



Modelling concrete deterioration in sewers using theory and field observations



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ABSTRACT

Samples of new and 70 year old pre-corroded OPC concrete were exposed for up to 48 months in 6 sewers throughout Australia. Corrosion losses at each site followed the bi-linear trend originally proposed by Wells and Melchers [1]. During an initial phase (lasting <2 years) negligible loss of material occurs however once the surface pH = 6 losses commence and accumulate linearly at a rate that is likely to remain constant over time. Corrosion rates were found to be sensitive to humidity but insensitive to concrete alkalinity. A first pass model which predicts the rate of concrete sewer pipe corrosion from a knowledge of local average sewer gas temperature, humidity and H₂S concentrations was also developed. The equation predictions were in good agreement with rates determined from field observation and historical data.

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1. Introduction

Long-term deterioration is a major challenge for the continued reliable and economic performance of reinforced concrete sewer pipelines in wastewater infrastructure systems used in many major cities worldwide. In many cases the concrete sewer piping currently in use has been in place for many decades. For example Kaempfer and Berndt [2] estimated that 35% of the sewer network in Germany is over 60 years old. This raises serious issues for wastewater utilities of prioritizing remedial and replacement work and of minimizing disruption and other costs, including consequential costs. It follows that predicting the time to failure of existing reinforced concrete sewer systems is an important issue for wastewater utilities. However, reliable prediction requires an adequate understanding of the deterioration process and how this is influenced by operational and environmental conditions. Ideally such understanding is then translated into a quantitative model that builds on actual, quantified, observations of corrosion losses (such as obtained from visual and other inspections of pipes in service) together with quantified information on the most relevant environmental and other influences. Construction of such a model, using in-service data from a variety of sources, is the aim of the present work.

Much of the deterioration of concrete sewer pipes is the result of microbial induced corrosion (MIC) of the internal concrete surfaces under moist acidic conditions. The main mechanisms for such deterioration are well established and involve interactions between the sewage (or wastewater) stream in the lower part of the sewer and the gaseous phase above. A key factor is that biofilms form on the concrete in the

part of the sewer submerged in wastewater. The sulphate reducing bacteria (SRB) present in the biofilms convert the sulphates in the wastewater to H₂S. Diffusion and turbulence of the wastewater stream then facilitates the release of the H₂S into the sewer atmosphere. There it is absorbed onto the moist upper surfaces of the concrete pipe where both abiotic and biotic processes produce a number of sulfur species including sulfuric acid (H₂SO₄) [3]. The acid so produced reacts with the alkaline minerals within the concrete matrix to produce highly expansive minerals such as gypsum and ettringite. These are known to reduce the structural strength of the concrete and this, ultimately, may lead to structural failure of the pipe.

Based on the main features of the deterioration process outlined above, a previous study [1] showed that the long-term corrosion behaviour of concrete sewer pipes under aggressive conditions could be represented by a bi-linear function of exposure time. This was based on in-situ corrosion data obtained for concrete pipe samples exposed for 31 months in a sewer with very high H₂S concentrations at Perth, Australia. The purpose of the present paper is to examine, using other field data and observations, whether the bi-linear functional form is applicable more generally and in particular under less aggressive conditions, as well as to evaluate the effect of such conditions on the quantification of the bi-linear model. As described below, this includes the influence of sewer gas H₂S concentration, humidity and temperature. Developing such quantification will allow the bi-linear model to be compared with theoretical concepts for concrete sewer pipe deterioration that have been developed previously in the literature. It is acknowledged that in-situ field trials of the type used here involve an unavoidable level of inter- and intra-site variation in environmental conditions as well as sampling issues for corrosion coupons. However, provided this is recognized in the analysis and the model-building

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process, the results have a higher chance of representing reality than can be achieved purely with laboratory testing. Similar issues are well-recognized in comparable research activities [4].

In summary, the aims of the present study are:

- (1) Assess whether the corrosion trends observed in the previous study under highly corrosive conditions are also relevant under more benign exposure conditions,
- (2) Assess the degree to which the bi-linear model can be rationalized against known concrete sewer corrosion mechanisms and in particular examine the links between the onset of corrosion and surface pH, and
- (3) Calibrate the bi-linear model for concrete sewer corrosion against local average environmental conditions such as sewer gas H₂S concentration, humidity and temperature.

2. Background

Microbiologically induced corrosion (MIC) of reinforced concrete sewer pipe was first observed over a century ago by Olmstead and Hamlin [5] and linked to the activity of *Thiobacillus* bacteria in the mid-20th century [6–9] and later to other bacterial and fungal species [10–15]. Recently concrete losses in sewers have been observed, mainly anecdotally, to have increased considerably possibly as a result of the reduction in discharge to sewers of biologically toxic metals [16,17], increased discharge to sewers of high sulfur detergents [18] and the increased length of sewer lines (and hence sewage residence times) [19].

The evolution of concrete sewer pipe corrosion as a function of time is the result of the complex interaction between abiotic and biotic processes, the latter involving many different microbial communities [20]. Islander et al. [20] proposed that this evolution could be represented by a three stage model, dominated in the early stages by abiotic corrosion, followed by two overlapping stages of biotic corrosion governed by neutrophilic species once surface pH fell below 9 and then by more aggressive acidophilic bacteria once the surface pH has fallen below 4. Quantitative assessment of the Islander model has focused primarily on sewer pipes in advanced stages of corrosion (e.g. [12,21,22]) with only a small number of studies concentrating on the initial stages of sewer pipe corrosion [23,24] while very few studies, [1], have managed to assess initial and advanced stages of the corrosion process.

Because of the complexity of the processes involved, the reliable prediction of concrete sewer pipe deterioration has been considered difficult to achieve [25]. Many factors impact on the proliferation and activity of corrosion causing bacteria on the pipe surface and hence have a bearing on the rate at which losses occur. Factors include: pipe wall and sewer atmosphere temperature; pipe wall moisture content (via capillary condensation, contact with aerosols or direct contact with the wastewater stream [20]); surface pH [20]; the physical and chemical nature of the corrosion product layer formed; concrete porosity and the supply of nutrient to the bacteria (sulfur in a number of forms, nitrogen, carbon and mineral salts [26]). Sulfur supply is dictated by the chemistry of the wastewater stream and the rate of transfer between the wastewater and sewer atmosphere (a function of sewage pH, temperature and gas and liquid turbulence [27]) as well as the rate of absorption of H₂S back into the moisture film in the upper pipe [28].

The chemistry of the cementitious and aggregate material within the concrete is also believed to impact on the rate or corrosion. The use of high-alumina cements, calcareous aggregates and the inclusion of substances toxic to microorganisms have all been shown to have varying degrees of success in slowing down the rate of corrosion experienced by concrete sewer pipes (for example see [19,26,29,30]). Thistlethwayte [26] however reports that there is little evidence to show that increasing concrete strength or decreasing permeability improves corrosion resistance.

Ideally predictive models should consider all of the above parameters however the above list includes a number of parameters that may not be known, as well as a number of abiotic and biotic processes not all of which are, as yet, fully understood. The traditional approach used to overcome this issue has been to use semi-empirical methods, based primarily on the equations developed by Pomeroy [31]:

$$C = 11.5k\phi_{sw}A^{-1} \quad (1)$$

where C is the average rate of corrosion of concrete by acid (mm year⁻¹), ϕ_{sw} is the flux of H₂S to the pipe wall (gS m⁻² hr⁻¹) and A is the alkalinity of the sewer pipe wall expressed as CaCO₃ equivalent. The factor k accounts for the fraction of acid produced that contacts the sound concrete and takes part in the corrosion process (acknowledging that a fraction of the acid produced is washed back into the wastewater stream). When acid formation is slow k may approach unity but it may be as low as 0.3 to 0.4 in conditions where acid production is rapid. The value assigned to k therefore has a large influence on the corrosion rate predicted by Eq. (1). Despite its importance, there appear to have been no studies to assess the magnitude of k , [3].

Attempts to build less empirical models include that of Böhm et al., [32] who used a moving boundary idealization to model the sewer pipe corrosion process and which incorporated the porous structure of the pipe wall. A large number of parameters were required in the formulation and the authors concluded that this rendered the model unlikely to produce reliable predictions for real systems. More recently, Jensen [28], when combining models for H₂S removal rates and oxidation kinetics with an existing model of dissolved oxygen and organic matter transformations in sewers (WATS model, [33]), noted that the concentration of H₂S oxidising biomass on the concrete surface is a key parameter in predicting the rate of corrosion.

The present study follows a semi-empirical approach, by-passing much of the complex and poorly understood microbiological detail. It concentrates, as in the earlier work [1], on the effect of the local sewer environment on the rate at which corrosion losses occur. The next section describes an experimental program conducted over several years to attempt to overcome the acute scarcity of reliable data for sewer corrosion and the relevant environmental parameters. This is followed by the further investigation of the bi-linear model and the effect on it of three key parameters – sewer gas temperature, humidity and H₂S concentration. It includes consideration of the physical and chemical processes involved, the concrete chemistry and how these are reflected in the bi-linear relationship. In addition, historical literature data is considered to test the validity of the model.

3. Experimental design

The methodology employed for the experimental work in this study is similar to that previously reported [1]. A summary is as follows.

3.1. Materials

Two different OPC concretes cut to 100 mm nominal cubes were exposed in each sewer. One set of samples, ('new' coupons), were cut from newly manufactured 1.2 m ID spun cast standard reinforced concrete sewer pipe. A second set, ('old' coupons), were cut from reinforced concrete slabs that had served for 70 years as covers for a typical operating sewer carrying domestic, industrial and trade waste. Old coupons retained a ~2 mm covering of crystalline corrosion product. The use of the two concretes permitted direct observation of corrosion of realistic concretes at different time intervals – at the commencement of the life of the sewer pipe and at a more advanced stage when corrosion is well established. The aggregates for the new coupons primarily consisted of sub-rounded 10–15 mm dacitic volcanics and chert with lesser amounts of quartzite, quartz and rhyolitic volcanics. Old coupon

aggregate was generally larger (15–30 mm) and was dominated by rounded andesitic, dacitic and rhyolitic volcanic gravel and cherts with lesser amounts of sandstone, quartzite and quartz. The aggregate was tested and found to be inert under acid attack.

XRF (Spectro X'Lab 2000) and XRD (Philips X'Pert MPD diffractometer operating at 40 kV, 40 mA) analysis of the new and old concrete coupon cement binder chiselled from the centres of the new and old concrete coupons revealed a similar mineralogical makeup. However, instead of the portlandite ($\text{Ca}(\text{OH})_2$) component present in the new concrete coupon cement binder, the old concrete coupons showed the presence of calcite and vaterite (both forms of calcium carbonate), probably indicating that a large portion of the original alkaline mineralogy had been carbonated during the 70 year previous exposure period. For the old concrete coupons, XRD analysis and microscopic inspection of the pre-existing corrosion product present on their surfaces showed it consisted mainly of gypsum and quartz. The alkalinity of the new and old coupon concrete was determined according to the method of Snell and Ettre [34] to be 0.215 and 0.154 g CaCO_3 (g concrete)⁻¹ respectively. The permeable pore space fraction of the new and old coupon concretes was determined via the boiling water saturation technique [35] to be 6.0% and 11.4% respectively. Density of concretes was determined at 2430 kg m⁻³ and 2260 kg m⁻³ for the new and old concretes respectively.

3.2. Exposure sites

In situ corrosion behaviour was examined in gravity sewers located in three cities across Australia (Fig. 1a). In Perth, Western Australia the two exposure sites, (Perth A, Perth B), were located within a sealed sewer system. At these locations the H_2S gas concentrations were higher than normal for unsealed (conventional) sewer systems. The two trials in Sydney and the two in Melbourne were in naturally ventilated sewer pipes. Some corrosion observations at Perth A have been reported earlier [1].

At each exposure site extensive environmental monitoring was undertaken to determine average conditions as well as to establish the range of diurnal and seasonal variations in sewer gas temperature, humidity and H_2S concentration. Sewer gas temperatures and H_2S gas phase concentrations were logged every 5 minutes over a 2 week period every 3 months throughout the study period. Sewer humidity was recorded at the same frequency at the two Perth sites. At the Melbourne and Sydney sites humidity measurement proved problematic as a result

of repeated sensor failure. For this reason the values reported for humidity at the two Melbourne sites are based on a limited pool of data (2 days observations) while humidity observations at the Sydney sites are restricted to four records each 2 weeks long for the entire study period.

3.3. Mounting of the concrete coupons

Individually numbered coupon pairs comprising one old and one new concrete sample were embedded in epoxy resin in stainless steel containers (Fig. 1b inset). This configuration allowed for the direct comparison of the corrosion behaviour of the two concrete types. Each pair of samples were mounted to ensure that the inner pipe surface of the new coupons and the previously corroded surface of the old coupons protruded approximately 10–20 mm above the resin surface.

At each of the Sydney and Melbourne sites the stainless steel sample containers were bolted in an inverted position along the crown of the sewer (for example see Fig. 1b). As the sewers in Perth were lined, samples at these sites could not be affixed to the pipe itself but instead were mounted in an inverted position in stainless steel racks fitted in man-holes at each site. The racks were positioned so that the samples were at sewer crown height. In all locations the samples were well above the normal 'tidal range' of the wastewater stream and were only immersed at times of high stormwater assisted flow.

3.4. Coupon analysis

3.4.1. Loss of material and corrosion product layer depth

The magnitude of corrosion loss and the depth of corrosion product on each sample were determined using photogrammetric analysis of the exposed coupon surface. The details of the process have been described earlier [1] and are summarised here.

Prior to installation triplicate sets of 5 images were taken of each coupon surface and a surrounding fixed targeting template (Fig. 2). A photogrammetric imaging software package (Photomodeler Scanner®) was then used to create 3D representations of the exposed surface and an average height of the surface relative to the plane passing through the target template was then calculated (Z_0). The imaging process was repeated immediately upon recovery of the coupons (with the corrosion product layer intact) and again after the corrosion product had been stripped away. At each condition the average height of the exposed surface relative to the target template plane was calculated. The

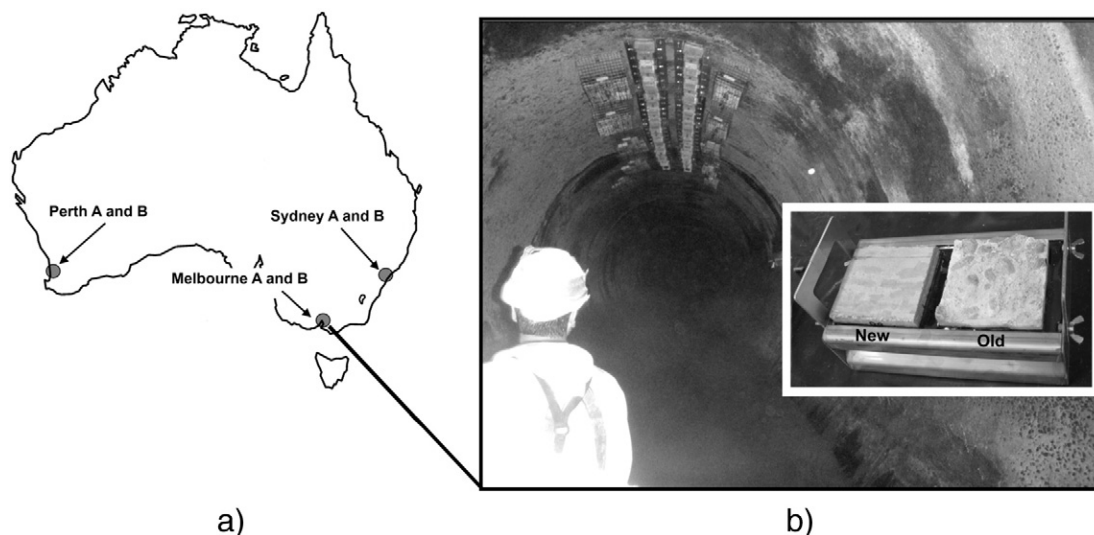


Fig. 1. Locations of sewer field sites employed in this study (a). Sample pairs mounted in inverted position along the crown of the sewer at Melbourne B site (b). Single new and old coupon pair in stainless steel container (inset (b)).

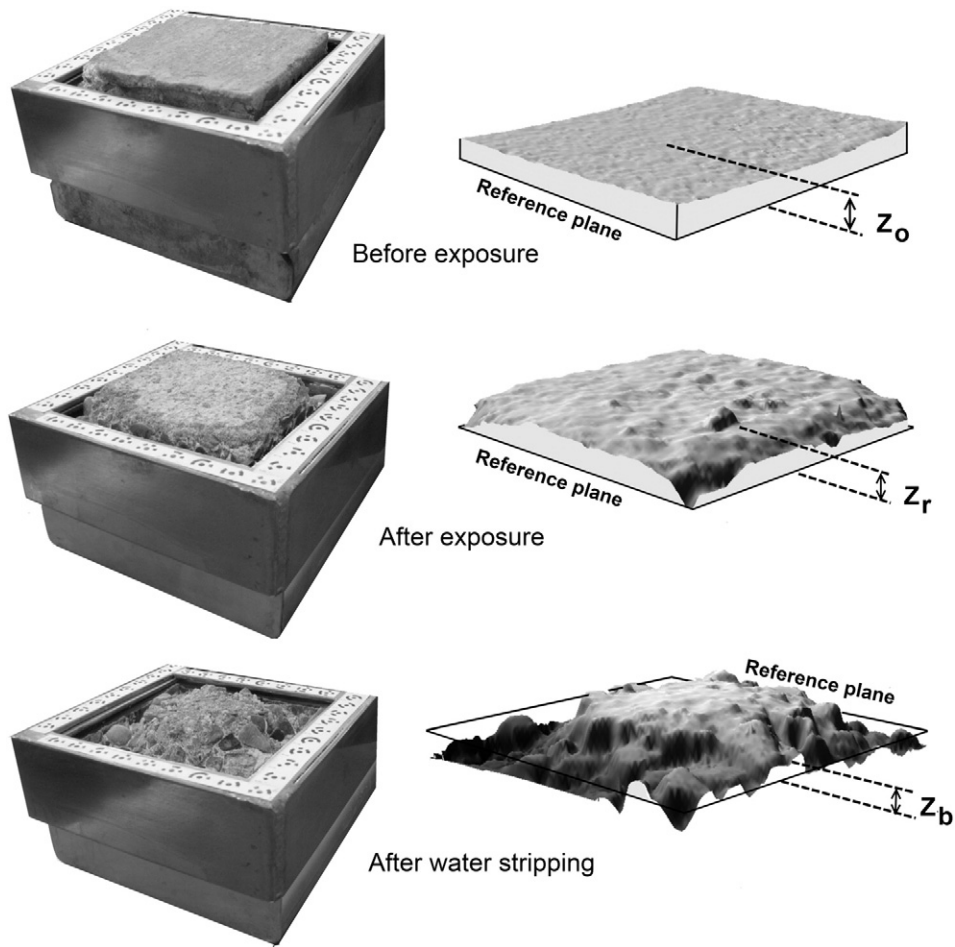


Fig. 2. Photogrammetric method for determining corrosion loss and corrosion layer depth. Depth of corrosion loss: $\Delta Z_c = Z_0 - Z_b$ and depth of corrosion product layer: $\Delta Z_l = Z_r - Z_0$.

depth of corrosion losses and corrosion product formed were determined as follows:

$$\Delta Z_c = Z_0 - Z_b \quad (2)$$

$$\Delta Z_l = Z_r - Z_0 \quad (3)$$

where ΔZ_c is the average depth of the sound concrete lost to corrosion (m); ΔZ_l is the average depth of the corrosion product layer (m); Z_0 is the average initial surface height prior to exposure (m); Z_b is the average surface height after high pressure water blasting of the sample; and Z_r is the average surface height of the recovered coupon (m). Calibration of the procedure has shown that the photogrammetric process can reliably detect changes in surface movement as small as 0.1 mm [1].

3.4.2. Surface pH and appearance

Coupons were removed from the sewers with the corrosion product layer intact. After recovery the surface of the corrosion product was inspected under a low powered microscope for changes in surface morphology. Thereafter the average surface pH of the corrosion product layer for each recovered coupon was determined using a flat-faced pH probe (Extech pH100). A series of de-ionised, distilled water droplets were first placed on the surface and allowed to equilibrate before the pH of the droplets were measured. Typically 9 individual pH readings were made for each coupon although porous and/or uneven surfaces presented difficulties and the number of readings sometimes was as low as 4. Surface pH measurements of new and old concrete control samples kept in a dry, temperature controlled laboratory were made

to assess the extent of background changes in surface pH brought about, for example, by atmospheric carbonation of the samples.

4. Experimental results

Coupons were installed in sewers between September 2009 and June 2010. Melbourne A and B coupons were recovered after approximately 7, 13, 18, 24, 32 and 39 months of exposure; Perth A and B coupons after approximately 6, 14, 19, 26, 31, 37 and 48 months of exposure; and Sydney A and B coupons after 9, 12, 24, and 36 months of exposure. The trial at the Sydney sites was abandoned after 36 months as a result of inadvertent contamination during sewer maintenance operations. All results reported here are the average values for each set of 3 coupons retrieved at each recovery.

4.1. Environmental conditions

The average values for the gas phase H_2S concentration, the relative humidity (H) and the gas phase temperature for each field site over the complete study period are listed in Table 1. As expected, H_2S gas concentrations recorded at the Perth A and Perth B sites (average 125 and 423 ppm respectively) were many times higher than those observed at the Melbourne A and B sites (2 and 6 ppm respectively) and the two Sydney sites (2 ppm). The two Perth sites (26–27 °C) were on average considerably warmer (5–6 °C) than the remaining sites. Perth B and Sydney A and B sites also were on average drier ($H \leq 0.94$) than the remaining sites where the sewer atmosphere was close to saturated.

Diurnal variation in sewer gas temperatures was typically less than ± 1 °C with larger variations observed at sites (such as Sydney)

Table 1

Details of the daily average environmental conditions within the sewers over the study period. (* Note humidity data for the two Melbourne sites are based on a limited pool of data (Melbourne A - 3 days and Melbourne B - 2 days)).

	Perth		Melbourne		Sydney	
	A	B	A	B	A	B
$H_2S_{(gas)}$ (ppm)						
Average	125	423	1.5	6.0	2.7	2.2
Minimum	26	107	0.0	0.0	0.0	0.0
Maximum	222	828	4.3	17.6	19.4	59.9
St. Dev.	113	163	1.1	3.2	2.4	2.9
T_{gas} (°C)						
Average	25.8	27.0	19.7	20.7	21.6	21.3
Minimum	22.2	14.0	16.5	17.7	15	15.3
Maximum	28.8	36.6	22.9	24.5	29.7	28.8
St. Dev.	1.8	5.1	1.6	2.0	3.4	3.0
Relative humidity (-)						
Average	0.97	0.91	1.00*	1.00*	0.94	0.94
Minimum	0.93	0.54	1.00*	1.00*	0.83	0.83
Maximum	1.00	1.00	1.00*	1.00*	1.00	1.00
St. Dev.	0.04	0.05			0.03	0.03

where the sewer was more exposed and internal conditions were therefore influenced to a greater degree by external atmospheric temperature fluctuations. Over longer time periods seasonal average sewer gas temperatures (Fig. 3) generally fell within a 6–8 °C band at all locations with the exception of Perth B where a higher seasonal variation (up to 14 °C) was observed. H_2S gas concentration, however, was much more changeable with concentrations varying by as much as 100% from the daily mean value over the diurnal cycle. Considerable variations in H_2S concentrations were also observed from season to season (Figs. 4)

with levels generally highest during the summer/autumn months and lowest during the winter months. Seasonal average humidity (Fig. 5) declined slightly at the Perth A site over the study period. However, there was a noticeable increase in humidity at the Perth B site towards the end of the field trials.

4.2. Surface morphology and mineralogy

At any one time the appearance and chemistry of the coupon surface varied considerably from site to site. Nevertheless, the general pattern of surface deterioration was similar at all sites for comparable exposure periods, noting that the deterioration process occurred at slower rates at the less aggressive sites. On new coupons the original smooth, light grey surface present prior to exposure became a fine layer of white crystalline material. With further exposure this became a more amorphous layer of soft, pasty corrosion product with numerous fissures and perforations. On the old coupons the pre-existing crystalline layer material that initially encrusted the surface was converted to a soft, fissured, pasty white coating similar in texture and chemistry to that present on the corroded surfaces of the new coupons. By the end of the study, the surfaces of the new and old coupons at Perth A, Perth B and Melbourne B sites were almost indistinguishable in appearance. Only at the more benign Melbourne A and Sydney sites was there a noticeable difference in appearance at the end of the study between the new and old coupons. XRD/XRF analysis of the corrosion product indicated that the corroded material comprised mainly gypsum, ($CaSO_4 \cdot 2H_2O$) with entrained quartz (probably a remnant of the sand particles present in the original mortar). Ettringite, a mineral often associated with sewer pipe corrosion, was not found.

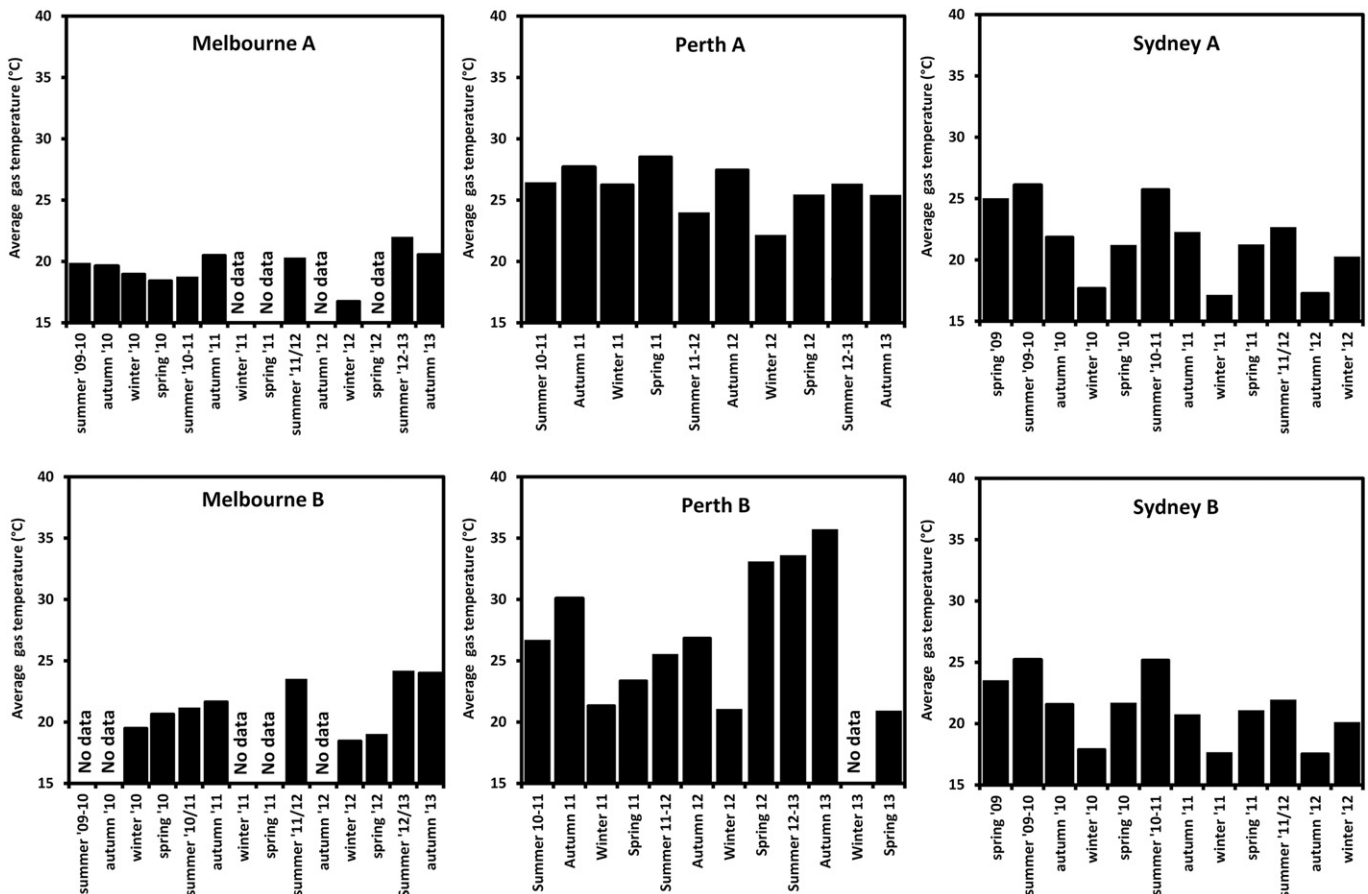


Fig. 3. Seasonal trends in sewer gas temperature.

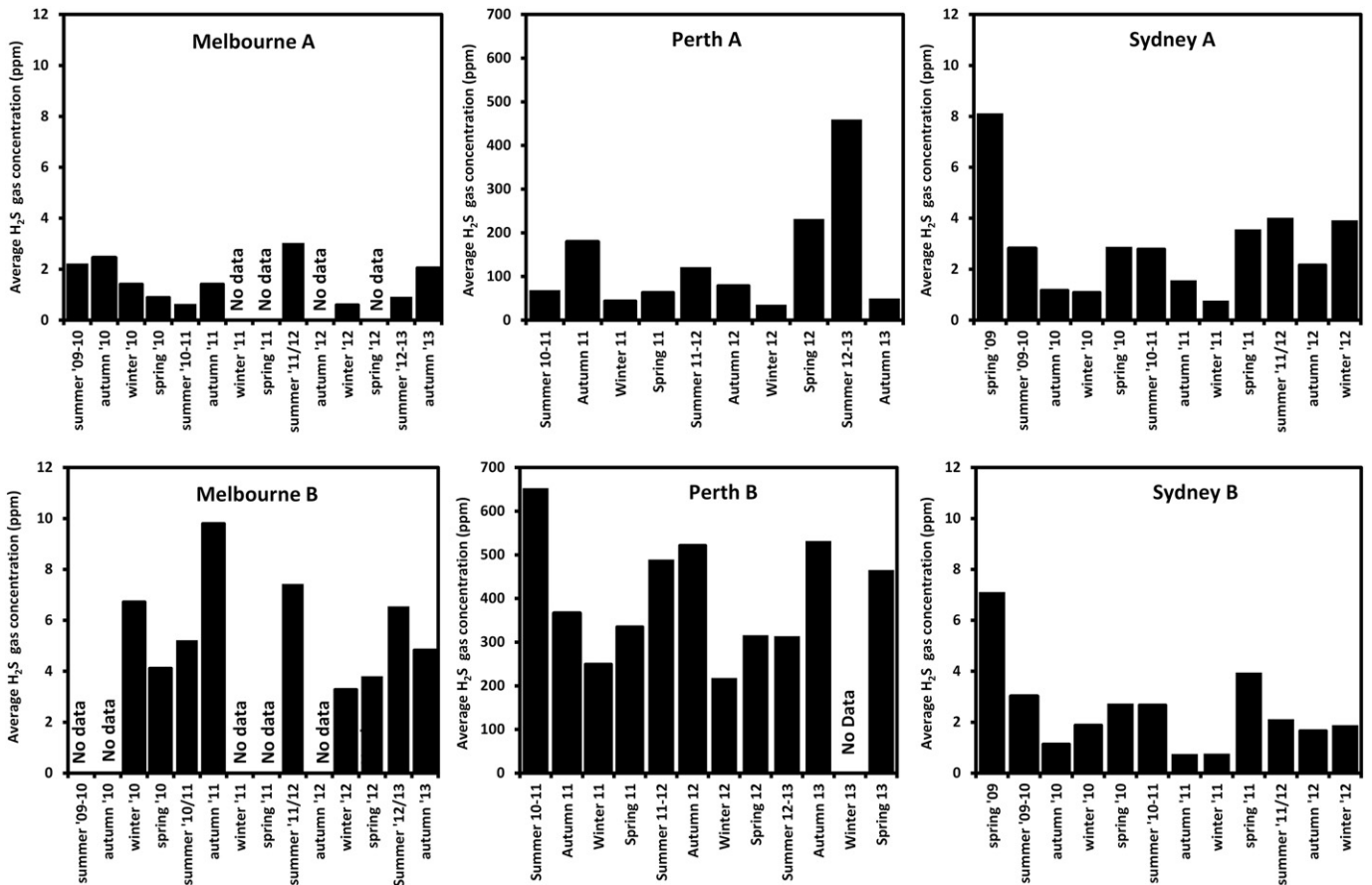


Fig. 4. Seasonal trends in H_2S gas concentrations.

4.3. Changes in surface pH

Both abiotic and the biotic processes in sewers lower the surface pH of the sewer pipe with increased exposure time [9]. The surface pH dictates the makeup of the bacterial and fungal communities colonising the pipe surface and ultimately this affects the dynamics of the corrosion process [12,20]. It follows that surface pH and the changes it undergoes can be considered as an indicator of the changing corrosivity of the local sewer environment and of the changing activity of the bacterial and fungal species present on sewer walls. Surface pH values for the new and old concrete coupons and for the control samples are shown in Fig. 6.

For the new concrete coupons the average surface pH of 10.1 determined at the commencement of the trials was considerably below the range $pH = 11\text{--}13$ typical for fresh concrete [36]. Once these coupons were exposed to the sewer environment the surface pH declined, but not at a uniform rate (Fig. 6a). For the first 18 months the rate of decline in surface pH was 0.3–0.34 pH units/month at all sites except Melbourne A where the decline was slower (~ 0.2 pH units/month). After 18 months the rate of decline in surface pH was 0.07–0.10 pH units/month at all sites. As a result, by the end of the study the surface pH of new coupons at all sites except Melbourne A and the two Sydney sites (where the trial ended prematurely) had fallen to $pH = 2.6\text{--}3.6$. After 39 months exposure the average surface pH of Melbourne A new coupons was rather higher ($pH = 4.9$). Over the same period the new concrete control samples showed little variation in surface pH (average 10.0 ± 0.2). Before the premature termination of the Sydney trials the surface pH appeared to have stabilised at $pH = 6$.

For the old concrete coupons the pattern of surface pH decline with increased exposure time differed considerably from that observed for new coupons (Fig. 6b). Old concrete coupons had an initial average

surface pH of 8.2. This value is consistent with their exposure history, the presence of an existing layer of corrosion product and the carbonation of the portlandite content of the coupon. The surface pH of the old concretes declined rapidly after the coupons were installed in the sewers. After 6–7 months exposure (first recovery) the average surface pH was $pH = 3.5\text{--}4$ at all sites except Melbourne A, where it had declined to $pH = 6.1$. After 12 to 14 months exposure (second recovery) the surface pH values were $pH = 3.7\text{--}4.5$ at all sites. The subsequent surface pH readings (i.e. for the remainder of the study period) were in the range $pH = 2.8\text{--}4$ at all sites except Perth A where they were slightly lower ($pH = 2.2\text{--}3.6$). Over the same period the surface pH of the control samples for the old concrete showed little variation (7.9 ± 0.3).

4.4. Corrosion losses and corrosion layer development

4.4.1. New coupons

The loss of concrete from new coupons installed at the Melbourne and Perth sites is shown in Fig. 7. The new concrete coupons showed little or no corrosion loss during the initial period of exposure. This initiation period lasted between 9 and 25 months. At the time the two Sydney trials were terminated negligible corrosion or corrosion product was found for the new concretes. It can therefore be assumed that for these two sites the initiation period was >36 months.

The new concrete coupons at the Melbourne and Perth sites started to corrode after the initiation period. In all cases corrosion losses then increased in a linear manner with time, except for Perth B. Here a slight increase in the rate of corrosion is apparent toward the end of the study period. Concrete losses for the new concrete coupons at the end of each exposure period ranged from 36.4 mm at Perth A (48 months), 16.3 mm

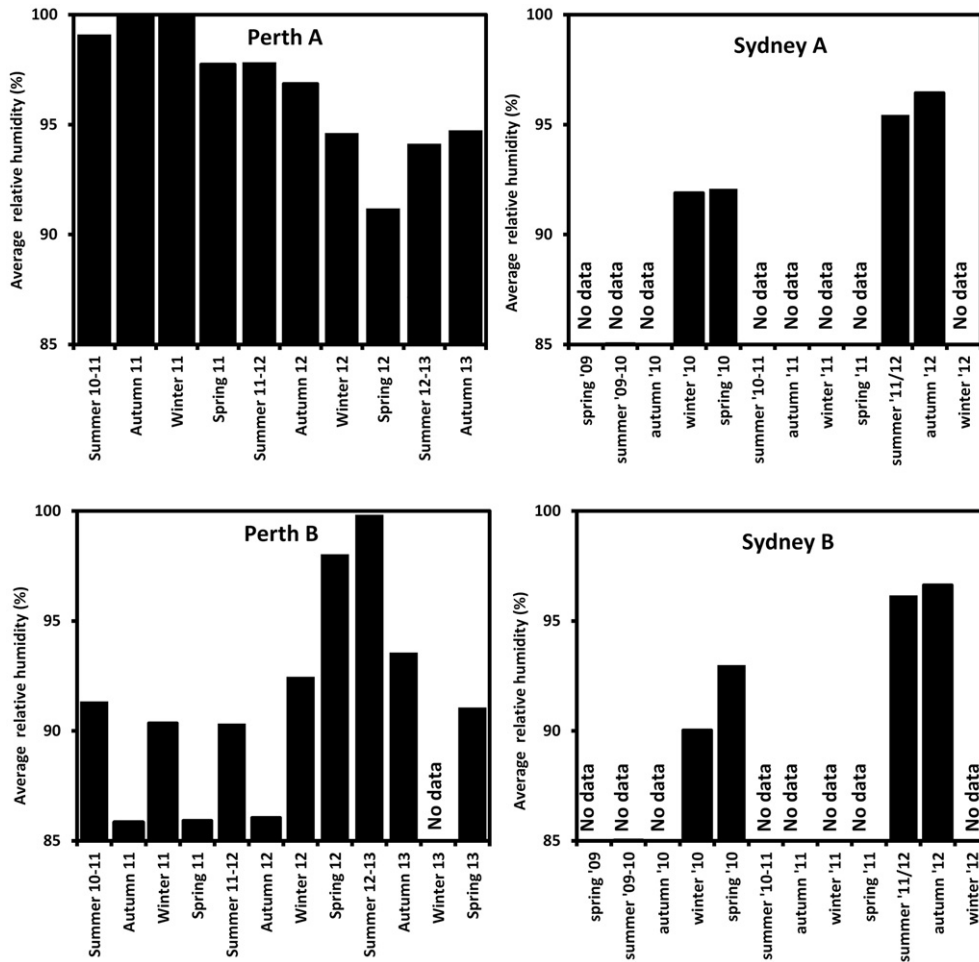


Fig. 5. Seasonal trends in gas phase relative humidity (note limited data from Melbourne sites not shown).

(Perth B, 48 months), 11.6 mm (Melbourne B, 39 months), to 0.8 mm at the Melbourne A site (39 months).

During the initiation period, the build-up of corrosion product on the surfaces of the new coupons was almost negligible (<0.5 mm). Some of this product was recovered however, and XRD analysis showed that it consisted of a layer of fine gypsum crystals. The thickness of the corrosion product that formed after the initiation period increased in a non-linear manner with time (Fig. 7). The corrosion product had a white pasty appearance with numerous fissures and perforations. The

thickness of the corrosion product at the end of the study period ranged from 9 mm on Perth A new coupons down to 1.4 mm depth on Melbourne A coupons.

4.4.2. Old concrete coupons

At all sites the coupons made from old concrete commenced corroding almost immediately on exposure in the sewers. Analysis of the corrosion loss data showed that corrosion was a linear function of time throughout the study period (Fig. 8). In the last few months at Perth B

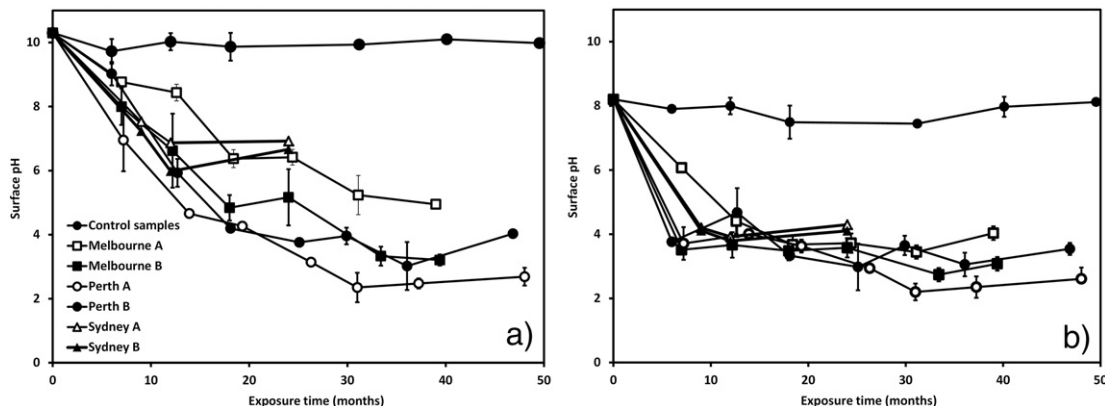


Fig. 6. Trends observed in average surface pH at each field site for new coupons (a) and old coupons (b).

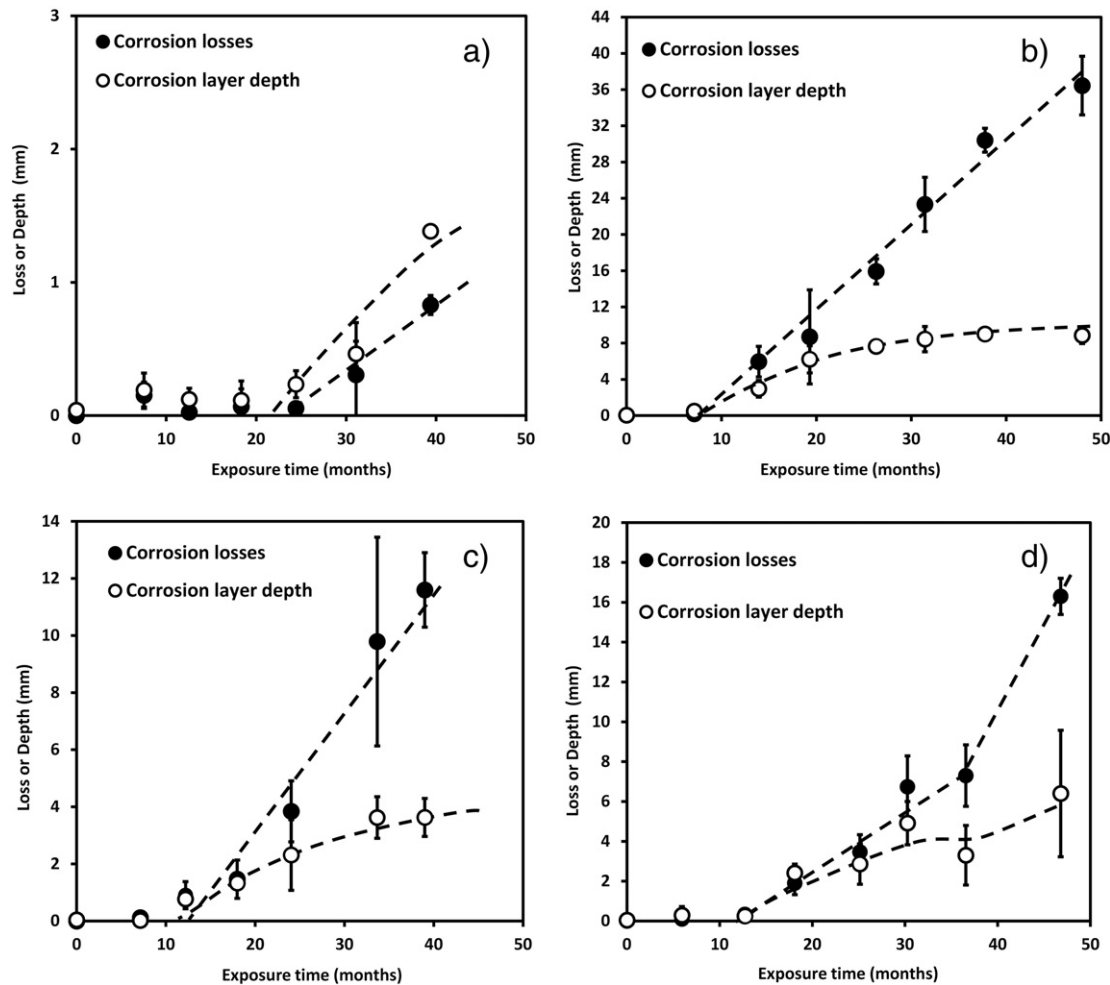


Fig. 7. New coupon corrosion losses and depth of corrosion product: (a) Melbourne A, (b) Perth A, (c) Melbourne B and (d) Perth B. Each data point is the average of observations for 3 coupons and error bars are the standard deviation of the three measurements.

a small increase in the rate of loss similar to that determined for the new coupons was found for the old concrete coupons. Analysis of the corrosion loss data showed the old concrete coupons to have greater variability between samples compared to that found for the new concretes. This could be the result of the effect the removal of individual (larger) pieces of aggregate from the old concrete coupons had on calculated losses. The average corrosion losses for the old concrete coupons ranged, at the end of the study, from 38.1 mm at Perth A (48 months), 23.9 mm (Perth B, 48 months), 19.0 mm (Melbourne B, 39 months), and as low as 4.8 mm, 2.7 and 2.0 mm at Melbourne A (39 months), Sydney A and B (36 months) respectively.

The pre-existing 2 mm thick layer of hard crystalline corrosion product initially present on all old coupons was soon transformed to a soft, pasty consistency upon exposure to the sewer environment. This was followed, at all sites except Melbourne A and Sydney A and B, by a steady but non-linear increase in the thickness of the corrosion product (for example see Fig. 8). At the Melbourne A and two Sydney sites no substantial increases in corrosion layer depth were over the study period. The average corrosion layer depths ranged from 14.7 mm at Perth A to <1 mm at Sydney A.

5. Discussion

5.1. Validity of bi-linear model

On the basis of data obtained from a very aggressive sewer corrosion site a bi-linear corrosion model was previously proposed [1]. Two other

previously reported observations support the linearity of corrosion after initiation. Firstly, linear behaviour was also observed in short term exposure tests of model concrete sewers in controlled laboratory and pilot scale experiments [3,37,38]. Secondly, there are data from field experiments conducted on actual concrete sewers near Melbourne, Australia [39] that show linear behaviour for exposures up to 18 years, under moderately aggressive conditions (H_2S concentrations ranging from 5 to 40 ppm, Fig. 9). This appears to be the first study to show that linear corrosion behaviour is experienced under real sewer conditions over a wide range of conditions (temperature, humidity and H_2S concentrations) and is likely to be experienced over the entire lifetime of the pipe.

The bi-linear model is also supported by the results of this study. First it is noted that the additional data collected at the Perth A site in the present test program (see Figs. 7 and 8) are consistent with the earlier findings [1]. In both cases the corrosion losses accumulate linearly with time. Linear regression shows a best fit linear function with $r^2 = 0.98$ for both new and old coupon losses as a function of exposure time.

The data collected at the remaining sites may be used to provide an independent check of the applicability and validity of the bi-linear corrosion model under a range of sewer environmental conditions. The relevant data for this analysis are summarized in Table 2. Here attention is confined to the rate of corrosion once initiation has occurred. Linear regression of the data for new concretes at the Perth and Melbourne sites showed regression coefficients $r^2 > 0.98$, except for Perth B where $r^2 = 0.92$. For the old concretes linear regression showed regression coefficients $r^2 = 0.98$ and 0.96 for Perth A and Melbourne B respectively

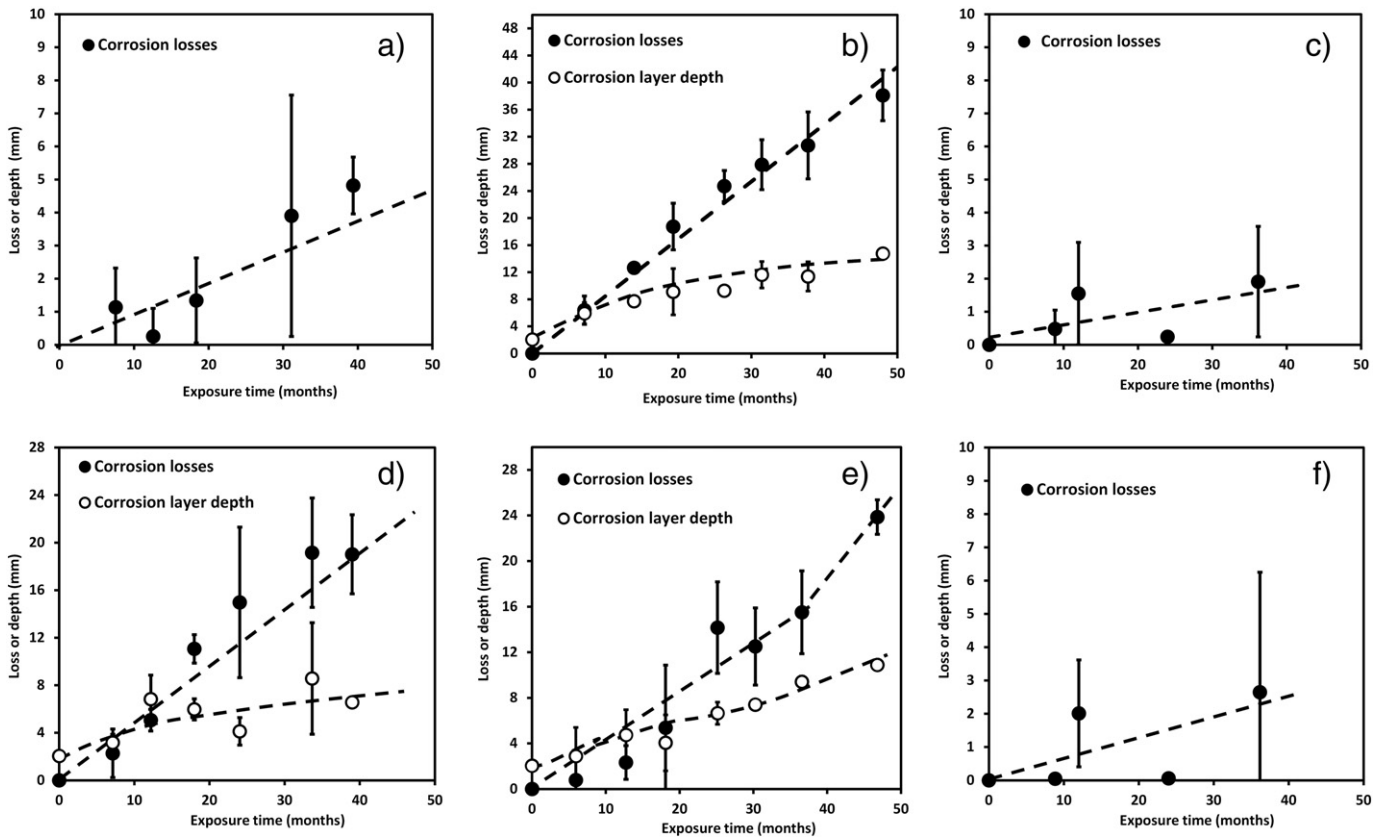


Fig. 8. Old coupon corrosion losses and depth of corrosion product: (a) Melbourne A, (b) Perth A, (c) Sydney A, (d) Melbourne B, (e) Perth B and (f) Sydney B. Corrosion product depth for Melbourne A, and the two Sydney sites not shown. Each data point is the average of observations for 3 coupons and error bars are the standard deviation of the three measurements.

and 0.92 for Perth B, indicating that a linear trend is reasonable. For the Sydney sites linearity is plausible but less clearly defined due to the low levels of corrosion, high sample to sample variability and the premature truncation of the test program.

The data obtained in this study along with the data reported by CSR Humes [39] indicates that sewer corrosion rates are likely to remain closely constant with time (Fig. 9). It is reasonable, therefore, to assume that linearity remains valid for the expected life of most sewer pipes, provided the exposure conditions remain the same.

Comparison of coupon surface pH and new coupon corrosion losses (Fig. 10) shows that the onset of corrosion losses coincides with the

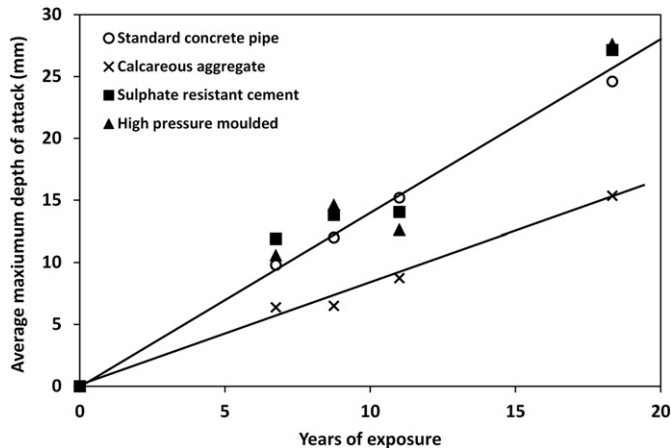


Fig. 9. Corrosion losses experienced by a variety of concrete sewer pipes near Melbourne, Australia between 1975 and 1993. H_2S conditions are reported to range between 5 and 40 ppm. (Modified from CSR Humes [39]).

point at which the surface pH is reduced to 6, irrespective of the aggressiveness of the site. The time taken to reach this surface pH and therefore the time to the onset of corrosion losses will depend on the local interior conditions in the sewer. These findings concur with the view put forward by Thistlethwayte [26] who states that concrete surface pH must first be reduced to $pH = 6$ before *Acidithiobacillus thiooxidans* will proliferate fast enough to account for corrosion observed in sewers. Morton [38] also reports that corrosion products first appeared on concrete samples after 8 weeks of exposure to the atmosphere inside sewage sedimentation tanks, at which time the surface pH had decreased to between $pH = 6$ and 6.5. Data presented by Okabe et al. [24], however, shows that in a moderately aggressive sewer environment (H_2S average concentration = 30 ppm) mass losses from concrete coupons did not commence until the coupon pH had fallen to around 2.

In summary, the above observations point to a pattern of corrosion behaviour that is consistent with the previously proposed bi-linear model:

- there is a period of time immediately after first exposure of new concrete when little or no corrosion loss occurs,
- when the surface pH falls to $pH = 6$, this is followed by a well-defined transition to a state in which corrosion losses are essentially linear with time,
- the results obtained in this study as well as field trials over 18 years, indicates the linear part of the model can be expected to extend over the full service life of concrete sewer pipes.

5.2. Impact of concrete mineralogy

The impact of cement chemistry/mineralogy and aggregate type on sewer pipe corrosion processes is well documented (for example see

Table 2

New and old coupon steady state corrosion rates. Corrosion rates are determined from a line of best fit through the corrosion loss. The likely error in the corrosion rate is characterised by the 90% confidence interval for the slope of the line of best fit.

	New coupon			Old coupon	
	Initiation time, t_i , (months)	Corrosion rate, r , (mm/yr)	+/- 90% CI	Corrosion rate, r , (mm/yr)	+/- 90% CI
Melbourne (A)	24	0.6	0.5	1.1	0.5
Melbourne (B)	16	6.1	1.3	6.5	0.6
Perth (A)	9	11.6	1.7	10.2	0.5
Perth (B)	15	5.3	1.7	5.5	0.7
Sydney (A)	>36			0.6	0.5
Sydney (B)	>36			0.7	0.7

[23,29,30,39,40]), and numerous cases have been reported where the inclusion of high alkalinity materials and/or aggregates have lowered the rate of corrosion experienced by concretes exposed to sewer environments (for example see [30]). In this study the corrosion rates for new coupons were measured side by side against old coupons with significantly lower alkalinity and more open pore structure. The differences in concrete properties suggest that corrosion rates experienced by old coupons should be significantly greater than new coupons at each field site. Based on the generally accepted inverse relationship between the corrosion rate and the level of concrete alkalinity (see Eq. (1) and [26,31]), the old coupons would be expected to have experienced rates of loss up to 40% faster than the new coupons at each site. This, however, was not the case (Table 2). The corrosion rates recorded for the new and old coupons at the two Perth and Melbourne field sites were in reasonably agreement (Table 2) and while the rates of corrosion

of the old coupons were marginally higher at Melbourne A, B and Perth B sites the differences were well within the level of uncertainty in the measured rates. At the Perth A sites the corrosion rates of the old coupons were marginally lower than those of the new coupons.

The agreement between new and old coupon corrosion rates is an interesting outcome particularly as it was noted at all sites over a wide range of environmental conditions. The results indicate that, at least for OPC based concretes, the relationship between the rate of corrosion and material properties may be more complex than previously thought and alkalinity as determined by the method of Snell and Etre [34] does not fully characterise the acid resistance of OPC concretes. Possibly a more comprehensive examination of the acid neutralisation capacity of the concrete using an approach such as outlined by Glass and Buenfeld [41] is required to fully characterise the acid resistance of OPC concretes in sewers.

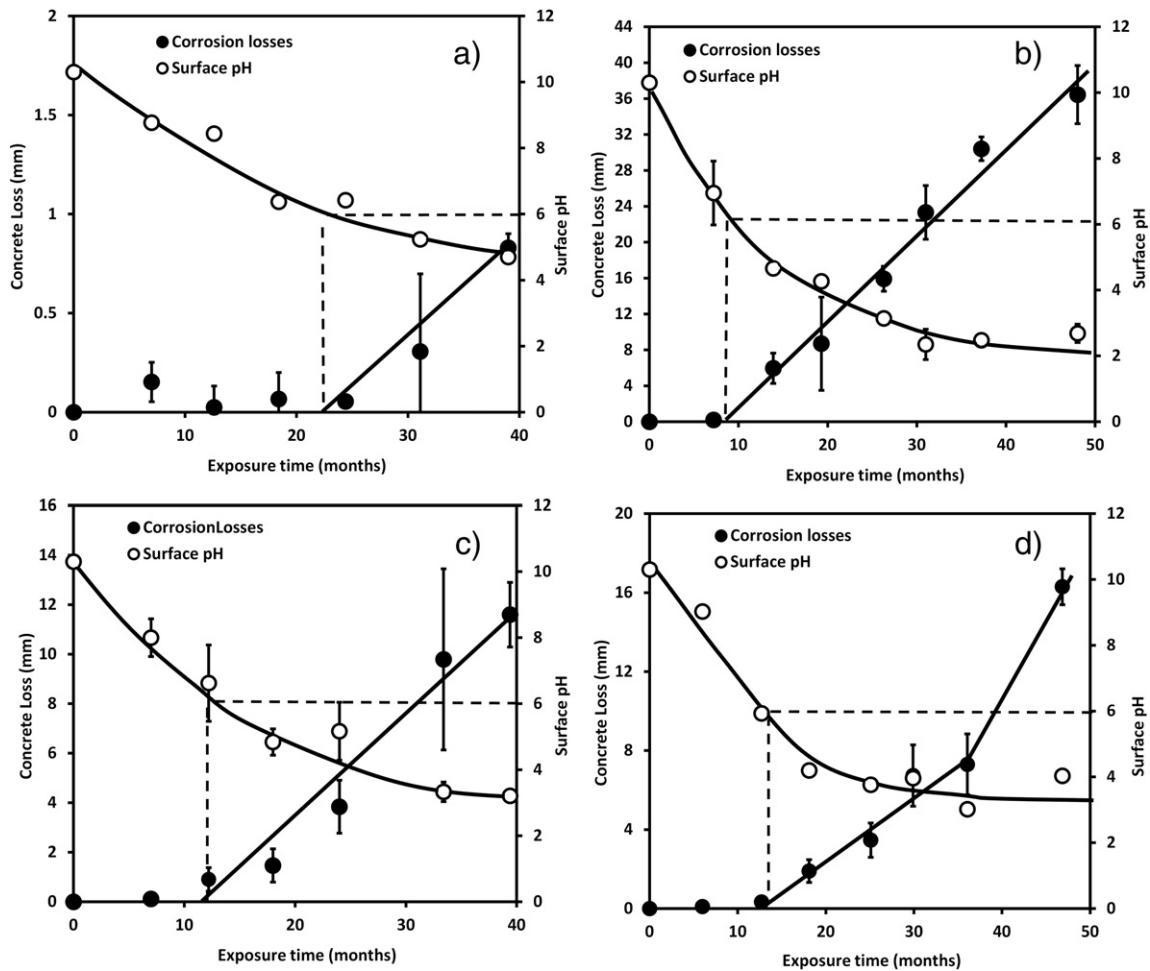


Fig. 10. Corrosion losses and surface pH trends for new coupons at individual sites. (a) Melbourne A, (b) Perth A, (c) Melbourne B and (d) Perth B.

Another possible explanation behind the better than expected performance of the old coupons may lie in its mineralogy. XRD analysis of the old coupon concrete revealed the presence of significant levels of calcite throughout the old concrete. The presence of calcite within the concrete pore structure has previously been linked to increased resistance to microbial acid attack [42,43] although it has been suggested [26] that the resistance is due to pore blocking which limits the ingress of acid and microorganisms into the concrete pore system. Pore size distribution analysis and bulk porosity measurements of old coupons however reveal that the pore structure is more open than that of the new coupons. Nevertheless the impact of concrete chemistry and mineralogy on acid resistance will be the focus of future work.

A further possibility is that the corrosion product layer is having an impact on the rate of corrosion. As noted in the previous field trial [1] the rate of corrosion remained essentially constant at all sites despite the increasing thickness of the corrosion product that builds up on the remaining sound concrete surface (Figs. 7 and 8). This would at first suggest that the depth of the corrosion product layer and its properties have minimal effect on the rate of corrosion. The relatively porous and fractured nature of the corrosion product layer (especially in and around exposed aggregate) implies that it is unlikely to act as an effective barrier to the transport of acids produced by surface bacteria to the sound concrete interface below and hence the development of the corrosion product layer is not likely to impede the access of nutrients to the bacteria. Never the less it is noted that the corrosion product layer was generally thicker on the old coupons and therefore may have contributed to the observed corrosion behaviour. Again this will be investigated in future work.

Lastly the similarity in corrosion rates experienced by the two different concretes may indicate that the chemical reaction at the sound concrete interface is not the rate limiting step in the corrosion process. Other possible rate limiting steps include the rate of absorption of H₂S into the condensate film and the rate at which acid is produced by the bacteria present.

5.3. Development of a first pass model to predict rates of corrosion

Overall the progress of sewer pipe corrosion can be summarised as shown in Fig. 11. Initially, when a reinforced concrete sewer pipe is put into service there is little or no mass loss for some period of time (months to a few years). During this time the activities of abiotic and subsequently neutrophilic bacteria and fungi lower the surface pH of the concrete. Once the surface pH reaches about 6, corrosion of concrete commences. This will continue at a uniform rate semi-indefinitely. It is evident that the duration of the initiation period and the subsequent rate of corrosion will be functions of the local environmental conditions. As already mentioned the number of parameters which may influence

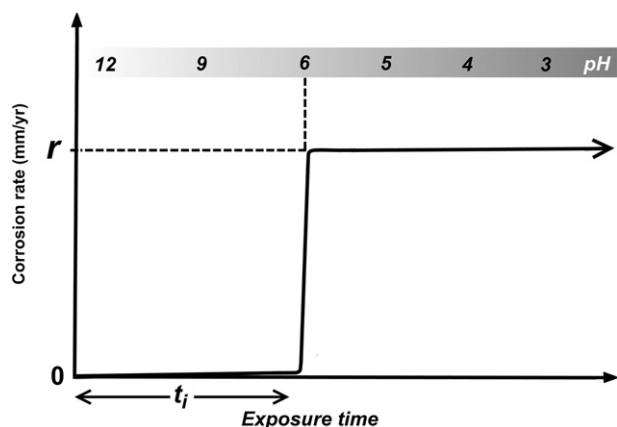


Fig. 11. Bi-linear model of sewer pipe corrosion. Both the initiation time, t_i , and the subsequent rate of corrosion, r , will be dependent on the local sewer environmental conditions.

the observed rate of material loss is considerable with many factors are difficult to quantify. In developing a first pass model of sewer pipe corrosion the present study will therefore focus on relating the observed loss of material to three primary gas phase factors: sewer gas temperature, H₂S concentration and humidity.

Although the field trials have provided information useful for verification of the form of the model, direct application of the data for model calibration is complicated by the differences in H₂S, temperature and humidity between sites. Progress can be made, however, by invoking the fundamental physical, chemical and biological processes known to take place and using this information to interpret the field data. The development of a first pass model calibrated to the available data and describing the relationship between these environmental variables and the rate of corrosion is now discussed.

For model calibration, data from actual operational sewer pipes are necessary. Data for steady state corrosion rates and long term averages of H₂S, humidity and temperature conditions are required. Such data, however, have rarely been reported in the literature and, as found during this project, are rarely measured in-situ. One reason is the practicality of collecting the necessary data. For example, while it is clear from the above exposition that good estimates of relative humidity are of particular interest, the instruments available to measure it last only a short time in the sewer environment. As a result, for model development work it has usually been assumed that humidity is at, or very close to, saturation. As noted above, this is not always the case. Also deviations from saturation can have significant influence on corrosion activity. Tables 1 and 2 shows the data available for model calibration obtained during the course of the present study. Some additional data is given in Table 3.

5.3.1. Effect of sewer gas temperature

The influence of gas phase temperature on the chemical, physiochemical and biological processes involved in sewer corrosion can be described as a function of the activation energy using the Arrhenius relationship:

$$k_r \propto e^{-E_a/RT} \quad (4)$$

where k_r , (mol s⁻¹), is the rate of the process in question at temperature = T (K), E_a is the activation energy for the process (J mol⁻¹) and R is the universal gas constant (= 8.314 J mol⁻¹ K⁻¹).

Activation energies ranging from 35 to 80 kJ mol⁻¹ have been reported for abiotic and biotic oxidation and for uptake of H₂S under aerobic conditions in wastewater environments [44,45] and in seawater [46]. In a study of bio-leaching kinetics Franzmann [47] reported an activation energy of 47 kJ mol⁻¹ for the oxidation of elemental sulfur by *At. Thiooxidans*. In an earlier study Parker [7] reports on the amount of acid produced over a 50 day period by *Th. Concretivorans* (later named *Acidithiobacillus thiooxidans*) for a range of temperatures. This corresponds approximately to an activation energy for acid production of 55 kJ mol⁻¹ for the temperature range 11–30 °C. Taken together, these results show a considerable range in activation energies. For the present study the field data shown in Tables 1 and 3 were used to obtain a best-fit estimate of the activation energy. This indicated a value at the midrange of reported values of $E_a = 45$ kJ mol⁻¹. The temperature dependence for the corrosion rate can be expressed as follows:

$$C \propto e^{(-45000/RT)} \quad (5)$$

5.3.2. Influence of sewer gas H₂S concentration

Gas phase H₂S levels often are used as the primary (and often sole) indicator/predictor of the corrosivity of the environment inside a sewer pipe. An insight regarding the validity of this approach can be obtained by plotting the available data for rates of concrete corrosion against the average H₂S gas concentration recorded at the same location

Table 3

Additional corrosion and average environmental data.

Source	Corrosion rate (mm/yr)	H ₂ S conc. (ppm)	Relative humidity (-)	Temperature (°C)
Morton et al., 1989	3.5	11.9	0.94	17.1
Parker and Jackson, 1965 (Perth)	1.1	12.4	0.86	25.8
Parker and Jackson, 1965 (Melbourne)	4.4	140.3	0.94	20.7

(Fig. 12a). From the scatter in the data it is clear that H₂S alone is not a good indicator (and therefore predictor) of concrete corrosion rate. For example, the rate of corrosion experienced at Perth B ($C = 5.3\text{--}5.5\text{ mm yr}^{-1}$) is lower than at Melbourne B ($C = 6.1\text{ to }6.5\text{ mm yr}^{-1}$), despite Perth B having H₂S levels approximately 70 times higher. This demonstrates that other factors, such as temperature (see above) and relative humidity (see below), also have an influence on corrosion activity.

The functional relationship between H₂S level and corrosion rate is of interest. Previous studies [48–50] have suggested Monod, exponential and simple power law relationships. Although it was not stated explicitly, the nature of the laboratory trials used in these studies suggests that all tests were conducted in conditions at, or close to, 100% humidity. A linear regression of a log-log plot of H₂S and temperature at sites involved in this study where average humidities are >0.97 (Melbourne A, B and Perth A – Fig. 12b), shows that corrosion rates are approximately proportional to the square root of the H₂S concentration under similar conditions. While this result is based on a limited number of sites, operating at different temperatures, it is consistent with Nielsen [3] who found that the oxidation of hydrogen sulphide by corroding concrete surfaces followed a simple nth order kinetics with a process order of 0.45 to 0.75. It is therefore proposed that the dependency of the rate of corrosion on gas phase H₂S concentration be expressed as follows:

$$C \propto [H_2S]^{0.5} \quad (6)$$

5.3.3. Influence of sewer gas relative humidity

Fig. 12a shows a significant disparity between corrosion rates at sites close to saturation (average humidity > 0.97, dark points) and sites where the sewer is drier (humidity < 0.94, white points). An extreme example is Perth B for which low corrosion losses correspond to a very high H₂S environment. The possibility that inhibition of sulfur oxidising bacteria by excessive H₂S levels could be responsible for suppression of corrosion activity at Perth B was discarded since according to data presented by Buisman et al. [51] that would require much higher H₂S levels. Support for the proposition that low humidities are

suppressing corrosion activity at Perth B is seen in the significant increase in corrosion losses, for both new and old coupons, after 36 months exposure corresponding to a period when sewer gas humidity at this site increased from <90% to >95% (Figs. 7 and 8).

Apart from the influence humidity may have on microbiological growth and activity and hence corrosion caused by such activity, it also has an effect on the moisture available for ionic reactions and related processes involved in abiotic corrosion. The relationship between humidity in the sewer gas space and the moisture content of the concrete sewer walls is therefore of interest. In the present study the field samples were located at the crown of the sewers. The concrete at that location can be wetted by aerosols generated by sewage turbulence, by occasional high rises in wastewater levels or by the condensation of water vapour from the warm, moist sewer atmosphere [20]. In the present study wastewater levels were considered rarely to reach the sewer crown. Also aerosol emissions are considered to be low in general. It follows that surface wetness and hence concrete moisture content most likely is the result primarily of capillary condensation of the surrounding water vapour and therefore in this case is sensitive to changes in sewer gas humidity. This is important since it is known (e.g. [26]) that for relative humidity less than ~85%, the moisture level within the concrete matrix is sufficiently low as to halt acid production by bacteria, (often described as a “dry pipe”). At higher humidity levels, water vapour condenses initially within the smaller diameter pores and then progressively in larger diameter pores with further humidity increases until eventually, at 100% humidity, the pore space is completely filled. At this point a condensate film forms on the concrete surface of the pipe.

Before considering the relationship between gas space humidity and concrete wall moisture and hence corrosion losses, two practical implications follow from the above. First, corrosion activity can be much reduced or even halted, even at very high H₂S levels, if humidity within the sewer is lowered sufficiently, for example by increased ventilation, forced or natural. Second, although many sewers are regularly monitored for H₂S, monitoring of humidity is much less common. As a result the sewer atmosphere humidity levels and their time dependent characteristics are not well known. The Perth B example shows that such data are important as without adequate knowledge of the humidity

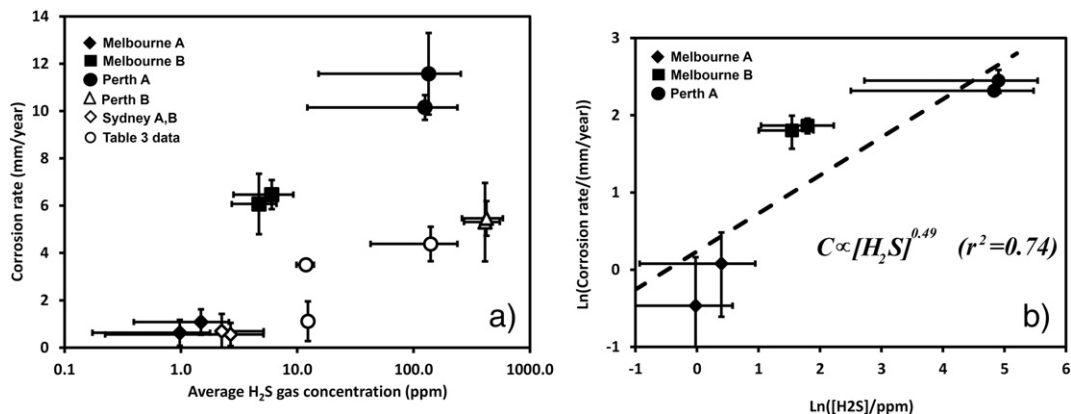


Fig. 12. The rate of corrosion as a function of average H₂S concentration. Plot of corrosion rate versus H₂S level at all sites (a) and log-log plot of data from Perth A and Melbourne A, B sites where average humidity is close to saturation (b).

conditions, (or the resultant levels of concrete moisture content), making reasonable estimates of corrosion activity is likely to remain problematic.

To estimate the influence of humidity on concrete corrosion it is necessary to have available a relationship between humidity and concrete moisture content. It will be a function of the pore size distribution of the concrete, the geometry and tortuosity of the individual pores as well as the temperature of the pipe relative to the sewer gas and as such will vary from concrete to concrete. There also may be hysteresis effects as the concretes undergo successive wetting and drying cycles (e.g. [52]). This suggests that the relationship is likely to be complex and to vary with location within the concrete pipe wall since the pore characteristics of the concrete pipe are unlikely to be homogeneous. It is also acknowledged that pore size distributions are likely to change over time as the pipe corrodes. A first approximation for the relationship between humidity and concrete moisture content was obtained as described below.

As a first step the maximum radius of a pore into which vapour will condense was estimated for relative humidities ranging from 85 to 100%. This was done using the Kelvin equation which relates the maximum radius of a pore at which capillary condensation occurs to the partial pressure of water vapour as follows [53]:

$$r_{cond} = \frac{-2\gamma V_m}{\ln\left(\frac{p}{p_{sat}}\right)RT} = \frac{-2\gamma V_m}{\ln(H)RT} \quad (7)$$

where r_{cond} is the radius of the pore in which condensation occurs (m), p/p_o is the partial pressure of the water vapour (equivalent to the relative humidity $H(-)$), γ is the surface tension of water (N m^{-1}), V_m is the molar volume of water ($\text{m}^3 \text{mol}^{-1}$), R is the universal gas constant ($= 8.314 \text{ J mol}^{-1} \text{ K}^{-1}$), and T is the absolute temperature (K). A number of assumptions were made in applying Eq. (7). These include that the pores are cylindrical (i.e. no hysteresis effects), that equilibrium exists at the pore-atmosphere interface and that the temperature of the pipe wall equals that of the sewer gas.

The pore size distribution of a representative sample of concrete cut from the centre of a new concrete coupon was then determined by Hg intrusion porosimetry, (Micrometrics AutoPore IV, (Fig. 13a)). Once the pore size distribution was established the fraction of the pore volume that is filled for a given humidity was calculated by determining the fraction of the total pore volume with pore radius $\leq r_{cond}$ as calculated in Eq. (7).

Under these conditions and assumptions the functional relationship between humidity and moisture content was obtained as shown in Fig. 13b. It illustrates the sensitivity of concrete moisture content to relatively small changes in humidity particularly for humidities above 90%.

The functional relationship can be approximated with an empirical curve of the form:

$$M_{v/v} = \left(\frac{0.1602H - 0.1355}{1 - 0.9770H} \right) \quad (8)$$

where $M_{v/v}$ is the moisture content of the concrete expressed as a fraction of the saturated volume (-).

As stated previously the relationship between moisture content and relative humidity is complex and dependent on the characteristics of the pore structure of the concrete in question so naturally will vary from concrete to concrete and also over time and space within the one concrete. Kelvin equation calculations carried out using the pore size distribution reported for other Portland cement concretes ([54,55]) and the old coupon concrete used in this study suggests that the maximum error in the prediction of $M_{v/v}$ using Eq. (8) is likely to be approximately ± 0.25 .

If it is assumed that the rate of corrosion is directly proportional to the moisture content present in the concrete pore structure then it follows that:

$$C \propto \left(\frac{0.1602H - 0.1355}{1 - 0.9770H} \right) \quad (9)$$

5.3.4. A simple model for predicting the rate of corrosion and pipe service life

From the above considerations a first pass model that predicts the rate of corrosion for a concrete pipe is proposed by gathering together Eqs. (5), (6) and (9):

$$C = A \times [H_2S]^{0.5} \times \frac{(0.1602H - 0.1355)}{(1 - 0.9770H)} \times e^{(-45000/RT)} \quad (10)$$

where C is the rate of corrosion (mm yr^{-1}), $[H_2S]$ is the concentration of hydrogen sulphide in the sewer atmosphere (ppm), H is the percent relative humidity of the sewer atmosphere (-), R is the universal gas constant ($= 8.314 \text{ J mol}^{-1} \text{ K}^{-1}$) and T is the temperature of the sewer atmosphere (K).

The value of the scaling constant, A ($= 207750 \text{ mm yr}^{-1} \text{ ppm}^{-1/2}$), was determined in the following manner: Average humidity, temperature and H_2S conditions associated with corrosion measurements obtained in this study and elsewhere (Table 3) were first inputted into Eq. (10) along with nominal values of the constant A . The resultant rates of corrosion predicted by Eq. (10) were then compared with the observed rates. The value of A was then varied to obtain the minimum least squared error between predicted and observed values of the concrete corrosion rates.

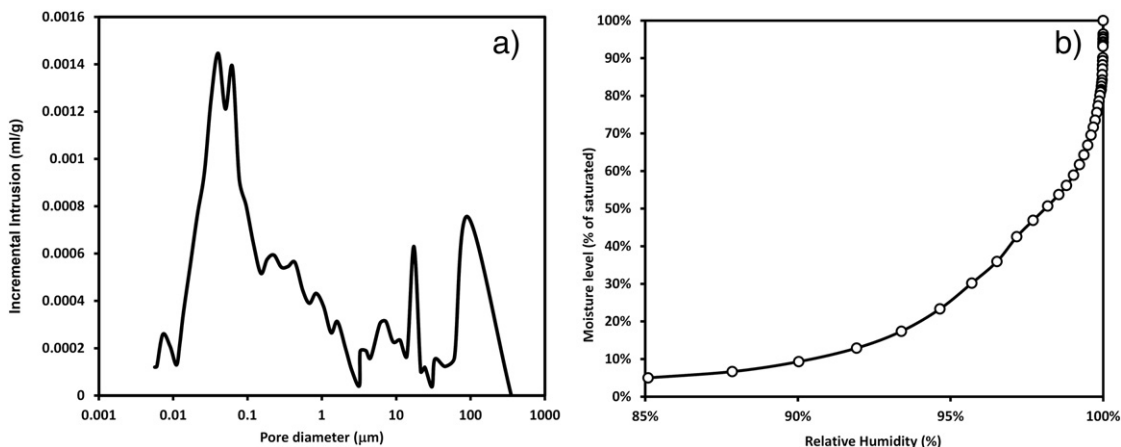


Fig. 13. (a) Pore size distribution of new coupon concrete and (b) calculated response of concrete moisture level to changes in sewer gas relative humidity.

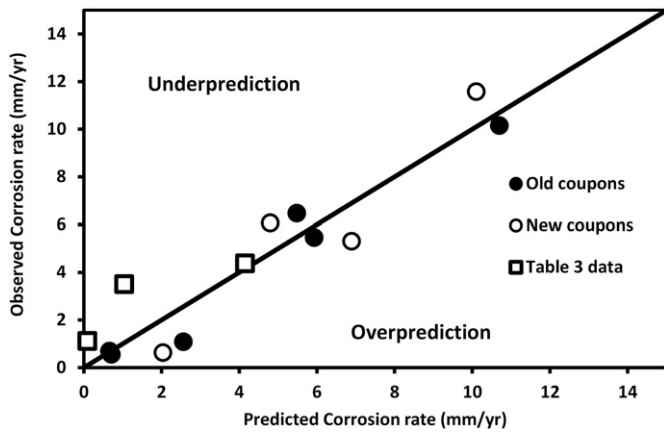


Fig. 14. Comparison between observed and predicted rates of corrosion. The solid diagonal line represents perfect agreement between predicted and observed values.

The rates of corrosion predicted by Eq. (10) and those extracted from the field observations (Table 2) and the historical data sources (Table 3) are compared in Fig. 14. It shows excellent agreement between predicted and reported corrosion rates. This means that Eq. (10) provides a good first pass estimation of the rate of concrete sewer pipe corrosion provided the relevant long term average gas phase properties (temperature, humidity and H_2S concentration) are known.

The use of OPC concrete loss and pore size distribution data in the development of the corrosion model restricts its likely validity to OPC concrete sewer pipes. Eq. (10) serves as a first estimate of the rate of corrosion and it is acknowledged that there are numerous factors contributing to the corrosion process (such as those listed earlier) not included in the model which may have an additional impact on the final rate of corrosion. Nevertheless Eq. (10) provides a starting point for estimating service life of pipes using (relatively) easy to gather data.

To estimate the long-term service life of a concrete sewer pipe it is reasonable to assume that the relatively short time to onset of corrosion, t_m , (in this study 9–24 months) can be ignored when calculating pipe service life. Assuming the service life of a concrete sewer pipe is defined as the time taken for concrete corrosion to reach the steel reinforcement in the pipe wall, the service life (SL) estimate can be obtained from:

$$SL = \frac{CD_{t=0}}{C} + t_m \approx \frac{CD_{t=0}}{C} \quad (11)$$

where SL is the service life (yrs), $CD_{t=0}$ is the original depth of concrete cover to the steel reinforcement (mm) and C is the rate of corrosion (mm year^{-1}) calculated from Eq. (10).

6. Conclusions

Corrosion loss data relating to new and 70 year old concrete coupons exposed in sewers at six Australian field sites over a four year period is presented. The data supports the earlier proposal that concrete sewer pipe corrosion follows a bi-linear trend as a function of period of exposure. This trend was observed at all sites. The bi-linear trend consists of an initial phase lasting < 2 years during which little or no loss of material occurs although the concrete surface pH lowers with time. Once the surface reaches $pH = 6$ corrosion losses commence and then continue indefinitely at a constant rate with time. This observed corrosion rate was similar for both new and old concretes despite difference in the material properties of the two concretes. The corrosion rate was shown to be a function of H_2S concentration in the sewer gas space, temperature and also the relative humidity. An important observation is that even with high H_2S low relative humidity results in very low corrosion. This highlights the importance of sewer humidity in the corrosion process.

A first pass model is proposed for the rate of sewer concrete corrosion as a function time and of the local sewer gas temperature, the relative humidity and the H_2S concentration. Testing of the model predictions against reported corrosion rates shows good agreement. This indicates that the model can be used to make a first pass estimate the rate of OPC concrete sewer pipe corrosion and to predict the service life of the pipe.

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