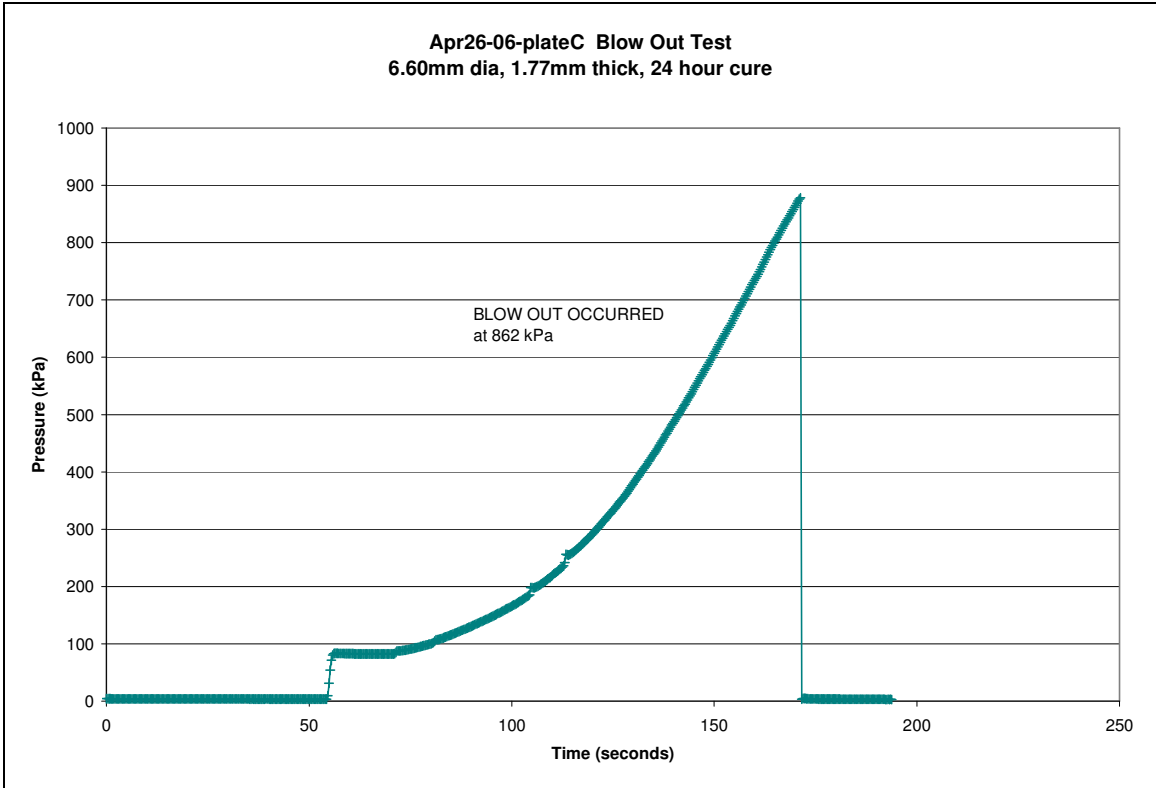
Apr24-06-plateD Blow Out Test
5.18mm dia, 1.74mm thick, 96 hour cure

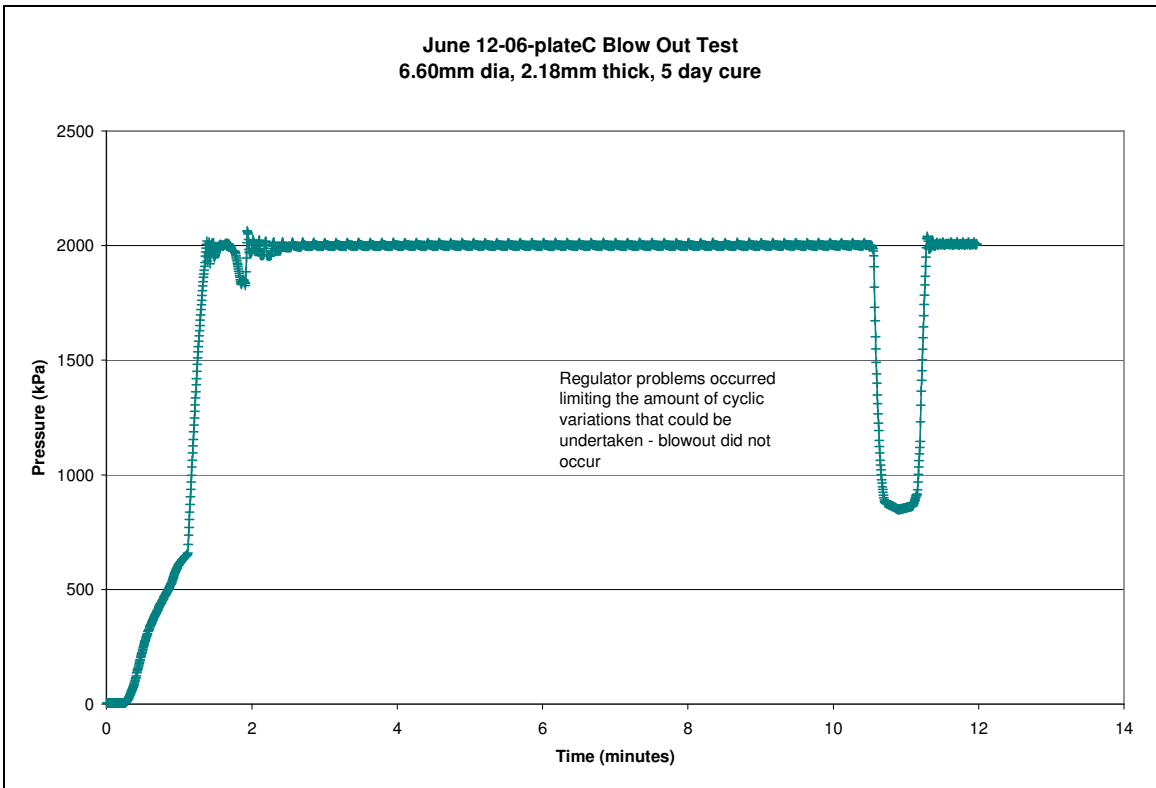
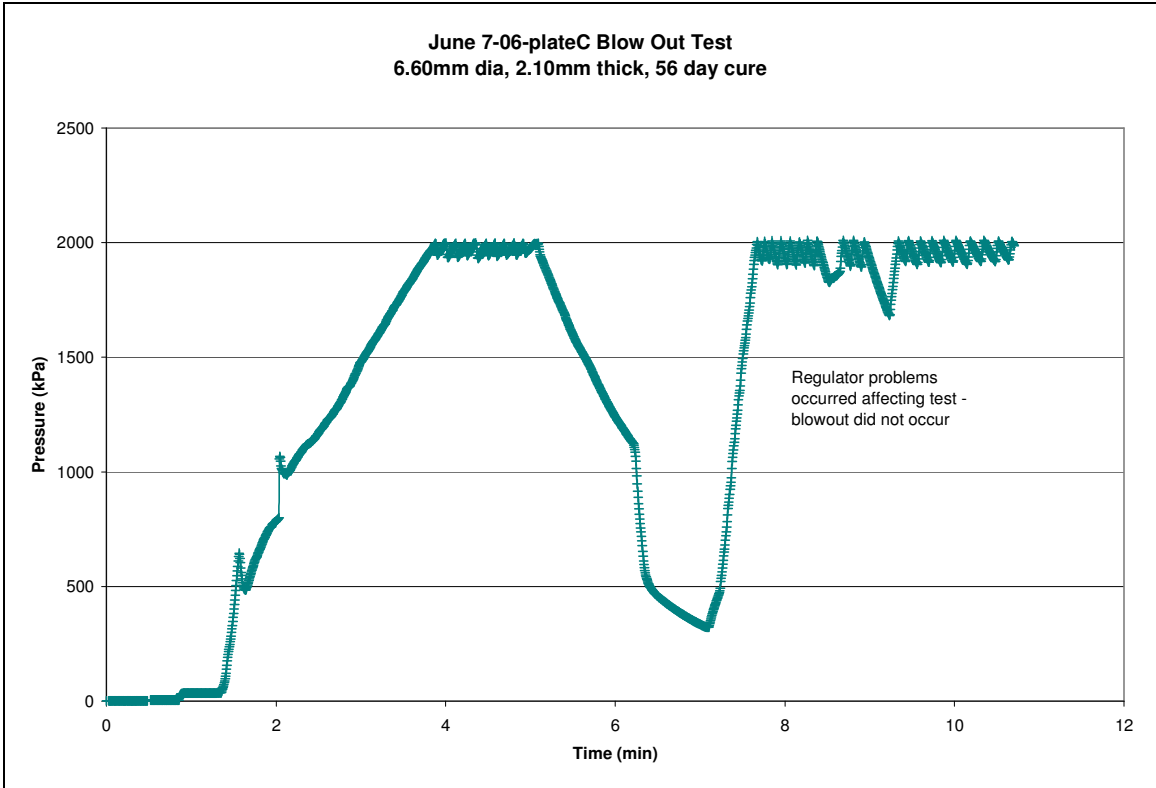


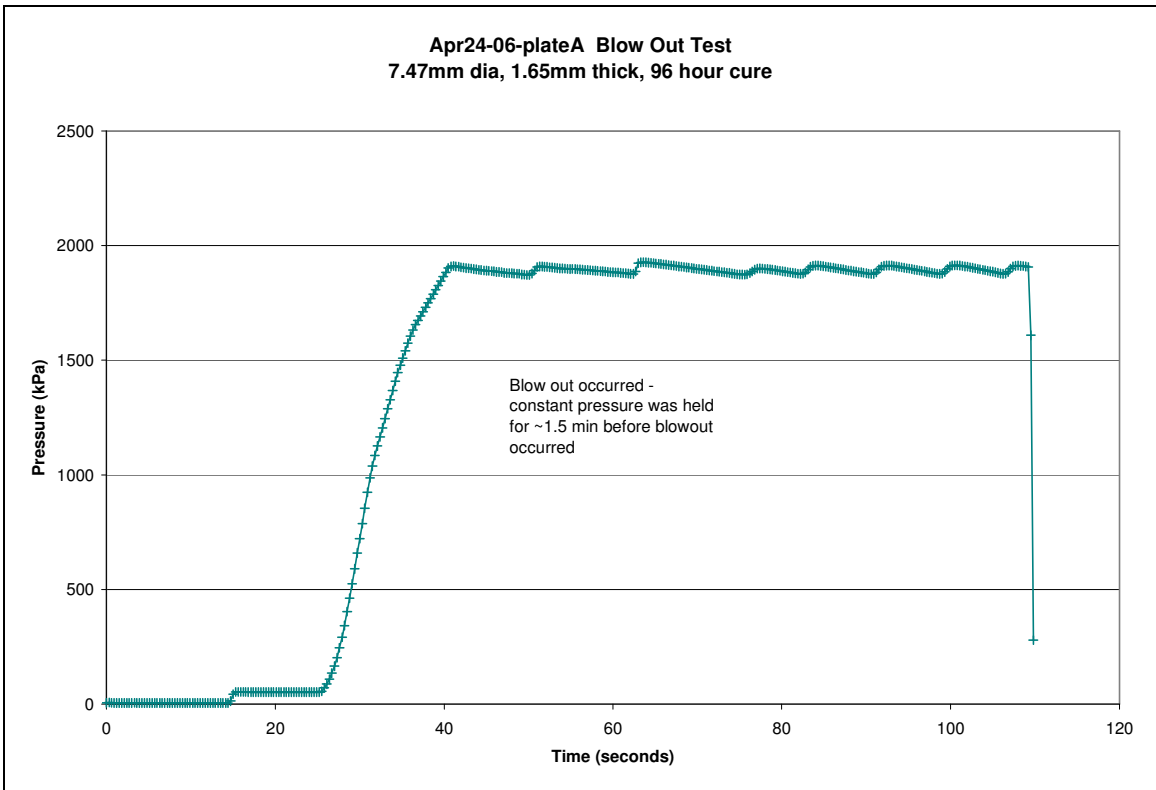
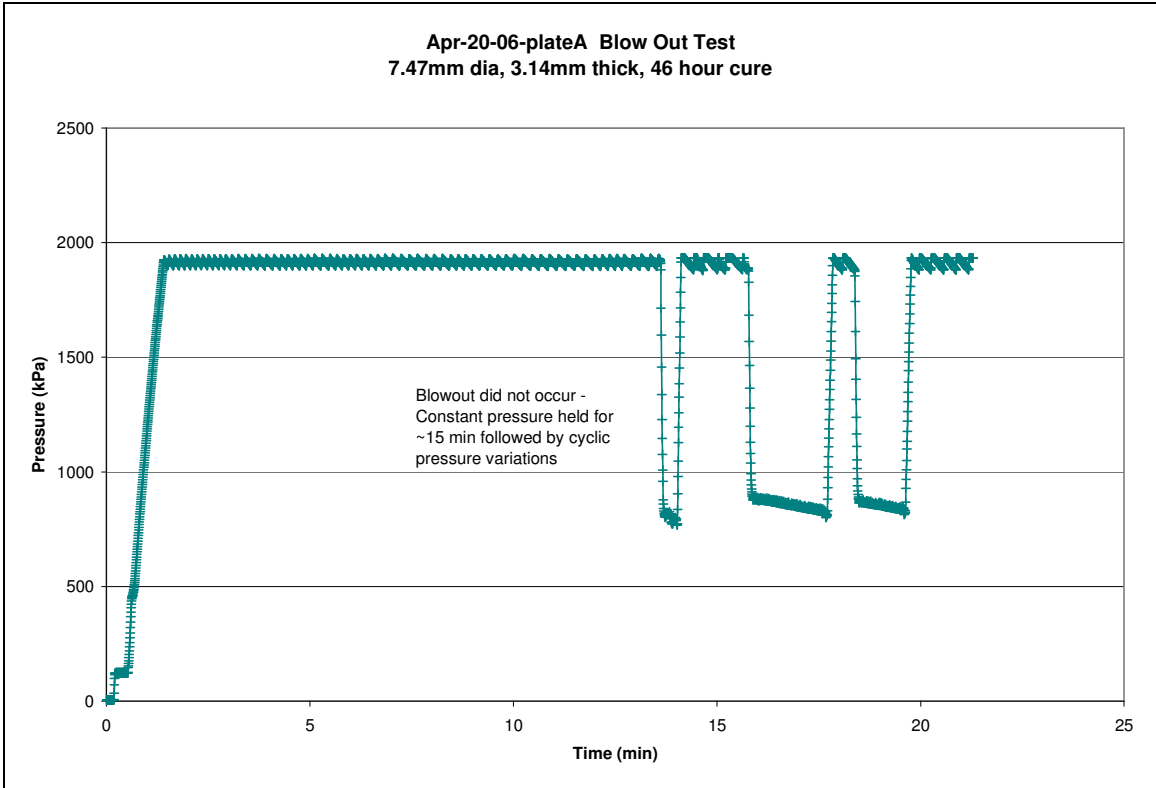


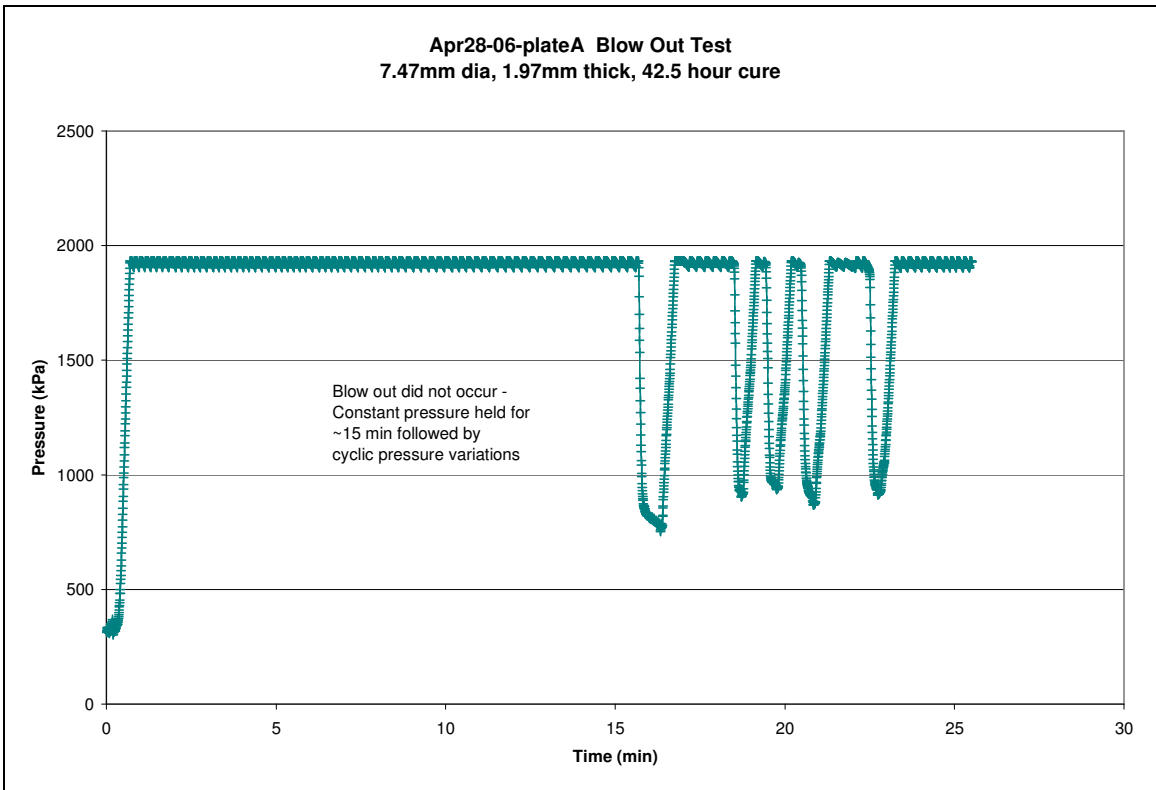
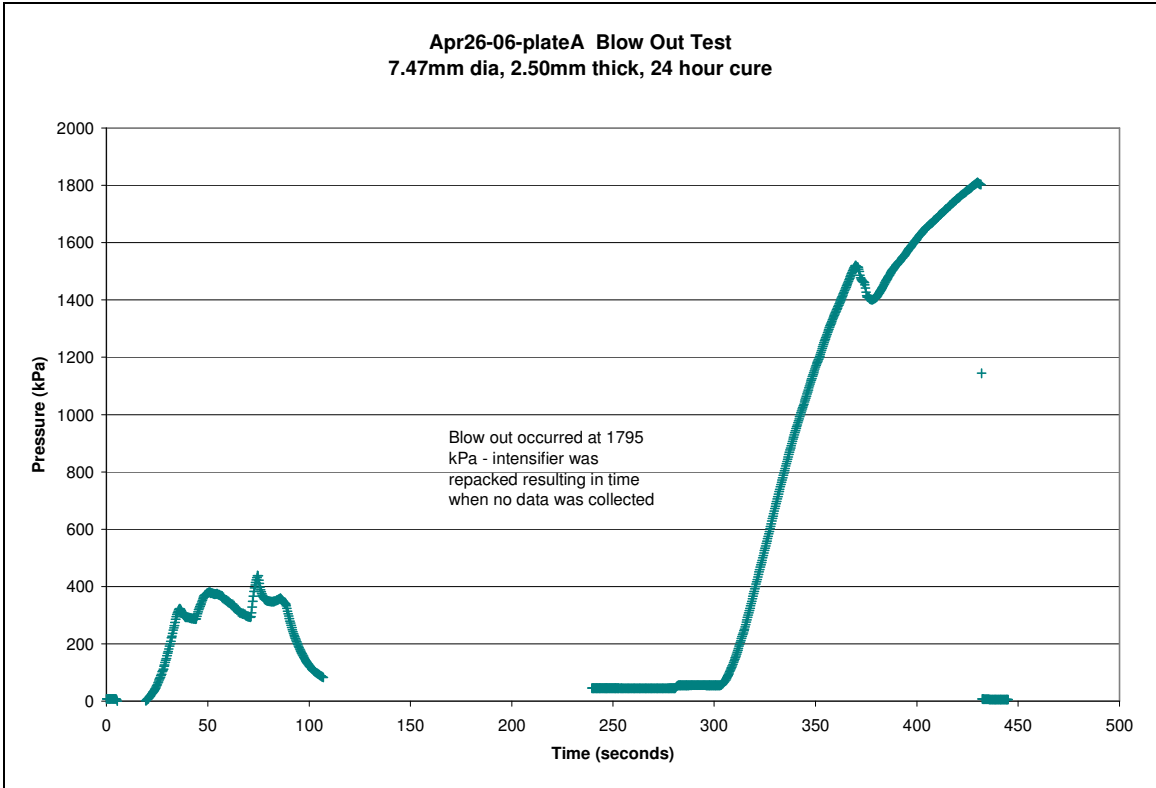


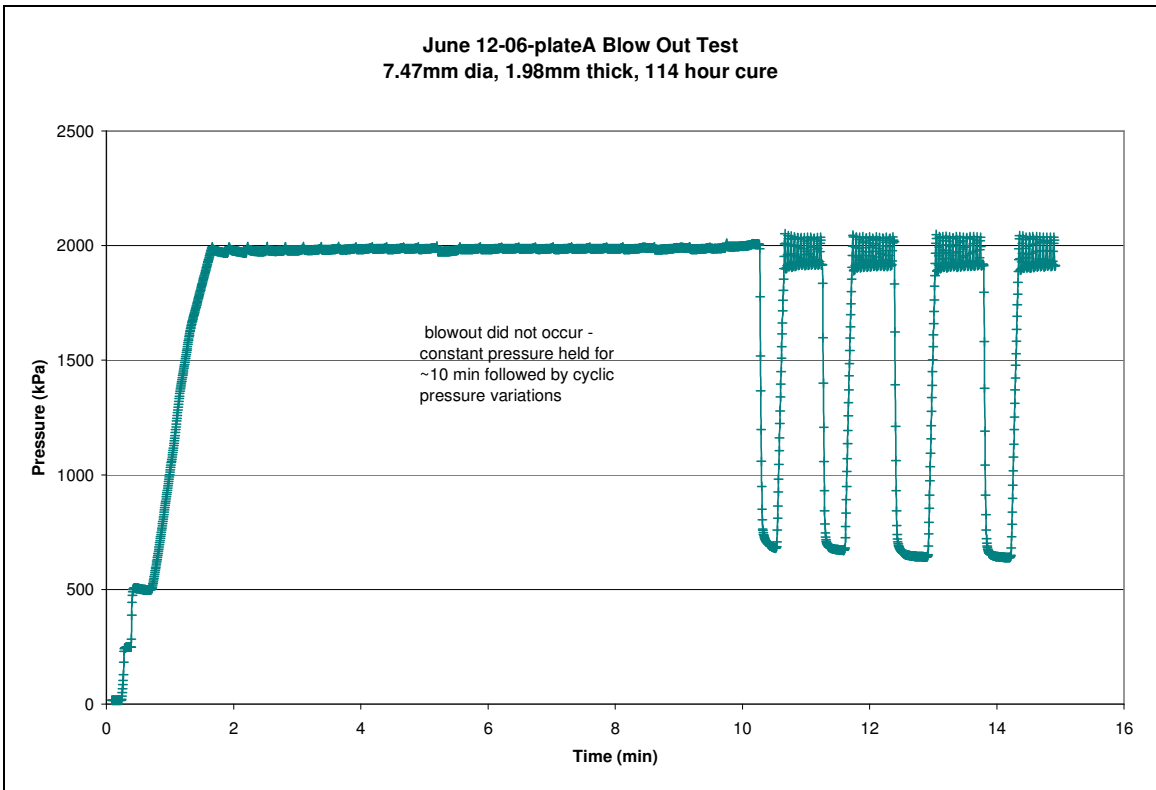
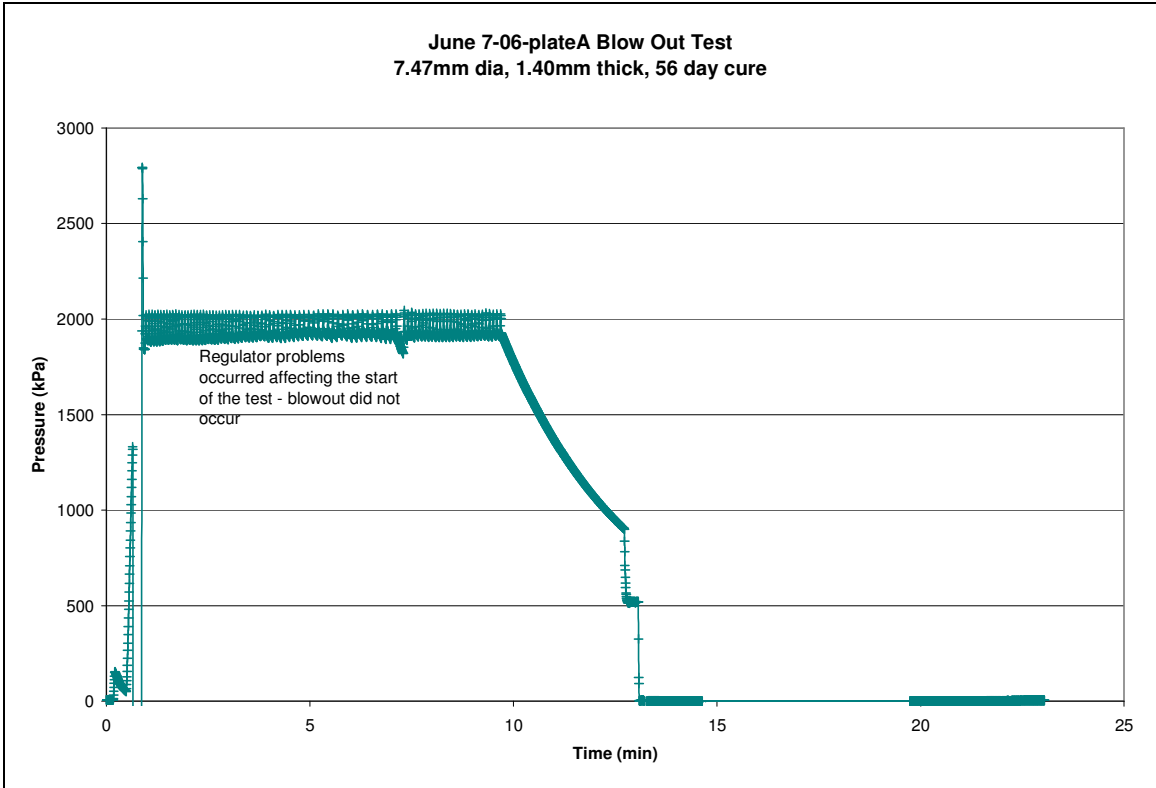




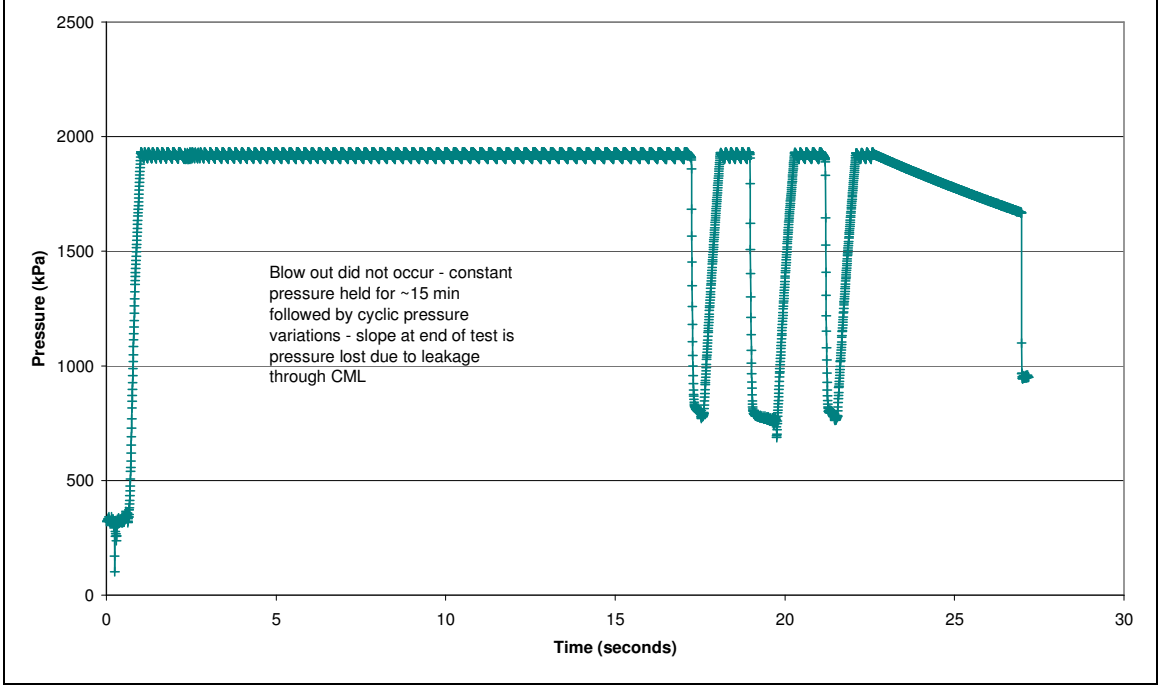




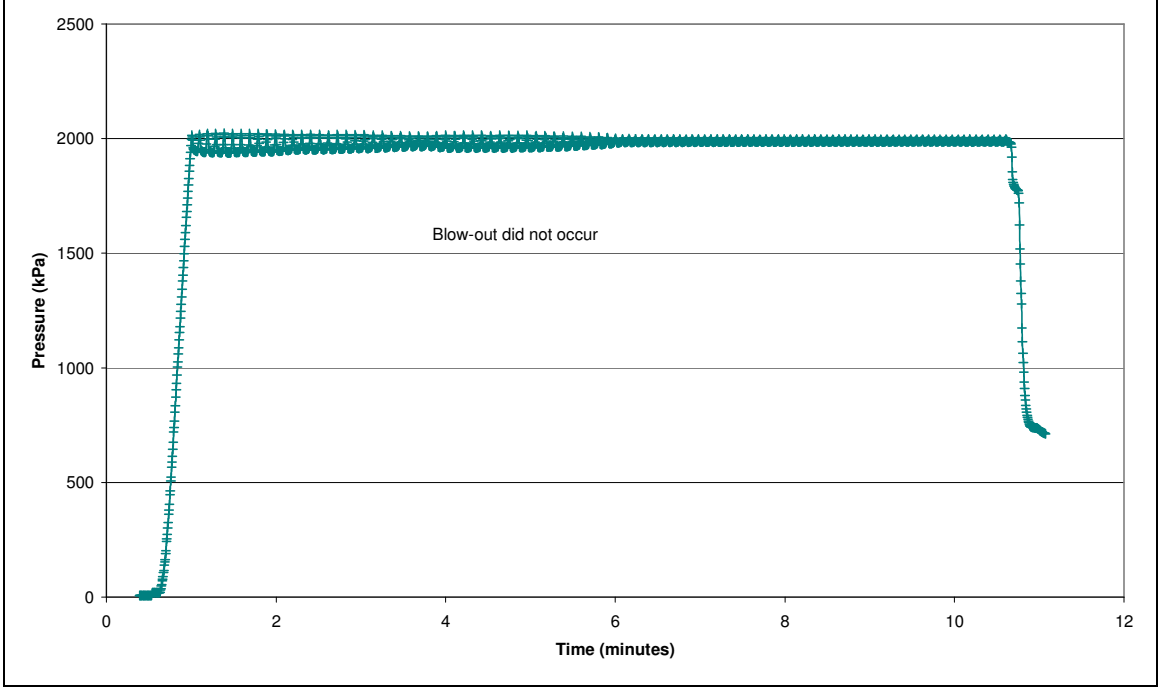


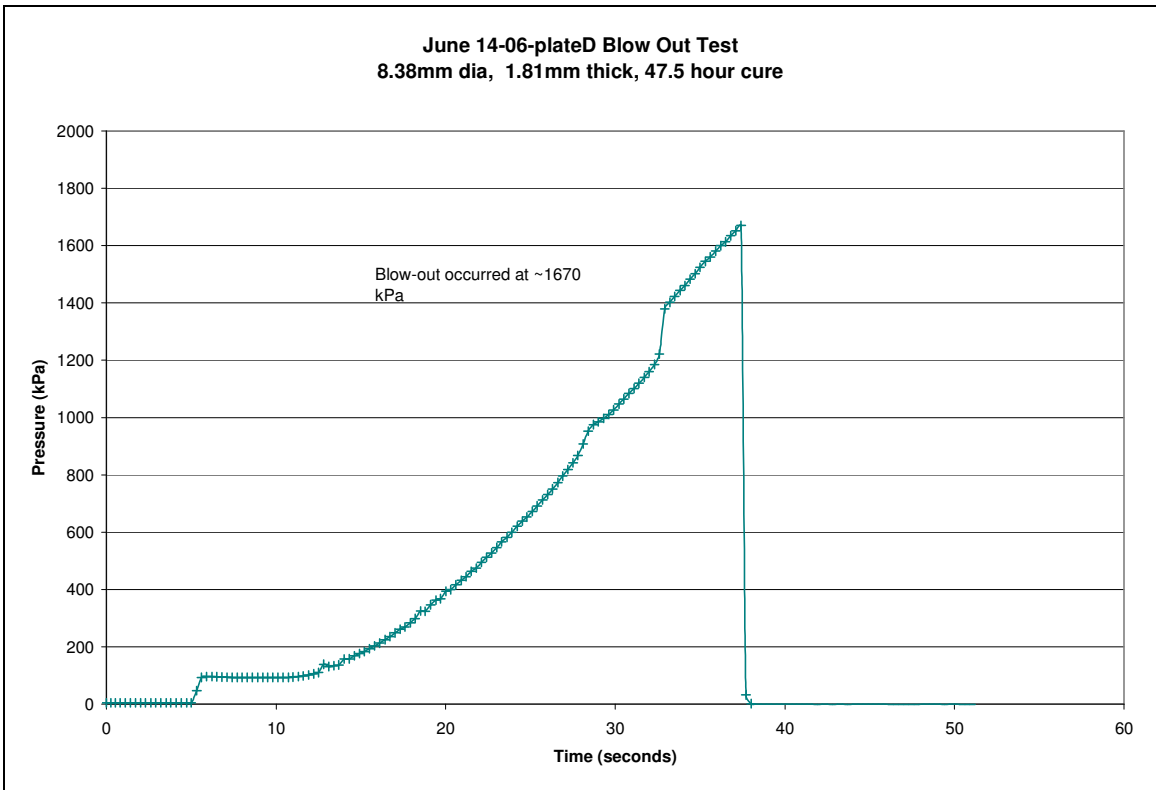
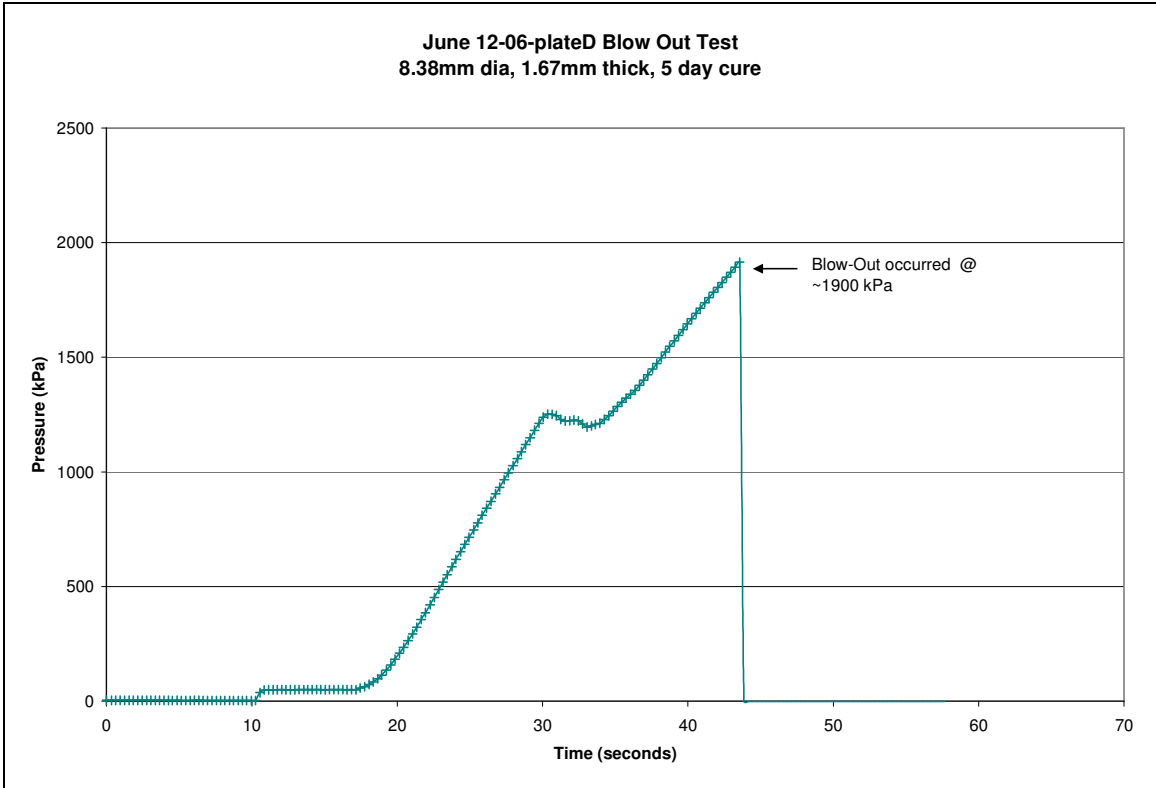


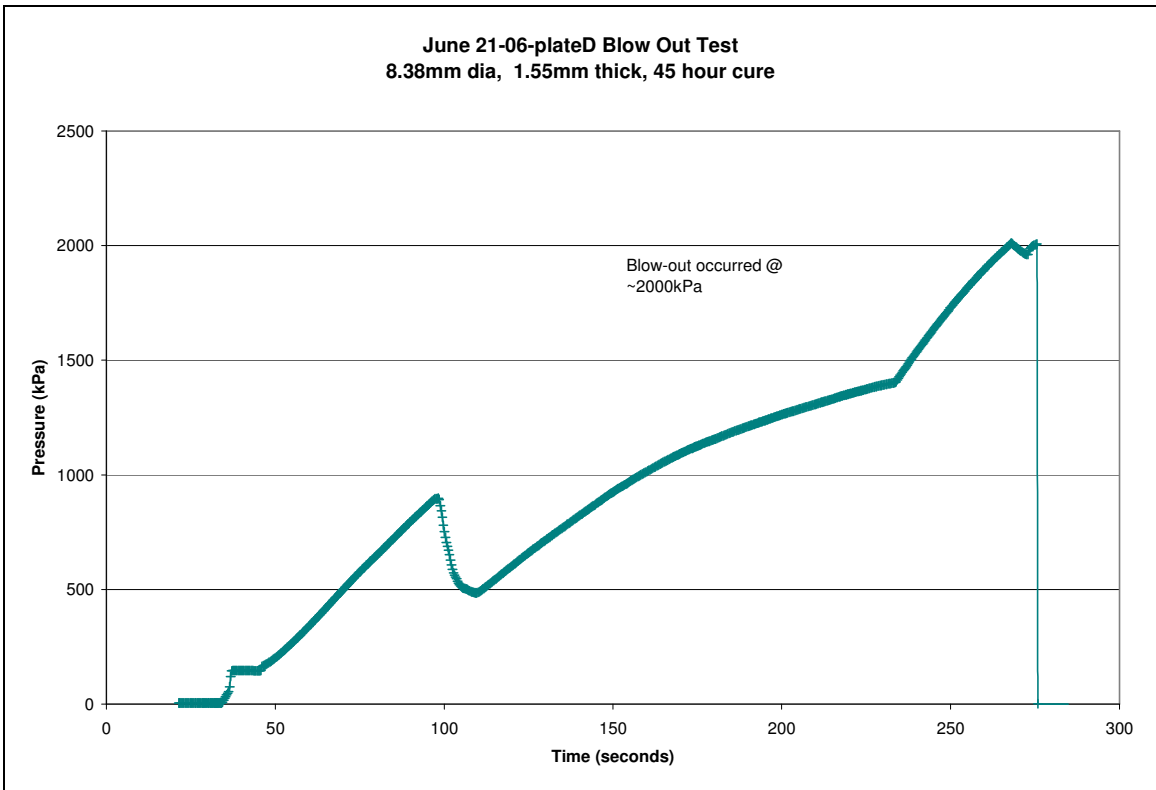
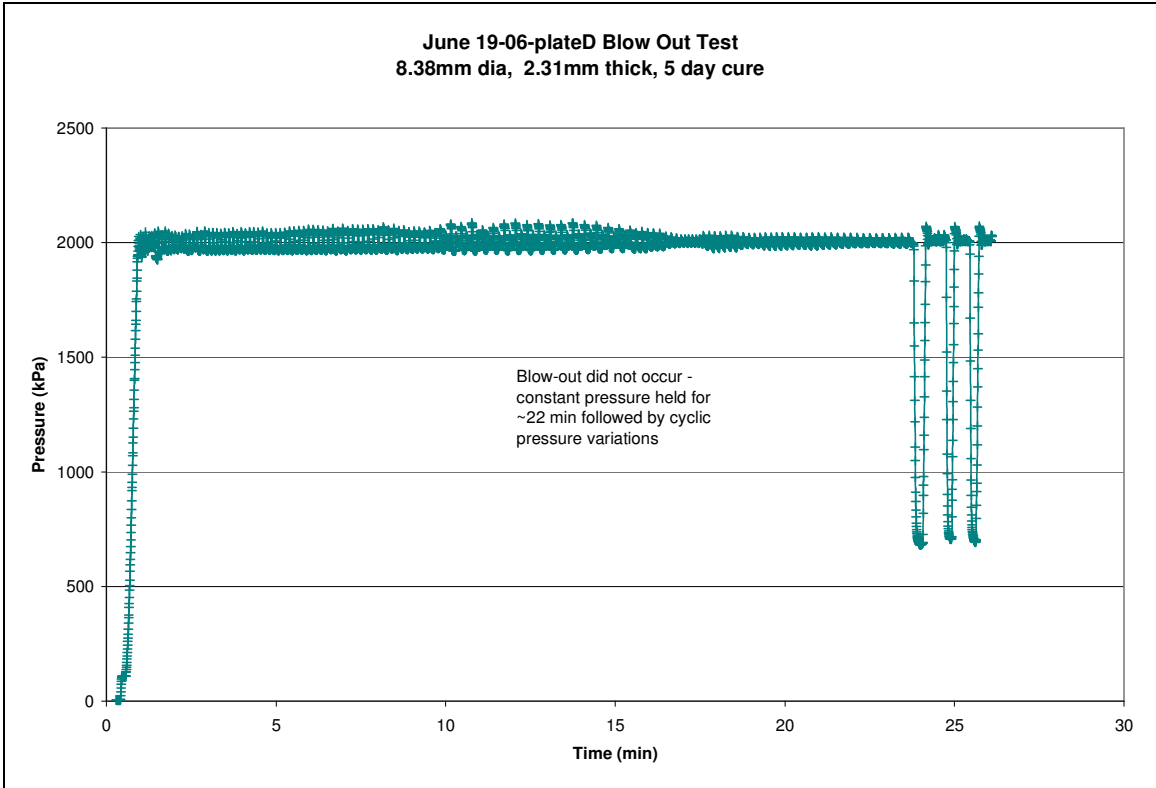
Apr28-06-plateD Blow Out Test
8.38mm dia, 2.21mm thick, 43.5 hour cure

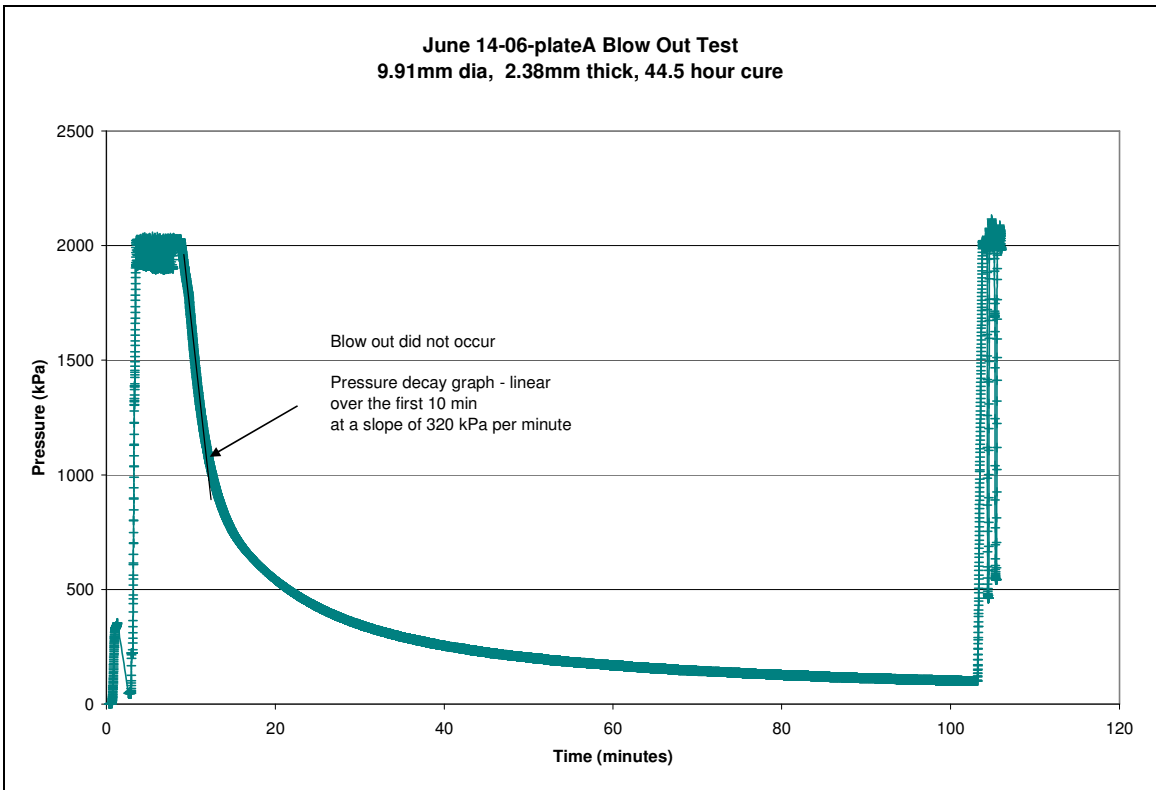
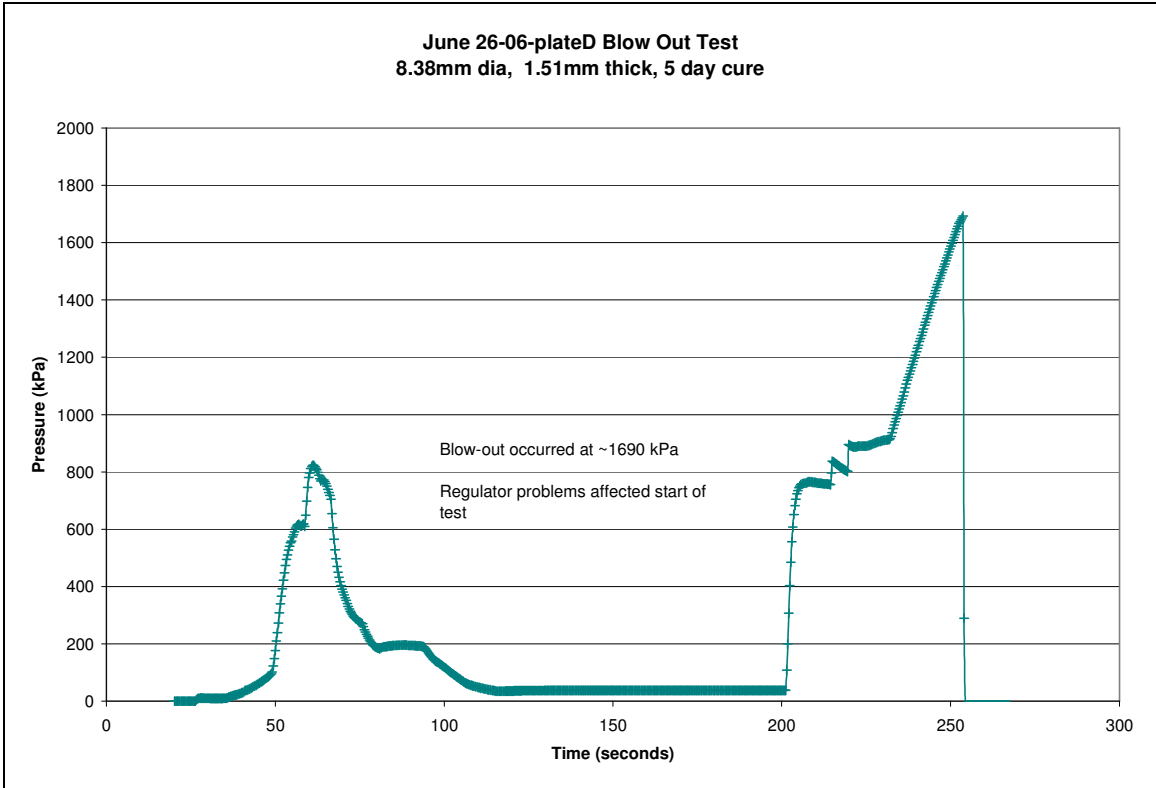


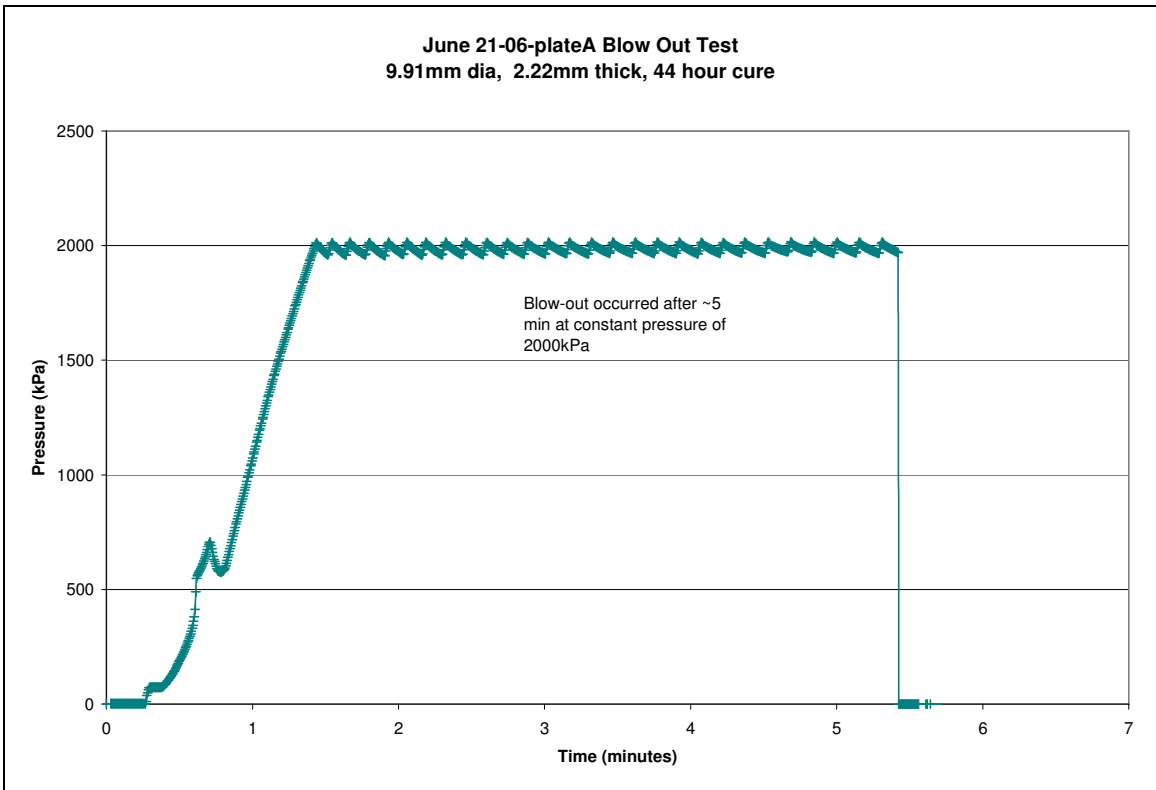
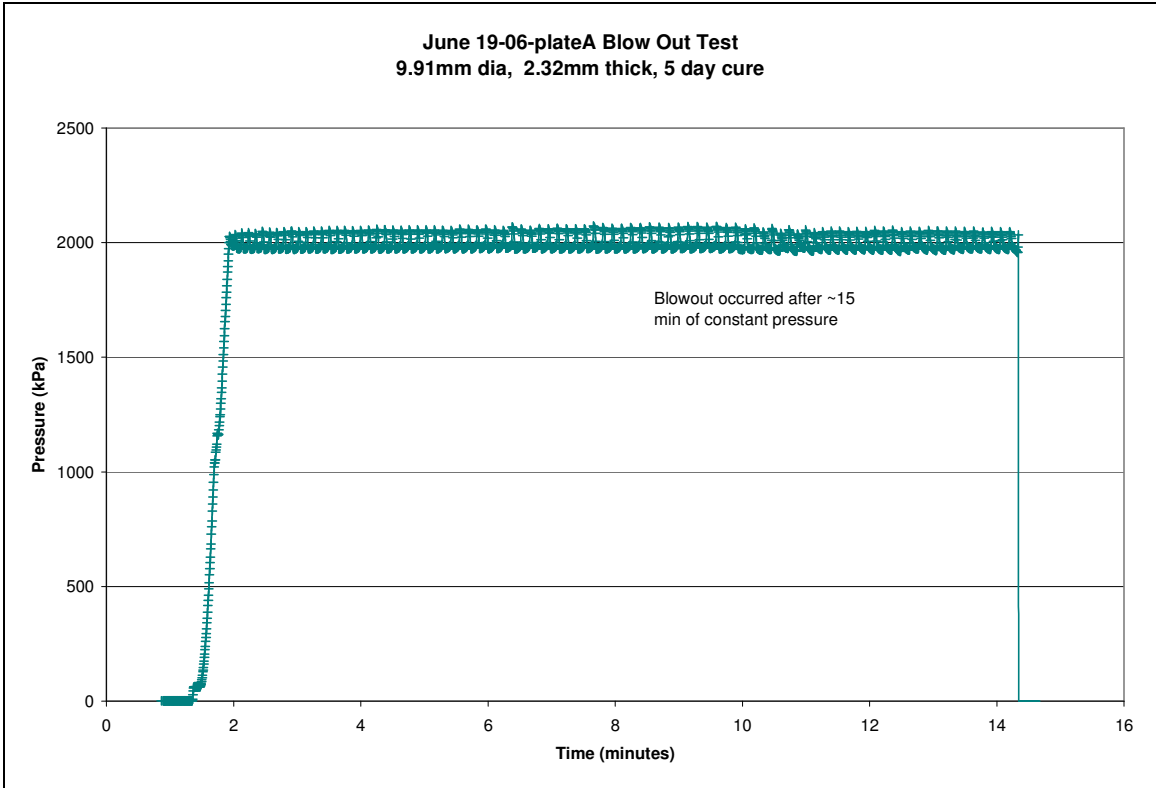
June 7-06-plateD Blow Out Test
8.38mm dia, 1.56mm thick, 56 day cure

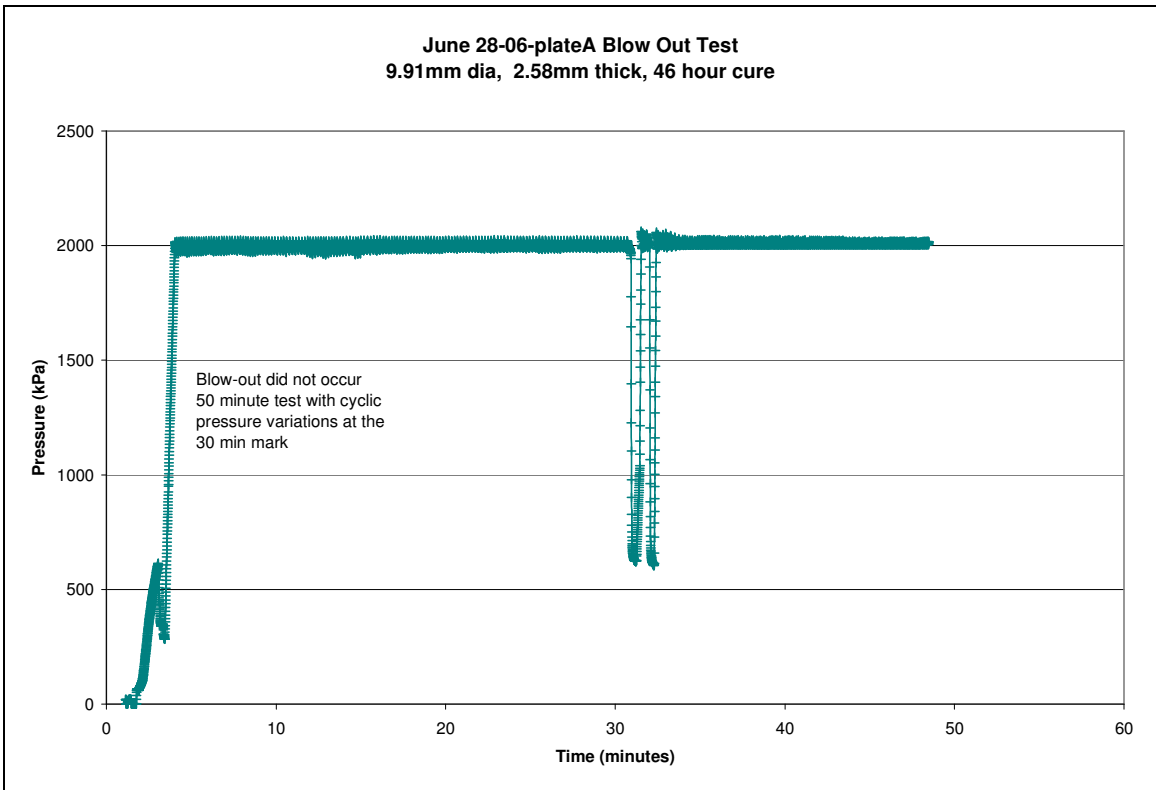
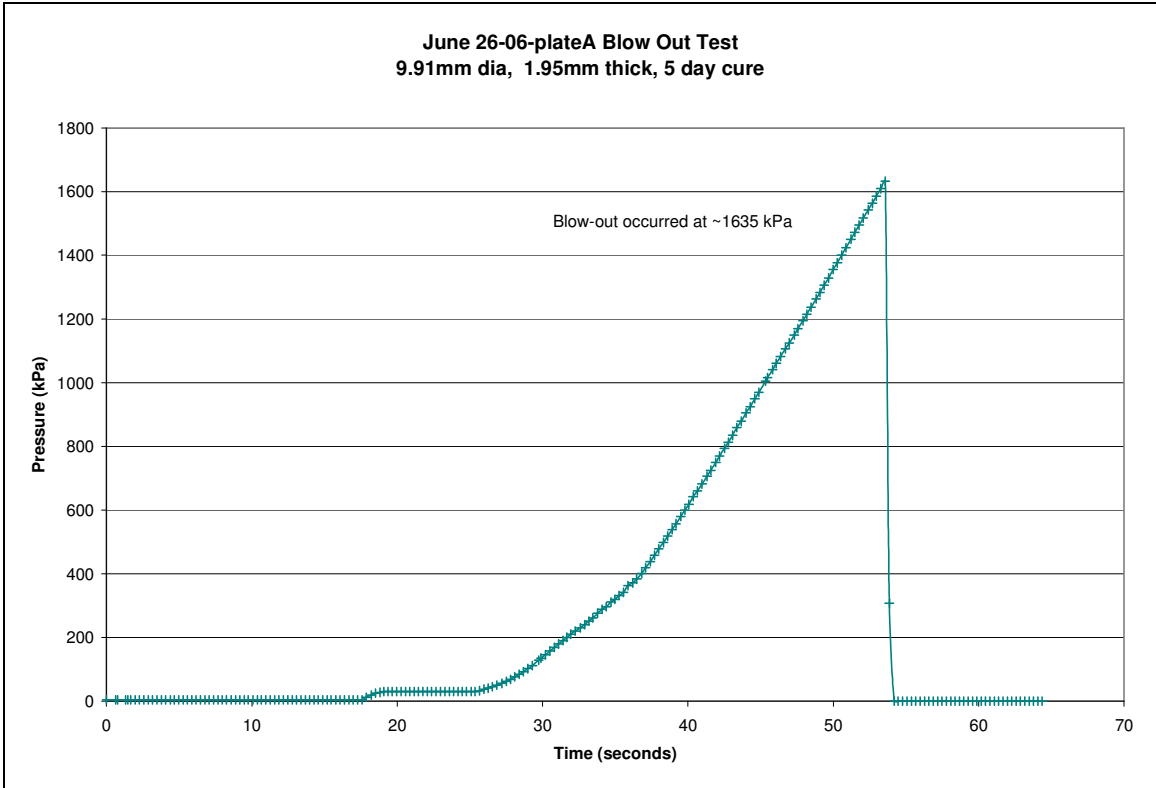


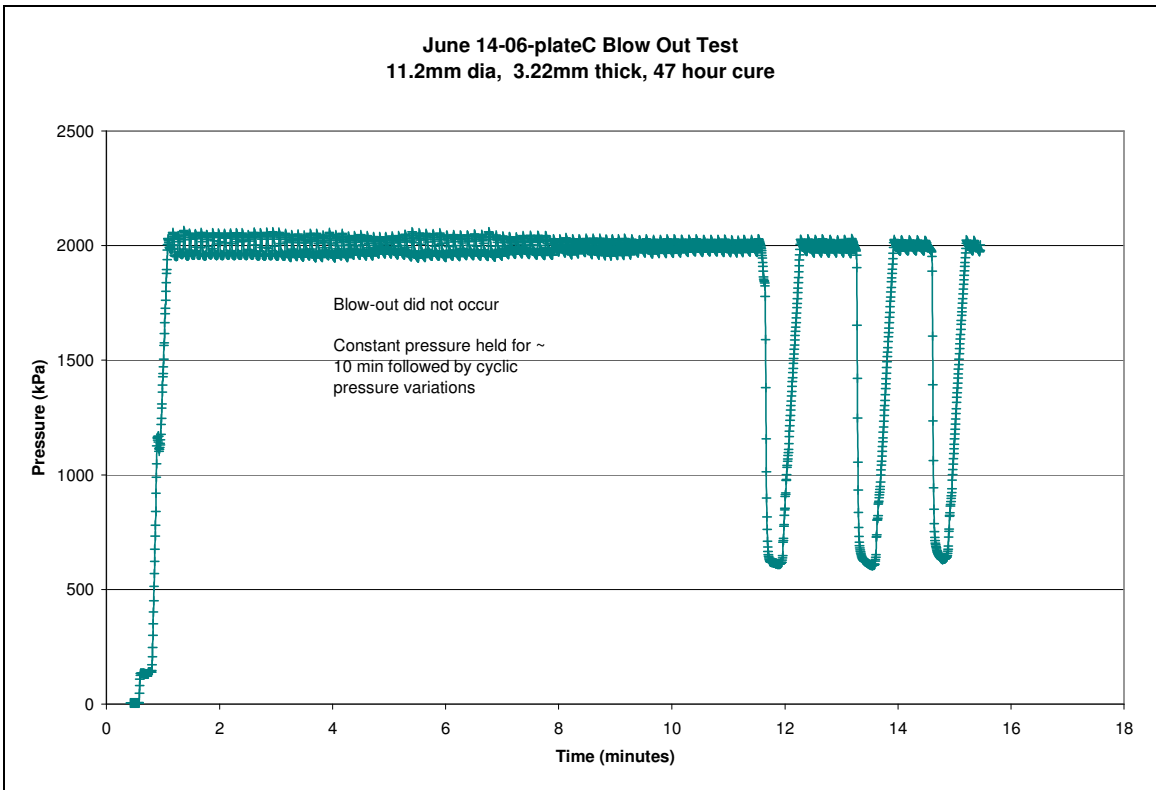
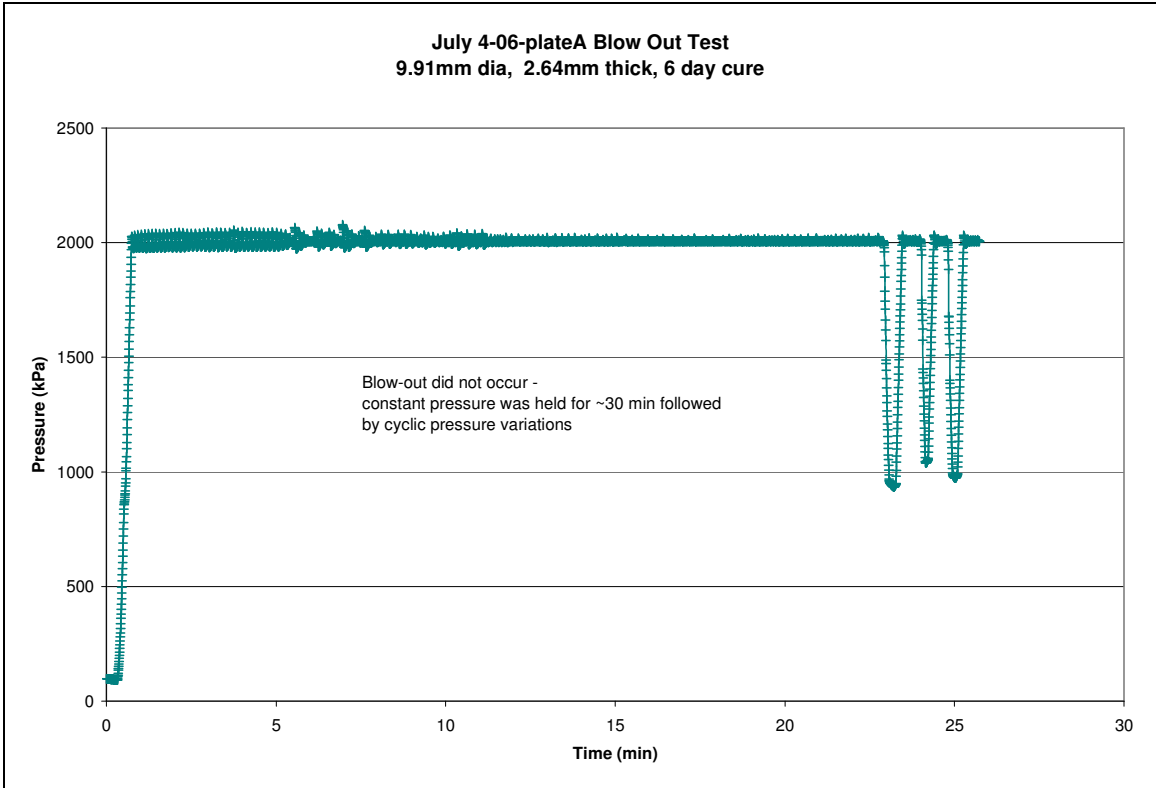


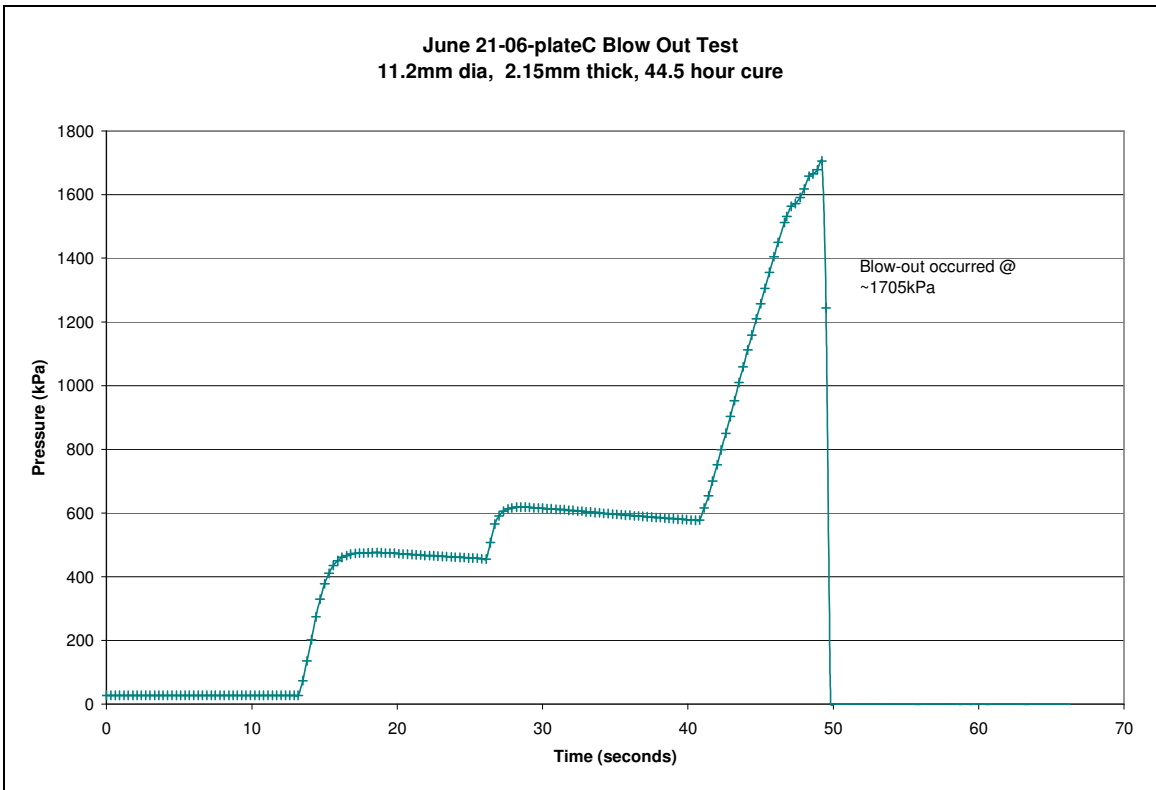
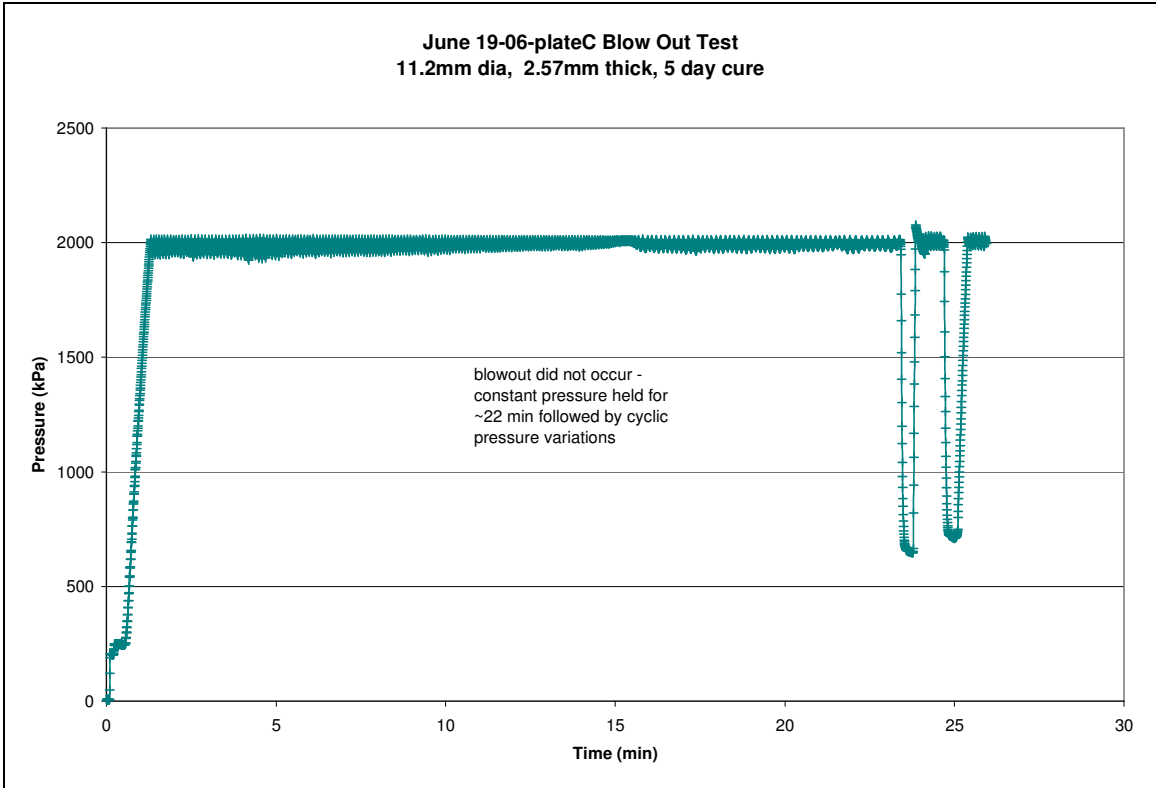


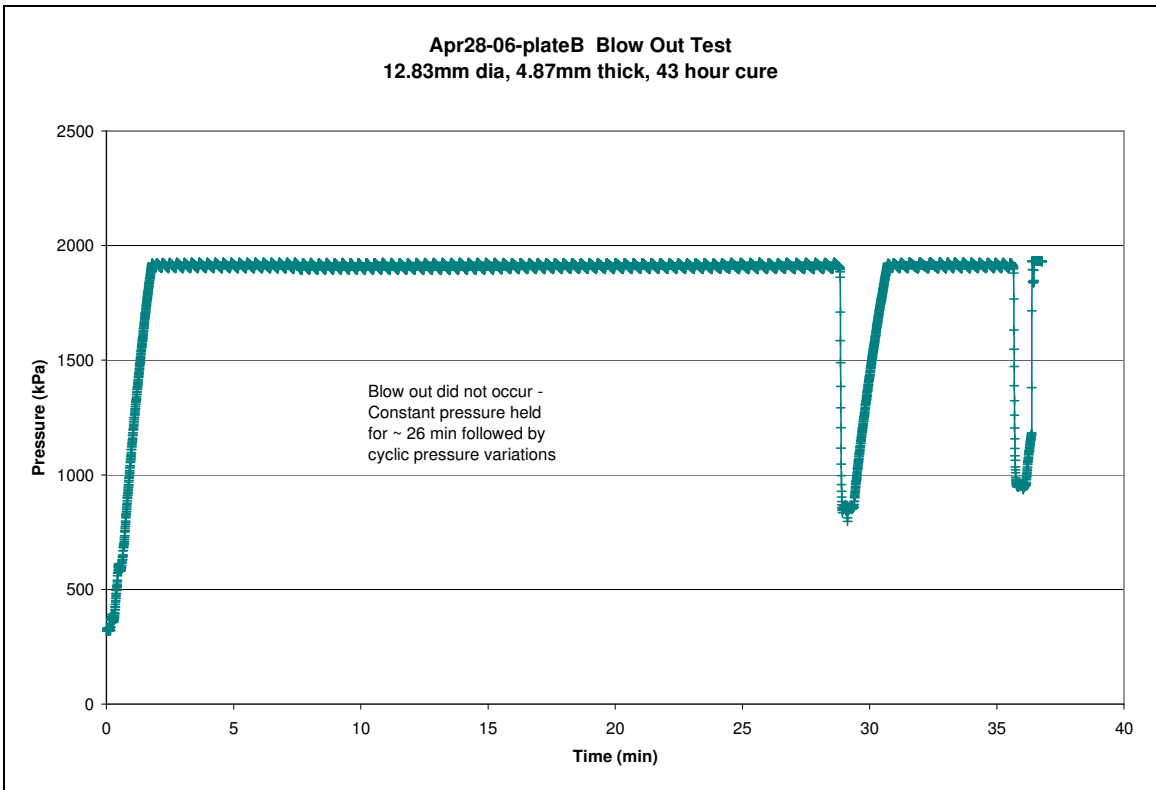
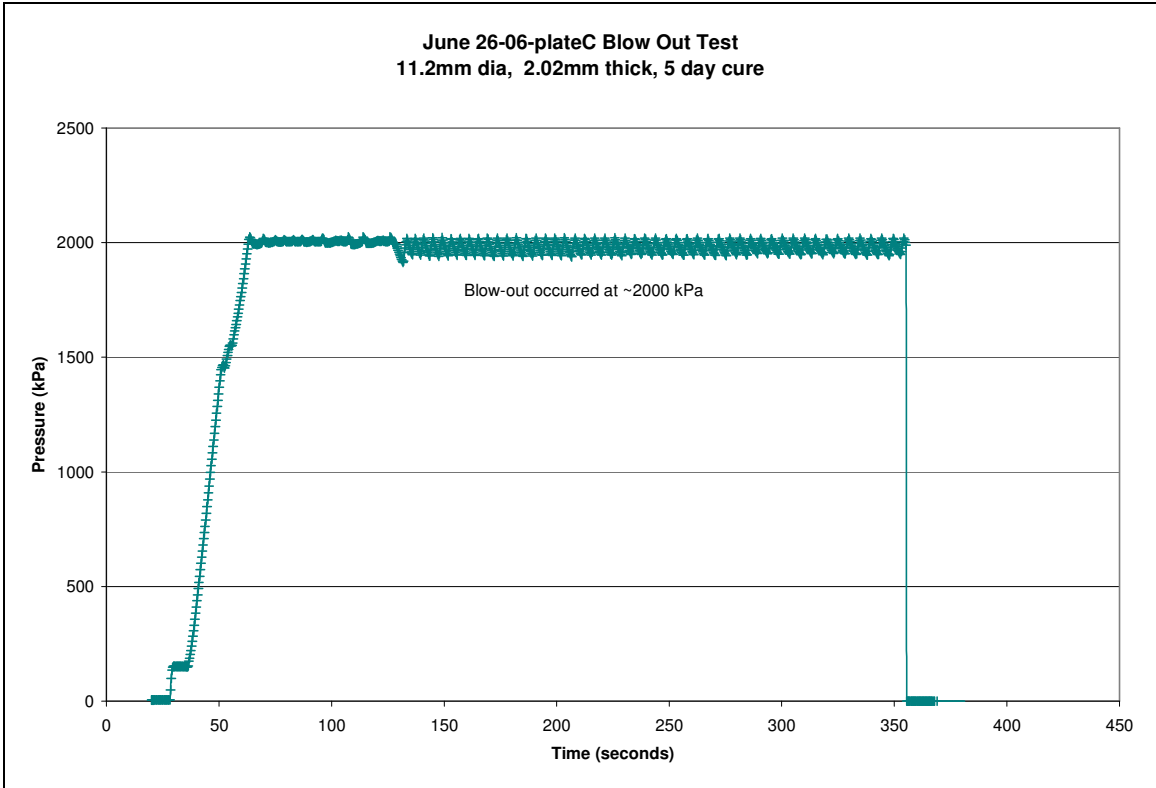


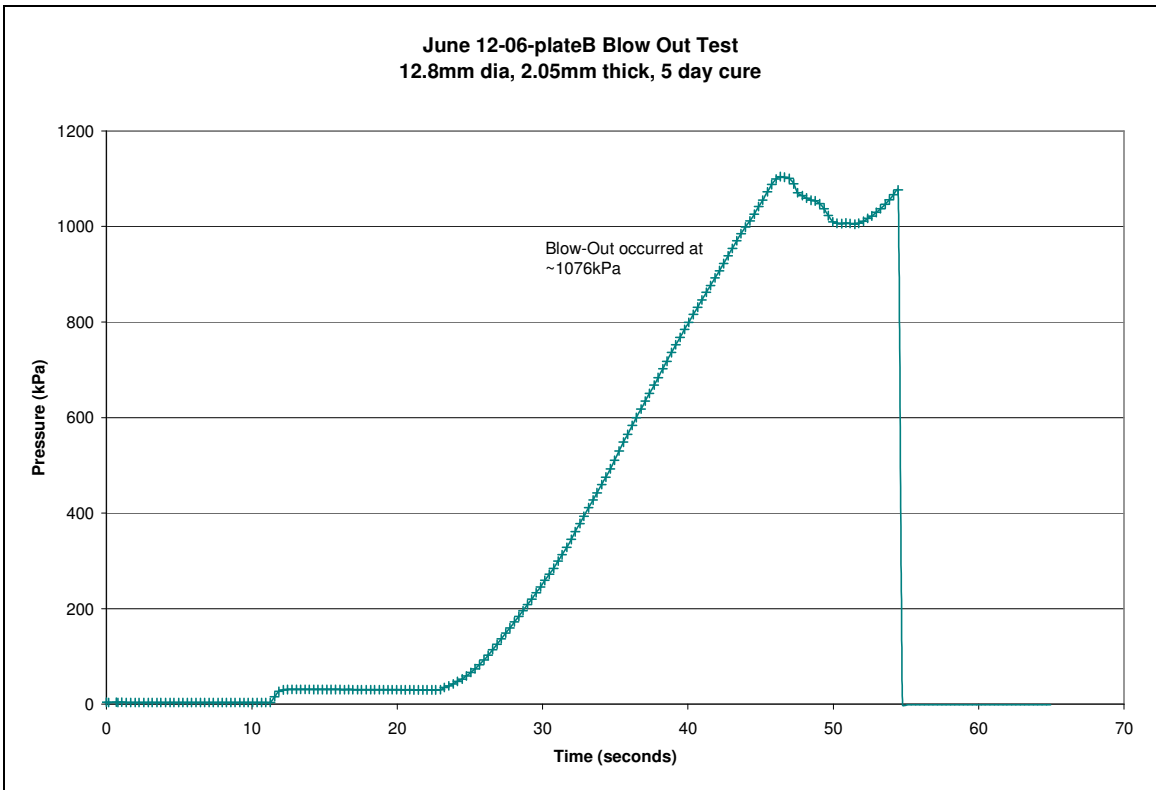
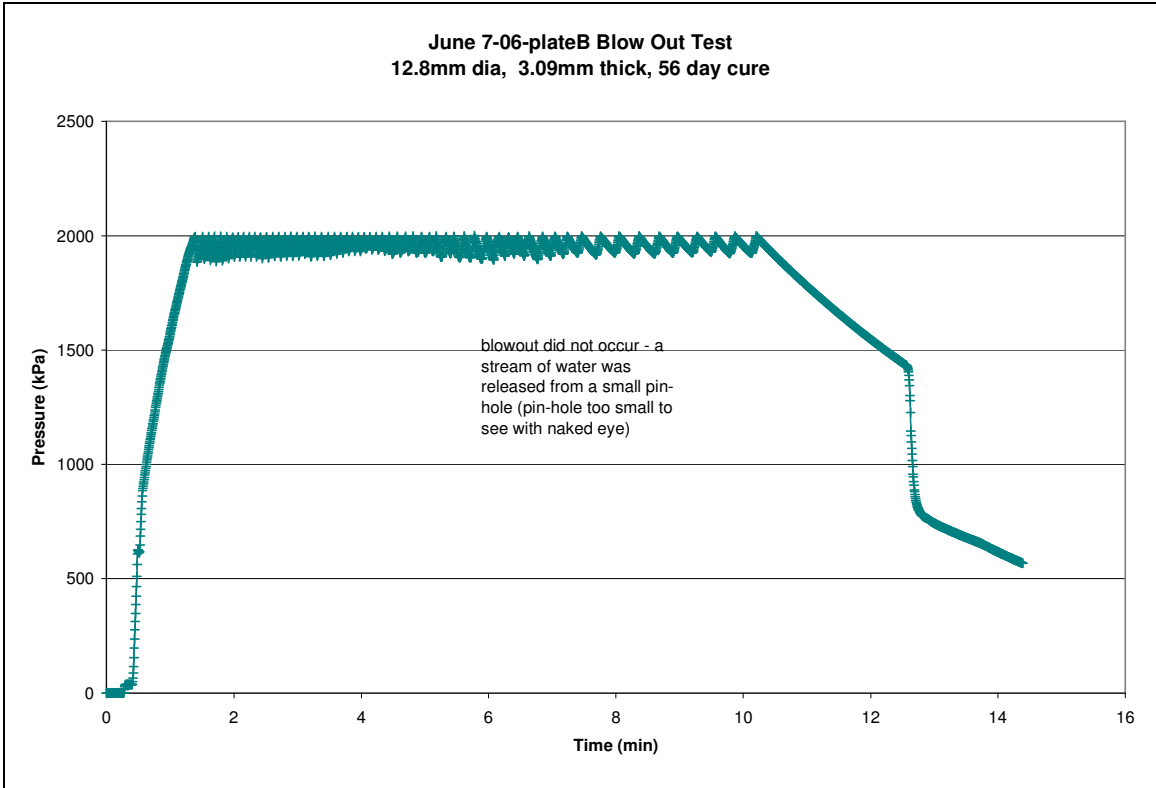


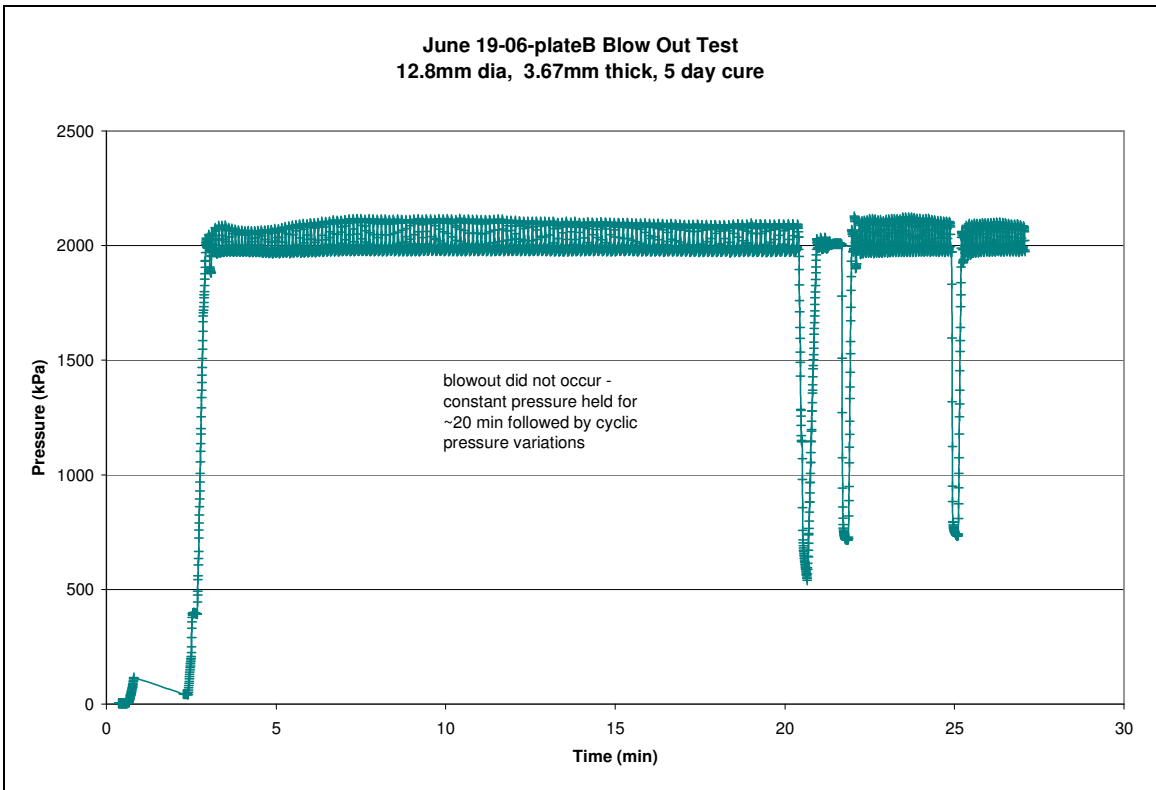
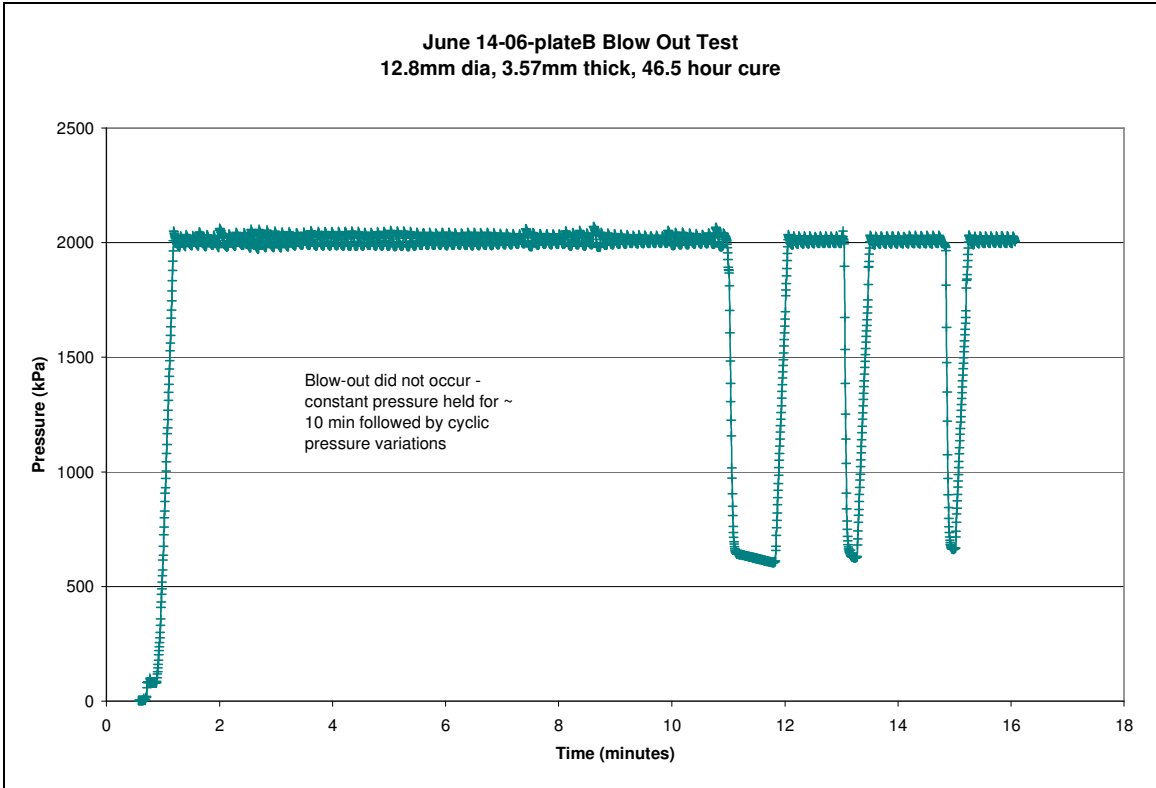


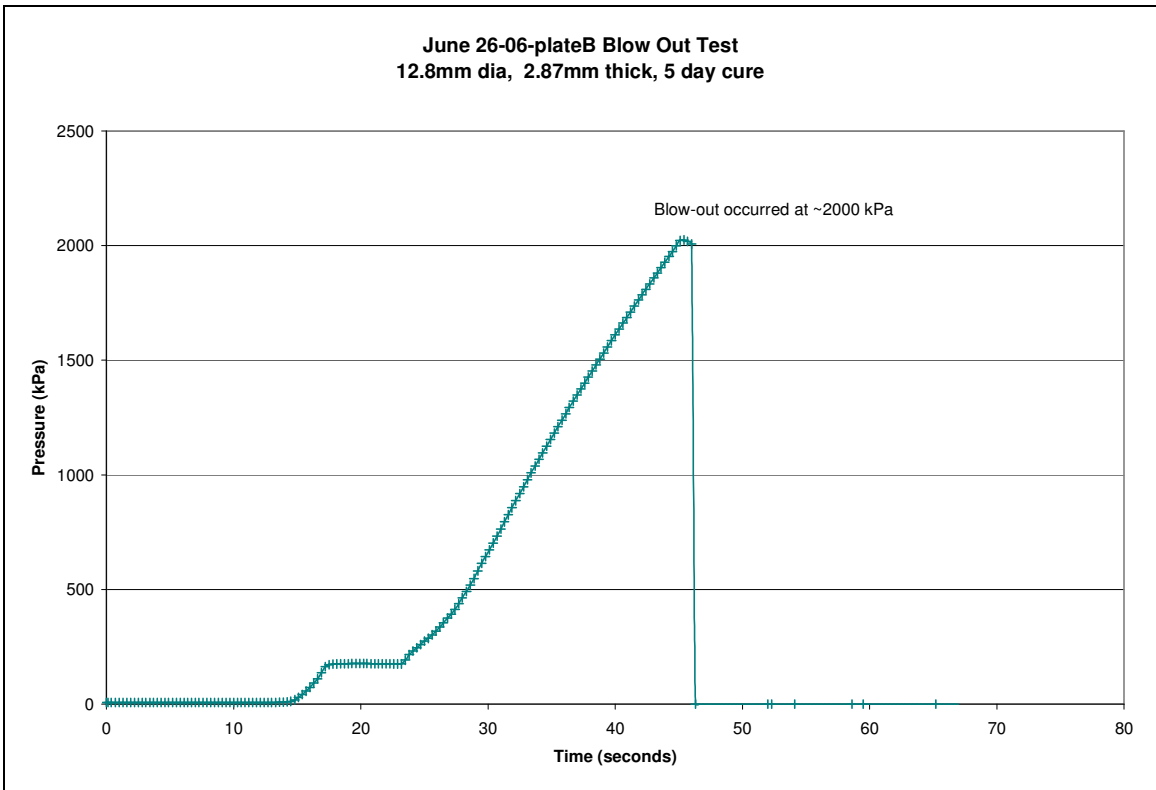
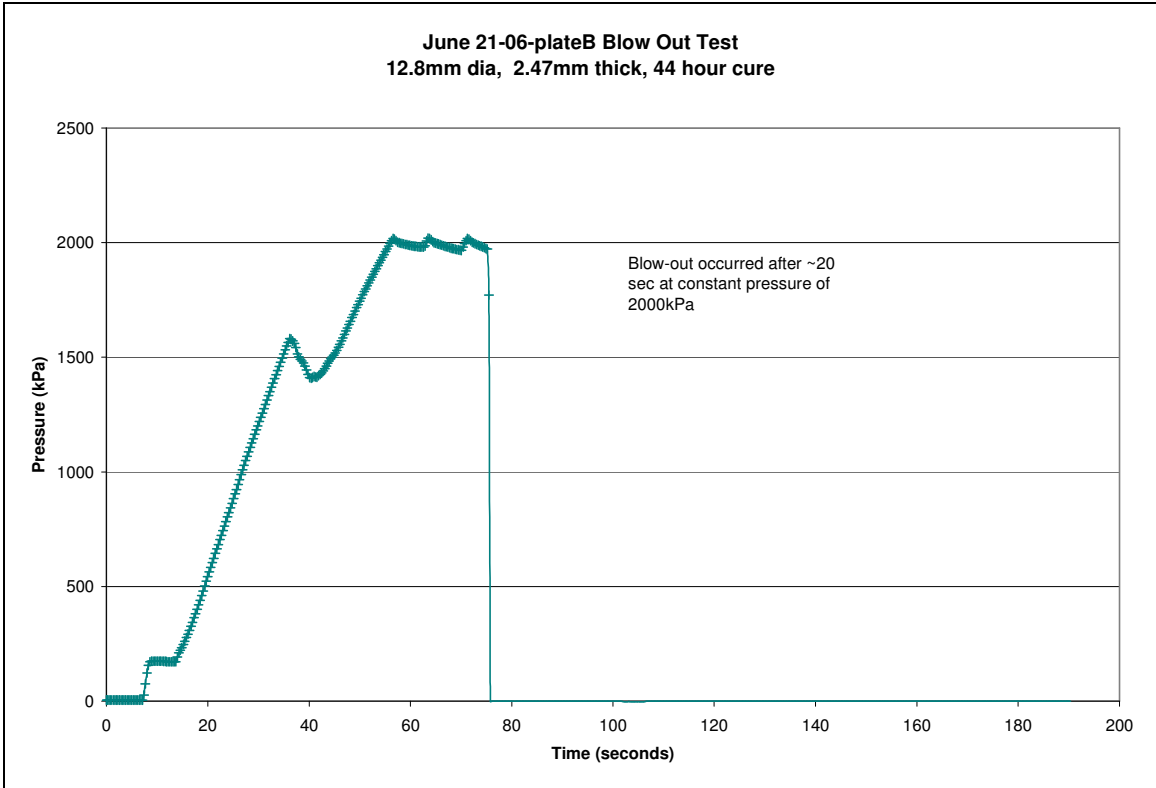




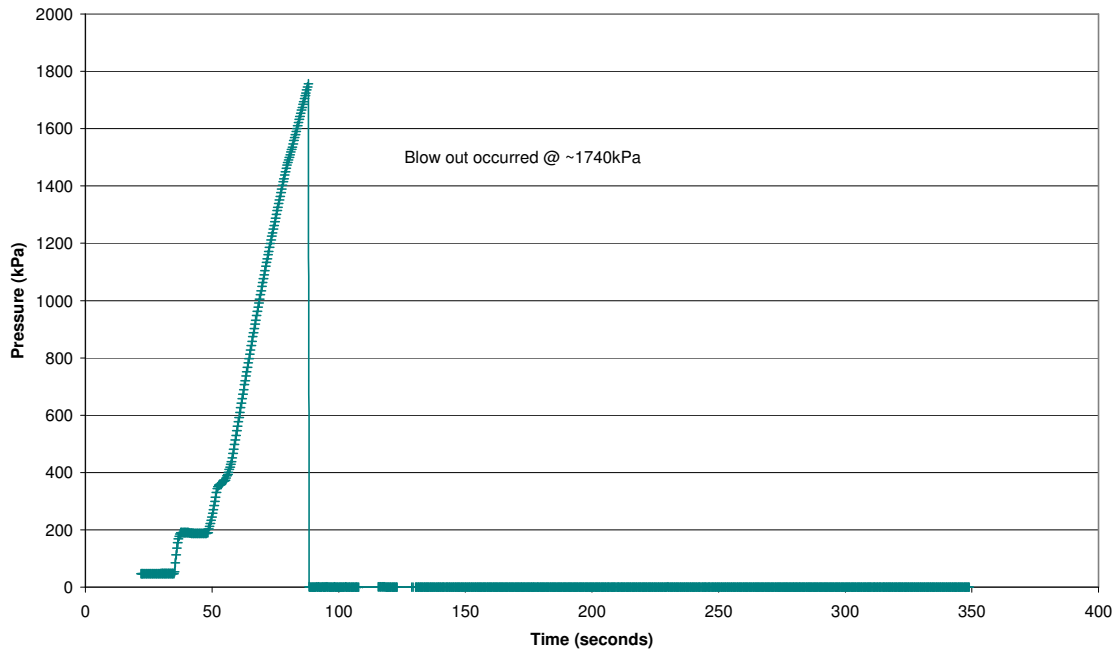




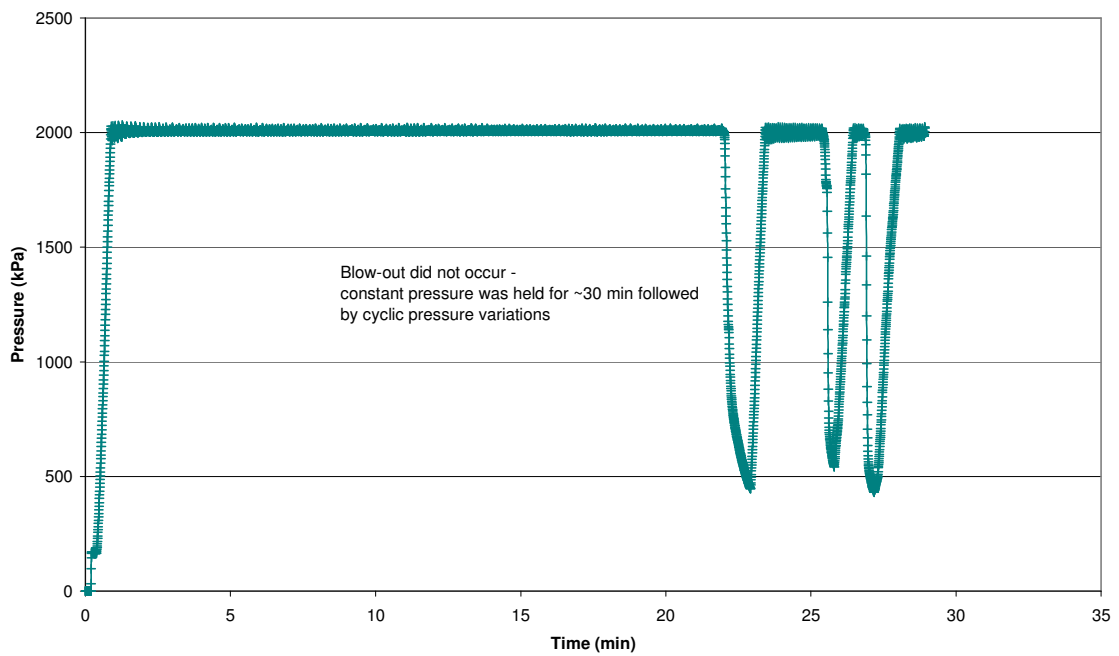


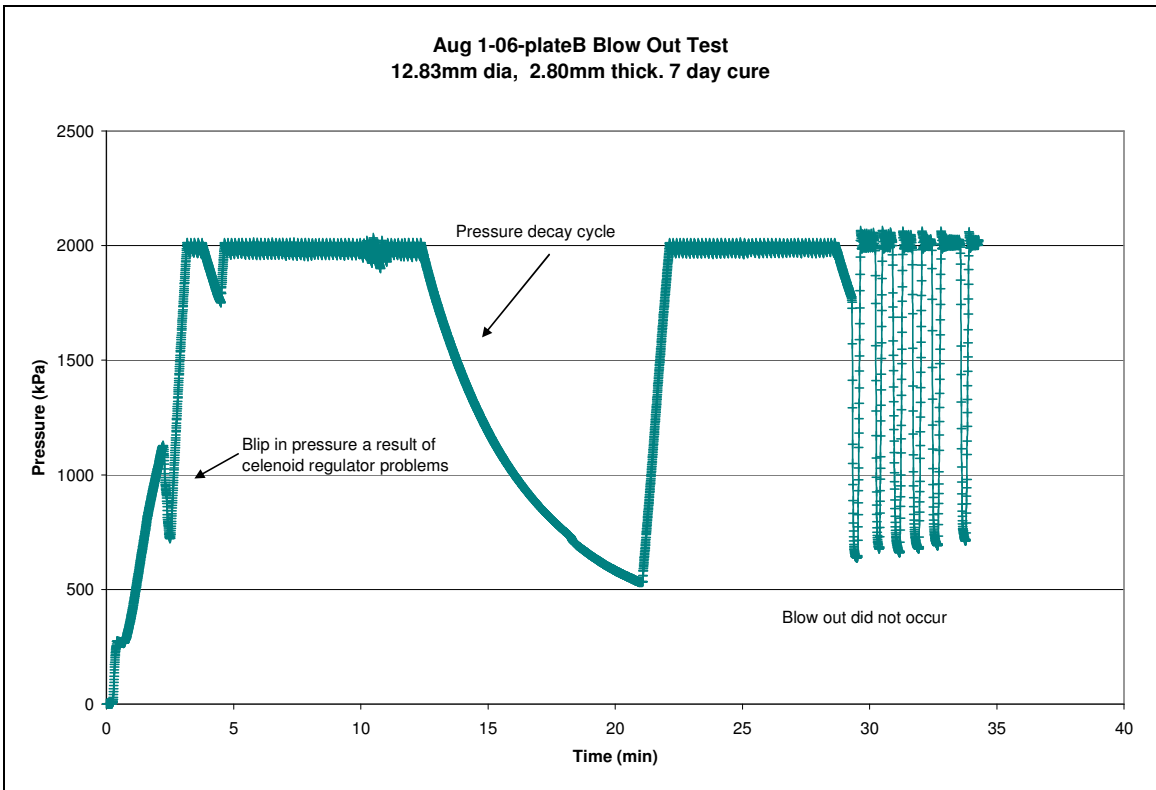
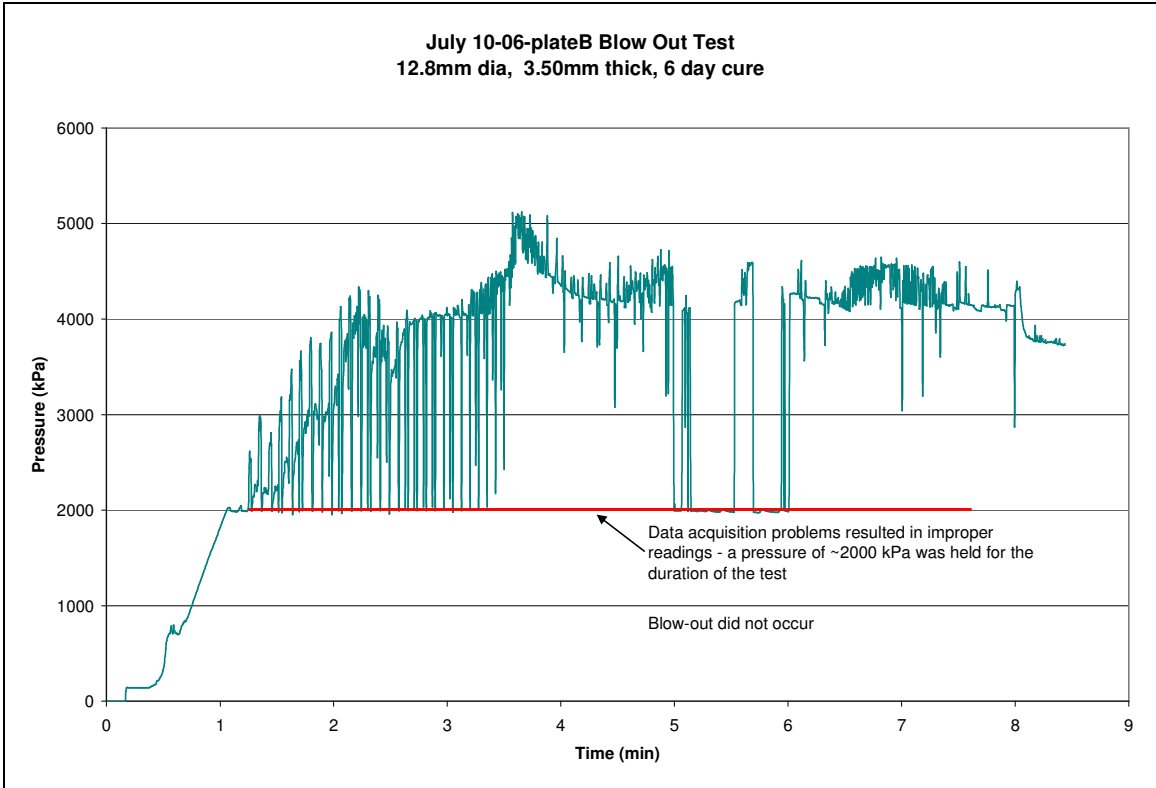


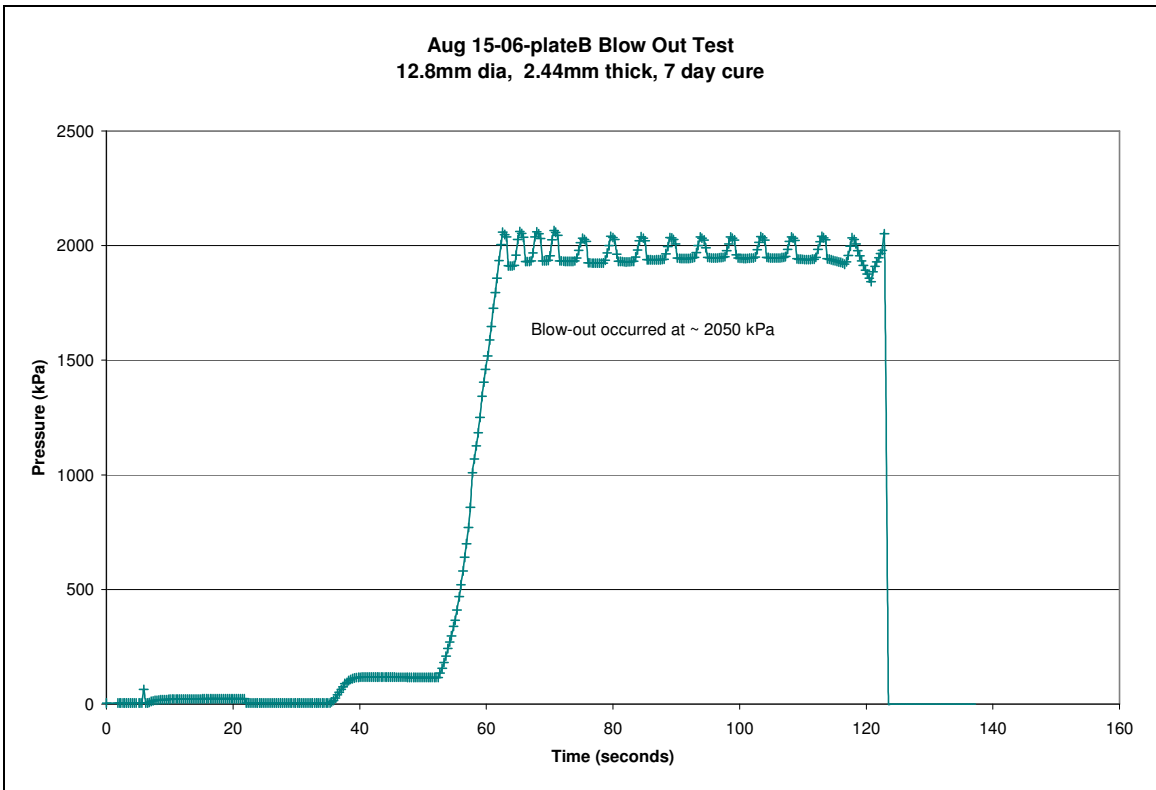
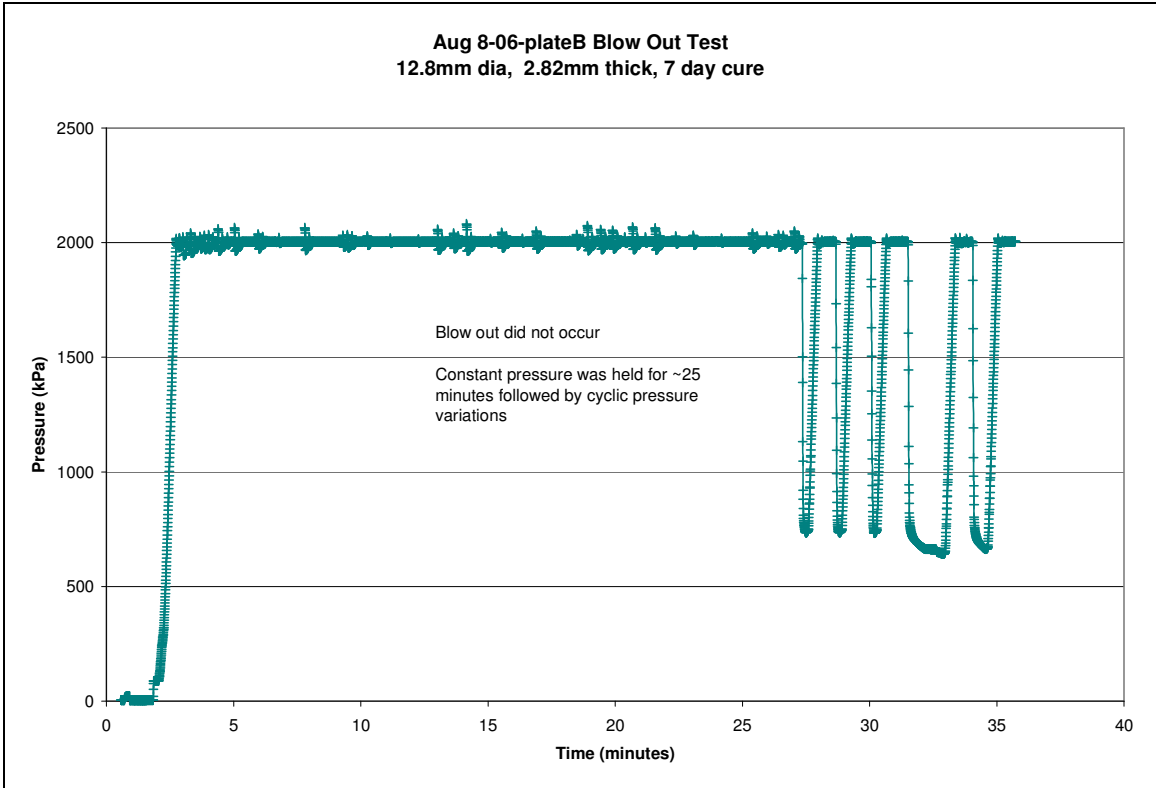
June 28-06-plateB Blow Out Test
12.8mm dia, 2.13mm thick, 46.5 hour cure

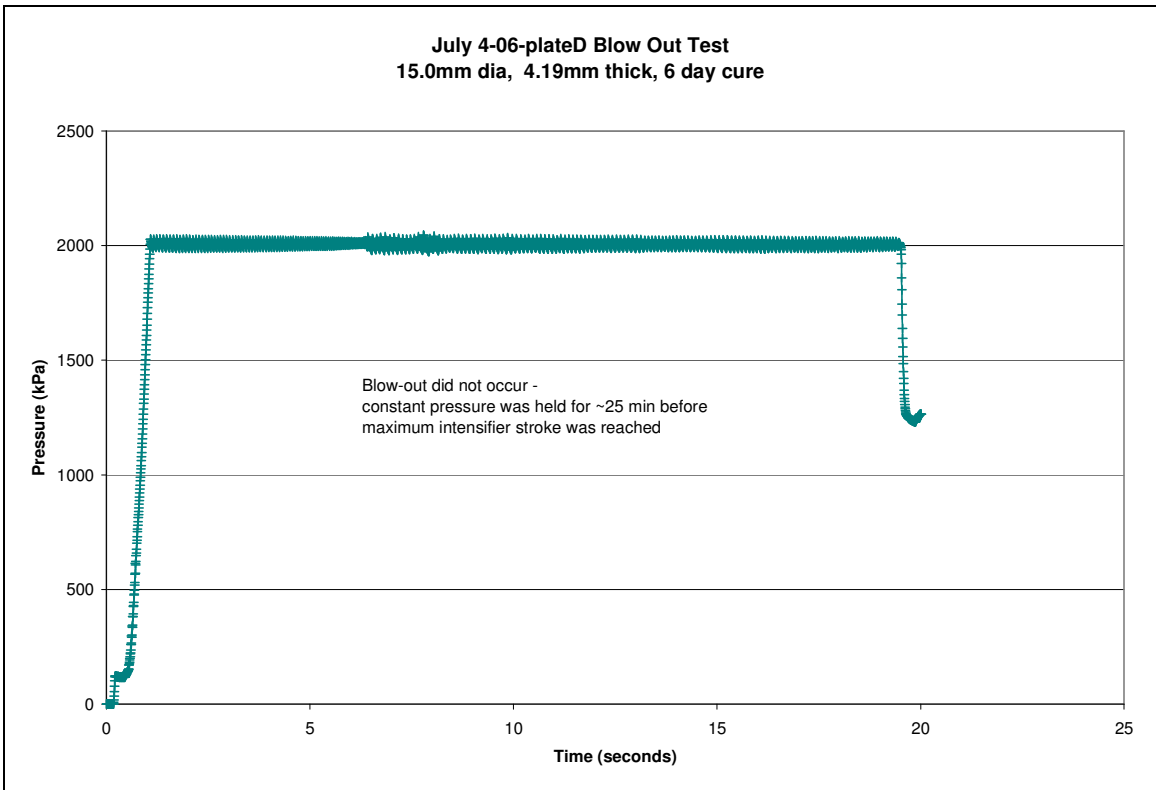
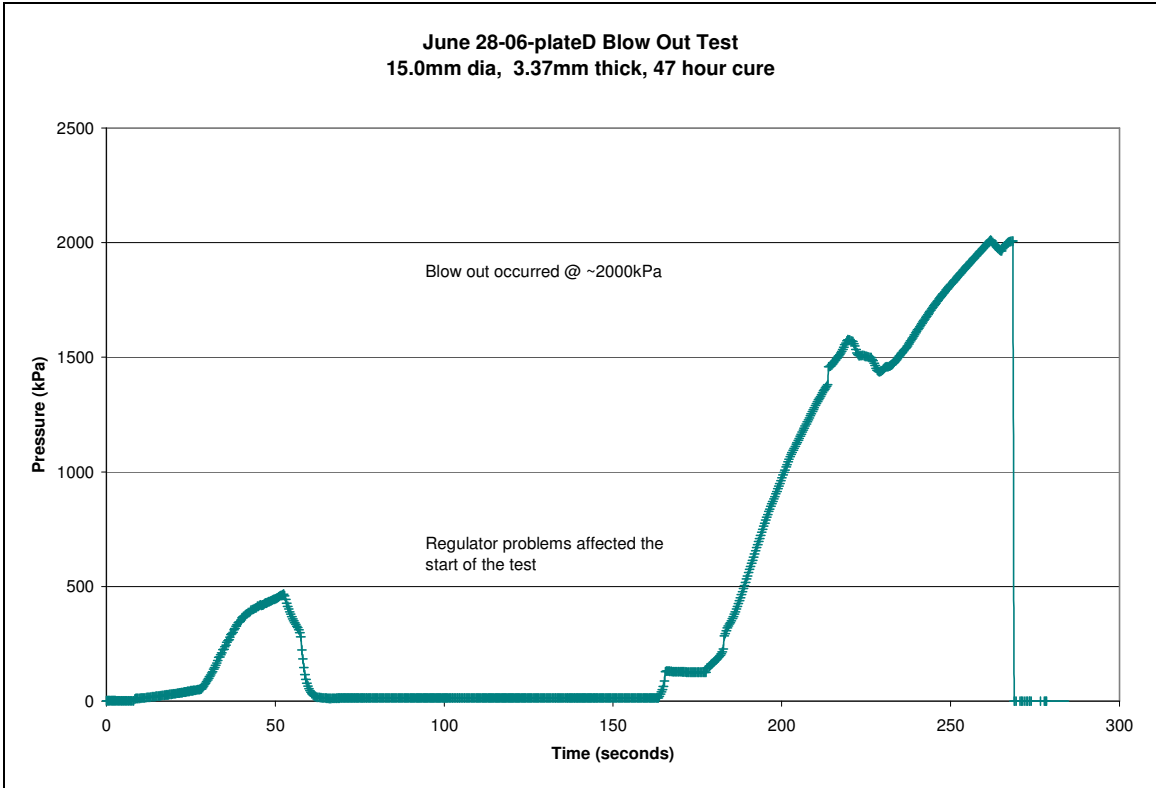


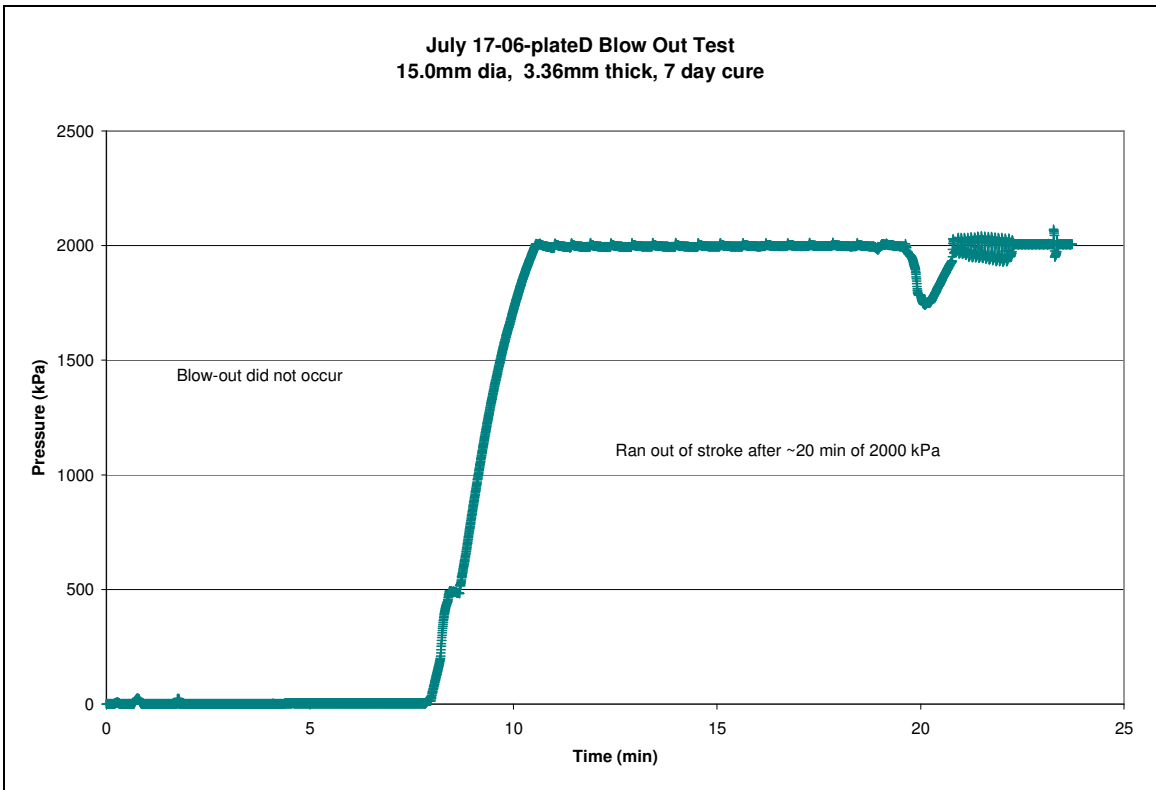
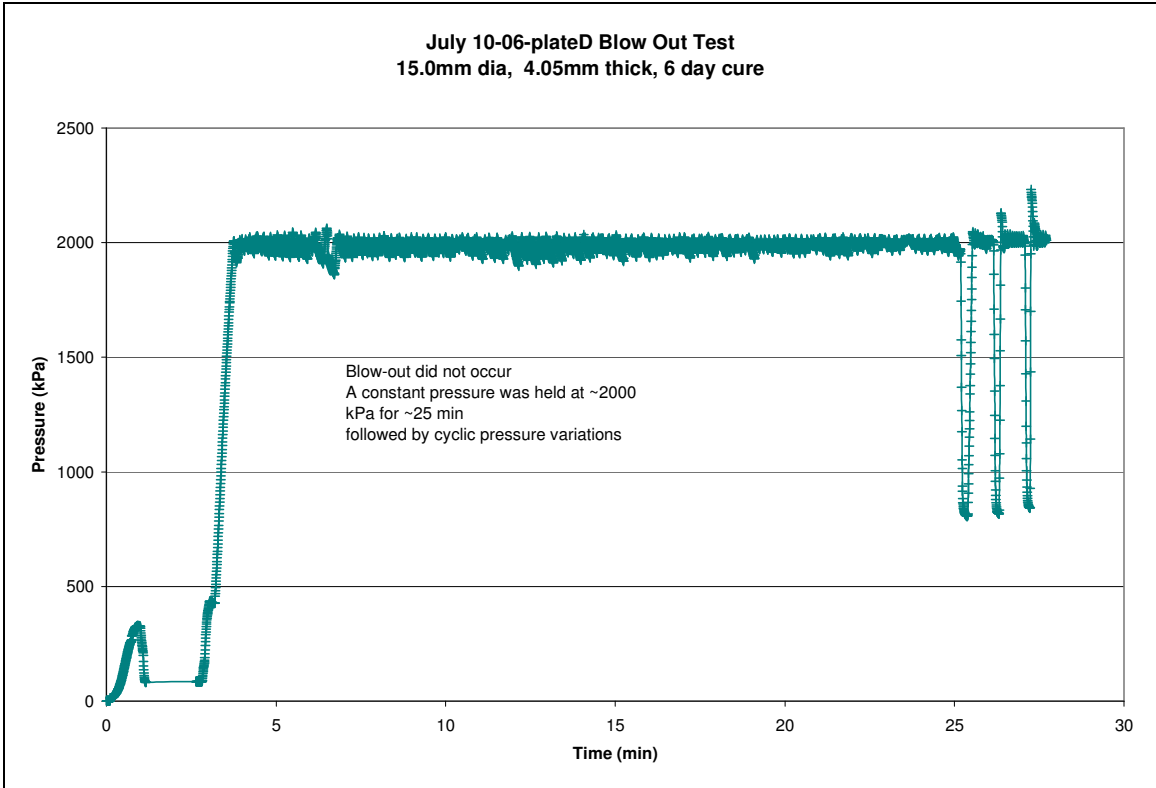
July 4-06-plateB Blow Out Test
12.8mm dia, 3.30mm thick, 6 day cure

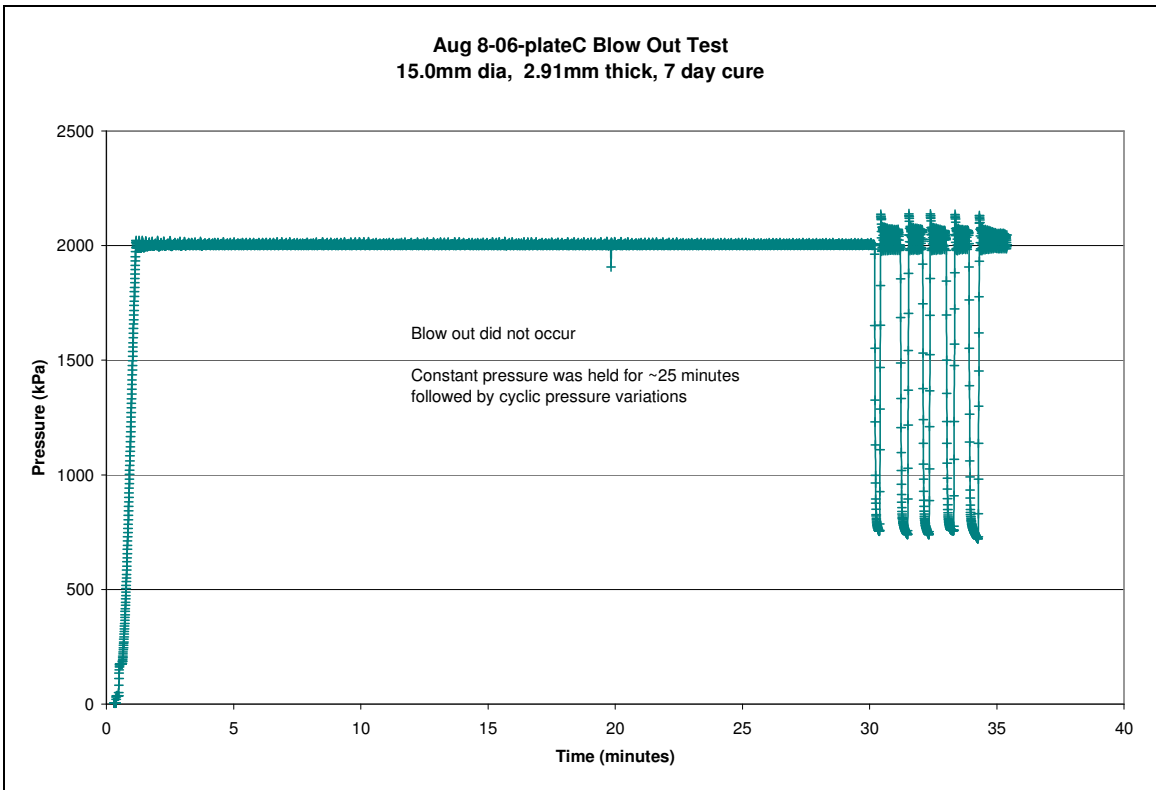
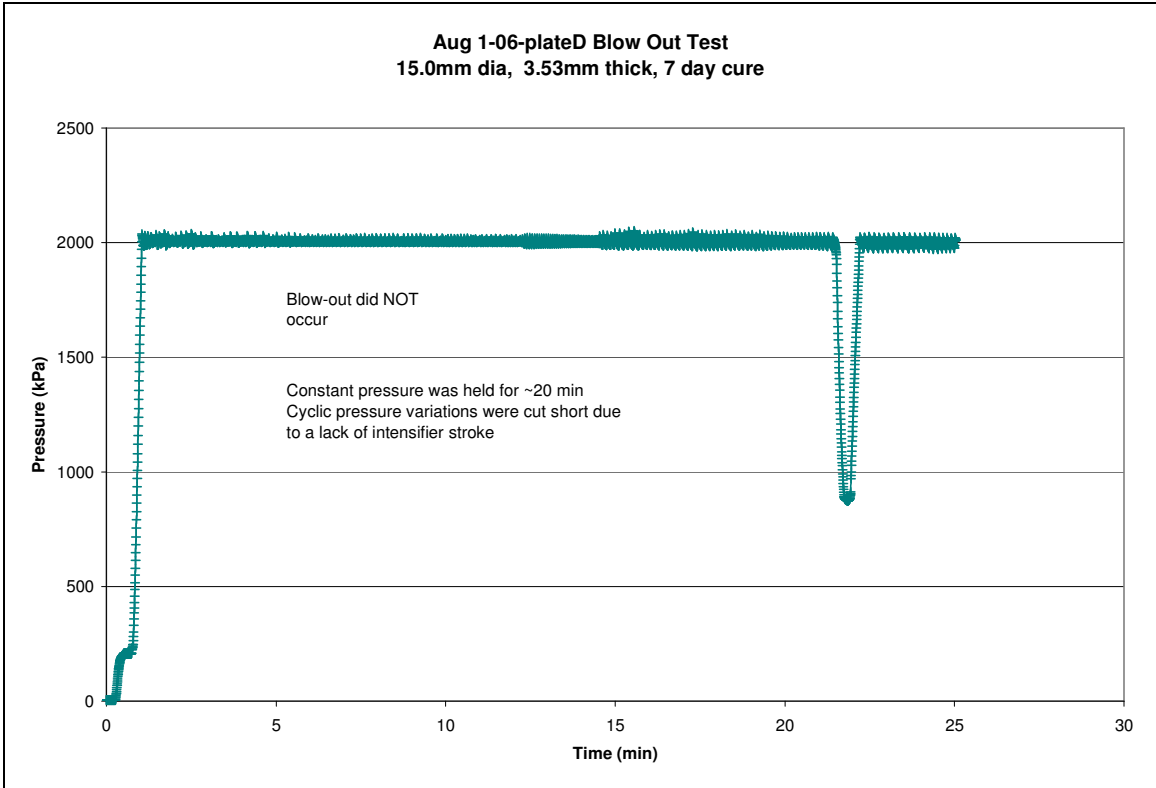


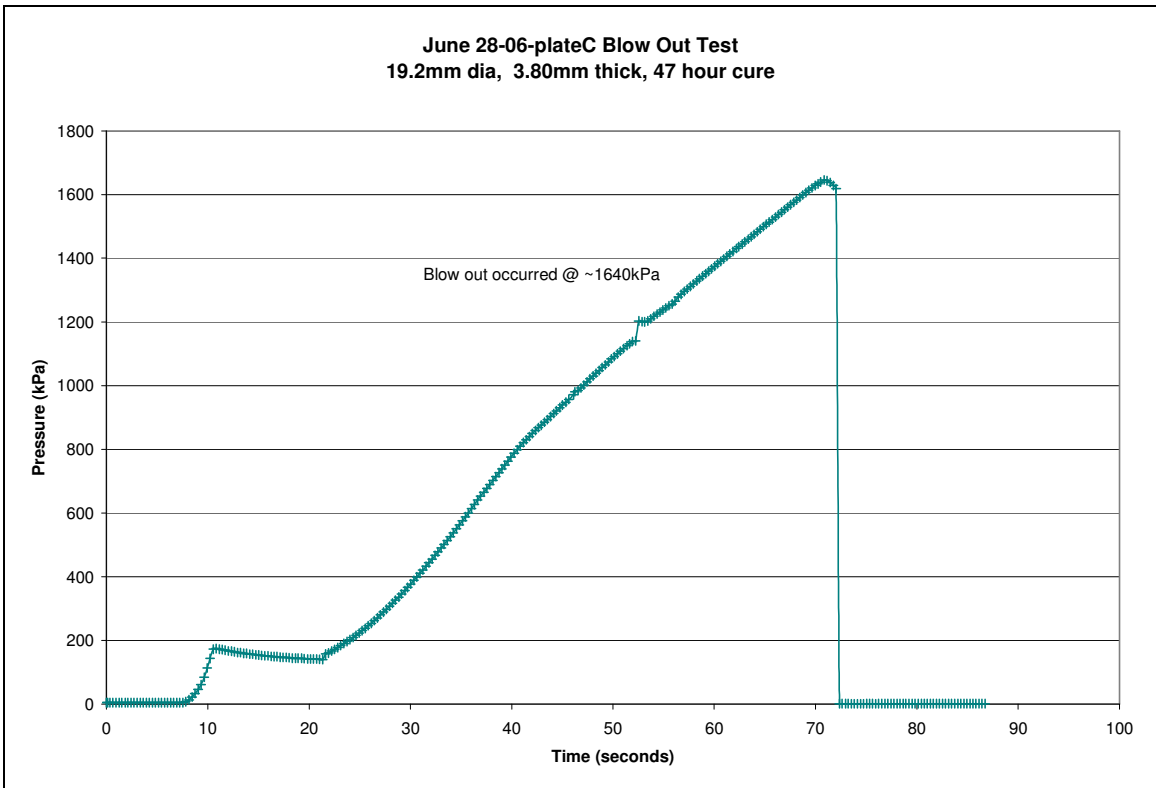
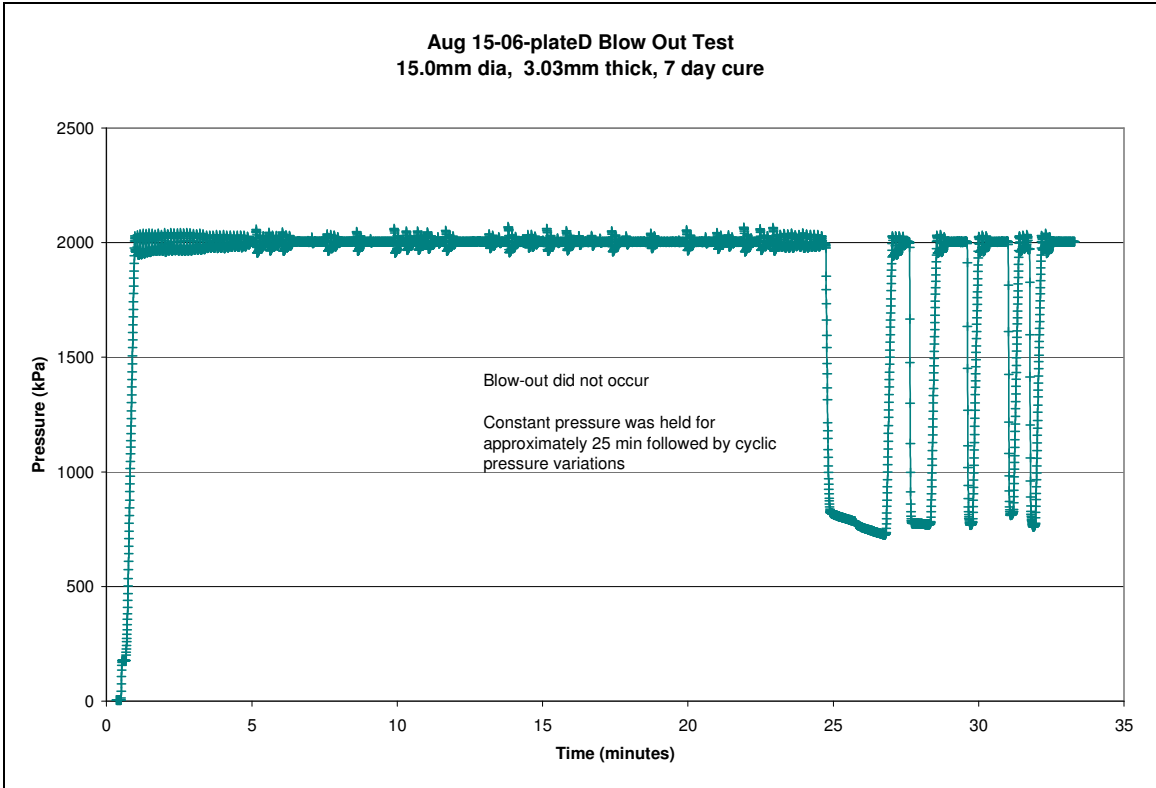


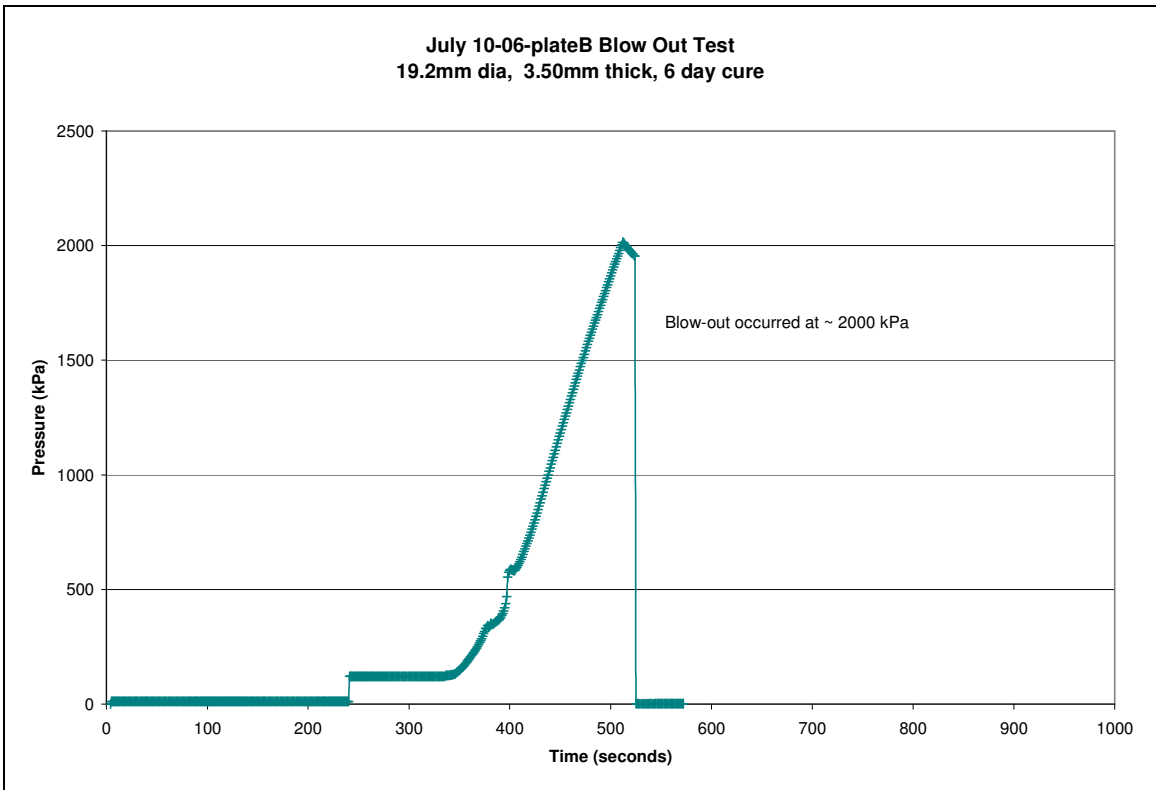
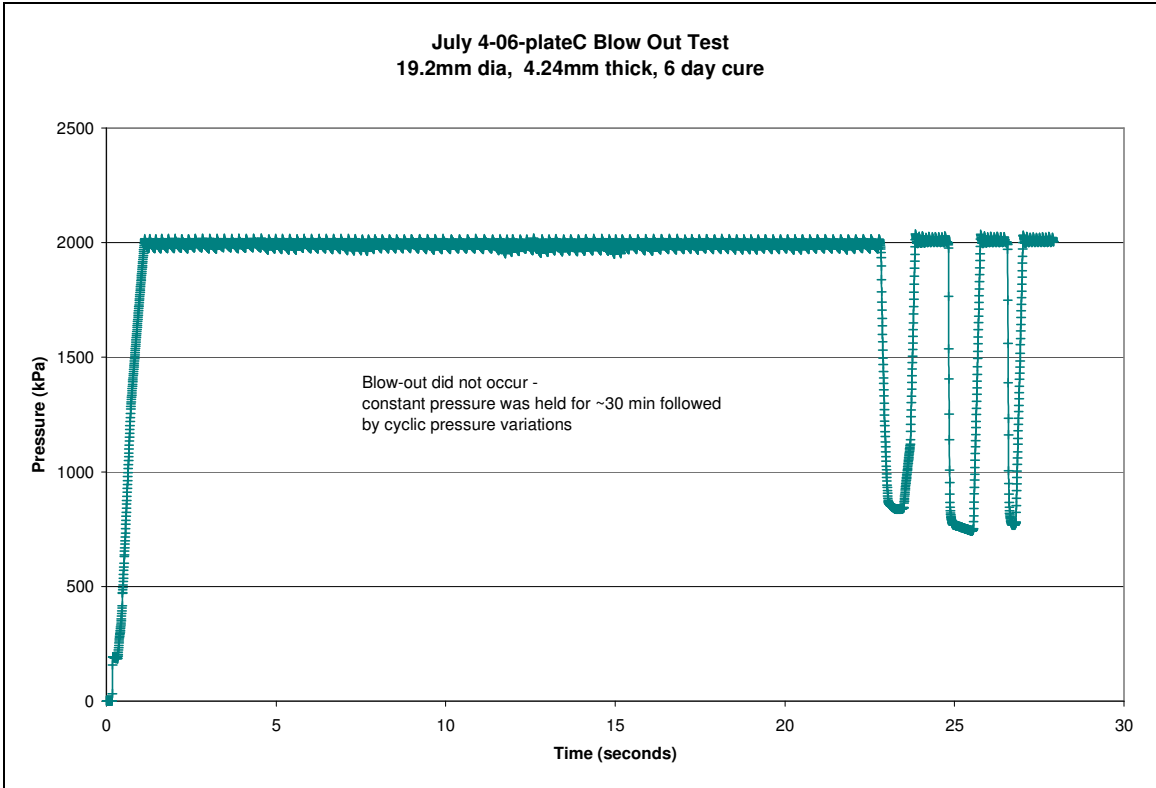


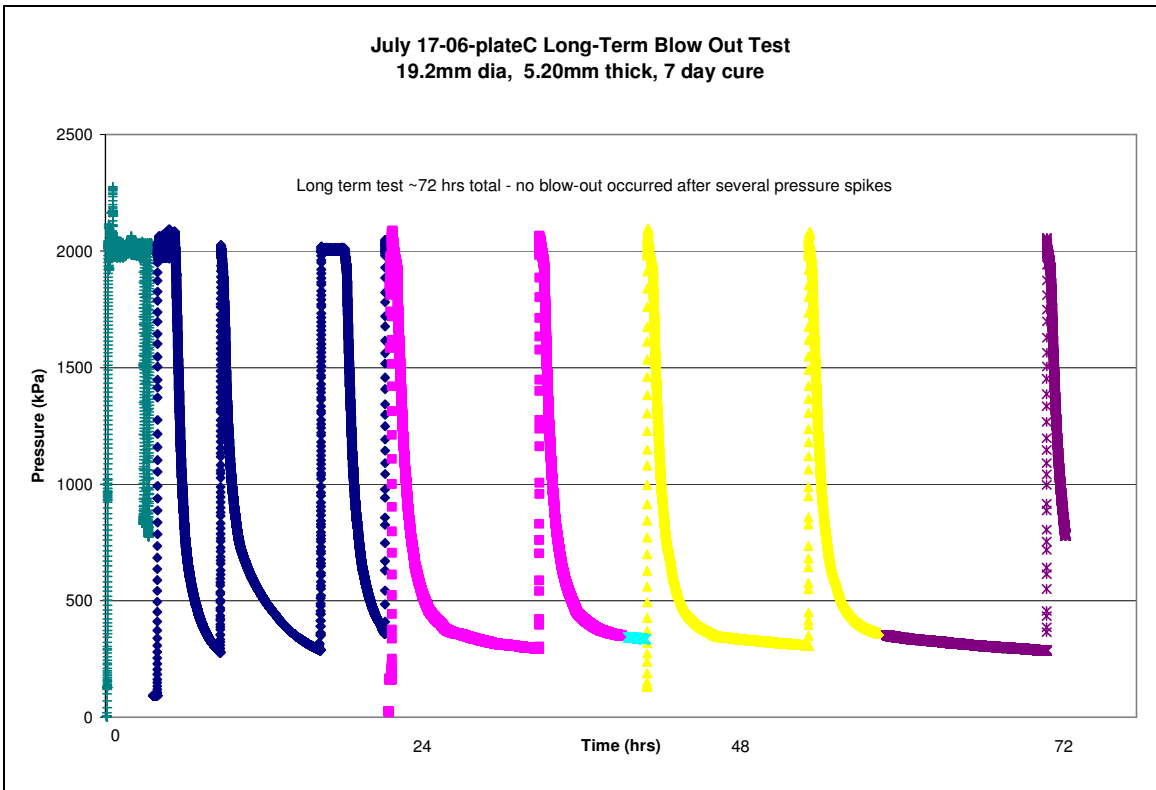
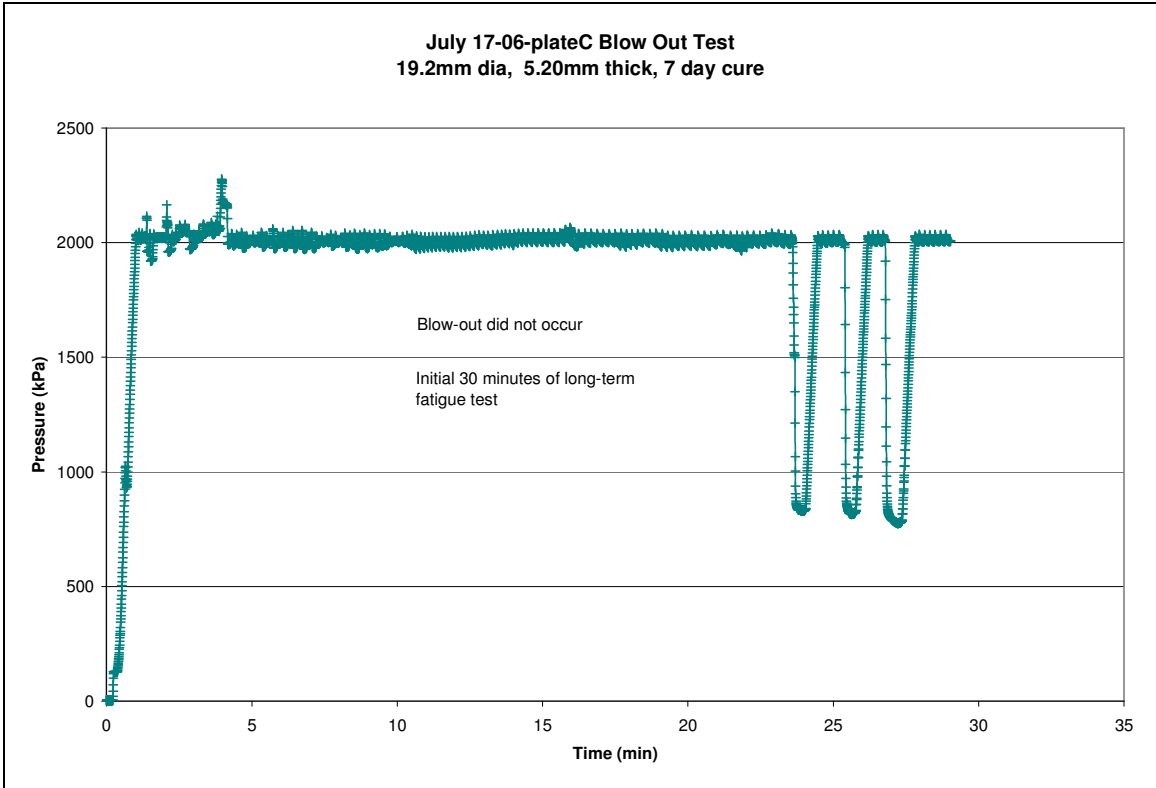


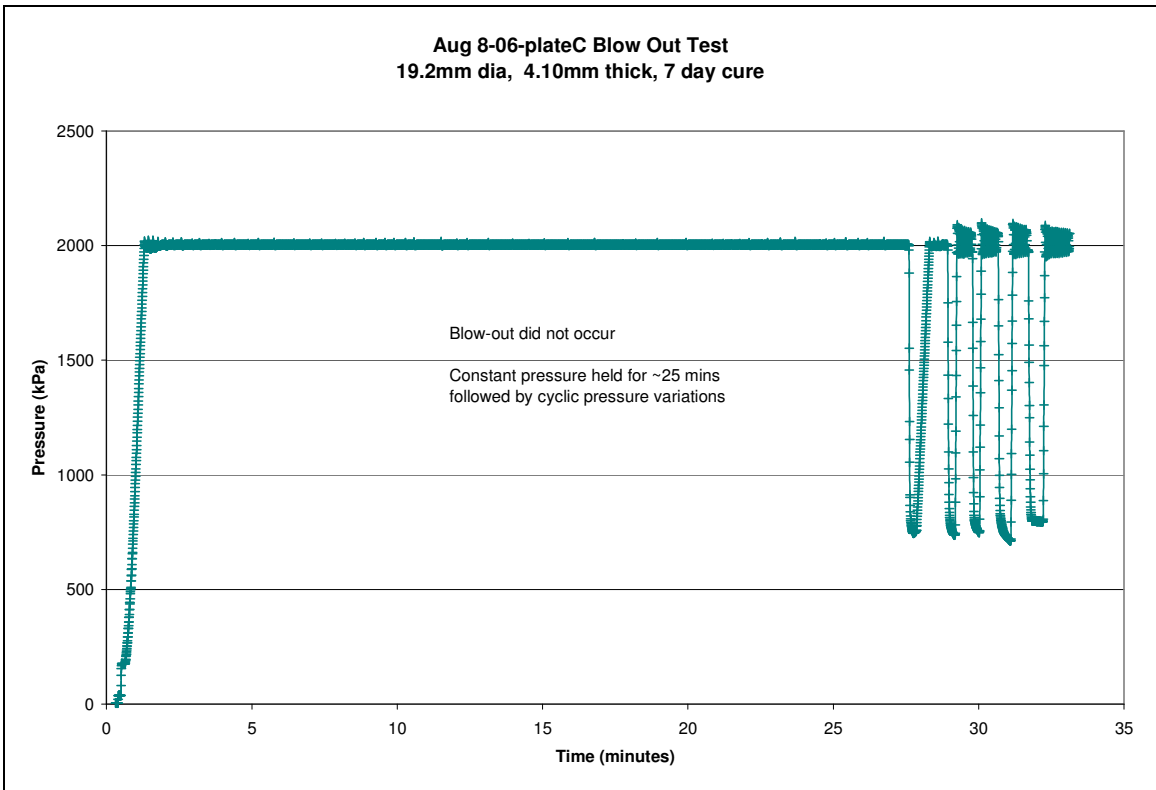
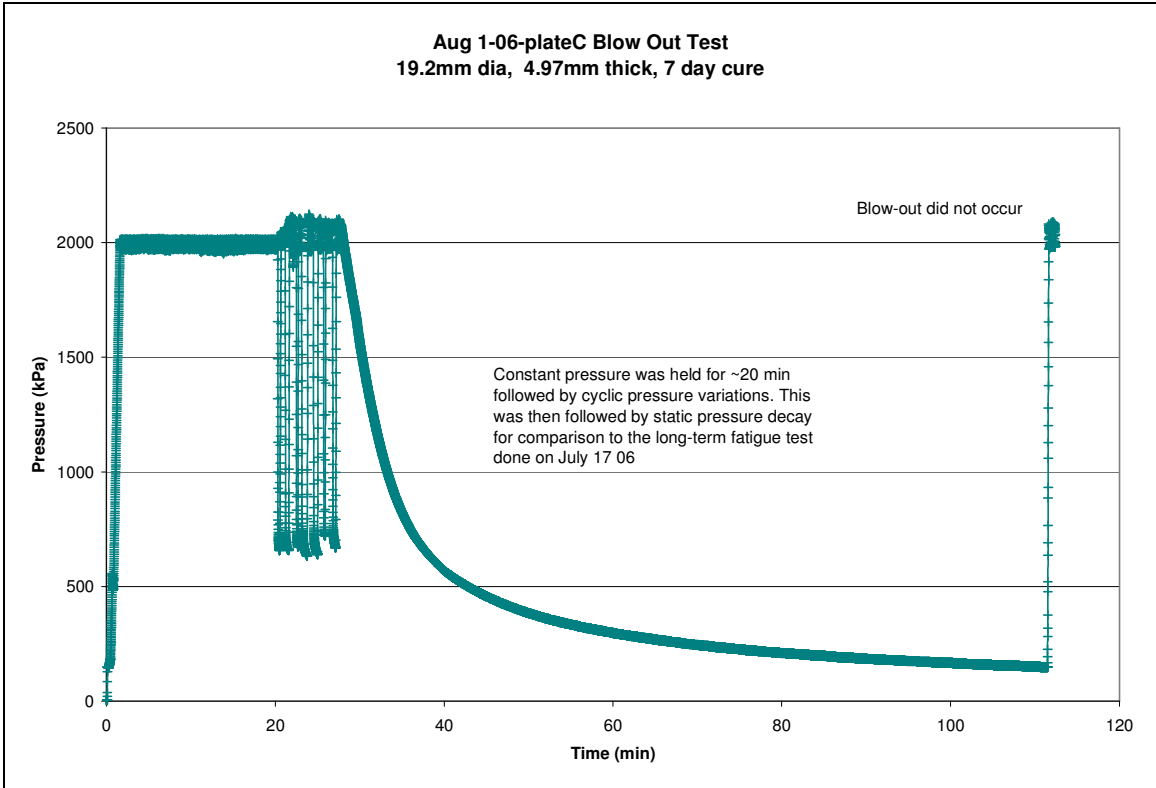


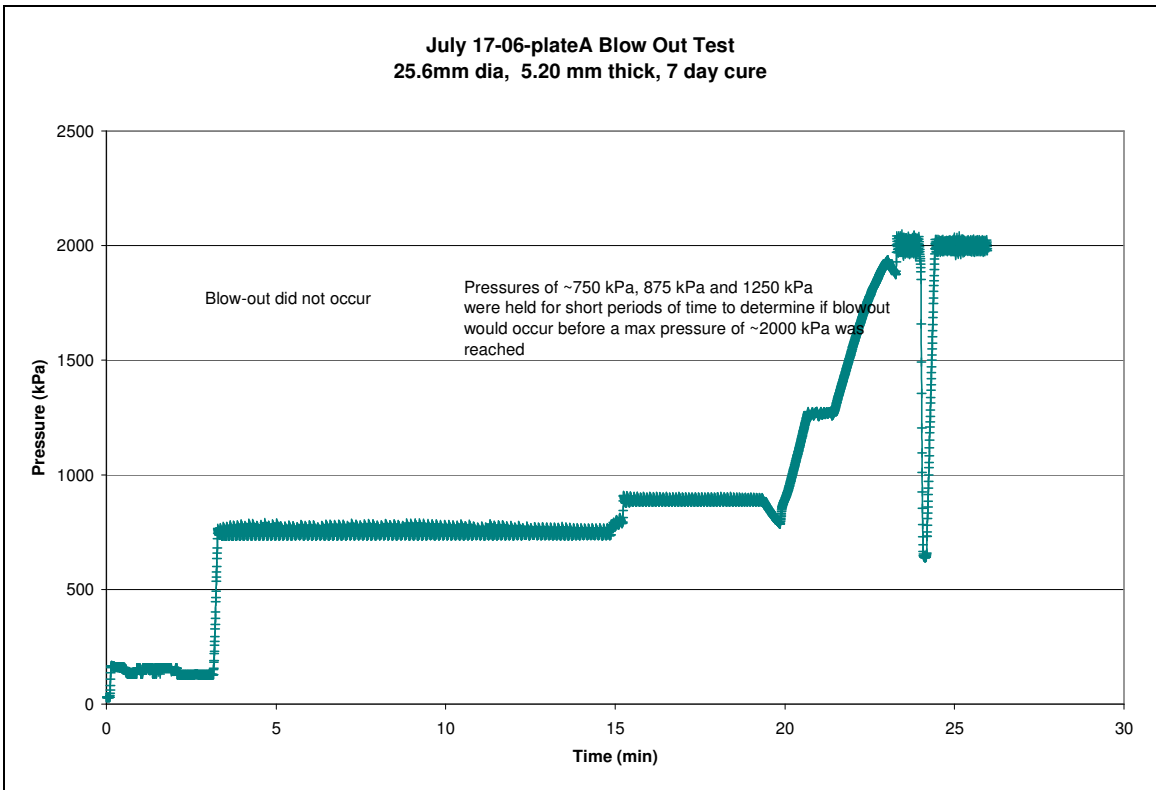
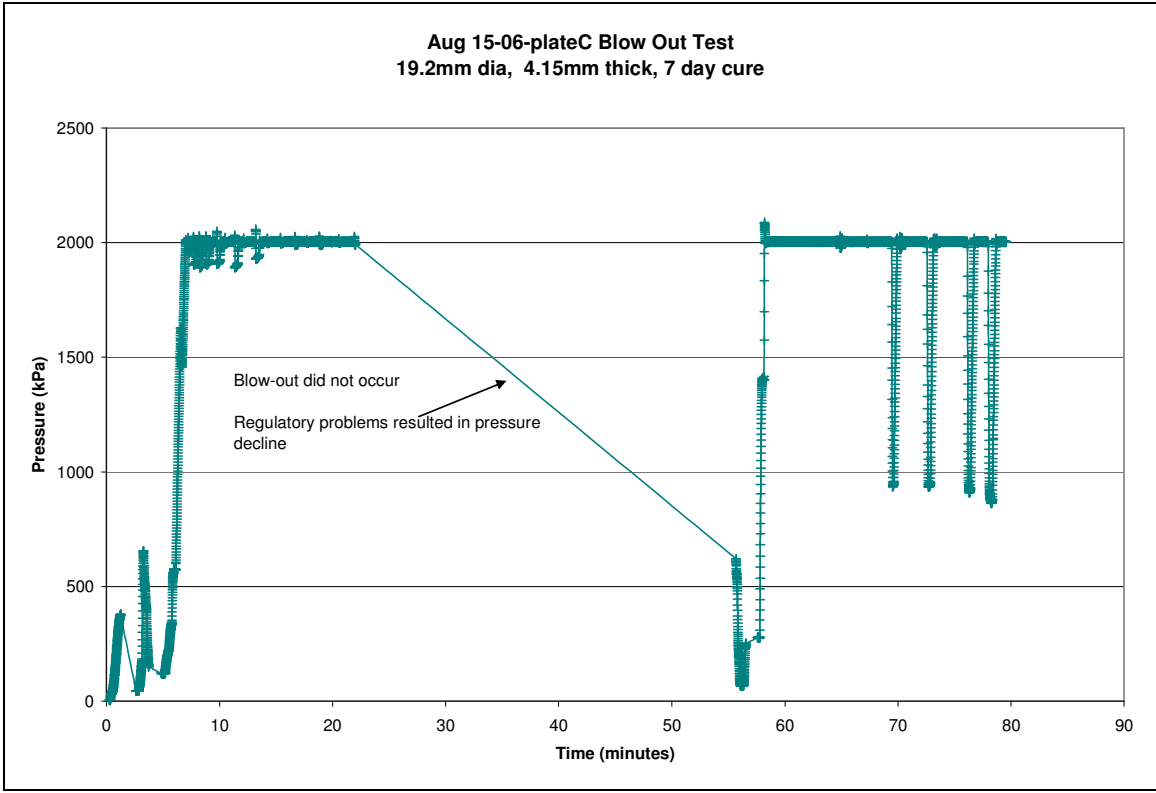


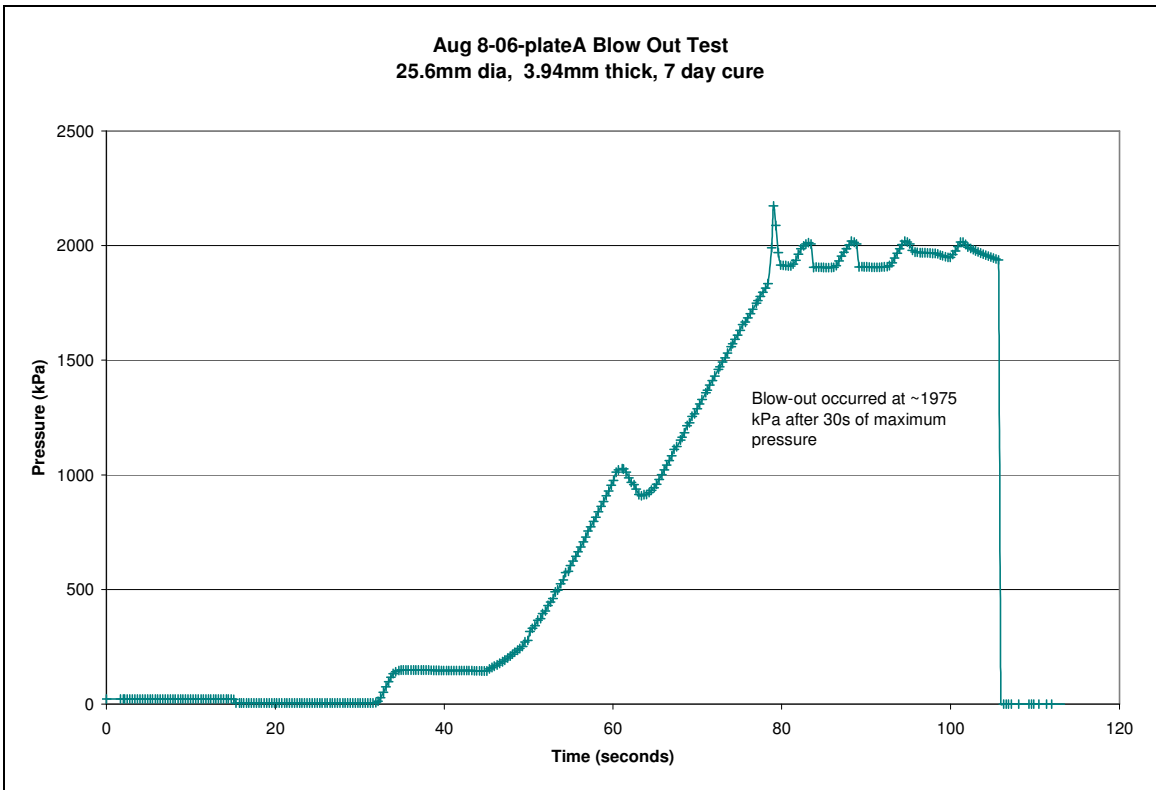
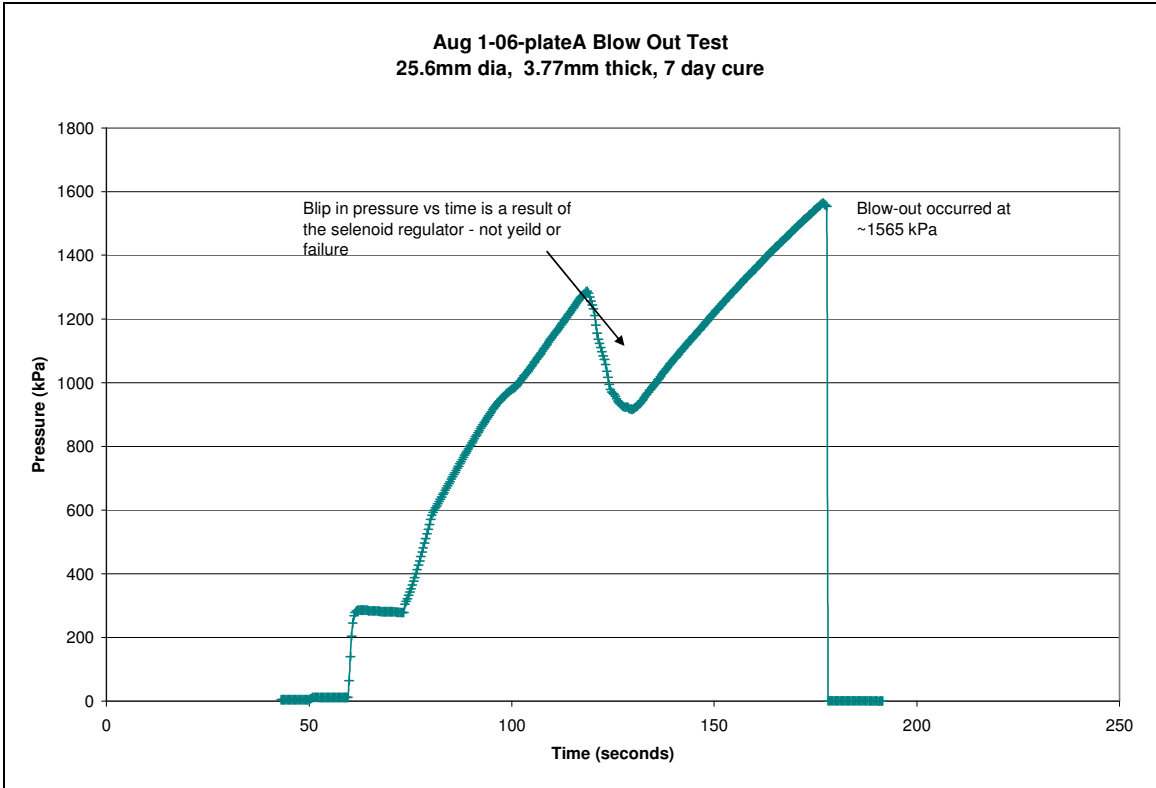




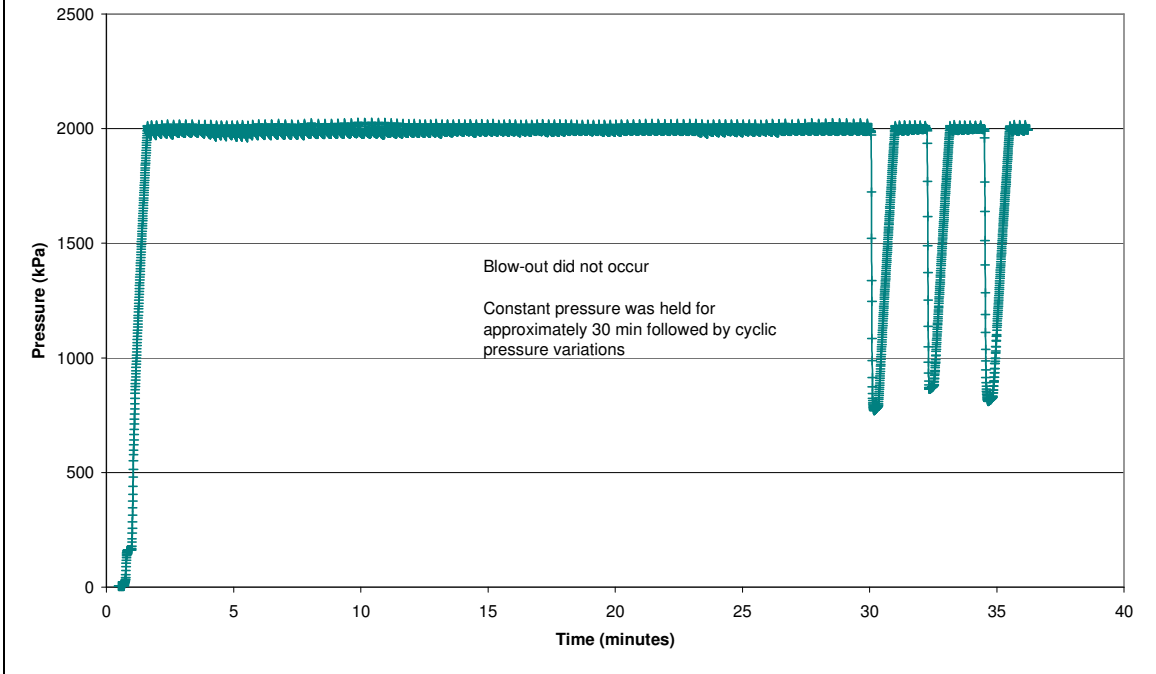








Aug 15-06-plateA Blow Out Test
25.6mm dia, 5.65mm thick, 7 day cure



Appendix D
Distribution Fitting - Probability Paper Plots

