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**TENSILE STRENGTH OF AGED GRAY CAST IRON WATER MAINS IN
GATINEAU, QUEBEC**

by
Shelley McDonald

A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfilment of the requirements
for the degree of
Master of Engineering *

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario, Canada
January, 1996

* The Master of Engineering Program
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administered by the Ottawa-Carleton Institute for Civil Engineering

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Tensile Strength of Aged Gray Cast Iron Water Mains in Gatineau, Quebec

Submitted by Shelley McDonald
in partial fulfilment of the requirements for
the Degree of Master of Engineering

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ABSTRACT

Gray cast iron water pipes constitute 53% of Canada's existing water distribution network (McDonald et al., 1994). This type of pipe had a rate of failure of 35.1 breaks/100 km in 1992 and 36.7 breaks/100 km in 1993. With reducing municipal budgets, utility operators need technologies to locate "hot spots" (areas close to failure) and replace these pipes before they fail. This problem could be solved by introducing a tool which could examine the pipe *in-situ* and measure the pipe wall thickness. Knowledge of remaining wall thickness could be used with a data base of tensile strength values for various pipe thickness and soil environment conditions, to predict residual useful life.

This thesis is a first attempt at developing a framework for the collection of tensile strength values for gray cast iron pipe of various wall thickness. A correlation between tensile strength and remaining material is established. Also investigated was the relationship between the pipe pitting rate and surrounding soil resistivity. A modest trend is observed indicating an increase in pitting rate for a decrease in soil resistivity. However, the trend is weak due to the narrow range of soil resistivities that were available from the data.

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CHAPTER 1

INTRODUCTION

Safe and reliable distribution of the nation's drinking water has always been a concern of municipal operators. When a water main breaks, the reliability of the system suffers and the health and safety of the public is potentially at risk.

The majority of the water in Canada is distributed to households and industries via cast iron pipe. This material has been in place since the early 1900's in some cities. The existing pipe material will continue to deteriorate since it is no longer used (since the early 1970's) for new installations, repairs or rehabilitation.

As municipalities battle with shrinking budgets and increases in the number of repairs and rehabilitation, one would assume that the older sections of the cities' water distribution networks should be replaced first. However, this may not always be the case. In some cities, gray cast iron pipes, 75 years or older, are still in place and do not have a high occurrence of breaks. In other cities, gray cast iron pipe was replaced after as little as 5-10 years of service.

It is difficult to manage a system that is out of sight and where there is no warning before a main failure. The actual condition of these pipes is not readily apparent without excavation. There is a need for a management tool like that in the pavement industry (e.g. Paver). For such a management system to work, a data base has to be established to assist in the determination of the residual life of gray cast iron pipe. Until now, there has not been significant mechanical testing performed to determine the remaining strength of this material.

This investigation is the first step in determining the framework for testing and data collection for the determination of the residual strength of gray cast iron water pipe. Other researchers have performed mechanical testing on gray cast iron pipe of various ages, however, some of the parameters that influence the residual strength have not been collected (e.g. soil resistivity).

Gray cast iron and ductile cast iron are the two forms of cast iron that will be referred to in this thesis. Gray cast iron has a gray fracture surface since fracture occurs along the graphite plates or flakes. The elements and their composition ranges are carbon (2.5-4.0 %), silicon (1.0-3.0 %), magnesium (0.2-1.0 %), phosphorus (0.002-1.0 %) and sulfur (0.02-0.25 %) (Davies, 1992).

The ductile cast iron also contains the previously mentioned elements at the following composition ranges carbon (3.0-4.0 %), silicon (1.8-2.8 %), magnesium (0.1-1.0 %), phosphorus (0.01-0.1 %) and sulfur (0.01-0.03 %) (Davies, 1992). The graphite component of ductile cast iron forms into balls like little cabbages. Its fracture surface is silver-gray and this form of cast iron has measured ductility. The increase in ductility of the ductile cast iron was a measured improvement over the once used gray cast iron for water main pipe material (Davies, 1992).

The objectives of this study are to correlate the tensile strength of gray cast iron pipe and the corrosion level for a sample of pipe that is the same age and establish the pipe pitting rate and the influence of the surrounding soil resistivity.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This section introduces some of the factors considered to influence water main failures and the various modes in which these pipes fail. The last section of this chapter presents work previously done in the areas of water network condition assessment based on physical testing and corrosion pitting analysis.

2.2 Water-Distribution Networks Failures

2.2.1 Water Main Failure Causes

Water pipes fail when the strength of the pipe is exceeded by the stresses induced in the material through external or internal pressure on the pipe. Large diameter pipes (> 250 mm) tend to fail predominately with a longitudinal crack due to overloading which leads to crushing stress, while small diameter pipes tend to break circumferentially due to beam action (Ciottoni, 1983). A study of 50 Canadian water distribution networks by McDonald and Félis (1995) indicates the predominant mode of failure was circumferential (38.5%). The majority (71.6%) of pipe installed in these cities had diameters up to 200 mm (8 ").

A number of factors can decrease the strength of or increase the stress on the pipe thus leading to failure. Morris (1967) found that water main breaks result as a consequence of the following factors.

- inadequate design

There are many factors to be considered in the design of a water distribution network. The principal factors are the working pressures and the type of material for backfill and bedding.

The working pressure and the operation procedures must be known in order to have a proper design. These pressures and procedures will be used to assist in the determination of the pipe type, wall thickness and pipe strength needed. A safety factor of 2.5 is normally used to allow for some uncertainties in design and for pressure surges that may take place during the operation.

The type of pipe chosen is dependant on the type of bedding and backfill materials. Knowledge of the soil conditions can assist in determining whether the soil promotes corrosion of the pipe material or if it has the capacity to swell or shrink. The level of expected corrosion depends on soil resistivity, moisture content and oxygen content. At this point, the provisions needed to deter corrosion of the water main can be incorporated in the design process. The problems associated with soils that swell/shrink will be discussed in a later section.

- improper installation

Improper installation has led to water main failures. Negligent installation is characterized by boulders, rocks and blocks of wood left in the bedding material. The bedding provides support for the pipe and should be smooth without hard spots caused by clumps of clay, rocks, boulders, etc. Over time, if a leak occurs, the bedding is washed away, the pipe settles and could rest on the rocks, boulders or block of wood. An increase in tensile stresses on that area could occur as the pipe settles and deflects over time due to the point load on the bottom of the pipe (Howard, 1994).

- pressure surge or water hammer

A water main failure can also result from a pressure surge or water hammer. A pressure surge occurs as the result of rapid change in water velocity which can be the consequence of opening or closing valves, pump operation or fire hydrant operation. The change in velocity causes a pressure wave in the system which has energy to dissipate. If the energy is dissipated within the pipe, an increased radial pressure acting on the pipe results. For a pipe whose strength is already reduced due to corrosion, the increase in radial pressure may be sufficient to cause failure.

Signs that indicate the occurrence of a pressure surge or water hammer failure are those failures which occur immediately after a hydrant flushing program. Upon inspection of the section, the failure resembles a circular hole where a carbon plug has popped out of the pipe due to the increased radial pressure. Pressure surges are suspected to be the cause of recent (August, 1995) multiple failures in a subdivision in Ottawa-Carleton (Scothorn, 1995) caused by either the closing or opening of valves required to isolate a section of the distribution network for hydrant flushing. In these cases circular sections (carbon plugs) had popped out of the pipe leaving holes.

- soil movement and volume shrinkage or swelling

Active soil can also cause problems for a utility operator. Knowledge of the soil type is not only important for determining the corrosion potential, but also for its swell/shrink potential.

An expansive soil is one which, upon wetting (or drying) swells (shrinks) significantly (Clark, 1970). Materials that undergo such a volume change should be avoided when specifying the type of bedding and backfill material for water main placement. To determine whether a soil is susceptible to volume changes, the mineralogy of the material must be investigated.

Kaolinite is an example of a soil that does not expand or shrink due to the strong bonds between its alumina and silica sheets. This strong bond does not permit water and cations to enter.

A clay material that can shrink and swell considerably is *montmorillonite*. This clay has a sheet of alumina bound between two sheets of silica. The sheets are held loosely together therefore, cations and water are able to enter inducing volume changes in the soil.

If the placement of the pipe in these materials cannot be avoided, provisions should be made to limit the change in moisture content and thereby control the shrink/swell of this type of soil. This could be achieved by placing the pipe in a deeper trench,

below the zone of groundwater fluctuations. The environment there is more stable and there is a greater soil bearing on the pipe to offset the swelling pressure below it. If this option is not viable, a granular material could be placed in the trench to assist in the drainage around the pipe.

It has been shown (Morris, 1967) that as the soil expands, the corrosion potential of the soil increases. Soils with a swelling index of 144 MPa (3000 psf) and higher, generally, have a soil resistivity of less than 700 ohm-cm. This low saturated resistivity is considered a severely corrosive soil condition (Heino and Taylor, 1987).

At the present time, if a soil has a swelling potential of 144 MPa (3000 psf) or greater, it is considered to be an expansive soil. Therefore, pipe selection is a special consideration as is corrosion protection under this soil condition.

Even though distribution pipes are probably able to withstand small amounts of movement, excess main failures in some cities have been attributed to volume changes due to expanding and shrinking soils (Morris, 1967). Morris (1967) determined that by increasing the moisture content by 10%, for a material with an 18 per cent volume of shrinkage, the swell pressure reached 72 MPa (1500 psf).

Problems could also arise from layers of different soil materials above and below the pipe. For example, when the moisture content is increased in an expansive soil, it tends to lift the pipe. If a non-expansive soil is above the pipe, it could restrain the pipe from moving. Depending on the pressures and stresses induced, the pipe could possibly rupture due to this restraining backfill system.

- internal corrosion

There are three types of internal corrosion: chemical, galvanic and bacterial, also called, microbiological. Chemical will be discussed in this section as well as factors to inhibit microbiological corrosion. Galvanic and microbiological corrosion will be discussed in the external corrosion section.

Chemical corrosion of water mains occurs from the chemicals present in the drinking water. Waters with a low pH or waters high in dissolved oxygen, total solids, and carbon dioxide, will corrode most ferrous metals (gray cast and ductile cast iron).

Zamanzadeh et al. (1990) have reported that internal corrosion caused by water is influenced by the following properties of water: hardness, alkalinity, chloride ions, sulfate ions and silica, concentration of dissolved gases, pH, temperature and water velocity. In a study of Ontario water main corrosion by Wakelin and Gummow (1990), it was concluded that the pipe diameter influenced the occurrence of internal corrosion. For large diameter mains (> 250 mm), the internal corrosion was greater than that experienced by smaller diameter pipes. This could be due to the carrying capacity of the larger diameter pipes which are able to provide a greater amount of oxygen which is an essential component in the chemical process of corrosion.

To determine whether internal corrosion occurs within a water distribution system, a section of excavated pipe is examined. Tubercules, which are red-brown scales, will line the inside of the pipe. The soluble ferric salts are oxidized and hydrated ferric oxide (rust) is deposited in the pipe (Evans, 1960). Over time, the carrying capacity of the pipe will be reduced as the concentration of tubercules increases. For a pipe in service, indicators of internal corrosion are: colored water complaints and/or a reduction in pressure.

There are ways to protect or reduce the likelihood of internal corrosion. Reedy (1966) proposed an increase in the calcium carbonate (CaCO_3) concentration in the treated drinking water which promotes the deposition of a thin protective film inside the pipe wall. This protective film inhibits or retards the internal corrosion of the metal surface. The precipitation of the calcium carbonate can also be initiated by the adjustment of the pH of the water.

Bacterial corrosion can easily be reduced or eliminated by maintaining a chlorine residual in the water distribution system. At the present time, chlorine residuals (trihalomethanes) have been linked with cancer and other means of the final disinfection of water before it leaves the water treatment plant are being evaluated. The replacement chemicals would need to have a long life since some treated water

travels along distance before reaching the homeowners' tap. For example, in Ottawa-Carleton (ON), water leaving the Britannia Water Treatment Plant on the west end of town can take up to two days to reach consumers on the east end of town (Orleans).

- **external corrosion**

External corrosion is defined as a natural occurring phenomenon which deteriorates a metal or an alloy by an electrochemical reaction (Fitzgerald, 1968). External corrosion types include chemical, electrochemical and bacterial or microbiological.

One example of a chemical corrosion process is termed graphitic corrosion. In the case of gray cast iron, the reaction which takes place between the metal pipe and the surrounding environment results in the depletion of the ferrite component of the gray cast iron leaving graphite, as shown in Fig. 1.

Graphitic corrosion is the conversion or removal of the iron which leaves behind corrosion by-products in gray cast iron pipe. When an externally corroded pipe is excavated, the pipe is apparently in good condition because the corrosion by-product (graphite) is black and when still attached to the pipe surface may resemble the original gray cast iron (Wakelin and Gummow, 1990). The apparent strength of this corroded pipe is deceiving since the graphitized portion of the pipe has a much lower strength than the original metal and the original metal may only form a small fraction of the pipe wall. The graphitized portion of the gray cast iron is easily removed from the pipe by sandblasting which exposes the remaining wall thickness. This reduced wall thickness is vulnerable to failure if, for example, a surge in pressure occurs (Zamanzadeh et al., 1990).

Fig. 1. Gray cast Iron pipe graphitization

(from Gummow, 1984)

Galvanic corrosion is an example of electrochemical corrosion in which a cell may form from either contact between dissimilar metals in an electrolyte or contact similar metals in a dissimilar electrolyte. Galvanic corrosion due to dissimilar metal contact has been studied by Wakelin and Gummow (1990) and by Gummow (1984, 1988) and is shown in Fig. 2. The figure illustrates the direction of current flow which is from the anode (iron pipe) to the cathode (copper service line).

Wakelin and Gummow have investigated the corrosion rates of metallic pipes with respect to the attachment of a lead or a copper service lines. Values for gray cast and ductile cast iron pipe and the two types of service fittings were not separated and identified. Copper service lines attached to metallic pipe generate higher corrosion rates than lead lines. Since 1955, copper lines have been used for service connections (Wakelin and Gummow, 1990). The use of copper service lines could be one component influencing metallic pipe failures of pipes of less than 40 years of age and therefore requires special consideration for corrosion protection. Lead lines still exist as service connections but many cities and regions are in the process of removing them because of health related problems.

The driving force behind galvanic corrosion can be related to the difference between the surface area of the copper service line to that of the gray cast iron pipe. The corrosion rate increases for an increase in the copper service line surface area compared to the exposed surface area of gray cast iron pipe (Gummow, 1988). For gray cast iron water pipes that are protected with a coating, care must be taken to ensure the entire surface of the pipe is covered. A break in the coating results in an increase of localized pitting due to the increase in the ratio of copper service line to exposed gray cast iron pipe (Gummow, 1984).

Fig. 2: Interconnection of copper service piping to gray cast iron pipe

(from Gummow, 1988)

External corrosion of metallic pipe is also influenced by the type of soil surrounding the pipe. Soil resistivity can be used to measure indirectly the corrosiveness of the soil. Soil resistivity is a measure of the ability of soil to conduct electrical energy. The following table (Table 1) illustrates the relationship between soil resistivity and the severity of corrosion.

Table 1. Soil resistivity and harshness of corrosion attack

(from Heino and Taylor, 1987)

Soil Resistivity (ohm-cm)	Harshness of Corrosion Attack
< 500	extremely corrosive
500 - 1,000	severely corrosive
1,000 - 2,000	corrosive
2,000 - 5,000	moderately corrosive
5,000 - 10,000	mildly corrosive

Jakobs and Hewes (1987) have found a correlation between soil resistivity and pitting corrosion on metallic pipes. The present study will try to establish the same type of relationship.

In Canadian urban areas, the soil has increasingly been contaminated with deicing salts. This is a problem since an increase in the concentration of deicing salts (> 1000 ppm in some areas) reduces the soil resistivity which in turn can increase the iron corrosion rate (Gummow, 1984).

Microbiological corrosion results from the formation of sulfate-reducing bacteria under anaerobic conditions. This bacteria removes hydrogen from the cathodic processes thus enabling corrosion to continue. The cathodic protection is disabled. To eliminate the formation of the sulfate-reducing bacteria, the pH of the soil should be greater than 9 (Evans, 1960).

Corrosion due to stray current may also be a problem. Stray currents may be generated from such systems as electrical equipment, grounded D-C power, cathodically protected services (e.g. gas mains), faulty cables and electric rail lines (Ferguson and Nicholas, 1984 and Reedy, 1966). Boston (MS) has had problems with stray current corrosion from subway systems. The degree of the stray current corrosion is dependent on the soil resistivity.

Corrosion failures of gray cast iron pipe mains are usually characterized by an increase in the number of water main failures over a period of time (Lochbaum, 1993). Corrosion, by either galvanic, stray current or bacteriological actions, has severely damaged water mains by thinning the walls of these pipes (Zamanzadeh et al., 1990). Failures associated with other causes were not found to vary greatly with the age of the pipe (Fitzgerald, 1968).

The maximum pit depth is an important parameter in determining the extent of external corrosion and consequently the time to failure. However, graphite corrosion plugs which prevent the perforation from failing may prolong the time to failure of a pipe even though the pit has fully penetrated the pipe wall (Wakelin and Gummow, 1990). Recently (August, 1995) in the Regional Municipality of Ottawa-Carleton, graphitized plugs were popped out by a pressure surge or water hammer in the system. The age of the pipes at the time of the failure was 35 years; however, the time for complete wall graphitization is less than 35 years.

External corrosion by-products of gray cast iron water pipe have a strong adhesion to the graphite component and to the remaining metal. This adhesion strength may result in the corroded pipe being able to withstand considerable internal pressures even with through-wall graphitization (De Rosa and Parkinson, 1986). Gray cast iron graphitic corrosion is more likely to plug a perforation which will offset the occurrence of a leak or break in the main.

One method for reducing corrosion failures is to increase the pipe wall thickness. This is especially true for smaller diameter pipes which have a smaller wall thickness. The time to failure of these smaller pipes is shorter than larger diameter pipes. Another method is to isolate the metallic surface from the electrolyte (i.e. soil)

(Lehmann, 1964). This isolation can be achieved through the application of coatings or protective polyethylene wraps. The durability and resiliency of the material is important when determining the type of protective coating. The protective material must perform, at times, under adverse conditions (installation, freeze thaw cycles, etc.). A discontinuity in the protective coating or wrap can accelerate corrosion attack.

- temperature differential

Many case histories indicate that there is an increase in main failures between November and February (Russell, 1988). In a survey conducted by McDonald et al., (1994a), it was found that 58% of the water main failures in 1992 occurred during the four coldest months of the year. How temperature (air and water) influences water main failures is discussed.

As water travels through the pipe, water pressure exerts a radial pressure on the pipe wall. During the winter months, the buried water mains are exposed to cool conditions from the water that flows through them and the soil surrounding them which causes the main to contract. However, due to the close contact with the surrounding soil, the pipe may be restrained from moving. Longitudinal (tensile) stresses along the pipe will develop which will magnify those already imposed on the pipe by external loadings and internal pressures. These coupled with either a reduced wall thickness or external resistance induced hoop stresses (from inadequate bedding or swelling of clays) could cause the pipe to fail.

A majority of the water main failures experienced in Canada in 1992 and 1993 have been attributed to temperature differentials both in the summer and the winter (McDonald et al., 1994). The study shows that while low temperatures during the winter months influence the number of water main breaks, there have been reported cases of an increase in water main failure during the summer, and in particular during August. For a pipe that is already corroded, the increase in water demand in the summer months may be enough to burst the pipe. Utility operators are advised to increase the pressures slowly to avoid a pressure surge.

Another possible explanation for the increase in summer breaks is shrinkage of the clay soils. The cities that report a greater number of main failures in the summer have largely surficial clayey soil. As discussed previously, clays with a high concentration of montmorillonite are susceptible to volume changes and thus are able to impose tensile stresses on the pipe. These stresses may be sufficient to cause the failure of a corroded pipe.

It has been noted that penetration by frost may double the external stresses on the pipe (Habibian, 1994). The frost enters the ground after the air temperature has remained below freezing temperature for a few days.

- **manufacturing defects**

Manufacturing defects are suspected to be the cause of premature water main failures. Once the pipe has been delivered to the site and after it is moved either off the delivery truck or into the trench, visual inspection and ring tests are conducted to determine whether there are any cracks, splits or other manufacturing defects. The pipe is discarded if any defects are found.

- **impact**

Problems could arise for water mains when gas lines or sewer mains are repaired or replaced. During water main repairs, sewer laterals are sometimes broken. It is possible that the reverse could happen, that is, during sewer repair, the excavator could knock and break the water main. This could be avoided by cooperation and coordination between companies during repairs that must be done to the utility.

As discussed, there are many causes to water main failures. The age of the main is not the only legitimate parameter to use to determine whether a water main should be replaced. The condition of the main, not the age, should be the governing factor when establishing a replacement plan.

2.2.1 Water-Main Failure Modes

There are three basic types of water pipe failures: blowout, ring failure and beam failure. The type of failure can usually give an indication of its cause.

A blowout is the result of excessive hoop stresses at the weakest point of the pipe and is generally caused by excessive internal pressure. This pressure may be the result of improper design (pressure in excess of what the pipe was designed for) or a normal pressure on a weakened pipe. This weakness can be either due to corrosion or due to temperature induced stresses (Cave, 1987).

Ring failures result in longitudinal splits caused by excessive external pressures or loads and may be caused by improper embedments, inadequate soil depth to reduce the influence of traffic or frost loads, variation in site conditions where the external loadings on the pipe are increased and from a pipe that is weakened from other means (Morris, 1968, O'Day, 1982 and Cave, 1987).

Beam failures are characterized by circumferential failures and occur as a result of differential loadings on the pipe. The causes of the differential loadings may be due to uneven bedding, unstable soils, variation in soil types along the pipe length, point loads created by adjacent objects and finally, stress concentrations from pipe entering a structure (Morris, 1968, O'Day, 1982 and Cave, 1987).

The type of failure usually depends on the pipe diameter. For example, beam failures usually occur in pipes with diameters between 150 - 200 mm (6-8 ") while ring or crushing failures occur in pipes with large diameters, usually greater than 250 mm (10 ") (O'Day, 1982).

Table 2 summarizes the causes of water main failures as described by Ciottoni (1983).

Table 2. Types of water main failures and the causes

(from Ciottoni, 1983)

Type of Failure	Causes
Circumferential	temperature changes, beam loadings on small diameter pipes
Longitudinal	improper installation, traffic loads or construction, overloading of large mains
Blowout/holes	intense galvanic corrosion

Clarke (1968) has also defined types, causes and preventive measures for pipe failures. These are shown in Fig. 3.

Bahmanyar (1982) found that 75% of the breaks for an eleven year period in Madison, Wisconsin, were circumferential and the majority of failures occurred in 150 mm (6 ") and smaller diameter pipes. As shown in Table 2, circumferential failures can be caused by temperature changes and beam loadings on the pipe. Beam loadings on the pipe can occur as a result of non-uniform support or soil movement by consolidation, settlement or frost loading. Temperature differentials for smaller diameter pipes pose a problem especially during the colder months. The heat source provided by the water is less than that for larger diameters pipes, since small pipes carry less water. This smaller heat source may result in a possible deeper frost penetration (Bahmanyar, 1982).

Type of Failure	Appearance	Cause	Preventative measures
a) Overload fracture		Excessive vertical load or inadequate bedding	Higher bedding class, or stronger pipe, or concrete surround
b) Burst socket		Differential thermal or moisture expansion of jointing mortar	Resilient jointing material which does not cause excessive radial pressure on the socket
c) Distortion fracture		Differential heating or cooling or moisture content	Protection of uncovered pipes against sun or cold night (or drying wind, with concrete pipes)
d) Beam fracture		Uneven resistance of foundation, or soil movement, or differential settlement	Flexible joints and uniform hardness of foundation
e) Pull fracture		Thermal or drying shrinkage of pipe or site concrete, drying shrinkage of clay soil	Flexible-telescopic joints and gaps in site concrete at pipe joints
f) Shear fracture		Differential settlement of wall relative to pipe or vice versa	Flexible joints at least at A and B making AB not more than 3 ft.
g) Bearing fracture		Hard spots in pipe bed	Elimination of hard spots
h) Throat fracture		Restrained thermal or moisture expansion of pipe or compression due to subsidence	Flexible-telescopic joints which do not cause excessive radial pressure on the sockets. Spigot and not hard up in socket.
i) Leverage fracture		Excessive angular displacement	Avoidance of excessive slew when laying

Fig. 3. Types and causes of pipe failures

(from Clarke, 1968)

The occurrence of circumferential breaks in gray cast iron pipe presents a problem for gas utilities as well as for water distribution utilities. A model for predicting gas main breaks was proposed in a study by Lochbaum (1993) after completing a survey of 20 gas utilities across the United States. Interestingly, Lochbaum (1993) found that the indices of risk (rating system based on the size of the pipe and the break frequency) of large and small mains are almost equal even though the larger diameter mains carry a higher volume of gas and potentially pose greater problems upon failure. This equality in the risk index is due to the high incidence of circumferential failures of the small diameter mains. The index of risk is determined by multiplying the index of escaping gas (higher for larger diameter mains) by the frequency of breaks (higher for smaller diameter mains).

In determining the predictive model for gray cast iron gas main failures, the following relationships were determined (Lochbaum, 1993):

- the frequency of main breaks increases as the diameter decreases
- mains installed after 1949 have had a higher incidence of failures (pipe lengths were increased to 5.5 m (18 ') from 3.7 m (12 ') lengths)
- the likelihood of a failure within a main that has previously failed is higher than that for a main that has never experienced a failure
- soil resistivity influences the main break rate

In Lochbaum's study (1993), age was found not to influence the occurrence of main breaks for the following reasons: data were only available for the past 20 years and not all the installations of gray cast iron pipe had been documented. Also, since 1949, the pipe lengths have increased from 3.7 m to 5.5 m (12' - 18'). The increase in pipe length has led to an increase in beam type (circumferential) failures for small diameter pipes.

2.3 Case Studies

Little information exists to assist the water distribution operator in deciding when to replace a water pipe. Models have been developed to help predict pipe breaks as a function of pipe age, type of pipe, pipe diameter, occurrence of previous breaks and temperature. Using these prediction models requires extensive data, which is not always available as indicated in a recent survey (McDonald et al., 1994) of Canadian and American water distribution utilities. Often, the operators do not know when the pipe was installed nor the type of material of each main within the system.

A predictive model would have to be specific to an individual city. The rate at which water mains break varies from city to city, as indicated by the Canadian and American study. For example, in 1992, the water main break rate (number of breaks/100 km of pipe) for Vancouver (BC) was 4.7 and in North York (ON) it was 57.9 for the same year. In Los Angeles (CA) and Boston (MS) the main leak/break rates for 1992 were 23.1 and 1.7, respectively. Break rates also can vary within a city. The variance in deterioration can be attributed to varying soil conditions, pipe materials, pipe sizes and ages, varying environmental conditions, installation procedures, water quality and pressures.

The network condition is of great importance in deciding when and which mains to rehabilitate. The number of breaks for a city section is an important criterion in determining the network condition. However, physical testing of the mains should be performed and knowledge of the surroundings should also be used to confirm the condition of the deteriorated system. An analysis should be carried out to determine the extent of internal and external corrosion as well as the degree of tuberculation. Soil conditions should be considered since external corrosion may occur in corrosive soils and expansive soils may increase the external loads on the pipes.

The following section discusses testing programs carried out by various cities across North America and in Britain and Japan.

2.3.1 Britain

In 1975, in response to approximately 3000 complaints about colored water, members of the Soar Division of the Severn-Trent Water Authority devised a research program to study the performance of the existing system (Lackington and Large, 1980). Pipe samples were retrieved and corrosion levels were measured to determine whether a relationship between age, strength and degree of corrosion for 75, 100 and 150 mm (3, 4 and 6 ") water mains existed. The results of correlations for the Lackington and Large (1980) study had not been completed in this paper.

It was found that in sections of the distribution systems where bursting of water pipe was a problem, the residual tensile strength was as low as 80 MPa (compared to 170 MPa for new pipe) while in other areas, strengths were as high as 170 MPa (Lackington and Large, 1980). Soil parameters such as pH, moisture content and in particular resistivity were not analyzed with respect to the frequency of main bursts in this report.

This study also discussed the relationship between the number of pipe bursts or burst frequency with time (from 1951-1978), average age of pipe and air and ground temperature (from 1976-1977). After 1974, the number of unlined gray cast iron breaks (50% of the 4 559 km network) dramatically increased (from 0.3 bursts/km to 0.75 bursts/km). It was found that there were three times as many main bursts in the 8 weeks that the air temperature was below the ground temperature than during the remaining year.

This report has shown trends for the Leicestershire region with respect to the number of pipe bursts and the air and ground temperature. The history of the network with respect to the inventory of the system and the break data is impressive. The authors had spent 2 years evaluating the parameters affecting the Leicestershire's network. They had pointed out that still more work needs to be done before conclusions can be drawn. They had also noted that the water industry should strive to supply a level of service to its customers comparable to that supplied by the gas and electrical utilities.

2.3.2 Edmonton, AB

In 1985, gray cast iron pipe accounted for 40% of Edmonton's water distribution system; however, gray cast iron pipe accounted for 90% of the breaks (Heino and Taylor, 1987). There are variations from year to year, but the general trend is that the break rate is increasing. This increase can probably be attributed to the weakening and deterioration of the system. A long range plan to remove 20 km of gray cast iron pipe per year was initiated in 1987.

Corrosion in the form of graphitization, has weakened nearly all the gray cast iron pipe in place. The following factors, at pipe depth, have led to the destruction of the gray cast iron pipe:

- low soil resistivity
- minerals, in particular sulfates
- clay soil type

The criterion used for replacement of gray cast iron pipe in the city of Edmonton was (Heino and Taylor, 1987):

$$\text{breaks/km/year} = \text{number of breaks in 5 years} / \text{total network water main length} \quad (1)$$

If the break frequency is greater than 5, the section of pipe is up for renewal.

2.3.3 Calgary, AB

A study was conducted in Calgary (AB) between 1982 and 1985 to determine the causes of water main failures (Jakobs and Hewes, 1987). The majority of the 2 750 km of water-distribution pipe in place as of 1980 was gray cast and ductile cast iron.

Six gray cast iron pipe samples were analyzed to determine the pitting rate. The average pitting rate for these samples was determined to be 0.41 mm/year (16.14 mil/year) and this rate increased with decreasing soil resistivity (Jakobs and Hewes, 1987). The pitting rate corresponds to a range of soil resistivities from 560 to 4 300 ohm-cm. The peak in water main failures occurred during the winter months in the

period from 1969 - 1975. The failures during the colder months (from October to March) out-numbered those in the remaining months by as much as 5 to 1.

It was found that 60% of the water main failures in the period from 1956 to 1980 were associated with corrosion. The gray cast iron pipe samples exhibited localized corrosion in the form of pitting. The soil type in the Calgary area is either wet clay or silty clay, containing a considerable amount of dissolved solids.

2.3.4 Philadelphia, PA

The city of Philadelphia initiated a physical testing program in 1982 (Philadelphia Water Department, 1985 and Durrant, 1986) as part of a program to determine the present and future condition of their water distribution network. The present and future condition of the water distribution network was assessed based on analysis of the main breaks. A model was developed to determine the causes of the main failures and also developed were a main break profile and cost models for assessing the repair, replacement and maintenance alternatives. Durrant (1986) presents the soil and pipe sampling and testing methodology followed for the program and the final procedures that will be carried out to achieve the above mentioned goals.

From the Water-Main Pipe Sample Physical Testing Program Report (1985) the tensile strength testing program was based on 46 samples collected with ages ranging from 34 to 134 years. Pipes 34 years old had a range of tensile strengths between 151.7 and 227.5 MPa (22 - 33 ksi). No trend between strength and age was observed for pipes that were between 50 and 134 years old was found. Most samples had a strength of 103 MPa (15 ksi) and the range in reported tensile strength values was 186.2 MPa (27 ksi) for an 80-year old sample and 69 MPa (10 ksi) for a sample that was 65-years old (Philadelphia Water Department, 1985).

The average rate of external corrosion was 0.04 mm/year (1.5 mils/year) but ranged from 0.007 to 0.12 mm/year (0.28-4.85 mils/year). There was no relationship between the age and the pitting rate. A weak correlation existed between the soil resistivity and the rate of external corrosion. It was thought that stray currents accelerated the

corrosion rate apart from the rates expected for soil resistivities ranging from 1 500 - 10 000 ohm-cm.

2.3.5 Washington, DC

A study on water main condition was performed by the Washington Suburban Sanitary Commission from November 1986 to June 1987 (Russell, 1988). Parts of the study dealt with the analysis of pipe rings to determine the extent of external and internal corrosion and to determine a correlation between pipe failures over time, with respect to environmental conditions, and pipe characteristics (age, material, size).

The analysis of data (1976-1985), indicated that 57.5% of the breaks took place during the four coldest months (November-February). It was found that when the air temperature dropped most rapidly for the first time, there was an increase in the number of water main failures. The extra loading on the pipe (frost) was substantial to cause the pipes, already weakened by corrosion, to fail.

External corrosion in approximately 90% of the samples was found to be in the range of 0-20% of the original wall thickness. A reduction in wall thickness of up to 20% is not expected to lead to a water main failure when distributed uniformly over the pipe, therefore, other factors should be considered as contributing to water main breaks. The average pitting rate of these samples was found to be 0.08 mm/year (3.06 mil/year) with a range from a low of 0.02 mm/year to a high of 0.19 mm/year (0.86 - 7.7 mil/year).

2.3.6 Japan

Yamamoto et al. (1983) carried out a testing program including tensile testing, ring tests and fatigue tests to establish a relationship between graphitic corrosion and the strength of gray cast iron pipe. The tensile strength for 200 mm (8 ") ID samples ranged from 134.9 - 147.2 MPa for samples not exhibiting metal loss due to corrosion (Yamamoto et al., 1983). A relationship was established between graphitization (metal loss) and the tensile strength.

Yamamoto et al. (1983) determined that the rate of corrosion of gray cast iron water pipe was between 0.167 mm/year and 0.25 mm/year (6.57 - 9.84 mil/year). These rates

were for samples with ages ranging from 22 to 79 years. Internal corrosion progressed at a greater rate than external corrosion.

2.4 Summary

Tensile strength values from the literature for gray cast iron water mains can be summarized as follows.

Table 3. Reported values of tensile strength for gray cast iron pipes

Investigator	Tensile Strength, MPa (ksi)
Lackington and Large (1980)	80* (11.6)
Lackington and Large (1980)	80 -170 ** (11.6 - 24.7)
Beer and Johnston (1985)	170 (24.7)
Ciottoni (1985)	124 (18)
Bonds (1995)	124 (18)
Yamamoto et al. (1983)	134.9 - 147.2 (19.6 - 21.3)
Ferguson and Nichols (1984)	180-300 (26.1 - 43.5)

Note: * where bursting pipe a problem and ** where no previous bursts were recorded.

The rehabilitation of water distribution mains must be carried out in a more rational manner because of higher demands on city and municipal budgets. A pipe renewal program can only benefit from performing physical tests to determine relationships between the remaining strength with respect to remaining material thickness, the pitting rate and the corresponding soil resistivity. General relationships between the failures of pipes and the design parameters can be established if a data base is well documented. These design parameters should include construction practices, material type, soil conditions, vehicle loadings, proximity to existing structures and accidental impact.

CHAPTER 3

PRELIMINARY STUDIES

3.1 Introduction

In this chapter, material related to the inventory of Canada's water-distribution networks is presented. Outlined will be the specifics of the pipe components, the types of failures and the materials involved. Finally, the gray cast iron water main break patterns for 24 cities and a comparison between a city's freezing index and the number of gray cast iron main failures will be discussed.

3.2 Inventory

At the present time, information is not readily available on the inventory of an individual city's water-distribution network. A questionnaire (McDonald and Félio, 1995) was sent to various cities across Canada in order to obtain this information. Four population groups were targeted. These groups were based on population of less than 10,000, 10,000-50,000, 50,000-100,000 and greater than 100,000. The response rate was at least 75% for all groups targeted.

The exact length of water distribution pipe in service in Canada is not known. However, a representative sampling (approximately 30% of Canada's population) has been obtained. The survey (McDonald and Félio, 1995) corresponds to a total population of 8.5 million, the area is close to 8 800 km² and the total length of pipe in the ground is approximately 30 000 km.

The responding cities, along with their population, area, length of water pipe and their density (length of pipe in km/km²) are shown in Tables 4 and 5. It is to be noted that Montreal (QC) serves the largest population and has the most densely developed network, Peterborough (ON) covers the largest area and Calgary (AB) has the longest water-distribution system.

Table 4. City, population, area, length and density of water-distribution pipe for cities with a population greater than 50,000 for 1992
(from McDonald and Félio, 1995)

City/Region	Population	Area (km ²)	Length of pipe (km)	Density (km/km ²)
Vancouver, BC	475 000	115.5	1 458	12.6
Victoria, BC	88 200	30	425	14.2
Burnaby, BC	166 000	95.6	655	6.8
Calgary, AB	732 000	695	3 435	4.9
Edmonton, AB	625 300	450	2 754	6.1
Regina, SK	190 000	110	750.6	6.8
Winnipeg, MN	622 000	280	2 477	8.8
Sault Ste. Marie, ON	81 000	80	430	5.4
London, ON	315 000	166	1 270	7.6
Windsor, ON	216 000	125	900	7.2
Kingston, ON	60 000	n/a	230	n/a
Peterborough, ON	75 000	3 942	360.4	.1
Sudbury, ON	90 000	300	400	1.3
Etobicoke, ON	300 000	96	1 000	10.4
Scarborough, ON	510 000	185	1 250.4	6.8
North York, ON	560 000	160	1 380	8.6
East York, ON	n/a	21	209.6	9.9
York, ON	134 000	14.6	281	19.2
Ottawa-Carleton, ON	600 000	400	2 300	5.8
Chicoutimi, QC	64 000	157	392	2.5
Sherbrooke, QC	75 000	32	373	11.7
Montreal, QC	1 500 000	60	2 720	45.3
Quebec City, QC	170 000	88.9	700	7.9
Halifax, NS	165 619	83	445	5.4
Saint John, NB	65 000	100	403	4.0
Moncton, NB	50 000	66	380	5.8
St. John's, NF	109 240	490	420	.9
Total	8 038 959	1 182.2	27 799	-

Table 5. City, population, area, length and density of water-distribution pipe for cities with a population less than 50,000 for 1992

(from McDonald and Félio, 1995)

City/Region	Population	Area (km²)	Length of pipe (km)	Density (km/km²)
Prince Rupert, BC	17 000	n/a	n/a	n/a
Trail, BC	8 000	13.2	68	5.2
Kimberly, BC	6 500	n/a	88	n/a
Medicine Hat, AB	45 850	47	314.8	6.7
Yellowknife, NWT	16 000	15	62.2	4.1
Drumheller, AB	7 500	25	60	2.4
Moose Jaw, SK	36 500	48	259	5.4
Yorkton, SK	15 500	12	114	9.5
Weyburn, SK	9 800	6.13	76.5	12.5
Brandon, MN	39 000	24	211.3	8.8
Pembroke, ON	14 000	24	92	3.8
Perth, ON	5 500	8.9	40.5	4.6
Shawinigan, QC	20 900	27.2	145	5.3
Lennoxville, QC	4 000	n/a	n/a	n/a
Grand Falls, NB	6 000	n/a	n/a	n/a
Fredericton, NB	45 000	100	290	2.9
Yarmouth, NS	9 500	10.9	57	5.2
Amherst, NS	10 000	30	77.2	2.6
Sydney, NS	29 000	23.3	145	6.2
Charlottetown, PE	16 000	40	112.7	2.8
Summerside, PE	9 000	15	75	5.0
Corner Brook, NF	23 660	16.5	140	8.5
Mount Pearl, NF	26 000	15.5	100	6.4
Total	420 110	501.6	2 528.2	-

The dominant soil material in which the pipes are installed is clay. Clay accounts for approximately 62% of the native materials, as shown in Table 6. Coarse and other fine-

grained soils account for approximately 15% each of the native soil materials. Seven cities -- Edmonton (AB), Regina (SK), Yorkton (SK), Winnipeg (MN), North York (ON), Chicoutimi (QC) and Moncton (NB) -- indicated that their networks were installed exclusively, or almost exclusively, in clay. In St. John's (NF), on the other hand, pipes are installed, for the most part, in coarse-grained soils and rock. The water-distribution systems of the remaining cities have various concentrations of the different soil types as listed in Table 6.

Table 6: Distribution of soil type for the water-distribution networks for 1992

(from McDonald and Félio, 1995)

Material	Percentage
clay	61.6
fine grained	15.2
coarse grained	15.7
rock	5.6
other	1.9

The dominant pipe size used is 200 mm (8 ") or less as shown in Table 7. This pipe size is used for nearly 72% of the water-distribution networks.

Table 7. Distribution of water-distribution pipe size for 1992

(from McDonald and Félio, 1995)

Pipe Size (mm)	Percentage
0 - 200	71.6
200 - 500	22.0
> 500	6.4

Table 8 presents the types of pipe materials and the proportion of each used for the water-distribution networks. Approximately 74% of the networks are constructed of gray cast or ductile cast iron pipe while PVC and asbestos-cement constitute 10.4% and 10.1% respectively, of the water-distribution networks. However, the proportions of

materials used by a city vary from one region of the country to another. Some Western cities, for example, have a significant portion of their network made of asbestos-cement: Regina (SK), 71.3%, Weyburn (SK), 77.1% and Drumheller (AB), 45%. The use of PVC varies across the country.

Table 8. Distribution of water-distribution pipe materials for 1992

(from McDonald and Félio, 1995)

Material	Percentage
gray cast iron	52.8
ductile cast iron	21.5
PVC	10.4
asbestos-cement	10.2
concrete	3.3
steel	1.2
steel with lining	0.4
PE	0.1
other	0.1

The age distribution of the pipe is shown in Table 9. The age group 11 to 30 years old is the most common (32%), while 26% of the total length of all the pipe is over 50 years old. There are four cities with at least 65% of their network greater than 50 years: Saint John (NB), Charlottetown (PE), Victoria (BC) and York (ON).

Table 9. Age distribution of the water-distribution network for 1992

(from McDonald and Féllo, 1995)

Age (year)	Percentage
0-10	14.3
1-30	32.0
30-50	25.9
50-75	14.0
>75	12.2
unknown	1.6

3.3 Water Main Failure Modes

Thirty-three of the cities listed in Tables 4 and 5 provided responses that could be used for comparisons of the type and proportion of each type of water-main break experienced. These thirty-three cities account for 24 708 km or approximately 81%, of the total length of water pipe represented in these questionnaires. Circumferential breaks are the type of failure that occur most (Table 10) frequently. The next two most prevalent types were those associated with service connections and pipe bursting. Pipe bursting normally results from a pressure surge and can be quite dramatic resulting in a large hole in the pipe wall. The types of material associated with these breaks, in most cases, were either gray cast iron or ductile cast iron. As shown previously, these materials account for 74% of the total networks.

Table 10. Distribution of the types of water main breaks for 1992

(from McDonald and Félío, 1995)

Type of Failure	Percentage	Type of Pipe Material
Circumferential	38.5	Gray cast iron/Ductile cast iron/Asbestos cement
Service Connections	22.3	Gray cast iron/Ductile cast iron
Bursting of the pipe	14.4	Gray cast iron/Ductile cast iron/Asbestos cement
Corrosion	9.3	Gray cast iron/Ductile cast iron/Steel
Longitudinal	7.9	Gray cast iron/PVC
Joints	7.6	Gray cast iron/Ductile cast iron/Concrete

It is interesting to note that gray cast iron pipe, which accounted for 53% of the water-distribution networks, has failed or been associated with all of the above failure modes. It is noted that the average age of gray cast iron pipe is between 40 and 50 years while the oldest ductile cast iron pipe is 34 years (Rajani and McDonald, 1995) and the oldest PVC pipe is 34 years old (Rajani and McDonald, 1995). Other factors which influence the occurrence of gray cast iron water main failures such as soil bedding, backfill materials, ground water conditions, climate, installation and operation procedures were not considered when determining this number.

3.4 Gray Cast Iron Break Patterns

Cities establish their rehabilitation programs for pipes based on a number of criteria. Common criteria used include inadequate pressure, colored water, other utilities in the area under repair, unaccounted water loss, high break history, etc. Without digging up the pipe or passing a non-destructive tool through the distribution system, operators do have the means of determining the condition of their networks. The condition of the network can be estimated from viewing the number of water main breaks/month for consecutive years.

Gray cast iron is the oldest group of pipe in place at the present time and comprises over 50% of the networks. For these reasons gray cast iron pipe will be the only pipe material examined in this study. The following figures show the gray cast iron water main breaks per 100 km of cast iron water pipe for 1992 for twenty-four major Canadian cities. The cities are grouped together according to the month or months in 1992 where the maximum number of water main failures occurred. The data were obtained through a survey on water main break data on different pipe materials (Rajani and McDonald, 1995).

Figs. 4 to 7 illustrate various seasonal pipe-break patterns for 1992 for gray cast iron pipe. The cities were grouped according to the type of break pattern they experienced; that is, the pattern generated by the number of breaks per 100 km of gray cast iron pipe per month. The total pipe length in each city is indicated in the legend of each figure.

Figs. 4a-4d illustrate a one-peak pattern in breaks/100 km of gray cast iron pipe/month. Fig. 4a illustrates cities that have their greatest number of gray cast iron pipe breaks in the month of January. The cities included in Fig. 4a are North York (ON), Scarborough (ON), London (ON), Pembroke (ON) and Halifax (NS).

Fig. 4b shows cities with a one peak in gray cast iron pipe breaks. The peak for the cities of Calgary (AB), Sherbrooke (QC), Moncton (NB) and Mount Pearl (NF) is in November.

Fig.4a. One-peak break pattern: January peak

Fig. 4b. One-peak break pattern: November peak

The cities included in Fig. 4c also have a one-peak pattern of gray cast iron pipe breaks/100 km of cast iron pipe. The cities of Victoria (BC), Vancouver (BC), Windsor (ON), Peterborough (ON) and St. John's (NF) have their greatest number of gray cast iron pipe breaks in December.

Fig. 4d includes cities with a peak in pipe breaks in either February, March or June. Ottawa-Carleton (ON), Fredericton (NB) and Moose Jaw (SK) are included in this figure.

There are three cities which had two peaks in gray cast iron breaks in 1992 as shown in Fig. 5. Sudbury (ON) had a peak in August and again in November, Sydney (NS) had a peak in July and in December and Corner Brook (NF) had a peak in June and again in November.

Fig. 6 illustrates the pattern of pipe breaks for Edmonton (AB) and Regina (SK) and they experience their main peak during the late summer. Edmonton (AB) and Regina (SK) have a peak in August. This pattern could be attributed to failure of corrosion weakened pipe due to the increase in water demand during the summer months or the increase in tensile stresses on the pipe induced by the surrounding shrinking clay soil. As noted from the legend, Regina (SK) has only 15.15 km of gray cast iron pipe in place.

Fig. 4c. One-peak break pattern: December peak

Fig. 4d. One-peak break pattern: various months

Fig. 5. Two-peak break pattern

Fig. 6. Late summer peak

There are two cities, Summerside (PE) and Burnaby (BC) (Fig. 7) in which the number of gray cast iron breaks/100 km of gray cast iron pipe does not follow a particular pattern. Summerside (PE) had all of its gray cast iron breaks from January to April and in December and Burnaby (BC) had the same number of breaks in January, November and December.

Figs. 8 and 9 illustrate the number of gray cast iron pipe breaks/100 km of gray cast iron pipe for 1992 and 1993, as measured against the freezing index. The freezing index is the summation of the average daily temperatures less than 0 °C for the year. With approximately 58% of the breaks occurring during the four winter months (January, February, November and December), it will be interesting to see whether there is a relationship between the number of breaks and the freezing index. In a study by Lothbaum (1993), the number of gray cast iron gas main failures increased exponentially when the freezing index was greater than 1200 degree days. Fig. 8 does not clearly confirm the exponential increase in breaks for a high freezing index. However, Fig. 9 shows an exponential increase in breaks for an increase in the freezing index. But this trend was not clearly established so it can be concluded that other parameters other than temperature (e.g. soil corrosivity, soil type, pipe size, material and age) influence the occurrence of gray cast iron water main failures in these cities for 1992 and 1993.

Gray cast iron water pipes, at the present time, constitute approximately 53% of the water distribution networks across Canada. The age of this pipe material in the networks is potentially as young as 25 years to greater than 100 years. There is a need to determine the present condition or remaining strength of these water distribution pipes with such a large length of the networks constructed from a material that is aging. One way is to perform physical testing on the pipe and try to correlate the strength with age and remaining pipe material.

Fig. 7. Random break pattern

Fig. 8. Comparison of the number of gray cast iron pipe breaks/100 km of gray cast iron pipe versus the freezing index for 1992

Fig. 9. Comparison between the number of gray cast iron pipe breaks/100 km of gray cast iron pipe versus the freezing index for 1993

3.5 Summary

Gray cast iron pipe still forms the majority of the water-distribution pipe in place today. This pipe material has also failed in all the modes of failure as listed in Table 10.

The gray cast iron break patterns for 1992 vary from city to city. No apparent trend was observed between the number of gray cast iron breaks for each city and their freezing index for 1992. For 1993 there was slight agreement with an increase in the number of breaks for an increase in the freezing index as found by Lochbaum (1993). Water main failures due an increase in the freezing index (frost penetration) are also influenced by pipe diameter, native backfill and bedding material.

CHAPTER 4

FIELD STUDIES

4.1 Introduction

The excavation of gray cast iron water distribution pipes in Gatineau (QC), as part of a water main renewal, provided an opportunity to obtain corroded samples of pipe for mechanical tests. Prior to mechanical testing, the pipes' pits were measured and the soil resistivity analyzed to determine whether this is a relationship between the pitting rate and the soil resistivity. This chapter discusses the methods and equipment used to determine the pitting rate of the gray cast iron pipe and the soil resistivity of the trench soils.

External corrosion was believed to be the major problem of the pipes in Gatineau. From geophysical surveys that were carried out before the trench was open as part of another project, it appeared that the soil resistivity was low (Baker et al., 1996). From the literature review, low resistivity can be an indicator of high external corrosion pitting. For that reason soil resistivity was the only external corrosion parameter investigated in this thesis. Internal corrosion was not investigated due to the lack of tubercles on the inside of the pipes and because of this, water quality data was not collected for this project.

4.2 Gatineau

The following section discusses the area from where the pipe samples were obtained and describes the condition of the pipes that were tested. The replacement section covered approximately 165 m (540 ') of Carle St. in Gatineau (QC) and in total 33 pipe sections of 5.6 m (18 ' 5 ") were excavated during late August, 1994. These gray cast iron pipes were installed in 1970. As a result of numerous water main failures over a period of time on this street, the 150 mm (6") diameter gray cast iron pipe was replaced with 200 mm (8") diameter cement-lined ductile cast iron pipe, cathodically protected.

The depth to the invert was 2 m (6' 7") and the depth to the obvert was 2.3 m (7' 7"). In most sections of the trench, the bedding and backfill material was clay and a mixture of sand and clay was observed through one 3.0 m (10 ') segment of the trench. When the pipe was excavated, soil samples were taken every 3.0 m (10 ') to determine the soil resistivity from the base of the trench and where the bottom of the pipe lay.

4.3 Soil Resistivity

Soil resistivity is an important parameter in determining whether the soil environment is corrosive. It is anticipated that metallic pipes excavated from low resistivity soils would potentially exhibit higher levels of external corrosion damage resulting in a greater degree of metal loss or more pitting of the pipe.

The soil box method was used to determine the soil resistivity. The equipment used included a M.C. Miller Co. Soil Box, an external 12 V battery and a MCM B3A2 analog multimeter.

Typically, the sample was prepared by compacting the soil into the soil box to approximately the same level of compaction as the material in the field. Two brass pins were removed before filling the box then replaced before testing. The power supply and the milli-ammeter were connected to facilitate the current passage between the end terminals of the soil box. The voltmeter was connected to the two brass pins.

The soil resistivity is determined by the following procedure. The potential between the two brass pins without the applied current is first measured and recorded. Then, the potential between the pins with a current applied is measured. The voltage across the pins and the corresponding current applied is recorded.

The soil resistivity in ohm-cm is given by:

$$\text{soil resistivity} = \frac{\text{change in potential (mV)}}{\text{current (mA)}} \quad (2)$$

The dimensions of the box are such that when it is filled, the corresponding units for soil resistivity will be ohm-cm.

It should be noted that the change in potential is the absolute arithmetic difference between the potential with and without current, because the initial potential may be of opposite polarity to that when the current is applied.

Soil resistivity is a function of both temperature and moisture content. In order to duplicate field conditions, it is therefore advised to run the tests at a temperature similar to that in the field. Once the sample has been collected, it should be properly sealed and measurements made as soon as the sample is removed from the container, to ensure the moisture content is as close to field conditions as possible.

The soil resistivity results are shown in Fig. 10. It has been noted that soil resistivities below 2 000 ohm-cm have been observed in corrosive soils (Heino and Taylor, 1987). By observing the soil box plot of the test section one can see that the entire region has a soil resistivity below 2 000 ohm-cm. These measurements will be the reference readings of soil resistivity for the test section. Table A-1, in Appendix A contains information dealing with each pipe number and the corresponding road chainage.

Fig. 10 shows 5 distinct dips in soil resistivity over the test section. The first incidence of low resistivity occurred at approximately the 24.4 m (80 ') mark. The second low resistivity was at approximately 48.8 m (160 '). The third low section was between 73.1 and 85.3 m (240 and 280 '). The next low soil resistivity occurs at approximately the 118.9 m (390 ') point. The final point of low resistivity occurred at approximately 115.4 m (510 ').

The pipes chosen for the mechanical testing were picked from areas of low and high resistivity. These pipe numbers are indicated on Fig. 10 according to their road chainage. This selection was made to confirm the effect of soil resistivity on the pipe condition and it was hoped to obtain samples with varying levels of metal loss. All the pipes appeared to be in good condition upon visual inspection. Figs. 11 and 12 show the condition of the pipes after they were removed from Carle St. At this point, it was difficult to determine which areas have been subject to corrosion attack. The areas of corrosion were evident after the pipes were sandblasted. Fig. 13 shows an example of a severely pitted piece of pipe. Fig. 14 shows the cross-sectional wall thickness reduction due to the external corrosion.

Fig. 10. Soil resistivities for Gatineau test site and location of pipe sections for pitting analysis and tensile testing

(Baker et al., 1996)

Fig. 11. Excavated pipes prior to sandblasting: sample A

Fig. 12. Excavated pipes prior to sandblasting: sample B

Fig. 13. External corrosion pitting

Fig. 14. Wall thickness reduction due to external corrosion

4.4 Pipe Pitting Analysis

The sample group consisted of gray cast iron water mains, ID 150 mm (6") from Carle St. in the City of Gatineau (QC). These pipes were manufactured by CANRON (CSA Class 22) between August 24, 1970 and September 14, 1970. The age of the pipes, at the time of excavation, was 24 years.

The excavated samples were sandblasted to remove the dirt and graphite from the surface. Before this was done the pipe had no visible defects and it was difficult to identify sections of the thirty-three 5.6 m (18' 5") pipes that had corrosion. However, after the sandblasting, the areas affected by corrosion were apparent.

The pipes were brought to the National Research Council (NRC) campus on Montreal Road in September, 1994. At that time, the pits in each pipe section were measured. Each pipe was divided into top and bottom sections of 0.61 m (2') lengths along the length of the pipe with the bell end of the pipe defining the starting point. The top of the pipe had been identified at the time of excavation by spraying it with paint to maintain the *in-situ* orientation.

Pit hole measurements were characterized by depth and diameter (or length and width) dimensions. The equipment used for measuring the dimensions were a depth gauge and a caliper. The depth gauge had a 12.7 mm (0.5") stroke with a resolution of 0.01 mm (0.0001") and a dial marked in inches. The caliper was 150 mm (6") in length. It had a digital output with a 0.001 mm (0.0005") resolution.

The majority of the pits were circular and randomly distributed along the pipe, however, other shapes and placement patterns were observed. There were a few long and thin slits that ran longitudinally. Also, bands of pits were observed circling the pipe near the bell. The bands were approximately 10 mm (0.4") wide and there were smaller pits within the band width. Finally, a greater density of pits was observed at the point where the corporation fitting (main stop) enters the main. This could be attributed to increased galvanic corrosion caused by the dissimilar metals (gray cast iron and copper).

Pit diameters and depths were measured from seven sections. From Fig. 15, it is shown that the majority of the pits in these pipes had diameters between 1 and 10 mm.

Fig. 16 illustrates the distribution of the pit depths. The majority of the pits are between 0.1 and 2 mm. This corresponds to a metal loss up to 20% of the original wall thickness. This value was determined based on the average wall thickness (9.2 mm) divided into the largest depth within the range of maximum distribution (2 mm). It has been found that up to 20% material loss was common from other studies done on water distribution gray cast iron pipe removed from service after a period of time (Russell, 1988).

4.4.1 Pitting Distribution Determination

An analysis was conducted to determine whether the pitting rate of the pipes was random. The pits depths and volumes for pipe sections 4a, 4b, 6, 10, 26b, 29 and 32 were fitted to statistical distributions. Volume was chosen for this analysis instead of diameter since volume loss is a more significant corrosion measure than diameter alone. When the data were collected, the diameter and depth of each pit was recorded enabling the volume of each pit to be calculated.

A known failure-rate distribution, the Weibull 2 parameter, was chosen for fitting the data. The probability density function ($f(x)$) for the Weibull distribution is the following:

$$f(x) = \lambda \beta (\lambda x)^{\beta-1} e^{-(\lambda x)^\beta} \quad (3)$$

A trial and error process was performed by observing the probability density function of the data, assuming values for the shape (β) and scale (λ) parameters and comparing the actual and generated cumulative density functions. The cumulative density functions for each pipe are shown in Appendix B. The Kolmogorov-Smirnov Goodness of fit test was used to determine when appropriate values of the shape (β) and scale (λ) parameters were found. The results of this analysis are presented in Table 11 for depth and Table 12 for volume.

Fig. 15. Pit diameter distribution for Gatineau cast iron water distribution pipe

The Kolmogorov-Smirnov goodness of fit test used compares the actual or experimental cumulative frequency with the generated or theoretical cumulative frequency. A comparison is made between the maximum difference between the experimental and theoretical cumulative frequencies (d_n) and the critical value (c) which is defined by the following equation:

$$\text{at 10\% confidence} \quad c = \frac{1.22}{n^{1/2}} \quad (4a)$$

$$\text{at 5\% confidence} \quad c = \frac{1.36}{n^{1/2}} \quad (4b)$$

Where n is the sample size. The value of maximum difference between the experimental and theoretical cumulative frequencies (d_n) must be less than the critical value (c).

The pit depths all follow a form of the 2 parameter Weibull distribution. Pipes 4b and 10 with a shape parameter of 1 are forms of an exponential distribution. Pipe 26b with a shape parameter of 2 follows a Rayleigh distribution. The exponential and the Rayleigh are special cases of the Weibull distribution. All of the pit volumes follow the 2 parameter Weibull distribution with varying scale and shape parameters.

The purpose for determining the extreme value distribution type is to be able to determine the central tendency (mean), the maximum (95th percentile) and the minimum (5th percentile) values, in this case pit depths or volumes for each pipe. Once the distribution type and parameters are determined, the mean 5th and 95th percentile are easy to determine instead of manipulating the data set itself. Many extreme value distributions were tried, for example, the poisson, log-normal, exponential, but the 2 parameter Weibull characterized the data best.

Another parameter to investigate is the rate of pitting. The rate of pitting is determined by averaging the pits' depth measurements made and dividing by the number of years the pipes were in service (24 years).

Table 11. Depth distribution results

Pipe Number	Distribution of Pit Depths	Goodness of Fit	
	Parameters	Significance	Parameters
4a	$\lambda = 2$ $\beta = 0.75$	5%	$d_2 = 0.0912$ $c = 0.1475$
4b	$\lambda = 1.4$ $\beta = 1$ (Exponential)	5%	$d_2 = 0.0757$ $c = 0.0797$
6	$\lambda = 1.5$ $\beta = 0.4$	5%	$d_2 = 0.035$ $c = 0.1062$
10	$\lambda = 0.9$ $\beta = 1$ (Exponential)	5%	$d_2 = 0.0666$ $c = 0.07218$
26b	$\lambda = 0.31$ $\beta = 2$ (Rayleigh)	10%	$d_2 = 0.0718$ $c = 0.072$
29	$\lambda = 3.9$ $\beta = 0.9$	10%	$d_2 = 0.0498$ $c = 0.05088$
32	$\lambda = 1$ $\beta = 0.71$	5%	$d_2 = 0.0602$ $c = 0.063$

Table 12. Volume distribution results

Pipe Number	Distribution of PIt Volumes	Goodness of Fit	
	Parameters	Significance	Parameters
4a	$\lambda = 2$ $\beta = 0.1$	5%	$d_2 = 0.0592$ $c = 0.1475$
4b	$\lambda = 1.75$ $\beta = 0.1$	5%	$d_2 = 0.0721$ $c = 0.07972$
6	$\lambda = 1.75$ $\beta = 0.4$	5%	$d_2 = 0.1037$ $c = 0.1062$
10	$\lambda = 1.75$ $\beta = 0.1$	5%	$d_2 = .0546$ $c = .0721$
26b	$\lambda = 1.5$ $\beta = 0.08$	10%	$d_2 = .0519$ $c = .06016$
29	$\lambda = 2.97$ $\beta = 0.1$	5%	$d_2 = .0406$ $c = .042459$
32	$\lambda = 4$ $\beta = .075$	5%	$d_2 = 0.0557$ $c = 0.631$

The pitting rate varied from pipe to pipe (Table 13) for those excavated in Gatineau (QC). The average pitting rate was found to be equal to 0.049 mm/year (1.92 mil/year) with a range of 0.0018 mm/year (0.07 mil/year) to 0.348 mm/year (13.72 mil/year) for an average soil resistivity of 1 200 ohm-cm.

Table 13. Results of pipe soil resistivity and pitting rate for Gatineau (QC) gray cast iron pipe

Pipe No.	Soil Resistivity (ohm-cm)	Average Pitting Rate (mil/year)	Pitting Rate- Low (mil/year)	Pitting Rate- High (mil/year)
4a	1 487.1	0.72	0.07	2.40
4b	1 439.6	1.38	0.07	5.50
6	934.8	1.83	0.14	13.72
10	1 141.4	2.26	0.18	11.82
26b	1 397.5	3.08	0.14	13.65
29	961.9	1.36	0.14	9.99
32	1 051.9	2.83	0.18	13.23

The following linear relationships ($y = mx+b$) were established between the soil resistivity and the pitting depth (low, average and high). The constants of each equation and the correlation coefficient (r^2) value for the regressions are shown in Table 14. The corrosion pits determined from the linear equation are shown with the test points in Fig. 17. It can be noted that there is a trend for an increase in pit depth with a decrease in soil resistivity especially for the maximum pit depth. The r^2 value of 0.43 is low but still indicates a relationship. However, a linear relationship does not characterize well the distribution of average and low pit depths and soil resistivity, as indicated by the values of r^2 obtained. This relationship may be higher if a larger range of soil resistivities were used. All the samples from the Carle St. site are in a corrosive resistivity range and form a narrow band of soil resistivities (930 - 1 500 ohm-cm). Other factors thought to influence galvanic corrosion such as moisture and oxygen content may have affected the pitting rate or the pit depth of these pipes.

Fig. 17. Linear relationship between the pit depths and the soil resistivity

Table 14. Regression constants determined for the pitting depths

Pit depth group	r^2	slope (m)	intercept (b)
average	0.041	-5.3E-04	2.02
low	0.486	-9.7E-05	0.21
high	0.428	-8.93E-03	17.97

A summary of the pitting rates obtained by other researchers as well as the rate established for gray cast iron pipe from Gatineau (QC) is shown in Table 15.

Table 15. Comparison of pitting rates for gray cast iron pipe found by others

Reference	Average pitting rate mm/year (mil/year)	Range of pitting rates mm/year (mil/year)	Soil resistivity (ohm-cm)
Current study	0.049 (1.92)	0.0018 - 0.348 (0.07 - 13.72)	930 - 1 500
Jacobs and Hewes (1987)	0.41 (13.67)	n/a	560 - 4 300
Philadelphia Water Department (1985)	0.045 (1.5)	0.008 - 0.15 (0.28 - 4.85)	1 500 - 10 000
De Rosa and Parkinson (1986)	0.15 (5.0)	n/a	1 667 - 5 405
Russell (1988)	0.09 (3.06)	0.025 - 0.23 (0.86 - 7.7)	<2 000 , > 6 000
Romanoff (1964)	0.25 (9.86) ^a	n/a	17 800
Romanoff (1964)	0.27 (10.64) ^b	n/a	5 200
Romanoff (1964)	0.71 (27.8) ^c	n/a	62
Romanoff (1964)	1.27 (50.0) ^d	n/a	190
Romanoff (1964)	0.18 (7.0) ^e	n/a	943
Romanoff (1964)	0.13 (5.0) ^f	n/a	1 660
Romanoff (1964)	0.28 (11.14) ^g	n/a	84
Yarmamoto et al. (1983)	n/a	0.167 - 0.25 (5.6 - 8.3)	n/a

It should be noted that in the study conducted by Romanoff (1964), gray cast iron pipe samples were buried in various soil types and locations across the United States. In Table 15, seven samples from the study were listed and their soil types and locations are Cecil Clay Loam ^a (Atlanta, Ga.), Hagerstown Loam ^b (Loch Raven, Md.), Docas Clay ^c (Cholame, Calif.), Acadia Clay ^d (Spindletop, Tx.), Sharkey Clay ^e (New Orleans, La.), Carlisle Muck ^f (Kalamazoo, Mich.) and Tidal Marsh ^g (Charleston, S.C.).

The study conducted by Romanoff (1964) clearly shows that although soil resistivity is a parameter to indicate the likelihood of corrosion, it is not the sole indicator. Soil resistivity is a more reliable measurement for the potential magnitude of soil corrosion when supplemented with soil drainage or pH, when other factors that influence corrosion are not present (e.g. galvanic effects).

It is difficult to draw conclusions on the various values reported by other researchers without all of the pertinent data available (i.e. soil resistivity and pit depths). However, the average pit depth for gray cast iron pipe reported by Jakobs and Hewes (1987) is much greater than that found in the present study. The difference could be attributed to the range of soil resistivities used since they had values of pit depths for soil resistivities values as low as 560 ohm-cm, which corresponds to a severely corrosive environment (Heino and Taylor, 1987). Other factors influencing the corrosion of gray cast iron water pipe (galvanic corrosion, moisture and oxygen content, pH) may have enhanced the pitting rate of these pipes as compared with those of the present study.

4.5 Summary

Using the soil box method, the soil resistivity of the excavated gray cast iron water pipes' surrounding soil was determined. The range of soil resistivities was 650 - 1 600 ohm-cm. For the pipes used in the pit analysis study, the average soil resistivity was 1 202 ohm-cm with a range of 930 - 1 500 ohm-cm.

The average pitting rate was 0.048 mm/year (1.92 mil/year) with a range of 0.0018 - 0.348 mm/year (0.07 - 13.72 mil/year). All pipe pits were fitted to a 2 parameter Weibull distribution of varying scale and shape parameters.

A trend between soil resistivity and pit depth was observed. While a linear relationship is not able to characterize a central tendency for pit depth versus age, it appears to be able to explain, to some extent, the relationship between maximum pit depth and age. It would have been beneficial to have pit depth data for pipes surrounded by soil with resistivities less than 900 ohm-cm and greater than 1 500 ohm-cm.

CHAPTER 5

EXPERIMENTAL PROGRAM, RESULTS AND DISCUSSION

5.1 Introduction

From the thirty-three pipe lengths excavated, six were chosen for tensile testing. The six sections were chosen based on the soil resistivity readings taken from the geophysical surveys done on the street prior to the excavation (Baker et al., 1996). Varying degrees of resistivity were used corresponding to a wide range of corrosion levels. Some pipe sections still maintained their original pipe thickness. Others had locally a 100% reduction in wall thickness, e.g. a hole.

Coupons for tensile tests were selected in order that varying degrees of corrosion levels were encountered. The samples were selected visually on the basis of the degree of corrosion (i.e. wall thickness). For this study, there were thirty-seven tensile tests carried out and the results for the ultimate strength and the Young's modulus (where applicable) are shown in Table 16. A discussion of the tensile strength results from the present study and those from previous work, and defined values, concludes the chapter.

5.2 Experimental Apparatus

The tests were performed at NRC using a MTS model 801.22 hydraulic load frame and microconsole (Fig. 18 and 19) with a capacity, in tension, of 100 kN (22 000 lbs.). The hydraulic grips were fitted with specially manufactured wedges to accommodate the curvature of the 100 mm ID Gatineau (6 ") pipe. These wedges and grips are shown in Fig. 20. Fig. 21 shows a sample in place in the testing machine.

Fig. 19: MTS hydraulic testing apparatus

Fig. 20: Microconsole for MTS hydraulic testing apparatus

Fig. 21. Picture of wedges and grips

Fig. 22. Test coupon in load frame

The data acquisition system was driven by a custom program written by W. Lei of NRC on "TEST LINK" (software). The parameters collected for each test were load (lbs.), strain (%) and displacement (mm). The test was conducted at a constant displacement rate of 0.05 mm/min. An MTS, model number 632.11C-20, extensometer was attached to the sample in order to determine the percent strain experienced by the sample during the test. The MTS cards placed in the microconsole for each parameter were 100 kN (load), 10% (strain) and 50 mm (displacement).

5.3 Tensile Tests

The tensile specimens were prepared by staff in the machine shops of NRC's Institute for Research in Construction (IRC). The tensile coupons were cut according to ASTM standard E-8-94a. Fig. 22, illustrates a typical specimen and its dimensions. The thickness (T) is not given since this dimension can vary from sample to sample.

The location on the pipes for the specimens was chosen based on the degree of corrosion surrounding them (number of pits through the gauge length). In total, thirty-seven samples were tested to failure. Two samples that failed during the test setup were not included in the analyses.

Fig. 23 shows a typical (sample 1) stress vs. strain curve generated from the results of the tensile tests. The other stress strain curves are included in Appendix B.

The values for the engineering stress (ultimate strength), percentage of material remaining and Young's Modulus were determined for each sample, where applicable. The results are shown in Table 16.

The engineering stress was determined using the following equation (5):

$$\text{Engineering stress } (\sigma) = \frac{\text{Force}}{\text{Initial Area}} \quad (5)$$

Dimensions

$B = 75 \text{ mm}$

$G = 50 \text{ mm}$

$W = 12.5 \text{ mm}$

$C = 17.5 \text{ mm}$

$A = 57 \text{ mm}$

$R = 12.5 \text{ mm}$

STANDARD

ASTM E 8 - 94 a

Fig. 22. Illustration of the tensile test coupon

Fig. 23. Stress strain curve for sample 1

The percentage of material remaining was determined by dividing the measured effective area (reduced by corrosion) by the net or initial cross sectional area. Finally, Young's modulus was determined from the stress/strain curve at 0.2% strain using the secant method.

Table 16. Ultimate tensile strength, percentage material remaining and Young's modulus results

Sample Number	Ultimate Strength (MPa)	Percentage Material Remaining (mm²/mm²)	Young's Modulus (GPa)
1	201.8	100.0	166.7
2	194.3	100.0	122.7
3	208.6	100.0	116.7
4	207.2	100.0	110.0
5	119.0	78.8	-
6	182.7	100.0	107.5
7	202.8	100.0	113.9
8	185.1	100.0	125.3
9	197.8	100.0	105.8
10	179.3	100.0	112.0
11	219.3	100.0	115.4
12	198.5	100.0	112.3
13	207.0	100.0	114.9
14	170.8	100.0	119.4
15	133.4	80.9	-
16	218.9	100.0	115.4
17	230.8	100.0	125.6
18	205.6	97.4	-
19	212.6	100.0	116.7
20	159.5	100.0	103.9
21	131.4	90.6	-
22	136.4	90.5	-
23	132.0	87.6	-
24	173.6	95.5	-
25	176.7	94.1	-
26	104.1	61.0	-
27	139.4	66.6	-
28	179.4	82.8	-
29	93.4	54.1	-
30	177.7	86.2	-
31	119.6	100.0	92.3
32	174.2	100.0	113.6
33	120.1	65.1	-
34	191.5	93.8	-
35	161.2	88.9	-
36	132.8	67.5	-
37	71.6	58.0	-

Nineteen of the thirty-seven specimens did not have any metal loss from corrosion through their gauge lengths. The mean ultimate strength value for these samples was 193.2 MPa (28 ksi) and the mean Young's modulus was 116.3 GPa (16.9 ksi). Table 17 compares these results with other published values of ultimate strength and Young's modulus.

Table 17. Comparison of test results for ultimate tensile strength and Young's modulus

Source	Ultimate Tensile Strength (MPa)		Young's Modulus (GPa)	
	mean	range	mean	range
Test results (ASTM E 8-94a, CSA class 22)	193.2	120-230.8	116.31	92.3-166.7
Beer et al., 1985 (4.5% C, ASTM A-48)	170	n/a	70	n/a
Weast, 1983 (ASTM A48-48, class 25)	n/a	n/a	89.8	n/a
Yamamoto et al. (1983)	n/a	134-147.15	n/a	n/a
Ferguson and Nichols (1984)	n/a	180-300	n/a	n/a
Lackington and Large (1980)	80 *	n/a	n/a	n/a
Lackington and Large (1980)	170 **	n/a	n/a	n/a
Philadelphia Water Department (1985)	n/a	69 - 227.5	n/a	n/a

Note: * indicates strength for pipe where previous bursts were recorded and ** indicates strength for pipe where no previous bursts were recorded.

The mean ultimate strength of 193 MPa was greater than the values obtained from the other sources listed in Table 17, with the exception of Ferguson and Nichols (1984). This difference could be attributed to the class of gray cast iron. It is not known what class or grade of gray cast iron was tested.

5.3.1 Relationship Between Ultimate Strength and Remaining Material

One of the objectives of this thesis was to determine whether a relationship exists between the ultimate strength and the remaining material. Fig. 24 shows the results of the ultimate tensile strength and the percent of material remaining. As can be seen in this figure, there was a positive correlation between ultimate tensile strength and remaining material. A linear relationship (6) was established between the ultimate strength and the remaining material.

$$y = mx + b \quad (6)$$

Where:

y	=	ultimate strength (MPa)
x	=	remaining material (%)
m	=	slope
b	=	intercept (0)

The slope (m) of this line is equal to 1.896. The correlation coefficient (r^2) was 0.648 which is fairly high for this type of data and the confidence interval was 95 %. It was assumed that when no remaining material exists, there is no strength.

It is interesting to note the variability in the test results for a material without metal loss. At the 100% material remaining, the ultimate strength ranges from 120 - 221 MPa (17.4-33.5 ksi). This corresponds to a 48% difference in strength. These pipes were all from the same manufacturer and put in the ground at the same time. Gray cast iron is a heterogeneous material. Depending on the microstructure details (in particular the spacing between graphite flakes) the strength of the gray cast iron varies. This will be discussed further in the next chapter.

It was found that different pipes had differences in their initial wall thickness. For example, samples from pipe 4 had a wall thickness of approximately 10.88 mm while pipe 26b had an initial wall thickness of 9.28 mm. This represents a 14.7% difference in wall thickness. This parameter becomes important when determining the ultimate strength of a non-tested gray cast iron pipe based on a relationship similar to that determined in Fig. 24. If all other conditions such as material, age, diameter and

environmental condition remain unchanged, the strength of the pipe could be determined if the remaining wall thickness is known and if the percentage of remaining thickness is determined by dividing the current thickness by the original thickness. Only an estimate of the remaining material can be determined without a proper reference to the original thickness. This estimate can introduce an error into the determination of the remaining strength.

5.4 Summary

Thirty-seven tensile specimens were tested to failure in a constant displacement test. The loading rate for all specimens was 0.05 mm/minute. The load (lb.), strain (%) and displacement (mm) were collected for each test.

In total, nineteen were free of corrosion along their gauge lengths. The mean ultimate tensile strength was found to be 193 MPa and the Young's Modulus was 116 GPa for these samples.

The regression coefficient was 0.648 and the slope of the straight line is 1.896. It was assumed that a section with no material remaining would not have any strength.

Fig. 24. Linear relationship between the ultimate tensile strength and the remaining material

CHAPTER 6

GRAY CAST IRON METALLURGY DISCUSSION

6.1 Introduction

When the pipe was removed from Gatineau in 1994, the records showed that the material was identified as ductile cast iron. On further investigation, it was realized that the material was gray cast iron. The microstructure of the two samples viewed under a microscope confirmed the identification of gray cast iron.

A considerable amount of tensile stress is concentrated at the graphite components during events of either compression or tension. The graphite components are discontinuities which provide an opportunity for crack propagation to occur (Minkoff, 1983). The shape of the graphite components (spheroid or plate flakes) determines the relative amount of energy needed for crack propagation. Ductile cast iron contains graphite in the spheroidal form which allows this material to be more ductile than gray cast iron.

The strength of the gray cast iron pipe is influenced by the weight fraction of graphite flakes. A difference in volumetric fraction of graphite from 5% to 15% corresponds to a decrease in Young's modulus in tension from 140 GPa to 85 GPa (Minkoff, 1983).

This chapter will outline the procedure followed for mounting the gray cast iron pipe samples, present microscope photos of the samples and identify the three components of the gray cast iron pipe.

6.2 Sample Preparation

This section discusses the "standard mounting practice" that was followed for preparing the sample surface to be viewed under the microscope

Two samples of pipe were mounted and polished. The samples are mounted in a "puck" of thermosetting plastic, in this case, bakelite using the Simplimet[®] II sample setter. A series of sanding and polishing sessions are required. Four successively finer

grades of grit (240 to 600) are used. The sanding was conducted in one direction. When scratches on the surface of the sample did not change then the sample was sanded with the next finer level of grit. The sample finally was rinsed off with water to remove the grit on completion of the sanding.

The specimen was polished with a fine abrasive, in this case, 0.3 micron alpha alumina solution. The polish was applied to the wheel at the onset and again when the wheel was dry. When this phase was complete, the sample was washed well with warm water.

The structural details were revealed by etching the polished surface with a chemical solution. In this case, after removing the water with an alcohol rinse a 2% nital etch (methanol) was applied generously. It was then removed with water, taking care not to place fingers on the sample. The sample was again rinsed with alcohol and was dried with an air blower. This process of applying a chemical to the surface, removes the atoms from the grain boundaries, enabling viewing of the microstructure.

6.3 Microscope Scan

Two samples of gray cast iron were prepared using the previously mentioned procedure and viewed under a microscope. It was hoped that the components of gray cast iron would be readily visible. These components are ferrite, graphite and pearlite. The rate of cooling, temperature and amount of carbon dictate whether there will be an abundance of pearlite or ferrite.

Graphite is a low pressure form of carbon that is stable at different temperatures. It is soft, black and lustrous. Graphite flakes are the black particles in the microscope scans. Ferrite is iron containing less than about 0.02 w/o carbon. It will show up as the light gray or white areas in Figs. 25 and 26. Pearlite is a lamellar mixture of ferrite (almost pure iron) and cementite or iron carbide (Fe_3C) often occurring in gray cast iron (Parker, 1984). Pearlite is formed by the decomposition of austenite of eutectoid composition. It will appear gray on the microscope scan. These defined components are shown in Figs. 25 (X 100 magnification) and 26 (X 500 magnification).

Fig. 25. Gray cast iron microstructure (X 100)

Fig. 26. Gray cast iron microstructure (X 500)

The graphite and ferrite are easily found and distinguishable in Fig. 25. The graphite flakes are black and the ferrite is the gray background. The pearlite is harder to distinguish. There are clusters of pearlite on the left hand side of the figure and at the top right hand corner.

All three components of the gray cast iron are in view in Fig. 26. The ferrite is the light gray background and the graphite is the black section at the middle left and right of the picture. Pearlite is the gray area, that resembles layers or plates of material covering the remainder of the figure.

6.4 Summary

The composition of gray cast iron pipe is determined by the volume fraction of graphite present. The higher the percentage volume fraction of graphite the lower the Young's Modulus (Minkoff, 1983).

By observing the microstructure, the material has been identified as gray cast iron. Example microstructure scans were shown in Figs. 25 and 26. The graphite flakes are long and uniformly distributed across that section. By changing the position of the sample, an entirely different picture could appear. The material is variable but the scans shown in Figs. 25 and 26 are examples of the microstructure of this gray cast iron pipe. Variability in concentration of graphite and pearlite in a scan of gray cast iron material has been noted by the ASTM Subcommittee XV on Impact Testing (1933).

CHAPTER 7

SUMMARY and CONCLUSIONS

Gray cast iron pipe still forms a major portion of the water distribution networks across Canada and the United States. This material has not been used for new installations or replacement since the early 1970's. As a result, the gray cast iron pipe in the ground will continue to age and understanding the causes of gray cast iron deterioration will become more important as the frequency of these main failures increases.

The average pitting rate of the gray cast iron pipes from Carle St. in Gatineau (QC) was 0.048 mm/year (1.92 mil/year). A weak relationship was found between the soil resistivity and the pitting rate. This relationship could be due to the narrow range of resistivities that were used (935 - 1 500 ohm-cm). According to Heino and Taylor (1987), these resistivities are in the severely to very corrosive range, therefore some degree of pitting corrosion is expected.

The wall thickness was reduced up to 20% of the original material for a majority of the sampled pipes after 24 years in service. Structural failure of the pipe should not be prominent with this degree of metal loss, without external influences. The pitting of the pipes can be characterized using a 2 parameter Weibull distribution.

The tensile tests indicated that for a 150 mm (6 ") ID pipe after 24 years in service, in a soil with an average resistivity of 1 202 ohm-cm, the remaining material ranged from 54 - 100%. The ultimate tensile strength decreased with the reduction in the remaining material at a rate of 1.896 MPa/1%. The range for ultimate strength for 100% remaining material or no corrosion through the gauge length was 120 - 231 MPa.

There was a significant variation between the original wall thickness of the pipes tested. These pipes were all put in the ground at the same time and from the same manufacturer. This variation could be a consequence of the production method (spun cast or vertically cast). The original wall thickness is important in order to use a relationship similar to the one established in this study between ultimate strength and the remaining material. The percentage of remaining material is dependent on the

original thickness. For example, a pipe with a remaining thickness of 6.5 mm whose original thickness was 10.5 mm has a remaining material of 62%. Conversely, if the original material thickness were 9 mm, the pipe has 72% remaining material, a difference of 10%. Therefore, accurate records of wall thickness at the time of pipe placement are essential.

Microstructure analysis shows that there was a large amount of variability in the specimen depending on the location of the sample. Gray cast iron is heterogeneous in that the size and shape of the graphite, ferrite and pearlite components may not be uniform throughout the sample. Figs. 25 and 26 illustrate one example of the microstructure of this type of gray cast iron. This variability of the concentration of each component in the microstructure has been reported by others (ASTM Subcommittee XV, 1933).

This investigation was performed to test a framework that could be used for mechanical testing of metallic pipes in order to determine the residual tensile strength.

Relationships were established between pitting rate and the soil resistivity and the tensile strength and the remaining material. At the present time, this framework is being enhanced at the Urban Infrastructure Laboratory of the Institute of Research in Construction at the National Research Council.

CHAPTER 8

RECOMMENDATIONS

The present study provides a methodology and initial data for the determination of the tensile strength of gray cast iron water pipes. It could be enhanced with further testing of gray cast iron pipe of varying ages and over a larger range of soil resistivities. This study could determine whether a correlation exists between the soil resistivity, remaining material, age and the ultimate tensile strength. Since the gathering of samples usually depends on an existing break or is limited to a section up for renewal, a variety of soil resistivities and ages is not always possible to obtain.

Other parameters that could be collected include the soil's moisture content (trench), pH and redox potential. Correlations between tensile strength and these parameters could be established. To better understand internal corrosion, the collection of water quality data would be an asset. Since tuberculation or the presence of internal pits were not evident, the water quality data were not obtained for this study.

For this study, the graphitized material was removed from the exterior surface of all pipes but one, in order to choose samples of varying corrosion levels. Additional tests should be conducted with the graphitized material on the pipe to determine whether the graphitized material adds strength to the pipe.

Without accurate information on the original wall thickness and year of installation, the values determined present a wide scatter. The assessment of the residual strength is based on the original wall thickness and the time of installation. Most cities do not have the time nor the resources to properly record these valuable pieces of information. A management system developed for and used by the utilities is essential to ensure that these two pieces of information, along with pipe type, bedding and backfill material, are recorded.

PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

PRECISIONSM RESOLUTION TARGETS

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APPENDIX A

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Table A.1 Road chainage values for excavated Gatineau pipe

Pipe Number	Road Chainage (feet)	
	start	finish
1	0	15.17
2	15.17	31
3	31	50
4a	50	58.25
4b	58.25	68.58
5a	68.58	73.75
5b	73.75	87
6	87	91
7a	91	99.5
7b	99.5	107.92
8a	107.92	115.75
8b	115.75	126.33
9a	126.33	131.25
9b	131.25	140
10	158	140
11	158	178
12a	178	185.42
12b	185.42	194
13	194	212
14	212	230
15	230	249
16	249	267
17	267	286
18	286	303
19	303	323
20	323	339
21	339	356
22	356	374
23	374	393
24a	393	400.42
24b	400.42	410
25a	410	417
25b	417	424.25
25c	424.25	428
26a	434	438
26b	438	443
26c	443	451
27	451	470
28	470	486
29	486	505
30a	505	521
30b	521	524
31	524	540
32	540	561
33	561	570

Table A.2: Tensile Test Results

Sample Number	Net Cross-Sectional Area (mm ²)	Effective Cross-Sectional Area (mm ²)	Net Ultimate Strength (MPa)	Effective Ultimate Strength (MPa)
1	124.6	124.6	201.76	201.76
2	118.55	118.55	194.35	194.35
3	115.95	115.95	208.6	208.6
4	119.27	119.27	207.2	207.2
5	120.33	94.81	119.03	151.1
6	118.04	118.04	182.7	182.7
7	116.93	116.93	202.8	202.8
8	125.46	125.46	185.1	185.1
9	115.4	115.4	197.8	197.8
10	118.32	118.32	179.3	179.3
11	121.22	121.22	219.31	219.31
12	119.48	119.48	198.45	198.45
13	127.27	127.27	206.99	206.99
14	133.18	133.18	170.84	170.84
15	146.89	146.89	133.37	133.37
16	120.11	120.11	218.94	218.94
17	118.72	118.72	230.79	230.79
18	121.85	118.72	205.6	211.02
19	118.26	118.26	212.6	212.6
20	121.06	121.06	159.46	159.46
21	132.32	119.86	131.41	145.07
22	133.39	120.77	186.37	205.84
23	135.06	118.36	132.64	151.36
24	131.65	125.72	173.56	181.74
25	135.01	127.09	176.69	187.65
26	120.05	73.26	104.14	171.95
27	123.70	82.42	139.40	209.21
28	122.45	101.45	179.44	217.19
29	115.34	62.42	93.44	172.65
30	115.62	99.72	177.72	206.06
31	119.79	119.79	119.55	119.55
32	116.89	116.89	174.17	174.17
33	119.23	77.67	120.13	184.39
34	112.79	112.79	191.54	191.54
35	120.02	106.72	161.24	181.33
36	120.52	81.41	132.75	196.52
37	115.01	66.70	71.64	123.53

APPENDIX B

Fig. B-1. Pipe 4a Weibull Distribution Determination using Kolmogorov-Smirnov Test

Fig. B-2. Pipe 4b Weibull Distribution Determination using Kolmogorov-Smirnov Test

Fig. B-3. Pipe 6 Weibull Distribution Determination using Kolmogorov-Smirnov Test

Fig. B-4. Pipe 10 Weibull Distribution Determination using Kolmogorov-Smirnov Test

Fig. B-5. Pipe 26b Weibull Distribution Determination using Kolmogorov-Smirnov Test

Fig. B-6. Pipe 29 Weibull Distribution Determination using Kolmogorov-Smirnov Test

Fig. B-7. Pipe 32 Weibull Distribution Determination using Kolmogorov-Smirnov Test

Fig. B-8. Pipe 4a Volume Distribution Determination using Kolmogorov-Smirnov Test for the Weibull Distribution

Fig. B-9. Pipe 4b Volume Distribution Determination using Kolmogorov-Smirnov Test for the Weibull Distribution

**Fig. B-10. Pipe 6 Volume Distribution Determination
using Kolmogorov-Smirnov Test for the Weibull
Distribution**

**Fig. B-11. Pipe 10 Volume Distribution Determination
using Kolmogorov-Smirnov Test for the Weibull
Distribution**

**Fig. B-12. Pipe 26b Volume Distribution Determination
using Kolmogorov-Smirnov Test for the Weibull
Distribution**

**Fig. B-13. Pipe 29 Volume Distribution
Determination using Kolmogorov-Smirnov Test
for the Weibull Distribution**

**Fig. B-14. Pipe 32 Volume Distribution
Determination using Kolmogorov-Smirnov Test
for the Weibull Distribution**

Fig. B-15. Stress Strain Curve For Sample 2

Fig. B-16. Stress Strain Curve For Sample 3 **Fig. B-17. Stress Strain Curve For Sample 4** **Fig. B-18. Stress Strain Curve For Sample 5**

Fig. B-19. Stress Strain Curve For Sample 6 **Fig. B-20. Stress Strain Curve For Sample 7** **Fig. B-21. Stress Strain Curve For Sample 8**

Fig. B-22. Stress Strain Curve For Sample 9 **Fig. B-23. Stress Strain Curve For Sample 10** **Fig. B-24. Stress Strain Curve For Sample 11**

Fig. B-25. Stress Strain Curve For Sample 12 **Fig. B-26. Stress Strain Curve For Sample 13** **Fig. B-27. Stress Strain Curve For Sample 14**

Fig. B-28. Stress Strain Curve For Sample 15 **Fig. B-29. Stress Strain Curve For Sample 16** **Fig. B-30. Stress Strain Curve For Sample 17**

Fig. B-31. Stress Strain Curve For Sample 18 **Fig. B-32. Stress Strain Curve For Sample 19** **Fig. B-33. Stress Strain Curve For Sample 20**

Fig. B-34. Stress Strain Curve For Sample 21 **Fig. B-35. Stress Strain Curve For Sample 22** **Fig. B-36. Stress Strain Curve For Sample 23**

Fig. B-37. Stress Strain Curve For Sample 24 **Fig. B-38. Stress Strain Curve For Sample 25** **Fig. B-39 Stress Strain Curve For Sample 26**

Fig. B-40 Stress Strain Curve For Sample 27 **Fig. B-41. Stress Strain Curve For Sample 28** **Fig. B-42. Stress Strain Curve For Sample 29**

Fig. B-43. Stress Strain Curve For Sample 30 **Fig. B-44. Stress Strain Curve For Sample 31** **Fig. B-45. Stress Strain Curve For Sample 32**

Fig. B-46. Stress Strain Curve For Sample 33 **Fig. B-47. Stress Strain Curve For Sample 34** **Fig. B-48. Stress Strain Curve For Sample 35**

Fig. B-49. Stress Strain Curve For Sample 36 **Fig. B-50. Stress Strain Curve For Sample 37**

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