

A Thesis submitted for the degree of Doctor of Philosophy

**The Expansion of Pyritic Mudrocks: A  
consideration and application of physical  
tests for better understanding of the Irish  
pyrite problem.**

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# Abstract

Although the problem of the expansion of pyritic mudrock has only come to light in the last 10 years, the possibility of the problem occurring is known to engineers across the world. The mechanism behind the reaction process, the oxidation of pyrite and secondary precipitation of gypsum, is reasonably well understood and is highlighted in a series of case histories. This piece of work collated this information and brought it into a single document such that the reaction information and the case histories can be referred to and used to prevent or better understand future cases.

Irish Standard IS 398-1 was used to analyse a series of Irish properties in order to determine whether it is possible to correlate between the level of damage seen in a property and the chemical and geotechnical information obtained from the fill. Although initial results are unclear due to the influence of factors not accounted for in the data available and the variation of the fill material, further work on this method is encouraged as data refinement and numerical/statistical modelling are likely to produce clearer results.

This thesis also considers the current cases in Ireland, and the experimental work that has been carried out, in order to present a) a better understanding of the Irish situation, and b) to show the influence upon the development of laboratory tests to study the factors that influence the reaction process. This information was used to guide the development of both laboratory work and field monitoring systems, the latter included placement of a monitoring system below a domestic floor slab. Although data available from the system was incomplete, initial analysis shows that the temperature in the fill is influenced both by changes in the external temperature and in the temperature of the room above the slab.

In the laboratory, tests were designed to build upon previous work, confirming the influence of the density of the material upon the reaction rate and amount, and also showing the influence of the grading of the material on the reaction process. This indicated that the process of using well graded fill material gives a long-lasting reaction process wherein the fine material reacts initially whereas the coarse material reacts slower but for longer as the air and water take time to reach the pyrite in the centre of the larger rock particles.



# 1 Introduction

In Ireland in the early to mid-2000's, it was noticed that there was a distinct increase in the number of relatively new properties reporting instances of damage. Initially this damage was attributed to settlement of the foundations and was remediated accordingly, however, these properties again began to show further damage a few years after remediation efforts.

Around 2005, several parties began to investigate the causes of damage which was manifesting in various locations. These parties included HomeBond (the Irish equivalent of NHBC), Menolly Homes (the developer responsible for one of the housing estates showing damage) and Ballymun Regeneration (the developer of a community centre in Ballymun that was showing significant damage) (Tuohy et al., 2012, Hawkins and Stevens, 2014). Eventually the damage was determined to be due to the fill material used beneath the floor slab of the damaged structures.

Tests carried out on the fill material found that much of the material was weak mudrock that contained fine grained pyrite. The pyrite was oxidising in the damp fill material and leading to the production of gypsum, which occupies more space in the material than the minerals it replaces. Despite the literature available detailing instances of pyritic heave phenomena in Canada, Wales, the UK and other places, this was a relatively unknown phenomena in Ireland.

In 2011 a panel of experts was formed by the Irish government with the intention of investigating and advising upon the pyrite problem in Ireland, and their report, published in 2012 summarised the history of the Irish situation and how many people were likely to be affected by it. The following information is summarised from the Pyrite Panel Report (Tuohy et al., 2012), and highlights a number of key facts relevant to the pyrite situation in Ireland.

The majority of structures affected by pyritic heave in the Dublin area were constructed during the Celtic Tiger housing boom of the late 20th and early 21st century. In the period between 1995 and 2007, the Irish economy, like that in much of Europe, expanded rapidly. Residential construction comprised around 65% of all construction output in 2006, with the number of dwellings completed in 2006 up 88% on the number complete in 2000. This led to 93,000 properties being constructed in 2006.

During this period of increased construction, there was an increased demand for materials, including stone for fill - the amount of stone quarried tripled from 1993 to 2007, with production up to circa 130 million tonnes/year by the end of that period.

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Specifications for underfloor fill material in Ireland at that time primarily focused on the compactability of the material, with recommendations for maximum grain size and a “good quality hardcore: clean, crushed, well graded stone”. Although the presence of sulphates was occasionally mentioned, it was never flagged as a serious problem that a material should be tested for prior to use. Overall, this led to some developers simply requesting “Clause 804 material” for use beneath domestic properties, which defines material for various uses based primarily upon the grain size. It is clear that in many cases, however, that no testing was carried out to ensure material met the Clause 804 standard.

There seems to have been several assumptions throughout the supply chain at this time, with builders assuming that the material they requested had been tested and was good for use beneath domestic properties, and with the quarry owners assuming that the builders knew what the material was and what it should be used for considering they were requesting Clause 804 material. Discussions as to appropriation of the blame are still ongoing, and will not be covered in this document.

As to the scale of the problem in Ireland, the Pyrite Panel report provides the following figures (Tuohy et al., 2012):

- 74 estates showing signs of pyritic damage were recorded, with a total of 12,250 ground floor dwellings.
- Claims (at the time of the report) were ongoing for around 850 properties, with a further 1,100 that had been, or were in the process of being, remediated.
- This gives a pessimistic number of 10,300 properties that have pyritic infill still beneath the floor slab.

At present, the only recognised method of remediation in Ireland is to remove all defective material and replace it with good quality, inert rock. This is a significant investment in terms of both time and money, and causes major disruption to the homeowner, as all ground floor fittings and the floor slab itself must be removed as part of the remediation process. This can take between 6 & 16 weeks (Tuohy et al., 2012).

In August 2011, HomeBond formally denied all responsibility for the damage caused to properties that were part of its warranty scheme if related to pyritic heave. This included a refusal to cover remediation costs for any properties not within the claims process at this time. It should be noted that the HomeBond scheme was not specifically an insurance scheme, and that it was the builder that registered the properties they constructed with the scheme rather than the homeowner. It was, in 2008, amended to be an insurance scheme, but at this time pyritic damage was under a list of exclusions (Tuohy et al., 2012). This withdrawal left Premier Guarantee as the primary insurance agent working on the pyrite problem in Ireland.

Liberty International and Premier Guarantee Ireland have since then been working with engineers and geologists to gain a better understanding of how the reaction progresses

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and how the damage is caused. Partially, this is with the aim of determining a time line of events, from when the fill is placed, through when the damage begins to occur, to the point that the damage is significant enough that it is recognised by the homeowner and requires remediation.

Figure 1.1 details the primary starting points for this consideration of the pyrite problem in Ireland: the mechanism of the reaction and the factors that affect it, and the damage that the expansion of pyrite is known to cause.

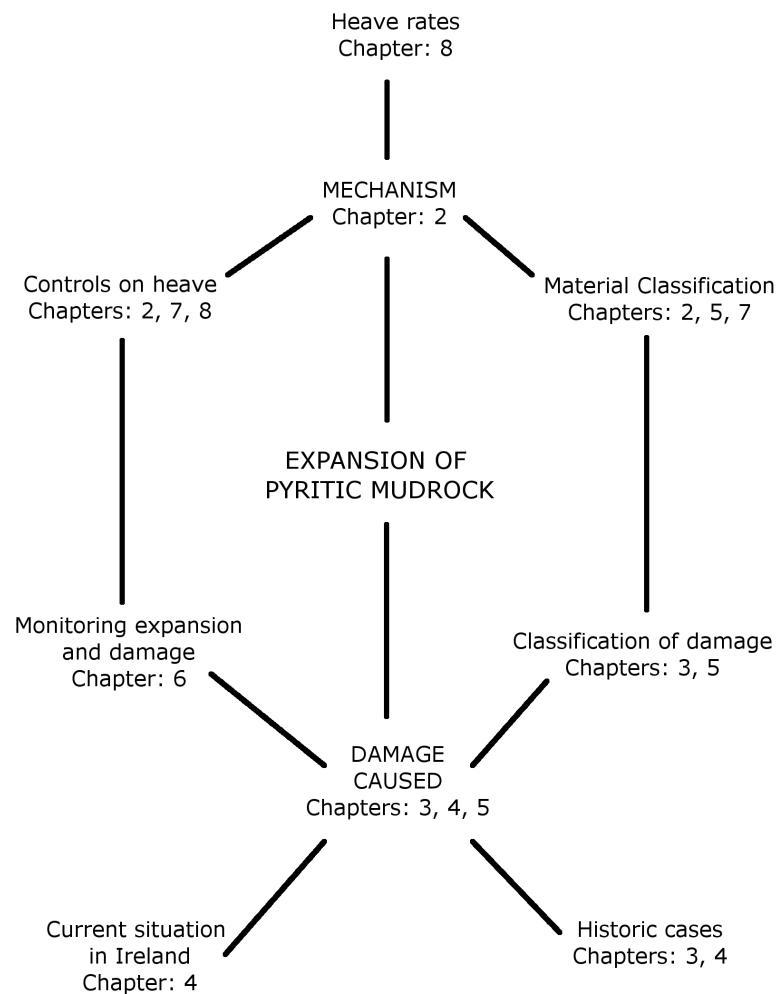


Figure 1.1: Link between the topics covered within this thesis.

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As detailed in Chapters 3, 4 and 5, the damage caused by pyritic expansion below the ground floor slab produces very distinct forms of structural damage that can be identified. This includes, but is not limited to, stellate cracking of the floor slab, uplift of internal walls and structures seated on the floor slab, cracking to internal walls, and cracking to external walls particularly on level with the damp proof membrane.

The mechanism behind the expansion of pyritic mudrocks is pretty well understood and described throughout the literature (Chapter 2), as is the damage that it causes (Chapter 3). Similarly, it is understood that there are a number of factors that influence the reaction process, what is not so well understood is the exact part these factors play, individually or combined, and how much of an influence they have on the reaction process, rate and final amount of expansion.

To that extent, the sections of this thesis have been devised with the intent of considering the available information on pyritic heave, and to build on that knowledge to further the understanding of the pyrite problem, particularly the effect it is currently having in Ireland.

There has, however, been much argument over the cause of the expansion, which was ultimately decided in the High Court. Some argued that gypsum formed passively in the void space without causing expansion, whilst others produced evidence that crystal growth could produce expansion in excess of the volume of gypsum (Chapter 2).

In part this has involved considering the systems available for classifying the damage to properties and determining whether that information may be used to correlate with the chemical properties of the fill material in order to better link the two together (Chapter 5). It has also involved a series of laboratory tests, building on existing work with the intention of clarifying the effect of various factors upon the amount and rate of expansion (Chapter 8). In order to better understand the environment below the ground floor slab, a system was designed to passively monitor the environment whilst providing minimal disruption to both the fill and the homeowner (Chapter 6). Finally, the conclusions bring together these studies to determine what has been discovered or confirmed, and leads into suggestions for further work that will hopefully answer any questions that have been raised.

## 2 The Pyrite Oxidation and Expansion Process

As mentioned in Chapter 1, the primary cause of the current problems in Ireland is the presence of Pyrite in the mudrocks used as fill material.

As the pyrite oxidises it causes acidic groundwater conditions, which can itself cause problems, but also causes reactions with calcite in the mudrock and the eventual production of gypsum. This Chapter focuses on this reaction process, the production of gypsum and the factors that affect both the amount and rate of heave caused.

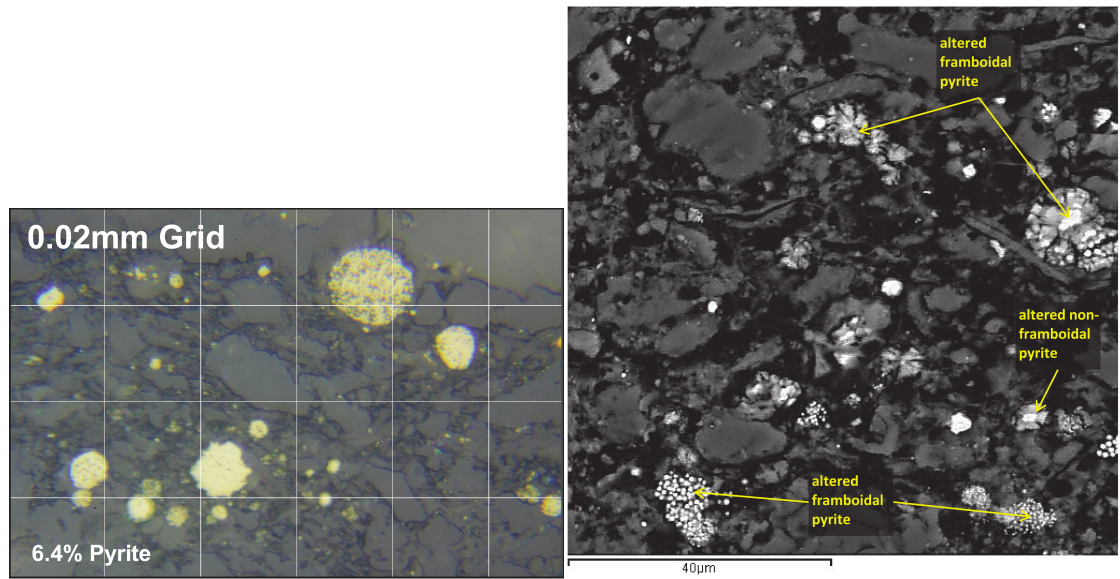
### 2.1 Pyrite

Pyrite ( $\text{FeS}_2$ ) contains both iron and sulphur in their reduced states. For iron, this is the Fe(II) state, meaning that it is unstable under atmospheric conditions, and oxidises readily to its Fe(III) state. Under normal atmospheric conditions sulphur will preferentially form sulphate ions, it is only under reducing conditions that sulphide ions, and therefore pyrite, are formed.

This means that pyrite cannot form under oxygen-rich atmospheric conditions and so is primarily found in rocks that formed within environments such as igneous formations, ore veins, and sedimentary rocks that were deposited in an anaerobic or anoxic environment (Hawkins and Pinches, 1992). Sedimentary rocks formed under such reducing conditions generally suffer little or no bioturbation, few animals being able to survive under such conditions. The resulting rocks often have a shaley fissility and commonly contain pyrite as either small disseminated grains ( $0.1\text{--}0.5\ \mu\text{m}$  in diameter) or small clusters of such grains known as framboids ( $2\text{--}40\ \mu\text{m}$  in diameter). These aggregates have a large specific surface and are, therefore, more reactive than some other forms of pyrite (Hawkins and Pinches, 1992).

It is common to find both framboidal, disseminated and cubic pyrite within dark-coloured sedimentary rocks, although the amount of pyrite is not always linked to the reactivity of the material. Rocks with quantities of pyrite as low as 0.1% by weight have been reported to react and cause significant mounts of heave (Penner et al., 1973). Figure 2.1 below shows the main forms of pyrite.

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(a) Pyrite crystals under high magnification direct light (with permission from Aidan O'Connell Associates). (b) Pyrite crystals as seen with SEM (with permission from Ground Investigations Ireland).

Figure 2.1: Pyrite crystals as seen in Irish mudrocks.

Iron sulphide can also be found as pyrrhotite ( $\text{FeS}$ ), which is its most reactive form. Due to the rapidity with which this form oxidises, it is very rarely found in rocks and, to date, has only been recorded as trace amounts in test cases in the Dublin area (Hawkins, 2012, 2014b). Hawkins (2014b) suggests that the mineral may be more common in the Irish bedrock than previously thought, and has not been identified due to its quick reaction time. At the present time, without significant investigations being carried out on fresh material, it is difficult to say whether the apparent absence of pyrrhotite in the Irish cases is due to its general absence in the rock material, or if it is due to the oxidation of any pyrrhotite that was present prior to testing.

## 2.2 The Oxidation Process

The stages of the reaction process presented here are taken from the literature currently available and generally represent the processes observed and recorded at different sites globally, all of which have shown expansion of pyrite-rich sedimentary rocks. It should be noted that, depending on individual site conditions, there may be some overlap or absence of the parts of the reaction process outlined in Figure 2.2.

## 2 The Pyrite Oxidation and Expansion Process

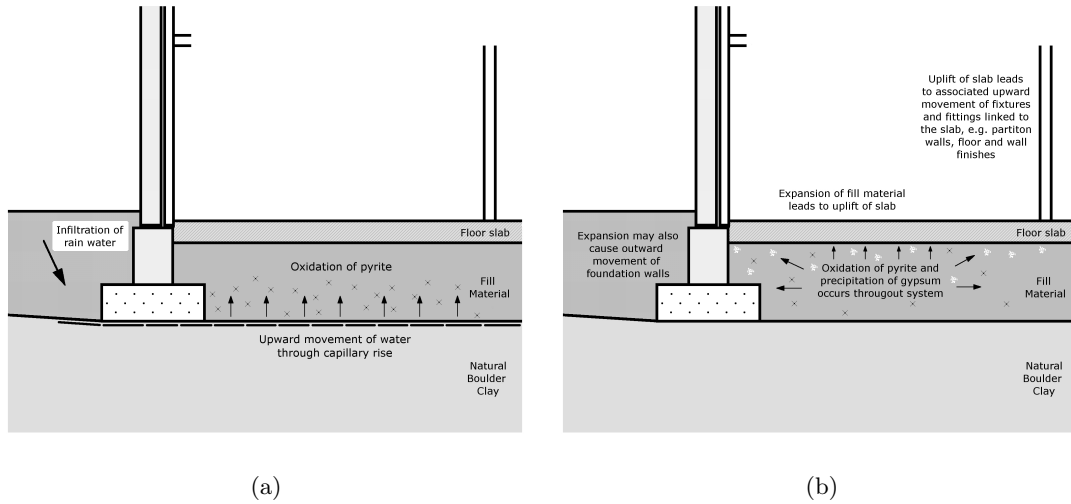


Figure 2.2: The mechanism of the reaction process from initial oxidisation to precipitation.

Figure 2.2 shows the method by which water is most likely to be introduced into the below slab environment in the Irish cases (see also Chapter 6). In the Irish cases, the local water table is located within the natural material, in many cases a heavily consolidated boulder clay, and so although the clay is likely to be damp, there will be minimal upward movement of water through this material. Instead, due to the impermeability of the clay, local rainfall and surface water runoff moves through the top soil to pool on the upper surface of the clay. As shown in Figure 2.2a, this water, when sitting on the boundary between the clay and the fill material, moves upward through the fill via capillary rise, providing a water source to support the reaction process, as outline in Equations 2.1 through 2.7 below.

Figure 2.2b shows how precipitation of gypsum within the fill material connects to other parts of the structure and begins to cause damage. The primary result of the precipitation of gypsum, and expansion of the fill, is upward movement of the floor slab. This causes upward movement of parts of the structure that are sitting on the floor slab, including internal partition walls and structures, and fittings attached to the outer walls, such as skirting boards and plasterboard wall finishes. This movement, especially where differential movement is occurring between one section of the wall and another, leads to cracking of the wall finishes. This is shown schematically in Figure 2.3.

Differential movement of the floor slab also occurs, as constraint of the slab occurs along its edges, due to its interaction with the foundation walls, leading to more upward movement in the centre of a span. This is seen in the floor level survey data obtained from properties (Chapter 5) and in the stellate cracking that is so indicative of pyritic heave (Chapters 4 and 5).

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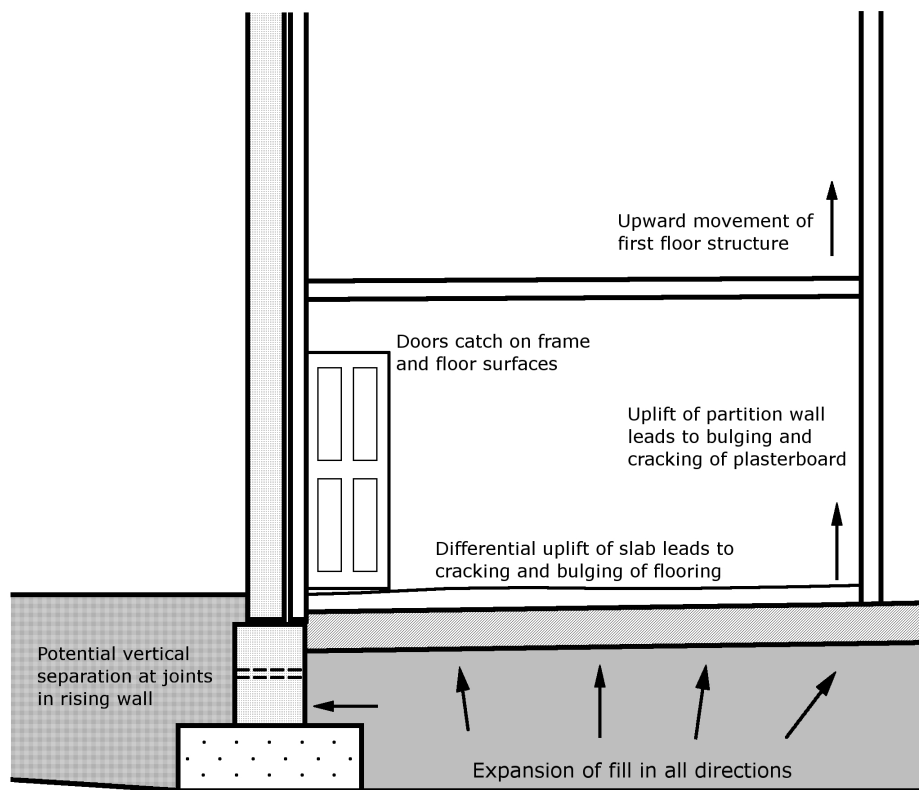


Figure 2.3: Mechanism by which expansion of the fill causes damage to the ground floor of a structure (adapted from Hawkins, 2014a).

The reaction begins with the introduction of air and water into the system. The system as referred to here can be either that of fill material beneath a floor slab or the natural rock material in a quarry or beneath a foundation. The initial introduction of air and moisture into the system occurs when the rock is excavated either to form a base for the foundation or when it is quarried to be used as fill. Similarly additional air and moisture will be introduced, especially with fill material, when the material is stored before use and later when compacted in situ. In the Ireland cases, fill material is typically placed at optimum moisture content or up to 2% below optimum in order to maximise compaction, this is discussed further in Chapter 4.

For the case of the Irish properties affected by the heave, air and water are initially introduced into the fill at the time of quarrying, which may lead to some oxidation occurring before placement, and preserved during the compaction process. Additional water comes from run off that ponds along the upper surface of the water, as mentioned

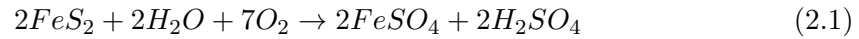


## 2 The Pyrite Oxidation and Expansion Process

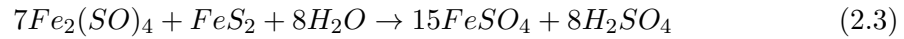
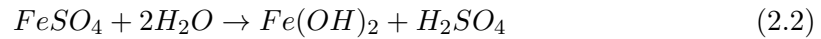
above, and movement of this water downwards through the clay is impeded by its low permeability. Air is introduced to the system through this water, which is oxygenated, and through changes in barometric pressure, which also introduces additional water vapour.

The material used as fill also has natural moisture content; so far in the testing of fill material in Ireland, this is generally between 2 and 10%. As the analysis of the fill material generally takes place between 3 – 8 years after construction, and therefore placement of the fill, this is a long-term value for the material. Further discussion on the effects of capillary rise and thermal gradient on the fill material is contained in Chapter 6.

The initial reaction between the pyrite, water and air leads to the production of  $\text{SO}_4^{2-}$ ,  $\text{Fe}^{2+}$  and  $\text{H}^+$  ions, which in turn react to form sulphuric acid and ferrous sulphate, as shown in reaction (2.1). However, this reaction is normally inhibited after this stage as the Fe(II) ion is stable in acidic conditions (Cripps, 2009; Hawkins and Pinches, 1992; Hawkins, 2014a; Hawkins, 2014b).



For the reaction to progress beyond this point, the acid must either be removed or neutralised. Within some systems this occurs by the production of ferric sulphate, which is itself a strong oxidising agent capable of breaking down pyrite, shown in reactions (2.2) and (2.3).



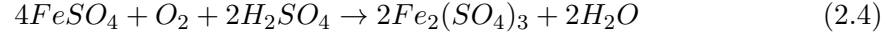
The main result of the oxidation and breakdown of pyrite is that the reaction products themselves are highly reactive leading to further reactions within the system. However, it is the presence of bacteria within the rocks and groundwater that have the most influence on the speed and progress of the reactions.

Bacteria of the genus *Thiobacillus* occur in soils and rocks; however, only one of the species, *Thiobacillus ferrooxidans*, can convert Fe(II) to Fe(III) as well as converting the sulphides to sulphates. In an acid environment, this is a key feature of pyrite degradation (Bell, 1983; Taylor and Cripps, 1984; Hawkins and Pinches, 1987b, 1992).

*Thiobacillus* operate best in the temperature range of 25 – 40°C (Holt, 1977; Hawkins and Pinches, 1987b), with a decline in the number of bacteria at temperatures beginning around 50°C. The additional carbon that the bacteria use as a food source is supplied to the system both as atmospheric  $\text{CO}_2$  and via the reaction process outline in reaction (2.6).

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The reaction involving *Thiobacillus ferrooxidans* to oxidise Fe(II) to Fe(III) is shown in reaction (2.4).



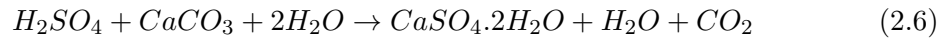
Hawkins and Pinches (1992) cite examples that suggest that reaction (2.1) may also be affected by bacterial action so that the reaction would progress as in reaction (2.5).



The ultimate result of these reactions is that the water present within the system is now low in pH and rich in sulphates. Movement of this water, even over small distances, allows it to react with other minerals, such as  $CaCO_3$ , or man-made structures within the system such as buried concrete structures such as blockwork or mortar, which may in turn lead to the formation of minerals such as ettringite and thaumasite. This is detailed further in Section 2.3.

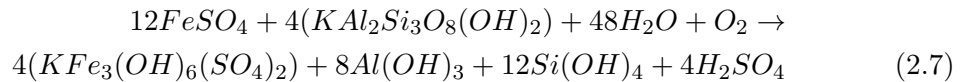
In the case studies where heave has been attributed to pyrite degradation, gypsum is often noted to be present in the rock or fill material. In many instances it is present as the acicular crystal form of selenite, often occurring in cracks and along bedding plane laminations (Section 2.4).

Gypsum is formed due to the reaction of sulphuric acid with calcium minerals that often occur in the rock formations, which in many cases is calcite,  $CaCO_3$  (Cripps, 2009; Hawkins and Pinches, 1992; Taylor & Cripps, 1984).



The effect of this gypsum precipitation is considered further in Section 2.4.

Other minerals common in areas where pyrite degradation has occurred are brown, iron hydroxide minerals (such as goethite and limonite) and the bright yellow mineral jarosite, which is also known to contribute towards heave (Berube et al., 1986). jarosite is most often formed as a result of chemical reactions involving clay minerals (especially illite) common in mudrocks, though it is generally only found in systems with a pH of less than 4.



Jarosite, for example, was found in cases in the UK (Llandough Hospital (Section 3.1.1) and Teeside (Section refNixon)) and Canada (Bell Canada Building (Section 3.2.1) and Sainte Foy (Section

Brub et al. (1986) also found fibroferrite ( $\text{FeSO}_4(\text{OH}) \cdot 5\text{H}_2\text{O} \pm \text{Al}$ ) to be present at the Sainte-Foy site. The fibroferrite in that case was believed to have been the main cause of heave in the system with gypsum being the secondary cause, likely due to a limited amount of calcite in the rock material at that location.

## 2.3 Sulphate Attack

As shown in reaction (2.1), the initial pyrite oxidation produces sulphate ions ( $\text{SO}_4^{-2}$ ) that as part of the reaction process can produce sulphuric acid. This can lead in the first instance, to acid attack on buried structures such as foundation walls and metal pipes or reinforcement.

The production of the sulphuric acid can lead to ground waters with a pH of 2 or less, this can cause corrosion of buried metal elements, as well as dissolution of the cement matrix of any buried concrete. This may present as corrosion of the surface of the concrete, or may also involve precipitation of gypsum crystals, as the acid reacts with calcium products present in the cement. Generally, it is considered that for this acid attack to cause significant damage, it requires that the acidic groundwater must be mobile (BRE, 2005).

However, when the buried concrete structure is in contact with ground water containing sulphate ions, these ions can migrate into the concrete as the water phase is considered to be continuous across the soil/concrete interface (BRE, 2005). This process occurs as a consequence of the concentration gradient from the ground to the concrete, and will continue if the ions are consumed by reactions with the cement and/or aggregate particles. The reactions that can then occur are complex, but the mechanism for sulphate attack can generally be split into 2 cases: conventional and thaumasite sulphate attack.

Conventional sulphate attack requires:

- A source of sulphates,
- Mobile groundwater, and
- The presence of calcium hydroxide ( $\text{Ca}(\text{OH})_2$ ) and calcium aluminate hydrate ( $\text{CaAl}_2\text{O}_3(\text{OH})_2$ ) in the cement matrix.

When these conditions are met, and in the presence of alkaline pore-water in the concrete, the reaction process can lead to formation, amongst other minerals, of ettringite ( $3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 31\text{H}_2\text{O}$ ). Formation of ettringite primarily causes damage to the concrete by expansion – it has a greater volume than the minerals that it replaces during the reaction process.

The first effect of conventional sulphate attack is to increase the strength and density of the concrete as minerals precipitate into the available pore space. Beyond that point, further precipitation increases the internal stress of the concrete until it exceeds the

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tensile strength of the material, after which damage to the concrete occurs (BRE, 2005). It is also possible in this case to get precipitation of gypsum in the concrete pore spaces, which also causes damage by expansion within the material, however this is generally secondary to the precipitation of ettringite.

Thaumasite sulphate attack requires:

- A source of sulphates,
- Mobile groundwater,
- A source of calcium silicate hydrate (found in Portland cement),
- A source of carbonate,
- Low temperatures (ideally less than 15°C), and
- a pH of 10.5 or greater.

The availability of carbonate ions changes the reaction process from that in the conventional form of attack, and when combined with a low temperature, mobile groundwater and a high pH, thaumasite ( $\text{CaSiO}_3 \cdot \text{CaCO}_3 \cdot \text{CaSO}_4 \cdot 15\text{H}_2\text{O}$ ) is formed (Floyd, 2003; BRE, 2005; Czerewko et al., 2010).

In the United Kingdom at shallow foundation depths (down to around 1.2 m, as would be the likely maximum depth for a domestic dwelling) the average temperature ranges from a minimum of +4°C in March to a maximum of +17°C in September. Once progressing to a depth of 3 m, these values change to +8°C and +12°C respectively (BRE, 2005).

The calcium silicate phases required for the reaction process to produce thaumasite, are one of the main binding agents of Portland cement, providing much of its strength. So, in addition to causing damage by expansion as with the conventional form of sulphate attack, thaumasite attack also causes damage by weakening the concrete. In severe cases this can cause such significant damage to the cement matrix that it is often described as being reduced to “mush”, in which the thaumasite and surrounding material can be broken by hand (Floyd, 2003; BRE, 2005; Reid et al., 2005).

Since the thaumasite form of attack does not depend on calcium aluminate hydrates, it is possible for it to occur in specialist “sulphate resistant” cements (BRE, 2005).

Generally, sulphate attack on concrete is rarely seen in Irish structures undergoing pyritic heave. The main considerations as to why it has not been seen in the Irish cases are as follows:

- Evidence of sulphate attack has been missed due to a focus on the fill material and heave damage to the floor slab and structure.
- The reaction process and subsequent appearance of damage progresses quickly enough that remediation occurs before significant sulphate attack takes place.

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- The minimal amount of water movement through the fill beneath the slab inhibits the movement of fresh sulphate ions into the concrete and cement that is required for the reactions to progress.
- In the case of thaumasite attack, the large amount of sulphate ions give the water in the fill a sufficiently low pH that it is difficult to achieve the necessarily high pH required for thaumasite to form. It is unclear from the current evidence whether or not the formation of gypsum and associated reduction of the  $\text{SO}_4$  concentration would have such an effect on the pH as to allow the thaumasite reaction to occur.

In any case, removal of the defective fill material is considered to remove the threat of sulphate attack, such that any that may have begun to occur will not progress further.

One case where sulphate attack was noted to have occurred in Ireland was the Ballymun Youth Centre, and is detailed in Section 4.3.1.

### 2.4 Causes of Expansion

Recent work has suggested that there may be a certain amount of heave caused by the initial breakdown and oxidation of the pyrite framboids (Hawkins, 2012). Due to its low specific surface, cubic pyrite often shows only small alteration rims; however, framboidal pyrite has a much higher specific surface and so is more reactive. This is shown by recent SEM images of partially oxidised framboids, in which cracks have formed during the oxidation process and increased their volume, as shown in Figure 2.4.

Although this volume increase is small, even a 100% volume increase of a 20  $\mu\text{m}$  framboid would not cause noticeable expansion, the fracturing of the framboids is significant because it increases the access of moisture and oxygen to the pyrite and so aids progression of the reaction.

However, the main cause of expansion identified in pyritic mudrocks, especially in Irish properties, is the precipitation of gypsum and other sulphate minerals (such as jarosite) along fractures within the mudrocks and void space within compacted fill material.

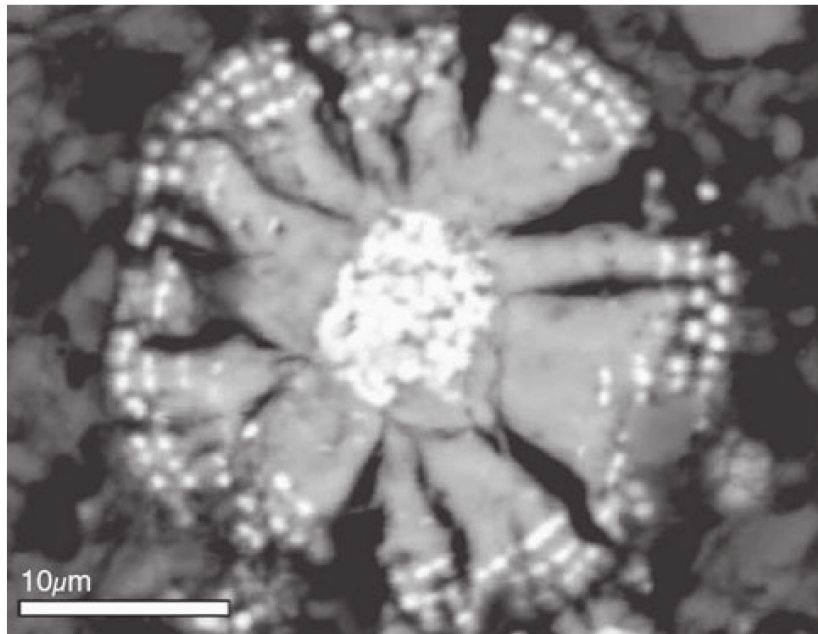


Figure 2.4: A pyrite framboid under SEM showing cracking in the outer material that furthers the reaction in the core material (Hawkins, 2012).

Gypsum is formed, in solution, by the interaction of sulphuric acid and calcite. The gypsum then precipitates into the void space of the material once the fluid has reached its saturation point. One of the key properties of gypsum that contributes to its role in heave of rock/fill is that it has a low solubility (120 g/litre); this means that once it has crystallised out of solution, it is not easily dissolved back into solution or flushed out by fresh water (Hawkins and Pinches, 1992).

Primarily, the gypsum has a lower specific gravity than the pyrite that it replaces:  $G_s$  2.3 for gypsum and  $G_s$  4.8-5.1 for pyrite. In the case of the amounts of pyrite and gypsum present at Llandough Hospital (Section 3.1.1), Hawkins & Pinches (1987a) cite a maximum possible heave of 100 mm due to the replacement of pyrite and calcite with less dense minerals such as gypsum and jarosite.

In addition, both the location and the crystal habit in which the gypsum precipitates have an impact on the amount of heave. Generally in cases of pyritic heave, gypsum will form in one or both of the following habits: needle-like, acicular crystals or flat, tabular euhedral crystals, shown in Figure 2.5. The acicular crystals (more often found in cracks within mudstone particles) will exert more pressure on the system than larger euhedral crystals (often found in the fine material surrounding larger particles) (Taylor and Cripps, 1984).

## 2 The Pyrite Oxidation and Expansion Process

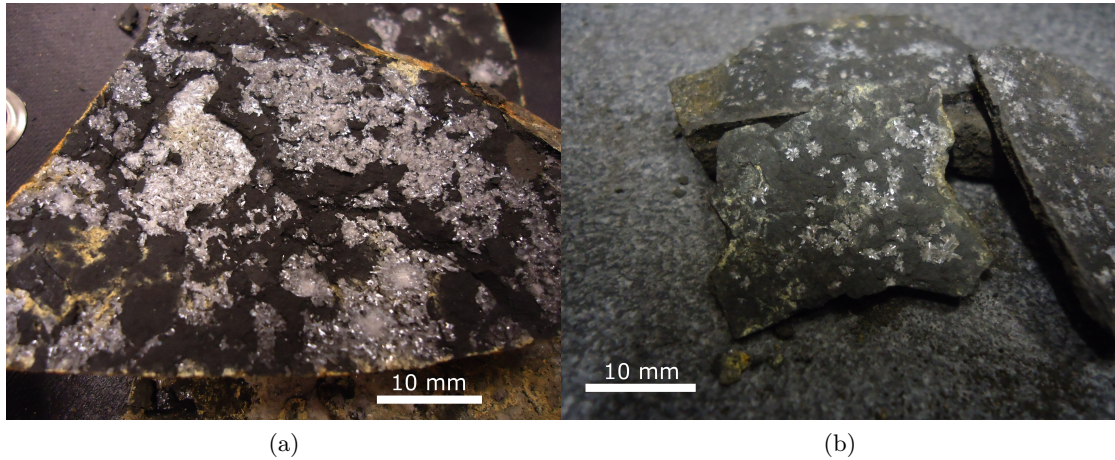


Figure 2.5: Gypsum crystals seen on lamination surfaces of mudrock particles.

The habit and precipitation location of the gypsum crystals within the fill depends upon the energy required to form the crystals. If the energy required to form a larger crystal, of whatever habit, is less than that required to begin formation of a new crystal, then the larger crystal will continue to grow, even in cases where it is restricted (Taylor and Cripps, 1984).

The growth of gypsum crystals in this way causes an increase in pressure within the rock and so expansion of the material. Table 2.1 below lists key values taken from the literature to show the range of pressures believed to be induced in rock systems due to precipitation of gypsum.

Source	Value	Note
Lutenegger et al (in Berube et al., 1986)	28 kPa	Suggested minimum value
Quigley et al (in Berube et al., 1986)	72 kPa	Minimum value from damage
Quigley and Vogan, 1970	74 kPa	
Fasiska et al (in Berube et al., 1986)	500 kPa	Suggested minimum value
Maher et al., 2011	600 kPa	Value calculated from testing

Table 2.1: Potential crystallisation pressures for gypsum, taken from the literature and based on either site investigations or test results.

It is also possible to calculate the theoretical pressures that will be imposed upon the system by considering the degree of saturation of the solution within the system, as proposed by Winkler and Singer and applied to gypsum by Taylor & Cripps (1984, Cripps, 2009) and shown in equation (2.8).

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$$P_c = \frac{RT}{V} \ln \frac{c}{c_s} \quad (2.8)$$

Where: R = gas constant, 8.3143 J/Kmol; T = absolute temperature (0°C = 273°K); V = molecular volume of gypsum (54.8 cm<sup>3</sup>/mol); c = concentration of solution (g/litre); c<sub>s</sub> = concentration of saturated solution (g/litre);

This gives the theoretical pressure values shown in Figure 2.6, which shows how the potential crystallisation pressure increases with both temperature and the concentration of the solution. It should be noted that these are the maximum values for the given mineral, and so may not necessarily be reached under all possible ground conditions (Taylor & Cripps, 1984).

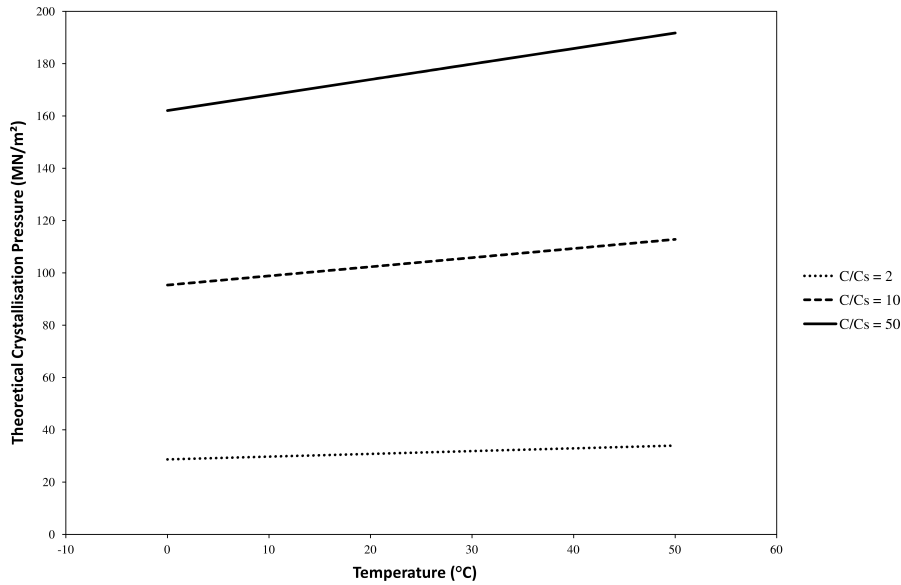


Figure 2.6: Theoretical crystallisation pressures for Gypsum in MN/m<sup>2</sup> (after Taylor and Cripps, 1984).

In rocks with a range of pore sizes, crystallisation will occur preferentially in the larger pores before the smaller ones. Expansion only begins to occur when crystallisation begins in smaller pore spaces (Taylor & Cripps, 1984). The amount of pressure induced within a system by crystallisation also depends upon the habit of the crystal, with acicular crystals that grow perpendicular to laminations or bedding exerting higher pressure than prismatic or tabular crystals that tend to grow parallel to laminations.

Estimates of pressure increases due to crystallisation vary widely, likely due to the complexity of the reactions and the range of secondary minerals involved (see also Table 2.1).

The amount of expansion in a system due to precipitation of secondary minerals can also be considered as percentage increases, as shown in Table 2.2.



## 2 The Pyrite Oxidation and Expansion Process

Original Material	Product Mineral	Expansion %
Pyrite, FeS <sub>2</sub>	Jarosite	115
	Melanterite	536
	Anhydrous ferrous sulphate	350
Calcite, CaCO <sub>3</sub>	Gypsum	103

Table 2.2: Percentage expansion due to mineral alteration (Taylor & Cripps, 1984).

It should be noted that the figures given in Table 2.2 are approximate maximum values for expansion, since it is rare that in an open system precipitation of secondary minerals occurs at the same location as the initial pyrite reaction. This is because the presence of oxygen and water are key to the initial oxidation of pyrite, so an open or dynamic environment, in which air, water and minerals in solution will circulate relatively freely, is required to maintain the reaction process. Therefore, the amount of heave is dependent upon the supply of solution and ability of the secondary minerals to precipitate out of solution (Taylor & Cripps, 1984).

It should be noted, however, that this free movement of water through the rock material is unlikely to occur in this same way when considering material isolated beneath a floor slab. There may be some free movement of water from below the foundation level, however, the foundation walls themselves form a barrier to water movement laterally through the system.

The material that the Irish structures are founded on is a heavily over consolidated boulder clay. This material has a low porosity, and is likely to become saturated due to rainwater runoff from garden and other open areas around the structures. Although in some areas the local water table may be high enough for the upper section of the boulder clay to be saturated in this manner, the runoff mechanism will provide water in area where the water table is several metres below surface level. In either case, the moisture in the boulder clay will be drawn up into the fill by capillary rise.

Another possible source of water for the below-foundation environment, is the seepage of rainwater, either through the blockwork below the damp-proof course, or along the boundary between the impermeable boulder clay and the fill material. In order to consider the below-foundation environment better, along with the temperature and moisture profiles that are expected to be present in the fill, an Arduino based monitoring system was designed. This is described further in Chapter 6.

One of the key points discovered in the literature is the rapidity with which the reaction can progress given the correct atmospheric conditions. This is discussed briefly below and in more detail in Section 2.6.

Structures subject to pyritic heave in Canada generally show significant signs of damage within 10-15 years of final construction (CTQ-M200, 2001). In Ireland these signs typically appear within 5-8 years (Sutton et al., 2013); in this case the difference is believed

to be largely due to the lower average annual temperature in Canada, which slows the overall reaction process (see Section 2.6).

This rapid oxidation and expansion process has implications when testing samples during a site investigation as the reactions can and often do continue after sampling (Hawkins and Pinches, 1987b). This has led to a sampling protocol being implemented in Ireland for all tests related to insurance claims for pyritic heave, which includes storage of samples in sealed plastic bags under cool conditions between sampling and testing. The reduced temperature is intended to slow bacterial activity in the rocks thereby slowing the reaction rate.

## 2.5 Damage Caused by Pyritic Heave

The majority of damage seen in Ireland is related to the expansion of the fill, there is generally little damage seen due to sulphate attack (Sections 2.3 and 4.3.1).

The precipitation of gypsum increases the pressure within the rock material, as detailed in Section 2.4 and as such causes an increase of stress upon the floor slab and foundation walls. Although the damage seen is detailed in Section 5.1, it is also discussed here to illustrate the key types of damage seen with pyritic heave.

A lot of the damage is linked to the movement of the floor slab as caused by expansion of the material beneath it. Other damage not directly linked to the slab movement includes outward movement of the external walls and cracking/movement of the foundation walls (Tuohy et al., 2012). However, damage directly related to movement of the slab includes:

- Heaving and cracking of the floor slab, cracking to tiled finishes or development of gaps at joints in wooden finishes.
- Upward movement and bulging/cracking to finishes of partition walls, especially those resting upon the floor slab.
- Sticking of both internal and external doors.
- Upward movement of cupboards and other such fixings as rest on the floor slab, leads to tilting of work surfaces and sticking of doors.
- Potential stress added to service pipes, causing distortion that may lead to blockage and leaks, especially at joints.

This is detailed further in Chapters 4 and 5.

## 2.6 Factors Affecting Heave

The effect of heave of the fill material on the walls and floors of a property is dependent on a number of factors including the type of structure and the pressures involved (caused by both the building and expansion of the fill). It is generally found to be the more lightly loaded areas of the structure that are affected the most, for example the floor slab and paved areas (Cripps, 2009).

The amount of expansion seen at a given site is variable as many of the controlling factors will vary greatly even across a small site. Some of these factors are outlined below, and the effect they have is discussed within this section.

- Rock lithology/composition
  - Amount of pyrite
  - Amount of mudstone/limestone
- Depth/amount of fill
- Temperature
- Density of fill
  - Amount and availability of void space
- Water content
  - Source of water
- Grading of fill
  - Specific surface

### 2.6.1 Rock Lithology/Composition

In terms of the Ireland situation, the Pyrite Panel Report (Tuohy et al., 2012) discusses the rock formations from which the quarried fill material was obtained, these are discussed in detail in Chapter 4.

In general though, the majority of the framboidal pyrite in the Irish cases is contained in the mudrocks. Still, there is a large amount of variation seen in the mudrocks, with variations most often seen in the amounts of calcareous and carbonaceous material present, along with the proportion of material that is shaley/laminated. There is also a varying proportion of muddy limestone, be this argillaceous or carbonaceous.

The variation of the amounts of materials is largely dependent upon how the materials were interbedded in the quarry, and it is therefore difficult to predict how this will vary.

Pyrite is also present in the more pure limestone materials, however, this is generally of the less-reactive cubic form. Therefore, the amount of mudrock is believed to be linked to the amount of pyrite, and the limestone provides the calcium carbonate required to produce the gypsum.

### 2.6.2 Depth/Amount of Fill

Experimental results (Sutton et al., 2013; McCabe et al., 2015) and field data (Ballivy et al., 2002) indicate that the amount of expansion relates to the depth of the fill. A greater thickness of rock material contains more reaction products and will show more expansion: as Ballivy et al. (2002) state “a vertical expansion of 1% has a greater impact on a 1.5 m thick rockfill than on a 20 cm thick rockfill”.

This is supported by the result of experiments carried out by Sutton et al. (2013) at the National University of Ireland, Galway, in which a series of swell tests on Irish mudrock were carried out (similar to those detailed in Chapter 8), with material thicknesses of 500, 750 and 1000 mm. Analysis of test results implied that the expansion rate for the material was proportional to the thickness of the fill material, with the tests showing an average rate of 2 – 3 mm/year/m thickness of fill.

### 2.6.3 Temperature

The effect of temperature on the oxidation of pyrite is currently the source of some debate.

Past testing has shown that an increase in temperature increases the production of sulphate within pyritic rocks, as it speeds up the reaction process. In addition, an increase in temperature also speeds the rate of precipitation of gypsum and the uptake of water through the capillary zone.

Hawkins and Pinches (1987b) carried out tests on samples of pyritic rock from the Westbury beds, in which samples were stored in humid conditions at varying temperatures and the pH and acid soluble sulphate (%SO<sub>4</sub>) were measured over the course of 15 weeks.

As shown in Figure 2.7a, the samples kept at the highest temperature (41.5°C) show the highest production of SO<sub>4</sub>, and similarly the lowest pH value (Figure 2.7). Whereas the samples kept at 7.5°C show the lowest rate of production and highest pH values. The low level of activity in the latter case is likely due to a lack of bacterial activity at this temperature (Section 2.2).

Of interest however, is the way in which the samples held at 18.5, 29.5 and 41.5°C all show a similar rate of SO<sub>4</sub> production, at an increase of between 0.056 and 0.074 % SO<sub>4</sub> per week. The sample held at 7.5°C however shows an increase of 0.016 % SO<sub>4</sub> per week.

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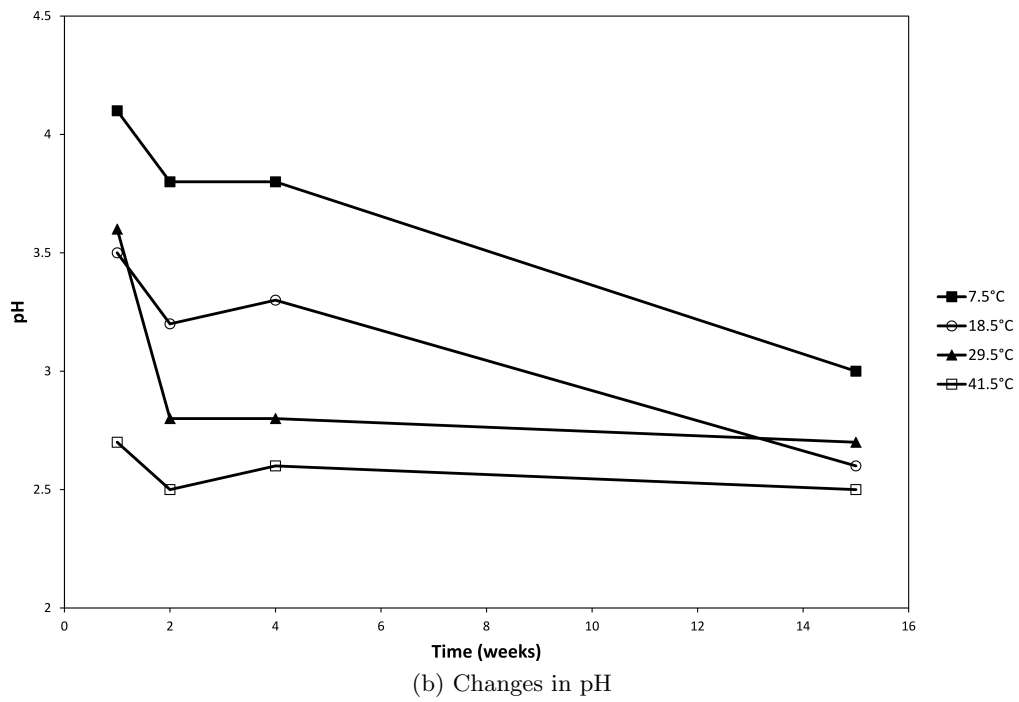
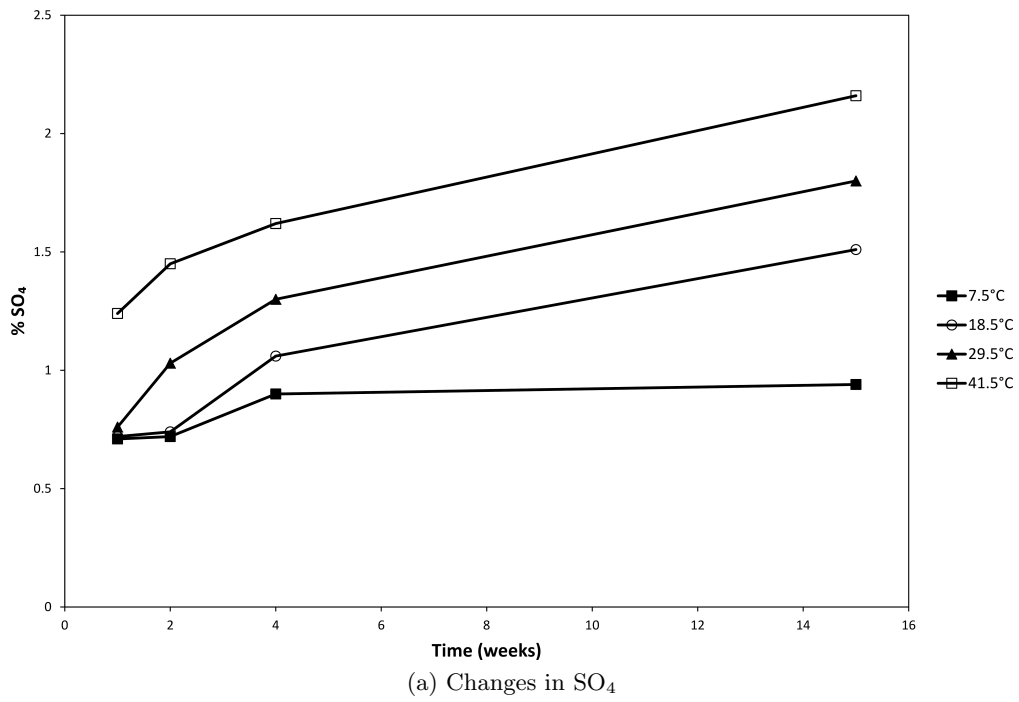


Figure 2.7: Changes in the chemistry of a pyritic mudrock over 15 weeks at varied temperatures (after Hawkins, 2014b).

## 2 The Pyrite Oxidation and Expansion Process

It is for this reason that the sampling procedure implemented in Ireland includes the specification to store all samples in an airtight container at a temperature of 2 – 4 degrees and to be tested as soon as possible.

It is also implied that an increased temperature speeds the reaction by the damage seen at both Llandough Hospital, Wales and Rideau Health Centre, Canada (Chapter 3), in which the temperature below the slab was increased due to the presence of a service duct as part of the foundations.

Similarly, Ballivy et al. (2002) theorised that temperature was a factor in the different amounts of expansion seen in the fill below basement floor slabs compared to that beneath garage floor slabs. The basement areas were frequently heated during the colder winter months, whereas the garages were very rarely heated at any time of year.

The reason behind this implied increase in reaction rate at higher temperatures is the preference of the *Thiobacillus* bacteria to operate at temperatures between 25 – 40°C (Holt, 1977), which catalyse the reaction process.

In contradiction to these points of view, recent work carried out at National University of Ireland, Galway (McCabe et al., 2015) implies that an increase in temperature has a minimal effect on the rate of the reaction process.

Initial work at Galway (Sutton et al., 2013) showed that there was a temperature effect, with tests showing a decrease in reaction rate when there was a significant temperature drop (in that case from 11.3 to 7.9°C). Such a temperature drop is suggested to be significant enough to cause the bacteria to go into thermal shock. It was also noted that the thermal expansion, although not specifically recorded during the tests, was unlikely to account for more than 10–15% of the total expansion recorded during the time period (Sutton et al., 2013).

This would also seem to correspond with the shorter reaction time in the Irish cases compared to those in Canada. The average time between placement of fill and the appearance of damage in Ireland is 3–5 years at an average annual temperature of 10°C (Tuohy et al., 2012) compared to 10–15 years in Canada where the average annual temperature is around 4°C (CTQ-M200, 2001).

However, a more recent set of tests were carried out in a temperature controlled room, where the temperature was cycled and the results upon the samples measured.

The temperature changes varied, with periods of time spent at or around 10, 15 and 20°C. Temperatures were held at each particular level for between 9 and 223 days. For all of the tests, once the data was normalised, there was an initial increase in expansion of the tests, followed by a return to the expansion rate that was seen prior to the temperature change.

Calculations of the thermal expansion of both the aluminium frame supporting the dial gauges and the mudstone within the swell tests was used to determine that this initial apparent increase in expansion at the point at which the temperature was increased, was

## 2 The Pyrite Oxidation and Expansion Process

due to movement of the frame giving a false reading (McCabe et al., 2015). In order to confirm this theory, the rates at each temperature were compared, minus the first week after each change to allow the initial thermal effect to be discounted.

Allowing for some scatter in results, when plotting the displacement rates against the temperature, there was no significant difference in those rates between the different temperatures.

It is not clear within the paper (McCabe et al., 2015) whether or not there is some initial expansion of the material associated with the temperature change, or whether the movement seen in the material was purely due to thermal expansion. If there was some initial expansion of the material that would not be lost once the material has settled after the change, that would provide an increase in the amount of expansion of the test, even if it did not effect the overall rate of expansion.

Also of note is that the maximum temperature the tests were carried out at was circa 20°C (the paper states that the room was maintained within 1°C of the intended temperature), where as the literature suggests that in some cases the temperature below the floor slab may reach or even exceed 30°C, as seen at Llandough Hospital and Rideau Health Centre (Chapter 3).

It would be interesting to see these tests carried out at both a higher temperature, within the optimal operating range of the *Thiobacillus* bacteria, and for longer periods of time, in order to see if the results are duplicated.

### 2.6.4 Density of Fill

An increased density in the underfloor fill does not necessarily equal more precipitation of gypsum and therefore more expansion. However, a higher density of fill means less void space and so precipitation will occur along fractures and laminations in the mudstone. This means that a denser material will show signs of expansion before a less dense material that can accommodate a certain amount of precipitation before expansion. This is also acknowledged by the Pyrite Panel Report (Tuohy et al., 2012), wherein they comment that properties where the fill density is low, the amount of damage related to expansion is similarly low.

Work by McCabe et al. (2015) followed on from earlier work carried out at the National University of Ireland, Galway (previous work by Sutton et al., 2013), by looking in part at the effect of density upon the rate of expansion of pyritic mudrocks. In those tests, as sample density increased, so did both the amount of expansion and the expansion rate. Chapter 8 details the expansion of the Galway samples.

### **2.6.5 Water Content**

Water is a key requirement of the reaction process, as shown in Equation (2.1), although should the material be submerged, the reaction will be inhibited since the availability of oxygen under full saturation is considerably less than under partial saturation.

In the case of the Irish properties, the material was recommended to be compacted whilst damp in order to achieve maximum compaction (NRA, 2013), and frequently this material was left exposed to the elements, and so may have had a significant moisture content before being placed. The moisture contents of material tested as part of the insurance process range from around 2 – 11%, suggesting that there is likely to be a secondary source of water for the reaction process.

In cases where the structure is sited on the bedrock material, water will generally be sourced from rainwater run-off, the ponding of this run-off on the boulder clay, and, in some cases, the local water table.

Work carried out by Sutton et al. (2013) at National University of Ireland, Galway placed swell tests of the type detailed in Chapter 8 in varying depths of water. It was found from these tests that the depth of water had minimal effect upon the amount and rate of expansion that was seen. In fact, the control test that was contained in a tank with no water showed a faster reaction rate than the tests in tanks with water. This is likely due to a portion of the other test being below the water level and therefore submerged with no reaction taking place in this small section of the test and implies that there is sufficient water in the fill as compacted to sustain the reaction process.

### **2.6.6 Grading of Fill**

There is little information available about the effect of grading upon the amount and rate of expansion, however, it would be expected that material with a higher proportion of fine material, and therefore a larger specific surface, would react quicker due to the increased availability of reaction products.

Much of the material used in Ireland was likely to have been ordered to Clause 804 standard, which provides an upper and lower limit for particles sizes of material for use in construction. This means the material should be well graded, and can be easily compacted in situ. Figure 2.8 shows the limits, and the effect of grading is discussed further in Section 7.3.



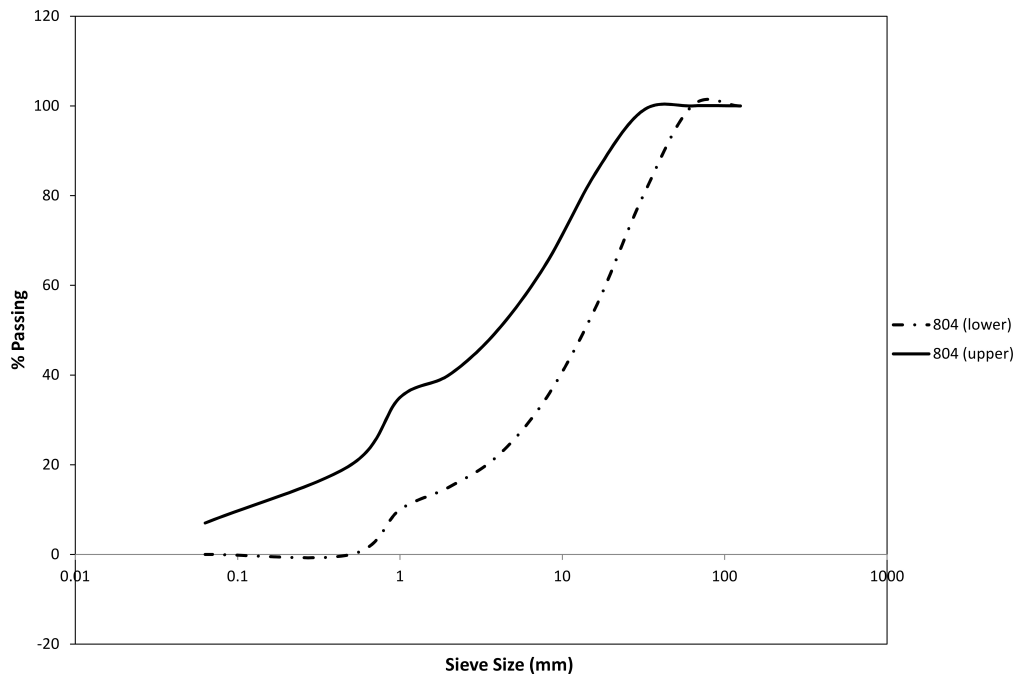


Figure 2.8: PSD limits for Clause 804 material, intended to supply a well graded material for compaction.

## 2.7 Identifying the Potential for Heave

Previous work carried out (Hawkins and Pinches, 1992; Nixon, 1978; Reid et al., 2005) has identified various factors that indicate the potential for heave in pyritic mudrocks, which has been expanded upon by more recent work in Ireland to create the scheme discussed in Section 5.1. Some of the key points are detailed below, and many are expanded upon within this report.

Hawkins and Pinches (1992) present the following list of factors that should be considered at the design stage of a project in order to minimise the impact from pyrite related heave.

- Presence of visible pyrite – in hand samples the cubic form of pyrite is often visible, however, thin sections or photomicrographs will be needed to confirm the presence of framboidal or disseminated pyrite.
- Colour – rocks that are high in organic and sulphide content are likely to be dark grey to black in colour.
- Assessment of weathering – if chemical weathering has already taken place, it reduces the possibility of future weathering. The weathering state may be determined by the presence of secondary minerals.

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- Rock structure – heave is more likely in laminated and fissile rocks such as shale. The laminations and discontinuities allow circulation of air and water to aid the oxidation reaction.
- Sulphur analysis – the values for both total acid soluble sulphur and total sulphur should be obtained and considered. In particular, the difference between total sulphur and the amount of sulphate indicates the potential for oxidation of sulphide.
- XRD – used to obtain weight percentages for pyrite, gypsum and other primary and secondary minerals.
- Acidity – sulphuric acid is produced by the initial oxidation reaction, thus giving the rock system a low pH.
- Microbiology – the oxidation reaction is catalysed by the presence of bacteria of the species *T. ferrooxidans*.
- Proposed structure – any occupied and heated structure will raise temperatures beneath the floor slab, thereby accelerating the reaction process. Care should be taken when designing structures seated upon pyritic bedrock material.

However, by far the most reliable method of identifying a material's probability to expand due to oxidation of pyrite, is to study the chemistry and geology of the material. The specifics of the limits contained within the various standards concerning material close to buried concrete or steel structures are discussed in Section 4.2 and mentioned briefly below.

The values taken across much of the literature are those suggested in the TRL report (Reid et al., 2005), and are as follows for material within 0.5 m of buried concrete:

Property	Value	Source
Water soluble sulphate	1.5 g/l	Reid et al., 2005
Total potential sulphate	0.3%	Reid et al., 2005
Oxidisable sulphides	0.06%	Reid et al., 2005
Acid soluble sulphate	0.2%	Clause 804
Pyrite	1%	Nixon, 1978

Table 2.3: Limiting values according to the standards in place at the time of writing.

It should be noted that even if a material exceeds the limits in Table 2.3, it does not necessarily mean that it is totally unsuitable for use, it does indicate that the material has a potential to expand and therefore is unsuitable for certain uses.

## 2.8 Remediation Options

Considering the literature available considering the pyrite problem, there is relatively little detailed information regarding the remediation options that are available, and what success has been achieved with each method. Generally, these remediation options fall under one of two categories:

1. Removal of the pyritic material and subsequent repair of the structure, and
2. Creation of unfavourable conditions in the ground below the structure in order to inhibit the reaction.

Removal of the pyritic material is by far the most widely accepted and used method of remediation, especially in cases where the pyritic material is imported fill rather than the natural bedrock. It can, however, be a costly and time-consuming process – for example, in the case of the Irish properties, the average turn-around time for the remediation is 12 weeks.

This process also involves considerable disruption, all of the fixtures and fittings on the ground floor must be removed before the floor slab itself is dug out and the fill material removed. The cost of this process will also increase as the size of the floor slab increases. However, this is generally a “one-off” cost – once the process has been completed, it is highly unlikely that it will occur again.

The situation of remediation in Ireland is expanded upon and discussed in Chapter 4.

Creation of unfavourable conditions below a structure is, on paper, a more efficient and less disruptive solution to the problem of pyritic heave. However, at the present time, all of the options that are known to have potential in remediating the problem are costly to both initiate and maintain.

The most prominent example in the literature, is that of the remediation at the Bell Canada building (Section 3.2.1), where the main portion of the remediation work focused on both raising the local water table and the addition of alkaline salts to raise the water pH. This used some 12 tonnes of potassium hydrate and required 16,000 litres of water per day to maintain the new water level (Penner et al., 1970, 1973). It is not specified in the literature what the long-term actions were at this site, nor is it mentioned if there were any detrimental effects of the remediation other than some minor settlement.

There has been some discussion in the past few years in Ireland of the use of foam-injection techniques to force either a neutralising solution or micro-fine cement through holes in the floor slab and thereby render the pyritic material inert. However, as far as this author is aware, there has been little investigation into this beyond the initial discussion phase. Maher et al. (2011) make mention of the use of foam injection in tests carried out by Golder Associates on pyritic fill material, however, there is no indication of what effect this had or if further tests were carried out.

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Another idea following on from this was to introduce a bacterium into the system to consume available oxygen and thereby inhibit the reaction process. However, no further information on this has been provided in the past three years, either regarding the feasibility of the project, nor whether options or testing procedures had been decided upon.

In some instances, the remediation of a site includes methods that combine both removal of material with engineering solutions to limit further damage – these are generally applied in instances where a structure is situated on pyritic bedrock. For example, the Sainte-Foy site in Canada (Section 3.2.5) combined removal of weathered material with sealing of the newly exposed rock, creation of a gap into which future expansion could occur without affecting the floor slab, and the installation of collapsible units to absorb the swelling material (Berube et al., 1986).

By far the easiest option is to avoid construction on or involving pyritic material, however, if this is not possible, the following options should be considered during the design process (after Hawkins and Pinches, 1992).

- Ensure the structure and construction process does not encourage the reactions:
  - Maintain the ground water level.
  - Begin construction upon the existing weathered material and keep exposure of fresh material to a minimum.
  - Reduce access of oxygen into the ground by using impermeable membranes or material such as bitumen.
  - Avoid sub-floor heating or use suitable insulation below the floor slab.
- Design the structure so as to be able to absorb heave:
  - If possible use deeper foundations that are sited either below the problematic material or below the local water table, although this does not eradicate the possibility of heave below the floor slab.
  - Use suspended floor slabs or incorporate collapsible units.
  - Use flexible slabs or joints.
  - Use rafted foundations.
- Work to counter the effects of the oxidation process:
  - Introduce lateral drainage to move oxidation products away from the foundations.
  - Raise the local ground pH .
  - Consider the introduction of bactericides to slow the reaction process.

- Raise the local water table, saturating the pyritic material and inhibiting the oxidation reaction.

## 2.9 Discussion

Pyrite is a naturally occurring iron sulphide mineral, found in many rocks and environments. Its framboidal form, however, poses a threat in its increased tendency to react in the presence of air and water to form numerous secondary minerals including gypsum and jarosite, and in some cases minerals such as ettringite and thaumasite that are part of sulphate attack processes affecting buried concrete.

Gypsum and jarosite are associated with the expansion of the host rock, which occurs when the gypsum is precipitated into void spaces within the material, and also when precipitating along laminations within the host rock. When this host rock is comprised of a weak material such as mudrock, the precipitation process forces the laminations further open causing both expansion of the material and also increasing access for more air and water to continue the reaction process. This is discussed further in the Case Study examples shown in Chapters 3 and 4.

When occurring in a restrained environment, such as that below the floor slab of a domestic dwelling, pressures are induced upon the floor slab and the foundation walls, potentially leading to movement of both and causing cracking and other damage to the structure. Ranges for the pressures caused by pyritic heave range from 28 kPa (Berube et al., 1986) to 600 kPa (Maher et al., 2011) based upon both laboratory and site investigation data.

The damage seen in different structures is explored further in other sections of this thesis (Chapters 3, 4 and 5 in particular), but includes heave and stellate cracking of the foundation slab, movement and cracking of walls and fittings that sit upon the floor slab and cracking of the outer foundation walls at the level of the damp proof membrane.

Factors that are thought to affect the pyrite reaction process and rate are considered throughout the literature, with laboratory testing confirming the influence or lack thereof of several of these factors. Tests carried out at the University of Galway have considered the influence of the depth of fill and the water supply (Sutton et al., 2013) and the effect of temperature and compaction (McCabe et al., 2015), and have led to the development of the tests carried out at Sheffield (Chapter 8) to look at factors such as the grading of the material.

All of these factors contribute to not only the analysis of existing cases of pyritic heave, but also to the identification of rock materials that have the potential for heave. Chemical analysis will detail materials that contain excessive levels of pyrite and are not safe for use near buried concrete or steel (Chapter 4), but for materials that are borderline with their chemistry, details such as what environment they will be used in can be important.

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For example, it may still be possible to use a mudrock that has a borderline level of pyrite in an environment where those factors that contribute to heave are absent or mitigated, such as where the local water table is always going to saturate the material and therefore inhibit the reaction process.

In cases where pyritic heave is ongoing, such as the current situation in parts of Ireland (Chapter 4), remediation options are currently limited. Although ongoing maintenance of the fill surface (Section 3.2.5), and efforts to raise the local water table, flooding the material and inhibiting the reaction (Section 3.2.1) have been used at other sites, the only accepted remediation option currently in place is to remove the defective fill material (Section 4.4.2).

This process is considered to be the most reliable method of removing the threat of pyritic heave, as all of the defective fill material is removed and replaced with clean, inert fill. Since this requires removal of the existing floor slab, it is a costly process that is both time consuming and involves considerable disruption to the homeowner. This is discussed in more detail, as inferred above, in Chapter 4.

Chapter 3 details the damage seen in historical cases of pyritic heave, primarily in Canada and the UK, and discusses how these cases can and should influence the approach to future cases of pyritic heave.

### **3 Case Studies – Examples of Pyritic Heave Worldwide**

Although the process of pyritic heave is relatively well understood and described in the literature (Chapter 2) and in both European and British standards (Chapter 4), the damage caused by the heave will vary depending on various factors. The factors that will affect the rate and amount of heave, such as the moisture content and composition of the material, are discussed in Chapter 2. However, there are other factors that can be considered within the literature, including the source of the material (either fill or natural bedrock), the type of foundation and structure, and any environmental factors that would slow or speed the reaction rate.

To that extent, a search of the literature reveals evidence of problems linked to the oxidation of pyrite and subsequent expansion of gypsum. Although the problem with pyritic heave in Ireland only started to become evident circa 2005 (Tuohy et al., 2012), there is evidence of cases in other areas of the world that date back into the 20<sup>th</sup> Century, with well documented cases from the 1970's onwards. It can be argued that these should have been of relevance to the situation in Ireland, and this is discussed further in Chapter 4 along with the relevant Standards in use at the time that mention the problems with pyritic materials.

Many of the cases presented below relate to problems with construction on pyritic bedrock, although cases in Teeside, UK and Montreal, Canada show the implications of using pyritic material as fill beneath domestic properties. The relative scarcity of cases involving pyritic fill might be assumed to be due to the increased likelihood of fill material being tested before use, whereas testing may not be so thorough on bedrock material that has been constructed upon at an earlier date.

Some of the most prominent cases from the literature, sorted according to geographical distribution, are presented and discussed below, along with a consideration of how this information can be compared to the situation in Ireland (Section 3.5).

## 3.1 United Kingdom

### 3.1.1 Llandough Hospital, Cardiff

Llandough hospital was constructed primarily between 1927 and 1933, and comprises a series of two-storey wards that extend southwards from a main corridor. During the construction process, up to 3 metres depth of material was removed in order to level the site. It is very likely that this exposed fresh, unweathered material to air and water across a large area (Hawkins and Pinches, 1987a).

The construction of the building involved casting the floor slab in direct contact with the ground and including a service duct approximately 1.2 m deep by 1.0 m wide as part of the foundation. This service duct (as at the Rideau Health Centre (section 3.2.2)) increased the temperature gradient in the material adjacent to it well above the natural ground temperature.

Damage was initially noted to the structure in 1982, and a structural survey recorded that “significant movement had occurred at wall-plate level”, that there was bulging of the external walls at first floor level and cracking was visible to the brickwork. Two weeks later, further movement was noted to have occurred and the external walls were shored up in an effort to stabilise the structure (Hawkins and Pinches, 1987a). Internal inspections revealed that cracking up to 10 mm in width was present to the internal walls and that there was a difference in floor levels of up to 60 mm that indicated an overall arching of the floor slab (Figure 3.1). This arching was determined to be the cause of the damage to the internal walls.



### 3 Case Studies – Examples of Pyritic Heave Worldwide

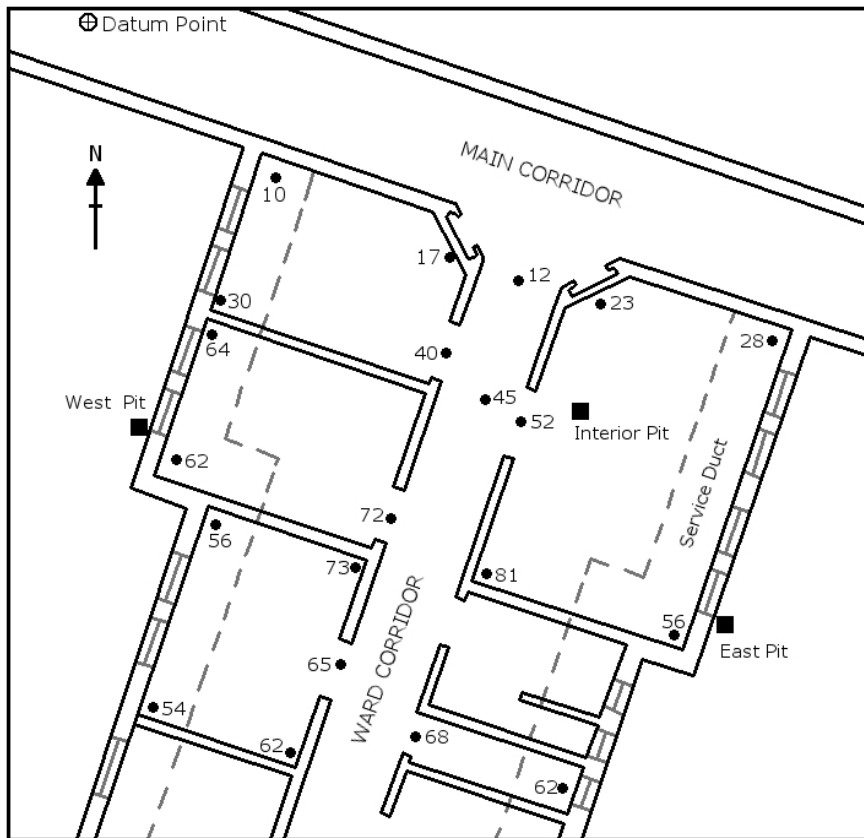


Figure 3.1: Location of test pits and height of floor slab above datum at Llandough Hospital (after Hawkins and Pinches, 1987a).

The local geology is predominantly comprised of Triassic age calcareous mudstones of the Westbury formation, and it is known that the Penarth Fault runs approximately parallel to the main corridor of the hospital. The mudstone is described as being laminated and contains two distinct limestone bands (Hawkins and Pinches, 1987a) that were used for stratigraphic correlation across the site.

Part of the site investigation consisted of a series of trial pits at the site; one within the foundations of the structure and two outside, as shown in Figure 3.1. These trial pits highlighted the difference between the external and internal geology. Material in the interior pit was described as being more oxidised and containing gypsum along both bedding planes and joint discontinuities (Hawkins and Pinches, 1987a, 1992).

Sulphate analyses were carried out on samples from the trial pits, which showed a distinct difference between the material internally and externally. The material from the upper section of the internal trial pit was considerably more acidic than that sampled externally (pH values around 3.6 – 3.75 from the internal pit, compared to 7.10 from the external

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pit at equal depths) and had higher  $\text{SO}_3$  values, as shown in Figure 3.2.

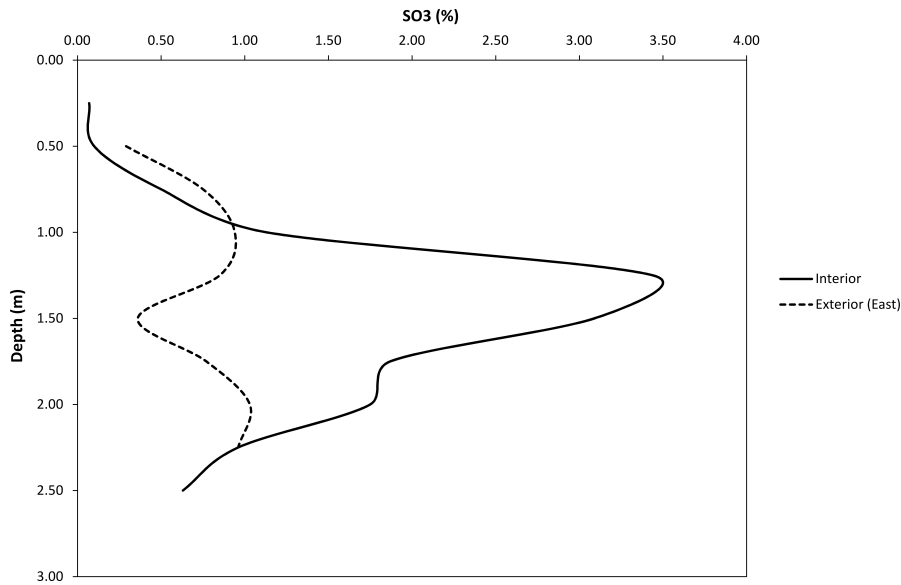


Figure 3.2: Changes in the  $\text{SO}_3$  values with depth in the internal (blue) and external East (red) trial pits at Llandough Hospital (after Hawkins and Pinches, 1987a, 1992).

The increase in  $\text{SO}_3$  values in the upper section of the internal pit corresponds to the oxidation of the material in this area, with material below 2.5 m being un-oxidised and showing pH and  $\text{SO}_3$  values comparative to those at a depth of 1.5 m in the external pits.

During the initial investigations, three theories were proposed regarding the cause of the heave (after Hawkins and Pinches, 1987a):

1. The presence of expansive clay minerals
  - XRD analysis confirmed that smectite and illite (clay minerals most often responsible for such damage) comprised approximately 1% of the total clay mineralogy at the site. Therefore, it was considered that any volume changes from such a small amount of swelling clay minerals would not be enough to have caused the differential heave seen at the site.
2. Hydration of the mudstones
  - Moisture contents at the site were reasonably high. The internal pit showed a higher moisture content at shallow depths than the external pits, believed to correspond to the layer in which gypsum was present (0.75-1.50 m), with water present in the voids between crystals. The external pits were believed

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to have been affected by a prolonged period of dry weather. Natural ground water level was recorded at 3.2 m below ground level at the site.

#### 3. Volume change due to mineralogical changes

- In this instance any volumetric changes would be due to the oxidation of pyrite and the precipitation of gypsum. Hawkins & Pinches (1987a) cite a maximum possible heave of 100 mm due to the replacement of pyrite ( $G_s$  4.8-5.1) and calcite ( $G_s$  2.7) with less dense minerals such as gypsum ( $G_s$  2.3) and jarosite ( $G_s$  3.2). It is not stated what assumptions were made to calculate this maximal heave, although it would be likely that they assumed full replacement of all minerals.

From evidence obtained in the site investigations and subsequent laboratory testing, it was determined that formation of gypsum was exerting the most uplift pressure. Crystals up to 1 mm thick were found on bedding planes and laminations, and it was reasoned that crystal growth in these locations would cause expansion of the material (Hawkins and Pinches, 1987a).

Hawkins & Pinches (1987a) present the following as the most likely series of events that led to heave at the site:

- Removal of overburden at the site lead to relaxation of stress along bedding planes and lamination. This in turn allowed access of weathering agents into the mudstone, in particular allowing oxygen and water to react with pyrite within the material.
- Alteration of pyrite lead to production of iron sulphates and sulphuric acid.
- Other minerals present in the system (particularly calcite) react upon contact with the sulphuric acid.
- Change in pore fluid chemistry leading to eventual precipitation of crystals out of solution into weak zones such as laminations and large pore spaces.
- Continued growth of crystals that results in gradual opening of discontinuities. This increases pressure and void space within the system, the latter allowing further ingress of weathering agents and movement of pore fluid through the system.

Remedial works at this site included underpinning of the external walls, creation of an air gap and secondary brick wall around all service ducts to prevent heat transmission and installation of units to separated the bedrock from the floor slab (Hawkins and Pinches, 1987a).

#### 3.1.2 Domestic Properties, Teeside

Damage was noted around 1970 to a series of domestic properties in the Teeside area; by 1975 this damage was much more widespread and significant in nature. The damage

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included lifting of the floor slab and interior walls and bulging of the external walls (Nixon, 1978).

Fill material at the affected properties fell into two main lithologies:

1. Cleveland Ironstone Shale
  - Waste material from an ironstone mine
  - Approximately 50% of this material was described as hard, the remainder as soft and laminated
2. Cleveland Shale
  - Quarried for specific use as fill
  - Generally soft and laminated
  - Not frost resistant, identified as likely to be subject to sulphate attack if used in damp conditions
  - Split along laminations when subjected to wetting/drying cycles in laboratory tests; gypsum and jarosite deposited between laminations

Although both materials were in a weathered condition, analysis confirmed the quarried material of lithology 2 to be responsible for the damage. Large amounts of gypsum and small amounts of jarosite were found both in the material taken from beneath the properties and in comparison material taken direct from the quarry. Test results confirmed the cause of the heave to be oxidation of pyrite in the quarried shale and subsequent precipitation of gypsum. The increase in volume from the disseminated pyrite grains and framboids to the platelets and rosettes of gypsum was estimated to be between 1 and 2% (Nixon, 1978).

It should also be noted that evidence of sulphate attack was found at this site, with small amounts of ettringite detected on a concrete floor slab that had undergone heave. At the time of testing, no deterioration of the concrete had occurred, despite the weight percentage of  $\text{SO}_3$  in the material being higher than the BRE recommended limit of 0.5%.

Nixon (1978) cites potential volume increases from pyrite to jarosite calculated to be as high as 115% and pyrite to ferric sulphate as 170%, with this increase in volume and the expansive pressure of crystal growth between laminations being the driving force behind the expansion process.

Remediation options suggested by Canadian workers and implemented at some sites (as discussed in Section 3.2.1) included keeping the water table high enough to exclude air from the shale and removing the shale completely. The first option is only viable at certain sites, may lead to problems such as penetration of damp into buildings, and is costly and difficult to maintain. Although expensive, removal of the pyritic material is currently the best remedy due to its success rate.

Nixon (1978) highlights the fact that previously all concerns regarding the amount of sulphur present related to the potential for sulphate attack, rather than that of the expansion process. There are two main factors that indicate a potentially expansive material:

- Material containing an ‘appreciable’ amount of finely divided pyrite
- Material containing a ‘sufficient’ amount of calcite to be able to form gypsum

Reports stemming from Canadian cases of pyritic heave report rocks having pyrite contents as low as 0.1 percent by weight. The rocks beneath the Bell Canada building contained 8.2% calcite and 4.25% pyrite (Penner et al., 1970, 1973). The shales used at these sites have average calcite content of 1.49% and between 3.14 - 8.76% pyrite (average 5.4%) (Nixon, 1978). This supports the idea that the amount of pyrite is not always a direct indicator of the amount of damage that will occur, as there are other factors that will influence both the reaction process and the manifestation of the heave.

Nixon (1978) advises carefully studying and analysing any rocks that may pose a problem before use. A combination of studying hand specimens, chemical analysis and XRD would be able to identify a problem rock material.

#### 3.1.3 Barry, Glamorgan

Constructed in 1906, the Old Town Hall in Barry is located on the site of an old, partially back-filled quarry within Lower Jurassic age rocks.

The quarry was in use prior to 1906 to excavate limestones of the Blue Lias which were interbedded at the site with black, shaley mudstones. Site investigations revealed that 50 – 70% of the material at the site was weak to moderately strong mudstone. Waste mudstone from the excavation process was also used as fill at the site after the active life of the quarry. Weathering of the mudstones was determined to be shallow, with fresh, unweathered material found below levels of 2.1 to 3.5 m below ground level across the site (Wilson, 1987).

Damage to the structure was determined to be caused by heave of the basement floor, which was constructed, along with the foundations, directly onto the mudstones of the quarry floor. Damage was enhanced by the poor construction quality of the floor slabs, with total slab thickness in some areas barely reaching 100 mm including the screed that was added later in an attempt to level and repair the floor.

Although it was determined that there was not enough heave to affect operation of the building, visual inspections in 1981 suggested a maximum heave of 20-50 mm in the centre of the largest floor slab span, which was around 7 m. Trial pits confirmed that more heave was seen in areas where floor slab construction was poor. This effect of poor construction upon the amount of damage caused in properties suffering from pyritic heave can also be seen in some of the Irish cases currently under consideration

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(Chapter 5). Also that in some areas the material was bound together with a clay matrix, these areas showed less precipitation of gypsum and subsequently less heave of the floor slab (Wilson, 1987).

In this case, it is supposed that where the floor slab was thinner and poorly constructed, there would have been less restraint upon the pyritic material, and therefore the expansion was shown more easily in uplift of the slab.

This effect of poor construction enhancing the damage caused by pyritic expansion is also seen in Ireland. However, the poor construction there tends to be shown in cases where, for example, instead of sitting on a foundation wall, an internal wall will be offset such that it rests partially on the floor slab. In that case, any uplift of the slab in that area is directly translated into movement of the wall. This is discussed in Section 2.2 and Figure 2.2.

Based on work by Nixon (1978), discussed in Section 3.1.2 chemical analyses were carried out on rock samples taken from different depths within the boreholes, as indicated in table 3.1.

<b>Borehole no.</b>	<b>2</b>	<b>2</b>	<b>5</b>	<b>5</b>
<b>Depth (m)</b>	3.00	4.50	4.00	5.00
<b>Total sulphur as SO<sub>3</sub> (%)</b>	0.38	0.79	1.20	1.32
<b>Total (acid soluble) sulphate as SO<sub>3</sub> (%)</b>	0.06	0.09	0.11	0.13
<b>Total (acid soluble) CaO (%)</b>	18.5	18.6	18.9	11.3
<b>pH</b>	8.15	8.25	7.9	7.7

Table 3.1: Chemical analyses of four drill core samples from boreholes around the quarry (after Wilson, 1987)

Nixon (1978) defined ‘troublesome’ mudstones as those containing more than 0.5% calcium and a total sulphur content greater than the acid soluble sulphur content. Based on these definitions, the quarry material can be easily classified as showing the potential for pyritic heave.

Due to the absence of a void beneath the floor slab, it was concluded that the heave was due to expansion of the fill material, rather than concrete expansion due to sulphate attack. This was worse in locations of both poor slab construction and a large floor span. Where the slab was better constructed and the floor span smaller, the slab resisted the heave leading to an increase in the density of the fill with negligible associated floor heave.

## 3.2 Canada

### 3.2.1 Bell Canada Building, Ottawa

The Bell Canada Building in Ottawa, initially constructed in 1929, was founded on Ordovician age shales of the Billings Formation, known to be rich in pyrite and calcite. A large extension was added to the original building in 1961, this was also founded on the same material and heave was first noticed in the basement floor of this extension around 4 years after construction. The area affected by heave was around 225 m<sup>2</sup> and investigations into the problem began in 1966 (Penner et al., 1970, 1973).

The floor slab within the extension to the building is 300 mm thick reinforced concrete on a 150 mm layer of crushed limestone containing a drainage system. The limestone was placed directly onto the bedrock and up to 2.7 m of shale material was removed in parts of the site, with the floor slab sitting approximately 1.5 m below the original ground level.

The rocks beneath the building were described during investigations as black, fissile shale, rich in pyrite and calcite. In the area of the Bell Canada Building, the Billings Formation is around 6 m thick and there are known to be two major and numerous minor faults in this area that may have contributed to the fissility and weathering of the rocks (Penner et al., 1970).

Precise level surveys were carried out in April 1967, revealing two areas of pronounced uplift within the slab. The most pronounced heave was seen in the area designated Area A, which heaved 51.8 mm within the measurement period (a rate of 1.8 mm/month) giving a total heave since construction of 94.5 mm. A second area, designated Area B, showed less uplift, with a total heave since construction of 55 mm, with a rate of heave during the monitoring period of 0.5 mm/month (Penner et al., 1970).

In addition to the survey of the floor slab, ground investigations were also carried out at the site to determine what, if any, changes had taken place to the rocks below the floor slab. These investigations determined that two distinct changes in stratigraphy occurred at the site (Penner et al., 1970, 1973).

- **Altered Zone – 0.76 to 0.91 metres below surface level:** The rock within this zone showed distinct horizontal layering, was soft, friable, damp and merged gradually downwards into the intact rock described as the second lithology below. Laminations ranged in thickness from a few millimetres in material close to the surface to several centimetres in material at the centre of this zone. Lamination surfaces were noted to be coated with yellowish-brown powder and colourless crystals (later confirmed by XRD analysis to be jarosite and gypsum respectively), and the rock was noted to contain less pyrite and carbonate than the unaltered material below (Penner et al., 1970, 1973).
- **Unaltered Zone – more than 0.91 metres below surface level.** As with the

### 3 Case Studies – Examples of Pyritic Heave Worldwide

altered zone above, this layer showed distinct laminations although they were much thicker, up to several centimetres. The rock below this level was much stronger, requiring mechanical methods to break up the rock for excavation. This material had a pyrite content of 1.3 – 1.6%, with little sulphur present as sulphate, and showed little to no evidence of alteration (Penner et al., 1970).

Although the position of the local water table at the time of this investigation is not specified, the difference between the altered and unaltered zones described above highlights the variation that is shown by these mudrocks both with depth and level of weathering.

The pH in the altered zone ranged between 2.8 – 4.4, while in the unaltered zone it was in excess of 7. The drop in pH in the altered zone is due to the production of acid as a result of the pyrite oxidation process and provides a suitable environment for bacteria of the *Ferrobacillus* and *Thiobacillus* genera that catalyse the reaction process (Section 2.2).

Penner et al (1970) proposed a series of factors that contributed to the heave seen at this site:

1. The water table at the site appears to have dropped since construction of the building. Air entry is facilitated by underfloor drainage and fracture sets.
2. The basement is warm (c. 30°C) due to plant machinery which enhances bacterial activity. Optimum growth temperature for the bacteria is around 35°C.
3. The shale was rich in pyrite supplying minerals for the alteration process.
4. Molar volumes of the initial minerals (pyrite and calcite) are less than those of the precipitated secondary minerals (gypsum and jarosite), meaning that the secondary minerals occupy more space within the system.

#### *Remedial Treatment*

Remediation at this site involved creating unfavourable conditions within the rock beneath the structure so as to limit bacterial activity and therefore reduce the rate of the reaction process. In this case it was decided to neutralise the acidic conditions by introducing an alkaline solution to the rock. The alkaline selected was potassium hydroxide in solution in was added to the groundwater beneath the site.

Pumping of the alkaline solution started in January 1970 and ran through to May 1971, by which time more than 12 tonnes of potassium hydroxide had been used and approximately 16,000 litres per day of water were required to maintain the new local water level and had led to a small amount of settlement of the structure (Penner et al., 1973; Hawkins and Pinches, 1987a). The pH levels at the time of the initial investigation were around 3, however, 2 years later, after the treatment, the local pH had returned to between 6.4 – 7.1 (Penner et al., 1973).



### 3.2.2 Rideau Health Centre, Ottawa

The Rideau Health Centre is also located in Ottawa, and over a 20 year period was recorded to have undergone up to 76 mm of heave. The health centre is a two-storey building, with some sections underlain by basement structures and the remainder founded directly onto the shale bedrock (Quigley and Vogan, 1970).

The portion of the structure that had shown the most heave was the two storey part of the structure founded directly onto the bedrock. The foundations for this part of the structure include poured concrete service ducts that contain various services and support the external masonry walls. The service ducts are approximately 1.5 m wide by 2.1 m deep and are constructed within a trench in the shale bedrock. The bedrock material between these foundation service ducts was not removed and instead forms a plug of shale material beneath the floor slab.

The service ducts that comprise the foundations contain heating pipes and electrical cables and during the winter months the temperature within these ducts can reach close to 30°C (Quigley and Vogan, 1970; Hawkins and Pinches, 1992). This in turn is translated to the rock plug between the service ducts and is combined with the heat that is conducted through the concrete floor slab from the heated ground floor of the structure. During the summer months, the rock is believed to be much closer to the ambient temperature of the surrounding bedrock. The heat introduced to the system creates a thermal gradient within the plug of shale beneath the floor slab, which in turn creates a zone of permanent capillary rise above the water table. This creates movement of solution through the rock in this area (Quigley and Vogan, 1970).

The floor slab was also poured concrete, separated from the plug of bedrock by around 46 mm of granular material. It can be noted that this design is similar to that of Llandough Hospital mentioned in Section 3.1.1 above, and is detailed in Figure 3.3 below.

Damage to the structure centered around the heave of the floor slab. There was a long, continuous fracture through the slab that was determined to correspond roughly with the edge of the shale plug, and the floor outside this fracture was sloped towards the external walls. There was also damage to the interior walls and upward bowing of the first storey floor caused by upward movement of the interior columns that were founded on the shale plug. The average rate of heave was calculated to be about 2.5 mm/yr and is thought to have been more or less constant during the period heave was occurring (Quigley and Vogan, 1970).

The building is founded on Ordovician age shales of the Lorraine and Billings Formations, which are generally medium to dark grey in colour, with distinct lamination and fissility along bedding planes (Quigley and Vogan, 1970). Of particular note is the groundwater level in this area. Drains are located alongside the foundations, meaning that the local ground water level is maintained at around 3 m below ground floor level. Therefore, the shale bedrock beneath the basement areas is fully saturated, whilst the plug of bedrock beneath the floor slab in the two storey portion of the building is located within the

### 3 Case Studies – Examples of Pyritic Heave Worldwide

zone of capillary rise above the water table, as indicated in Figure 3.3.

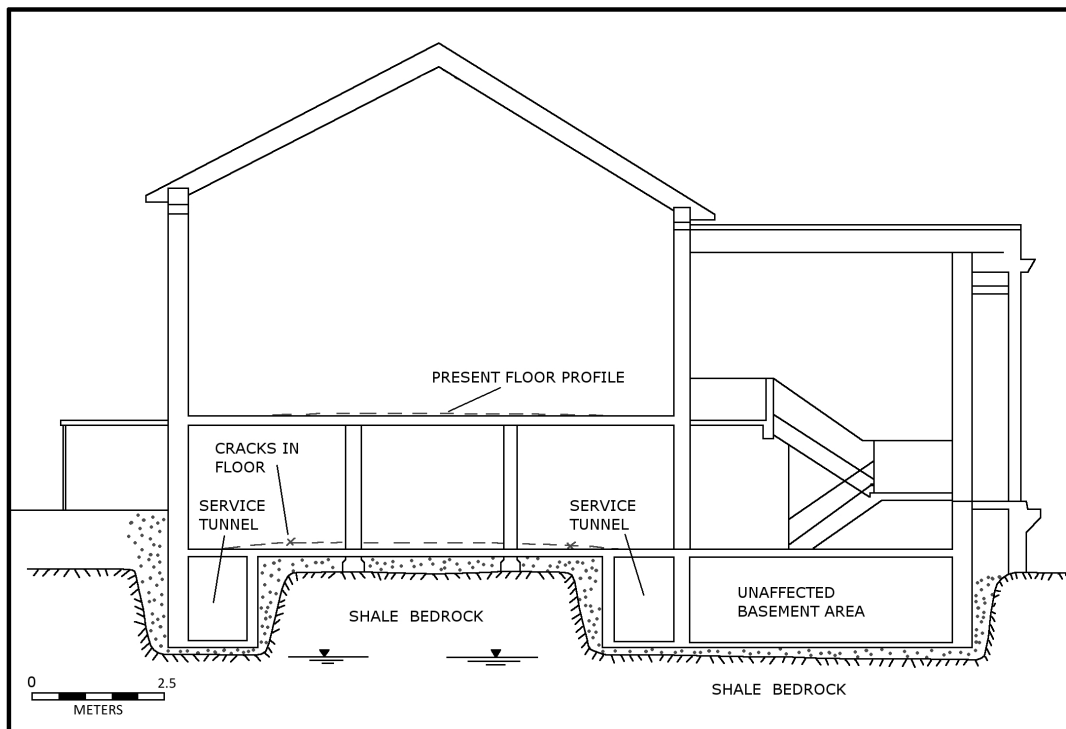


Figure 3.3: Cross section of the Rideau Health Centre (after Quigley and Vogan, 1970).

Boreholes drilled through the plug of rock beneath the floor slab during site investigations revealed the following stratigraphy beneath the site:

- 0 – 0.3 m: Bands of oxidised, orange-coloured shale with soil-like material between bedding planes.
- 0.3 – 0.9 m: Weathered, orange-stained material. Jointed with occasional poor recovery, gypsum crystals present along joints.
- 0.9 – 1.5 m depth: Poor core recovery due to the jointed nature of the material, gypsum crystals were present along joint surfaces in what material was recovered.
- 1.5 – 2.7 m: Rock showed a distinctive wavy bedding style; bedding planes occasionally propped open by small gypsum crystals.
- Below 2.7 m: Rock sampled from below the local water table. Core below this depth was intact and structurally sound.

It should be noted that it is likely that more gypsum was present in the ground than was

### 3 Case Studies – Examples of Pyritic Heave Worldwide

recovered in these cores, as the material was subject to “torque, grinding and abundant wash water” as part of the drilling process (Quigley and Vogan, 1970).

Quigley & Vogan (1970) present two possible mechanisms to explain the heave seen at this site:

1. Hydration and expansion of swelling clay minerals
2. Geochemical alteration of sulphides to produce secondary sulphates and subsequent heave from pressures of crystallisation

Testing was carried out on clay minerals known to be present in the rocks of the Lorraine and Billings Formations, and it was determined that the maximum heave that could be induced by these minerals was around 25 mm compared to the 76 mm of heave seen at the site. It was determined that although this process may have contributed to the heave, it could not be the principal mechanism responsible and so mechanism two was considered in further detail (Quigley and Vogan, 1970).

The environment above the water table, i.e. within the shale plug, was warm and humid due to the capillary rise and heat from the service ducts. This kind of environment is ideal for the growth of aerobic bacteria, which are known to catalyse the oxidation reactions of minerals such as pyrite. The dark shales found at the location are typical of organic shales known to contain pyrite in fine, disseminated forms, which are more readily susceptible to oxidation and subsequent chemical alteration of the rock system. The amount of precipitated gypsum propping apart the bedding planes within the shale plug was calculated to correspond roughly to the amount of heave recorded. Also, this gypsum only occurred above the local water table in the partially saturated zone of capillary rise (Quigley and Vogan, 1970).

It is implied that, for this method of heave to be accurate, the gypsum would have to exert a crystallisation pressure equivalent to the weight of the rock above it. In this case it is approximately a 3 m thickness of rock, which correlates to an implied crystallisation pressure of 0.75 kg/cm<sup>2</sup>, which is equivalent to approximately 74 kPa. It is not stated whether this calculation includes the weight of the slab and structure (Quigley and Vogan, 1970).

#### 3.2.3 St Luke’s Church, Ottawa

St Luke’s Church in the New Edinburgh area of Ottawa was built in 1913 and was known to have required repairs to the basement floor some 15 to 20 years after construction. Further investigation of continued damage to the basement floor was carried out in 1974 and implied up to 63 mm of heave had occurred at that time. These investigations also revealed that the initial repairs had consisted of placing a second layer of concrete directly over the original basement floor.

The stratigraphy through the basement floor was detailed as follows.

### 3 Case Studies – Examples of Pyritic Heave Worldwide

Minimum thickness (m)	Maximum thickness (m)	Detail recorded
–	–	Floor level
0.19	0.19	Layer of bitumen
0.57	1.19	Layer of ‘good’ concrete
1.84	2.46	Approximate thickness of original concrete
1.84 +	2.46 +	Weathered shale bedrock

Table 3.2: Floor layers discovered during investigations at St Luke’s church, Ottawa (after Grattan-Bellew and Eden, 1975) .

The original concrete was described during the investigations as having a ‘mushy’ consistency, and the ‘good’ concrete was of an uneven thickness, confirming assumptions that the initial repairs were required because of heave of the basement floor (Grattan-Bellew and Eden, 1975).

The geology in this area of Ottawa is comprised of the black, carbonaceous shales of the Eastview Formation, which contain up to 4.25% pyrite and 8.2% calcium carbonate. The material below the basement floor slab was weathered to a depth of 0.4 m, with the uppermost section described as being heavily weathered. Abundant gypsum was present along joints and bedding planes within the unweathered material found around 1 metre below floor level (Grattan-Bellew and Eden, 1975).

Pyrite present at this site was found to occur in three distinct forms:

1. Octahedra and cubes around 8–10  $\mu\text{m}$  in size
2. Circular ‘bodies’ 10–20  $\mu\text{m}$  in diameter. These are described as irregular circular masses under SEM, not framboids as suggested by optical microscopy (Grattan-Bellew and Eden, 1975)
3. Fine grained pyrite replacing fossils

Similarly, there were two main forms of gypsum identified at this site:

1. Fibrous bundles of needle shaped (acicular) crystals found growing perpendicular to laminations within the shale in Flat, blade-like crystals found growing parallel to laminations within the shale

The acicular crystals were also often associated with small amounts of jarosite and other sulphates. However, their relative scarcity compared to the amounts of gypsum present suggest that the majority of the heave is due to the presence of gypsum, particularly those crystals precipitated perpendicular to the laminations.

The crystallisation process suggested by Grattan-Bellew & Eden (1975) is summarised below and discussed in more detail in Chapter 2.

### 3 Case Studies – Examples of Pyritic Heave Worldwide

- Due to overburden pressure within the system, crystallisation is restricted to a narrow rim at the base of the system, i.e. along a lamination or joint.
- Addition of further material is limited to this small area, and so growth of the crystal proceeds upwards to form a needle-like morphology.
- Capillary action moves solution through the shale, bringing new material into the area where the crystal is forming. Further upwards growth of the crystal slowly forces apart the laminations.
- Although initial heave is localised to areas of crystal growth, as the cracks within the shale develop and grow, the pressure in these layers is reduced. This leads to a pressure gradient within the system, inducing further movement of solution towards cracked zones.

Examination of the oldest layer of concrete revealed that the cement had largely been removed, leaving large voids between the aggregate particles. This is believed in this case to be due to acid attack on the concrete (Grattan-Bellew and Eden, 1975), although it is not stated whether minerals such as thaumasite or ettringite were found at this site. Small amounts of gypsum were found within the concrete; however, this comprised platy, tabular crystals unlike those found to have induced heave in the slate.

#### 3.2.4 Domestic Properties, Montreal

As many as 10,000 houses are thought to be affected by pyrite-bearing rock fill in the Montreal south-shore area. Damage to these properties varies but mostly consists of heaving of the floor slab and/or sulphate attack to the concrete of the slab. In this instance, Ballivy et al. (2002) report upon damage recorded to 224 of these domestic properties and consider several case studies that show the range of damage seen in these structures.

In all cases, damage was determined to be due to expansion of the fill material caused by oxidation of the pyrite. Additionally, the fill material beneath all properties included in the studies was the same shaley fill, with some exceptions beneath the garage bases as discussed below. This type of material is particularly susceptible to swelling due to factors such as weak mechanical properties, the presence of clay minerals and the presence of laminations and pre-existing cracks (Ballivy et al., 2002).

At each of the 224 properties included in the study, a series of investigations were carried out to gather information including property age and type, and a visual survey of all damage to the property. These surveys were based upon recommendations put forward by the Quebec Technical Committee for Pyritic Heave (CTQ-M200, 2001), and the classifications developed are summarised in Table 3.3.

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Criteria	High	Medium	Low
Maximal heaving	> 5 mm/m	1-5 mm/m	0-5 mm/m
Cracking:			
No. of star cracks	>5	1-2	0-2
Crack opening	>2 mm	from 1-2 mm	<1 mm
Cracking pattern	Ramified network	Single crack	Single crack
White powder present *	Sparse – Abundant	Sparse	Undetected
Foundation wall:			
Presence of cracking	Yes	No	No
Displacement	Yes	No	No

Table 3.3: Description of the classification criteria to give low/medium/high probability that damage is caused by pyrite oxidation. \* refers to the presence of gypsum and thenardite at the surface and within cracks (after Ballivy et al., 2002).

Ballivy et al. (2002) do emphasise that in many cases, especially those properties showing only slight damage, interpretation of the symptoms of expansion is largely subjective. However, the identifying factors highlighted in Table 3.3 can be placed in decreasing order of importance as follows: heave of slab, cracking and lateral displacement of the garage foundation wall, cracking to floor slab, presence of whiteish powder on the slab.

The classification for each property was based upon the severity of the criteria summarised in Table 3.3, and are expanded upon as follows:

- Maximum relative heave of the floor slab, as measured with a 2 m long level and expressed as linear millimetres.
- The crack pattern and size of crack openings:
  - Crack opening – indicates the stress induced by expansion of the fill and/or sulphate attack.
  - Crack pattern – indicates the cause of the cracks. Those formed by drying/shrinkage are generally less abundant and regularly spaced, whilst those formed by pyritic heave often join at a central point to form a star.
- The presence of whitish powder that is a blend of secondary minerals along cracks (also atop the slab where there is a very strong thermal gradient). This material is often removed by homeowners when cleaning the area.
- Both cracking *and* lateral displacement of the garage foundation walls must be present to conclude that there is pyritic expansion of the fill.

For the latter point, it should be noted that the lateral displacement of the garage foundation walls (often signified by cracking where two walls meet) is due to the increase in pressure against the foundation walls by expansion of the fill. Similar effects may not be seen in basement foundation walls if the layer of fill is too thin to generate high

### 3 Case Studies – Examples of Pyritic Heave Worldwide

enough pressures, or if the position of the fill relative to the footings restricts its ability to induce pressure upon the walls. For this reason, cracking or displacement of the basement foundation walls was not included in the study criteria (Ballivy et al., 2002).

Additionally, the damage seen with pyritic heave is progressive – as long as the material is present and available to react, it will do so. The only thing that will end this reaction process is a change in environment – for example, an absence of air and/or moisture, or the consumption of all pyrite present. If the damage were due to settlement, it would be expected to stop within a much shorter time-frame than that of the pyrite reaction, and shrink-swell changes would show a more pronounced cyclicality.

Based upon the above criteria, it was determined that the number of garages that indicated a high probability of damage due to pyrite oxidation was more than double that of basements – around 78% of garages compared to around 32% of basements. Ballivy et al. (2002) proposed two factors that contribute to these statistics.

1. The thickness of the fill material beneath the garage floor slabs is generally much larger than those beneath basement floor slabs, and a theoretical vertical expansion of 1% will be more readily noticeable on 1.5 m thick layer of fill than it would on one only 0.2 m thick.
2. Although the fill material used beneath the basement floor slabs was subjected to quality control before use, no such system was in place for the material used beneath garage floor slabs. The material beneath the basement floor slabs was described as a ‘clean gravel without fine particles’, whilst the material beneath the garage floor slabs was much more varied, consisting of different and well graded material often giving poor quality fill.

From the study it was determined that damage had arisen from heave of the floor slab due to expansion of the fill, sulphate attack of the slab due to sulphates generated by reactions within the fill, or a combination of these two processes (Ballivy et al., 2002). It is implied that the likelihood of a particular method occurring at a given property is largely dependent on the aggregate material used for the fill.

As stated above, the garages that were a part of the study showed a higher probability of damage due to pyrite oxidation; the largest amounts of heave recorded over the study were in garage areas, with a maximum heave in a single room of 75 mm and a general maximum of 25 mm.

Ballivy et al. (2002) propose the following reasons that more heave is seen in the garage floor slabs than in the affected basement floor slabs:

- The layer of fill beneath the majority of garage floor slabs was thicker and of poorer quality than that beneath the basement floor slabs, as detailed above.
- The local water table was much closer to the basement floor slabs than to the garage floor slabs, this would have 2 principle effects:

### 3 Case Studies – Examples of Pyritic Heave Worldwide

- A shorter distance for sulphates to travel to reach the basement floor slab - this would explain why there were a larger number of basement slabs suffering from sulphate attack
- Any fill material below the water table will be less likely to react, whereas capillary rise in material above the water table will aid the reaction process.
- The thermal gradient is greater below the basement floor slabs, due to basements being heated during colder months while garages are not. This gradient speeds the movement of sulphates in solution towards the basement floor slabs and contributes to the increased instances of sulphate attack.

Out of all the properties that were surveyed as part of the original study, five were selected for further study and monitoring, and are summarised below. The properties were all of different ages and were selected because all damage observed to the properties was believed to be due to reactions within the fill material. The reaction was believed to be active at three of the properties at the time of investigation, and instrumentation at all five properties included tell-tales and specifically designed ‘fissure-loggers’, which measured both ambient temperature and the lateral movement of individual cracks.

*Property 1:* Property was 18 years old at the time of inspection and installation of the logging equipment. Two cracks were measured; the first (garage floor slab) showed little movement over the monitoring period, the second (garage foundation wall) showed relative outwards movement of around 1 mm over 2 years.

*Property 2:* Property was 19 years old at time of installation of monitoring equipment, the slab had been previously repaired. Repairs included removal of a thin layer of fill, partial removal and replacement of the slab, addition of reinforcement to foundation and chimney walls; the house floor slab is now independent of the garage. Over the two years that the instrumentation was in place the crack width did not vary much, with a total of 0.5 mm growth recorded; this is believed to be due to stabilisation of the slab from the added reinforcement.

*Property 3:* The property was 4 years old at the time of installation of the monitoring equipment. Laboratory analysis of the fill material showed that it was comprised entirely of soft, friable, dark-coloured shale. Readings from the tell-tale did not show movement of the crack in the first few months of observations, however, at this point the home owner stopped taking readings. During the time period of around 7 months, the logger recorded an upward movement of the slab of 7.2 mm, which was the limit of the logger, giving a rate of uplift of 1.2 mm/month for this time period (Aug - Feb). The instrumentation was re-installed after this time and a second monitoring period (Jul - Nov) gave an uplift rate of 0.9 mm/month. It should be noted that the uplift continued despite temperature fluctuations, implying that expansion of the fill is to blame, although it should be noted that the logger recorded ambient temperature not that within the fill.

*Property 4:* Property was 6 years old at time of installation of monitoring equipment, and the rockfill consisted completely of soft, friable, dark shale, with a trace amounts



of intrusive rocks. The crack that was monitored opened at a relatively constant rate, despite temperature fluctuations, to reach a width of 5.25 mm. The data from this property showed that the reaction rate was largely constant despite large temperature variations in the garage, this implies that the temperature and humidity conditions in the fill are fairly constant and independent of changes in the property.

*Property 5:* This property was 7 years old at the beginning of the monitoring period and showed the most damage of the 5 properties - damage was noted as first appearing within 2 years of construction. Only 36% of the fill beneath this property was the friable black shale seen at other properties, with the remainder comprising hard syenite and metamorphic rocks. There was no indication of sulphate attack to the floor slab of this property. Four instruments were installed at this property; the crack width was noted in some monitored cases to vary with temperature changes, with up to 0.75 mm total opening noted after 1 year. Expansion pressures were high enough to push apart the foundation walls, with an exterior crack at the corner between two walls widening by 3 mm over the recording period. Uplift of the slab was around 2.3 mm and outward movement of the foundation walls up to 2.2 mm over the monitoring period. This property did show temperature variations, with a decrease in movement in the winter and a slight increase in the summer.

#### 3.2.5 Sainte-Foy, Quebec

Berube et al. (1986) present the example of a college in Sainte-Foy, Quebec, and in particular two buildings that were constructed in 1971 and showed significant amounts of damage in the time period before investigations began in 1983.

These investigations recorded a maximum heave in one of the buildings (listed as Building J) of around 100 mm, and recorded damage to the structure including cracking of the floor slab, ‘dislocation’ of concrete blocks that formed a partition wall in the basement area and damage to walls on the upper floor. Heave was noted to be ongoing at the time of the survey and an initial estimate gave heave rates of more than 10mm/year (Berube et al., 1986).

Boreholes drilled into the ground below Building J found 40 mm of sand blinding separating the floor slab from the natural bedrock, which was described as a highly laminated shale that contained large amounts of “white to rusty secondary minerals along the laminations” (Berube et al., 1986). These fissures were noted to be parallel to bedding in the shale and whilst the upper section was highly weathered, the weathering state decreased rapidly around the level of the local water table.

The other building on the site that showed signs of damage due to heave is listed as Building B, and here the amount of movement of the floor slab that was recorded was over 200 mm and was noted to have caused damage to service pipes and supporting rods. The rock material encountered in boreholes beneath this building was similar to that

### 3 Case Studies – Examples of Pyritic Heave Worldwide

under Building J, although heating in the building meant that humidity and temperature at this site was much higher than below Building J.

Chemical testing on rock material from the Sainte-Foy site revealed framboidal pyrite concentrations between 1–5%, with an average of 2%, and also the presence of gypsum and jarosite. The gypsum occurred primarily as acicular crystals, parallel to laminations and fractures, and often forming rose-shaped aggregates. Similarly the jarosite formed rosettes of crystals, although they were more commonly found between the gypsum crystals (Berube et al., 1986).

Berube et al. (1986) cite several estimates of the pressure induced in the rock system by crystallisation of gypsum ranging from a minimum value of 28 kPa to a maximum of 500 kPa, including the value calculated by Quigley and Vogan (1970) of 72 kPa. This is a minimum estimate of the pressure induced in the system, based upon damage seen at the Rideau Health Centre (section 3.2.2) and determined to be enough pressure to cause a 3 m rock column to heave by up to 76 mm (Quigley and Vogan, 1970).

Although there was some movement of partition walls within the basement area, there was no movement of the main structure recorded at this site. This implies that whatever the heaving force is, it does not exceed the gravitational forces acting on the main pillars and external walls. This increased pressure within the system may also explain why sulphate crystals were present under the floor slab but not under the footings: sulphates are more soluble at high pressures (Berube et al., 1986).

Unlike other Canadian case studies presented herein (section 3.2), fibroferrite ( $\text{FeSO}_4(\text{OH}) \cdot 5\text{H}_2\text{O} \pm \text{Al}$ ) was found at the Sainte-Foy site. This mineral has similar habits to gypsum and is considered to have contributed significantly towards heave at the site. As fibroferrite does not contain calcium, it is inferred that primary calcite and secondary gypsum may not have been as important in the heave process as they were at other sites (Berube et al., 1986).

Penner et al (1970) give average heave rates in the Ottawa area as not exceeding 20 mm/yr. At the Sainte-Foy site, maximum heave in excess of 200 mm was noted between 1971 and 1983; since heave does not usually begin until the building is a few years old (allowing the pH to drop, bacteria to form and precipitation of secondary minerals to begin) heaving rates at this site are also believed to be no greater than 20 mm/yr. However, considering the movement in building J between the time of survey and publication of the paper, heave was thought at that time to be progressing at a rate of more than 10 mm/yr.

Previously suggested corrective measures for pyritic heave that are highlighted by Penner et al (1970) include:

- Penner et al (1970) - application of bactericides.
- Penner et al (1973) - increasing the pH of ground solutions by adding chemical products, e.g. potassium hydroxide. The aim of this is to minimise oxidation of

### 3 Case Studies – Examples of Pyritic Heave Worldwide

pyrite as well as inhibiting bacterial growth. However, this is not an ideal long term solution.

- Penner et al (1970, 1973) and Quigley and Vogan (1970) both suggest raising the groundwater level to inhibit oxidation, however, this is difficult to achieve and maintain, especially in areas with a low water table.
- For new excavations, and where conditions permit in remedial works, it is often practicable to remove any altered rock and apply a layer of insulating material to the newly exposed surface of fresh material. This reduces any air inflow and subsequent alteration of the rock.
- The most effective method is to avoid building lightly loaded structures on rocks that are at risk from pyritic heave; instead, pillars should be on fresh rock and suitably counterbalanced against heave. It is not specified as to what in these conditions would be considered “suitable”.

At the Sainte-Foy site, Berube et al. (1986) discuss remediation measures had been undertaken or were ongoing at the time of publication of their paper:

- Building B - since there is access to the underfloor area, enough rock was removed to clear the service pipes and it is planned to monitor the area for further heave.
- Building J had not been remediated at that time, but the following two measures had been proposed:
  - Remove all of the altered rock from beneath the floor slab (approximately 1.3 m thick), apply an asphalt material to the fresh rock and fill the excavated space with a low porosity, well-compacted granular material. This measure does not totally rule out the possibility of future heave.
  - Excavate a limited amount of rock and support the floor on a new series of pillars resting on fresh rock, a gap would be left between the floor and ground to allow for heaving between the pillars.

## 3.3 United States

### 3.3.1 Kentucky

Several instances of pyrite related damage have been recorded in East Kentucky, most of them associated with the Chattanooga Shale, also known as the Ohio and New Albany Shale, which is a Devonian age organic shale containing up to 5% pyrite. The rocks of the Chattanooga Shale within the area discussed below are between 30 – 50 metres thick and are generally found to be fissile, silty brown-black shales that contain abundant pyrite. There are also numerous fracture sets in the area, many of which are mapped in close proximity to the affected buildings, that facilitate water movement through the rock

### 3 Case Studies – Examples of Pyritic Heave Worldwide

mass. The formation also contains limestone and other calcareous material (Anderson, 2014).

Anderson (2014) presents a study of the Chattanooga shale and affected sites, with investigations for some beginning in 2000 and others not until 2012. Presented below are details of the damage at each site, the information determined during site investigations and any remediation options that were implemented.

The specific sites discussed below are located at different stratigraphic depths within the shale, and so are located on varying thicknesses of material. For example, the shale thickness beneath the hospital is thinner than that beneath the school (a maximum thickness of 30 m compared to 49 m), this variation in thickness attributed to both localised faulting and variations in original depositional thicknesses (Anderson, 2014).

XRD and XRF were used to determine the composition of the shale beneath the middle school, with analyses indicating that the shale is 60% clay, 30% quartz silt, 5% pyrite nodules and 5% calcite. Sulphur values ranged between 2.95 to 5.47%, although this includes sulphur as sulphate as well as sulphide, as confirmed by the presence of secondary sulphates seen in core samples (Anderson, 2014). Within the core samples taken at the middle school, the bedding planes contained sulphate minerals including jarosite and copiapite.

#### *Estill County Middle School*

Problems were noted to floors, walls and doors shortly after the opening of the Middle School, cosmetic repairs were carried out to cracking along wall and floor finishes, including cracks up to 25 mm wide in some blockwork walls. In addition, significant heave was noted to the gymnasium floor, to such an extent that the movable bleacher seating could not be moved back to its closed position. In 2005 the foundation material beneath the gymnasium was removed and the bedrock outcrop sealed with resin to prevent further heave, similar remedial works were carried out to other parts of the school in 2006 & 2007. It should be noted that the nearby High School suffered no apparent damage of this kind, although it is founded lower within the stratigraphic sequence (Anderson, 2014).

Also recorded at this site was damage to a 250 mm thick concrete retaining wall alongside the car park. The wall was noted to be bulging, developing tension cracks along the face of the wall and many of the drainage holes along the wall were becoming blocked due to precipitation of sulphate minerals (Anderson, 2014).

#### *Carhartt Factory*

Located approximately 1 mile southwest of the Middle School and higher within the stratigraphic column, the Carhartt clothing factory also showed signs of damage due to floor heave, including cracking to walls and floors. At the time of writing, the factory was not damaged enough to impair function, although remediation carried out included trimming doors and adding steel ramps to allow forklift access, at the time of writing,

### 3 Case Studies – Examples of Pyritic Heave Worldwide

more permanent options were being studied (Anderson, 2014).

#### *Clay City Water Treatment Plant*

Damage at this site included heaving and cracking of the floor slab, and cracking of the walls that in turn was threatening power cables and conduits. Similarly heave of the floor slab threatened the useability of water filtration and pumping equipment. Several inches difference was noted between floor levels in the centre of the room and towards the load bearing outer walls (Anderson, 2014). Of interest at this location is that the structure was almost 20 years old when signs of heave were first noticed.

#### *Marcum and Wallace Hospital*

Unlike the other properties mentioned above, the Hospital showed signs of damage within the basement area as well as at the ground floor level, although many cracks seen in basement walls were noted to be adjacent to floor slabs that were showing signs of heave. The basement floor had signs of iron staining and water marks, indicating upwards movement of water through the slab, in addition to seepage through the foundation wall from a nearby runoff area. In the main part of the structure, interior floors and non-load bearing walls showed signs of heave and associated cracking, although external, load-bearing walls did not show signs of similar damage. The internal damage also included bending and breaking of copper pipes, cracking to ceiling plaster and damage to the central heating system (Anderson, 2014).

Primary repairs at this location included installation of water lines, repairs to fixings and finishes, although modification of the drainage system will be necessary to prevent further heave of the structure (Anderson, 2014).

#### *Other incidents*

In 2012, there was a large explosion to a natural gas pipeline in eastern Kentucky that ran through the black shale described above. Anderson (2014) reported that in the last 30 years, there have been 10 major explosions of gas pipelines in Kentucky; approximately 30% of these have occurred to pipelines located within the Chattanooga Shale. Many of the explosions were classified during investigations afterwards to be due to corrosion or seam failure, either of which can be caused or exacerbated by the acidic nature of the surrounding rock and any groundwater present in the area. Although in such areas it is common to protect the pipeline through use of liners or inert material, it is unlikely that these were used when constructing older pipelines, implying that there may be other such explosions in future.

## 3.4 Other Types of Pyrite-Related Damage

### 3.4.1 Damage to embankment dams, Roadford and Carsington

#### *Roadford Dam*

Roadford Dam is an earth embankment dam in west Devon. The structure is 430 m long, a maximum of 41 m high and was constructed between 1987 and 1989. The material obtained for use in the structure was locally obtained mudstones, siltstones and sandstones, to which was added an upstream asphalt membrane. These materials were intended to create a free-draining, homogeneous material that would compact well. However, pyrite was found to be present in the fresh mudstone; pyrite content in the mudstone was around 1.3%, which would give an overall value in the embankment of around 0.6%. The minimal amount of alkaline minerals such as calcite led to pore fluid in the embankment becoming markedly acidic, with a pH range of 3.1 to 6.4 (Reid et al., 2005).

Concern during the planning and site investigation stages was that the pyrite would lead to degradation of the embankment material and pollute local groundwater. Trial embankments were constructed and drainage was monitored, which was discovered to be highly acidic and contain large amounts of sulphates and iron (Reid et al., 2005). Allowances were made in the design for deterioration of the materials, along with provisions for monitoring of drainage from the dam as a continuing process. An altered dolerite was used in the drainage blanket, this material had a calcite composition of around 11.6% and was used with the intention of the calcite neutralising the acid produced by oxidation of the pyrite.

It was determined in the period following construction of the embankment and filling of the reservoir that treatment to remove sulphates, iron and manganese was necessary before allowing drainage to discharge into the River Wolf downstream of the dam (Reid et al., 2005).

#### *Carsington Dam*

Carsington Dam is an earth embankment dam constructed in Derbyshire in the early 1980's. Problems were first noted at the site in 1984 when a large slip occurred on the upstream face as the dam approached its full height. Detailed investigations, including site studies, were carried out that led to modification of the design of the dam, although the location and alignment of the structure did not change, and construction recommenced between 1987 and 1992.

The materials used for construction were a clay for the core and weathered mudstone for the outer shoulders of the dam. The mudstone was highly weathered and potentially not free-draining, therefore, drainage layers of limestone were incorporated into the original design. The fresh mudstone at depth contained around 3.5% pyrite, 7.5% calcite and

### 3 Case Studies – Examples of Pyritic Heave Worldwide

had a neutral pH, whereas the weathered material used in the construction had lower pyrite and calcite contents and a weakly acidic pH (Reid et al., 2005).

The main problems caused by the pyritic material at Carsington were related to the production of acidic groundwater within the fill, and included:

- Reaction with the limestone drainage layers, resulting in precipitation of minerals such as gypsum and iron hydroxides (ochre) that clogged the drains, and generation of carbon dioxide that entered the manhole system thus presenting a secondary hazard.
- Corrosion of buried concrete structures, especially where these were in close proximity to the limestone layers, such as the concrete spillway.
- Runoff from the dam was highly acidic and contained high concentrations of iron and other metals. Runoff had to be collected and treated before being discharged into local water courses.

All of the above aspects were noted during the investigations that followed the 1984 failure of the dam and were flagged as requiring attention during the redesign of the structure prior to 1987. Remediation specifications included re-designing the dam so that it did not require internal drainage along the slopes, and where drainage was necessary, non-calcareous materials were used, detailed analysis of the mudstone material, and the construction of lagoons on the downstream side of the dam to allow treatment of runoff (Reid et al., 2005).

#### 3.4.2 Damage to buried road infrastructure, A564 and M5

##### *A564 Road Improvement*

Problems involving the corrosion of buried galvanised steel culverts was noted soon after construction of the road. In some instances this included the formation of holes within the culverts, which had to be replaced before the road could be opened to allow traffic access. The fill material used at this site was locally obtained alluvial sand and gravel, which, before use, was only tested for pH. The values obtained from these tests indicated that the material was suitable for use. The fill was later determined to have a pyrite content of around 0.4% sulphur, or a total potential sulphate concentration of around 6.0 g/l SO<sub>4</sub>, which is significantly above the limit of 0.3 g/l SO<sub>4</sub> that was later imposed (Reid et al., 2005).

Reid et al. (2005) cite this as an example in support of carrying out proper screening tests even in materials that are believed unlikely cause construction problems.

##### *M5 motorway bridges*

In 1998 damage was noted to several bridge foundations along the M5 motorway in Gloucestershire, constructed some 25 years before the damage was investigated. All

of the bridges were founded on Lias clay, excavated material was also used to backfill around the foundations, and showed signs of thaumasite attack upon the concrete (Reid et al., 2005). Tests prior to construction indicated that the clay was sulphate class 2, and therefore suitable for use. However, tests carried out in 1998 showed that the material was class 4 or 5. The change in sulphate levels is believed to be due to oxidation of pyrite present in the Lias clay.

In these cases only a portion of the sulphur released after oxidation of the pyrite appears to have reacted to form gypsum, the remainder had reacted with the concrete to form thaumasite and/or been lost to groundwater.

### 3.5 Discussion

The types of damage seen at these sites is often similar, and is discussed in more detail in Section 5.1, however, the main signs of damage as seen in these case studies and in Irish cases are summarised below.

- Heave/uplift of the floor slab.
- Damage to internal walls, fixings and floor finishes associated with uplift of the floor slab.
- Bulging and cracking of external walls.
- Damage to pipes and other services.
- Damage to machinery and other equipment that requires a level floor. It is common to see differential heave of the slab, leading to uneven floor surfaces. This can cause operating concerns and more obvious problems, such as preventing the opening/closing of doors.

The case studies available in the literature provide useful information relating to the effect of pyrite oxidation, both the effect of the change in acidity of the groundwater and the expansion caused by precipitation of gypsum, upon different types of structure. Of particular use is the way that not only do the available case studies detail the types of damage seen, but they also show some of the factors that had an influence on the expansion of the material.

These factors include, but are not limited to, those outlined below:

- The source of the material that caused the expansion. Within the scope of these case studies this is generally either the local bedrock material (for example Sections 3.1.1 and 3.2.1) or is fill material sourced from a secondary location (for example Sections 3.1.2 and 3.2.4).
- What type of foundation was used, and what was the depth.



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- What were the groundwater conditions, for example what was the local water table depth.
- Was the damage caused progressive or delayed due to environmental factors.

Although in some cases the information available is present in some detail, for other factors the available information is limited. For example, all of the case studies state whether the material is local bedrock or imported fill, whereas the details of foundation type and depth are often limited. In these cases it is possible that the information was either not considered of relevance to be included in the paper, or it was unavailable to those authors at the time of publication.

Two sets of information that are present in many of the case studies discussed within the literature is the chemistry of the material and details as to the amount of heave and the timescale over which it occurred. Table 3.4 summarises these values as present in the literature for each of the above case studies.

This comparison highlights the point that significant damage can be caused by even a small amount of pyrite when the material is in an environment that fosters oxidation. It also highlights that it can be difficult to find an exact correlation between the amount of pyrite and the amount of heave that will be induced in the system, due to the large number of factors that affect the amount of heave that occurs at a given location (see also Section 2.4).

The rates given in Table 3.4 are also discussed further in Chapter 8, where the comparison between some of the expansion rates seen in the case studies are compared to those obtained from the laboratory tests carried out on Irish pyritic mudrocks.

Many of the case studies presented above show damage caused by pyritic material to a single structure or at a single location. Although the damage that occurs at these locations still shows the distinct characteristics associated with pyritic heave (detailed further in Chapter 5), it is when multiple structures within a given area have suffered damage that people tend to look towards being able to classify the damage.

With the current situation in Ireland, this drive for classification has led to the introduction of Irish Standard IS398-1 (NSAI, 2013, discussed in Section 5.1), which is intended to determine whether damage seen to a property is likely to have been caused by pyritic heave before the homeowner commits to expensive testing procedures.

Of the case studies above, the only one that presents a classification system for the damage seen is that of Ballivy et al. (2002) with their consideration of the properties affected in the Montreal south-shore area (Section 3.2.4). This classification system is specific to the types of damage seen in the properties in this area.

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Location	Amount of pyrite	Average rate of heave	Damage
Llandough Hospital	TS: 2.20; WSS: 4.7 g/l	1.7 mm/yr	60 mm max difference in floor level, 10 mm width cracks in walls
Teeside	3.14 – 8.67%; average 5.4	N. S.	Uplift of floor slab and internal walls, bulging of external walls
Barry, Glamorgan	2.2% (calculated)	0.6 mm/yr	20 – 50 mm uplift of basement slab
Bell Canada Building	1.3 - 1.6%	18.9 mm/yr	94.5 mm to floor slab since construction
Rideau Health Centre	N. S.	3.8 mm/yr	76 mm heave
St Luke's Church	4.25%	1.4 mm/yr	63 mm uplift
Houses in Montreal	N. S.	14.4 mm/yr	25 mm general maximum uplift, one case with 75 mm uplift
Sainte-Foy	1 – 5%; average 2%	up to 20 mm/yr	200 mm maximum uplift
Kentucky	Up to 5%	N. S.	More than 10mm uplift, wall cracks up to 25mm wide, difference in floor levels between centre and walls of “several inches”

Table 3.4: Comparison of the amount of pyrite present in the rock material for each case study, along with the primary type of damage shown at the site and an estimated value of the rate of uplift. All values taken are the maximum values present in the literature. N.S. – no value specified in the relevant paper.

The classification system they present combines recommendations from the Quebec Technical Committee for Pyritic Heave (CTQ-M200, 2001) and factors seen in properties known to be suffering from pyritic heave in order to create a series of criteria that may be used to determine the possibility that other properties in the area are also suffering pyritic heave. Table 3.3 shows the classification system presented for the Montreal cases, and is compared with that for the Irish cases in Section 5.1.

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However, constructing a universal system of classification for damage due to pyritic heave is incredibly difficult due to the inherent complexity of each different situation when considering factors such as the type of structure, the type of material, the material density, chemistry and so on. The variation of these factors as seen in Ireland are discussed in Chapters 5, 7 and 8.

Although both the classifications presented by Ballivy et al. (2002) and Irish Standard 398-1 (NSAI, 2013) are focussed on the respective geographical areas that they represent, they do have some similarities. In particular, both consider the amount of movement of the floor slab and the presence and shape of cracks to the floor slab.

Stellate cracking and differential uplift of the slab are considered to be the primary indicators of pyritic heave – most other damage is linked to the movement of the slab, such as cracking of internal walls and movement of fixtures/fittings. Cracking and, in particular, uplift of the floor slab is mentioned in all of the case studies discussed above. Where available this is listed in Table 3.4 as either the maximum amount of uplift, or as the rate of uplift seen in each case.

Also of interest in the case studies, are the different remediation options that have been applied to the structures in order to repair the current level of damage, and to prevent further damage from occurring. The specific methods and variations mentioned in the case studies are summarised below.

- Llandough Hospital:
  - Underpinning of the external walls; creation of an extra wall and air gap to insulate the service ducts from the bedrock; installation of “units” to separate the bedrock from the floor slab.
- Bell Canada:
  - Increase of local water table; creation of alkaline conditions to inhibit the reaction process and bacterial activity.
  - Used a total of 12 tons of potassium hydroxide; required 16,000 litres of water per day to maintain the new water table; caused some settlement to the structure.
- Sainte-Foy:
  - Removal of rock material to create a void below the slab that would accommodate any further expansion; application of asphalt to the fresh rock surface to prevent air ingress intended to inhibit the reaction process; filling of the space between the rock surface and the slab (where necessary) with inert, granular material.
- Kentucky:

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- Removal of rock; fresh rock surfaces sealed with resin intended to inhibit the reaction process.

These remediation methods fall into one of two categories:

1. The removal of pyritic material.
2. The creation of unfavourable conditions.

Although the second of these methods is intended to prevent further damage by ensuring that the reaction process is either stopped or slowed to such a degree that the damage would not be significant within the life of the structure, it is generally costly both to initiate and to maintain. As seen in the case of the Bell Canada Building, maintaining a new water level required significant pumping of water into the area beneath the structure. This, in addition to the required correction of any settlement, is thought to be the primary reason that this method has not been attempted in other locations.

As discussed in Sections 2.8 and 4.4, similar methods have been considered in Ireland – such as injection of grout or bactericide foam. However, in Ireland the only accepted remediation method at the time of writing falls into the first category – removal of the pyritic fill material.

This is because, in cases where fill material is the cause of expansion, the first method, while also being costly and requiring significant disruption to the property, is generally a single-step measure requiring no maintenance once complete.

Although some of these case studies could be considered to be within hard to obtain sources, there is clean mention within the literature to instances where the expansion of pyritic mudrocks has led to damage of structures.

The combination of the case studies available in the literature, along with with the information present in the relevant standards at the time, suggests that the problem of expansion of pyritic mudrocks could have been anticipated were the relevant testing protocols adhered to. This matter is discussed further in the following chapter, with a detailed consideration of the relevant standards present in Ireland at the time of the Celtic Tiger housing boom.

## 4 The Pyrite Problem in Ireland

As stated in Chapter 1, a large-scale problem involving pyritic heave is currently being investigated in Ireland, primarily in the Dublin area. The reaction process, as covered in Chapter 2, has caused damage to a number of properties, some of which have been remediated, and potentially as many as 10,000 that are still being affected (Tuohy et al., 2012).

As indicated in Chapters 2 and 3, and as discussed further in Chapter 8, both the physical and chemical composition of the material has a significant effect on the reaction process. To that extent, the local geology of County Dublin is considered, in order to consider those materials that have been used as fill beneath properties known to be suffering from pyritic damage.

Following on from that, consideration is taken of the codes and regulations that were in place at the time of construction of many of these properties, looking into what factors should have been accounted for in order to restrict the use of pyritic materials as domestic fill. This includes limits introduced after analysis of cases of pyritic damage in the UK.

Some recent Irish case studies are considered in Section 4.3 (see also Chapter 5) before moving on to a consideration of what the insurance and remediation process currently involves for Irish homeowners.

## 4.1 Geological Setting

### 4.1.1 Regional Setting

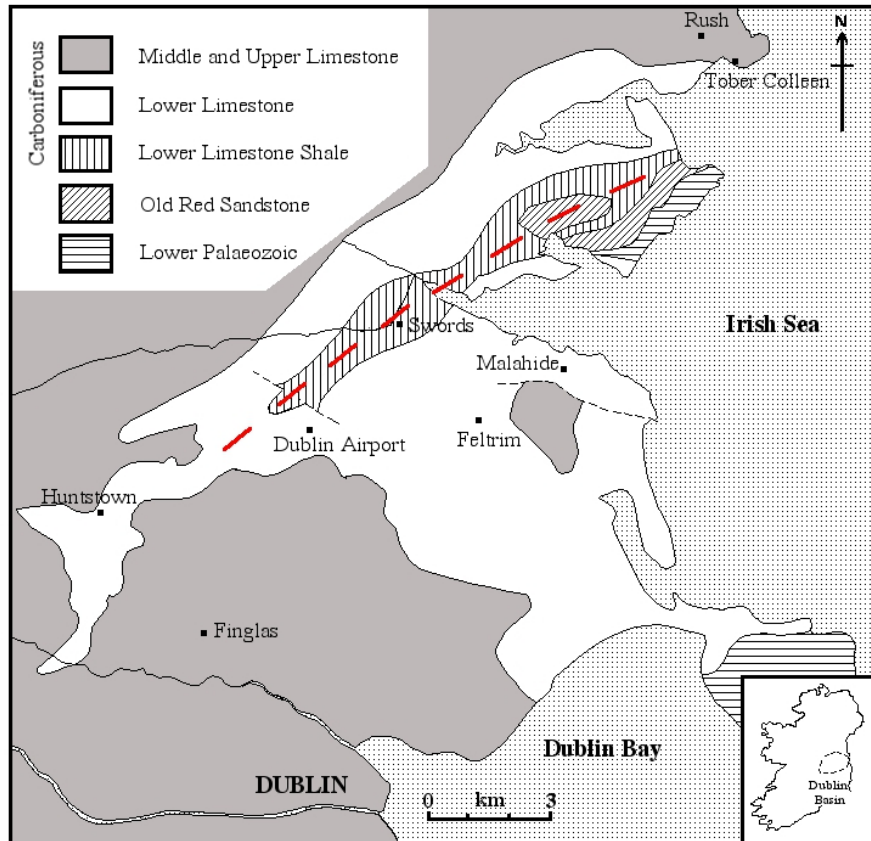


Figure 4.1: Outcrop of basinal and shelf facies in the Dublin area, including the inferred position of the Swords Anticline in red (after Jones et al., 1988).

Much of the Dublin area is underlain by Carboniferous age rocks that formed during the Mississippian (approximately 359 to 323 million years before present) as part of the Dublin Basin. The Dublin Basin was a fault bounded graben that likely extended well into the Irish Sea, possibly linking with the Craven Basin in the UK, and covered much of the northern area of the Irish Midlands during the late Tournasian and Visean Ages (Sevastopulo and Jackson, 2009).

Since framboidal pyrite is known to be a relatively common constituent of certain types of sedimentary rock, particularly those formed in low energy and deep marine environments. These rocks tend to be rich in carbon and potentially contain fossil material, which when undergoing diagenesis and subjected to overburden pressure, show replacement of fossils

#### 4 *The Pyrite Problem in Ireland*

with pyrite and the formation of fine grained framboidal pyrite throughout the material (Hawkins, 2014c). The deep marine environment of the Dublin basin, and the rocks that formed therein, are a prime location for the formation of pyrite-bearing marine sediments. Although, it should be noted, that as with any such environment, there will be significant lateral and vertical variation in the amount of pyrite due to the nature of the original sediment deposition.

The basin was bounded by carbonate shelves, the Balbriggan Shelf to the north and the Leinster Granite (also known as the Kildare Shelf) to the south. Sedimentary material moved from these shelf areas towards the centre of the basin, particularly in the form of debris flows triggered by movement on the faults at the edges of the basin, leading to a general decrease of grain size of the rocks progressing towards the centre of the basin (Sevastopulo and Jackson, 2009).

Coastal exposures around Rush reveal the basinal facies of the Tober Colleen and Rush formations, and although the base of these formations is not seen in coastal exposures, in boreholes at other locations, they have been shown to overlie Waulsortian Limestones. It has been suggested that the older Tober Colleen mudrocks formed from deep water muds, whilst the younger conglomerates of the Rush formation formed from turbidite and sediment flow processes (Sevastopulo and Jackson, 2009). Overlying the Rush formation are the argillaceous shales of the Lucan formation, which also formed in a deep water environment, whilst the younger formations such as the Naul formation were formed in shallower waters, implying a period of progradation in this area at the time of formation.

##### **4.1.2 Local Setting**

Due to the fact that many areas of bedrock are covered by quaternary material or man-made structures, many of the type sections for the rocks in this area are taken from coastal exposures or quarry faces. The succession seen at the Bay Lane quarry is equivalent in age to the Tober Colleen (Jones et al., 1988) and Rush (Kalvoda et al., 2011) formations seen in coastal exposures. We can, therefore, consider this material to be comparative, particularly with the depositional environment across the area.

Figure 4.2 shows the vertical succession of the rocks within the Dublin basin.

#### 4 The Pyrite Problem in Ireland

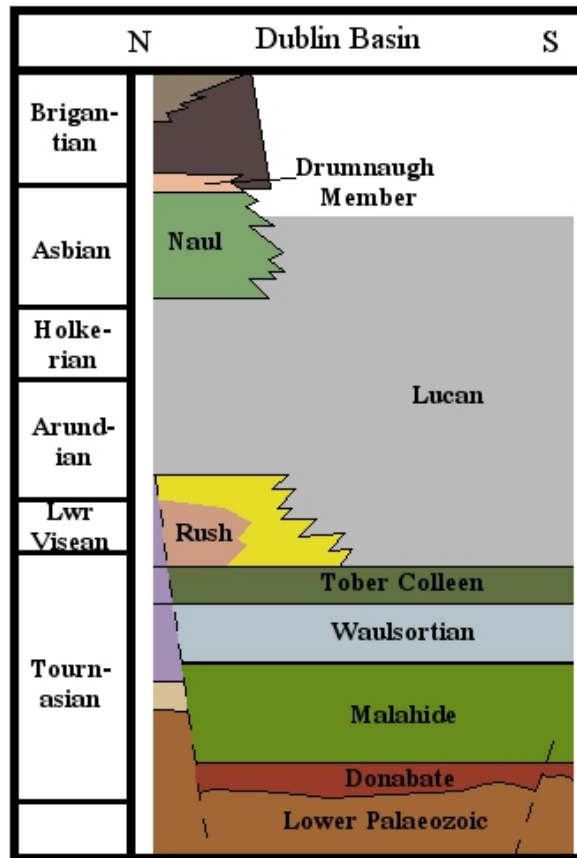


Figure 4.2: Vertical succession of the major rock formations in the Dublin Basin (adapted from Sevastopulo and Jackson, 2009).

Although Figure 4.2 gives more detail, the stratigraphic succession is simplified when applied for use by the Pyrite Panel (2012). In this case the Geological Survey of Ireland database is used to classify the rock according to their pyritic content. This database contains 7,362 mineral entries, of which 327 (some 4%) are pyrite and 9 are pyrrhotite.

Figure 4.3 shows the division of the rocks in the Dublin area into the 4 categories as assigned by the Panel. This division is discussed in more detail below.



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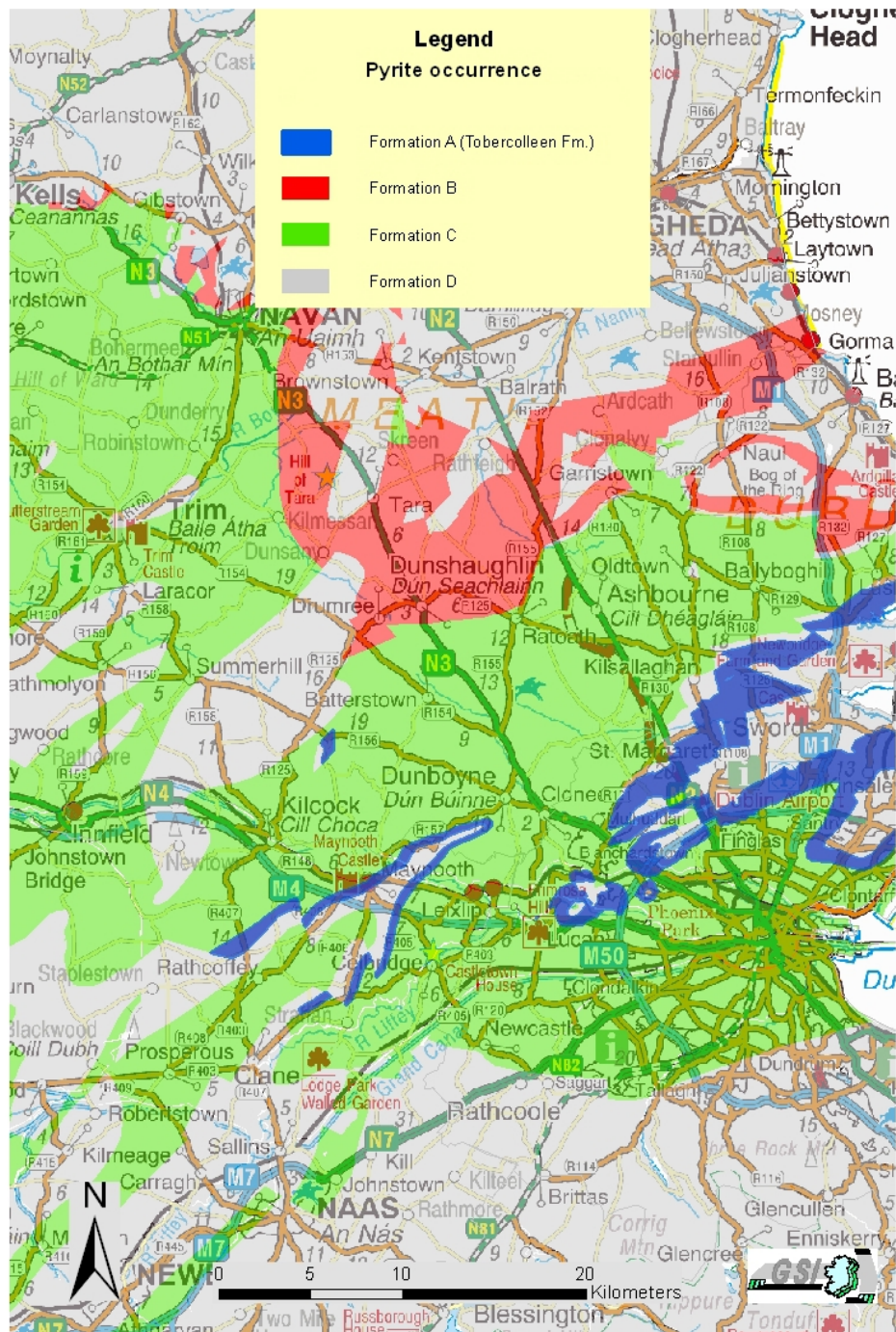


Figure 4.3: Outcrop of major rock groups in the Dublin area, A & B are those most likely to contain pyrite as discussed in the text (Tuohy et al., 2012). Note that some of these outcrops may be comprised of multiple rock formations.

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Rock group A is the outcrop of the Tober Colleen Formation, which is described in more detail below but primarily consists of interbedded mudstones and limestones, and is known to contain pyrite as a primary constituent of the material. Rock group B are rocks in the area that contain pyrite as a common constituent, but do not belong to the Tober Colleen Formation. Both of these rock groups are equally likely to contain pyrite. Rock group C are those formations in which pyrite may occur, but it is not a common constituent of the rock material, and group D are those formations in which pyrite is unknown or very rarely a constituent (Tuohy et al., 2012).

The main source of pyritic material in the Irish properties suffering damage due to heave is the material sourced for use as fill beneath the ground floor slab. In the case of the cases in the Dublin area, the material is known to have come from 5 quarries, with at least one quarry being located in each of the four rock groups detailed above (Tuohy et al., 2012).

There are three primary deep-marine facies identified in the northern Dublin area: Waulsortian facies limestones, cover mudstones and the Tober Colleen formation.

The Waulsortian facies limestones, identified in the older literature as the Feltrim limestones (Jones et al., 1988), are often referred to as reef limestones despite them not forming from coral/algal reef structures. These rocks commonly contain large amounts of micrite formed from multiple layers of carbonate mud. The associated rocks are massive with minimal layering or laminations, and generally lack the faunal structure seen in most limestones. Variations in the rock are normally associated with a shift in depositional environment, as the water depth fluctuates going from a deep to shallow marine conditions (Sevastopulo and Jackson, 2009).

The mudstones that overlie the Waulsortian rocks are referred to by Jones et al. (1988) as Cover Mudstones and by Sevastopulo and Jones (2009) as Waulsortian Mud Mounds. These are considered to be a deeper water facies than the Waulsortian limestones, indicated by the interbedded mudstones, micrites and shales that make up this formation.

The Tober Colleen formation is higher in the stratigraphy than the Cover mudstones, and outcrops in the Huntstown Quarries (Jones et al., 1988) and the Bay Lane Quarry (Kalvoda et al., 2011).

Jones et al. (1988) describe the Tober Colleen formation as “an alternation of cleaved mudstones and dark coloured, barren micritic limestones”. They are similar in composition to the Cover mudstones, although the Tober Colleen shows more variation in material between the mudstones and the more laminated shale material. Kalvoda et al. (2011) describe the outcrop of the Tober Colleen in the Bay Lane quarry as consisting predominantly of “dark grey, sandy calcareous siltstone” with a large  $\text{CaCO}_3$  component to the rocks in this location, with some beds containing calcareous nodules up to 30cm in diameter. It is inferred that this is the upper section of the Tober Colleen, and is associated with the turbidites described by Sevastopulo and Jones (2009).

## 4.2 History of the pyrite problem

The first instances of pyrite related heave to domestic properties were recorded in Ireland around 2006/7 (Tuohy et al., 2012). Although there is speculation about the single cause of the dramatic increase in the number of cases that were recorded at this time, it is most likely that it is due to a series of causes, some with more influence than others. Hawkins (Hawkins, 2014a) suggests a list of reasons for sudden large number of cases, including the following.

- Mis-diagnosis of earlier cases as settlement of foundations.
- A change in style/type of fill material used for domestic construction.
- A change in the recommendations for the amount of compaction of the material - requirements from the late 1990's onwards required the material to be compacted with a moisture content at or up to 2% below its optimum. This means that since this time, fill material has been compacted close to maximum density and is therefore less able to accommodate expansion.
- There was a possibility that material would be stockpiled under less than ideal conditions while on site, allowing the material to become wet and begin to oxidise.
- With an increase in the demand for fill material, it is likely that quarries may have used new strata that have not been examined and tested completely.
- There is also a possibility that quarries previously supplying material for use in road construction also started shipping material for other uses without additional testing of the new material.
- There were changes to the design/construction of properties in the 1990's/2000's that will also have had an effect on how the properties respond to expansion of the fill.
  - In increase in the use of polythene damp proof membrane led to a subsequent increase in the use of ground bearing floor slabs.
  - An increase in the popularity of both underfloor heating and accessibility of better insulation means that modern properties are more likely to be kept warm.
  - Many walls now finished with dry lining that is left to sit on the slab and will therefore bow and show signs of distress quicker should expansion occur.
  - Similarly, modern cement is more brittle than lime mortar and so is more likely to show signs of stress cracking.
  - In order to save time during construction, it is possible that best practice may not have been followed strictly, and floor slabs were poured in one continuous

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slab, even over rising walls. This will transmit any heave of the slab directly to the rest of the structure.

It is easy to see how the heave cases were mistaken for foundation settlement in the early stages of the problem becoming apparent. Figure 4.4 shows how similar some of the damage seen in cases of pyritic heave is to that seen with differential settlement of a foundation.



Figure 4.4: Damage to residential properties caused by pyritic heave (a & c) and foundation settlement (b & d). Images a & c used with permission of Taylor Tuxford Associates, images b & d used with permission of Aidan O'Connell Associates.

At the time of construction of many of the properties that went on to show signs of pyrite related damage, there were several specifications for fill that were in place and likely to have been referred to during the construction process. Some of the specifications in place

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at the time of construction of many of the Irish properties are mentioned briefly below, along with those that have since been introduced.

**The Homebond Manual** (HomeBond, 2007), which specifies that material used as fill should be “good quality hardcore: clean, crushed well graded stone” and “free from shale and 100mm maximum size”. Interestingly the manual also notes the possibility of problems due to the presence of sulphates, “sulphate can cause expansion and disruption of concrete, particularly on filled sites”.

**The Irish Building Regulations** (ECLG, 2004) specify that any fill places should be at least 150mm thick after compaction, and that it should be comprised of “broken stones, brick or similar suitable material, well compacted, clean and free from matter liable to cause damage to concrete”.

This led to the common practice of ordering “3 inch down” or Clause 804 (NRA, 2013) material for use as fill beneath domestic properties.

Prior to the pyrite related heave being noted in properties in Ireland, the main focus on sulphate in the standards was related to its potential to cause sulphate attack on concrete and therefore much of the documentation focused on the Water Soluble Sulphate (WSS) value of a material.

**The Highways Agency** in the UK and the **National Roads Authority** in Ireland both published documentation revisions in 2000 that stated that for fill material within 0.5 metres of concrete, the WSS should not exceed 1.9 g/l SO<sub>3</sub>, which is equivalent to 2.3 g/l SO<sub>4</sub>. Based upon recommendations from the Thaumasite Expert Group, the 2004 revisions of documentation by both agencies included a maximum Acid Soluble Sulphate (ASS) value of 0.2% SO<sub>3</sub> (or 0.24 % SO<sub>4</sub>) within 0.5 metres of concrete to go with the WSS value. The acid soluble sulphate represents the portion of the sulphur present as sulphate, which may be present in cases of pyritic heave in solution as sulphuric acid or precipitated as gypsum.

**The European Standards** in place in the early 2000’s focused more on the physical properties of the fill. Clause SR21 (the Irish documentation for EN13242:2002) chose Clause 4 for domestic fill material, meaning that testing of the material was required at the quarry, although they (like the UK) did not require at that time that a CE mark should be added to the material sold.

Both EN13242:2002 (BSI, 2002) and EN13285:2003 (BSI, 2003) draw attention to the responsibility of the producer with respect to their product. In particular they highlight that the producer was the one responsible for testing material and ensuring that it did not exceed “provisions valid in the place of use of the unbound mixture”.

The 2004 update of SR21 of EN13242:2002 includes the notation that the Total Sulphur (TS) value for a given material “may not of itself completely exclude the risk of swelling due to the presence of a reactive form of pyrite”, and that the TS levels should be tested, particularly where there is variability in the stratigraphy of a quarry.

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In 2005 the British Research Establishment produced Special Digest 1 (Establishment, 1995), considering concrete in aggressive ground. This recommended to account for the following chemical values of a material intended for use as fill in contact with concrete: WSS (g/l SO or SO<sub>4</sub>), pH, ASS (% SO<sub>4</sub>), TS (% S), Total Potential Sulphate (TPS as % SO<sub>4</sub>) and Oxidisable Sulphides (OS as % SO<sub>4</sub>). The TPS can be calculated as 3 times the TS value, and the OS can be calculated as the TPS - ASS values.

**The TRL laboratory report 447**, first published in 2001 and updated in 2005 (Reid et al., 2005), gave specifications for acceptable amounts of pyrite present in fill placed within 0.5 m of concrete or steel. Both considered the values given by other specification documents and revised some of the maximum values down based on more recent evidence. They also provided a comprehensive discussion of the chemical tests carried out to determine the values for a given material.

The 2001 edition of the TRL report suggests the following limiting values:

- For concrete:
  - 2.3 g/l SO<sub>4</sub> for WSS
  - 0.6 % SO<sub>4</sub> for TPS
  - 0.46 % SO<sub>4</sub> for OS
- For galvanised steel:
  - 0.3 g/l SO<sub>4</sub> for WSS
  - 0.06 % SO<sub>4</sub> for OS

The 2005 update revised the values for concrete as follows:

- 1.5 g/l SO<sub>4</sub> for WSS
- 0.3 % SO<sub>4</sub> for OS

This report also considers the significance of framboidal pyrite, stating that if it is present that the material should be considered unsuitable for use as fill unless the material has been used before with no problems and the reason why it will not cause problems is known and documented based on chemistry and mineralogy (Reid et al., 2005).

The **NRA Specification for Roadworks 800 Series** (NRA, 2013) contains the standard for clause 804 material that is commonly used as fill material in Ireland. The March 2000 edition of the standard notes that material within 0.5 metres of concrete should have a WSS value less than or equal to 1.9 g/l sulphate as SO<sub>3</sub> (equivalent to 2.3 g/l SO<sub>4</sub>), and that such material should be placed in layers less than 225mm thick leading to a maximum compaction thickness greater than 110mm.

Classification of material specified as Clause 804 includes:

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- The PSD of the material must fit within a specified grading envelope to ensure it compacts well.
- The Liquid Limit should be less than 21% to limit the amount of clay minerals
- Material should be placed in moisture content range optimum to optimum -2 %
- Water absorption should be less than 2 %
- Flakiness index should be less than 45
- ASS threshold of 0.2 % SO<sub>3</sub>

A 2010 update added Clause 809, which added requirements for material placed near steel. This specified a WSS threshold of 0.3 g/l SO<sub>3</sub> and stated that the OS should not exceed 0.06 % sulphur as SO<sub>4</sub>.

As stated above, many of these specifications have been introduced or amended since the construction of many of the properties now showing signs of pyritic heave. As discussed in Chapter 1, the Irish housing boom began around 1995, but did not peak until 2007 (Tuohy et al., 2012), and as such the standards that were in publication at that time can be considered, although as discussed below some might not have been readily available in Ireland.

Table 4.1 shows those standards in publication at the peak of the housing boom in 2007. As such, it is expected that anyone constructing domestic dwellings and utilising fill material in Ireland at that time would at least be familiar with those standards published in Ireland. The detailed references within each of these standards regarding the fill material that could be used, and the chemical values as applied by each standard are discussed above.

Although further amendments were made to many of these standards after 2007, including the addition of information specific to pyritic rocks, that information is not detailed within Table 4.1 as it was not available at the time of construction of the properties. More recent additions to the standards, especially the introduction of Irish Standard IS398-1, and the subsequent changing of limiting values is discussed in Chapter 5.

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Standard	Year of introduction	Country of origin	Relevant warnings
The Homebond Manual	2007 (5th Edition)	Ireland	Clean, well graded fill, free of shale. Warns about problems due to presence of sulphates
Irish Building Regulations	Up-dated/amended annually since 1997	Ireland	“free from matter liable to cause damage to concrete”.
National Roads Authority (Clause 804)	2000 edition and 2004 revision	Ireland	For material within 0.5 m concrete, WSS less than 2.3 g/l SO <sub>4</sub> ASS less than 0.24% SO <sub>4</sub> .
EN 13242:2002 & SR21	2004 revision	Ireland	Total sulphur “may indicate presence of reactive form of pyrite”.
BRE Special Digest 1	2005	UK	List of chemical values that should be considered before use as fill.
TRL report 447	2001 edition and 2005 revision	UK	WSS less than 1.5 g/l SO <sub>4</sub> OS less than 0.3% SO <sub>4</sub>

Table 4.1: Relevant standards that were in publication at the time of the peak of the Irish housing boom in 2007.

It is clear from the increasing amount of information specified in the various standards that as the pyrite problem became better understood there was a corresponding increase in the detail provided.

It also shows that there was, especially towards the end of the housing boom in 2005 – 2007, relevant information available on the chemical limits for material containing pyrite that was to be used within 0.5 m of buried concrete or steel, with the limits presented by the National Roads Authority and in TRL report 447. Both of these set reasonably low limits for chemical factors that should have been tested, at the very least, at the time of quarrying. As discussed in later chapters, the chemical properties of the fill material, even several years after placement, exceeds the limits that were present at or around the time of construction.

Although the intent of the creation of new standards, and the amendment of existing ones, is intended to limit the number of cases of pyritic heave in future by both making people aware of the problem and ensuring that testing should be carried out, it is uncertain whether this will be the case.



Considering that the problem of pyritic heave causing floor slab uplift has been known in the literature since the 1970's, and the requirement for testing of material were in place, pyritic material was still used as fill material in the construction of up to 12,250 domestic properties in Ireland (Tuohy et al., 2012).

### 4.3 Irish Case Studies

#### 4.3.1 Ballymun Youth Facility

Located in the north of Dublin, approximately 6km north of the centre, and close to Dublin International Airport, Ballymun was one of many areas in which new properties were constructed and other refurbished as part of the Celtic Tiger boom. Originally built in 1998, an extension to the Ballymun Youth Facility was planned in 2002 as part of regeneration works in the area, as is shown in Figure 4.5, with construction beginning at the site in August 2004.

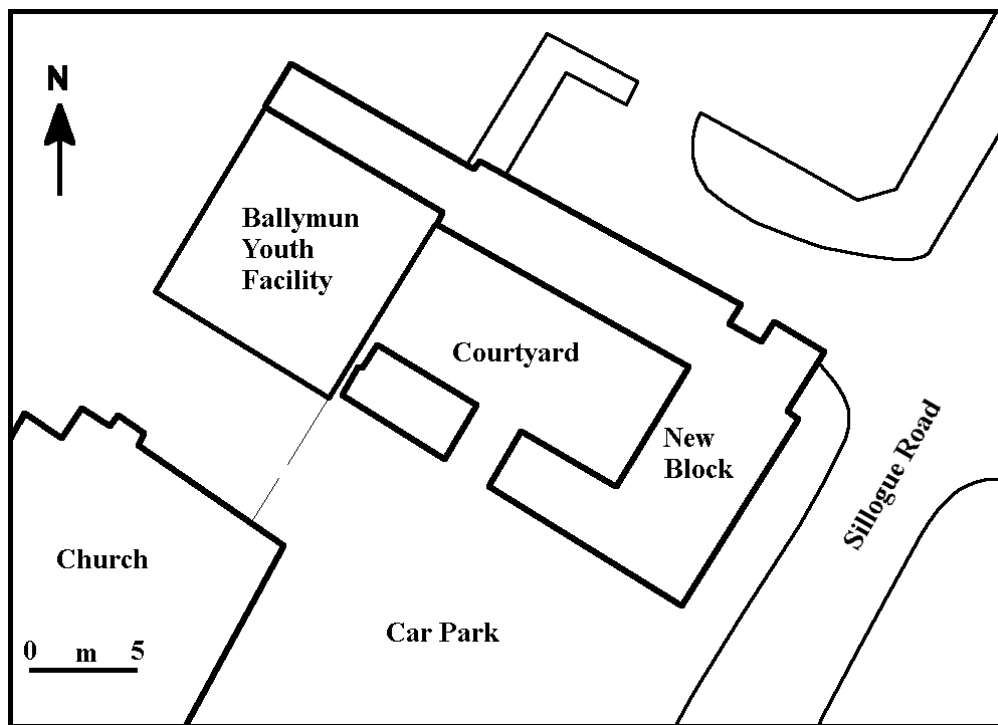


Figure 4.5: Layout of the Ballymun facility showing the original structure and the extension built as part of the regeneration works (after Hawkins and Stevens, 2014).

Ground investigations at the site consisted of four trial pits, dug to a depth of around

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2.9 metres below local ground level. These trial pits indicated that there was a varying thickness of made ground over the site, overlying a thin layer of brown glacial deposits (the Dublin Boulder Clay) before reaching the Dublin Black Boulder Clay. This latter material was selected as the base upon which mass concrete pier foundations would be sited in order to support the pad foundations for the new sections of the building (Hawkins and Stevens, 2014). Aggregate fill, specified in the schedule of works as required to be Clause 804 compliant, was placed between the concrete piers and compacted in layers of 150 – 225mm, and covered with a radon barrier, insulation and 150mm concrete floor slab. Pouring of the slab took place in December 2004 and “practical completion” of the facility was signed off on 2nd September 2005 (Hawkins and Stevens, 2014).

The first signs of damage to the building were recorded within the first month of completion and continued throughout early 2006 until the 3rd August when a snag list was put together, as is summarised in Table 4.2 below.

Floor	Room	No. cracks	Max length (mm)	
Ground Floor	Reception	16	1,300	
	Cafeteria	9	600	
	Kitchen	3	1,500	
	Corridor 1	5	700	
	Office	5	1,000	
	Workroom	9	1,000	
	Interview Room	4	full height of room	
	Corridor 2	5	full height of room	
	Toilets	3	full height of room	
	Corridor 3	6	full height of room	
	Information	3	600	
	Multipurpose Room	6	full height of room	
	Corridor/stairs	3	1,500	
	Gym	5	“a few metres”	
	First Floor	Internet (public)	4	1,000
		Internet (classroom)	2	1,000
		Lobby	3	2,000
Conference Room		4	2,000	
Bathroom		1	2,000	
Corridor 4		9	2,000	
Toilets		7	Not stated	
Corridor 5		4	3,000	
Arts room		2	2,000	
Staff room		1	1,500	
Stairs to lobby	4	1,500		

Table 4.2: Snag list for the Ballymun Facility as of 3rd August 2006 (Tuohy et al., 2012).

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Between September and November 2006, repairs were carried out to the facility, covering the items listed in table 4.2 as well as to displaced plasterboard that had caused gapping around plug sockets and to a crack on the ceiling of the gym. By October 2007, with problems still occurring, an independent assessment was commissioned. Some images of the damage present at this time are contained in Figure 4.6.

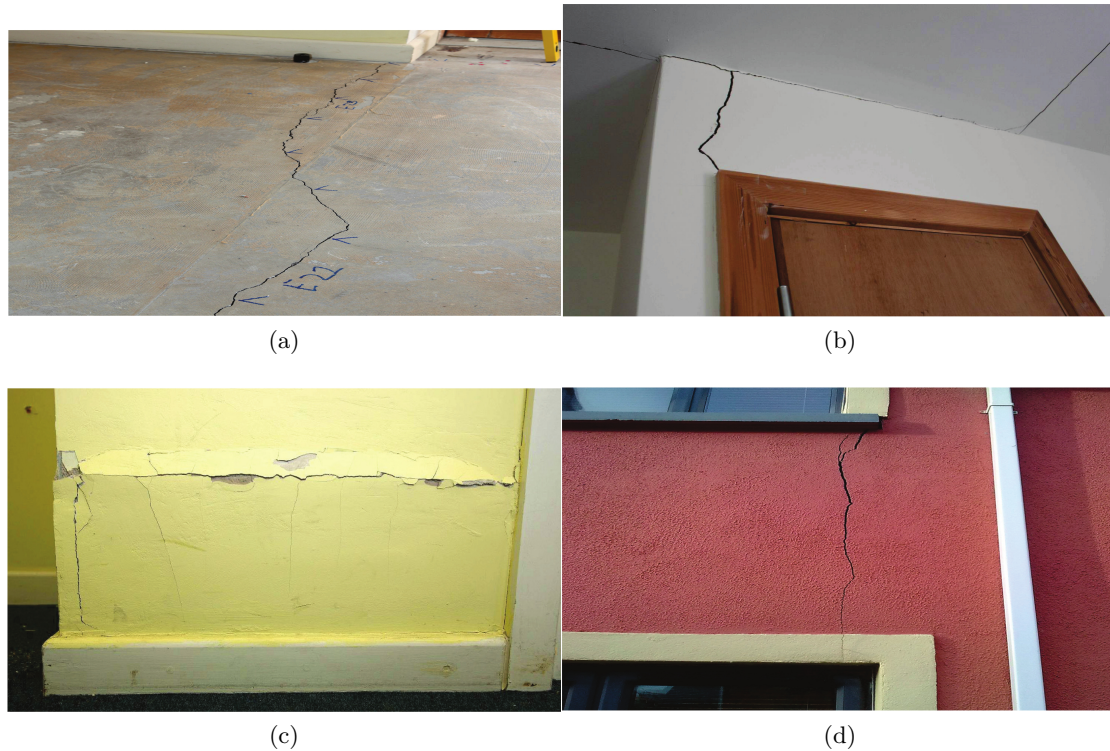


Figure 4.6: Collection of images showing the damage recorded at the facility (after Hawkins and Stevens, 2014).

It was suggested at this time that the damage may be due to sulphate heave, which was being identified in the Dublin area at this time, consequently, a more detailed investigation was commissioned and carried out, including sampling of the fill material for geotechnical and chemical analysis. A total of seven sampling locations were established at the site: five within the facility (to study the material beneath the floor slab) and two in the courtyard to sample the material beneath the brick paving in this area. The results of the tests carried out at this location are summarised in table 4.3 and are discussed below.

Tests carried out on samples from this site included physical tests (PSD, Atterberg limits and water absorption) and chemical tests (total sulphur, acid soluble sulphate and water soluble sulphate). The samples labeled as 1–5 herein were taken from beneath the floor

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slab of the structure, whilst those labeled as 6 & 7 were taken from the central courtyard area.

	<b>1</b>	<b>2</b>	<b>3</b>	<b>4</b>	<b>5</b>	<b>6</b>	<b>7</b>
Mudstone (%)	51	58	53	59	66	7	7
XRD – pyrite (%)	2	2	2	2	3	1	<1
XRD – gypsum (%)	4	3	2	2	3	0	1
Total Sulphur (%)	1.38	1.41	1.34	1.51	1.56	0.35	0.24
A.S.S. (% <sub>4</sub> )	0.55	0.46	0.46	0.51	0.43	0.08	0.06
W.S.S (mg/l <sub>4</sub> )	1614	1575	1612	1589	1603	538	144
Sulphur (% S)	0.83	0.95	0.88	1.00	1.13	0.27	0.18
Equivalent pyrite (%)	2.24	2.35	2.22	2.50	2.65	0.60	0.41
Original pyrite (%)	2.58	2.64	2.51	2.82	2.92	0.65	0.45

Table 4.3: Summary of tests results from the Ballymun Youth Facility (Hawkins and Stevens, 2014).

Samples 1 – 5 were generally dark grey to black in colour, whilst 6 & 7 were lighter and more brown in colour, which is likely due to the different lithologies present in the samples. PSD values generally fell within the Clause 804 limits, although there was an increase in fines at some locations, which was believed to be due to compaction methods. The liquid limit values were slightly higher than the recommended levels (23–30% rather than 20–21%), but were not high enough to indicate the presence of expansive clay minerals (Hawkins and Stevens, 2014). There was a clear division in the lithology of the fill material between samples 1 – 5 and 6 & 7. Samples 1 – 5 were composed predominantly of calcareous mudstone (43–60%), whereas 6 & 7 were composed of more than 60% pure limestone (Hawkins and Stevens, 2014). Pyrite was present as both cubes and framboids, many of the latter showing alteration rims.

The NRA recommendation in place at the time of construction states that material within 0.5 metres of concrete products should not exceed water soluble sulphate values of 2,300 mg/l SO<sub>4</sub> and, as of May 2004, acid soluble sulphate values of 0.2% SO<sub>4</sub>. All of the samples met the water soluble sulphate criteria, however only the samples from locations 6 & 7 met the acid soluble sulphate criteria as stated earlier; the samples from locations 1 – 5 were more than double the limit (Hawkins and Stevens, 2014).

Although standards such as the 2001 edition of the TRL report 447 (Reid et al., 2005) was in publication at the time of construction, it is uncertain whether this was widely available in Ireland in 2004 when the extension was constructed. Therefore, the NRA recommendation, which was published by an Irish authority and so should have been widely available, is the one to which these results are compared. The values cited above are those present in the NRA documentation in 2004 (WSS limits) and 2004 (ASS limits).

Beginning in May 2008, another phase of investigations was started, including levelling surveys and additional trial pits intended to check on the founding level of the concrete

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piers. The levelling survey revealed that several points in the building rose by more than 5mm over the survey period of around 400 days. Also of note, was an increase in the rate of heave between 18th June and 9th July 2008, which is believed to be linked to a period of warmer weather (Hawkins and Stevens, 2014).

In addition to checking the founding level of the concrete piers, the trial pits (dug in July 2008) were also utilised to assess the condition of the concrete forming the ground beams and the piers. The following things were noted from the concrete samples:

- Patches of gypsum were present on the outer surface of the concrete, some of which were more than 50  $\mu\text{m}$  thick.
- Within the sample taken from the ground beam:
  - Ettringite extended up to 8 mm into the concrete and was present in voids at some locations (10.6%  $\text{SO}_4$  at 1 mm into the concrete and 7% at 3 mm).
  - Thaumasite present up to 2.8 mm into the top of the ground beam, and in cracks up to 1.8 mm into the concrete.
- Within the sample taken from the top of the concrete pier:
  - Ettringite found in both pores and cracks to a depth of 5 mm into the concrete.
  - Up to 11%  $\text{SO}_4$  by mass cement at the top edge of the pier.
- : Within the sample taken from the outer edge of the concrete pier:
  - Thaumasite present up to 2 mm into the concrete (12%  $\text{SO}_4$  by mass cement).
  - High levels of Ettringite (up to 12%  $\text{SO}_4$  by mass cement) up to 5 mm into the concrete.

It was noted that despite the high levels of  $\text{SO}_4$  detailed above, there was little evidence that sulphate attack had already taken place, although it was suggested that it would have been highly likely to occur with the introduction of further sulphates (Hawkins and Stevens, 2014).

Remediation works at the site began in May 2009, largely because the building was becoming a hazard to people using the building. Hazards noted included: trip hazards due to uneven floors and lips in doorways, sticking doors, stress in areas around electrical sockets and cracking and bulging of wall plaster. Although multiple options were considered for the site, it was determined that the most practical solution was to remove the pyritic fill material and replace it with inert fill that had been thoroughly checked and approved for use (Hawkins and Stevens, 2014). The process used for removing the fill is similar to that used at other Irish sites, and is discussed in detail in Section 4.4.2.

Throughout the remediation process at this site, samples were taken of the underfloor fill, and the foundations were checked to ensure they had been placed upon the Black Boulder Clay, and thereby ensure the problem was not due to differential settlement

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and to check whether the fill had affected the integrity of the concrete. Due to the high amounts of sulphates, and the depth to which they had penetrated the concrete, it was decided to scabble the ground beams and concrete block rising walls before covering them in a Fosroc product intended to prevent the ingress of further moisture into the concrete.

During the remediation process, it was noted that in many locations there was a distinct horizontal crack present around much of the external wall, showing a clear separation along the rising wall around 0.5 metres below ground level (Hawkins and Stevens, 2014). The cracking was due to both vertical and horizontal expansion occurring within the fill, causing the structure to be lifted away from the supporting walls. Once the fill, and so any lateral pressures, was removed the structure settled back into place (Hawkins and Stevens, 2014). Similar cracking has been noted in other locations in Ireland (Section 4.3.2 and Chapter 5).

The remedial works were completed at this site in December 2009, with no damage recorded since (Hawkins and Stevens, 2014).

##### **4.3.2 Donnycarney Senior Citizens Dwellings**

Constructed between 2003 and 2004, the senior citizens' dwellings in Donnycarney (located approximately 4.5km to the northeast of the centre of Dublin) had started to show signs of damage within 18 months of completion of construction. Damage at this site included cracking of ground floor walls, and distortion of kitchen units and shower trays. As it was becoming clear at this time that there were numerous problems to properties in and around Dublin that showed similar signs of damage and were due to defective fill material, samples were taken from beneath the floor slab of the properties. Testing on this material indicated the presence of pyrite and gypsum (Finnegan and Hawkins, 2014). The layout of the site is shown in Figure 4.7.

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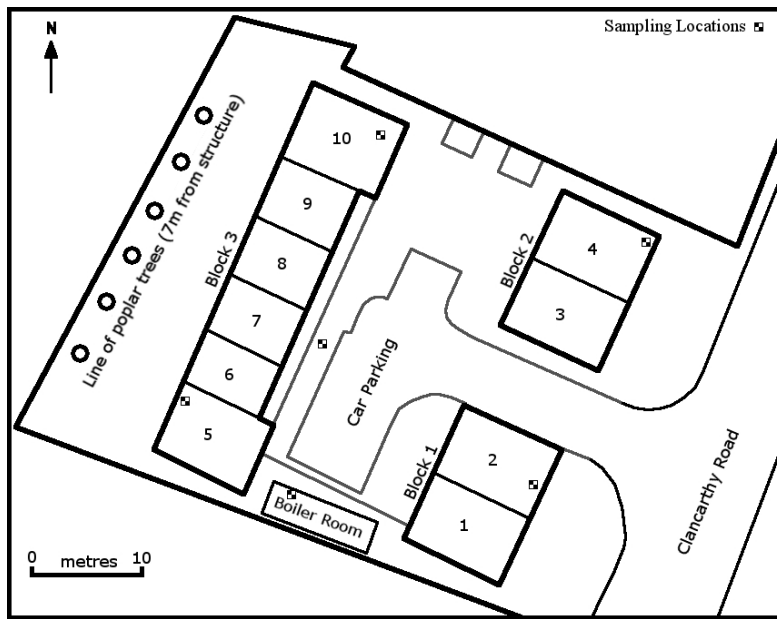


Figure 4.7: Layout of the Donnycarney site (after Finnegan and Hawkins, 2014).

Each of the three blocks at this site are comprised of two storey structures, each floor comprising multiple self-contained flats. There is a communal parking area in the centre of the site, and a laundry/boiler room to the south of the site. Each of the buildings are brick-block masonry construction, with load-bearing walls sited on strip footings, and the floor slab is reinforced concrete over the imported fill material that floats within the rising walls. The upper flats are access via external steps connected to a balcony structure that is supported by steel columns (Finnegan and Hawkins, 2014).

Initial damage seen at this site included cracking of tiles, arching of kitchen work surfaces, jamming of doors and changes in floor levels. In several flats it was also noted that there had been a rotation of the internal lintel, which was noted as being caused by upward movement of the internal partition walls (Finnegan and Hawkins, 2014). During investigations at the site, the floor slabs to a number of ground-floor flats were inspected, revealing the presence of “spider cracking” in the screed that extended down into the floor slab.

In some flats a general “doming” of the floor slab was recorded, whilst in others the slab was noted to have risen as a whole; this was noted particularly in Unit 2, in which there was a rise of 48 mm against the bathroom wall and 18 mm against the rear wall. Additionally in Unit 2, it was noted that the gable wall showed an outward movement of up to 20 mm at the base of the wall, which was associated with cracking seen on the exterior of the same wall. Cracking was also noted within the manholes sited in the parking area of this site and to the walls of the boiler room, in which movement of poles intended to support the pipework had moved upwards by up to 1 mm causing the roof

#### 4 The Pyrite Problem in Ireland

structure to be forced upwards by a similar amount (Finnegan and Hawkins, 2014).

Investigations at the site began in March 2009 and continued through to December 2009, with multiple trial pits located across the site in order for samples to be taken from the fill material. One of these trial pits, located beneath the floor slab of Unit 4, revealed horizontal cracking some 680 mm below floor level. It was possible for site engineers to insert a trowel into this crack, revealing it to be continuous throughout the wall, as the only resistance encountered was some 225 mm through the wall, likely due to the fill material present at the other side of the rising wall. A similar pit was placed to the rear of block 3, revealing a similar crack at this location. This crack was some 650 mm below the DPC and was 8-10 mm wide along its length, extending to 20 mm wide in locations where the mortar had crumbled away (Finnegan and Hawkins, 2014). The samples taken from these trial pits were sent for chemical and geotechnical testing, the information from which is summarised in Table 4.4 below.

Sample Location	Unit 2	Unit 4	Unit 5	Unit 10	B.Rm	Car Park
M.c. (%)	5.84	6.18	6.18	6.66	7.05	5.63
Mdstn (%)	68	73	73	73	70	5
Lstn (%)	30	21	21	23	27	76
Calcite (%)	28	31	31	29	27	58
Pyrite (%)	1.1	1.2	1.2	1.1	1.3	0.1
Gypsum (%)	4	4	4	5	6	0
TS	1.20	1.27	1.27	1.38	1.71	0.16
ASS	1.54	1.78	1.78	2.04	2.14	0.2
WSS	1,730	1,758	1,758	1,804	1,734	64
TS - SO <sub>4</sub>	0.69	0.68	0.68	0.70	1.00	0.09

Table 4.4: Chemical and geotechnical results obtained from tests on material taken from the Donnycarney site (after Finnegan and Hawkins, 2014). B.Rm = Boiler Room; m.c. = Moisture Content; Mdstn = mudstone; Lstn = limestone; TS = Total % sulphur by mass; ASS = acid soluble sulphur % by mass SO<sub>4</sub>; WSS = water soluble sulphur mg SO<sub>4</sub>/l; TS - SO<sub>4</sub> = total sulphur minus sulphur in sulphate as % sulphur.

The material tested was classified as unsuitable for use as fill in contact with concrete for the following reasons (after Finnegan and Hawkins, 2014):

- The total sulphur from all internal samples exceeded the 1% threshold recommended as per Irish Standard SR 21:2004;
- The acid soluble sulphate levels were above the 0.2% threshold given in both SR 21 and NRA 800;
- Although the water soluble sulphate levels are below the threshold given at the time of construction, they are above the threshold of 1,500 mg SO<sub>4</sub>/l given by TRL



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in 2005 relating to any fill within 500mm of any concrete structure;

- All of the internal samples had a total sulphur minus sulphur as sulphate value of greater than or equal to 0.68%, this is above the limit of 0.46% SO<sub>4</sub> given by TRL in 2001 that was in place at the time of construction, and more than double the revised limit of 0.6% SO<sub>4</sub> that was put into place in 2005.

Testing was also carried out to determine if there had been any deterioration of the concrete block rising walls due to sulphate attack. Samples were cored from units 5, 7 & 10, with SO<sub>4</sub> levels varying from 7 – 14.5% by weight, and a maximum depth of sulphate of 27 mm. In cases such as this, with the presence of concrete blocks, sulphates such as ettringite and thaumasite are usually precipitated within voids in the blocks. At the Donnycarney site, it was determined that whilst ettringite and thaumasite had started to form within some of the blocks, if the fill was removed there would be no further formation, and the amount that was present was not sufficient to cause damage to the blockwork (Finnegan and Hawkins, 2014).

Remediation works at this site began mid-2011, and involved removing the fill material from beneath each of the units at the site. It was initially thought that settlement of the structures would take some 7–10 days, however, the settlement was much more rapid than this, meaning it was difficult for engineers to record the settlement via planned means. Instead a dial gauge and time-lapse camera system were used, which revealed that the settlement was not a continuous process, but rather occurred in “bursts” of around 0.75mm followed by periods in which no movement occurred (Finnegan and Hawkins, 2014).

Overall settlement of the structures was noted by the authors (Finnegan and Hawkins, 2014) to occur in three stages:

1. Removal of internal partition walls led to straightening of lintels and closing of cracks in balconies as pressure was reduced on the first floor slabs;
2. Breaking of the ground floor slabs led to reduction in the confining pressure, allowing walls to return to their original position and alignment;
3. Removal of the stone fill allowed the structure to return to its original position, leading to closure of horizontal cracks.

It is also to be noted that the authors Finnegan and Hawkins (2014) highlight unique problems with the remediation process when applied to terraced structures. Most notably that the removal of fill in one section of the structure will relieve confining pressures whilst other sections will still be under pressure, this can lead to secondary cracking within the structure during the remediation process.

Following on from considerations of the method by which cracks closed at the Donnycarney site, studies were carried out at Avila Park, where a series of bungalows constructed in 2005 were suffering from damage related to expansive fill material. At this site, the internal walls had been “toothed” into the external walls, leading to cracking of the

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external wall above the lintel as the internal walls were forced upwards. In some cases, these external cracks were 18–25 mm wide (Finnegan and Hawkins, 2014).

In August 2011 experimental work was carried out to determine the effect of relieving the confining pressure. To this extent, internal walls were removed where possible and cut through where they connected with the external walls, similarly blockwork was removed where it was in contact with roof trusses and a strip of floor slab was removed along all edges that connected with external walls. A video camera was set up to record any movement of the external crack, which prior to commencement of works measured 12mm wide. The video showed that the crack itself closed over a period of some 10 seconds, and was barely visible along the wall one hour later. At the time of writing, the authors (Finnegan and Hawkins, 2014) point out that although no fill had been removed the damage to the structure seemed to have been halted, though a monitoring period was planned to determine if further damage was caused.

#### **4.4 The Insurance and Remediation Process**

Another problem arising from the pyrite-problem in Ireland is the number of properties that are not covered for the damage by an insurance policy, and those that are covered by Insurance have a somewhat complicated process to go through in order to claim and repair their property.

It is a requirement of the construction industry in Ireland that construction companies are required to provide indemnity to cover losses due to defects in design, construction, materials and workmanship. At the time of the housing boom in Ireland, such cover was provided by schemes such as that provided by Homebond, or through cover via insurance companies, such as the Premier Policy provided by Liberty Insurance.

The Homebond scheme was designed to cover the homeowners in the event that “major defects” appeared within the property within a 10 year period after construction. In the event that this happened, the builder would be required to make good any defects, with the Homebond system as a backup in the event that remediation was not carried out.

In the period prior to 2008, this system was not underwritten by an insurance company, and with the economic collapse that occurred in Ireland, many of the builders either did not have the financial capability to remediate their properties, or the companies ceased trading. In this event, it was required that Homebond would carry out the remediation on properties covered by their scheme.

Because of this unexpected increase in claims via the Homebond scheme, and despite various appeals and campaigns, the Homebond cover for pyrite related problems was withdrawn in August 2011 (Tuohy et al., 2012). This means that nearly all remediation currently ongoing in Ireland is provided by the Premier scheme or similar insurance policies, which operate in the manner shown in Figure 4.8.

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It should be noted in conjunction with Figure 4.8 that in order for an insurance policy to be activated on a given property, it must be shown that both the damage is serious enough at that time to warrant remediation and that there is proof that the fill is expanding and causing the damage. Fill that contains pyrite but cannot be proven to be expanding or causing damage is not covered. In many of the latter cases, monitoring is carried out on the property with secondary testing taking place at a later date. The damage must also be claimed against within the period of the insurance policy, which in the Irish cases is a period of 10 years from construction.

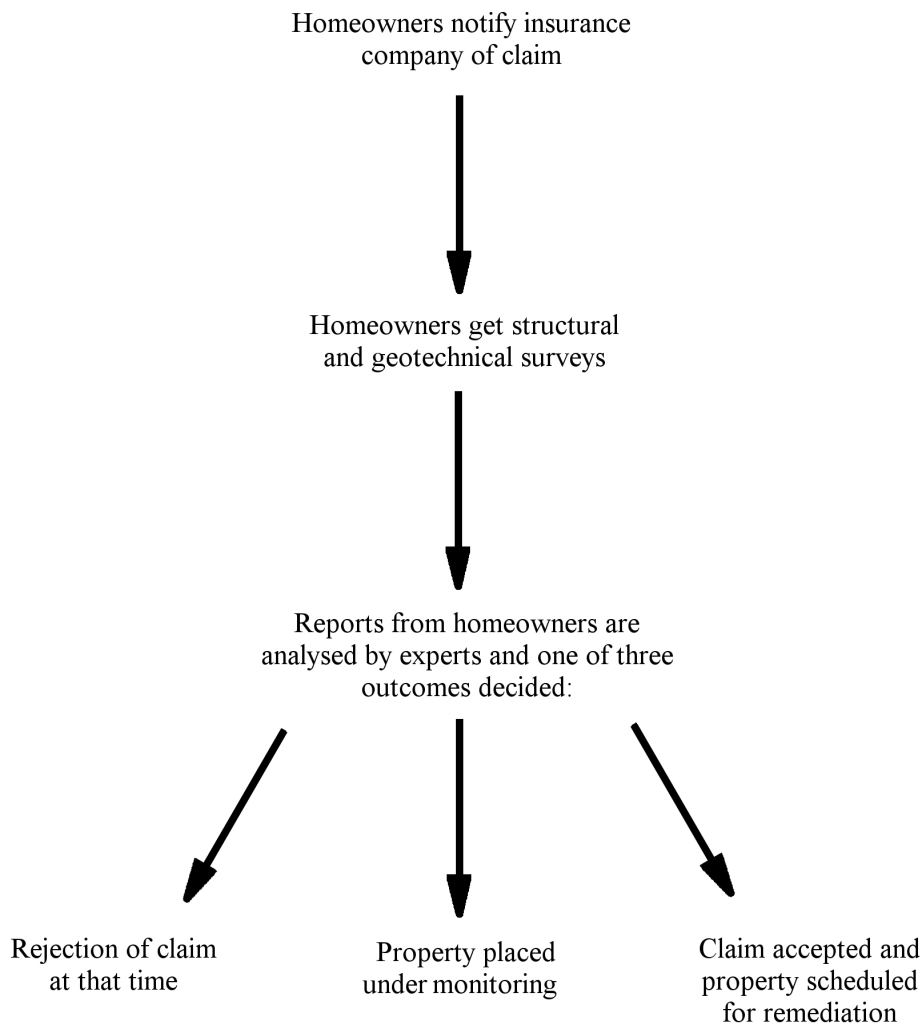


Figure 4.8: Process that is required for Irish homeowners in order to request remediation.

The initial investigation works are the responsibility of the homeowner, which is a costly

## 4 *The Pyrite Problem in Ireland*

initial outlay that will only be recovered should the claim be accepted. It can also be a lengthy process, with chemical tests results potentially taking several weeks to be returned from the date of sampling. Most often, the homeowner will organise the structural survey first, and then organise the more extensive and invasive geotechnical sampling based on the results of the first survey. The results of both surveys are then presented to the insurance company, who have them analysed to determine whether or not the damage is due to pyritic heave. If the fill material is confirmed to contain pyrite and gypsum, and to therefore be likely to have undergone pyrite oxidation and expansive gypsum precipitation, the claim is accepted.

It may be that either the fill material contains pyrite, but at levels that are not believed to cause significant damage, or cases where it is unclear that the pyritic fill is the cause of damage seen, in these cases the property will be monitored. This monitoring is also carried out at cost to the homeowner and under suggestion of their engineer – this is itself a risk, as there is no guarantee that there will be signs of progressive expansion.. If the property should shows signs of progressive damage at a later date, then further samples will be taken and tested, and the claims sequence progressed.

Properties where the claim has been accepted are scheduled for remediation as soon as possible - as this is a process that can take up to, or occasionally slightly longer than, 12 weeks, there is a continuous rolling program at the insurance company involving moving people to temporary accommodation and carrying out remediation work. The remediation process takes so long due to the fact that at the present time, the only accepted and accredited method in Ireland is to remove all of the defective material and replace it. As well as being time consuming, this is also a costly process that involves removing all furnishings to the ground floor of a property, removing the floor slab and all insulation, removing the fill that is present and replacing it with clean, inert material, re-laying a new floor slab and then replacing all furnishings and fittings to the ground floor of the property.

Although there are other suggested remediation methods mentioned within the literature, at the time of writing none of these have proven to be as effective as removing the defective fill, and so they are not approved for use in Ireland. Chapters 2 and 3 discuss some of the remediation methods that have been used in other cases of pyritic heave beneath occupied structures.

With the introduction of IS 398-1, the testing process has been more clearly defined, as is detailed in Chapter 5, with the primary points also being covered here.

### **4.4.1 The Investigation Process**

As briefly mentioned above, the first stage is a structural inspection of a given property. The procedure since 2013 with the introduction of IS 398-1 (NSAI, 2013) is a detailed desk study and structural inspection, this is discussed in Chapter 5 and will not be repeated here.

#### 4 *The Pyrite Problem in Ireland*

This structural inspection, both within the remit of IS 398-1 and for those tests carried out before its introduction, focuses on key types of damage that are solely linked to pyritic heave. Figure 4.4 shows how the cracking that is seen in many cases is not always a reliable indicator of pyritic heave. Other signs such as sticking doors and cracking on/around door frames is cautioned within IS 398-1 that it could be a sign of drying out of the wood rather than pyritic heave.

Some of the typical types of damage, those weighted more heavily in IS 398-1, are generally only seen with pyritic heave. In particular this is seen in the uplift and cracking of the floor slab – cracking associated with expansion of the fill generally radiates from a central point, i.e. the point of greatest uplift, and it is referred to as stellate or spider-web cracking. Where the floor slab can be exposed, or where it is covered with a brittle floor covering such as tiles, this cracking can be seen in many cases of heave. However, in cases where the floor covering cannot be moved, the uplift itself is used as an indicator, with factors such as a noticeable hump to the floor surface, or distinct changes in levels at joints in the floor slab being used to indicate movement.

Other factors that are given a high weighting are also those seen most commonly in cases of pyritic heave, and are mostly linked to or caused by the movement of the floor slab. This includes bowing of the plasterboard finish to ground floor walls, upward movement of the staircase, puncturing of the ceiling plasterboard at wall junctions and cracking to the outer wall of the foundation at DPC level (NSAI, 2013).

If the damage is such that it points towards the cause being expansion of the fill material, then geotechnical sampling is carried out, Figure 4.9 shows this process as used to take samples from a property tested in 2009. Since that time the sampling process has been modified and improved, with a larger area of the slab removed and the damp proof membrane cut and peeled back to the sides of the hole rather than removed as shown in Figure 4.9.

The geotechnical sampling first involves coring a hole through the floor slab and any fixed floor finishes in an area of the property indicated by the floor level survey to be undergoing uplift (Figure 4.9a). Once the hole has been cored, the section of floor slab is removed, along with the insulation such that the damp proof membrane is revealed (Figure 4.9b).

At this stage, as stated above, the damp proof membrane is cut and peeled back to sides of the hole such that the sand blinding (Figure 4.9c) and fill material (Figure 4.9d) can be accessed and sampled. Factors such as the thickness of the concrete slab, the presence of sand blinding and the depth of the fill are then recorded, the sample of fill is removed and sent for testing.

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Figure 4.9: The process of coring through the floor slab and taking samples for geotechnical testing.

According to IS 398-1, the samples are subjected to a geological analysis and are chemically tested, with the addition of XRD, thin section analysis and water absorption depending upon the damage condition rating of the property. Prior to the introduction of IS 398-1, the tests were generally the same, although both XRD and thin section analysis were more likely to be carried out no matter the level of damage displayed in the property. These tests are discussed briefly below.

### **Geological Analysis**

The purpose of this stage of the testing is to determine whether or not the petrographical make-up of the material contains rock types associated with pyritic heave. This is typically fine-grained weak or laminated sedimentary rocks such as mudstone.

### **Chemical Testing**

In order to determine the concentration and composition of the various sulphur species that are involved in the reaction process, the following tests are carried out on a representative sample of the material taken from the property.

- Total Sulphur
- Acid-soluble Sulphate
- Water-soluble Sulphate

All tests are specified to be carried out according to the TRL report (Reid et al., 2005), and from these values factors such as the current and original levels of pyrite in the fill material can be calculated.

### **XRD Analysis**

In order to obtain a detailed analysis of the elements and minerals present in the fill material, and their relevant quantities and phases, XRD analysis is carried out on a representative sample of the fill material. It is specified that testing should be carried out in the 2-theta ( $2 - 65^\circ$ ) range, with mineral phases compared to those in the International Centre for Diffraction Data database, with the full diffractogram shown in the final report. It also mentions the limitation to the detection limits, which, depending on the minerals present, is between 0.5 and 2%.

### **Thin Section Analysis**

This technique is used to support the other tests, by showing elements such as weathering of the rock particles, the presence and distribution of pyrite, and the presence and habit of secondary minerals such as gypsum. IS 398-1 specifies that the thin sections should account for all of the main lithologies present in the fill, use oil rather than water during sample preparation and should be studied under at least 500 times magnification in order to study the fine-grained pyrite. Examinations should be carried out with both transmitted and reflected light, and both images and a written report should be produced.

Figure 4.10 shows example thin sections taken from a sample of fill from an Irish property. Figure 4.10a shows oxidised pyrite crystals, for example those at grid point F/G3 and E5. Figure 4.10b shows a sample from the same property with crystallisation of gypsum (the mottled pale-grey material) along a lamination in a mudrock particle. This lamination runs from grid point B1 to G6.

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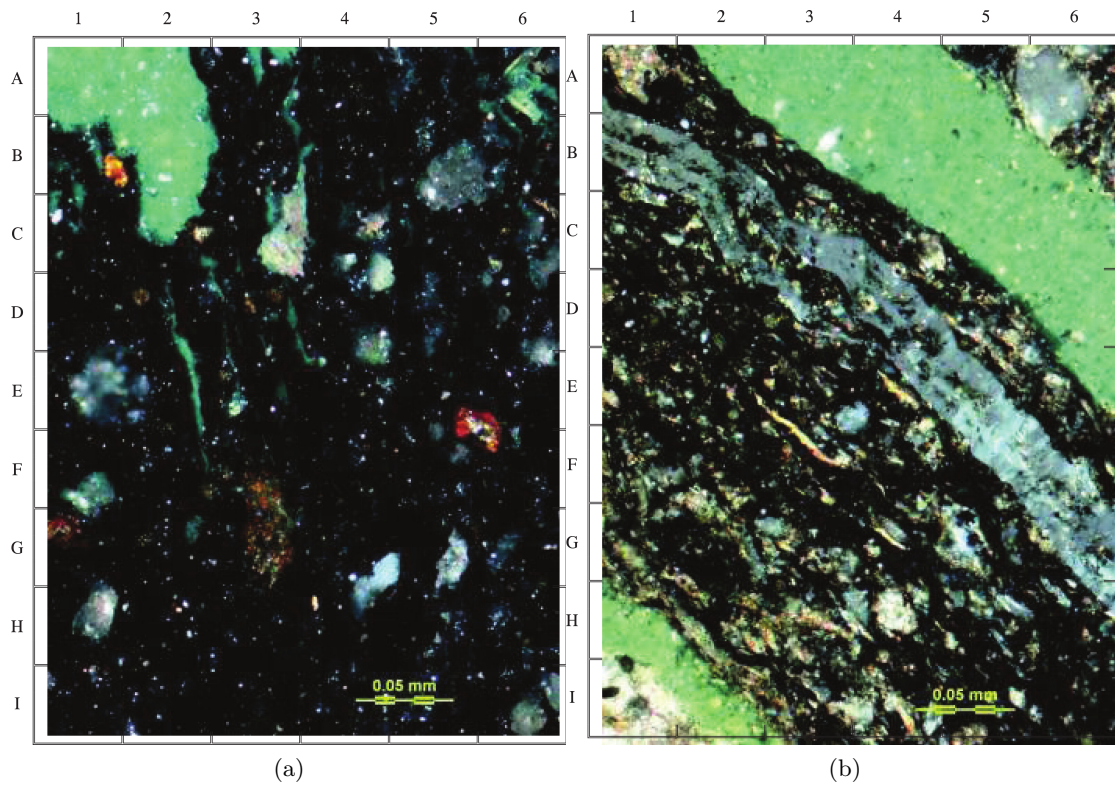


Figure 4.10: Example thin sections of pyritic mudrock, the green material is the resin used in the preparation of the samples (used with permission of Aidan O’Connell associates).

### Water Absorption

The standard specifies that this test should be carried out to IS EN 1097-6, and it is an indication of how well the fill material will absorb and transmit water. This then signifies the ability of the fill to allow water and air to access the pyrite and progress the oxidation reaction. It is given as a percentage of the dry mass of the sample.

### 4.4.2 The Remediation Process

As discussed above, the only currently accepted remediation process, at least in Ireland, is to remove the defective fill material. This is a lengthy and expensive process that requires the homeowner to vacate the property for up to 16 weeks (Tuohy et al., 2012) whilst work is being carried out. This process is outlined below.

- A detailed condition survey of the property is carried out, during which the damage due to pyritic heave is recorded. At this time, the floor slab will often be exposed



#### 4 The Pyrite Problem in Ireland

to record cracking that is present, cracking to wall finishes will be recorded and a detailed floor level survey carried out if required.

- Services are isolated and the strip out of the ground floor begins. This involves removal of kitchen units, doors, wall finishes and staircases. Any other defects to the ground floor structure, such as construction defects, are also recorded at this time/. Figure 4.11 shows the damage to a floor slab during the strip out process.



Figure 4.11: Stellate cracking of floor slab recorded during the remediation process (used with permission of Aidan O'Connell associates).

- Internal non-load bearing walls are removed at this stage before the floor slab is cut and removed. The insulation and damp-proof membrane are also removed at this stage, along with service pipes placed below the slab.
- All defective material is removed until the natural bedrock material is uncovered, as shown in Figure 4.12. The material is sampled if necessary before being removed from site and disposed of, and the rising walls are cleaned and inspected for any signs of sulphate attack.



Figure 4.12: Removal of defective fill material from an Irish property, showing the natural material at the base of the foundation (used with permission of Aidan O'Connell associates).

- Once all defective fill material has been removed and all surfaces are clean of material that may react in future, tested and certified fill material is placed and compacted, as shown in Figure 4.13. The services such as the radon sump are re-installed and a new damp-proof membrane placed.
- At this point, the steel reinforcement is placed and the concrete for the new floor slab is poured (Figure 4.14). It is worth noting that in many cases, the original ground floor slab did not contain reinforcement, as is revealed when coring through the slab in order to take the samples for the geotechnical tests.

#### 4 The Pyrite Problem in Ireland



Figure 4.13: Replacement of fill material and radon sump. (used with permission of Aidan O'Connell associates).

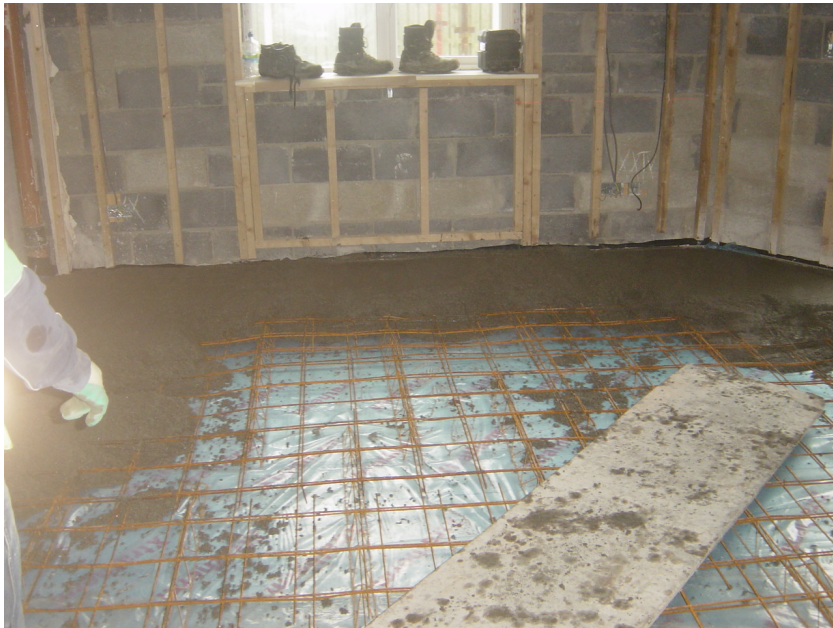


Figure 4.14: Placement of steel reinforcement and pouring of fresh concrete floor slab. (used with permission of Aidan O'Connell associates).

- Once the floor slab is in place, other services are reinstated, damage is repaired to any blockwork walls and structures such as non-load bearing walls and the staircase are replaced.
- The final stage is to replace all dry lining, fixtures and fittings such as floor coverings, decoration and kitchen units, and to carry out a final visual inspection.

## 4.5 Discussion

The presence of basinal Carboniferous age rocks under much of the Dublin area leads to a predisposition for large amounts of variation within a given area and so within a given quarry. This variation, along with the presence of rock types such as the Tober Colleen Formation that is known to contain pyrite, would require that detailed testing be carried out on rock material before it is used.

Official standards in place in Ireland at the time of the housing boom detail the quality of the rock material that can be used as fill primarily by size/grading. However, there are several standards that provide limits for sulphate levels for any material used within 0.5 metres of buried concrete. Considering the placement of the fill material beneath domestic floor slabs, and in direct contact with rising walls, it can be assumed that these limits should have been followed during construction of the properties.

It is currently being debated as to whether this responsibility to test the material should have been carried out by the supplier at the quarry, or by the builders that used the material. However, common sense would imply that both parties had a duty of care to ensure that testing had been carried out at some stage of the process to guarantee the material was suitable for use.

As briefly mentioned in Chapter 3, remediation in Ireland involves the removal of the defective fill material from below the floor slab. Prior to this, however, investigations are required to determine the type/extent of structural damage to the property, and the chemical composition of the fill material. This is carried out, as standard for all claims, at cost to the homeowner.

With the introduction of the new Irish Standard IS 398-1, this process is intended to be streamlined. A survey will initially determine, based upon the key signs of pyritic damage discussed in Chapter 3, whether the damage seen to a property is likely to have been caused by pyritic heave. This will then allow the property to either be monitored or recommend that the fill material be tested. The standard and its application is discussed in the following Chapter.

# 5 A consideration of IS398 for Classification of Pyritic Damage

## 5.1 Introduction

As discussed in Chapter 3, Section 3.2.4, and below in Section 5.2 attempts have been made in Canada to classify the damage caused by pyritic heave to domestic properties. Systems such as this can be used for comparing the amounts of damage at a given property or development to the chemical and physical properties of a material. The aim of this being to determine whether there is one aspect of the fill material that has more of an effect on the damage than others.

Released in 2013, the new Irish Standard IS 398-1:2013 (Reactive pyrite in sub-floor hardcore material - Part 1: Testing and categorisation protocol (NSAI, 2013)) is designed as an aid to identifying properties that may be damaged due to the presence of pyritic material used as fill beneath the floor slab.

Although it is intended primarily as a tool for identifying new cases of pyrite related damage, since it was designed with consideration of the Irish properties, it can also be used to consider the damage seen in existing cases. Data such as the engineering report and site photographs exist for many of the Irish cases, and can be compared to data concerning the chemical properties of the fill, such as the geotechnical reports produced as part of the Irish insurance claim process.

The idea behind this analysis was to assign a numerical value to the damage, via the weighted analysis contained in IS 398-1, such that a graphical analysis could be performed in order to determine whether correlations exist between known chemical values and the damage level. Following on from this, a basic statistical analysis would then be carried out to confirm any correlations that appeared to be present in the graphical data.

To begin with a detailed analysis of the standard was carried out. The first reason for this analysis was that, to the author's knowledge, such an analysis had not previously been carried out since the standard was released. Secondly, the analysis would consider the effectiveness of IS 398-1 as a tool to analyse the damage to properties that had already undergone remediation and therefore had a limited amount of information available.

The analysis started by considering the different aspects of the standard and then using them to consider two test properties. The test properties were taken from different areas,

5 A consideration of IS398 for Classification of Pyritic Damage

considered two different forms of construction, and were used to determine whether there was enough data available in the structural reports and site photographs to enable a thorough analysis and provide accurate results.

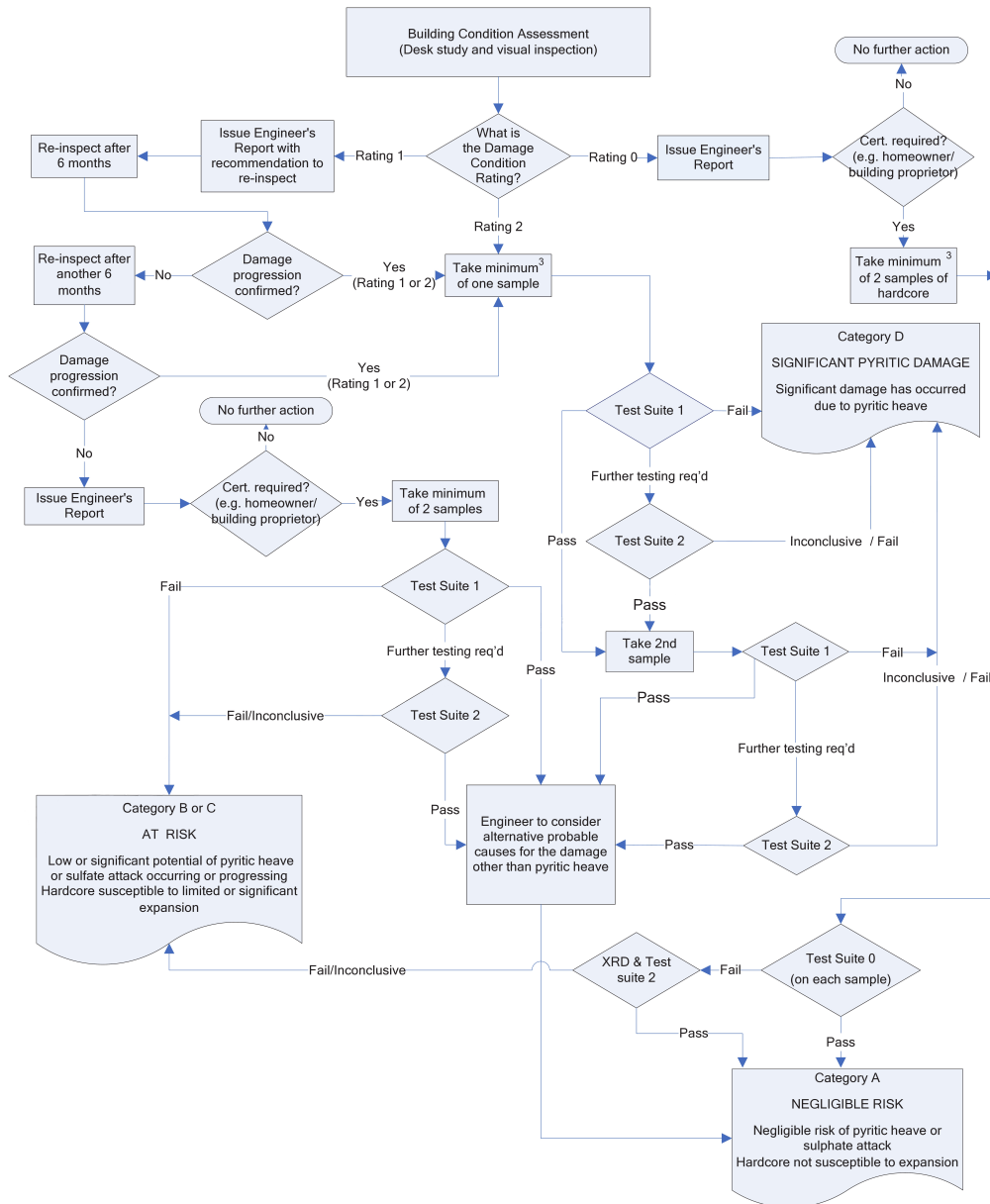


Figure 5.1: Flow chart detailing the process to be used with IS 398-1 to determine the likelihood that damage to a property has been caused by pyritic heave (NSAI, 2013).

## 5 A consideration of IS398 for Classification of Pyritic Damage

Based upon these results, a more detailed analysis was performed using the data obtained from a housing estate to the north of Dublin. This analysis considered some 103 properties and was intended to test the viability of using IS 398-1 as a tool for comparing the damage to a dwelling to the properties of the fill material.

The protocol within the standard can be summarised by the flow chart shown in Figure 5.1, and is detailed further below.

The first stage of the process is a desk study, intended to use available sources to determine the site history and local geology if applicable. This desk study should also consider available information on the the design and construction of the structure, including the location of any services, and will review the planning and certification documentation where available.

This is followed by a visual inspection of the property, which includes the details discussed Table 5.3 as well as the following:

- Details of general site features, this includes items such as the slope of the site, the presence of large trees, etc.
- Information on the history of problems with the property and any subsequent repairs. If possible dates should be attached to this information.
- A description of the construction of the structure.
- An inspection of both the interior and the exterior of the property, looking for evidence of damage due to pyritic heave. This includes details, photographs and sketches of marks of damage such as cracks, sticking doors etc.

These factors will contribute to give a Damage Condition Rating for the property, which, when used with the intended purpose of IS 398-1, will classify the property as showing significant, minor or none/aesthetic damage.

If the damage that is recorded at this stage is not severe enough to warrant moving on to the testing process, the property may undergo a monitoring period ending with a second inspection at least 6 months later.

Should the damage be such that further testing is required, IS 398-1 specifies that a minimum of 1 or 2 samples should be taken, dependent upon the damage condition rating and the size of the floor area of the property being less than 60 m<sup>2</sup>. The standard also specifies that samples should be stored in airtight containers, out of direct sunlight and at temperatures below 10°C. Should the samples need to be stored for longer than 14 days, they should be dried at 40°C before being stored as before.

The tests that would be carried out on the samples for a given property also depends upon the damage condition rating, as detailed in Figure 5.1. The test suites are summarised below:

## 5 A consideration of IS398 for Classification of Pyritic Damage

Test suite	Tests included
Suite 0	Geological inspection Chemical testing
Suite 1	As per Suite 0, plus Mineralogical analysis – XRD
Suite 2	As per Suites 0 & 1, plus Thin section analysis Water Absorption

Table 5.1: Tests included at each level as per IS 398-1 (NSAI, 2013).

The idea of this organisation of the testing is such that the most amount of information is obtained from the least amount of tests in order to still give a clear picture of whether or not the damage is due to pyritic heave.

As such, IS 398-1 specifies a series of chemical limits based upon the literature (as seen in Chapter 4) and upon the experience of experts studying the Irish cases. These are used to determine what the risk is that the material is susceptible to expansion and therefore is the cause of any heave beneath the floor slab. The principal values from IS 398-1 are shown in Table 5.2.

Parameter	Pass value	Fail value
Acid soluble sulphate	$\leq 0.2\%SO_4$	$> 0.2\% SO_4$
Water soluble sulphate	$\leq 500mg/l$	$> 1500 mg/l$
Total sulphur	$\leq 0.3\%S$	$> 1.0\% S$
Proportion of fine grained rock	$\leq 10\%$	$> 30\%$
Evidence of crystallisation	No	Yes

Table 5.2: Pass/fail values for chemical and geological tests as per IS 398-1 (NSAI, 2013).

These values, and how they relate to existing cases will be discussed below in Section 5.4.

## 5.2 The origin and design of IS 398-1

IS 398-1 currently presents the only official method of categorising damage to properties due to pyritic heave in Ireland. There is the system provided by Ballivy et al. (2002) for damage to properties in Montreal (Chapter 3), however, this focuses upon a very specific set of criteria based upon the damage seen in those properties.

The classification for the damage in Montreal considers the amount of heave of the floor slab, the amount and style of cracking, both to the floor slab and to foundation walls, and the presence of any white deposits on the slab or foundation wall that signifies the presence of secondary crystallisation of minerals such as gypsum (Ballivy et al., 2002).



## 5 *A consideration of IS398 for Classification of Pyritic Damage*

However, in the Montreal cases, they were generally considering heave of basement and garage floor slabs, which were less likely to be covered by carpet or other types of floor finish. In the case of the Irish properties, the floor slab undergoing heave belongs to the ground floor of the properties and is, more often than not, covered by some sort of floor finish. Therefore, it is frequently difficult to access the ground floor slab in these properties in order to determine if there is any cracking present.

Additionally, many of the signs of damage present as cracking to wall and ceiling finishes

In the case of IS 398-1 being applied to new cases where the property is being inspected for the possibility of damage due to pyritic heave, the standard can be applied in full from the first moment. However, as it is currently the only method specifically designed to categorise pyritic damage to Irish properties, it is of interest to see if that analysis can be applied retrospectively.

Since the confirmed cases of pyritic heave that are available for consideration have, in most cases, already undergone remediation, it is not possible to fully apply the standard for analysis. Therefore, the focus will be upon the data obtained from inspections of the properties – this includes, but is not limited to, structural surveys, photographs taken during remediation and floor-level surveys.

IS 398-1 breaks down analysis of a property into 5 main sections or element groups:

- Ground floor surface level;
- Fixtures and fittings;
- Ground floor walls and partitions;
- Ground floor ceilings;
- External.

The type of damage seen in each of these sections is then considered separately and a weighted value is assigned depending on how significant the damage is to the property. Table 5.3 shows the elements that are considered as part of IS 398-1 and the weighting applied to each element in order to determine their relevance to pyritic heave.

Although in most cases several observations from a given element group may apply to a property, only the highest value is considered when determining the overall damage rating for that property.

As briefly mentioned above, the various observations were selected and weighted according to their likelihood of being related to pyritic heave, with the more severe signs of damage being given a higher weighted value.

This weighting applies both to the categories of damage themselves as well as the observations within those categories. For example, many of the observations within the Fixtures and Fittings category, such as sticking doors and kitchen worktop displacement,

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are associated with the movement of the slab accounted for in the Ground Floor Level Surface category.

Element	Value	Observation
Ground Floor Surface Level	0	Floor appears reasonably level
	2	Local discrepancy in floor level $\leq 6\text{mm}$ in 1m
	2	Floor tiles cracked at internal threshold $\leq 2\text{mm}$ width
	3	Crack in slab/tiles $\leq 2\text{mm}$ separation
	3	Hogging of surface $\leq 10\text{mm}$
	5	Upward displacement of floor $> 6\text{mm}$ in 1m
	6	Upward displacement $> 2\text{mm}$ at external threshold
	6	Hogging of surface $> 10\text{mm}$
	8	Radial cracks in slab $> 2\text{mm}$ width
	9	Upward displacement of radial cracks $> 2\text{mm}$
10	Upward displacement at joint in slab $> 5\text{mm}$	
Fixtures & Fittings	0	Displacements not generally noticeable
	1	Internal door(s) catch at top/bottom
	2	Kitchen worktop upward displacement $\leq 6\text{mm}$ in 1m
	3	Top architrave displacement $> 5\text{mm}$ in 1m
	4	Fireplace hearth upward displacement
	4	Significant separation between door and frame
	6	External door(s) catching on floor, marks to floor finish
	6	Internal door(s) require significant force to open
	7	Upward displacement of staircase
	9	Internal door(s) won't open $> 45$ degrees
10	Internal door frames bowed $> 10\text{mm}$ in full height	
Ground Floor Walls & Partitions	0	No evidence of cracking or few cracks $\leq 1\text{mm}$ width
	1	Stress cracks at wall/ceiling junction
	2	Stress cracks at wall/wall junction
	3	Horiz/Vert cracks $> 1\text{mm}$ , $\leq 3\text{mm}$ width at junctions
	4	Diagonal crack(s) $\leq 2\text{mm}$ width over doors
	5	Several $\leq 1\text{mm}$ width
	6	Crack widths $> 2\text{mm}$ and tapered
	7	Crushing of plasterboard near slab level
	8	Bowing $> 10\text{mm}$ in full height
	9	Wall tiles/horizontal grout lines displaced $> 5\text{mm}$
10	Several cracks $> 3\text{mm}$ width	
Ground Floor Ceilings	0	No evidence of cracks or few $\leq 1\text{mm}$ width
	1	Stress cracks at wall/ceiling junction
	2	Upward displacement $\leq 4\text{mm}$ adjacent wall/partition
	3	Upward slope towards centre of room $\leq 6\text{mm}$ in 1m
	5	Many nails popped near wall junction

*Continued on next page*

Table 5.3 – Continued from previous page

Element	Value	Observation
	6	Puncture of ceiling at wall junction
	7	Plasterboard crack(s) > 3mm width
	10	Upward displacement > 4mm adjacent wall/partition
External	0	No evidence of wall cracks or few cracks $\leq$ 1mm width
	1	Stress cracks to render at DPC level
	2	Cracks at DPC lvl $\leq$ 1mm wide, no lateral displacement
	2	Diagonal/vertical cracking < 1mm width
	3	Cracks at DPC lvl $\leq$ 1mm, lateral displacement $\leq$ 1mm
	3	Diagonal crack at first floor head level > 2mm width
	4	Several cracks > 3mm width on 2 or more elevations
	7	Horiz cracking at DPC lvl, > 1mm lateral displacement
	8	Cracks at 1st floor window head level > 2mm width
	9	Continuous horiz cracking > 3mm width near soffit level
	10	Continuous horiz gap > 5mm under soffit

Table 5.3: Table used for evaluation of physical damage to a property (after IS 398-1, NSAI, 2013).

One of the key signs of heave of the floor slab is what is referred to within the Irish Standard as radial cracking, and by Ballivy et al. (2002) as star or stellate cracking. The presence of these radial cracks, especially with signs of lateral or vertical displacement along these lines, is therefore awarded some of the highest points in the floor level element, along with displacement of more than 5 mm at construction joints within the slab.

It should be noted that there are warnings within many of these sections for the engineer to be certain that the damage is due to pyritic heave rather than to some other factor. For example, in the fixtures and fitting section, sticking of internal doors or displacement of their frames is cautioned that it may be due to swelling of the timber rather than movement of the floor slab. Similarly, factors such as cracking to external concrete pathways is not included, as this may be due to thermal movement of the concrete rather than expansion of fill.

There are also some sections that may still be considered to be ambiguous in their wording. For example, within the fixtures and fittings element, it discusses “significant lateral separation of the gap between the top of the internal door and the frame”, with no indication of what should be considered “significant”.

Due to the occasional lack of information pertaining to some of the properties considered for this analysis, it was determined to be necessary to amend some aspects relating to the standard when looking at the data for the test properties. Listed below are the

differences between the standard that resulted from applying the standard to the data from the test properties.

- In many cases the floor level survey was carried out with a 1 or 1.2 m level rather than the 2 m level specified in the standard. Since the measurements in the standard, for example measuring an upward displacement greater than 6 mm, would be considered over a 2 m length, where a value of greater than 6 mm was recorded on a 1 m level, the points were awarded as for the 2 m level. The displacement seen over a 2 m length would likely be greater, although this cannot be said for certain.
- “Significant lateral separation” was taken to mean a gap equal to or greater than 5 mm.
- “Several cracks” (ground floor walls/partitions element) was taken to mean 5 or more cracks in the case of those up to 1 mm in width at their widest point, and 2 or more in the case of those greater than 3 mm at their widest.
- An addition was made to the ground floor ceilings element, to account for the number of properties showing large numbers of ceiling cracks that were around or greater than 1 mm in width, but did not meet the 3 mm width criteria. This was awarded at a similar weighting to the 1 mm width cracks in the walls/partitions element, alongside the separation of coving within the ceiling element.
- There is no mention of sulphate staining to the plinth, which can be seen on some properties. It is assumed that this has been left out as exposure to the weather would likely remove evidence of such staining. It has therefore not been added to this analysis, although it may be prudent to consider this in later uses.

Analysis of both the test properties and Development Alpha (Section 5.3) was carried out using the information that was available and could be corroborated as a primary source – this involved images and direct details in the structural reports and site photographs. Items that were only briefly mentioned within the structural reports but were not corroborated with images or detailed descriptions were not utilised at this stage. This is discussed further in Section 5.4.2.

### **5.2.1 Example – Property A**

The property discussed below, referred to as Property A, is situated on a development to the north west of Dublin city and is a timber-framed structure with an outer leaf of rendered blockwork. Surveys were carried out in spring and summer 2011, and the initial construction of the property is believed to have taken place in 2003/4.

Data for this property was somewhat limited, due to the lack of detail in the initial structural report, and the majority of photographs taken of damage to the structure were taken after remediation works had started. This means that the plasterboard

## 5 A consideration of IS398 for Classification of Pyritic Damage

coverings to most of the ground floor walls and ceilings had been removed and so signs of damage in some areas have been lost.

Each of the observations for the element groups detailed in Table 5.3 was considered based on the information available for the test property, and the analysis is presented according to element group below.

### Ground floor surface level

As the surveys for this properties were carried out long before the introduction of IS398-1, the level surveys were not carried out exactly as specified in the document. The survey at this property was carried out using 600 and 1500 mm levels, with level changes recorded as the difference along the length. Points noted from the surveys at this property include:

- Floor level changes of over 10 mm in 1.5 metres, as shown in Figure 5.2. Note that the numbers in Figure 5.2 indicate the increase in level, with the numbers at the end of the line showing the most uplift.

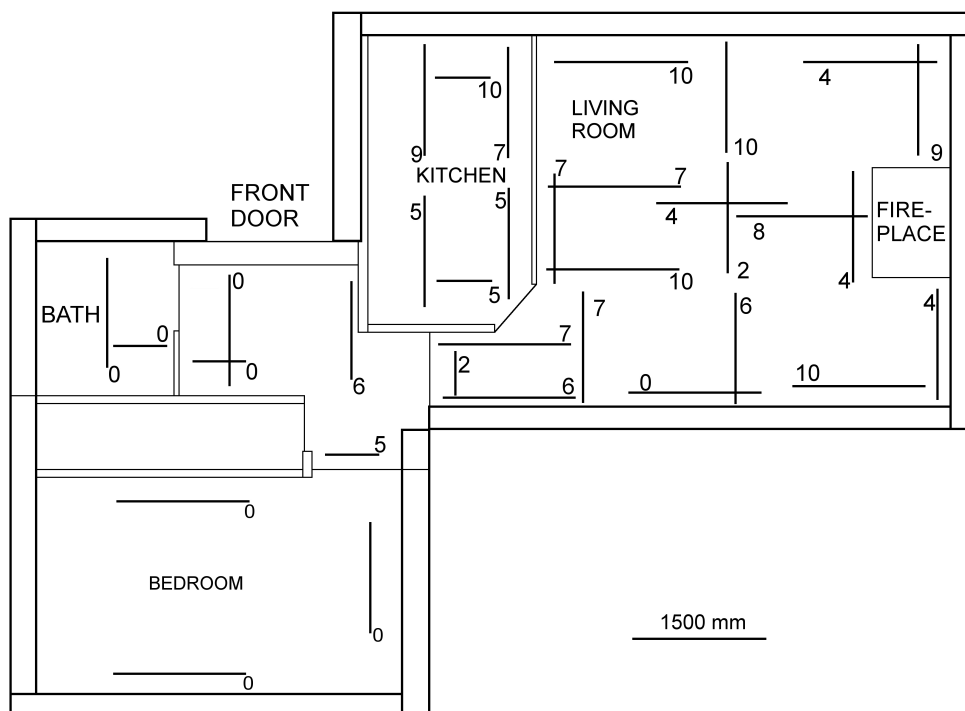


Figure 5.2: Floor levels from the floor level survey at Property A with permission from Aidan O'Connel Associates).

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- Radial cracks to the floor slab that are greater than 2 mm in width.
- Upward displacement greater than 5 mm at construction joints in the slab, as shown in as shown in Figure 5.3.



Figure 5.3: Uplift of the floor slab and displacement of skirting boards at Property A, uplift was recorded at around 6 mm in this area (with permission from Aidan O’Connel Associates).

The total “score” for this section is 10/10.

### Fixtures and fittings

As previously mentioned, the amount of detail available for this section is limited. Remediation had started before detailed photographs were taken, and the structural survey first carried out did not include details such as doors that were catching or other similar damage.

Points that were able to be noted are:

- Upward displacement of skirting boards by up to or more than 5 mm, as shown in Figure 5.3.
- Gaps present around window frames up to 5 mm in width, as shown in Figure 5.4.



Figure 5.4: Gapping at top of window frame in the living room of Property A (with permission from Aidan O’Connel Associates).

The “score” for this section is evaluated as 6/10, although it is likely that it would be higher with more detailed photographs pre-remediation considering the amount of floor heave at the property.

#### **Walls/partitions on ground floor**

This was another section where information was somewhat limited, although there was enough evidence to note the following points:

- Cracking to walls, including cracks above doors and stress cracks at wall junctions, the latter of which is shown in Figure 5.5.



Figure 5.5: Cracking of plasterboard at wall/wall interface in Property A (with permission from Aidan O'Connell Associates).

- Uplift of partition walls was noted during remediation once the plasterboard had been removed, shown in Figure 5.6.





Figure 5.6: Uplift of an internal partition wall at Property A by approximately 5 mm (with permission from Aidan O’Connel Associates).

The “score” for this section was evaluated as 6/10, although as stated above, it is likely that a higher score would be awarded if more information were available.

### **Ground floor ceilings**

This is the other section in which information was limited to that available within the structural report, although this was not necessarily considered to be a reporting priority prior to IS 398-1. However, this does contain pictures of damage to the ground floor ceilings, including Figure 5.7 below showing significant cracking to the ceiling.



Figure 5.7: Significant cracks to the plasterboard finish of the ground floor ceiling at Property A (with permission from Aidan O’Connel Associates).

The “score” for this section is taken as 7/10.

### **External**

There was a large number of images available covering the damage to the exterior of this property, as the damage was unusually noticeable to the exterior faces of the structure.

Points noted for this property included:

- Cracking to the external render – in the crack shown in Figure 5.8, the crack is greater than 5 mm in width, and shows more than one axis of movement.



Figure 5.8: Cracking to the render around the living room window at Property A (with permission from Aidan O'Connel Associates).

- Gapping around window frames and lintels
- Cracking to the plinth around DPC level, as shown in Figure 5.9



Figure 5.9: Damage to external walls of Property A (with permission from Aidan O'Connel Associates).

- Gapping at soffit level, greater than 5 mm in places as shown in Figure 5.10



Figure 5.10: Gap at soffit level at Property A (with permission from Aidan O’Connel Associates).

The “score” for this section is 10/10 due to the soffit gap.

Table 5.4 summarises the values for each section for Property A.

<b>Section</b>	<b>Score</b>
Ground floor surface level	10/10
Fixtures & Fittings	6/10
Ground floor walls/partitions	6/10
Ground floor ceilings	7/10
External	10/10
Total	39/50

Table 5.4: Summary of the values assigned to each section after analysis of Property A.

It is mentioned above that the data available through the structural report and site photographs is limited for certain sections of the analysis. Since this was a common problem encountered, as detailed in Section 5.3, the analysis progressed with the data available. Methods of addressing this problem are discussed in Section 5.4.2.

According to the analysis as directed within IS 398-1, and as discussed in Section 5.2.3 below, this property has a damage condition rating of 2.

### 5.2.2 Example – Property B

Property B is a 2-storey mid-terrace dwelling located in the Rush area to the north of Dublin. The structure itself is of blockwork cavity construction, with external finishes in both brick and render. Inspections were carried out in summer 2009, with an estimated construction date of mid-2004.

Data for this analysis was taken from both the initial structural inspection and from photographs taken during the remediation process. As with Property A the damage is divided by each element and discussed below along with photographs showing the damage as it presented in the property.

#### Ground floor surface level

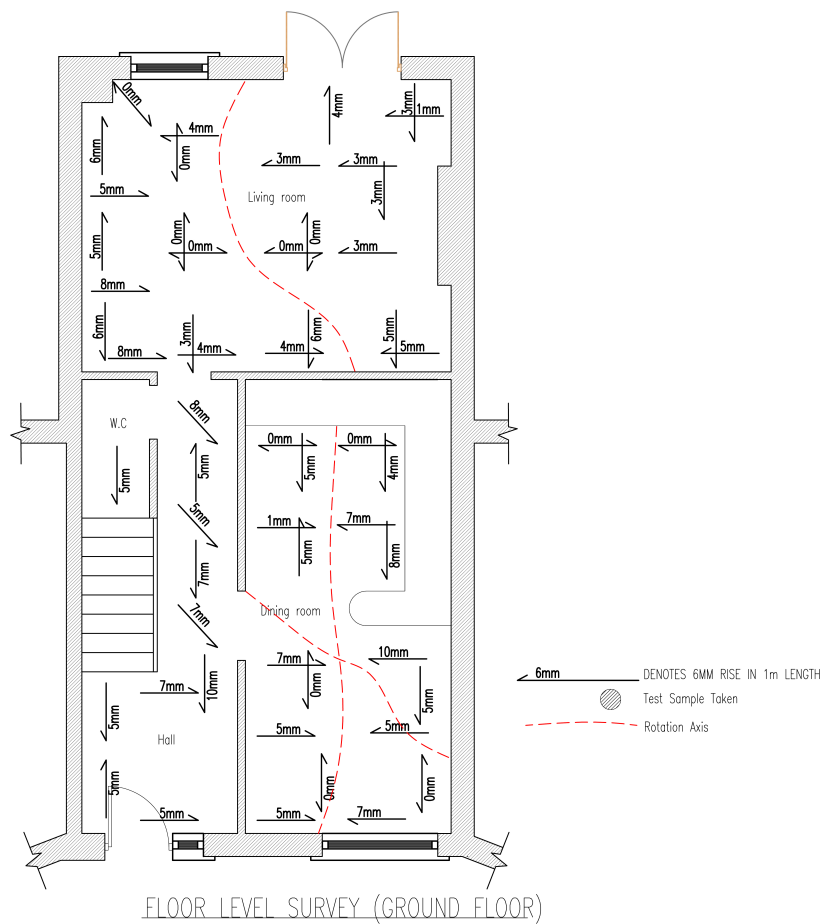


Figure 5.11: Floor levels from the floor level survey at Property B (with permission from Aidan O’Connell Associates).

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Points noted within this element were:

- Cracks were noted to the floor finishes in the hallway and dining room, the general width of these cracks was less than or equal to 1 mm.
- Measurements of the floor level were taken using a 1000 mm level, with a maximum rise of 10 mm over that length (Figure 5.11).

The total “score” for this section is 6/10.

### Fixtures and fittings

The amount of information relating to this section was incredibly limited, however, the structural report discussed signs of severe cracking in and around the stairwell.

This damage could have been caused by the upward movement of the floor surfaces around the stairwell. However, based upon the similarity of the damage in this case to other cases where the stairwell had moved, this was classed as indication of movement of the staircase.

The total “score” for this section is 7/10.

### Walls/partitions on ground floor

The cracking to the walls within this property was significant. There was cracking to walls on both the ground and first floors, and although the latter is not counted as part of IS 398-1, the level of cracking seen to the first floor landing area can be taken as a sign of upward movement of the stairwell and ground floor partition walls.

Examples of the cracking seen at this property are shown in Figure 5.12.



Figure 5.12: Cracking to ground floor walls at Property B (with permission from Aidan O’Connel Associates).

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The total “score” for this section is 10/10.

### Ground floor ceilings

The structural report for the property made mention of large amounts of cracking to the ceiling finishes, although pictures were of poor quality.

The total “score” for this section is 6/10.

### External

Damage seen throughout this site was generally low on the external finishes of the properties. Property B was no different, with only minimal cracking to some areas of the render noted during the inspection.

The total “score” for this section is 2/10.

Table 5.5 summarises the values for each section for Property B.

Section	Score
Ground floor surface level	6/10
Fixtures & Fittings	7/10
Ground floor walls/partitions	10/10
Ground floor ceilings	6/10
External	2/10
Total	31/50

Table 5.5: Summary of the values assigned to each section after analysis of Property B.

### 5.2.3 Damage Condition Rating

IS 398-1 then takes the highest value for each element group and awards a damage condition rating. The 5 element groups detailed above are also then weighted according to which is a more significant indicator of pyrite related heave and subsequent damage. The ways in which the sections are weighted for the final rating are detailed in Table 5.6 below.

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<b>Evaluation</b>	<b>Damage Level</b>	<b>DCR</b>
Attribute value 2 or less in any combination of element groups	None/Aesthetic	0
Any combination of attribute values not in DCR 0 or 2	Minor	1
Attribute value 6 or more in 3 or more element groups. OR Attribute value 6 or more in the ground floor surface level group plus and attribute value of 4 or more in two other element groups.	Significant	2

Table 5.6: The three damage levels and damage condition ratings that can be applied to a property based upon the site investigation (from NSAI, 2013).

It should be noted that a damage condition of 1 is intended to cover those properties that do not fall into either category 0 or 2, and as such is not defined in IS 398-1 other than as shown in Table 5.6.

Property A has a total value of 39/50, and a DCR of 2 that is classed as significant damage, and is one of the more substantially damaged properties recorded at that time. Property B has a total score of 30/50, which also corresponds to a Damage Condition Rating of 2, significant.

As discussed in Section 5.1, and shown in Figure 5.1, this rating determines what the next stage of the process will be. A rating of 0 effectively says that it is unlikely the damage has been caused by pyritic heave, and so no samples will be taken. A rating of 1 usually leads to the property being monitored for 6 months before another inspection is carried out, whilst a rating of 2 leads to the sampling and testing procedure being approved.

Under this system, both properties A and B would have proceeded to the testing phase, with the damage alone saying that the property was likely to have been suffering from pyritic heave.

Although it is not discussed further here, as it is not considered to be relevant to the purpose of this study, chemical testing on the fill at both properties confirmed the presence of both pyrite and gypsum. The fill in both cases was determined to be undergoing pyritic expansion and so both properties were accepted for remediation. As previously mentioned, at the time of this study, this remediation had already occurred.

### 5.3 Analysis – Development Alpha

As discussed briefly at the beginning of this chapter, the purpose of this analysis was to determine whether it would be possible to use IS 398-1 to classify the damage to existing



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cases of pyritic heave and to give a numerical value to represent that damage. Following on from that, the numerical value would be used to compare the damage for a given property to key factors such as the chemistry of the fill material.

For the intended use of IS398-1 (discussed in Section 5.1) the damage condition rating is the requisite value produced at the end of the analysis. However, in order to better compare the damage and the properties of the fill, a damage condition rating between 0 and 2 is too limited in scope.

Instead the values for each section of IS 398-1 were added together to give a value out of a possible maximum of 50 for each property. This is referred to as the Damage Rating (DR).

The idea behind using this rating for a given property is to try and compare this numerical description of the damage to known properties of the fill to determine whether there is a distinct link between any property and the amount of damage recorded.

An initial sub-group of properties were analysed using the factors set out in IS 398-1 and discussed with test properties A and B. The properties in this second round of analysis all came from a single development and included those that had been approved for remediation, those that were undergoing monitoring, and those that had been refused for remediation. However, this small number of properties was not considered to be large enough to give a conclusive idea of any relationships between damage and material properties. To this end, the data for the entirety of that development were considered, some 103 properties.

The data available for each property was, at minimum, a floor level survey, structural inspection information, and the chemical data for the fill material. Some properties included, in addition to this, XRD data, geological data and detailed photographs taken during remediation.

Although the delivery information is not available for all properties, it is believed that the fill material for this development was provided by one company, and was sourced from a single quarry.

The properties on this development include ground floor apartments, 2 and 3-storey terrace properties, detached and semi-detached 2-storey houses and dormer bungalows. Although exact completion dates for each property is unknown, all were constructed between 2003 and 2005.

Data for each of the properties was compiled, and a DR out of 50 was calculated, based on the available photographs and details recorded within the structural survey. The higher the value out of 50, the greater the level of damage seen at a given property. This data was then compared with factors such as the age of the property, chemical data obtained from tests carried out on the fill material and values calculated from the available data.

It should be noted that only limited information is available for properties inspected in

2008–9 when the extent and cause of the pyrite problem was still unclear. In particular, there is an absence of information relating to fixtures/fittings, beyond the occasional mention of internal doors catching on floor surfaces. These cases are discussed in more detail in Section 5.4, as are the methods used to modify the data in order to obtain better information.

Additionally, no account is taken of any construction defects that may have been present in the structure. Although the data for some of the properties included details about defects, for those that contained no such information it is impossible to tell whether this indicates that no defects were present or whether there is simply no information available. This is also covered further in Section 5.4.

Figure 5.13 shows the relationship between the DR and the chemical properties of the fill material determined during the testing phase. These are the key values discussed in Chapters 2 and 4 when considering, from a geochemical point of view, the potential for a material to be subject to pyritic oxidation.

The values in Figure 5.13 were selected for the initial comparison because they are available for all of the properties in Development Alpha, as well as being key chemical values for pyritic fill material discussed both in this study and in the relevant standards.

These are the values included in the standards when discussing limiting values for material in contact with buried concrete as regards limiting sulphate attack and pyritic expansion. These standards and the limits are discussed in more detail in Chapter 4, however, the ones referenced here are those presented in IS 398-1 (NSAI, 2013).

The red line in Figures 5.13b & c represent the limiting values for each of those chemical values according to IS 398-1. From this it can be seen how many of the properties exceeded the limits that constitute a fail value for the fill. The limiting value for the WSS of the fill is not shown on the graph because the value in IS 398-1 for a fail in this category is 500 mg/l, a figure which all of the properties exceeded where this information was available.

It should be noted that values in the graph in Figure 5.13c, the total sulphur, if multiplied by 1.87 gives the current pyrite content. Although this is not directly referred to in the graphs in the following analysis, it is used to calculate the percentage of the pyrite within the fill that has been oxidised, as shown in Figure 5.14.

The horizontal line in Figures 5.13a–c indicates the limiting values for each chemical value as defined by IS 398-1 (NSAI, 2013) and detailed in Table 5.2. This indicates the number of properties where the fill would be classed as failing the IS 398-1 limits.

Initially Figure 5.13 seems to show little obvious correlation between any of the values and the DR value for the properties of this development. However, some correlation can be seen between the damage and the water soluble sulphate (WSS), with higher WSS values linked to higher levels of damage.

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Further refinement of this data is discussed in Section 5.4, and the statistical covariance is discussed with reference to Table 5.7.

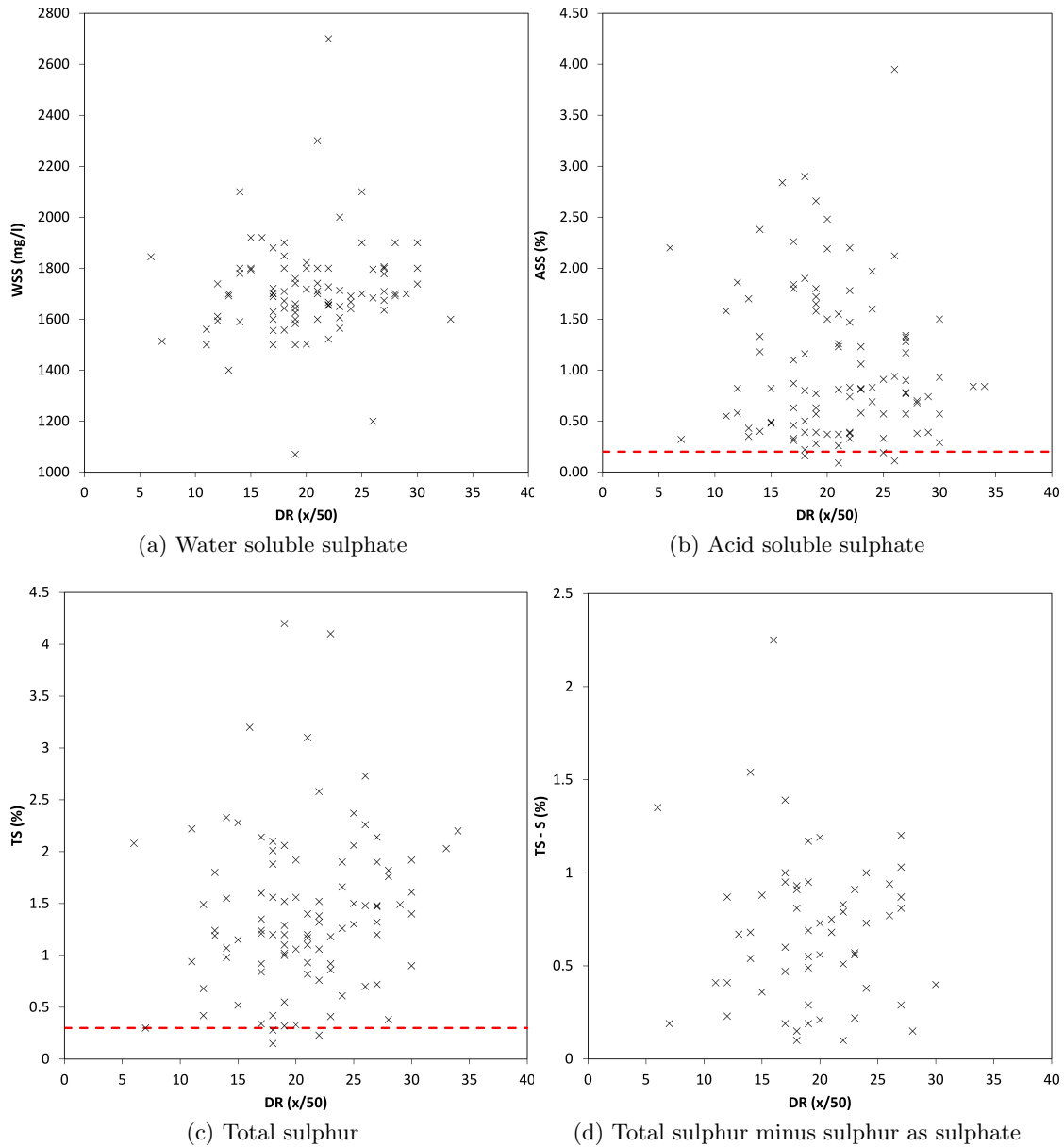


Figure 5.13: The relationship between the key chemical values and the damage rating for Development Alpha.

From the values determined via the chemical testing, it is possible to calculate other factors that give a better idea of the geochemistry of the material. As detailed in the

## 5 A consideration of IS398 for Classification of Pyritic Damage

TRL report (Reid et al., 2005), factors such as the total potential sulphate and the oxidisable sulphides can be calculated (detailed in Section 4.2) as shown in equations (5.1) and (5.2).

$$TPS = TS \times 3 \quad (5.1)$$

$$OS = TPS - ASS \quad (5.2)$$

Where TPS = total potential sulphate (% SO<sub>4</sub>), OS = oxidisable sulphides (% SO<sub>4</sub>), TS = total sulphur (% S), and ASS = acid soluble sulphur (% SO<sub>4</sub>).

From these values, it is possible to calculate the current percentage of pyrite present in the material and, when combined with the total sulphur values, also the original percentage of pyrite that was present in the fill material.

$$CP = OS \times 0.623 \quad (5.3)$$

$$OP = TS \times 1.87 \quad (5.4)$$

Where CP = current pyrite (%), and OP = original pyrite (%). These values and their relationship to the damage rating are shown in Figure 5.14.

Since the pyrite is directly linked to the expansion of the material, it would be expected that there should be a link between the original pyrite level and the amount of damage. Although this is hinted at in Figure 5.14a, it isn't truly clear.

There are similar hints of correlation in Figure 5.14b, where it would be expected that the more pyrite has been oxidised, the further into the reaction process it is, and so the more damage there should be.

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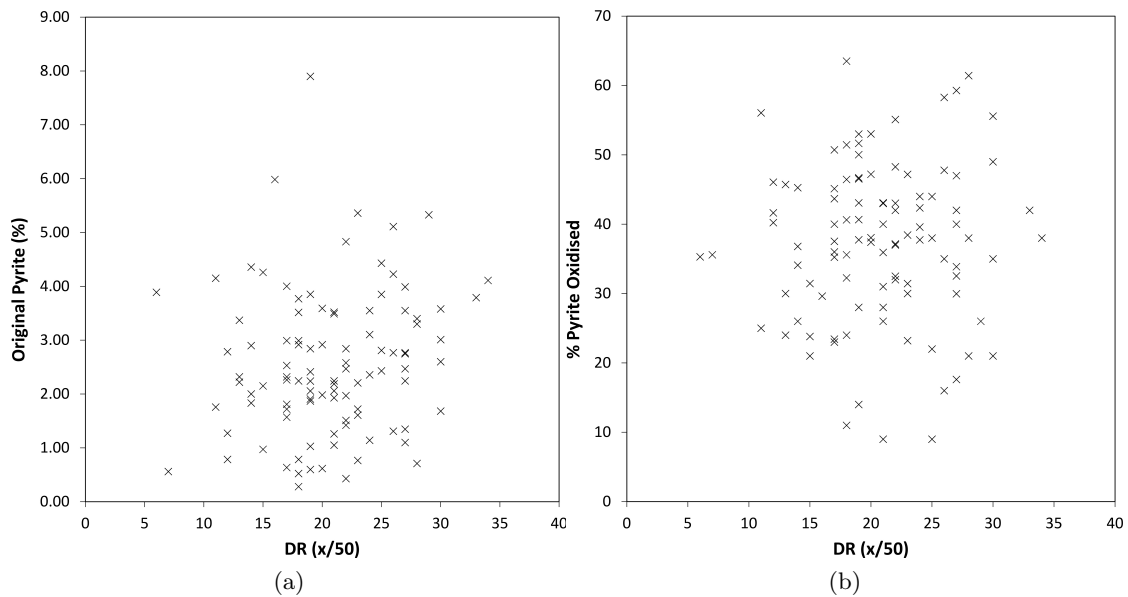


Figure 5.14: Comparison of the damage rating to both the original level of pyrite and the percentage of that pyrite that had oxidised by the time samples were taken.

However, there are several key assumptions made about the expansion process in both of these statements. Primarily, it is assumed that all of the pyrite is available to react. As discussed in Chapter 8, this is not always the case when considering larger mudrock particles, especially towards the start of the reaction process when breakdown of the material has not yet started.

Additionally, it assumes that gypsum produced by oxidation of the pyrite will always precipitate in a manner that leads to expansion of the material. Chapter 8 again discusses how the amount and rate of expansion is linked to the compaction density, although this can be summed up simply by the note that a higher compaction density leads to less void space and so more expansion that is visible in the system.

There is also the percentage of laminated material that has an effect on the amount of expansion and therefore the amount of damage that might be seen at a given property. The occurrence of precipitation along the laminations causes both expansion of the material as a whole, as well as improving access for moisture and air to continue the reaction. It would also be expected that factors such as the temperature and moisture content would have an impact on the values obtained.

In order to better consider the damage with respect to the overall reaction process, time is a factor that must be considered. With respect to this, we can consider the age of the property and the reaction rate – in this case the amount of pyrite oxidised since the

## 5 A consideration of IS398 for Classification of Pyritic Damage

property was constructed. As stated above, these dates are not accurate to more than a six month period, this lack of precision should be taken into account when considering the below data.

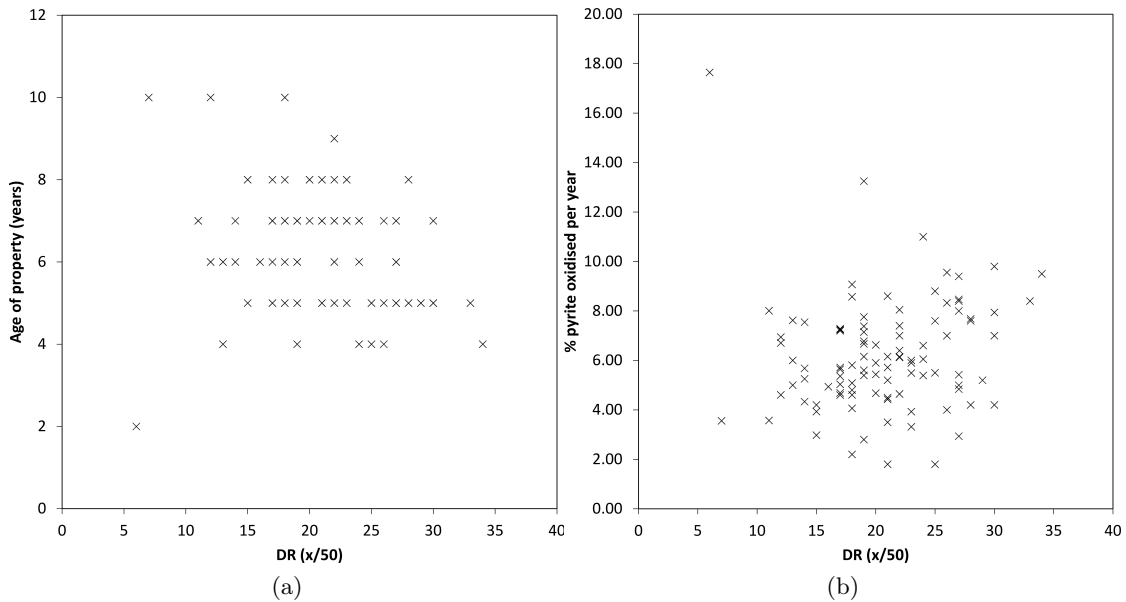


Figure 5.15: Comparison of the damage rating to both the age of the properties and the rate of pyrite oxidation.

As stated above, it is assumed that there is a link between the age of the property and the level of damage – this being based on the idea that as the reaction progresses, more pyrite is oxidised and correspondingly more expansion should occur. However, this is not reflected in Figure 5.15a, although as stated above, there is a lack of precision in this part of the data.

Instead, we can consider the age of the property in terms of the reaction rate – the amount of pyrite that has oxidised per year since construction (Figure 5.15b) and the rate of expansion (Figure 5.17).

Figure 5.15b shows much more correlation than previous graphs, showing the anticipated link that the more pyrite is oxidised per year, the more damage is caused. However, there is still a large amount of scatter in the data.

As said above, there are a number of key assumptions made when analysing the data, and there are some factors that are known to have a significant influence, but the data to account for them is not available. These factors include compaction density, the presence of structural defects, and the amount and accessibility of the pyrite. These factors are discussed in more detail in Section 5.4.

5 A consideration of IS398 for Classification of Pyritic Damage

In order to further consider the apparent poor correlation shown in the figures above, and to confirm the link between the level of damage seen and the expansion of the fill, the DR was compared with the amount of uplift seen at the properties. Since the uplift of the slab is the principal cause of much of the internal damage, it would be expected that there should be correlation between these two factors.

It should be noted at this point that there is a risk of autocorrelation between these two factors, since the uplift of the slab is accounted for in the initial assessment under IS 398-1.

However, as shown in Table 5.3, the uplift of the floor slab is measured in terms of displacement at cracks or joints and as a general notation that any hogging of the floor is greater than either 6 or 10 mm. Since it is not accounted for as a specific number, it is considered possible to compare the two with minimal interference from any autocorrelation.

To this extent, the floor level survey of each property was considered, and the largest uplift value identified – this being the maximum value, in mm, obtained along a length of 1.5 m. These are single point values that show the maximum uplift of the slab rather than the uplift rate or range.

Figure 5.16a shows the amount of damage compared to the largest single point of uplift in each property as shown in the floor level survey.

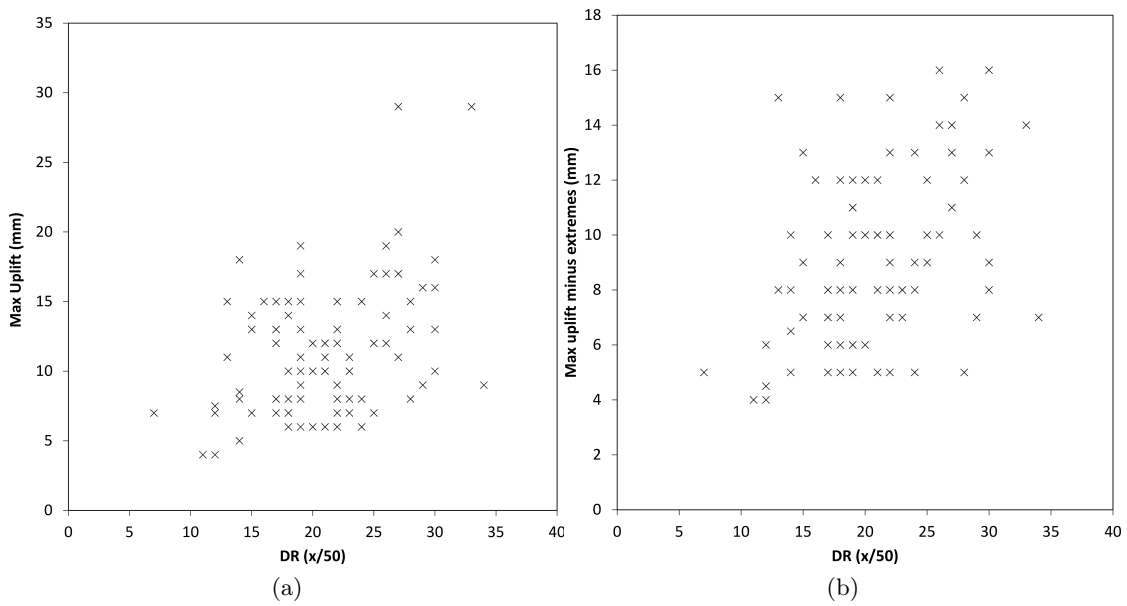


Figure 5.16: Comparison of the damage rating to both the maximum amount of uplift seen in a property, and the maximum when extreme, one-off values are removed.

## 5 A consideration of IS398 for Classification of Pyritic Damage

It was noticed during this section of the analysis, that in some cases the maximum value was significantly greater than the next highest value or values. In some cases this difference was greater than 10 mm, and it was not certain that the measurement was accurate.

In order to reduce this possibility for error, the highest uplift value for each property was taken as the highest value that was recorded at least twice within each property. This is shown in Figure 5.16b.

As anticipated, there is a more obvious correlation between the amount of uplift of the floor slab and the level of damage seen in Figure 5.16. Although scatter is still present, likely caused by the influence of factors as discussed above, there is still a link between the two variables. This is discussed further in Section 5.4 below

Following on from this, the damage was compared to the uplift rate – the maximum uplift over the length of time between construction and inspection of the property.

The uplift value used for this section of the analysis was the maximum uplift minus any extreme values as depicted in Figure 5.16b. The age of the property is that as described in Figure 5.15a, which is again only accurate to within 6 months.

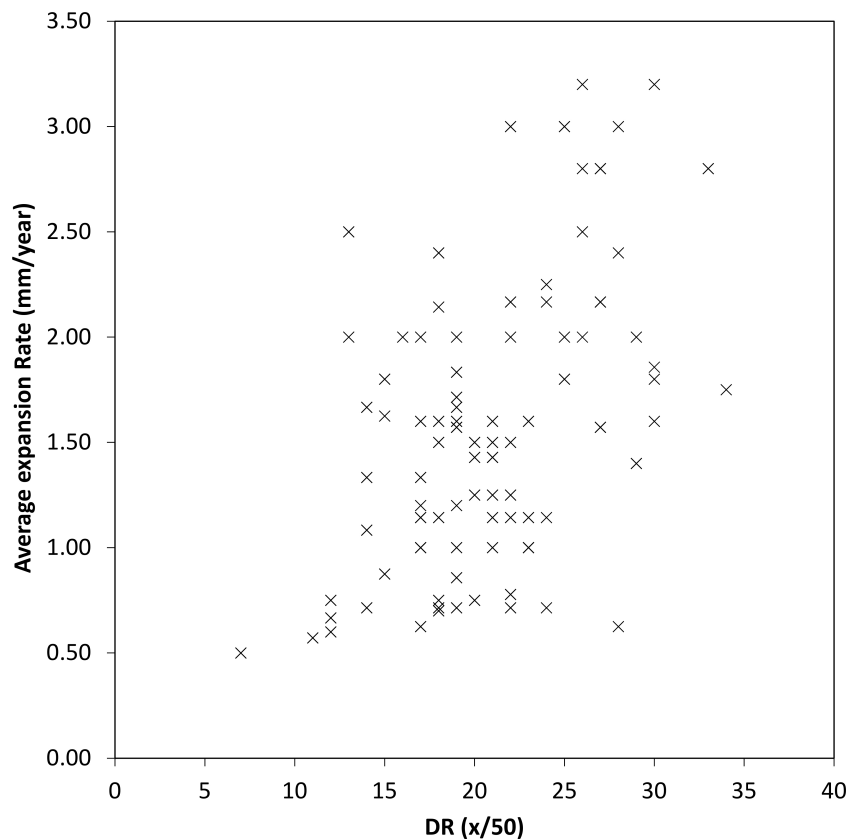


Figure 5.17: Link between the expansion rate and the damage rating.



This again shows correlation between the two values, although there is still a fair amount of scatter seen within the data.

In order to better understand the amount of variation in the data, and consider how important any correlation was, a simple statistical analysis was applied, as is discussed below.

## 5.4 Discussion

In order to consider the degree of correlation present between the various factors and the Damage Rating for the properties in Development Alpha, a Pearson correlation was used. This compares all of the values for a given factor against the corresponding Damage Rating and determines what, if any, correlation there is between the values.

Given 2 sets of variables, the Pearson analysis assumes that there is a linear relationship between the two and then returns a numerical value that represents the probability that the two variables are linked. The values returned range from 0 to -1/+1, depending on whether the correlation is negative (as one variable increases, the other decreases) or positive (as one variable increases so does the other), with a value of 0 indicating that there is no probability the variables are linked.

This data for the values presented above is contained in Table 5.7.

Value	Correlation	<i>p</i> value
WSS (mg/l)	0.112	0.3
ASS (%)	-0.012	0.94
Original pyrite (%)	0.147	0.14
Pyrite Oxidised (%)	0.035	0.73
Type of Structure	-0.039	0.7
TS – S (%)	-0.053	0.69
Age of property (years)	-0.248	0.01
% pyrite oxidised per year	0.041	0.68
Uplift, single high point (mm)	0.424	1.65 x 10 <sup>-5</sup>
Uplift, multiple high point (mm)	0.426	1.65 x 10 <sup>-5</sup>
Amount of mudstone (% by volume)	0.052	0.68
Minimum fill thickness (m)	-0.058	0.65
Uplift rate (mm/year)	0.446	2.67 x 10 <sup>-6</sup>

Table 5.7: Pearson correlation and *p* values for the link between the Damage Rating and other values for development alpha.

As can be seen in Table 5.7, the only variables that appear to show any real signs of statistical correlation with the Damage Rating are those relating to the amount of floor

## 5 A consideration of IS398 for Classification of Pyritic Damage

slab uplift.

In order to confirm this, the p value is taken for each variable to show the degree of confidence in the link between the DR and a given variable. The lower the p value, the more statistically significant the relationship between the two set of variables is. For example, a p value of 0.01 represents a 1 in 100 chance that the two variables are not related, i.e. a 99% chance that there is a relationship between the two variables.

Based on this, it can be said that there is a statistically significant relationship between the DR and the age of the property, the amount of uplift of the floor slab and the rate at which that uplift occurs. The uplift rate shows a very low p value, although this is expected as it is based upon the age of the property and the maximum uplift, both of which also show low p values.

The relationship between the DR and both the age of the property and the amount of uplift is also to be expected. As stated above, most of the damage accounted for in the DR, the cracking to walls, the effect on fixtures and fittings, is related to movement of the floor slab. Similarly, the greater the age of a property, the longer the chance for the material to react, expand and cause damage. However, considering the slightly higher correlation and p value for this, it can be concluded that there is less of a relationship between the DR and the age of the property than there is between the DR and the slab uplift.

One reason for this lack of correlation between the DR and variables that are known to affect the reaction process could be that IS 398-1 is not designed for this used. The final product of the IS 398-1 analysis is simply to assign the property in question to one of three categories in order to determine what should happen next, i.e. whether testing should be carried out or if the property should be monitored. IS 398-1 was not designed for use as a tool for classifying the damage and comparing it to chemical factors.

However, although the action of assigning numbers to the damage seen in a property may seem somewhat arbitrary, the weighting of these numbers is based on the knowledge of how the damage is most likely to present in cases of pyritic heave in Ireland. For this reason, it can be used as a reliable indicator of the amount of damage in a property as caused the expansion of fill material below the slab, even if it does not account for other, mitigating factors.

Factors that are not necessarily accounted for include the density of the fill, which is known to have a large impact on the amount of expansion seen in the material, but can be difficult to measure in situ, and is not included in the standard suite of tests carried out as part of the Irish testing protocol. Additionally, factors such as the thickness of fill and the accessibility of pyrite, both of which are discussed in Chapter 8 in more detail, are covered briefly below. The analysis also does not include variable such as the type of construction, or construction defects, which is discussed further in Section 5.4.1.

Some of the factors that are not accounted for, due to either the lack of data or the limitations of the analysis, are discussed below.

### **Compaction density**

At present it is not part of the testing protocol to measure the compaction density of the material, therefore, this information is unavailable throughout the data for Development Alpha. Although methods have been tested in recent years to calculate the density in situ, it has been determined that the only way to completely accurately measure the density beneath a given area of the slab is to remove all of the fill, weigh the material removed and calculate the area that it occupied.

This is a significant gap in the available information, and along with the lack of information about structural defects, is thought to be one of the main reasons that there is a lack of correlation where expected amongst the data.

If the compaction density of the fill is low, there is an increased amount of void space in the fill into which gypsum can precipitate without causing an associated expansion of the material. If this occurs in a material with even a high level of pyrite, there may not be the expected high degree of damage seen in the property. In this case, the fill could be present for many years before anything other than minimal damage is caused.

### **Structural defects**

As stated previously, the presence of structural defects or poor building practices is not accounted for in the data. This includes elements that might exacerbate the presentation of the damage caused by fill expansion.

For example, some of the Irish pyrite cases have had internal walls not sited correctly above the foundation, meaning that part of the internal wall rests upon the floor slab. In these cases, movement of the slab also causes movement of the internal wall and leads to damage that would likely not have been present were the wall correctly sited over the foundation.

This leads to instances where the damage rating is higher than should be warranted for the property in question, and is covered further in Section 5.4.1 below.

### **Amount of pyrite**

Figure 5.15b does not represent the amount of pyrite present at either the time of placement of the fill or the time at which it was sampled, it just measures the change between the two over time.

A low original level of pyrite in the fill means that expansion will occur, but to less of an extent. Therefore, again, the fill could be present for many years and only cause minimal damage.

### **Accessibility of pyrite**

As stated above, there is a key assumption that all of the pyrite present in the material is available to react at any given time and that the gypsum will precipitate in a manner that leads to expansion.

The grading of the material, except in a few cases, is not known in detail. Therefore, we cannot say whether there is a large proportion of coarse material in the fill that will lead to a slower reaction process.

Similarly, we cannot say whether there was a large proportion of fine material present that would allow a faster initial reaction rate that would in turn lead to the pyrite being consumed quicker and therefore causing a shorted reaction period.

This influence of the grading upon the reaction process is considered further in Chapter 8.

### **Other factors**

As mentioned earlier in this chapter, the types of property present on Development Alpha vary from single to 3-story, detached to terraced and house to apartments. The type of structure is likely to have some form of effect upon the behaviour of the fill material due to the differing foundation styles and stresses placed upon the foundation by the structure.

Although not considered within the scope of this initial study, this factor is considered worthy of further study, and is discussed within Section 9.3 both with respect to the IS 398-1 analysis and in terms of possible numerical modelling work.

Additionally, there could be an element of human error, since the analysis is somewhat subjective, depending on an individual's interpretation of the wording of the standard. However, considering that the analysis for the development considered in this case was carried out by one individual, it is unlikely that such an error has had a significant effect.

#### **5.4.1 The influence of other factors**

As previously stated, the analysis of the properties using IS 398-1 and comparison of these values to the chemical data does not account for other factors that are known to have an influence on the amount of expansion.

Perhaps the most uncertain of these factors is the presence of other structural defects within properties that are affected by pyritic heave. This can include poor construction of internal walls and ceilings, and placement of internal walls overlapping joints within the floor slab.

The information regarding structural defects not linked to the pyritic damage is variable, with some of the structural surveys showing clearly the structural defects, and other simply focusing on the damage that can be attributed to pyritic heave. Moreover, if the surveys do mention structural defects, these defects do not always have the effect of enhancing the damage due to pyritic heave. For example, a poorly formed party wall in the attic is unlikely to aggravate damage seen on the ground floor of the property. Conversely, an internal wall on the ground floor that is not properly situated over the foundation wall is likely to increase the damage caused by any uplift of the floor slab in that area.

## 5 *A consideration of IS398 for Classification of Pyritic Damage*

Much of this information is obtained not from the structural surveys, but from photographs obtained during the remediation phase. Due to the limited availability of this information, it has not been included in this section of the analysis. However, it should be possible in future to obtain this data in a more complete form and consider ways in which to adapt the analysis to account for mitigation by structural defects.

For some factors that may affect the amount of expansion of the fill, such as the position of the water table, no information is available. In the case of other factors, such as the grading of the material, information is only available for a very small number of properties. In the former case, there is information available about the moisture content of the material, and the local water table at Development Alpha is believed to be located within the natural boulder clay. The moisture content of the material and its relation to the amount and rate of heave is discussed in Chapters 6 and 8.

Temperature is highlighted throughout the literature as being a potential contributing factor when considering both the amount of expansion and the speed at which it occurs. This effect is discussed in both Section 2.6 and Chapter 6, and will not be repeated here.

In the case of information about the grading of the material, for Development Alpha, there was not enough information available to begin to determine if any correlation is present; there was only grading information available for 3 of the 103 properties. Considering that this is not part of the list of tests specified under IS 398-1, it is unlikely that this information will be available for future analysis of other developments without a specific testing system being implemented. The importance of the grading of the material is discussed further in Chapters 7 and 8.

The amount of mudstone present in the fill material is believed to have a significant impact on the amount of expansion, largely because this is the primary host of the pyritic material in the Irish cases (Section 4.1). Research is ongoing as to this effect, with initial results showing that there would be expected to be a correlation between the amount of mudstone and the amount of heave, in a similar way to the expectation that there is a correlation between the original amount of pyrite and the amount of heave.

In the case of the development considered herein, about two thirds of the properties had data pertaining to the types of rock present in the fill material, of which 86% contained detailed information on the volume percentage of mudstone, limestone etc rather than a simple percentage split into mudstone and non-mudstone. Figure 5.18a shows the association between the percentage of mudstone and the damage rating, which is currently being studied as to its effect on the amount of expansion.

## 5 A consideration of IS398 for Classification of Pyritic Damage

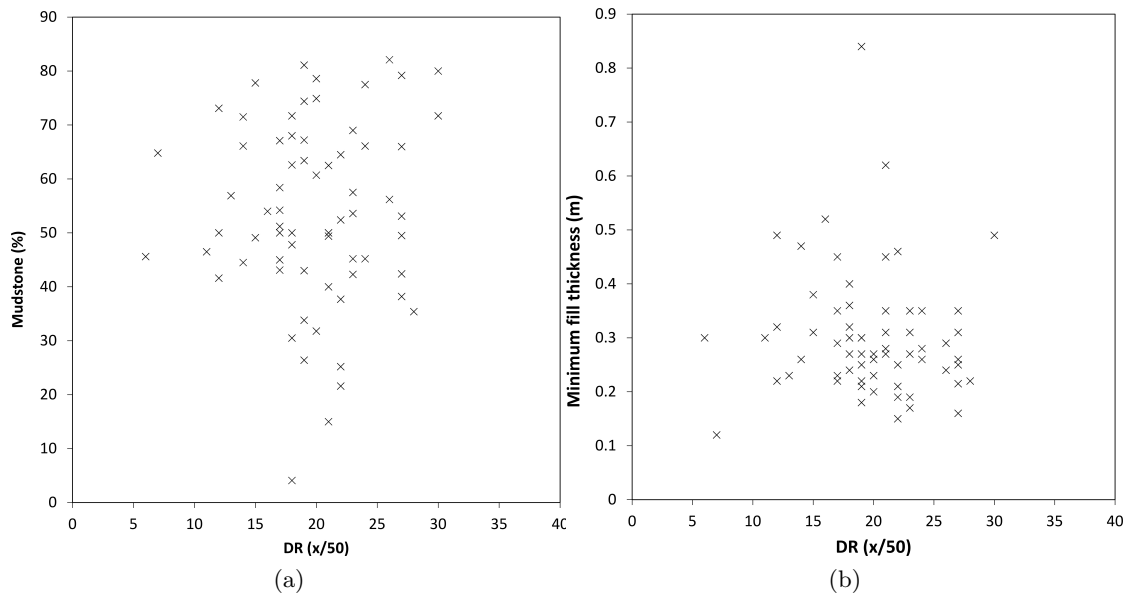


Figure 5.18: Comparison of the damage rating with the amount of mudstone and the depth of the fill.

Figure 5.18 also considers the impact of the fill thickness upon the amount of expansion, a factor that other research has shown to have a possible link (Ballivy et al., 2002. Sutton et al., 2013). The problem with considering the fill thickness in these cases is again a lack of information. For many properties, especially those inspected when the problem was just becoming apparent, the information is either unavailable or incomplete. For the comparison in Figure 5.18b, the fill thickness is that stated in the geological reports that are available. For some properties, the level of the natural material is stated, however, for most the lowest level is where digging stopped due to enough material already having been removed for a sample, or some barrier, such as buried services, was encountered. To that extent, the comparison is only with the minimum possible thickness of the material. The correlation between the damage and the fill thickness is summarised in Table 5.7.

Another factor highlighted by current research as having a large impact upon the amount of heave shown is the density of the fill material, which in turn effects the amount of void space available within the fill. Although the standards state that the material should be compacted at the optimum moisture content less 2%, there is no indication in any of the records of what this compaction density was at the time of construction.

At the time of writing, tests are still ongoing to determine a reliable way to determine the in-situ density of the fill material in the Irish cases. This is considered further in Chapter 8.

### 5.4.2 The shortage of data

In some cases, mention was made of damage to the properties that would have scored reasonably highly within a given element section, however, no supporting photographic evidence was given. In these cases, for the analysis above, values were assigned based on the information that was definitely available rather than what might have been mentioned in passing. In this case, the photographs and detailed information was considered to be primary evidence lacking any interpretation by other parties, a passing mention was considered secondary evidence that in many cases lacked enough detail to be certain that the damage was linked to the pyritic heave.

In order to determine whether or not the correlation would be improved by better information regarding the damage to the properties, the scores for each element were re-evaluated based on the damage seen to the adjacent properties. This primarily focused on the areas of fixtures/fittings and ground floor ceilings, which were the areas across most properties with a lack of information.

Only properties that had scored 19 or less out of the total of 50 were considered, which totalled 35 out of the 103 properties on the development for which structural and geotechnical investigations had been carried out. The structural inspections and available photographs for each of the properties was reviewed, and where mention was made of a defect the score was altered accordingly, even if no corroborating evidence such as a photograph was available. The scores were also altered where photographs showed partial evidence of a higher value.

It is appreciated that this method moves away from quantitative analysis of the problem, and introduces a larger chance of errors within the data. However, this portion of the analysis is simply intended to consider the viability of using IS 398-1 as a measure of the damage to a property should similar work be carried out with a more complete data set.

In order to keep this interpretation as accurate as possible, the only elements for which the scores were adjusted were those of the fixtures/fitting and ground floor ceilings. Although some of the properties showed minimal evidence of damage to the exterior of the building, this value was not adjusted. This is because evidence across the development indicated a low likelihood that damage would manifest in the external finish compared with the level of damage that could be seen internally.

Similarly, in order to limit the inclusion of error for those cases where no primary evidence of damage is due to the fact that the damage was not present, a limit was placed on the value to which a given element could be increased. For the ground floor ceiling element, this was set at “significant plasterboard cracks”, a maximum value of 4, which was commonly seen in properties throughout the development. For the fixtures and fittings element, the value was set at 6, corresponding to “front/rear door catching on floor” again because this was prevalent throughout the development.

5 A consideration of IS398 for Classification of Pyritic Damage

Figure 5.19 shows the graphs for those that provided the best correlation in the first half of the analysis, with the amended damage rating (ADR), and Table 5.8 shows the before and after Pearson's correlation coefficients for the same comparisons.

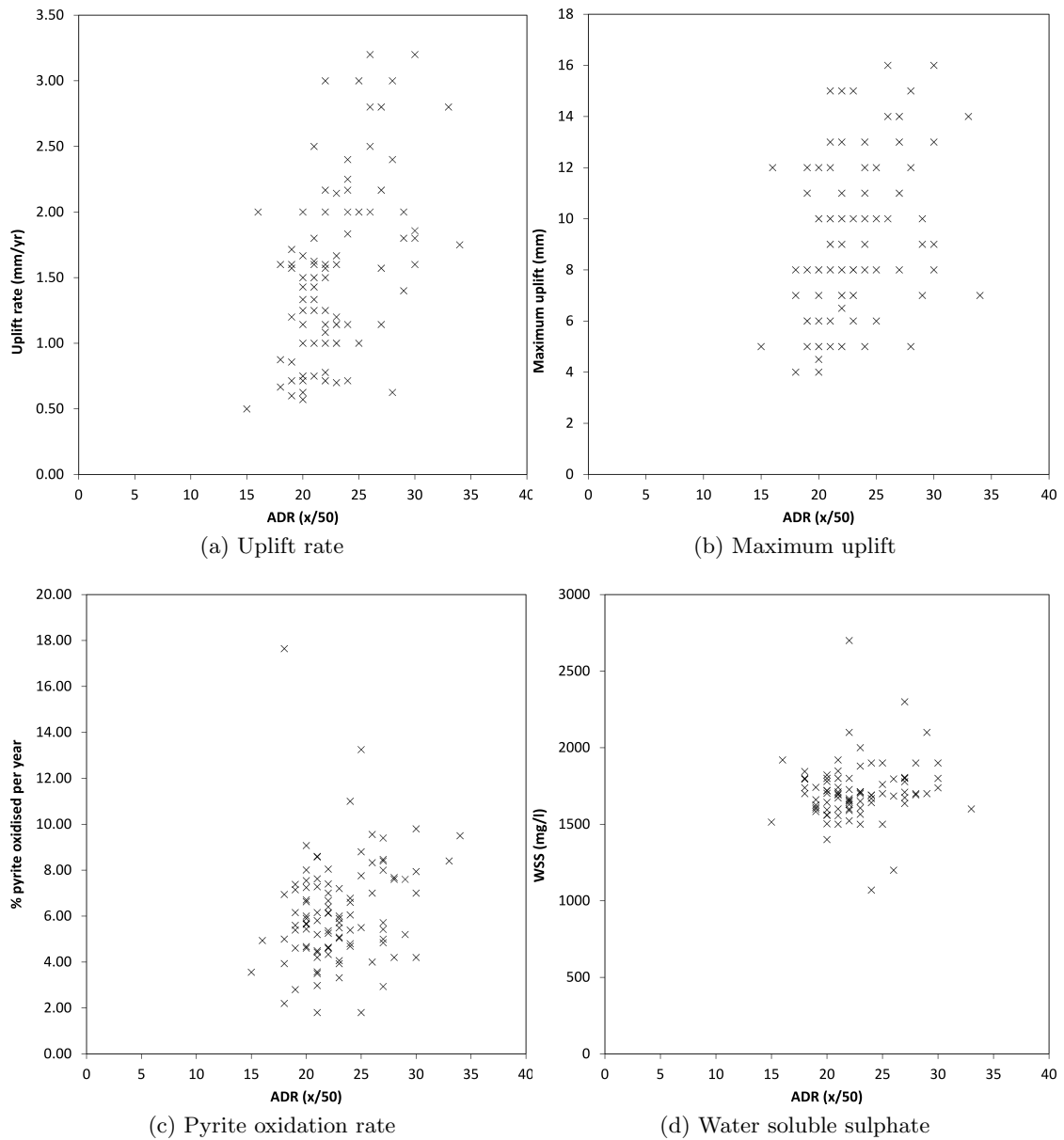


Figure 5.19: Comparison of the amended damage rating with the factors that indicated a minor correlation with the original damage rating.



Value	Original correlation	Adapted correlation
Expansion rate (mm/year)	0.446	0.497
Uplift, multiple high point (mm)	0.426	0.373
Original pyrite content (%)	0.147	0.153
WSS (mg/l)	0.112	0.115
% pyrite oxidised per year	0.041	0.221
Mudstone content (%)	0.052	-0.072

Table 5.8: Pearson correlation between the Damage Rating and other values for Development Alpha.

As hoped, some of the correlations improved, although in most cases this improvement is negligible, likely due to the influence of structural defects or factors not accounted for such as the density of the fill as discussed above.

Of some surprise is the change in the correlation between the uplift of the slab and the amount of damage, with a decrease from 0.426 to 0.373. The p value also changes from  $1.65 \times 10^{-5}$  before the adaptation of the data, to  $1.9 \times 10^{-4}$  after. This implies that the correlation after the adaptation of the data is less reliable than before, presumably via the introduction of errors when adapting the data.

This is yet another area that would benefit from a more detailed and robust statistical analysis of the available data.

## 5.5 IS 398-1 Analysis Conclusions

Although designed as a tool to identify new cases of pyritic heave in Irish properties, IS 398-1 provides a framework from which to analyse other cases of pyritic heave for which information is available.

Within the context of this study, the purpose of adopting IS398-1 for analysis of existing properties was firstly to classify the damage seen in the Irish cases – when considering a single development, this gives an idea of the variation in damage that can occur. Secondly it was to see if it was possible to directly link the damage seen to the chemical properties of the fill material.

The use of IS 398-1 as a tool for identifying damage caused by pyritic heave in pre-existing cases, and assigning a level of severity to that damage, is proven by the analysis carried out on Test Properties A & B, and by the analysis of Development Alpha. In all cases, either using the DCR given in the standard, or using the modified DR discussed in Section 5.3, it is possible to quantify the damage seen at a given property.

However, when comparing the different values to the Damage Ratings returned for the properties of Development Alpha, it can be seen that there is a lot of scatter to the data,

even when considering factors that are known to be linked to the amount of expansion.

Some of the factors do show signs of correlation with the Damage Rating, and a basic statistical analysis was applied to the data in order to determine the accuracy of the apparent correlations. Although this analysis proved some of the correlations, it was far from conclusive, and it is anticipated that a more robust and detailed analysis is required to continue the work on this aspect of the project.

Part of the problem with the amount of scatter present in the data is because certain factors are not considered. When using the standard for its intended purpose of identifying damage caused by pyritic heave in new cases, the absence of consideration of these factors does not pose a problem. It is when trying to adapt the system beyond its intended use, and comparing the damage seen to physical and chemical properties of the fill that these factors have more of an impact.

This is especially apparent when comparing the fill properties to the damage rating when considering the absence of factors such as the density of the fill material. This has been proven to have a direct effect upon the amount and rate of heave (Chapter 8), and yet it is unavailable within the Development Alpha data set as it is not routinely measured as part of the insurance protocol.

Similarly, the presence of structural defects is highlighted numerous times in this chapter. Although in some cases the presence of such defects will have little impact upon the damage seen to a given property, in other cases they may exacerbate the damage and the way that it is presented, by increasing the impact of the heave of the floor slab upon the rest of the structure. Within the data available for Development Alpha, the information on the presence or even absence of structural defects is limited at best. It should be possible to adapt the analysis to account for these defects, however, this is likely to introduce more errors to the result.

There are other factors that are not accounted for by the analysis that also play a part in the rate and amount of expansion seen in the fill. This includes the accessibility of the pyrite and the variability of the fill material itself.

Although the relatively small sample taken from the fill material of a given property is representative enough to confirm the presence of pyrite, and that the fill exceeds the limiting values provided as part of the testing protocol and within IS 398-1, there is still a lot of variability within the fill material. This will be discussed further in Chapter 7.

Discussion of the ways in which to improve the use of IS 398-1 by screening out factors such as the influence of structural defects is also touched up on in Chapter 9 and Section 9.3, as it is hoped that this will improve the methods described earlier and allow better comparison of the damage seen at a property with some of the physical or chemical factors.

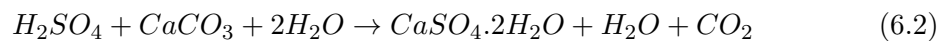
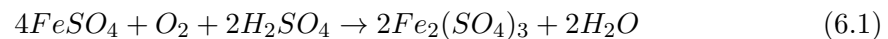
# 6 Field Monitoring – Development of an Arduino<sup>TM</sup> Based System

## 6.1 Introduction

Although there is evidence in the literature for the effect of temperature upon the reaction process (Section 2.6), and the fact that moisture is needed for the reaction process, there is a lack of information about what the environment is like below the floor slab in a domestic property.

As discussed in Section 2.2, water is a key component of the reaction process – without it, oxidation of the pyrite would not occur. Building regulations in force at the time of construction of many of the properties (Section 4.2) specified that the material should be compacted at 2% below the optimum moisture content. Tests at Galway showed that the material would react and expand at the moisture content at which it was compacted with no external water supply. This test continued to expand for over 150 days with no signs of a decrease in the reaction rate (Sutton et al., 2013).

It could, therefore, be assumed that the material in situ would react if damp, even if no external water source was available. This is potentially due to the way in which the reaction process itself produces water as a by-product, as shown in Equations (6.1) and (6.2).



However, overall more water is consumed by the reaction process than is produced. In laboratory tests, it may be assumed that some of this moisture will be taken from the air, as is discussed further in Chapter 8, Section 8.5.1. The most obvious sources of additional water in the environment below the floor slab are seepage of rainwater through or beneath the foundations, and capillary rise of water from the natural material below the foundations.

The natural material below many of the properties is stiff boulder clay, within which it is assumed that the local water table is located. Although this material has a low

porosity, capillary rise means that the material is often damp at the upper surface where it interfaces with the fill material.

From available air and soil temperature data, Figure 6.1, the insulation effect of the soil can be seen, with the soil temperature changes in air temperature, only slower and to lower extremes.

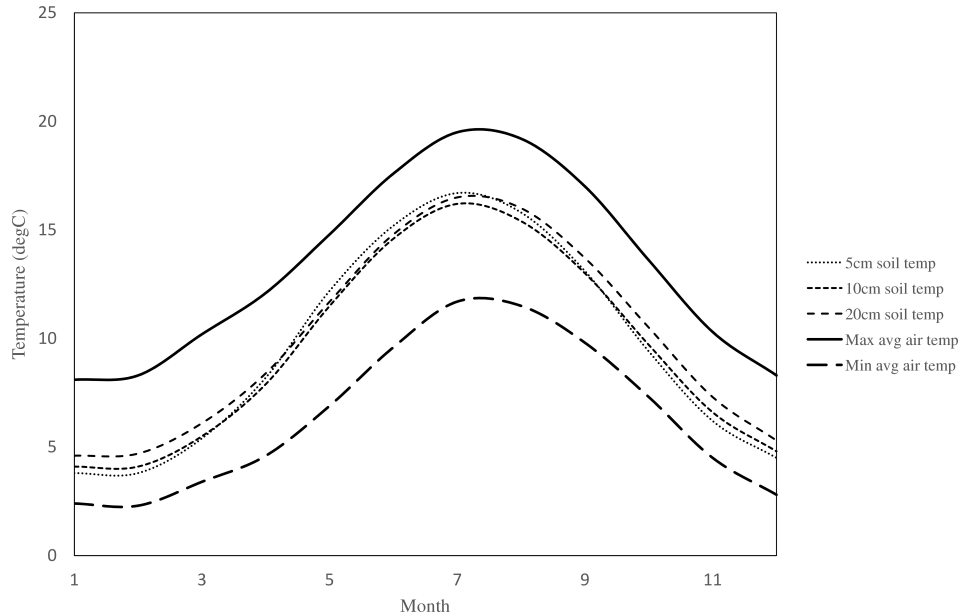


Figure 6.1: Annual air changes in air and soil temperature at Dublin Airport (Met.ie, 2010).

It can, therefore, be assumed that a similar variation in temperature would be shown in the material below the ground floor slab of a property. In addition to these seasonal variations, there would also be a human component to the temperature variations, especially in the colder months when buildings are heated.

This human influence, especially that of heating particular areas of a building, is of particular interest when considering the case studies of Llandough Hospital (Hawkins and Pinches, 1987a, Section 3.1.1) and Rideau Health Centre (Quigley and Vogan, 1970, Section 3.2.2), in which service ducts formed as part of the foundations were known to have raised the temperature of the material below the slab. This increase in temperature does not affect the reaction directly, but rather increases the activity of the bacteria that catalyse the reaction (Section 2.2).

Current research suggests that temperature changes have little effect upon the expansion rate (McCabe et al., 2015), although the maximum temperature in those experiments was 20°C whereas the *Thiobacillus* bacteria operate best at 25 – 40°C (Holt, 1977). Laboratory research is ongoing as to this aspect of the problem, and it is hoped that by

looking at the temperature below the floor slab, a better understanding of what, if any, effect temperature has in this situation will be gained.

## 6.2 The Arduino<sup>TM</sup> System

In order to study the below-slab environment, a system was needed that would be small, low cost and would allow for modifications depending on the number of sensors required. For the prototype system, it was decided to use an Arduino Uno<sup>TM</sup> – this is compatible with a large range of sensors, is reasonably priced and the code is open source meaning that a good support base is available. Figure 6.2 shows the Arduino Uno<sup>TM</sup> used in the laboratory testing.

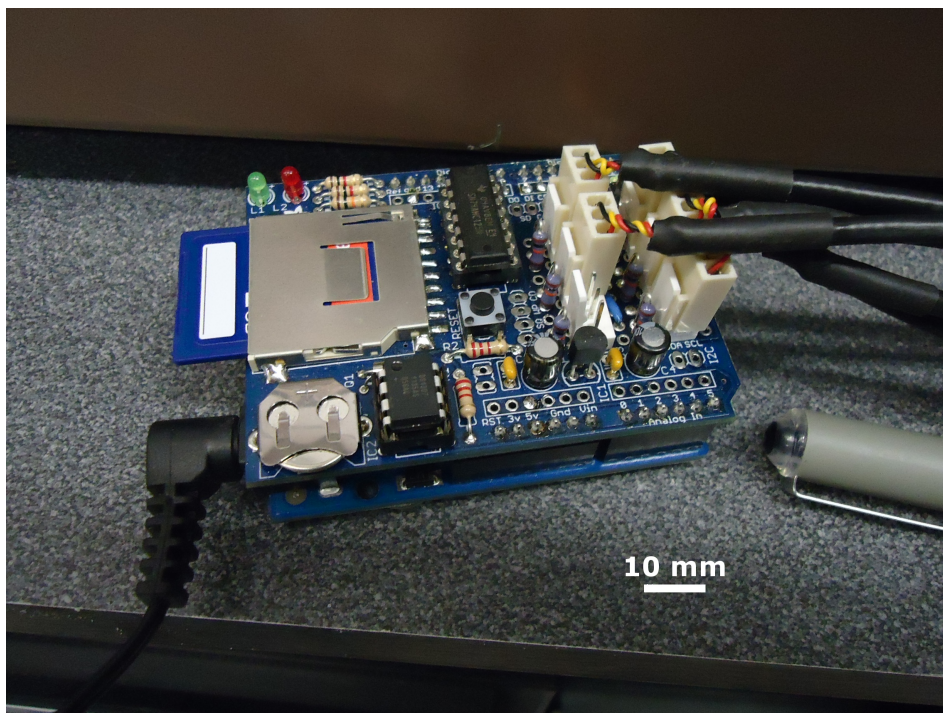


Figure 6.2: Image of the Arduino used in the first round of testing

For ease of access the system would be installed in a property at the time of the excavation for the geotechnical testing. The system for geotechnical testing requires a hole that is approximately 0.5 x 0.5 m, and is normally excavated downwards until either more than 20 kgs of material has been removed, or until an obstruction such as the natural bedrock is encountered.

Since the purpose of the system was to study the temperature and moisture content of the material below the slab, and how they vary, the layout for the sensors was planned

## 6 Field Monitoring – Development of an Arduino<sup>TM</sup> Based System

in order to determine any changes with depth through the fill. The reason being that any major temperature changes were likely to be induced from the direction of the floor slab, and any moisture changes from the direction of the natural material. Figure 6.3a shows the intended layout of the sensors in situ, in order to get the maximum amount of information. Three sensors was determined to give the most data while still allowing for space between the points, especially since the depth of fill would be unknown until installation.

Figure 6.3b shows the design for the housing for the sensors, with a larger tube housing the sensor itself to allow air to circulate which would be restricted were the sensor in contact with the fill.

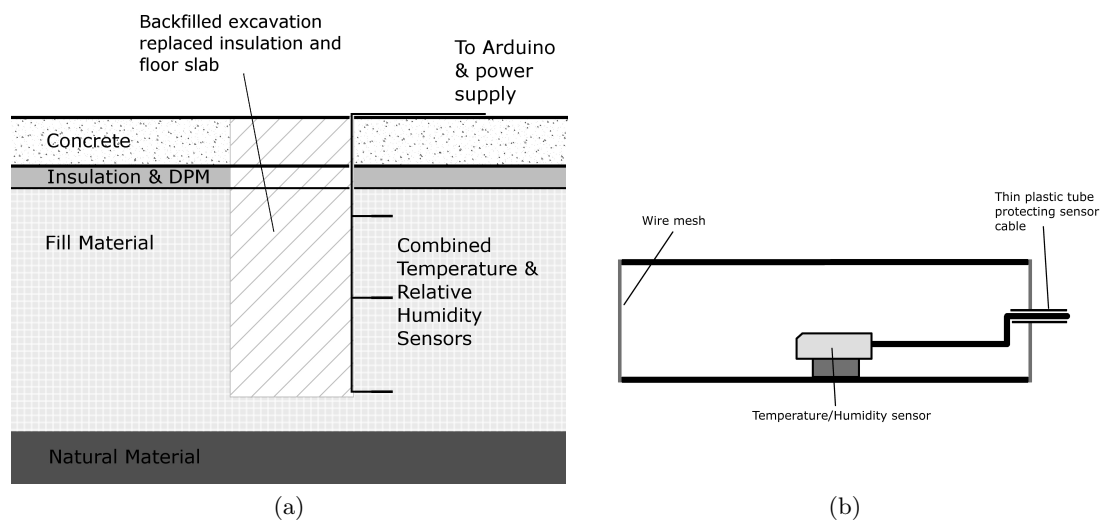
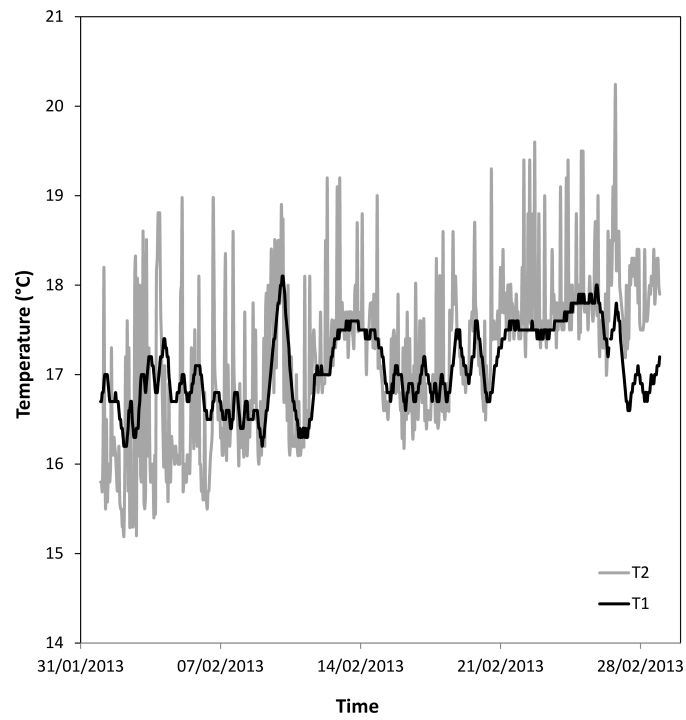


Figure 6.3: Planned method of installing the combined sensors in situ and the shield designed to protect the temperature and humidity sensors.

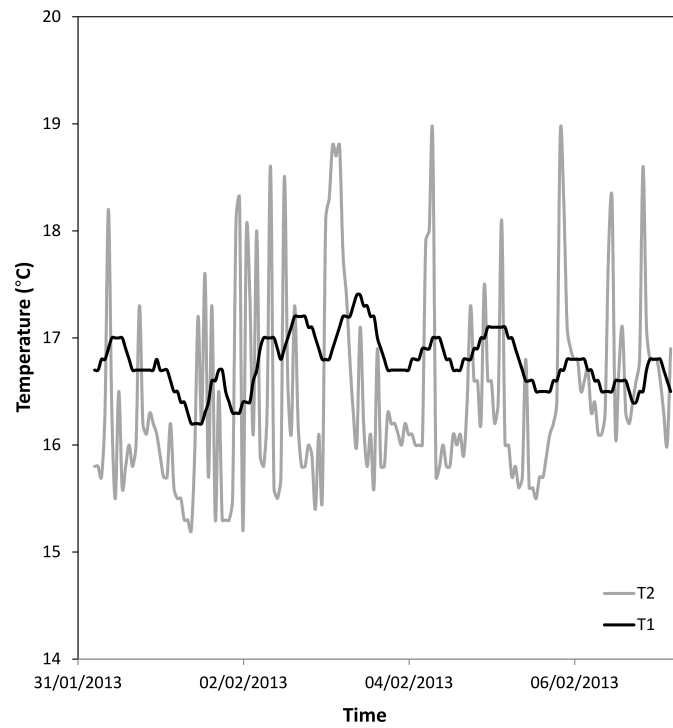
For this purpose, a system was designed using combined temperature and relative humidity sensors to measure the air within the fill. This could then be calibrated to give a reading of the moisture content of the fill after calibration in the laboratory. Additionally, the sensors were housed within a 50 mm diameter PVC tube covered at both ends by wire mesh – this was intended both to aid placement within the fill material, and to allow air to circulate over the sensors. Figure 6.3b shows this schematically.

This system was tested in the laboratory, with three sets of the temperature and relative humidity sensors compacted into one of the tubes intended for the swell tests. Since the exact density of the in situ fill is unknown, the material within the tube was compacted in layers of around 200 mm thickness using a 10 kg compaction hammer to give a final density of approximately  $2 \text{ Mg/m}^3$ . The swell tests and compaction methods are detailed further in Chapter 8.

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(a) Temperature changes over 31 days.



(b) Temperature changes over 7 days.

Figure 6.4: Mirroring of laboratory air temperature changes (T2) by the soil temperature (T1)..

Figure 6.4 shows the temperature data obtained from one of the buried sensors (T1) compared to one of the sensors situated in the air on top of the tube (T2). Figure 6.4a shows a full month of data from the tube, with recordings taken once per hour, Figure 6.4b shows an expanded view of the data from the first week. Both graphs show that the temperature within the material reflects the changes in the laboratory air temperature, although there is a distinct insulation effect in the sensor variations compared to the laboratory sensors. This insulation effect also causes a lag in the mirroring of the temperatures – high points in the material temperature occur slightly after the corresponding peak in the air temperature. This difference is generally on the scale of 3 to 4 hours, depending on the swing from one extreme to another.

Although the temperature function of the sensors were working as intended, all three were returning relative humidity values of 100%. Considering that the moisture content of the material when compacted was 8% and field values can be as high as 12%, it was decided to excavate the sensors to determine whether there was a fault with the sensors themselves or whether they were simply unsuited for the purpose.

Within 2 hours of being removed from the material, the humidity readings matched those of the sensors that had been left outside the tube to monitor the laboratory conditions. It was then decided to place one of the sensors into a box containing dry material to which water would be added until the sensors read 100% humidity, at this point the moisture content would be measured in order to determine the functional range of the sensors. The material used was the same as the material in the tube, although in this case it was not compacted around the sensors. Apart from the times at which water was added, the box was sealed to maintain it's own environment. Figure 6.5 shows the progress as the water was added, there is a gap in the data due to operator error in which the connector for the sensor was not seated correctly in the Arduino<sup>TM</sup>. The potential for this happening was taken into account when designing for the other sensors as detailed below.



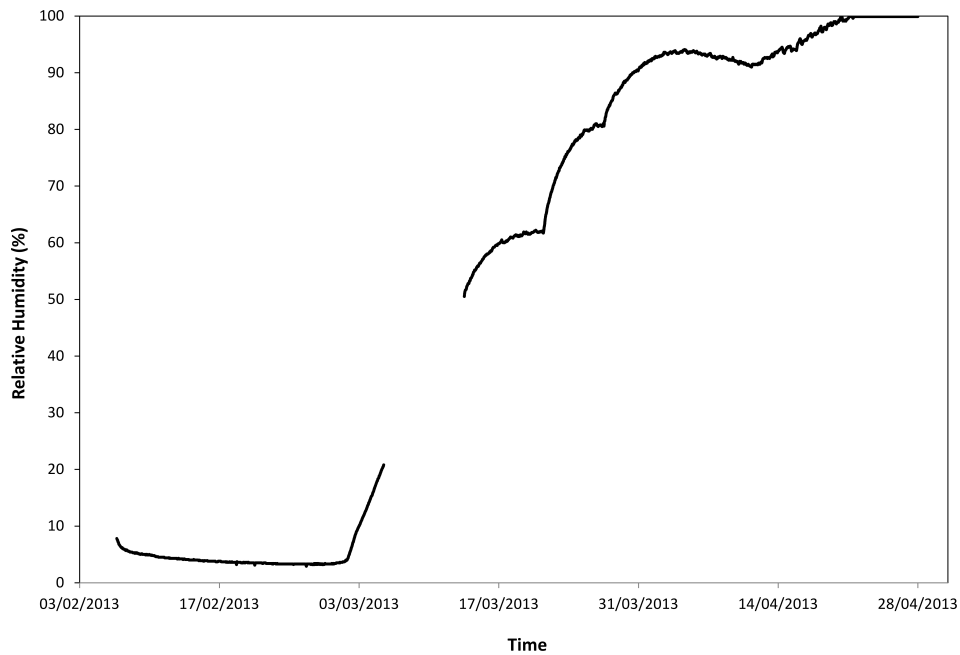


Figure 6.5: Addition of water to the sample containing the relative humidity sensor.

The moisture content at which the sensors recorded 100% relative humidity was calculated to be only 2.5%. Considering that many of the properties for which moisture content data is available are above this value (the recorded range at this time being 1.5 – 12%), the sensors would not be suitable for use in this case.

After looking at alternative sensors, it was decided to use dielectric probes – these could also be used with the Arduino<sup>TM</sup>, would be easier to install (as they do not require a separate housing), and ideally would give a quicker response to any moisture changes than the relative humidity sensors. Figure 6.6a shows one of the dielectric sensors used with the Arduino<sup>TM</sup> system. These are DFRobot moisture sensors (SKU:SEN0114), circuit diagrams and appropriate code are located in Appendix A.

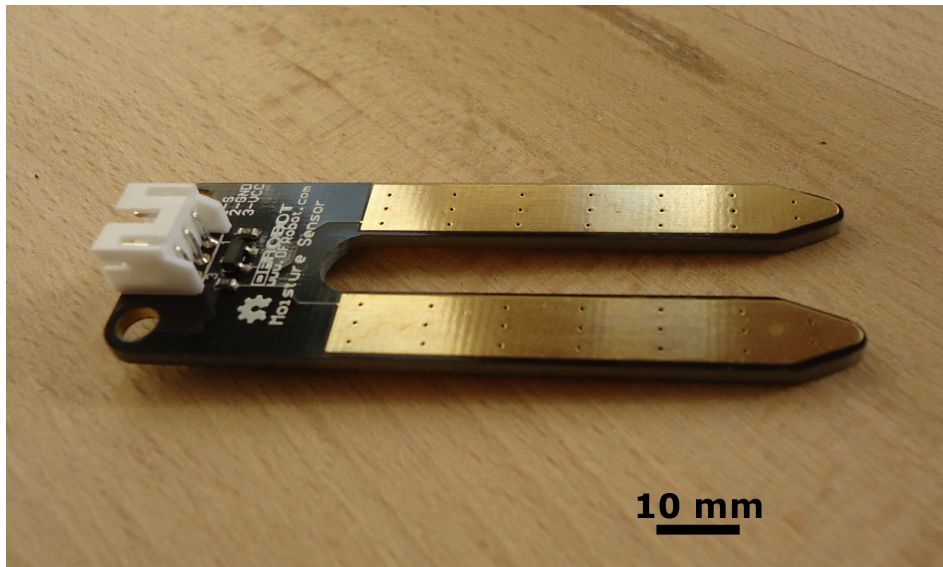


Figure 6.6: Dielectric sensor used with the Arduino to measure the soil moisture.

In addition to this, a separate system of monitoring the temperature would be needed, as the sensors no longer combined both functions. This actually made the design somewhat simpler, as smaller, digital temperature sensors could be used – these could be wired in sequence and simply placed along the edge of the excavation, and held in place by the backfill. The sensors used here were the Dallas One Wire digital temperature sensor (DS18B20), which had the benefit of having a reference library for the digital readings, meaning that calibration was unnecessary. The circuit diagram and relevant code for these sensors is contained within Appendix A. Figure 6.7 shows this new layout schematically.

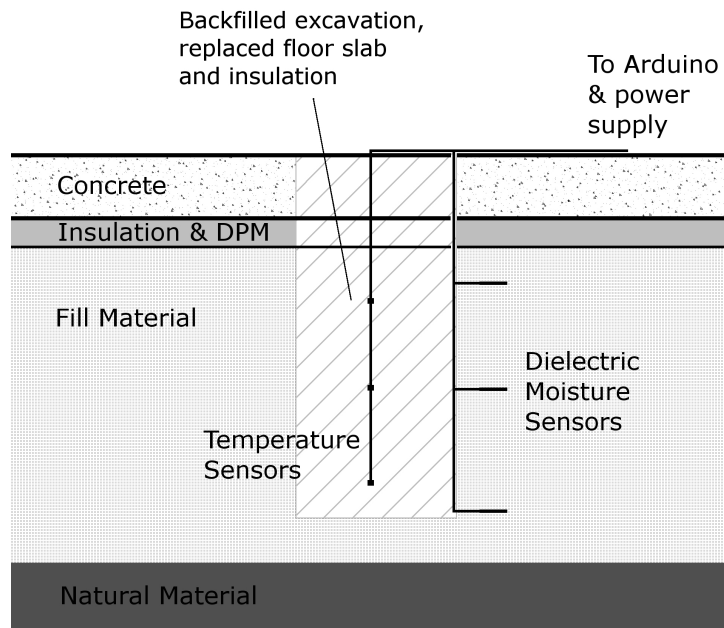


Figure 6.7: Planned method of installing the sensors in situ using the separate dielectric and temperature sensors.

### 6.3 Laboratory Testing

The new sensors were tested with a similar method to that used in determining the effective range of the relative humidity sensors – they were placed into a box with dry material, allowed to begin recording, and then water was added. In this case the water was intended to both test the sensitivity of the dielectric probes, and to make sure that both they and the temperature sensors would stand up to a damp environment.

The temperature sensors give a reading directly in °C, however the dielectric probes simply return a numerical value based upon the moisture content of the material.

The temperature sensors should be factory accurate and are assigned to refer to a digital reference library for the values they return before printing to file. As the temperature values returned in the laboratory were mostly the same as those returned by the temperature sensors on the swell test rig (Chapter 8), they were assumed to be factory accurate.

In order to determine the relationship between the reading returned by the dielectric sensors and the moisture content, readings were taken in dry material and in water to determine the upper and lower bounds of the range of the sensors. After this, readings were obtained from material at other known moisture contents, the average of these was

taken and the points plotted onto a graph.

It should be noted that there is some variation with the link between the sensor reading and the moisture content, depending upon the particle size distribution of the material. Therefore, it is necessary to calibrate the readings with a sample of the material into which the sensors are placed.

Figure 6.8 shows the relationship between the sensor readings and the moisture content for the material used in the laboratory. This relationship was assumed, based upon the clustering of points, to be linear, and so was calculated as such.

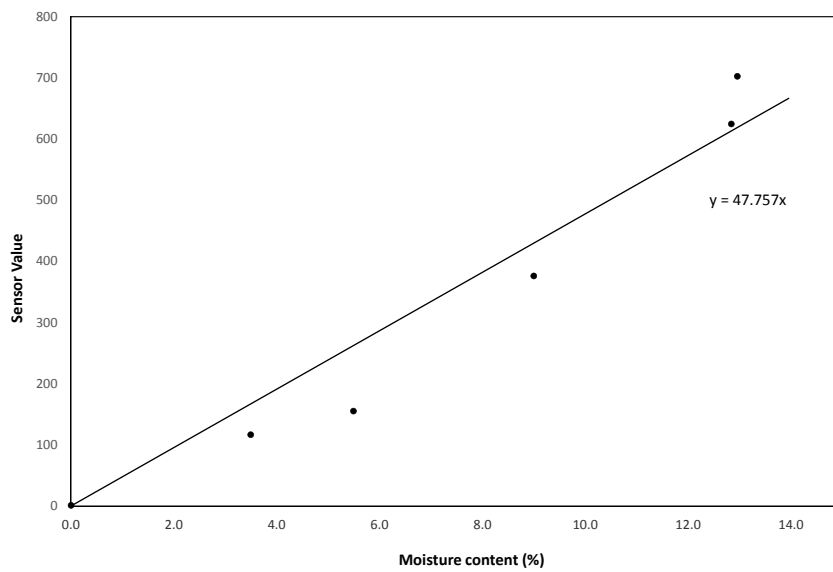


Figure 6.8: Calibration chart as used for determining the relationship between sensor reading and moisture content in the laboratory.

The relationship between the sensor readings and the moisture content, for the material in the laboratory tests, is shown in Equation 6.3, which was used to convert the sensor readings into moisture content values as shown in Figure 6.9.

$$MoistureContent(\%) = \frac{SensorReading}{47} \quad (6.3)$$

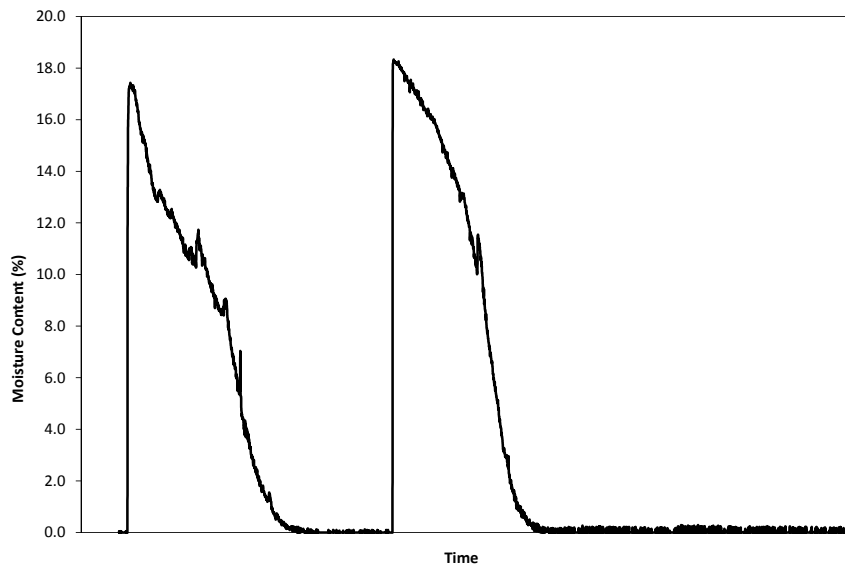


Figure 6.9: Variation of the moisture content as water was added to the soil sample in the laboratory.

Figure 6.9 shows the moisture content variations in the material as water was added. Since the box was not sealed, the water was allowed to evaporate and the material to dry out in order to see how variations occurred.

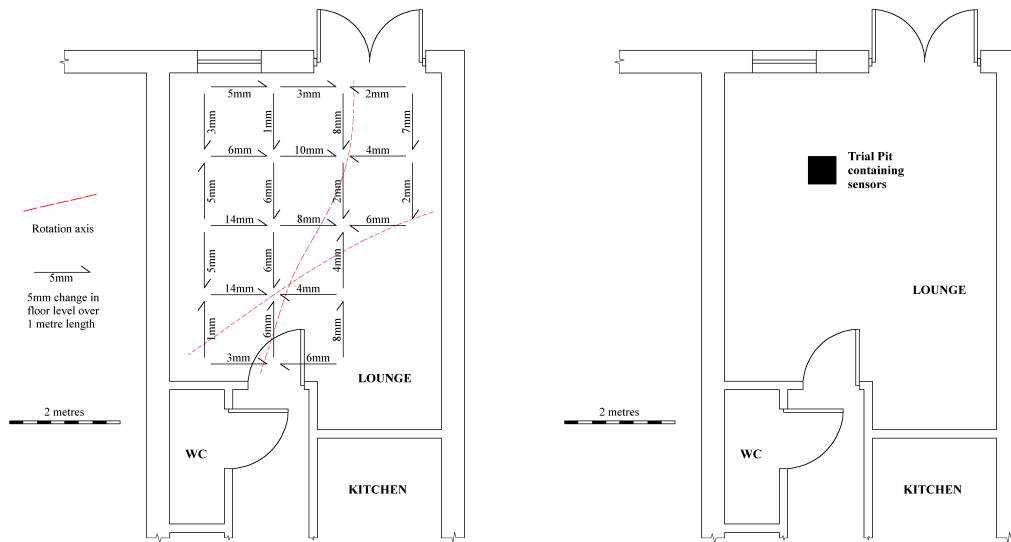
## 6.4 Installation

The property selected for installation is a three-story end of terrace dwelling, located on a development just to the north of Dublin that was constructed circa 2007 with inspections first taking place in 2015.

Analysis of the property using IS 398-1 gave a damage rating of 27/50, despite the damage seeming initially to be minimal. Movement of the staircase is suggested, with both cracking and bowing of the wall plaster in this area. The flooring by the front door had to be removed in order to be able to open the front door fully, and there are several instances in the ground floor ceilings where the nails can be seen to be protruding out of the plasterboard.

Figure 6.10 shows the floor level survey carried out at the property, and the location at which the sensors were installed.

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(a) Floor level survey showing the uplift of the floor surface in the lounge area where the Arduino was installed. (b) Installation location for the Arduino sensors.

Figure 6.10: Details of the lounge area of the property in which the Arduino and sensors were installed.

The trial pit was 0.57 m deep, this being the level at which the natural, clay material was encountered. The dielectric sensors were placed within the fill material on the side of the pit. The lowest was placed in the fill just above the natural material, the second placed roughly halfway down the side of the pit and the third into the fill just below the blinding beneath the damp proof membrane. This is shown schematically in Figure 6.11.

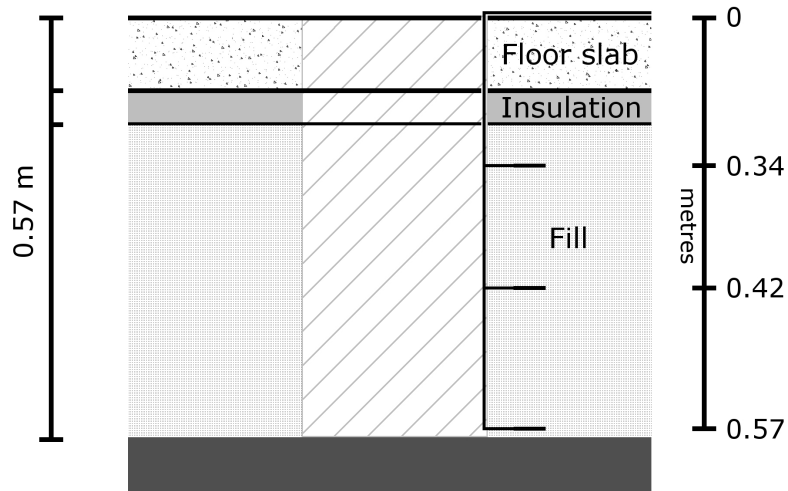


Figure 6.11: Layout of the dielectric sensors, including their depths, in the test property.

Once the dielectric sensors had been placed, back filling of the hole began. The hole was back filled with the standard material used by the site engineer for all such sampling tests carried out as per the sampling protocol. This material was a sandy gravel of low moisture content.

The temperature sensors were held in place in the middle of the pit whilst back filling was ongoing, allowing this material to support the cable holding the temperature sensors in a vertical position in approximately the centre of the pit.

A fourth temperature sensor was placed above the floor slab, close to the location at which the box containing the Arduino<sup>TM</sup> was located. To avoid the chances of the box being kicked accidentally by the house occupants, this box was placed close to the external wall at the rear of the property, with the temperature sensor taped to top of the box. This sensor was intended to give a reading for the temperature within the property to be compared to the temperatures recorded within the fill.

The material that was removed from the pit was taken away for testing as per the pyrite protocol, the information from which is not yet available. However, Figure 6.12 shows the sample that was brought back to the laboratory for calibration of the dielectric sensors.



Figure 6.12: Material taken from the pit in which the sensors were installed, shown at its natural moisture content.



(a) Mudstone fragment showing areas of orange-brown staining – to the right of the image. (b) Mudstone fragment showing areas of gypsum precipitation – silvery grey patches to the right of the fragment.

Figure 6.13: Images of opposite sides of a mudstone fragment taken from the sample shown in Figure 6.12.



The fill material had an in situ moisture content of 6.4% and is generally comprised of mudstone and siltstone, with a large proportion of fine material. The surfaces of the larger particles are frequently stained a brownish colour (Figure 6.13a) and are covered with small white or clear crystals (Figure 6.13b). Based upon visual inspection, these crystals are similar in habit and appearance to the gypsum crystals seen in hand specimens from multiple other locations.

Figure 6.14 below shows the installation of the dielectric sensors in the sample pit at the property. The dielectric sensors were seated into the fill material in the side of the pit with the full length of the pins inserted into the material. The cable containing the temperature sensors was held in place in the middle of the pit as the hole was back filled in order that the cable stayed vertical.



Figure 6.14: Installation of sensors at the test property.

Once the hole had been back filled, the damp proof membrane was repaired, the insulation replaced and the hole through the slab filled with fresh concrete. At this point the sensors cables were made safe and the sensors were connected to the Arduino<sup>TM</sup> in order to begin recording.

## 6.5 Data Analysis

The sensors were left in place after installation with the intention that they would continue monitoring the property until remediation occurred or, if remediation were to

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be declined, until the property owner asked for them to be removed. The data set presented below covers the first 8 weeks that the sensors were installed and covers the changes in both the temperature and moisture of the fill during that time.

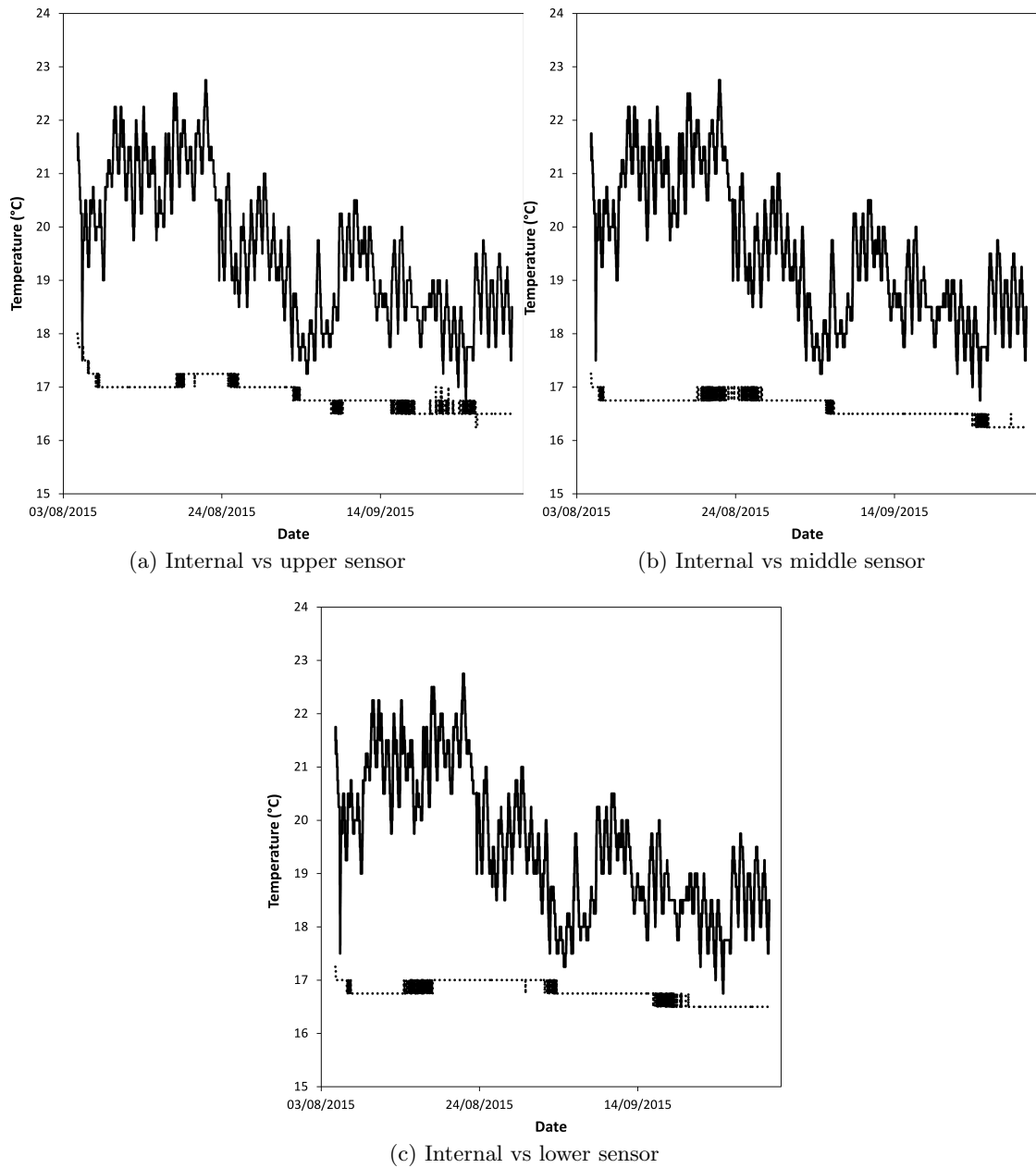


Figure 6.15: Variations in the air temperature (solid black line) and corresponding changes in ground temperature (dotted line) over the same time period.

Figure 6.15 shows the temperature fluctuations in the upper, middle and lower temperature sensors over the 8 week period. It should be noted that in all cases, the initial 4 hours of data were discarded both to allow the sensors to equalise to the ground temperatures, and to remove any effects the initial curing of the concrete might have had. Additionally, although the datapoints were recorded every 15 minutes, it should be noted that generally the swing recorded from a high temperature peak to a low temperature is on the order of several hours.

From these graphs, there is currently such little variation that it is difficult to see whether there is any effect on the ground temperature, particularly the lower sensors, with the variation of the internal room temperature. There is a gradual lowering of the internal temperature over time, as the local temperature decreases through the autumn months, with a matching gradual decrease in ground temperatures, although at the present time, it is unclear whether this effect will persist.

In order to determine whether there was a reflection of the temperatures, the temperature data for the internal and middle sensors was plotted with data points every 6 hours instead of every 15 minutes as in Figure 6.15. This was intended to remove the smaller temperature fluctuations in order that a better comparison of the temperatures could be made, and is shown in Figure 6.16.

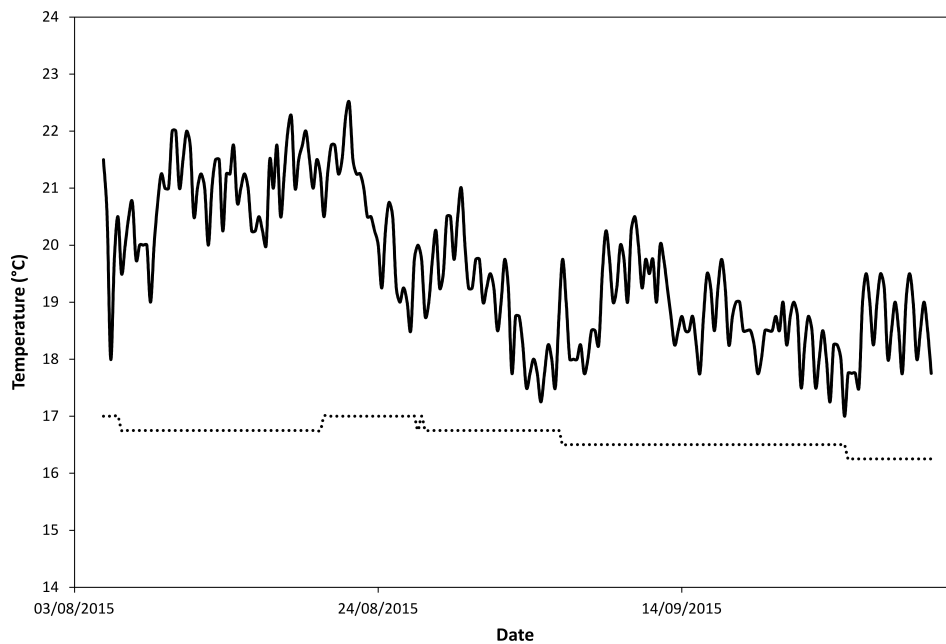


Figure 6.16: Modified data from the internal and middle temperature sensors, reflecting the gradual temperature decreases.

As the figure shows, although a general decrease with time of both temperatures can be seen more clearly than in Figure 6.15, at present there is not enough data to confirm

whether the trend will be reflected with a warming once factors such as central heating and colder winter temperatures start to have an influence.

Critically, however, the temperature in the fill still registers as between 16 and 17 degrees even in the lowest placed temperature sensor.

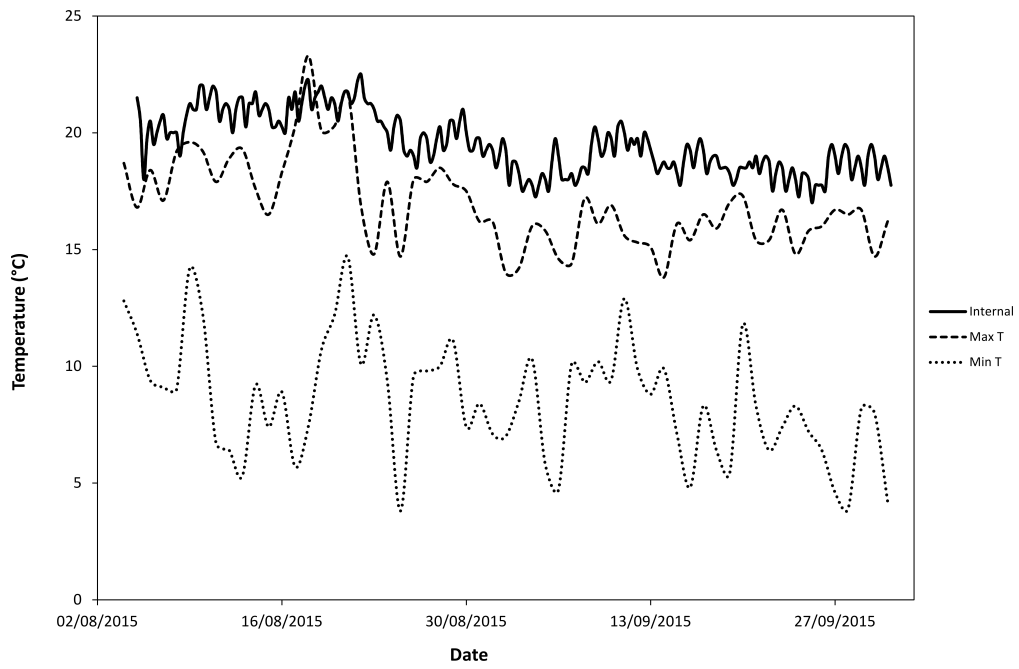
As shown in Figure 6.1, the average air and soil temperature data for Dublin Airport, located within a few miles of the test property, is available online via the Irish Meteorological Service (Met.ie, 2010). This data shows that in August and September 2015, the monitoring period described herein, the average soil temperature at 10 cm depth was 15.9 and 13.2°C respectively. Therefore, the sensor seems to be recording a higher reading, indicating the warming from the home.

To consider this further, the air temperatures for Dublin Airport during these months was accessed. Figure 6.17 shows the maximum and minimum air temperatures for August and September 2015 as compared to the values obtained from the internal sensor placed near the Arduino<sup>TM</sup> (Figure 6.17a) and the middle of the three buried sensors (Figure 6.17b).

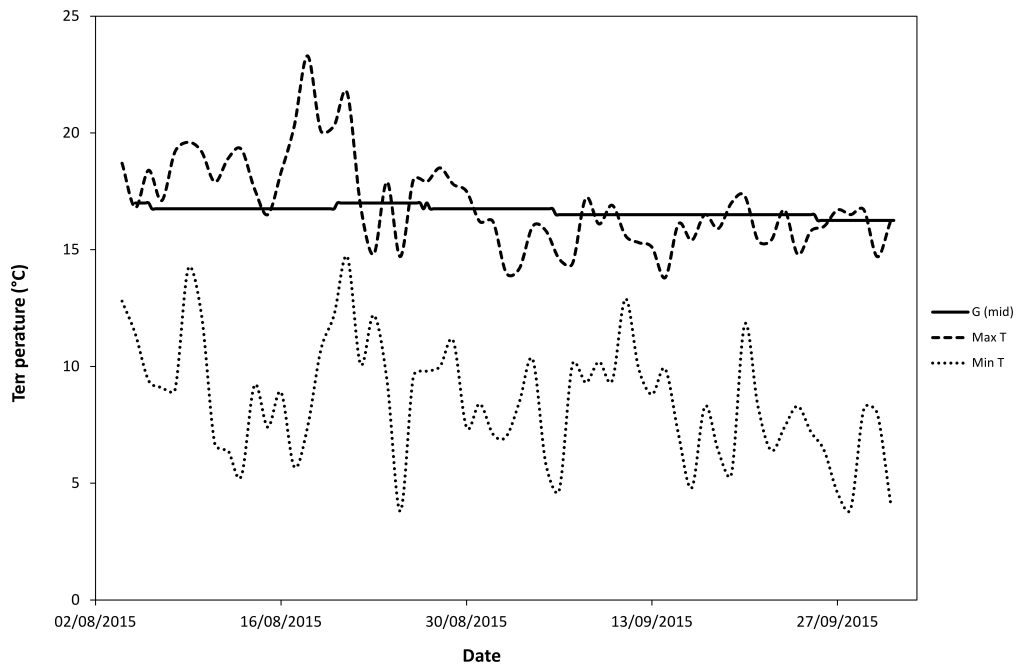
Figure 6.17b, seems to show that the temperature in the fill during the monitoring period is following the internal room temperature, which in turn is following the external air temperature. Monitoring is ongoing to determine if this will remain true during the colder months when the building is heated on a regular basis. More importantly, in the colder months the data should give a better idea as to which temperature has the greatest influence on the fill temperature – the internal room temperature or the external air temperature.

These temperature values are of particular interest when considering the laboratory testing as discussed in Chapter 8, since they give a range of likely temperatures at which the material is known to swell. More importantly, it confirms that the laboratory tests at Sheffield are being carried out under realistic temperature conditions, with the laboratory temperature monitored to be between 16 & 20°C.

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(a)



(b)

Figure 6.17: Variations in external and internal measured temperatures over the monitoring period.

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As mentioned above the dielectric sensors required calibration in order that the data returned could be converted to a relevant moisture content. And as the grading of the material will vary from one location to another, to ensure accuracy of the results the calibration for the test property sensors was carried out with a material sample taken at the time of installation.

Equation 6.4 shows the coefficient obtained from measuring the fill material obtained from the test property, whilst Figure 6.18 shows the initial values obtained from the dielectric sensors.

$$\text{MoistureContent}(\%) = \frac{\text{SensorReading}}{79} \quad (6.4)$$

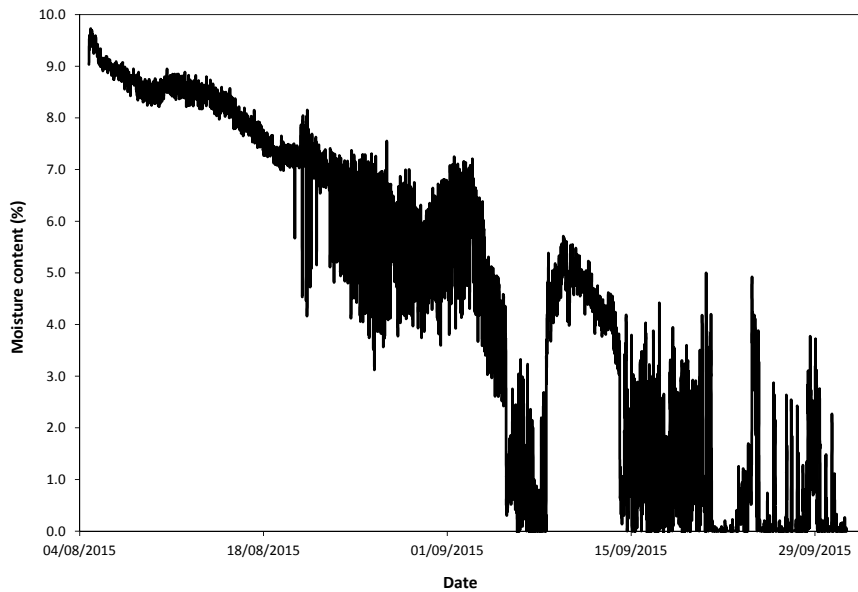


Figure 6.18: Initial data recorded with the uppermost dielectric sensor.

Figure 6.18 shows the raw data obtained from the uppermost dielectric sensor, converted to show the moisture content value, but with no data processing. Although the graph shows an overall decrease in the moisture content of the material, there is a lot of noise present obscuring the data.

The cause of this noise is unclear at the time of writing, as the sudden changes in moisture content were not seen in the laboratory experiments that were carried out to

test the system. Considering that in some cases, the moisture content can change from a value of 7.3 to 0.5 and back again within the space of 30 minutes, it is likely that the cause of these anomalies lies with the sensors or Arduino<sup>TM</sup> system, rather than with the fill itself. It is more likely that these anomalies are caused by a problem with the sensors or due to some sort of signal interference.

In order to remove some of this noise from the data, a basic ‘if’ analysis was carried out, where by if the difference between the original readings at 2 subsequent data points was greater than 100, the cell was assigned to be false. Where that difference was less than 100, the cell was true. Via this method, it was possible to see when these anomalies occurred, and in which sensors.

There were no anomalies present in the data prior to 9.30 am on the 20th August 2015, and with one or two exceptions, the anomalies generally presented in the data from all three sensors at the same time. By far the largest difference seen between two subsequent readings came from the lowest placed sensor, although all three sensors were showing moisture content differences of 2 – 3% on average. It is possible that this is a cabling issue, although it will be difficult to determine this until the sensors are excavated.

There was no apparent link between the times at which the anomalies would occur, seemingly ruling out some sort of interference with the Arduino<sup>TM</sup> system by something else within the property. Although, as can be seen from the data presented below, there was a general increase in the noise of the data with time. It was also noted that after the 5th September, the data showed a rapid increase in anomalies, with the overall moisture content decreasing rapidly from around 3.5 – 4% to less than 1% across all three sensors.

In order to better analyse the initial data, the points registering as an anomaly in the ‘if’ analysis were removed, and all data up until the 5th September was plotted as shown in Figure 6.19 below.

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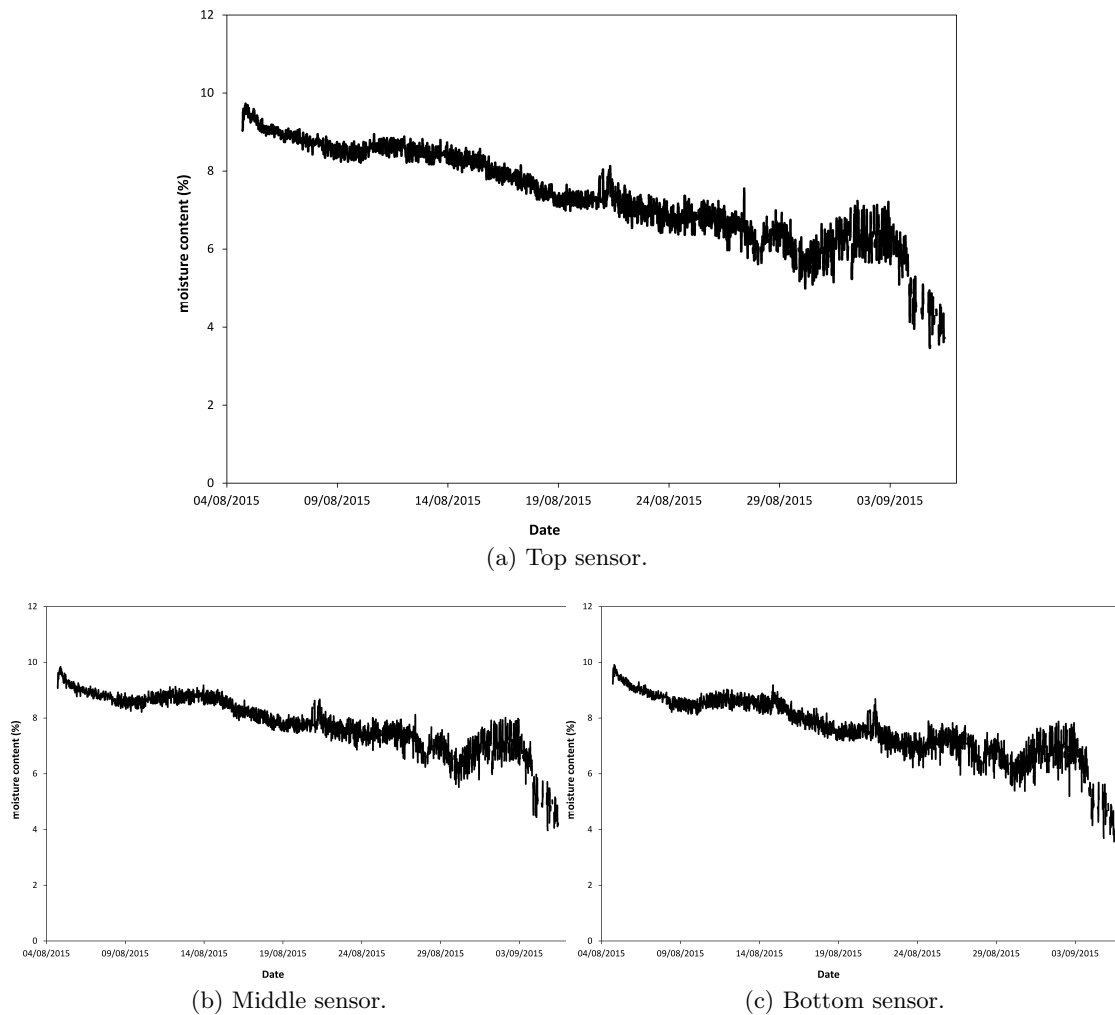


Figure 6.19: Moisture content data retrieved between 4th August and 5th September 2015.

Despite the analysis carried out to remove the worst of the anomalies, there is still a large amount of noise seen in Figure 6.19, and as mentioned above, there is an increase of this noise with time. Still, it can be seen that over time there is a gradual decrease in the moisture content of the material.

This is thought to be due to the moisture level in the material around the sides of the pit equilibrating to match that of the backfill that was placed after excavation. Although the backfill material was not completely dry, it did have a very low moisture content. As this was the only material that was available when installing the sensors, and the natural moisture content was not known such that water could be added to the material, it was decided to go ahead with placement and monitor the situation to see what occurred.



## 6.6 Discussion

From the point of view of requiring a small, easily modifiable system that can be placed in situ and left to monitor the environment, the Arduino<sup>TM</sup> system is, based on current results, a success. Once in place and recording data, in the case of the temperature and dielectric data acquired, the system could conceivably be left in place for months at a time without needing to retrieve the data, as long as the SD card contains enough storage and there is no power outage. Should a power outage occur, however, the system will resume monitoring once power is restored.

In the instance of using the system to monitor the environment below the floor slab, from a practical point of view, the system was small enough to install easily and locate at the side of the room without causing an inconvenience to the homeowners. Perhaps the only downside was that the connections of the sensors to the Arduino<sup>TM</sup> itself are somewhat delicate and require care to be taken when handling the system to retrieve the data. However, this is a problem that is easily solved when considering the system for future use.

Based on the data that has been obtained so far, there seems to be a general reflection of the internal air temperature within the fill material, with an overall cooling trend being seen in both as the time progresses. Interestingly, the temperatures within the fill are higher than the average temperatures at 10 cm soil depth recorded in those months at the nearby Dublin Airport monitoring station. The fill showed temperatures between 16 & 17°C, whereas the soil temperature at Dublin airport averaged 15.9°C in August and 13.2°C in September.

Considering that the annual average soil temperatures at 10 cm depth shown in Figure 6.1 vary between a minimum of 4°C and a maximum of 16°C, the fact that the temperature of the fill is already above this maximum heading into the colder months suggests both a heating effect of the foundation and house structure, and an impact by the human habitation in the property. Based on the current temperature data, it is unclear whether the house is insulating the fill from the cold or is actively heating it. It is anticipated that as more data is retrieved during the winter period, it will be possible to answer this question.

The temperatures currently being recorded within the fill are also comparative to those recorded in the Sheffield laboratory (see Chapter 8), confirming that at least the temperature element of the laboratory environment is representative of that seen in the field.

The moisture data shows the material drying out to match the moisture content of the material used as backfill within the pit. It is anticipated that as time progresses further, the fill will equilibrate further to show an increase in moisture content.

Were the opportunity to arise to place another of these systems within a property, there are amendments to both the system and the placement method that the author considers

vital. These are:

- Placement of damp material when back filling the pit. If the moisture content of the natural fill material for the location is not known, in the Irish cases an estimate of the moisture content could be obtained from the results of properties in the immediate area. This estimate could then be used to determine the upper bound for the moisture content of the fill material, and the equilibration period would be distinctly reduced.
- Although not of use when the moisture content is above 2.5%, the relative humidity sensors would theoretically be able to provide data on the humidity of the air within the fill when the material was too dry to give a reading on the dielectric sensors. The humidity data, combined with the dielectric moisture data would give more information about the movement of moisture and air within the fill material.
- Installation of a temperature sensor to monitor the external air temperature would provide further data on the influence of general climatic changes and enable us to determine whether the external or internal temperature has the most influence on the temperature of the fill material. Installation of such a system was considered before installing the above system, but was not permitted at this time.

The necessity to calibrate the dielectric sensors in order to accurately represent the material, led to a more in-depth study of the Irish mudrock material used in the laboratory testing. This is discussed in detail in the following chapter.

## **7 Characterisation of a Typical Irish Fill Material**

In order to gain a better understanding of the behaviour of the Irish mudrock material, a series of classification tests were carried out. This is the same material used in the initial Arduino tests (Chapter 6) and in the laboratory swell tests (Chapter 8).

### **7.1 Material Properties**

The material used for the laboratory tests that are detailed in this section, was taken from an Irish domestic property undergoing remediation. Unless stated otherwise, all material was taken from this property, and is believed to have been sourced from a single quarry.

The property in question is a 2-storey end of terrace house constructed in 2004. Detailed investigations into damage at the property began in 2009, and included the suite of investigations detailed in Chapters 4 and 5.

Figure 7.1 shows the changes in level to the ground floor of the property at the time of inspection. The maximum change in level over 1 metre was recorded at that time to be 16 mm, with 3 distinct ridges in the floor of the kitchen and living room.

## 7 Characterisation of a Typical Irish Fill Material

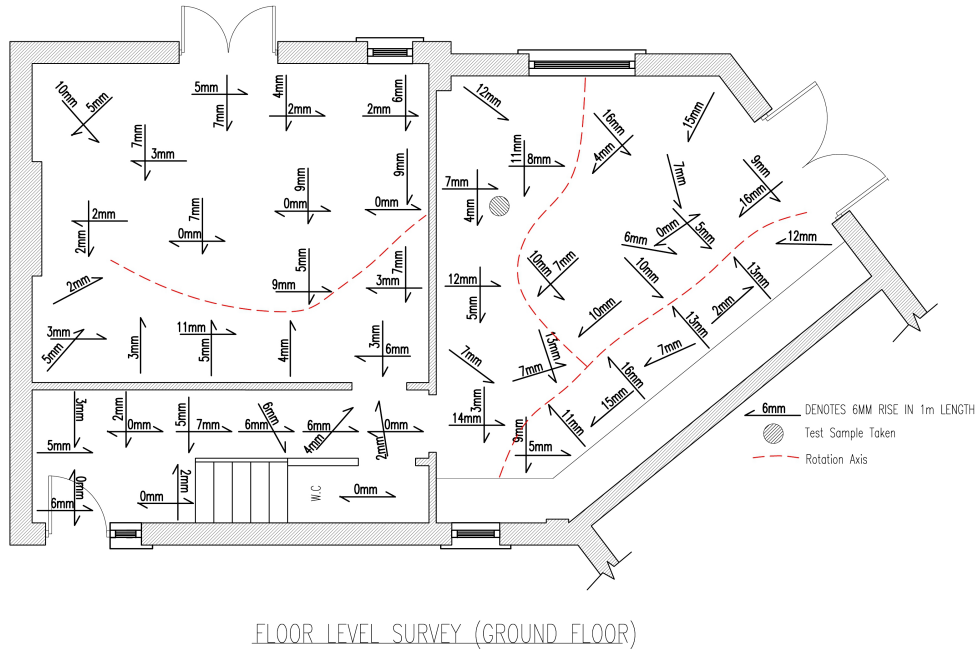


Figure 7.1: Floor level changes at the property from which material was taken for laboratory testing (Used with permission from Aidan O’Connell Associates).

The information available that showed the condition of the property prior to remediation was used to analyse the property according to IS 398-1. This process was covered in detail in Chapter 5 and will not be discussed fully again herein.

The analysis gave a minimum value of 30/50 for the amount of damage seen. This is stated as a minimum value since from the limited data available, it is unclear whether other damage might have been present that was not included in either the site photographs or the engineering reports. When compared to the properties that were analysed as part of the work detailed in Chapter 5, this is a relatively high value signifying significant damage, as is highlighted in Figure 7.2.

7 Characterisation of a Typical Irish Fill Material



(a) Cracking to the plasterboard above the living room door. Images b) and c) show the cracks in more detail



(b) 10 mm crack to upper left of living room doorway



(c) Cracking and spalling plaster to upper right of living room doorway



(d) Cracking to kitchen wall and ceiling, maximum crack width 8 mm



(e) Cracking and upward movement to stairwell, maximum movement 6 mm

Figure 7.2: Damage seen at the property from which material was removed for testing in the laboratory (Used with permission from Aidan O'Connell Associates).

## 7 Characterisation of a Typical Irish Fill Material

As part of the 2009 investigations, a series of geotechnical tests were carried out. At that time, 2 main lithologies were identified in the fill material:

1. A slightly calcareous SILTSTONE.
2. A calcareous MUDSTONE.

The surfaces of the mudstone were noted to contain clear and white crystals and orange-red deposits. It is unclear from the documentation available whether these were also present on the siltstone particles, although some cross-contamination can be assumed.



Figure 7.3: Mudrock fill material used in the laboratory testing in its damp state.

A petrographical examination of the fill material suggested that the pyrite content was between 1.0 – 1.5 %, and chemical testing on samples of the mudstone gave values as summarised in Table 7.1.

## 7 Characterisation of a Typical Irish Fill Material

Measurement	Value
Total sulphur (% S)	1.92
Acid soluble sulphate (% S)	0.93
Water soluble sulphate (mg/l)	1900
Water absorption (%)	2.90
Total potential sulphate (%)	5.76
Oxidisable sulphides (%)	2.94
Equivalent pyrite (%)	1.83
Original pyrite (%)	3.59
% Pyrite oxidised	49.00
% Oxidised per Year	9.80

Table 7.1: Pyrite calculations for the sample material, based on data obtained as part of the 2009 investigations.

As briefly discussed in Chapter 4, from the chemical data for the material, the following factors can be calculated as shown by both TRL report 447 (2005) and BRE Special Digest 1 (2005).

- The total potential sulphate (TPS) = 3 x total sulphur
- The oxidisable sulphides (OS) = TPS - ASS (acid soluble sulphate)
- The current or equivalent pyrite level of the material = OS x 0.623
- The original pyrite level of the material = total sulphur x 1.87

The relevant limiting values for material used in construction are discussed in Section 4.2, however, some of the relevant values are noted below:

Source & Year	Chemical Test	Maximum Value
TRL (2001)	WSS	2.3 g/l SO <sub>4</sub>
	TPS	0.6 % SO <sub>4</sub>
	OS	0.46 % SO <sub>4</sub>
NRA (2000)	WSS	2.3 g/l SO <sub>4</sub>
NRA (2004)	ASS	0.24 SO <sub>4</sub>
Clause 804 (2000)	WSS	2.3 g/l SO <sub>4</sub>
	W.A.	2 %
	ASS	0.24 % SO <sub>4</sub>

Table 7.2: Limiting values for material used within 0.5 metres of concrete, as relevant at time of construction in 2004. WSS: Water Soluble Sulphate; TPS: Total Potential Sulphate; OS: Oxidisable Sulphides; ASS: Acid Soluble Sulphate; W.A.: Water Absorption

## 7 Characterisation of a Typical Irish Fill Material

Comparison of Tables 7.1 and 7.2 show that the material as tested in 2009 exceeds all of the suggested limiting values with the exception of the Water Soluble Sulphate. However, reference to BRE Special Digest 1 (BRE, 2005) places the material in Design Sulphate class DS-3, which carries a warning as to the potential risk of sulphate attack if using this material in construction close to buried concrete.

Comparison of these values with current values for the material is presented and discussed in Section 7.4.

The rock material as received contained fragments mostly 100 mm or less along their longest axis, with occasional larger pieces up to 150 mm. Since the material was being prepared for placement in the swell tests (Chapter 8), it was put through a 63 mm sieve before being used for any of the laboratory tests. Since Clause 804 material is required to have 100% pass through a 63 mm sieve, this was not considered to be a problem for the PSD comparisons discussed in Section 7.2. However, it does show that the material was non-compliant, in that it contained material above the size range given by Clause 804.

Considering the generally small sizes and irregular shapes of the pieces of rock, whole rock strength tests could not be carried out. Similarly, re-constitution of a sample would have been difficult and unlikely to be representative of in situ values.

Simple tests carried out on a range of fragments of material showed that there is a variance in strength between the siltstone and mudstone fragments. The siltstone is generally stronger than the mudstone, which has a tendency to split easily along laminations when they are present. Additionally, once the mudstone has split along laminations, which are often coated with gypsum crystals, the remaining pieces can be broken with simple hand pressure, therefore they would be classified as weak.

Field tests estimate the strength of the mudstone to be weak (around 1.25 – 5 MPa, with the more heavily weathered material being at the lower end of this range) and the siltstone to be of medium strength (5 – 12.5 MPa) (ISO, 2003).

With the pyritic material being used as fill, there are two general locations in which gypsum crystals will precipitate: along laminations within the larger rock fragments, and within the fine material around and between the larger fragments.

In order to consider the fine or clay fraction of the fill material, PSD tests were carried out, as detailed in Section 7.2, and Atterberg limit tests were carried out to determine the plastic and liquid limits of the material.

The tests were carried out as per the Manual of Soil Laboratory Testing (Head, 1992) and BS 1377 Parts 1 and 2 (BSI, 1990; BSI, 1990). All samples for moisture contents were oven dried at between 65 and 75°C. This temperature range is required for materials containing gypsum – any lower and there is a risk of the heat producing more gypsum than was present when the sample was taken, and any higher and the water bonded to the gypsum molecule may evaporate.



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Table 7.3 shows the summarised values for the material and Figure 7.4 shows the position of the material on the plasticity chart.

Factor	Value
Moisture content (%)	7.8
Liquid Limit (%)	39.8
Plastic Limit (%)	24.8
Plasticity Index	15.0
Liquidity Index	-1.1
Relative consistency	2.1

Table 7.3: Atterberg limits and associated values as determined in laboratory testing.

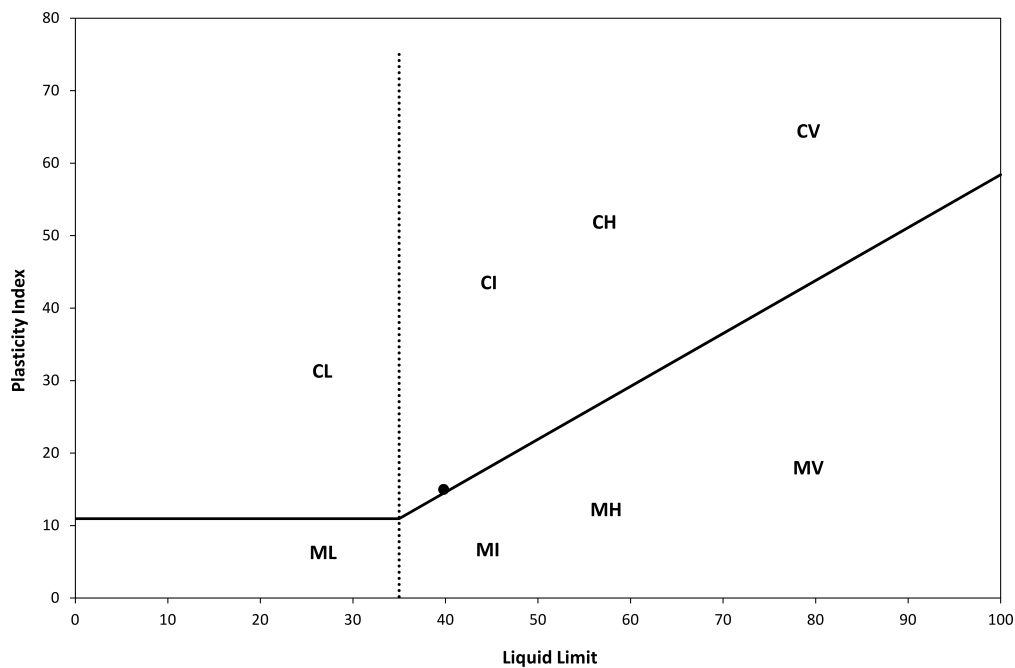


Figure 7.4: Position on the Casagrande chart of the material as tested in the laboratory.  
M: Silt, C: Clay. L: low plasticity, I: intermediate plasticity, H: high plasticity,  
V: very high plasticity.

## 7.2 Particle Size Distribution

As discussed in Section 4.2, when many of the damaged properties were constructed, it was common practice to simply specify Clause 804 material for use as fill beneath the ground floor slab, and the specification comes with upper and lower bounds for the particle size distribution, as shown in Figure 7.5. These bounds are intended to provide a

## 7 Characterisation of a Typical Irish Fill Material

material that is well graded and compacts well and easily in situ, as part of the National Roads Authority Ireland Specification for Roadworks (NRA, 2013).

One of the arguments often made in the Irish cases is that the material ordered from the quarries might have been specified as “Clause 804” material, but it often does not match this specification when studied in the laboratory after remediation.

Figure 7.5 shows the PSD for the material obtained as part of the initial testing in 2009 as well as the retest when it arrived in the laboratory after remediation, as compared to the Clause 804 upper and lower limits.

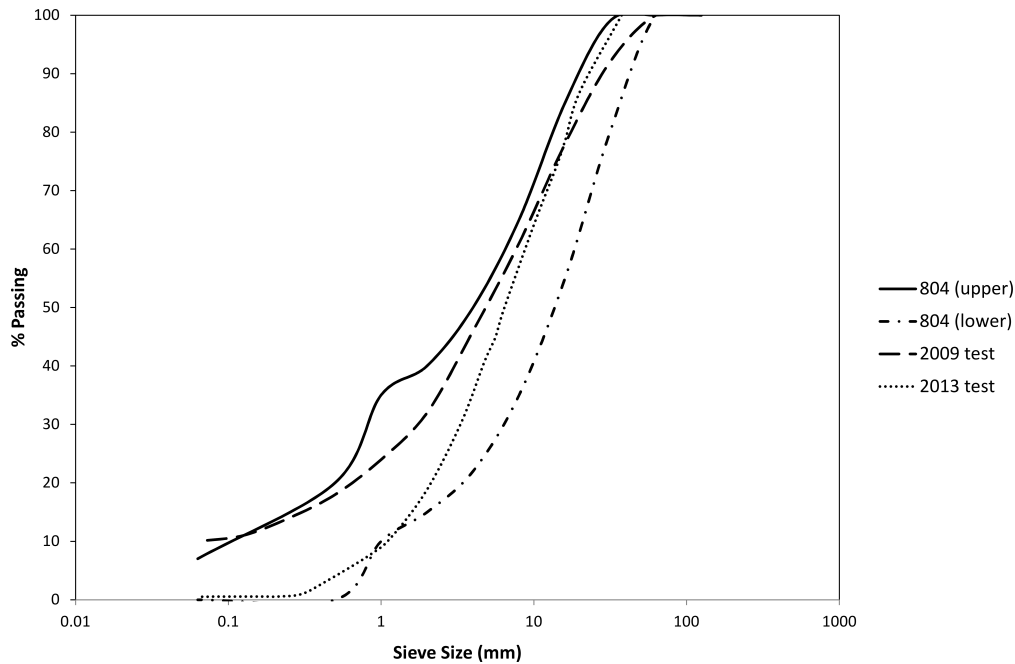


Figure 7.5: PSD for the material used in the laboratory testing from the 2009 geotechnical tests and as received in the laboratory.

There is a noticeable difference in the distribution between the 2009 and 2013 tests, with the 2013 test apparently containing less fine material than that in 2009. This is likely due to natural variations in the fill material – although Figure 7.1 shows the location that the sample was taken from, no such information was available as to which bags of material were taken from which location during the remediation process.

However, one thing that is immediately noticeable from Figure 7.5 is the fact that the material largely falls within the limits assigned to Clause 804 material, the main exception being that the 2009 test exceeds the upper limit in the very fine material present.

Considering the generally weak nature of the mudstone component, it was hypothesised

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that the material in many cases may have been within the limits when purchased as fill material, with the process of compaction within the foundations breaking down some of the material and leading to an increase in the fines component. To determine whether this was the case, and to determine how compaction should best be carried out in the planned swell tests (Chapter 8), a series of compaction tests were carried out on the material in the laboratory, with PSD tests carried out after each round of compaction to determine how the material changed.

Compaction was carried out in a 100 mm diameter mould, with a 2.5 kg compaction hammer as detailed in the Manual of Soil Laboratory Testing (Head, 1992) and BS 1377 Part 2 (BSI, 1990). Material for this part of the test was compacted at its natural moisture content, i.e. the moisture content when it was in situ, in this case a value of 7.8%. Figure 7.6 shows the PSD curves for the initial values of the material, and for the tests carried out after each compaction test.

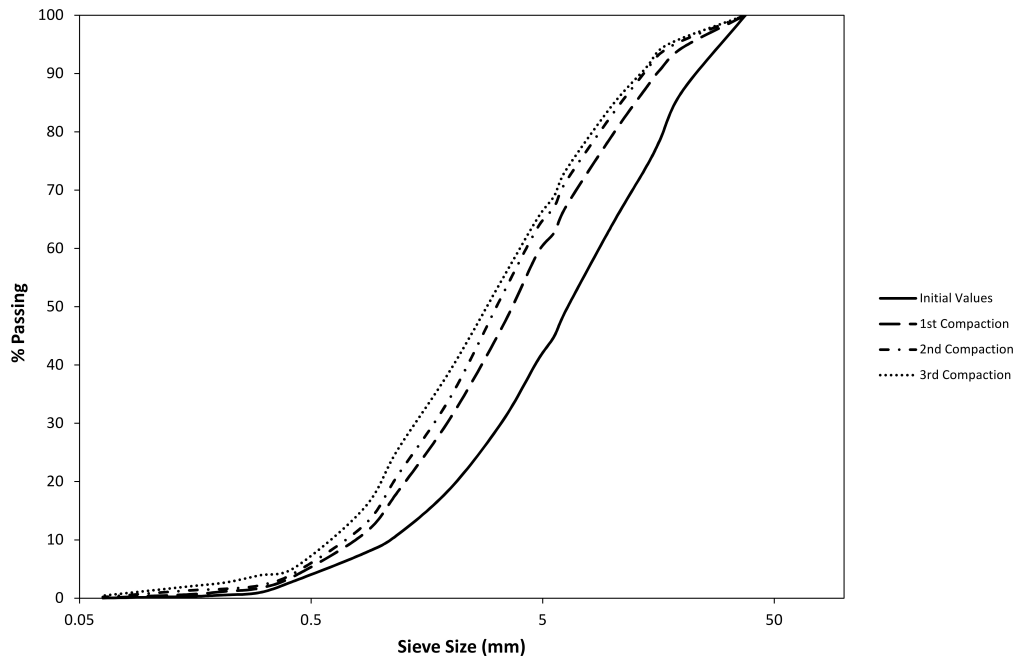


Figure 7.6: Fine-ward progression of the PSD curve with repeated compaction.

There is a general fine-ward movement of the PSD, showing the overall breakdown of the material with each subsequent round of compaction. Considering this breakdown of the material, it would seem that the initial grading of the material before placement as fill at a property would have met the limits of Clause 804.

However, two other factors should be considered before using this as proof that the material was suitable for use.

Firstly, the material was passed through a 63 mm sieve before beginning the PSD tests,

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removing a portion of the material that was larger than this. Indeed the largest pieces of rock material measured up to 150 mm along the longest axis – all of this material exceeds the grading specifications as given in Clause 804.

Secondly, part of the specification for Clause 804 material is that it should not contain rock that easily breaks down under compaction. Considering the breakdown of the material shown in Figure 7.6, even if the grading would appear suitable before placement, the strength of the particles would not be.

In order to consider this fine fraction separately, the material that passed through the 0.063 mm sieve was then analysed with the Malvern Mastersizer in the laboratory in order to see how this part of the distribution broke down from the initial material to that after three rounds of compaction.

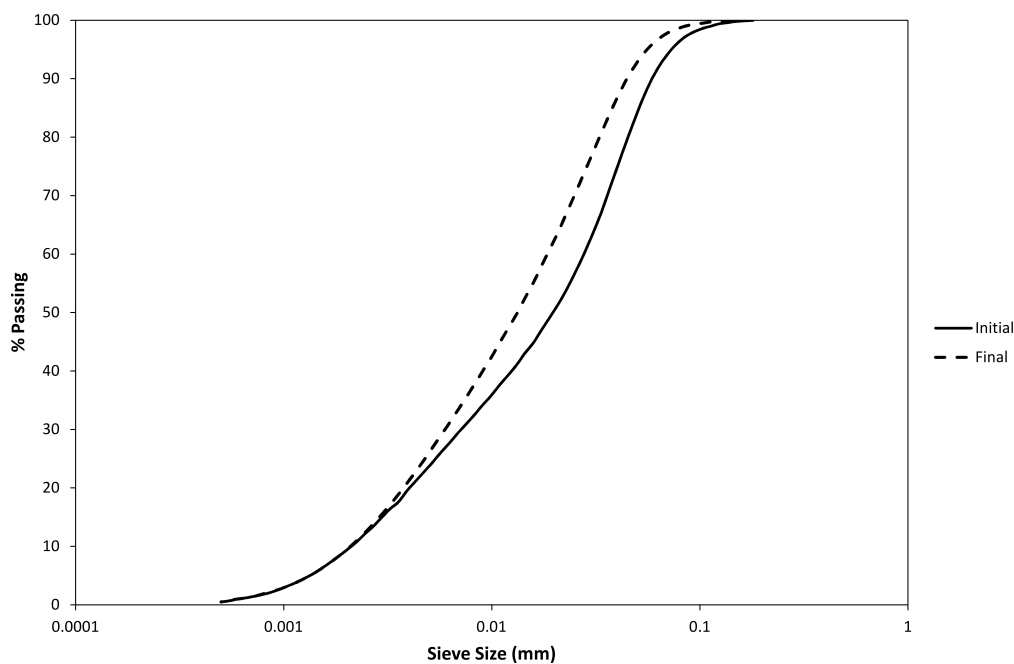


Figure 7.7: Change in the very fine content of the material between the initial test (as received in the laboratory) and after the third and final cycle of compaction, values as determined by the Mastersizer.

Figure 7.7 shows the change in the fine fraction from the initial laboratory tests to after the third compaction. No deliberate drying out of the material took place during these tests, and all material was stored in airtight containers in a cool location between testing phases, however, it is likely that a small amount of drying out occurred due to the nature of both the compaction and sieving tests.

There is a more pronounced increase in the material above 0.01 mm than there is below, which is likely due to the material below this size being too small to be broken down by

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the compaction process. Material above this point and below 1 mm or so is also unlikely to be overly affected by the compaction process, but material of this size is produced due to the breakdown of the larger particles.

This breakdown of the larger rock fragments is shown in Figure 7.8, which shows the difference between the Clause 804 limits and the PSD of the material after the compaction tests. This material had by this stage undergone 3 stages of compaction in the laboratory in addition to the initial compaction when placed in situ at the property.

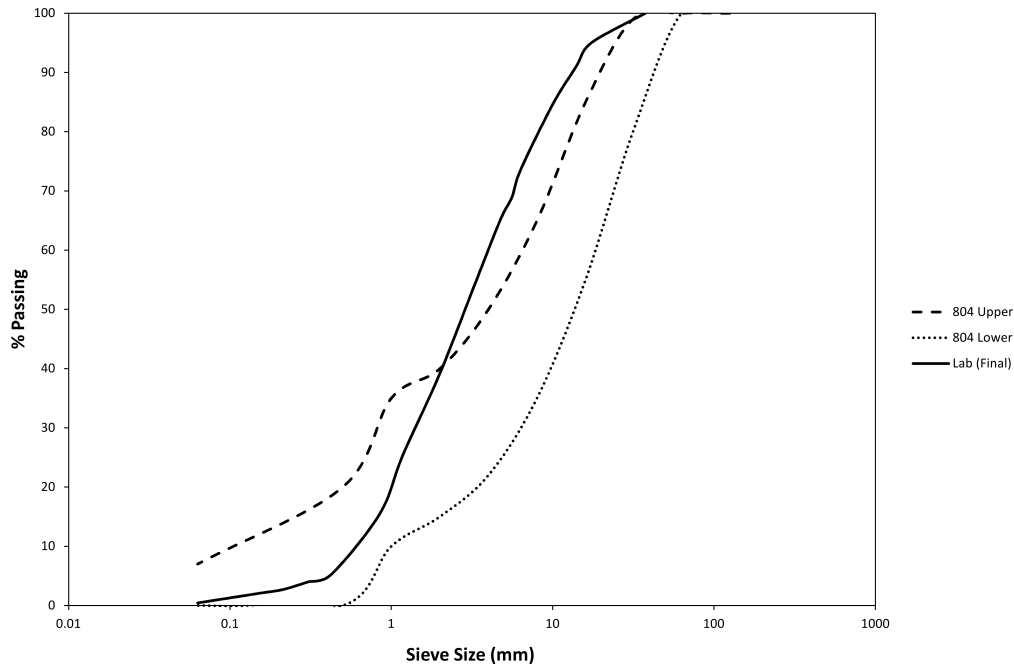


Figure 7.8: Comparison of the PSD for the material with the Clause 804 limits after the third round of compaction.

### 7.3 Specific Surface

The specific surface of a particle is defined as the ratio of surface area to mass, and generally speaking, for a given material, as the fines content increases, so too does the specific surface. Even a small increase in the fines content may affect the behaviour of a material, as the fines will fill spaces between larger particles that may previously have been filled by air or water. Most often, the specific surface is measured via gas adsorption or methylene blue absorption methods, however, more recent methods work by using the PSD of a material to gain an idea of its specific surface (Santamarina et al., 2002, Clarke et al., 2012).

The specific surface of the material is also linked to its reactivity: a material with a

## 7 Characterisation of a Typical Irish Fill Material

larger specific surface has a larger surface area over which reactions may occur. In the case of the pyritic mudrocks, this is a larger area over which air and moisture may access the pyrite to begin the oxidation reaction.

Based upon this, it would be expected that the fine fraction of the mudrock would allow the reaction in this material to progress quicker than in the coarser fraction with a smaller specific surface. However, the pyrite within this high specific surface material would be oxidised more quickly, and so while the reaction in the coarser material would progress slower, it would react for longer. This is discussed further in Chapter 8.

Clarke et al. (2012) cite Equation 7.1 as a simple measure of the specific surface of a material, in which all particles are assumed to be spherical.

$$S_s = \frac{3}{G_s r} \quad (7.1)$$

Where  $S_s$  is the specific surface,  $G_s$  is the specific gravity, and  $r$  is the radius of the particles.

Using this method, a specific surface value can be obtained when given a specific particle size, for example at the  $D_{50}$  value, this being the grain size at which 50% of the material passes through the sieve of that size, which would give a value of  $0.18 \text{ m}^2/\text{kg}$  for the material.

However, in order to obtain a better picture of how the material behaves as a whole, it is possible to obtain a specific surface value for the material at each sieve size, and then weight each of these values based on the % of material that passed a particular grain size. Table 7.4 summarises the values for the initial PSD curve obtained from the material in the laboratory.

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Sieve size (mm)	% passing	$S_s$	Weighted $S_s$
37.5	100	0.06	–
20	86.94	0.12	0.02
16	78.53	0.15	0.01
14	74.03	0.17	0.01
10	64.15	0.23	0.02
6.3	49.40	0.38	0.06
5.6	44.92	0.41	0.02
4.75	40.77	0.49	0.02
3.35	30.36	0.69	0.07
2	18.88	1.16	0.13
1.18	10.88	1.96	0.16
0.85	7.74	2.72	0.09
0.425	2.99	5.44	0.26
0.3	0.97	7.71	0.16
0.212	0.56	10.91	0.05
0.15	0.30	15.41	0.04
0.063	0.03	36.70	0.10

Table 7.4: % passing and associated values for the laboratory material.

Taking the sum of those weighted  $S_s$  values gives a specific surface of **1.196 m<sup>2</sup>/kg** for the material.

In comparison to this, the Santamarina et al. (2002) method considers the coefficient of uniformity, the specific gravity and the  $D_{50}$  value for the material. Equation 7.2 is that used by Santamarina et al. (2002).

$$S_s = \frac{3(C_u + 7)}{4\rho_w G_s D_{50}} \quad (7.2)$$

In which  $S_s$  is the specific surface,  $C_u$  is the coefficient of uniformity,  $\rho_w$  is the mass density of water,  $G_s$  is the specific gravity and  $D_{50}$  is the 50% passing value for the material.

The coefficient of uniformity,  $C_u$  is the ratio of the  $D_{60}$  value to the  $D_{10}$  value, and is a measure of the grading of the material. If  $C_u < 4.0$  the material is well graded, if  $C_u > 4.0$  the material is either well graded or gap graded, as determined by the PSD chart (Smith, 2006).

Figure 7.9 shows the PSD curve for the material with the different percentage passing values marked, these values are summarised in Table 7.5.

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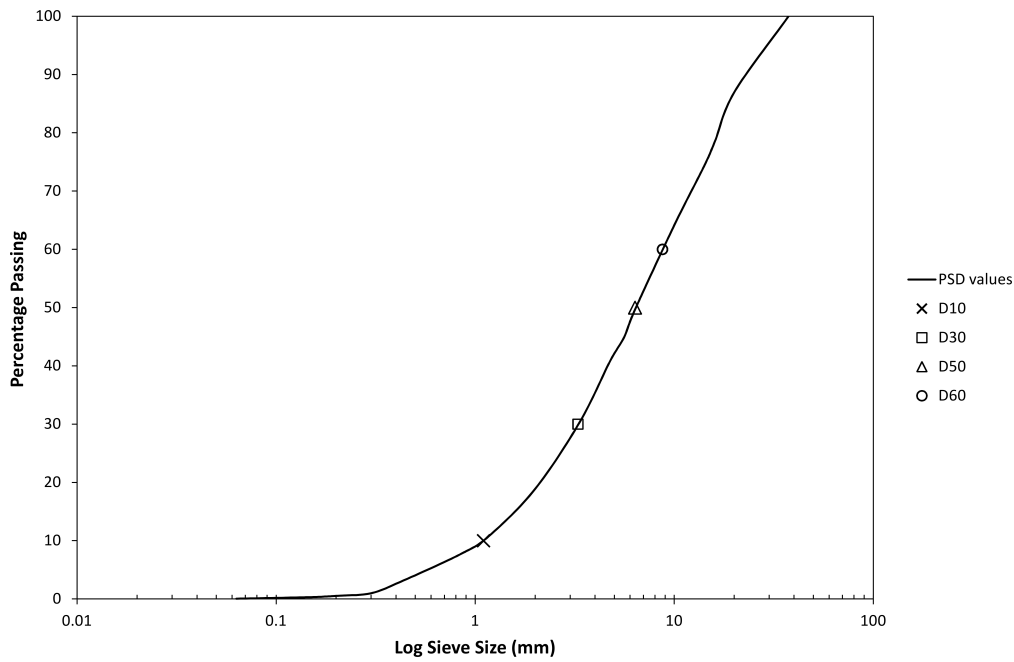


Figure 7.9: PSD chart showing the key percentage passing values.

Property	Value
D <sub>10</sub>	1.1 mm
D <sub>30</sub>	3.28 mm
D <sub>50</sub>	6.35 mm
D <sub>60</sub>	8.72 mm
D <sub>100</sub>	37.5 mm
C <sub>u</sub>	7.93
C <sub>c</sub>	1.12

Table 7.5: % passing and associated values for the laboratory material.

Taking these values into account, and taking a specific gravity of 2.6 for the mudstone, the values can be used along with Equation 7.2 to determine that the specific surface for the material is 0.68 m<sup>2</sup>/kg.

This is vastly different to the 1.196 m<sup>2</sup>/kg value given by the first method. In order to see if accounting for the other values from D<sub>10</sub> – 100 would provide a more accurate result, Equation 7.2 was applied to the other D values, and the totals averaged to give a specific surface of **1.30 m<sup>2</sup>/kg** for the material.

It should be noted, however, that this does assume that the particles are spherical in shape. Santamarina et al. (2002) discuss modifications to account for particles that are



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longer along one exist than the others, however, it still assumes a uniformity of particle shape throughout the material that is not seen in the Irish mudrocks.

The same analyses were performed on the PSD obtained after the three rounds of compaction, in order to determine how the compaction and break-down process affected the specific surface. Figure 7.10 shows the PSD curve for this analysis.

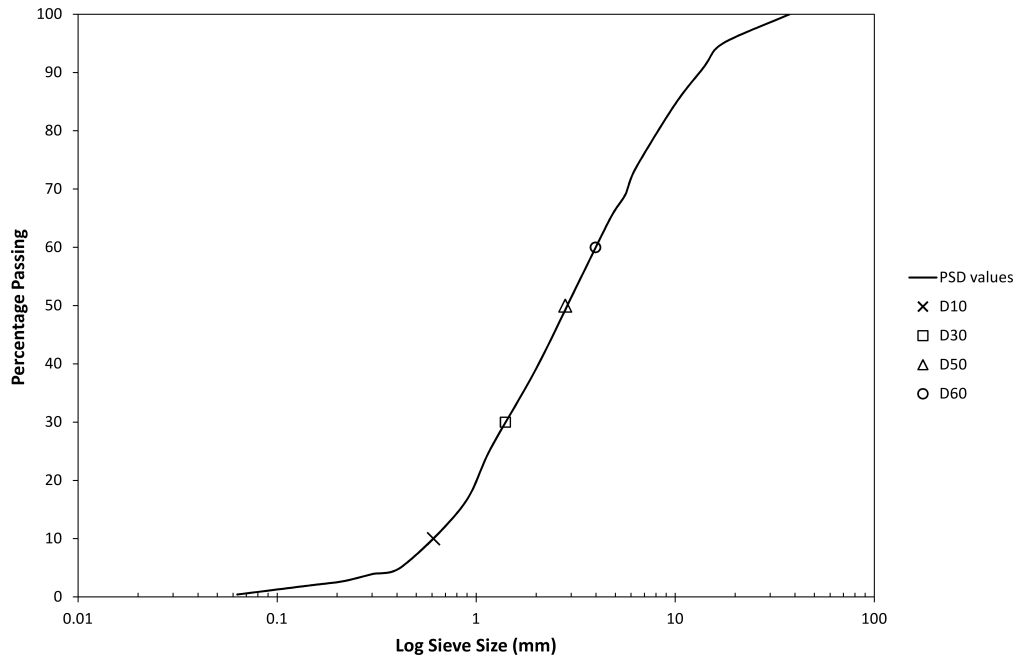


Figure 7.10: PSD chart showing the key percentage passing values after three rounds of compaction.

Equation 7.1 gives a specific surface of **2.38 m<sup>2</sup>/kg** and Equation 7.2 gives a value of **2.34 m<sup>2</sup>/kg** for the material after the third round of compaction, showing again the increase in fine material.

The  $C_u$  values for the material however show less of a change, being 7.93 for the initial tests and 6.51 after three rounds of compaction. The breakdown of particles applies to all but the finest material, suggesting that it remains well graded.

### 7.4 XRD Comparison Testing

One of the noticeable factors from considering the geotechnical data available for the various developments, is the amount of scatter that is seen in the data, which is partly covered in Chapter 5. The link between the XRD data and the damage rating for Development Alpha was not considered as part of the analysis carried out in Chapter 5

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largely due to the limited number of results available for the properties within that development – many of the properties were tested in 2009 – 2011 when the scale of the pyrite problem was first becoming apparent, and before the full testing protocol had been confirmed. However, for completeness of the discussion in this section, Figure 7.11 shows the XRD values where available plotted against the damage rating of the corresponding property.

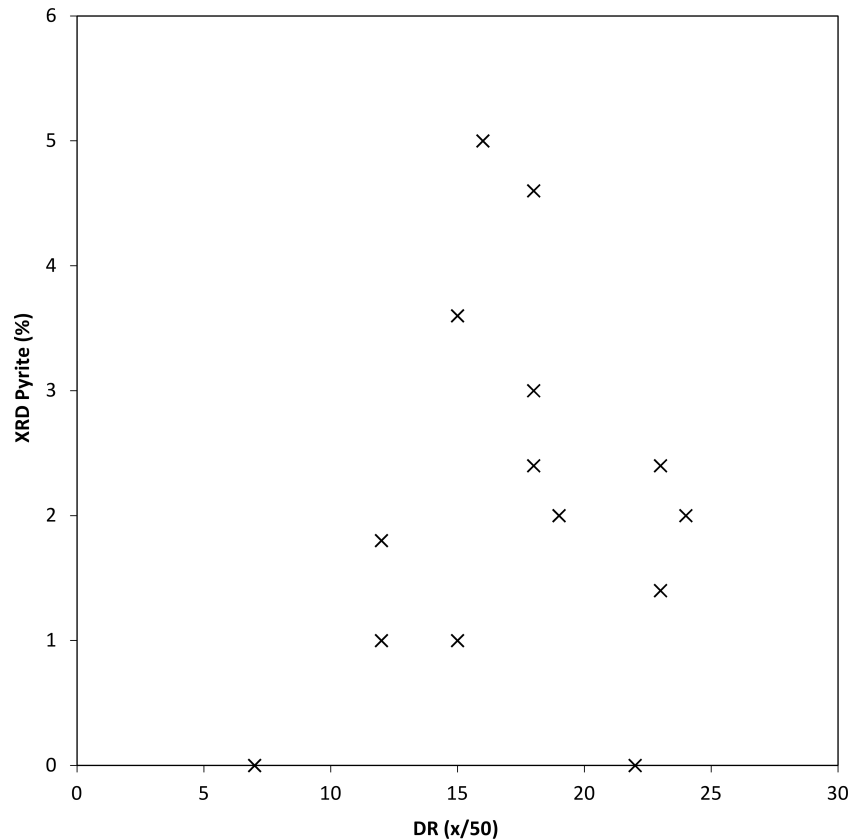


Figure 7.11: XRD pyrite values as linked to the Damage Rating for properties of Development Alpha

The correlation between the XRD pyrite value and the damage rating was calculated to be -0.011, meaning that there is no correlation seen between the data that is available. This would seem to suggest that there is no link between the two values, however, two factors need to be taken into account:

- The XRD value represents the amount of pyrite present in the material at the time of testing, i.e. after the damage seen at the properties had already occurred.
- The correlation between the calculated current value of pyrite (calculated from the chemical tests) and the damage rating was 0.096.

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The latter of these points in particular draws back to the discussion in Section 5.4 wherein there are factors such as structural defects that influence the damage rating. These influences mean that there is not always a correlation in the data between the damage rating and factors that are known to effect the reaction process.

However, the main point of discussion here is that there is a large amount of variation seen amongst the XRD results themselves, with values for Development Alpha ranging from a trace reading to 4.6%. The full range of data is contained in Table 7.6 along with the pyrite values calculated from the chemical tests.

<b>Property</b>	<b>XRD pyrite (%)</b>	<b>Chemical pyrite (%)</b>
A	1	0.42
B	trace	0.36
C	4.6	3.93
D	2	1.29
E	2.4	1.71
F	5	4.21
G	1	0.67
H	<1	0.19
I	3.6	1.64
J	1.4	1.06
K	3	1.73
L	1.8	1.63
M	2	1.88
N	2	0.92
O	2.4	1.69

Table 7.6: The calculated and XRD pyrite values as available for Development Alpha.

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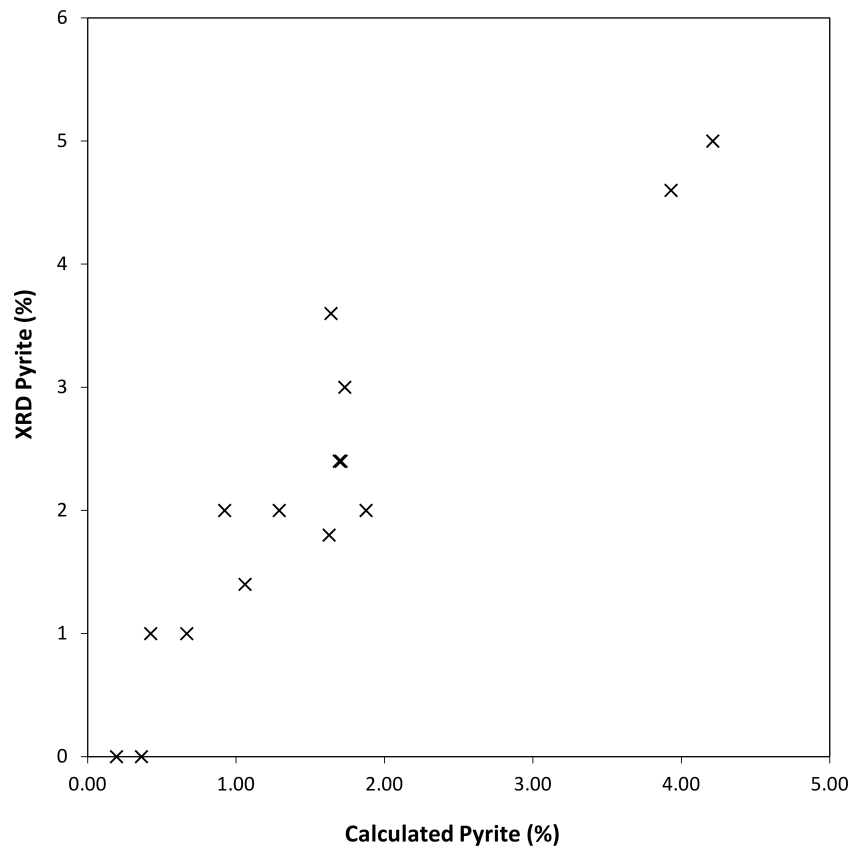


Figure 7.12: Link between the XRD and calculated current pyrite values for Development Alpha.

These values can be seen plotted against one another in Figure 7.12, and shows a Pearson correlation coefficient of 0.92, which indicates that as one increases so does the other, confirming that there is a linear correlation between the two sets of results when using either one to consider the amount of pyrite present in the material as it is examined.

The wide scatter in both the XRD and chemical values, but in particular the XRD, is likely due to the large variation in material over fill content of a single property. This in turn is due to the variation seen within the quarries, which was discussed previously in Chapter 4.

The question at this stage was whether the large amount of variation seen in the XRD results was simply due to this natural variation or whether it was due to differences in the XRD testing process between different laboratories. Considering the small size of the sample used for the XRD tests, it is likely that the differences are due to the natural variations, however, in order to confirm this, an anonymous series of tests were carried out.

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The purpose of these tests was that all laboratories taking part would receive samples of the material described in Section 7.1, which was taken from a single location at a single property. Although the exact location that the bag of material was taken from is unknown, the remediation method means that the material in one bag would have come from a small area of around 1 metre square. Although the primary aim of this section of the testing was to consider the variations in the XRD testing process, chemical tests were also carried out on the samples, as detailed below. Variation in the chemistry of the fill material is also looked at in Section 7.5.

Figure 7.13 shows the material as prepared and shipped out for testing. Prior to shipping the material was passed through a 20 mm sieve and was dried in the oven at 70°C. Prior to sieving the material had been stored in a cool, dry location, and after sieving and drying, before shipping, it was stored in the refrigerator.



Figure 7.13: Mudrock fill material used in the laboratory testing in its dry state as used for the Comparison Testing

Samples were sent to three different companies that are accredited to carry out tests as per the protocols in place for the Irish pyrite cases, and all are companies that currently carry out tests on Irish mudrocks. A fourth sample was sent to an independent laboratory in order to get a separate reference value for the pyrite and gypsum contents of the material via just the XRD analysis.

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The companies were asked to carry out both XRD and chemical tests on the samples according to their usual practices, either in house or via their usual testing laboratories. Table 7.7 shows the initial results for each sample.

Co.	Sample	XRD pyr	XRD gyps	ASS (%SO <sub>4</sub> )	WSS (mg/l)	TS (%)
A	1	1.3	0.2	3.9	2000	1.6
	2					
B	1	0.8	1.3	2.9	2000	1.4
	2			2.5	2500	1.1
C	1	1	1	3.69	1668	1.46
	2	1	1	3.92	1668	1.58
D	1	2.1	1.2	–	–	–

Table 7.7: Raw data obtained from the comparison testing. Please note: laboratory D was only commissioned to carry out the XRD testing. ASS: acid soluble sulphate; WSS: water soluble sulphate; TS: total sulphur.

From these base values, it is possible to see at a glance the variation that is present in the material. Added to this is the fact that the tests are carried out on an even smaller sub-sample of the material that was sent to the different laboratories, so it is understandable that a certain amount of variation is seen in the data.

IS 398-1 (NSAI, 2013) states that the detection limits for the material generally present in the Irish mudrock samples are between 0.5 – 2.0%. This implies that there is likely to be more error when detecting smaller amounts of a material present within a sample, and accounts for the difference often seen between the XRD pyrite and chemical pyrite values in the Irish cases. Table 7.8 shows the XRD and calculated values for the samples where both XRD and chemical test results were available – this table excludes the data from laboratory D.

Sample	XRD (%)	Current (%)	Original (%)	% Ox.
1	1.3	0.6	3.0	81.3
2				
3	0.8	0.8	2.6	69.1
4		0.5	2.1	75.8
5	1	0.4	2.7	84.3
6	1	0.5	3.0	82.7

Table 7.8: Comparison of the XRD and chemical values for pyrite present in the samples, along with the calculated original pyrite and percentage of this original value that has been oxidised.

From Table 7.8 it can be seen that the variation between the XRD and chemical values

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for the current pyrite level falls between 0.5 and 1.0%, within the limits suggested by IS 398-1 (NSAI, 2013).

In order to better consider these variations, and especially the amount of variation for each factor that was measured, the standard deviation for each was considered. Table 7.9 shows the mean values for the raw data and the standard deviations from these mean values.

<b>Factor</b>	<b>Low</b>	<b>High</b>	<b>Mean</b>	<b>S.D.</b>
ASS (% SO <sub>4</sub> )	2.5	3.92	3.38	0.64
WSS (mg/l)	1668	2500	1967	341
TS (% S)	1.1	1.6	1.43	0.20
XRD Pyrite	0.8	2.1	1.24	0.51
XRD Gypsum	0.2	1.3	0.94	0.43

Table 7.9: Average values and standard deviation from the mean for each of the measured factors. ASS: acid soluble sulphate; WSS: water soluble sulphate; TS: total sulphur.

The standard deviations initially seem to be relatively low, however, when compared to range of some of the values they reflect a relatively large amount of scatter. The standard deviations for the chemical values range from 10 – 20% of the mean, whereas the XRD values range 40 – 45% compared to the mean value.

What is initially unclear is whether this variation is due to the natural variations in the material, or due to variations in lab practices. For the latter, comparing the results between companies and laboratories gives some idea of what variation there is due to testing practices. All companies to which samples were sent, used different XRD testing facilities, however, only 2 different chemical testing laboratories were used. Companies A & B both used a single chemical testing laboratory, although the samples were further anonymised before reaching the chemical testing stage, as both companies assigned their own unique reference numbers to the samples they received.

It is therefore possible to consider variation in the chemical data as shown in Table 7.10.

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Lab.	Sample	ASS (% SO <sub>4</sub> )	WSS (mg/l)	TS (%)
1	A1	3.9	2000	1.6
	A2			
	B1	2.9	2000	1.4
	B2	2.5	2500	1.1
	Mean	3.10	2167	1.37
	S.D.	0.72	288.7	0.25
	% diff	23	13	18
2	C1	3.69	1668	1.46
	C2	3.92	1668	1.58
	Mean	3.81	1668	1.52
	S.D.	0.16	0	0.08
	% diff	4	0	6

Table 7.10: Raw data obtained from the comparison testing. ASS: acid soluble sulphate; WSS: water soluble sulphate; TS: total sulphur.

Although less data is available for laboratory 2, it would seem that its data is more consistent, with low variation of the data around the mean value. However, without more data is unfair to say that this is simply due to laboratory practices, when it may be that there was less variation of the material in the samples they received.

The data received from laboratory 1, although showing more variation than that from laboratory 2, is also relatively consistent, with a maximum variation of 23% of the mean value. Although this could be considered a large variation, even the lowest value exceed the failing values stated in IS 398-1 (2013) of 0.2% SO<sub>4</sub> for the acid soluble sulphate, 1500 mg/l for the water soluble sulphate, and 1.0% S for the total sulphur.

As all of the companies used different XRD facilities, the XRD results can only be compared according to those provided by a given company as already done.

### 7.5 Grid Comparison Testing

In order to further determine whether there is a direct link between the chemistry of the fill material and the amount of uplift seen in a given area of the floor slab, it was decided to use grid sampling in order to compare the two. Samples were taken from a property undergoing remediation, and the location of each sample recorded so that the chemical composition of the material at each location could be compared to the uplift in that area.

The property was constructed in 2004 and inspected for the first time in 2015, with



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remediation beginning in summer 2015. Analysis of the structural report with the IS 398-1 framework (as detailed in Chapter 5) gives a damage rating of 23 out of 50, with Figure 7.14 showing the initial floor level survey for the property from which the samples were taken.



Figure 7.14: Floor level survey for the property from which the samples were taken (used with permission of Aidan O’Connell Associates). All marked lines showing changes in floor level are 1 metre in length.

From the data included in this floor level survey, it is possible to consider the uplift of

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the slab as a whole. In this case, we focus on the area from which the samples were taken, which is the kitchen area at the rear of the property.

In order to create an image of how the surface level of the slab varies across its length and width, certain assumptions were made:

- All uplift was even along the length of each measured interval.
- The outer edges of the slab were counted as having an uplift of zero. Although it is likely that movement had occurred in these areas, no data for this is available. Therefore measurements relative to these areas were assumed to start at a value of 0.
- For all measurements not starting at the outer edge of the slab, the initial value for a given measurement was taken to be the final value from the previous measurement in cases where the uplift direction was the same.

Taking these assumptions into account, it is possible to create a grid of points across the section of the slab in question in order to obtain a series of contours for the height at each point on the grid. This is shown in Figure 7.15a.

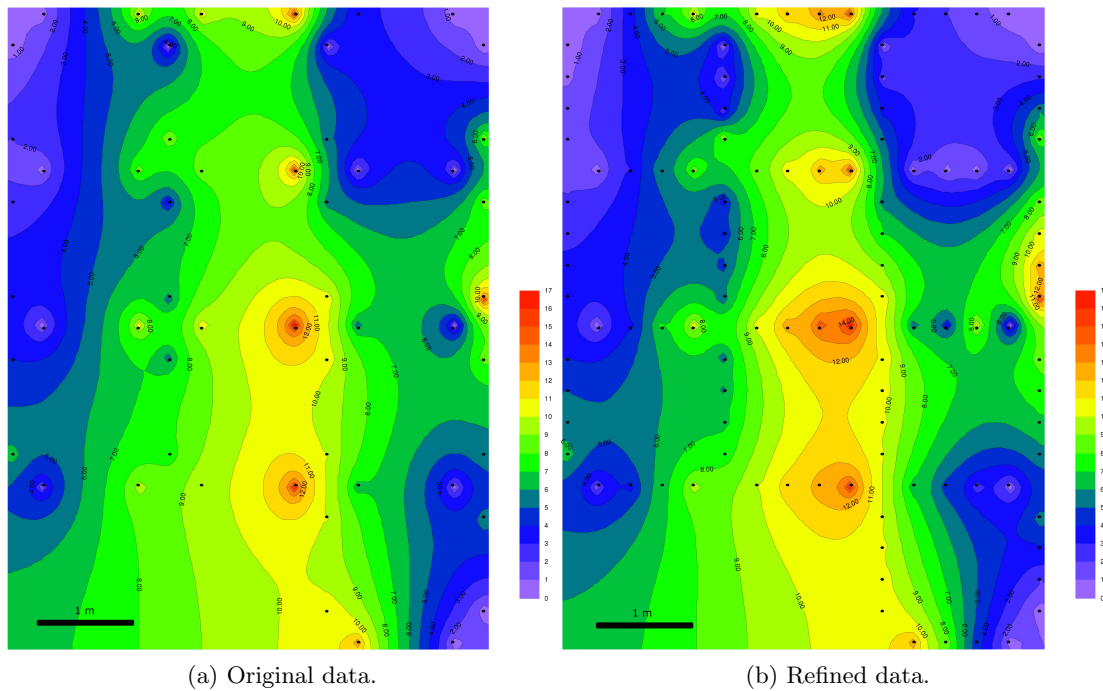


Figure 7.15: Contouring of floor slab levels in the kitchen of the property from which samples were collected. All contours given in mm above zero, grid values taken from the floor level data are shown as black dots.

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With the assumption that the uplift occurs evenly along each line of measurement, it is possible to add additional values for each line that was used for Figure 7.15a. This was used in order to attempt to refine the data to provide more information. However, as can be seen in Figure 7.15b, the improvement to the image is limited and it would require the addition of a large amount of data to smooth the contours enough to aid analysis.

Further improvement of the image would be likely with the addition of points of measurement within the grid itself, however, since these points in this case would be estimated from the levels around, it creates an additional layer of error within the analysis. In this case it was decided to work with the image shown in Figure 7.15a, which is amended in Figure 7.16 to show the walls of the property, as this gave a clear image of the uplift of the slab to compare to the chemical data, while minimising the errors.

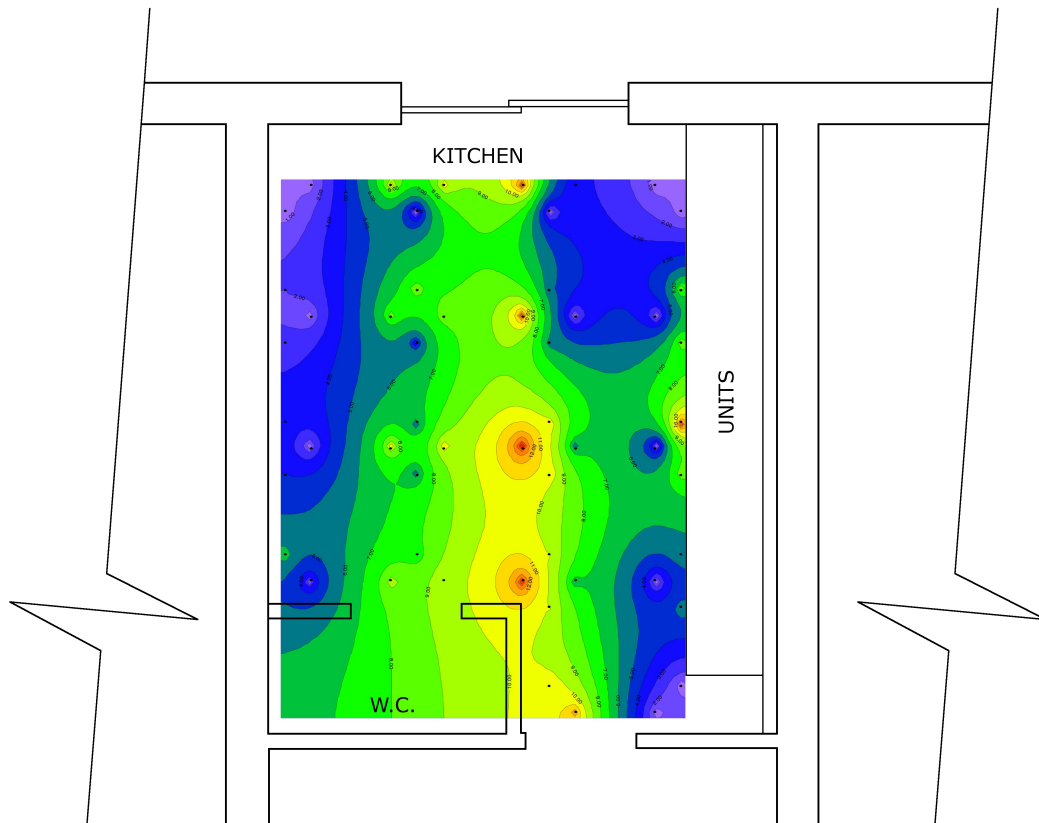


Figure 7.16: Uplift of the floor slab as relative to the structure, all contours given in mm above zero.

This process highlights the ridge in the centre of the kitchen floor slab that was shown in the floor level survey (Figure 7.14), with the majority of the uplift occurring in this

## 7 Characterisation of a Typical Irish Fill Material

area and to the right hand side of the slab near the units.

Samples were collected from the property during remediation, once the floor slab, insulation and DPC had been removed. They were taken on a grid as shown in Figure 7.17, measured relative to the external walls. At that time the analysis discussed above had not been completed, so the samples were taken based upon the floor level survey in order to have samples relative to areas that showed both large and small amounts of uplift.

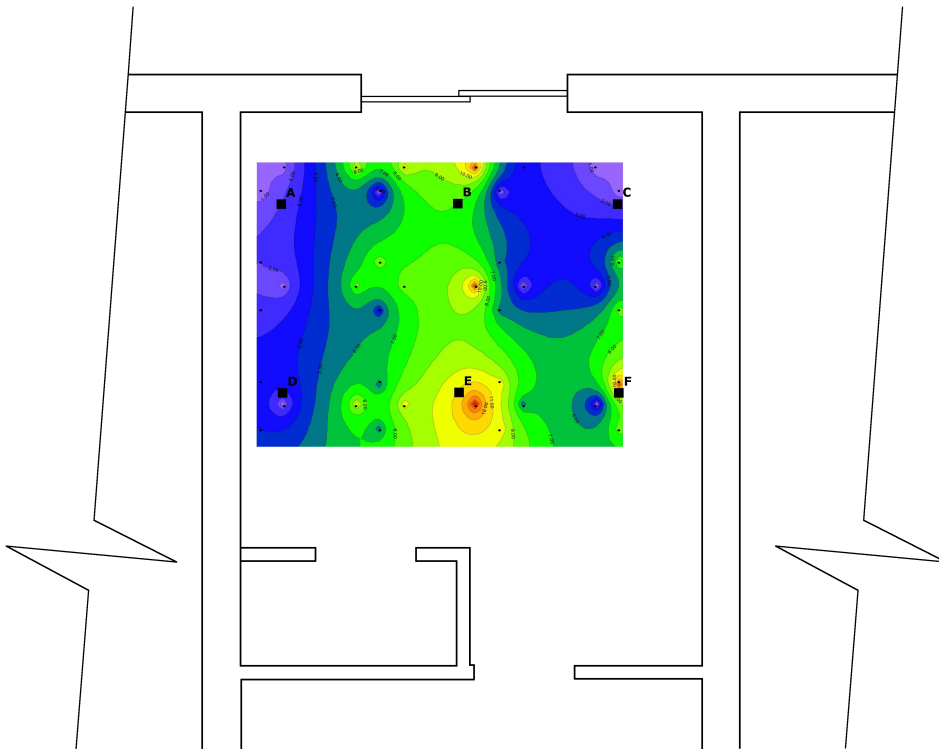


Figure 7.17: Sampling locations at the property relative to the uplift of the floor slab.

Figure 7.18 shows the sampling location and the material as seen in situ. In particular, Figure 7.18b shows the variation of the material that was seen on site.

## 7 Characterisation of a Typical Irish Fill Material



(a) Fill material below the kitchen area, with the double doors at the rear of the property.



(b) Limestone material (pale brown) mixed in with the mudstone (dark grey) fill material.

Figure 7.18: Images of the sampling location and the fill material.

The paler material was similar in colour and appearance to the clean limestone material used to replace the defective fill material after remediation. It is likely that a combination of the limestone and mudstone fill materials were used at the time of construction, with one replacing the other after the initial material had run out.

The limestone material was predominant in the lounge area of the property, with only small amounts in the kitchen area where sampling occurred. It should be noted that a large area of limestone was present close to the front wall of the property, where uplift amounts were low (Figure 7.14), but in areas where smaller amounts of limestone were present, such as in the doorway area between the lounge and kitchen, uplift rates were consistent with those in areas containing only mudstone.

Samples were taken primarily from areas designated by uplift, as discussed above, but were also taken from areas with minimal limestone content both laterally and vertically within the fill in order to limit the number of variables within this set of tests. All samples were taken within the top 300mm of the fill and were sealed in numbered plastic bags that were despatched the same day for analysis and chemical testing.

Table 7.11 shows the initial chemical results obtained from the six samples along with the calculated original and current pyrite contents of the material.

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Location	WSS	ASS	TS	Cur Pyr	Orig Pyr	% ox.
A	1594	1.69	0.73	0.31	1.37	77.2
B	1544	2.06	0.88	0.36	1.65	78.0
C	1214	0.70	0.32	0.16	0.60	72.9
D	1612	2.32	1.02	0.46	1.91	75.8
E	1666	1.32	0.62	0.34	1.16	71.0
F	1488	1.03	0.70	0.67	1.31	49.1

Table 7.11: WSS: water soluble sulphate (mg/l), ASS: acid soluble sulphate (% SO<sub>4</sub>), TS: total sulphur (% S), Orig Pyr: original pyrite (%), Cur Pyr: current pyrite (%), % ox.: percentage of pyrite oxidised.

These values can be plotted into contour plots similar to those shown for the floor level data above. In each case, the value of a given chemical measurement is plotted at each point and contours applied to show the variation across the area of floor slab in which sampling occurred.

Figure 7.19 shows the simplified topography of the floor slab adapted from Figure 7.17. In this simplified topography some of the contours have been removed in order to allow a clearer view of any intersections between the uplift and the chemical data. All contours shown in Figure 7.19 are in mm above zero – the level at which the floor slab would reside if no uplift had occurred.

## 7 Characterisation of a Typical Irish Fill Material

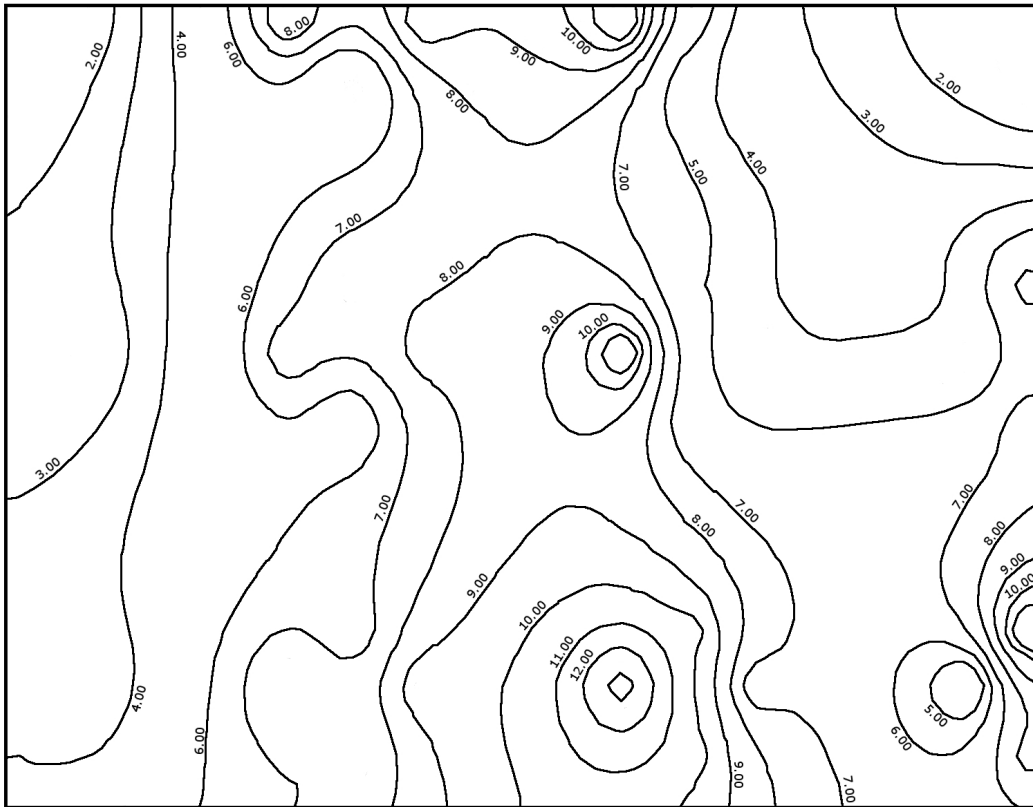


Figure 7.19: Simplified topography of the ground floor slab uplift. All line measurements in mm above zero.

This simplified set of contours is then superimposed over the contour plots for the chemical data as shown in Figure 7.20. This allows for comparison of the variation in the chemical values with variation in the floor slab uplift.

## 7 Characterisation of a Typical Irish Fill Material

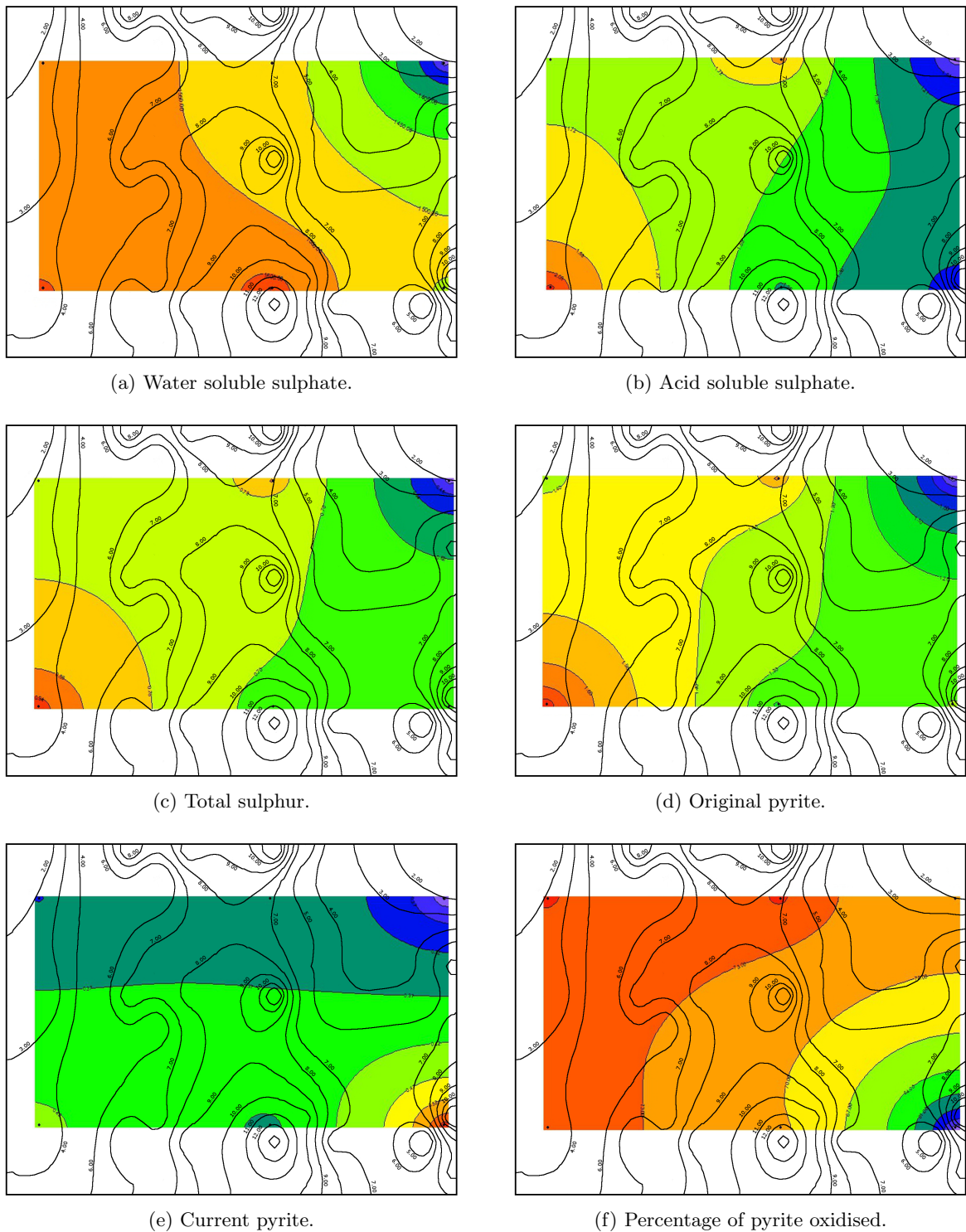


Figure 7.20: Contour plots showing the variation in measured and calculated values across the area of grid sampling. Black lines show floor slab uplift in mm, whilst colour changes show value variation from low (blue/purple) to high (orange/red) of the values returned for each chemical variation.



## 7 Characterisation of a Typical Irish Fill Material

When comparing these values to that of the ground floor uplift in Figure 7.17, it can be seen that the chemical data does not clearly reflect the ridge seen in the floor level survey. There is a mirroring of the chemical data around point C which also shows a lower level of uplift in the floor level survey, with all of the chemical values as well as the current and original levels of pyrite showing lower values in this area.

Of interest are the values at sampling point D, which show some of the highest chemical values from the samples obtained, yet only showed a few millimetres uplift in the floor level survey. There are a few potential explanations for this fact, although since the property was only seen after remediation had begun, these are mostly speculation as the opportunity to be examine the structure for things such as the presence of voids was unavailable. Potential explanations include:

- The presence of a void between the fill material and the floor slab into which the material could expand before exerting pressure upon the floor slab.
- A lower compaction density in this area, meaning that there were voids present into which gypsum could precipitate before causing expansion of the fill.
- There was movement along the outer edge of the floor slab in this area, meaning that the measurements across the slab did not start at zero as assumed when calculating the slab uplift levels, and that there was more uplift in this area than shown in the contour plots.

There is also the suggestion that around point F there was the potential for further uplift than the 10 mm that was seen, as only around 50% of the pyrite in that area had oxidised.

From the data presented here, the immediate conclusion is that due to the variation in the fill material, and the contribution of factors such as the fill density that cannot be easily measured, a simple correlation between the slab uplift and the chemical values is difficult to achieve with any level of certainty. It is possible that with more detailed data, more uplift measurements, more sampling points and even density information, that a more accurate comparison could be obtained. However, considering the amount of data that is routinely available, the difficulty of obtaining accurate density measurements and the cost of the chemical testing itself, it is unlikely at the present time that this can be achieved in a cost effective manner.

One method of better analysing the data, without the need for further testing, would be to use a method such as geostatistics to determine any correlation between the uplift and chemical data at a series of points across the floor slab.

# 8 Laboratory Testing – Development and Modification of the Swell Tests

## 8.1 Introduction

As mentioned in Chapter 2, various suites of laboratory testing have been carried out in the past to determine the methods of expansion and the rates at which this expansion will occur. In particular, when considering the behaviour of the Irish mudrocks, there have been a series of tests carried out more recently.

These tests, presented in Section 8.2, show the development of the equipment currently used in the testing of pyritic mudrocks, and the results obtained from Irish mudrocks in particular. This equipment is essentially the same as that used in the Sheffield tests, also discussed herein, as it has been shown to provide consistent results in an adaptable setting.

Following on from the details of these tests, the design, implementation and results of the Sheffield tests are also discussed within this Chapter.

## 8.2 Previous laboratory testing on Irish mudrocks

### 8.2.1 Golders Tests

Tests to study the expansion of pyritic mudrocks have been carried out previously as mentioned in earlier chapters. The first of these tests were carried out in Quebec as early as 1995, as was briefly discussed by Ballivy et al. (2002). Although no results were given at that time, the tests were described as being carried out in a 150 mm CBR mould.

This system was considered by Maher et al. (2011) for tests they carried out in 2007. However, a 150 mm diameter mould would limit the particle size for these tests to a maximum of 20 mm. Considering that the material they were using had particles up to 63 mm, this was considered to be too limiting and not representative of the material.

The initial design of the Maher et al. (2011) tests used a concrete manhole ring with a 1.2 m internal diameter and a maximum sample height of 1.02 m, that allowed a maximum particle size of 75 mm. The pipe did not have a base, but was instead free standing on a concrete surface – the assumption was that the combined mass of the tube and the

## 8 Laboratory Testing – Development and Modification of the Swell Tests

sample would be enough to keep it stable. However, after 17 weeks it was noticed that the pipe was tilting at the base and that a hairline crack was starting to develop along the side of the pipe. By the time the test had progressed to 50 weeks, this crack was 12 mm in width. Calculations at this time determined that the pressure within the pipe must be about 600 kPa in order to crack the pipe in such a way (Maher et al., 2011). It is not specified whether or not this calculation included the stress induced on the pipe as it tilted due to expansion of the material at the base.

This test setup was modified for tests carried out in 2009, to use a 600 mm internal diameter by 300 mm sample height pipe with a rigid base in order to ensure that expansion would only occur at the top of the pipe (Maher et al., 2011). The tests were placed into a trough in order to allow water to be introduced to the bottom of the samples, with the water being replaced in a 2 week cycle – this was intended to simulate a wetting-drying cycle that would likely speed the reaction process. Table 8.1 shows the test conditions for the modified tests.

Test	Conditions
1	Control sample. High-quality crushed limestone, 50 mm max particle size.
2	Recovered fill, 63 mm max particle size.
3	Recovered fill – duplicate of test 2.
4	Fine fraction of recovered fill, 5 mm max particle size.
5	Coarse fraction of recovered fill, 63 mm max particle size.

Table 8.1: Parameters chosen for the tests carried out by Maher et al. (2011).

The material used was comprised of approximately 95% calcareous mudstone/ siltstone and 5% limestone, and contained an average of 2.7% pyrite. Secondary gypsum was present at the start of the tests, mostly confined to laminations within the mudstone/siltstone particles (Maher et al., 2011). The samples were not kept within a temperature controlled environment, but the ambient temperature during the test was monitored, and ranged between 18 and 24°C. The sample was compacted at an optimum moisture content of 9.0% and had a bulk density equivalent to 2180 kg/m<sup>3</sup>.

Test	Movement after 52 weeks (mm)	Rate at 52 weeks (% fill/year)	Movement after 100 weeks (mm)	Rate at 100 weeks (% fill/year)
1 - control	0.00	0	0.00	0
2 - all in	1.53	0.51	3.49	0.57
3 - all in	1.27	0.42	N/A	N/A
4 - fine	0.83	0.27	1.03	0.17
5 - coarse	0.67	0.22	1.34	0.23

Table 8.2: Amount of movement and rate of expansion per year for each of the tests (after Maher et al., 2011).

## 8 Laboratory Testing – Development and Modification of the Swell Tests

Table 8.2 shows how the amount and rate of expansion of the tests over a 52 and 100 week period. The expansion rates are given as a percentage of the fill thickness per year.

Even though Test 3 was not continued for the full 100 weeks, the expansion rate for this and its duplicate, Test 2, are reasonably similar. As anticipated, the rate for the coarse material is initially the lowest, this is likely due to the decreased specific surface in the coarse material, which would accommodate the initial gypsum precipitation around the rock particles, and due to the way in which more time is required to oxidise the pyrite inside the larger rock fragments (Maher et al., 2011). By the 100 week point, the rate for the expansion of the coarse sample had exceeded that of the fine sample, this was attributed to the pyrite within the coarse fragments beginning to oxidise, and a greater proportion of the pyrite in the fine fraction having been consumed by the reaction process (Maher et al., 2011). As far as is known from the literature, only the chemical changes of the material were considered after testing, as is detailed below.

Test 3 was terminated at 80 weeks in order to see how the chemistry of the material had changed, these results are summarised in Table 8.3, which shows the progress of the oxidation reaction in the material.

	<b>Total sulphur (%)</b>	<b>S as SO<sub>4</sub></b>	<b>Equivalent pyrite (%)</b>	<b>% pyrite oxidised</b>
Start of test	1.30	0.84	1.91	22
80 weeks (top)	1.25	0.96	1.74	26
80 weeks (middle)	1.55	1.45	1.99	31
90 weeks (bottom)	1.40	2.05	1.34	49

Table 8.3: Chemical changes in the material after 80 weeks, with samples taken from the top middle and bottom of the sample (after Maher et al., 2011).

It is the sample taken from the bottom of the pipe that shows the most marked difference in properties compared to those at the start of the test. This is the section of the material that was exposed to the drying/wetting cycle, an area in which 27% of the pyrite that was present at the start of the test was oxidised. At the start of the test 22% of the original pyrite content had been oxidised, this means that in the 80 week period of wetting/drying more pyrite was oxidised than during the 5 year period the material was in situ beneath a property (Maher et al., 2011).

Maher et al. (2011) equate the heave rate for the all-in test to 1.8 mm/year and cite heave rates at that time in Irish properties as being between 3 – 6 mm/year. This is suggested as being caused by the increased fill thickness in situ – the tests had a thickness of 300 mm, while the fill beneath properties frequently exceeds 500 mm. The increased oxidation at the base of Test 3 was believed to be due to the breakdown of the mudrock particles by the wetting/drying cycles, as this breakdown would allow better access of

air and water into the larger particles. This suggests a link between the progression of the heave, the rate of oxidation of the pyrite and the breakdown of the rock particles.

Maher et al. (2011) also cite the grading of the rock as having an effect on the amount and rate of expansion, with the well graded material in these tests showing a higher rate of heave over the length of the testing period. It could also be assumed that the well graded material would show more consistent expansion over a longer period of time, with its fine material supporting oxidation early on in the process, followed by oxidation of the coarser material.

### 8.2.2 Galway Tests – Series 1

The design of the test used by Maher et al. (2011) was modified for use at the University of Galway in 2010. Unlike the initial tests, the first set of Galway tests were carried out in 229 mm diameter polyethylene tubes with a wall thickness of 6.5 mm. The tubes had holes drilled into the sides close to the top and bottom of the tube to allow movement of air and/or water (Sutton et al., 2013). Two variables were considered in this suite of tests – sample height and water depth, as shown in Table 8.4, along with the density to which each sample was compacted.

Test no.	Sample height (mm)	Water depth (mm)	Density (kg/m <sup>3</sup> )
1	500	30	2043
2	500	60	2050
3	500	90	1963
4	750	30	1959
5	750	60	1955
6	750	0	1949
7	750	90	2005
8	1000	30	1944
9	1000	60	1950
10	1000	90	1940

Table 8.4: Sample variables for the first set of Galway tests (after Sutton et al., 2013), with sample number 6 being the control sample that did not have an external water supply.

Two main lithologies were present in the material – a laminated, calcareous mudstone and a calcareous siltstone. The material was taken from below the floor slab of an Irish domestic property some 5 years after being placed. The amount of pyrite in the material at the start of the tests was calculated to be between 2.1 and 3.6%, with the original pyrite levels being between 3.4 and 4.7% (Sutton et al., 2013). The samples were stored in an area with no temperature control, and so any temperature controls in the material

## 8 Laboratory Testing – Development and Modification of the Swell Tests

reflected that of the surrounding environment. Figure 8.1 shows the expansion of some of these tests over a 175 day period.

The different water depths was determined to have minimal effect upon the reaction rate, to such an extent that the control sample that had no external water supply showed the fastest rate of expansion (Sutton et al., 2013).

The temperature of the water is also shown in Figure 8.1, and it was noted that when there was a sizeable drop in temperature, there was an associated reduction in expansion rate. This was thought to be due to a reduction in bacterial activity as the temperature moved below their active range, and less water evaporation leading to less precipitation of gypsum (Sutton et al., 2013). The effect of temperature was followed up on in the second set of Galway tests, as discussed in Section 8.2.3.

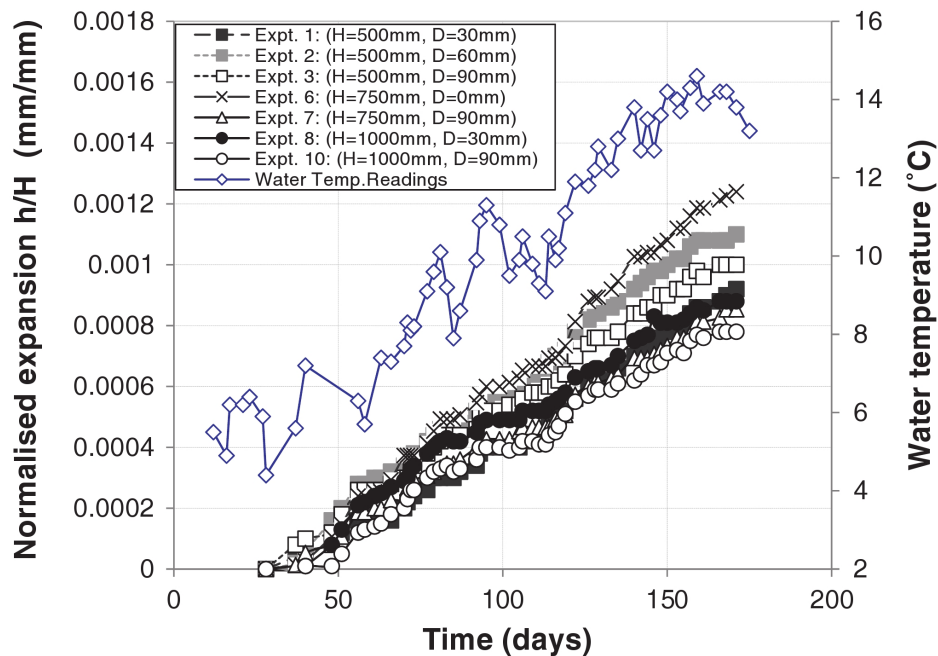


Figure 8.1: Normalised expansion rate of samples in the first round of Galway tests (Sutton et al., 2013).

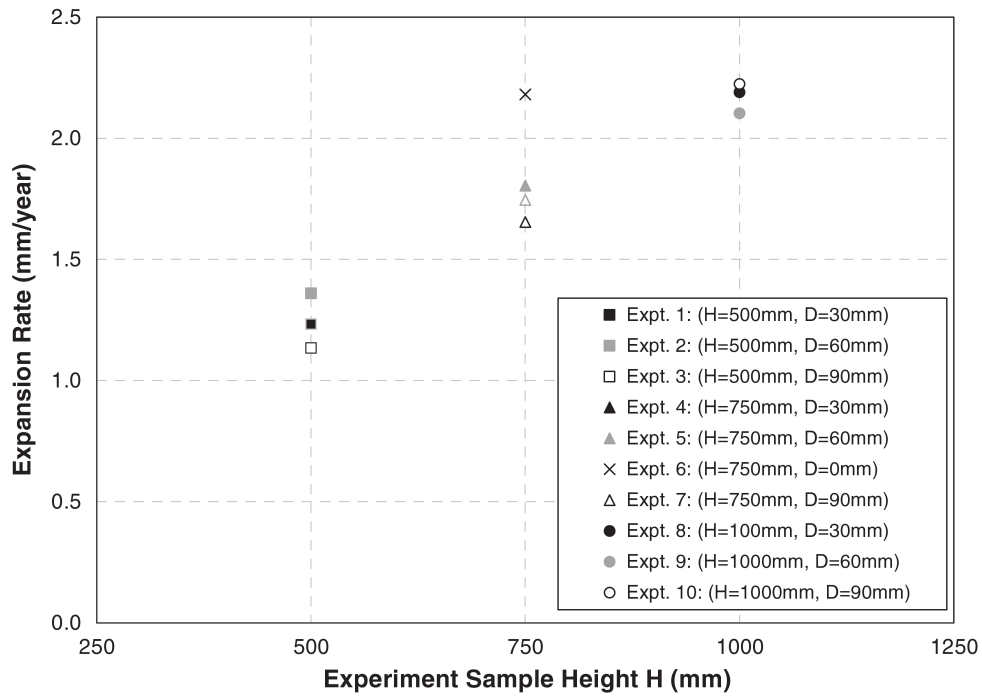


Figure 8.2: The effect of sample height on expansion rate (Sutton et al., 2013).

For the samples with an external water supply, the expansion rate was roughly proportional to the fill height, with an average expansion rate of 2 – 3 mm/year/mm of fill. When considering the Irish properties with between 500 and 750 mm fill thickness and an age of around 5 years, this would lead a maximum expansion of between 6 – 9 mm, which corresponds to the ranges seen in the floor level surveys (Sutton et al., 2013).

### 8.2.3 Galway Tests - Series 2

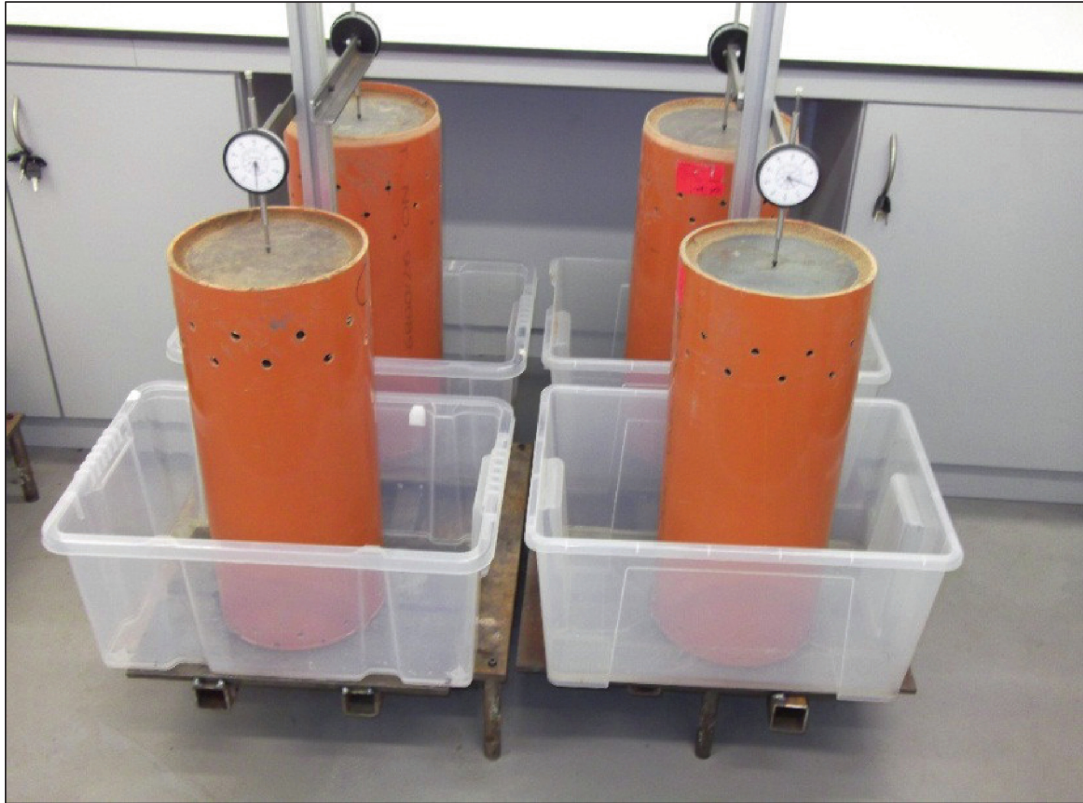


Figure 8.3: Design of the swell from the second series of tests at the University of Galway (McCabe et al., 2015).

Following on from the Sutton et al. (2013) tests, a second series of tests was planned at Galway, this time considering the effect of fill density and temperature upon the rate of expansion. The design of the tubes was the same as in the previous tests, as shown in Figure 8.3, although the height of all samples was kept at 500 mm and water levels maintained at either 10 or 30 mm. The full set of test variables are summarised in Table 8.5.



Test no.	Sample height (mm)	Water depth (mm)	Density (kg/m <sup>3</sup> )
B1	500	10	2169
B2	500	30	2230
B3	500	10	2000
B4	500	30	1952
B5	500	10	1799
B6	500	30	1817
E1	500	30	2035
E2	500	30	2016

Table 8.5: Sample variables for the second set of Galway tests (after McCabe et al., 2015).

The material used in these tests was taken from an Irish property undergoing remediation that had a pyrite level of 1.36% at the start of the tests. The original pyrite level when first placed in situ, some 6 years before the testing, was calculated from the total sulphur to be around 2.4% meaning that approximately 42% of the original pyrite had been oxidised when the tests started (McCabe et al., 2015). There were 3 lithologies present in the material: a laminated, calcareous mudstone, an argillaceous limestone and a pure limestone. Although McCabe et al. (2015) do not provide detailed specifics of the proportions of these materials, it was reported to contain around 78% mudstone by volume.

All tests were stored in a temperature controlled environment, with tests E1 & 2 acting as a control in a separate room to tests B1 – 6. The temperature of the rooms was varied between 10 – 20°C at different time periods with no intended pattern. The material used in tests E1 & 2 was pyrite-free Clause 804 material, and was used to determine the effect of temperature upon the test system (McCabe et al., 2015). Figure 8.4 shows the expansion of tests B1-6 as well as the temperature variations over the testing period. Expansion in the graph is normalised to show displacement of the upper surface of the test ( $h$ ) over the sample height ( $H$ ).

## 8 Laboratory Testing – Development and Modification of the Swell Tests

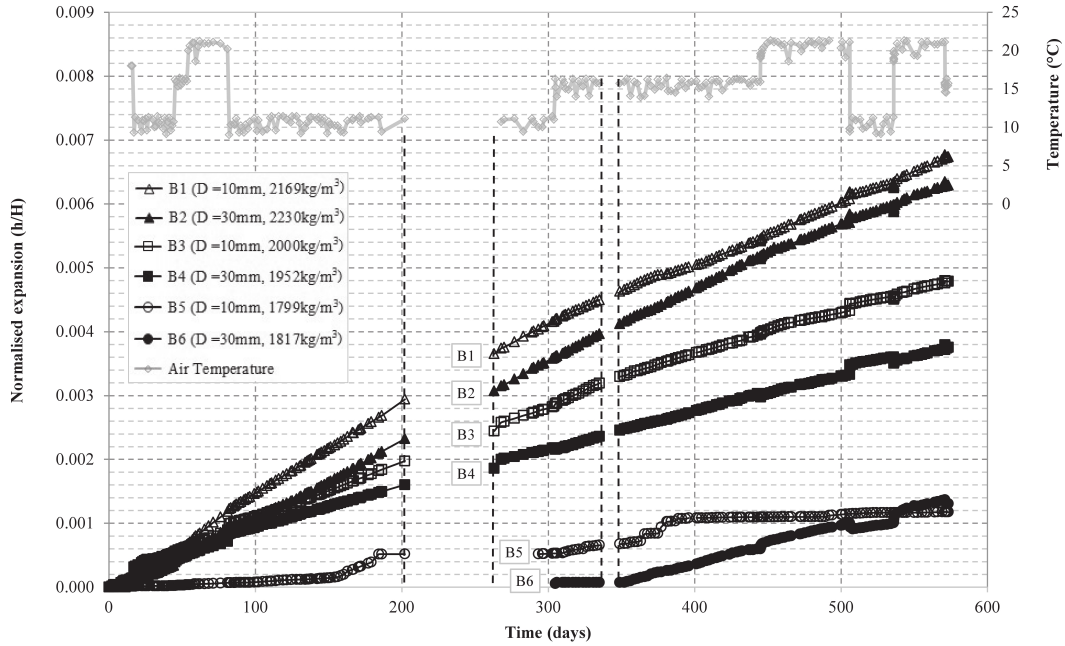


Figure 8.4: Expansion rates of the second series of Galway tests, with variations in temperature and material density (McCabe et al., 2015).

Although some variation can be seen in the expansion rates with the change in temperature, it is the effect of density that is the obvious factor when considering the expansion rates, with the higher density tests showing more expansion at a faster rate. This was expected, since a higher density material will have less void space into which the gypsum can precipitate before precipitating into locations that lead to expansion of the rock.

As shown in Figure 8.4, tests B1-4 generally show a constant rate of expansion, whereas the lower density tests behave differently. Test B6 shows a fairly constant expansion rate, although it did not register any expansion until the test had been running for 300 days. Similarly test B5 shows long periods with no expansion followed by shorter periods where the expansion rate seems to be greater than that of tests B1-4. Figure 8.5 shows the expansion rates compared to the density for the periods the test spent at 20°C (McCabe et al., 2015).

Similar to the tests carried out by Sutton et al. (Sutton et al., 2013), it was concluded that the sensitivity to the water level increased as there was less water available. Those samples with a water depth of 10 mm showed a greater expansion than those with a depth of 30 mm (McCabe et al., 2015). Again, this is expected, since the saturation of the material in this zone would greatly inhibit the oxidation reaction; McCabe et al. (2015) suggest that this reduction in reaction rate could be as much as 96%.

## 8 Laboratory Testing – Development and Modification of the Swell Tests

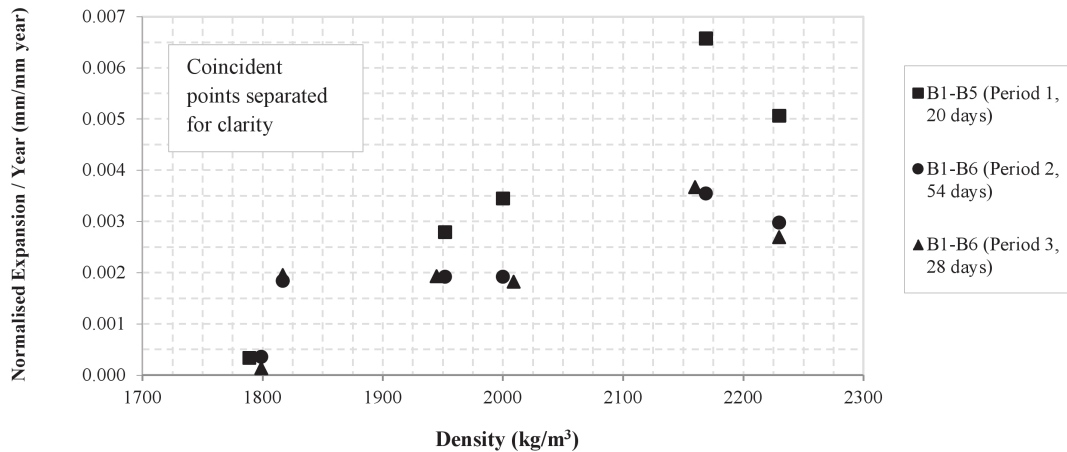


Figure 8.5: Effect of density on the average expansion rate for tests B1-6 at 20°C (McCabe et al., 2015).

In contrast to the clear evidence of the link between the amount of expansion and the density, the link between the temperature changes is less clear. At first it appears that there is a change in the expansion rate coinciding with each temperature change, however, work carried out by McCabe et al. (2015) shows that this apparent change in rate is due to thermal expansion/contraction of first the steel frame and then of the mudrock itself.

It is suggested by McCabe et al. (2015) that the effect of temperature on both the amount of expansion and the expansion rate is minimal. This is in contrast to the evidence of the examples given by Llandough Hospital (Section 3.1.1) and Rideau Health Centre (Section 3.2.2), and by comparing the rate at which damage is shown in the Irish cases compared to those in Canada.

The main explanation given for the way in which the Irish cases present signs of damage within 3 – 5 years and Canadian cases take between 10 – 15 years is that the average annual temperature in Ireland is higher than that of Canada (CTQ-M200, 2001), with the colder Canadian temperatures helping to inhibit the reaction process.

However, given the evidence presented by McCabe et al. (2015), it is likely that the influence of factors such as different construction methods, the fact that in many of the Canadian cases it is pyritic bedrock material as opposed to the fill material used in Ireland, and differences in the physical and chemical properties of the material have as much if not more influence on the reaction rate than the temperature.

As noted earlier, the maximum temperature reached during these tests was 20°C, whereas the *Thiobacillus* bacteria known to catalyse the reaction achieve peak efficiency between 25 – 40°C (Holt, 1977). It is unknown at this time whether or not tests are required

to be carried out at a higher temperature, although it would seem logical to amend the tests to further cover the temperature variable and determine whether this higher temperature has an effect in a controlled environment.

Results from the Arduino testing have shown that temperatures in the fill mirror the overall trend of the internal temperature of the room above the slab, with fill temperatures being a few degrees cooler than the internal temperature but still comparatively warmer than the external ground temperatures (Chapter 6).

Considering this, even with the effect of central heating, it is unlikely that the fill temperatures will exceed 25, however, cases such as Llandough Hospital and Rideau Health Centre have shown that structural designs incorporating service ducts into the foundations can cause larger temperature variations within the fill material.

### **8.2.4 Galway Tests - Series 3**

The most recent tests at Galway involve construction of a scale model of a foundation rather than using the tube set up of previous tests. This model comprises a masonry box approximately 1.5 m along each side, with a base of compacted boulder clay, and a layer of pyritic fill material topped by a DPC and a cast concrete slab (Mannion and McKeon, 2012).

The box is intended to give a sample approximately 1.12 by 1.12 m wide and 770 mm thick and have be constrained at both top and bottom as is the case in a domestic property. The test will be monitored with dial gauges on top of the slab to monitor the uplift, and with pressure gauges inside the box in order to measure pressure changes as the material expands.

At the time of writing, this test was ongoing and no results are available.

## **8.3 Sheffield Swell Test Design**

The design of the swell tests carried out at Sheffield was based upon that used by Sutton et al. (2013) and McCabe et al. (2015), as shown in Figure 8.6.

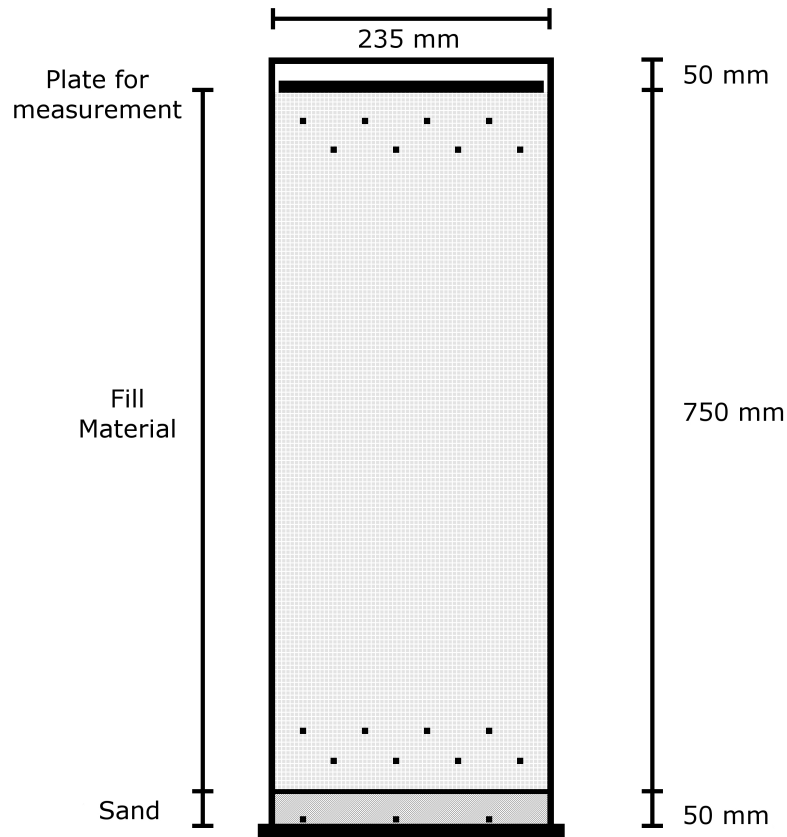


Figure 8.6: Design of the swell test tubes used at Sheffield.

The tubes are made, as with the Galway tests, from polyethylene, with a 235 mm internal diameter and a wall thickness of 12 mm, with a fixed 300 mm square and 20mm thick polyethylene base. 10 mm diameter ventilation holes are located immediately above the base, to allow water ingress, and beginning at 100 mm from both base and top to allow air to move through the material.

With the proof from the Galway tests that the amount of expansion is roughly proportional to fill thickness (Sutton et al., 2013), the fill thickness in these tests was maintained at or around 750 mm in order to consider other variables. The water depth was maintained at around 100 mm, and the test was housed in an climate controlled laboratory that was intended to maintain the air temperature at or around 20°C. The material used in this test sequence is described in Chapter 7.

Following on from the tests carried out by Maher et al. (2011), who stated that the grading of the material has an effect on the rate and amount of expansion, it was decided

to focus on varying this parameter. Table 8.6 shows the variables for each of the tests along with their density.

Test no.	Description	Mixed/ layered	Grading	Bulk density (kg/m <sup>3</sup> )
1	Control	well mixed	well graded	1950.7
2	Increased specific surface	Well mixed	High fine fraction	1907.9
3	Increased specific surface	Well mixed	High fine fraction	1876.6
4	Decreased specific surface	Well mixed	High coarse fraction	1881.3
5	Decreased specific surface	Well mixed	High coarse fraction	1917.2
6	Fining upward	Layered	Well graded	1849.7
7	Fining upward	Layered	Well graded	2018.2
8	Fining downward	Layered	Well graded	1844.7
9	Fining downward	Layered	Well graded	1900.6
10	High compaction	Well mixed	Well graded	2057.2
11	High compaction	Well mixed	Well graded	1908.2
12	Arduino test	Well mixed	Well graded	2009.6

Table 8.6: Variables for the swell tests carried out as part of this study.

It is mentioned in Chapters 4 and 7 that the material used in the Irish cases was intended to be subject to Clause 804 regulations, the PSD curves for which are shown in Section 7.2. This gives an upper range for the particle size of the material, with a requirement that 100% of the material pass through a 63 mm sieve.

As mentioned in Chapter 7, there was around 5% of the material that was initially received that exceeded this limit of 63 mm. Additionally, in order to avoid boundary problems, a maximum particle size of 37 mm (from the inner diameter of the tubes divided by 6) would be needed, and so all material was passed through a 35 mm sieve before starting compaction. It is not mentioned within the literature whether or not a similar process was carried out in the Galway swell tests, although it was discussed by Maher et al. (2011).

This leads to inevitable questions about the representation of the entire fill material in the swell tests. Up to 5% of material was lost in the initial sieving at 63 mm, with approximately another 35% lost when reducing the maximum particle size to 35 mm.

When losing up to 40% of the fill material that was supplied, it can be argued that the representation of the entire fill material, by these tests, is somewhat limited.

Despite this, there does not seem to be an immediate solution to the problem when using the tube swell test apparatus. In terms of cost-effectiveness, ease of use and flexibility to measure different parameters, the tube swell test equipment is known to work and produce repeatable results. However, this does mean that there will always be a fixed limit on the maximum particle size in order to keep the scale of the tests.

It will, however, be possible to compare the data from these tests to that of the foundation model once it is published (Section 8.2.4), which has a much larger maximum particle size since the boundary problems are not as restrictive.

The material was compacted at first using a 4.5 kg compaction hammer (used for Test 1) and then with a Kango vibrating hammer with a modified plate to fit the inner diameter of the tubes. This was used to make the compaction process more efficient and consistent than using the compaction hammer. Although there was some breakage of material using the Kango hammer, there was not significantly more than when using the compaction hammer.

Compaction of Test 1 was done according to the procedure in the Manual of Soil Laboratory Testing (Head, 1992) with layers of 200 mm uncompacted thickness, and compaction of all other tests using the Kango hammer was completed again on layers of around 200 mm uncompacted thickness for a period of 90 seconds per layer.

This compaction method did mean that it was difficult to achieve bulk compaction densities above  $2000 \text{ kg/m}^3$ . In order to achieve densities of this value, the compaction period with the Kango hammer would have needed to be much longer, at which point the chance of breaking down the material and changing the percentages for tests such as those with increased/decreased specific surface also increased.

For tests 2 – 9 the grading of the material was determined around the  $D_{50}$  point, with material above this size being considered coarse and that below being considered fine. Figure 8.7 shows the PSD curve for the material, with the  $D_{50}$  point marked. This point corresponds to a particle size of 6.35 mm, and so material was divided into fine/coarse by passing it through a 6.3 mm sieve.

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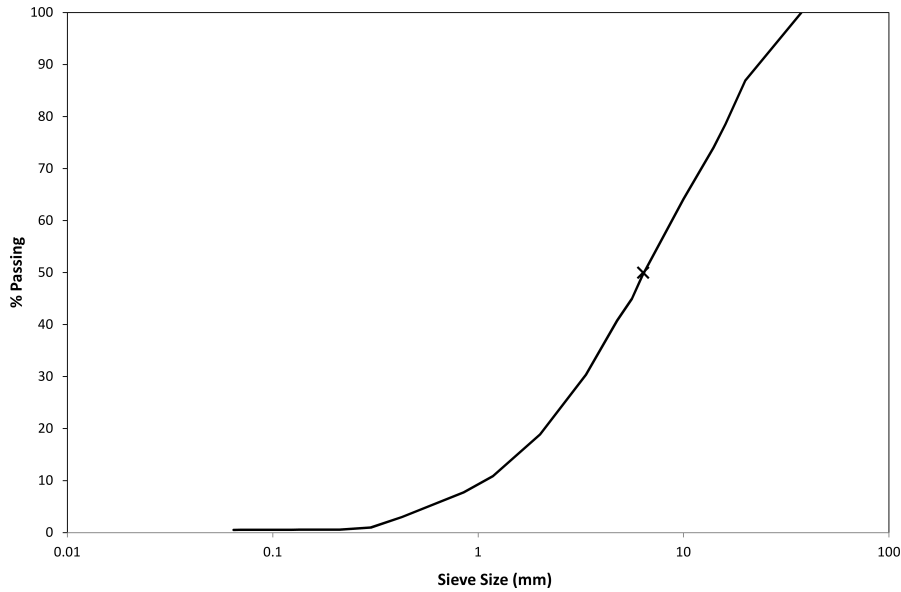


Figure 8.7: PSD chart for the material used in the swell tests with the  $D_{50}$  value marked.

The material for tests 2 – 9 was subdivided and compacted as follows:

- **Tests 2 & 3 – increased specific surface:** used 75% material less than 6.3 mm and 25% material greater than 6.3 mm. Material was measured according to weight for each layer and then mixed by hand before being placed into the tubes and compacted.
- **Tests 4 & 5 – decreased specific surface:** used 25% material less than 6.3 mm and 75% material greater than 6.3 mm. Material was measured according to the same method used for Tests 2 & 3.
- **Tests 6 & 7 – fining upward:** material divided at 6.3 mm and the coarser fraction further divided at 16 mm. Material compacted in layers so that the bottom third of the tube contained coarse material greater than 16 mm, the middle third contained material between 6.3 and 16 mm and the upper third contained material smaller than 6.3 mm. Each of these layers was approximately 250 mm thick after compaction.
- **Tests 8 & 9 – fining downward:** material divided at 6.3 mm and the coarser fraction further divided at 16 mm. Material compacted in layers so that the bottom third of the tube contained coarse material smaller than 6.3 mm, the middle third contained material between 6.3 and 16 mm and the upper third contained material greater than 16 mm. Each of these layers was approximately 250 mm thick after compaction.

The material in tests 2 – 5 was mixed such that the coarse and fine fractions were well



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mixed before compacting. Tests 6 – 9 were placed in layers that contained only the coarse, medium or fine fraction.

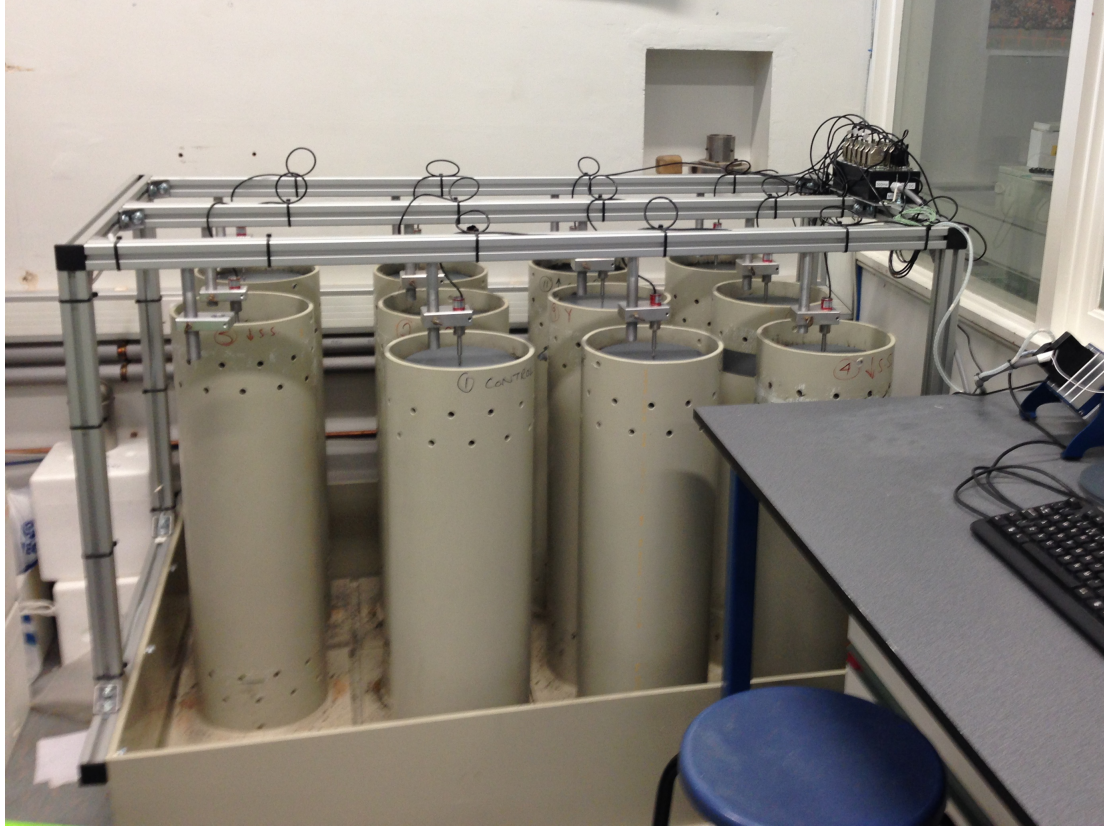


Figure 8.8: Set up of the swell test tubes in the laboratory before the addition of water. Note that the tube used for the initial Arduino system testing (Chapter 6) is missing in this photograph.

As shown in Figure 8.8, the tubes were housed within a polyethylene tank so that the water supply was continuous across all tests. This would ensure that all tests had the same depth of water, even in cases such as over holiday periods when the water would not be topped up as frequently. The exception to this was the control, test 1, which did not have the ventilation holes immediately above the base and so had restricted access to water. The tank itself was seated upon a 50 mm thick layer of polystyrene insulation so that the heat from the underfloor heating did not affect the water temperature.

The expansion of the material was monitored via a series of LVDT's (linear variable differential transformers) that were recorded electronically. These LVDT's measured the movement of a polyethylene plate that sat on top of the material in order that any uneven expansion would still be recorded. Figure 8.9 shows the RDP LVDT (DCTH 200AG) sensor that was used for all of the swell tests.

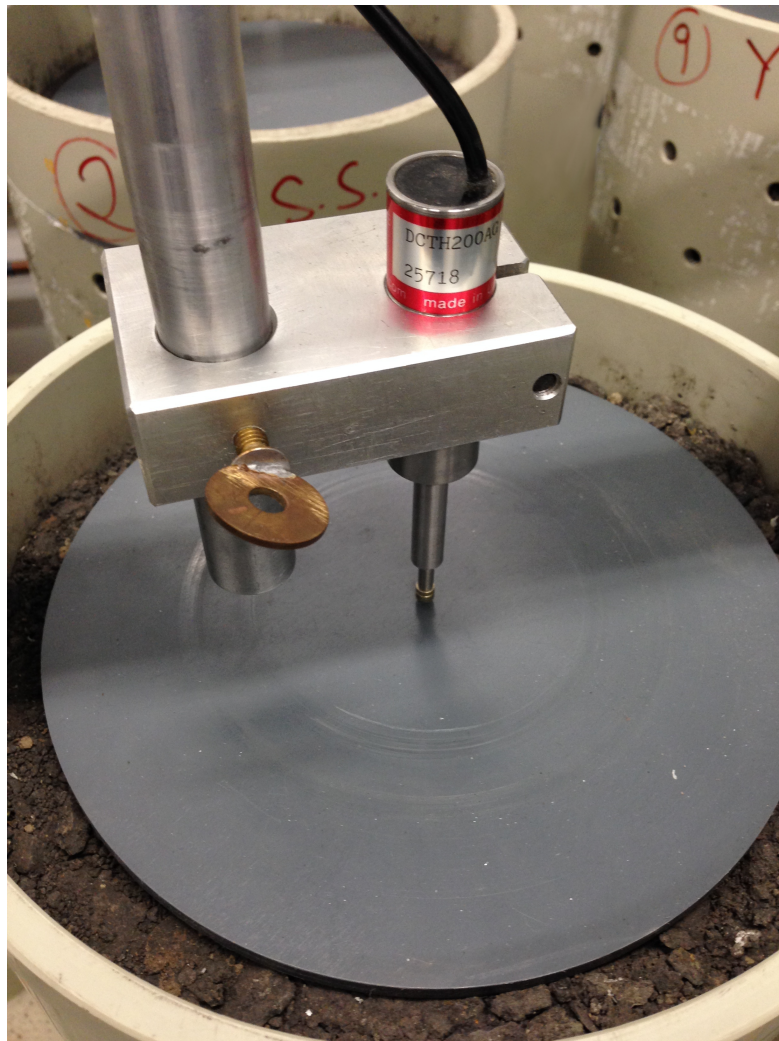


Figure 8.9: LVDT in use monitoring the swell tests.

In addition to the LVDT's monitoring the expansion of the fill, there were two temperature sensors fastened to the framework that supported the LVDT's. One of these sensors was placed on the upper section of the framework, 1 metre above ground level, while the other was placed at the opposite end of the framework, at the same level as the top of the water tank. This was to determine if there was any significant difference in the temperature across the system due to either the air conditioning units, the underfloor heating, or a combination of the two.

The LVDT's were connected to a computer and all data recorded using LabView. This allowed the user to view the data at a given time to ensure that the LVDT's were functioning correctly, and recorded all datapoints to file to be analysed at later. Samples were initially taken at 1 hour intervals, with an increase to 15 minute intervals as discussed

in Section 8.5.1.

The 1 hour interval was selected so as not to create an excessive number of data points when it was known that the reaction process was going to proceed relatively quickly. Even if an expansion rate of 20 mm/year was to be seen (one of the higher rates cited in the literature, see Table 8.10), this would still be an average expansion of 0.05 mm per day that would be seen over 24 data points.

The LVDT's were calibrated using a Mututoyo LVDT calibration rig, shown in Figure 8.10, in which each LVDT was monitored with the LabView software whilst being moved by measured amounts. Any difference between the amount moved on the calibration device and that recorded by the software allows for a calibration factor to be calculated for each LVDT. This is then used by the LabView software to correct the values obtained from the LVDT's before writing the data to file.

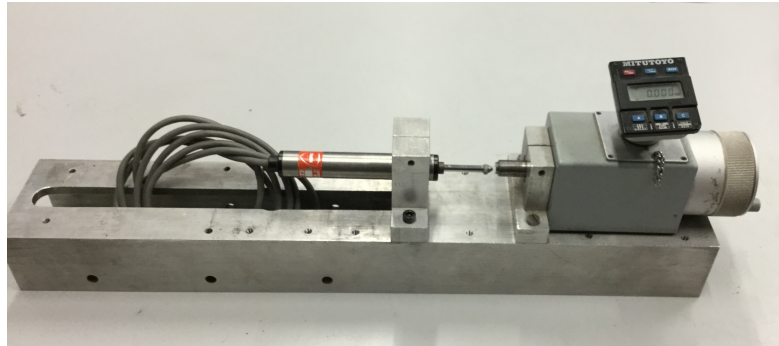


Figure 8.10: Mututoyo calibration device in use in the laboratory.

In order to cover the possibility of computer malfunctions, a manual dial gauge with a precision of 0.02 mm was also fixed to the frame in order to monitor the uplift of the control sample. Although data from the dial gauge was not monitored as often as the LVDT's, it provided a useful backup system in case of data loss, allowing for continuation of the dataset.

## 8.4 Initial Results

Figure 8.11 shows the swell tests results from April 2014 to August 2015.

The gaps in data recording are due to computer problems during which the computer used for recording the data required repair or replacement.

It is also noted across most of the datasets, that there was an initial period of very rapid movement after which the measurement settled into a more steady state of expansion.

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Some of the tests showed a small amount of settlement once the water was added, however, this generally only lasted a few days before signs of swell began to appear. This period of settlement was most pronounced in tests 4, 6 and 10. Since there was no apparent link between the method of compaction/material selection, this was attributed to hydrocompaction of the mudstone caused by the introduction of the water.

Recording of the samples began when water was added to the system, however the tubes were in place in the laboratory for 10 days prior to this. This standing period was to allow all the tubes to equalise to the same temperature as the laboratory, and to allow placement of the frame and LVDT's over the tubes. It is likely that during this period some drying out of the samples occurred, this will be discussed in Section 8.5.1 and is not covered here.

Figure 8.11 shows selected data from these initial tests, along with the point at which the test was modified by removing the water in June 2015 (Section 8.5.1). Full results of each of the pairs of tests are discussed later.

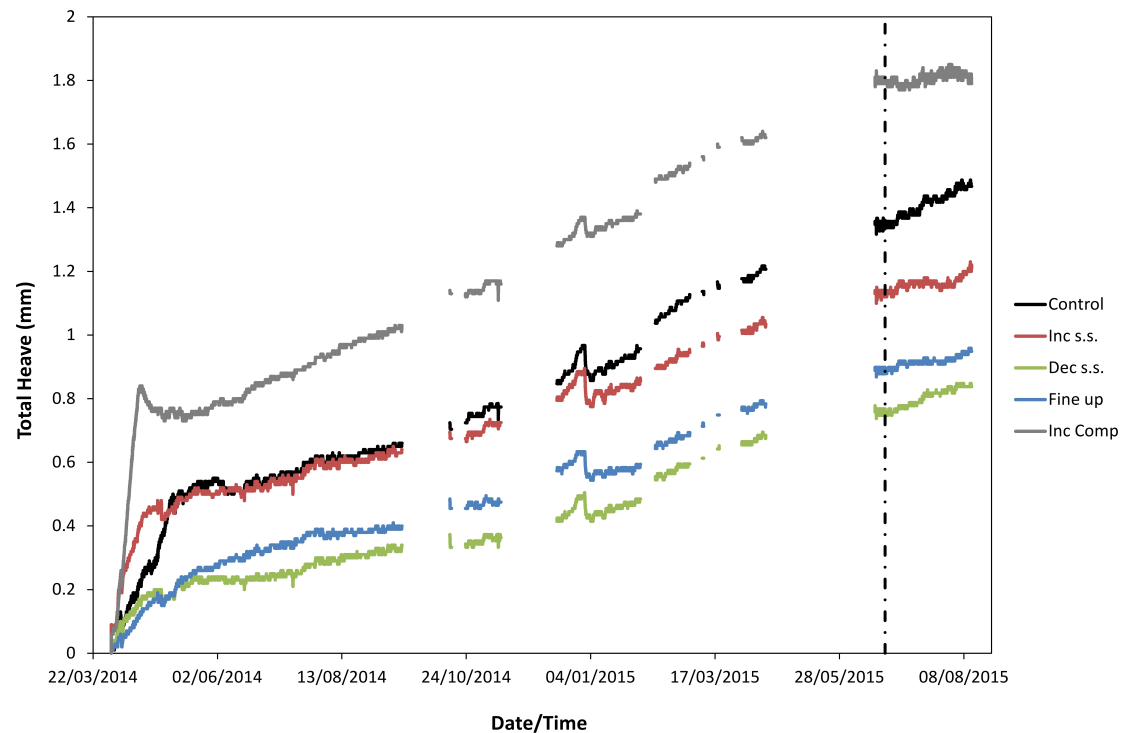


Figure 8.11: Expansion of selected swell tests from April 2014 to August 2015. The dashed black line indicates the point at which the water was removed from the tests.

The placement of the manual dial gauge allowed for correction of the data after the period during which data was lost, although this reading was only accurate for the rate

of expansion for the control test. The start point for the data after each break in the data was then calculated using the average rate of expansion for that tube, with the data from the dial gauge providing, in most cases, an upper limit for what expansion occurred while there was no recorded data.

The anomaly in the data around the 29th of December 2014 is thought to be caused by the air conditioning in the lab being turned down over the holiday period. This would explain the gradual increase in temperature from around the 21st December to a peak on the 2nd January, with a decrease after this point as people returned to work. The temperature increases from an average of 19.5°C at noon on the 21 December to an average of 21°C on the 2nd January before decreasing again at the start of the working week from the 5 January.

It was not expected, however, that a relatively small temperature change should have such an effect upon the expansion of the material. Although this expansion is only an average of 0.06 mm, it is enough to read on the LVDT data. Based upon the McCabe et al. (2015) data, this was initially thought to be due to thermal expansion of the aluminium frame holding the LVDT's. However, with a linear expansion coefficient of  $23 \times 10^{-6} \text{ K}^{-1}$ , the expansion of the aluminium frame due to a temperature increase of 2.5°C would be small enough that it is unlikely to be shown on the sensors.

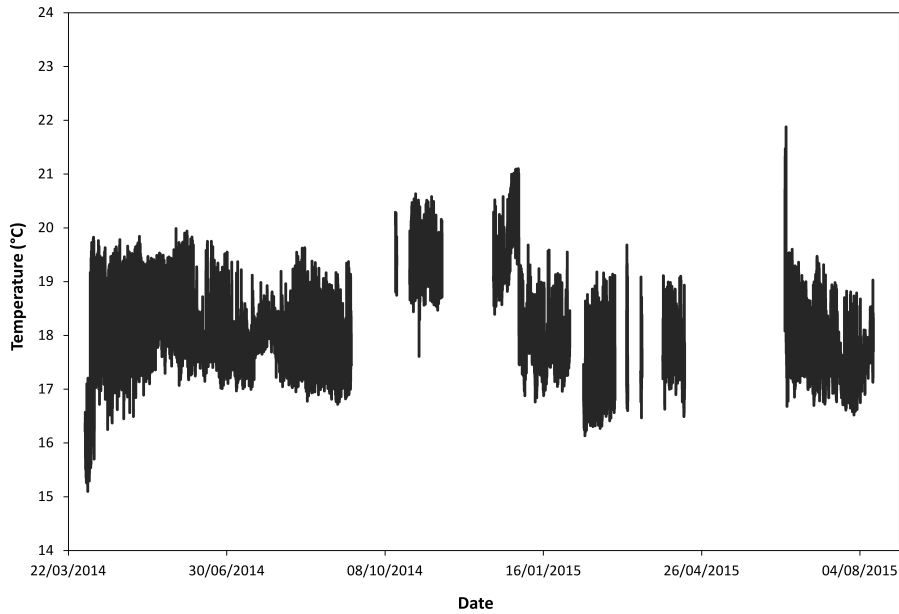
McCabe et al. (2015) showed a movement on the order of 0.00015 mm with a 10°C temperature change, which is consistent with that calculated from the expansion coefficient above for the change over a 1 metre length of aluminium. With the LVDT's in use at Sheffield measuring movement of the fill on a level of 0.01 mm, it is unlikely that any thermal movement of the frame would impact upon the data returned by the LVDT's. This is supported by the sampling period of 1 hour and the fact that any temperature changes progressed evenly over the space of a few hours, rather than the rapid temperature changes employed by McCabe et al. (2015).

This then leads to the assumption that it may be thermal expansion of the mudrock that caused the peak in the data. However, although most rock materials generally have a linear expansion coefficient of between  $15 - 33 \times 10^{-6} \text{ K}^{-1}$ , even at the upper end of this range, the expansion of the material due to a temperature increase of 2.5°C is not enough to create the 0.06 mm increase in the recorded rate.

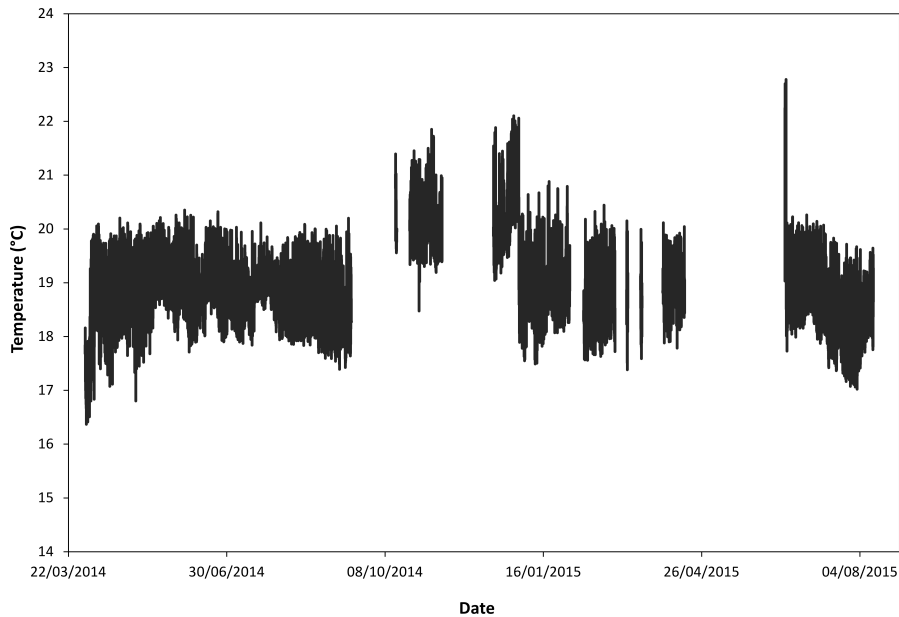
Although the increase took place over the holiday period, there were still people using the laboratory over this period. It is possible that the frame was knocked by one of these people, leading to the movement seen on all of the dial gauges, and that the apparent correlation with the temperature increase is coincidental.

The changes in laboratory temperature alluded to above are shown in Figure 8.12, which shows the variation in temperature over the testing period.

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(a) Temperature in the upper section of the test frame.



(b) Temperature in the lower section of the test frame.

Figure 8.12: Changes in laboratory temperature over the test period.

The large amount of variation seen in the laboratory temperatures is largely dependent upon the time of day and week, with higher temperatures being recorded during working hours in the week. Of interest is the fact that despite the laboratory being temperature controlled to 20°C, the air temperature only rarely reaches this temperature. Although

the data for that time period is incomplete, the average temperature in the laboratory was higher in the winter months when the underfloor heating has an effect. Despite occasional variations, the average temperature however, is reasonably consistent, meaning that apart from anomalies such as mentioned above, there should be little effect upon the tests and the data retrieved.

The values recorded at the top and bottom of the frame show changes in temperature at the same time, although that is to be expected, since the sampling rate at this time was once per hour. Temperatures are also similar between the two points, with the upper sensor recording temperatures, on average, half a degree cooler than the lower sensor.

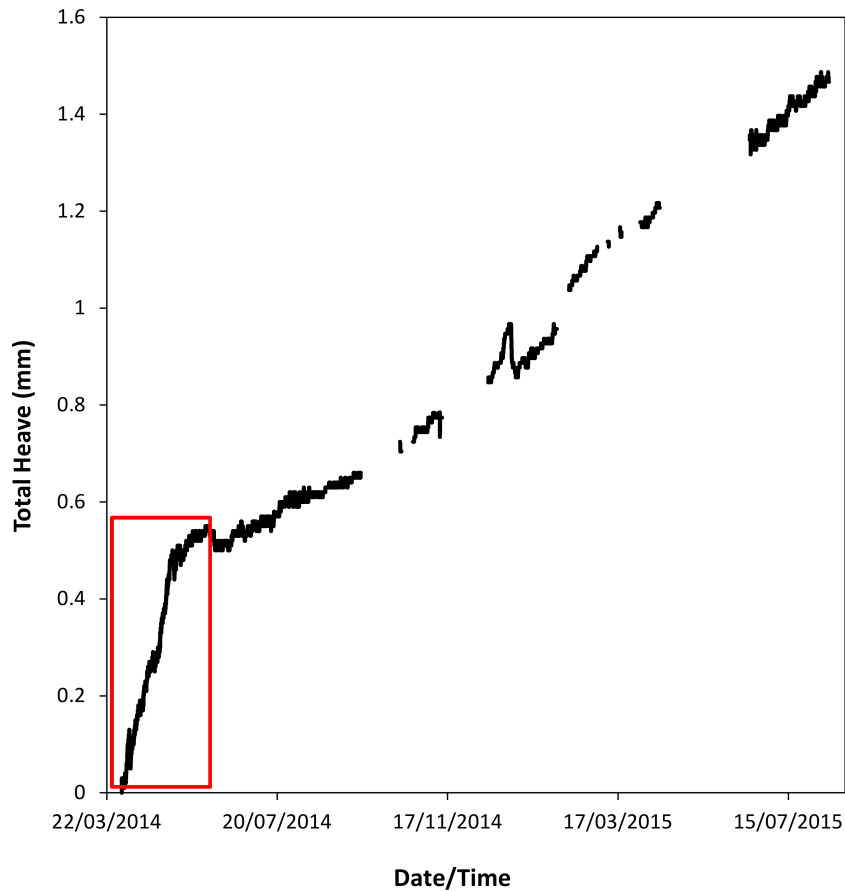


Figure 8.13: Expansion of the control sample over the testing period.

Figure 8.13 shows the expansion of the control sample over the testing period, with the initial rapid period of expansion that was seen in several of the tests highlighted in red.

An initial spike in reaction was expected, as the Galway tests (Sutton et al., 2013, McCabe et al., 2015) showed a period of increased activity at this time before the tests settled into a more steady rate of heave. However, in the tests carried out at Sheffield,

this initial period of expansion lasted much longer than the period of time seen in the Galway tests. In the Sheffield tests this period ranged up to 8 weeks compared to 3 weeks at Galway (Sutton et al., 2013).

Considering that some of the tests did not show this increased period of activity, it was determined that it was not caused by the system settling into a steady rate of heave, and was likely to be related to either a physical or chemical property of the tests.

This increased period of activity was seen to one extent or another in the control (Figure 8.13), increased compaction (Figure 8.14) and increased specific surface tests (Figure 8.15a), and to a lesser extent in the decreased specific surface tests (Figure 8.15b).

Initially it was thought that the rapid period of expansion in some tests was caused by expansion of the mudrock particles upon wetting, and was therefore linked to the compaction density, with denser samples being quicker to show the expansion as there was less space within the sample into which the mudrock could expand. However, when comparing the densities of the tests that showed the initial expansion and those that did not, there was no noticeable correlation, as indicated in Table 8.7.

Test	Variable	Rapid Expansion?	Density (kg/m <sup>3</sup> )
1	Control	Yes	1950.7
2	Inc s.s.	Yes	1907.9
3	Inc s.s.	Yes	1876.6
10	Inc comp	Yes	2057.2
11	Inc comp	Yes	1908.2
4	Dec s.s.	Yes*	1881.3
5	Dec s.s.	Yes*	1917.2
6	Fine up	No	1849.7
7	Fine up	No	2018.2
8	Fine down	No	1844.7
9	Fine down	No	1900.6

Table 8.7: Comparison of those tests that shows an initial rapid expansion and their bulk compaction densities. Those marked Yes\* showed some signs of a rapid initial expansion but not as significant as other tests. The density as listed is the bulk density at the time of compaction.

Another possible link is that all the tests showing the initial period of rapid expansion contained an even distribution of both fine and coarse material throughout the tube, even though in the case of the decreased specific surface tests (Tests 4 & 5) this was a reduced amount of fine material.

Referring to the tests by Maher et al. (2011), summarised in Tables 8.1 and 8.2, the ‘all in’ samples show a much higher expansion rate than those limited to either only the coarse or fine fraction.



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As stated above, the likelihood is that the fine fraction reacts earlier in the process than the coarse fraction due to its higher specific surface. Maher et al. (2011) show this in the higher initial expansion rate of the fine fraction that is later overtaken by the rate of the coarse fraction as the pyrite in the fines is used up and that in the coarse fraction begins to react (Table 8.2).

Similarly, it could be reasoned that the initial spike seen in the mixed, unlayered tests carried out at Sheffield were caused by the initial start of the reaction process in the very fine material contained in those tests. This reaction would be in addition to the reaction of the fine and coarser material and would provide a boost to the initial reaction rate. Once the pyrite within the very fine fraction had all been used up, the reaction rate would slow slightly as the material with a lower specific surface begins to dominate the reaction process.

This would account for the less rapid increase in the decreased specific surface material, as although the very fine fraction was present in this material, there was less of it than was present in the control, increased compaction and increased specific surface tests.

For the tests in which the fill was placed in layers, although there may have been small amounts of very fine material stuck to the coarse fragments, the fine fractions were generally only located in one third of the tube. In the case of the tests that were layered with the fine material at the the top of the tube, this material was initially limited in its water supply which would lower the reaction rate. For the tests where the fine material was located at the base of the tube, some of this would have been below the water level and some of it within the likely zone of capillary rise, meaning that some of the material was in a saturated or almost saturated state, which would also inhibit the reaction process.

At this time, however, this is conjecture, and more detailed, small scale tests would be required in order to study the reaction rate in highly controlled samples consisting only of material that was very fine, fine, coarse and so on. This is considered further in Section 9.3.

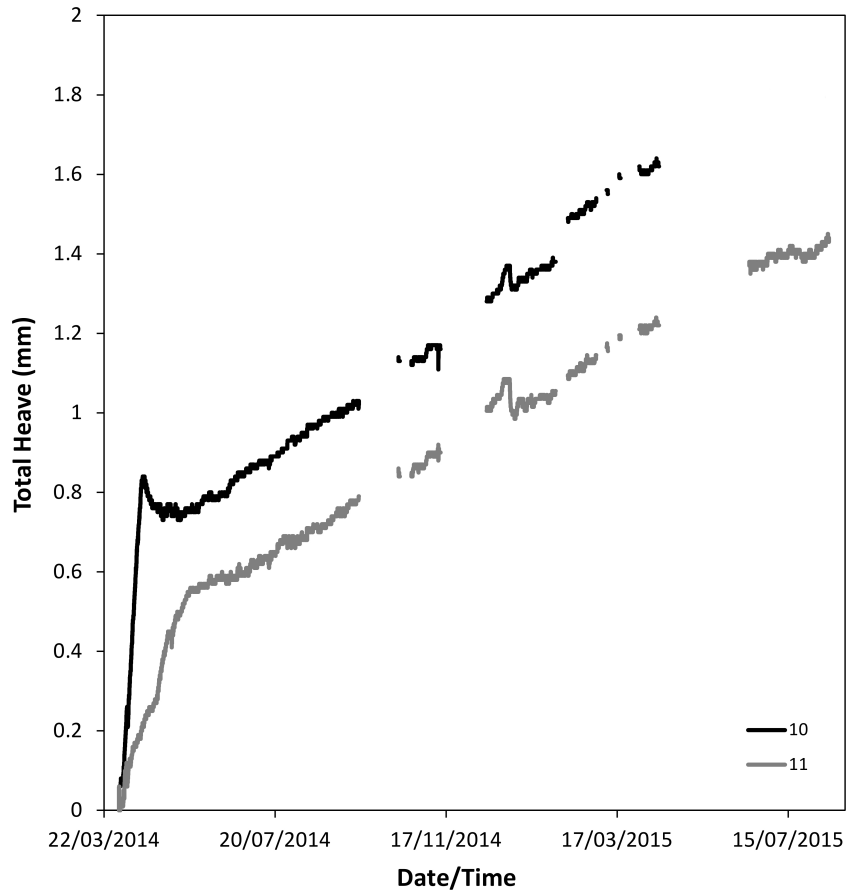


Figure 8.14: Expansion of the samples with increased compaction over the test period. Note that Test 10 was removed in April 2015 as discussed in Section 8.5.2.

Figure 8.14 shows the expansion of the tubes intended to be compacted at a higher bulk density than the other tubes. Results similar to those shown by McCabe et al. (2015) wherein the tests they compacted at a higher density showed a greater expansion rate and subsequently showed a higher level of total heave were expected.

As reference in Table 8.7, Test 10 had a bulk density of  $2057.2 \text{ kg/m}^3$  and Test 11 had a bulk density of  $1908.2 \text{ kg/m}^3$ . Although it was intended to have both of these tests above  $2100 \text{ kg/m}^3$ , this was not deemed possible during the compaction work without breaking down the particles of the mudstone.

Nevertheless, it can be seen that the test with the higher density of the two, Test 10, showed a more rapid rate of expansion in the initial phase and overall a greater rate of expansion per year. This is summarised in Table 8.8 and will be discussed further below.

These two tests can also be compared to the control, which had a bulk compaction density of  $1950.7 \text{ kg/m}^3$ , sitting between the densities achieved in the two increased

compaction tests. However, the amount of expansion seen in the Control test is slightly less than that of Test 11, with the expansion rate of the Control only progressing faster than that of Test 11 after more than 12 months into the test period.

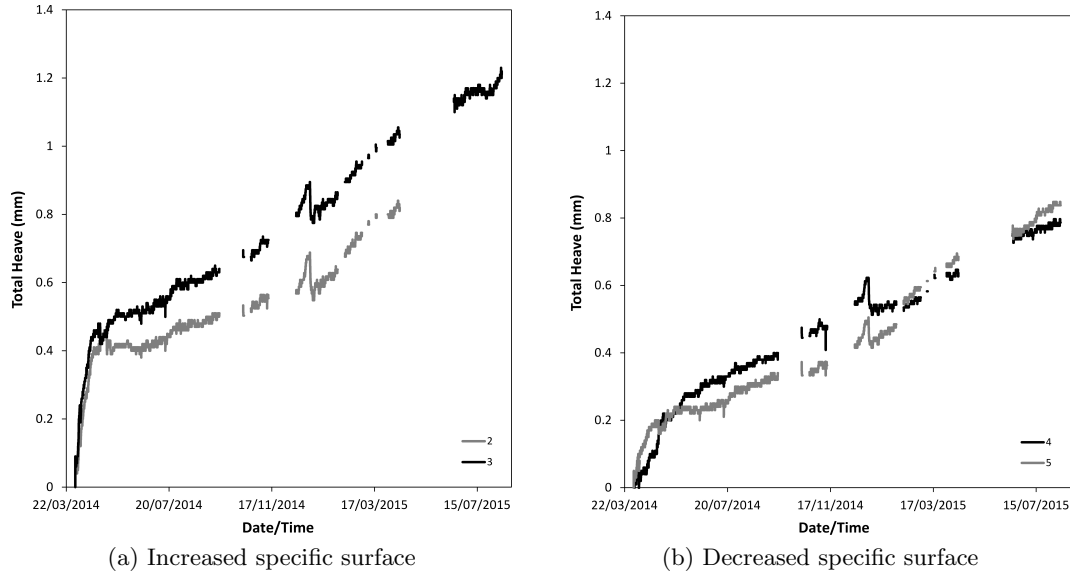


Figure 8.15: Expansion of the samples with both increased and decreased specific surface over the test period. Note that Test 2 was removed in April 2015 as discussed in Section 8.5.2.

Figure 8.15 shows the difference between the increased and decreased specific surface tests. At first glance the expansion rates of all four tests seem similar, however, the test containing the material with an increased specific surface shows considerably more expansion with Test 3 reaching a peak value of 1.22 mm total heave. By comparison, Test 5 only reaches a peak total heave of 0.84 mm.

In the case of the two tests studying increased specific surface, both tests show a similar, consistent rate of expansion once the initial rapid expansion phase has passed, however, because this phase lasted longer for Test 3, it shows a greater level of final expansion. Had Test 2 not been removed, it is likely that it would have progressed in a manner similar to that of Test 3, and would be showing a comparable total heave of around 1 mm.

It should be noted, that in contrast to the control and increased compaction tests, the test showing more expansion in this pair, Test 3, actually has the lower bulk density: 1876.6 kg/m<sup>3</sup> compared to Test 2 which is 1907.9 kg/m<sup>3</sup>. As stated above, the expansion rates beyond the initial rapid expansion phase are comparable suggesting that it is the initial rapid expansion period that had the most influence upon the difference in total heave seen between these two tests.

In contrast, the tests with decreased specific surface show less rapid initial expansion but then show variations in the settled expansion rate. Test 4 seems to show signs of a decrease in reaction rate in the latter third of the testing period, whereas Test 5 shows an increase in reaction rate over the same period of time.

Although the material for the decreased specific surface tests was separated into fine and coarse with set limits based upon the size required to prevent boundary conditions and the  $D_{50}$  point, it is not certain exactly what the distribution between these points is. In other words, we know that the decreased specific surface tests contain 75% material between 35 and 6 mm and 25% material finer than 6 mm, however we do not know the precise distribution of the material between these two points.

Since the aim of Tests 2 – 5 was to study the effect of increased or decreased specific surface, the material was divided simply by weight and an exact particle size distribution was not generated for each sample. Therefore, the exact distribution of particle sizes between 6 and 35 mm in Tests 4 & 5 is unknown.

It is possible that the material used in Test 5 contained more material closer to the 35 mm grain size than to the 6 mm size within the coarse fraction. Although the material was well mixed before being compacted in the tubes, this difference in grain sizes present is still possible. This presence of more of the coarsest fraction within Test 5 would account for the increase in reaction rate in the latter third of the testing period, as the pyrite in these coarse particles began to react.

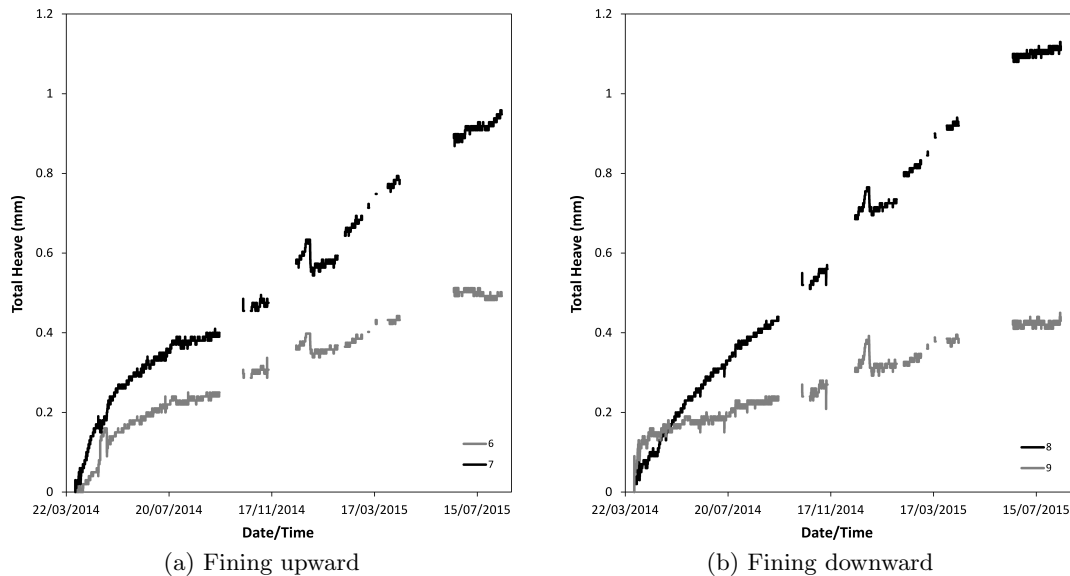


Figure 8.16: Expansion of the samples intended to have fining of the grain size upwards and downwards through the tubes over the test period.

As stated above, Tests 6 – 9 contain material placed into 3 sections that contain either

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all coarse, all medium or all fine material, with the divisions between these sections being at particles sizes of 16 and 6 mm respectively. Tests 6 & 7 have the coarse material at the base of the tubes and the fine at the top, with Tests 8 & 9 the opposite. In both cases, this was intended to determine what effect having a portion of either the fine or coarse material in a saturated or partially saturated state would have on the expansion of the tests.

For the tests that contained the fine material at the top of the tube (Tests 6 & 7), it is the sample with the highest bulk density that shows the most amount of expansion, with Test 7 having a density of 2018.2 kg/m<sup>3</sup> and Test 6 a density of 1849.7 kg/m<sup>3</sup>.

Both tests show an initial period of slightly faster expansion, although unlike Tests such as the control, it is significantly less pronounced, with both tests settling into a more steady state of expansion after a shorter period of time. Again, these tests show a change in reaction rate over the final third of the testing period, with Test 6 showing a decrease in reaction rate and Test 7 showing an increase. This will be discussed below in the context of the reaction rates of all the tests.

Tests 8 & 9 contain the fine material at the base of the tube, with some of this material correspondingly either below the water level or within the zone of capillary rise. Of the pair, Test 8 shows the most consistent expansion, with a rate that varies little over the testing period until the water was removed, after which time the rate slows (Section 8.5.1). Interestingly, this is also the test with the lower bulk density of the two: 1844.7 kg/m<sup>3</sup> compared to Test 9 which has a density of 1900.6 kg/m<sup>3</sup>.

Test 9 also shows a consistent rate of expansion, although this rate is less than that of Test 8, and also shows a small sign of a rapid expansion at the start of the testing period. Despite that, this is the test that shows the lowest overall expansion, showing just 0.43 mm of heave in the testing period.

From the data used to produce these graphs, it is possible to calculate both the rate of expansion over the test period, and the average expansion of the fill per year. In the literature, most expansion rates are presented as either mm/year or mm/m/year, the latter showing the normalised expansion where the fill thickness is known and representing the expansion as the % of the fill that expanded over a year.

For the Sheffield tests, considering the period of rapid expansion that many of the tests showed at the start of the testing period, three expansion rates were considered:

1. The rate of the period of initial rapid expansion, where it was present,
2. The settled rate of expansion, excluding any rapid expansion that occurred, and
3. The ‘all in’ rate of expansion, which uses all the data from the testing period and accounts for the rapid initial expansion as a part of the overall expansion process.

Table 8.8 shows the variation in each of these expansion rates for each test carried out.

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Test no.	Total Heave (mm)	Initial rate (mm/year)	Settled rate (mm/year)	All In rate (mm/year)
1	1.36	3.97	0.78	1.11
2	0.87	3.48	0.42	0.71
3	1.13	4.46	0.61	0.92
4	0.75	1.74	0.49	0.61
5	0.77	1.83	0.50	0.63
6	0.5	1.15	0.34	0.41
7	0.9	2.02	0.61	0.73
8	1.09	1.53	0.84	0.89
9	0.42	1.65	0.25	0.34
10	1.8	9.00	0.93	1.47
11	1.37	4.12	0.76	1.12

Table 8.8: Expansion rates for the Sheffield swell tests.

The values shown in Table 8.8 were calculated from the swell tests data gathered from the start of the test until the point at which the water was removed on the 23rd June 2015. The point taken for each test to separate the initial expansion period from the settled expansion period was that at which any variations in the gradient of the graph ceased and the settled behaviour began.

It is possible that the initial rapid expansion period is due to thermal expansion of the material, however, considering that the tests were positioned within the laboratory 10 days prior to addition of water and the start of the test, the temperature changes in the material should have been minimal. Additionally, considering how changes in the laboratory air temperature effected those in the fill as discussed in Chapter 6, Section 6.2, it seems unlikely that any thermal effects would last for the period seen in the swell tests.

For the rest of the analysis carried out herein it was decided to use the ‘all in’ expansion rate, as this better represents the full activity of the fill during the testing period.

Throughout much of the literature, it is common to see reference to ‘normalised’ data, in which the rate is shown as either the percentage of the fill that oxidises per year (% fill/year) or as the amount of expansion that is seen per year for every metre of fill present (mm/m/year). Both of these methods account for the fill thickness in a given situation, and allow that rate to be compared to fill thicknesses at other locations. Both values for the Sheffield swell tests are shown in Table 8.9, while Figure 8.17 shows graphically how the normalised expansion rate varies according to the variable considered in each test.

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Test no.	All in rate (mm/year)	Compaction density (kg/m <sup>3</sup> )	Fill thickness (mm)	%fill/year	Normalised rate (mm/m/year)
1	1.11	1950.7	750	0.15	1.48
2	0.71	1907.9	770	0.09	0.92
3	0.92	1876.6	740	0.12	1.24
4	0.61	1881.3	750	0.08	0.81
5	0.63	1917.2	730	0.09	0.86
6	0.41	1849.7	750	0.05	0.54
7	0.73	2018.2	770	0.10	0.95
8	0.89	1844.7	740	0.12	1.20
9	0.34	1900.6	750	0.05	0.46
10	1.47	2057.2	750	0.20	1.96
11	1.12	1908.2	770	0.14	1.45

Table 8.9: Averaged expansion rates normalised to % of fill that expands per year.

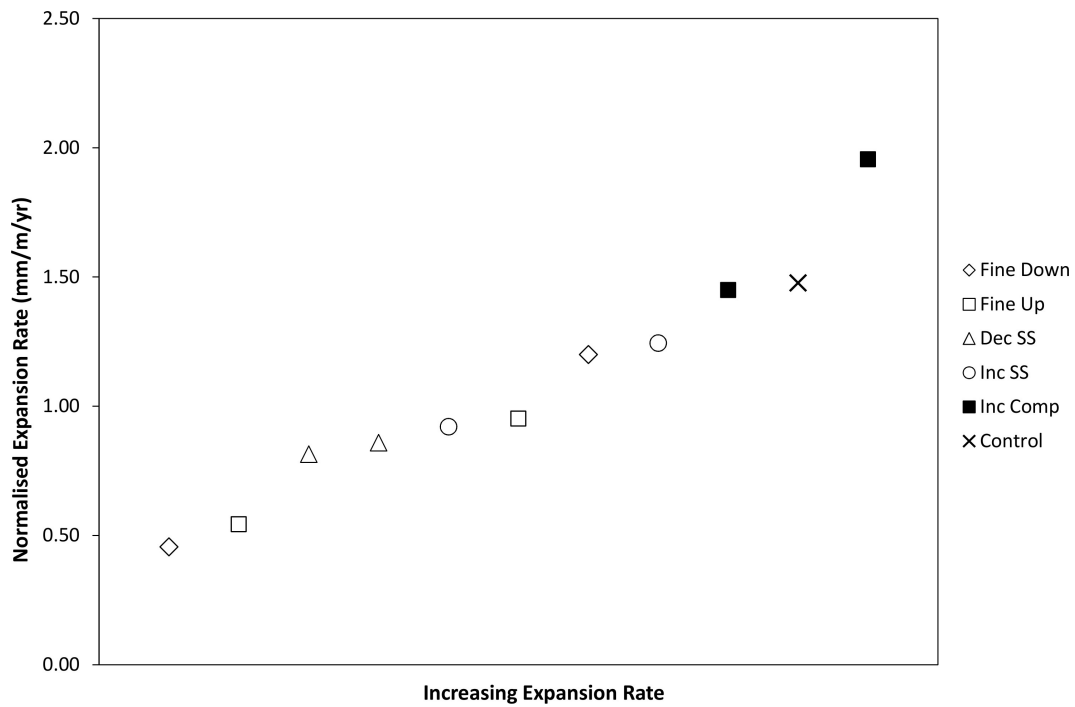


Figure 8.17: Variation of the normalised expansion rate of the Sheffield tests according to test type.

When compared to the values available in the literature, these values are at the lower end of the range seen in the laboratory tests on Irish mudrock, but are comparable to those seen in other locations, as shown in Table 8.10. The values for the rate of expansion

here are given as mm/year since this is the most common method through the older literature of reporting the data, and in many cases the fill thickness is not stated to allow normalisation of the values.

Source	Data source	Rate of heave
Sheffield tests	laboratory	0.34 – 1.47 mm/year
Maher et al. (2011)	laboratory	1.8 mm/year
Sutton et al. (2013)	laboratory	2 – 3 mm/year
McCabe et al. (2015)	laboratory	2 – 4.5 mm/year
Penner et al (1970)	field data	18.9 mm/year
Quigley & Vogan (1970)	field data	3.8 mm/year
Grattan-Bellew & Eden (1975)	field data	1.4 mm/year
Berube et al. (1986)	field data	up to 20 mm/year
Hawkins & Pinches (1987a)	field data	1.7 mm/year
Wilson (1987)	field data	0.6 mm/year
Ballivy et al. (2002)	field data	14.4 mm/year

Table 8.10: Comparison of expansion rates from both laboratory tests and case studies.

## 8.5 Modifications

### 8.5.1 Removal of Water

In order to see how the reactions progressed without an external water supply, the water was drained from the tank and data recording continued. No results are presented here for tests 2 (increased specific surface) and 10 (increased compaction) as they were used for the tests discussed in Section 8.5.2.



8 Laboratory Testing – Development and Modification of the Swell Tests

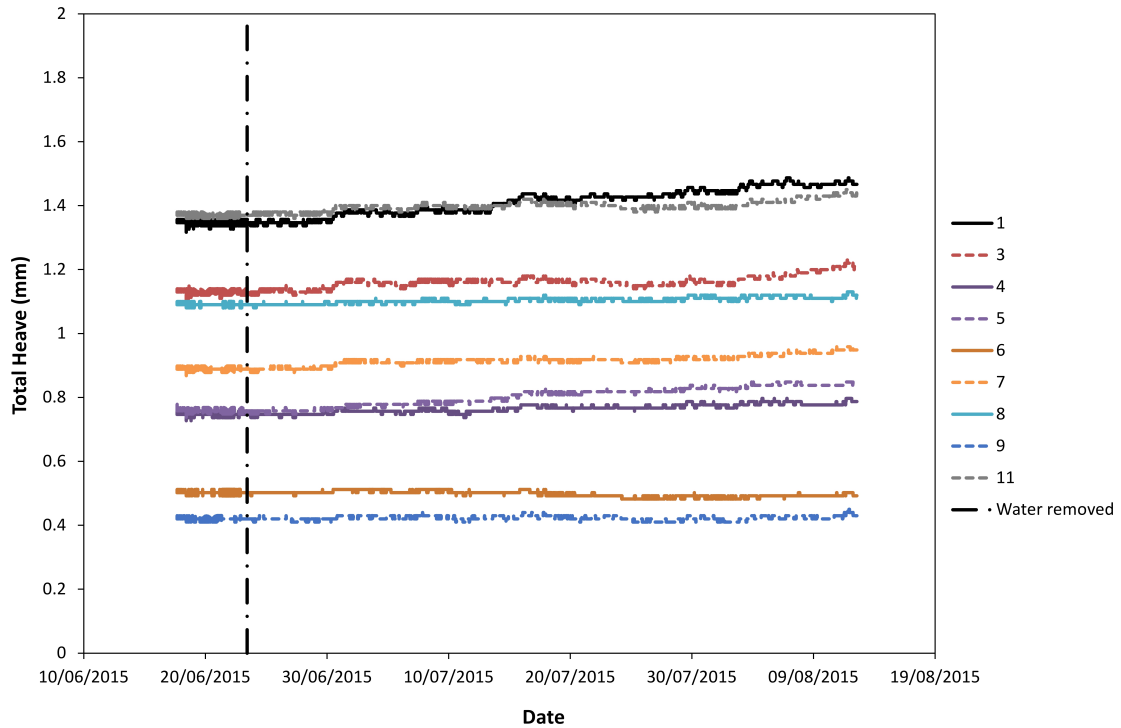


Figure 8.18: Expansion of the swell tests immediately after the water was removed.

Figure 8.18 shows the readings from the tube just before and for 6 weeks after the water was removed. At this point the data was being recorded every 15 minutes in order to better determine any changes to the rate of expansion.

Two points are immediately noticeable

- There is more variation in the readings – it is thought that this is due to the increased data sampling rate picking up temperature effects in the material.
- The expansion rate has slowed significantly in most cases, with some of the tests showing no expansion since the water was removed.

Table 8.11 compares the initial expansion rates for each test to those after the water was removed.

Test no.	Initial expansion (mm/year)	Initial rate (mm/year/m fill)	Second expansion (mm/year)	Second rate (mm/year/m fill)
1	1.11	1.48	0.78	1.04
3	0.92	1.24	0.59	0.79
4	0.61	0.81	0.26	0.35
5	0.63	0.86	0.52	0.71
6	0.41	0.54	-0.07	-0.09
7	0.73	0.95	0.39	0.51
8	0.89	1.20	0.20	0.26
9	0.34	0.49	0.07	0.09
11	1.12	1.45	0.46	0.59

Table 8.11: Expansion rates after water removal on 23 June to August 2015.

Based upon the control test used by Sutton et al. (2013), and the progress of Test 1 in the Sheffield tests, both of which had limited access to water and yet still showed signs of expansion, it was anticipated that once the water was removed from the tank the tests would all continue to show signs of expansion.

As can be seen from both Figure 8.18 and Table 8.11, although there was continued expansion in the majority of the tests, the rate decreased significantly once the water had been removed. The control test carried out by Sutton et al. (2013) actually showed more expansion than some of the tests containing water, so this decrease in reaction in the Sheffield tests was unexpected.

However, it was noted that the initial set of Galway tests were housed in a storage unit that had no temperature or climate control installed, whereas the Sheffield tests are located in an air conditioned laboratory.

Humidity sensors were in use in the laboratory during the initial testing of the Arduino system (Chapter 6), and although the data from within the tubes might not have provided the answers needed at that time, there were also 2 additional relative humidity sensors monitoring the relative humidity changes in the laboratory air. Figure 8.19 shows the temperature and relative humidity changes in the laboratory over the course of 24 hours. Note that these readings were taken before the placement of the swell tests, but there is no reason for the conditions to have significantly changed.

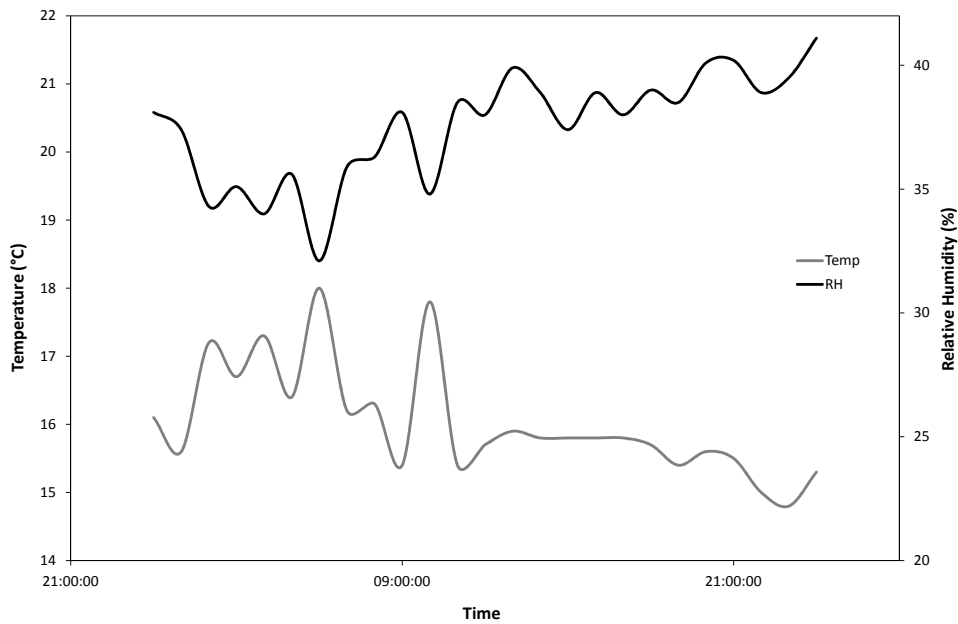


Figure 8.19: Variation of the laboratory temperature and relative humidity over a 24 hour period.

The relative humidity in the laboratory over the time shown in Figure 8.19 goes from a low of 32.1% to a peak of 41.1%, with these low points corresponding to times at which the temperature was increased by movement of people in and out of the laboratory.

Over the full length of time that this particular sensor was in place, some 88 days, the relative humidity in the laboratory ranged from 23% to 56%, with an average value of 36.2% relative humidity. Although climatic data for Galway that included humidity was difficult to obtain, similar data for Dublin Airport is easily available through the Irish Meteorological Office. Figure 8.20 shows the annual variation in the relative humidity in Dublin at 9 am and 3 pm, this data is averaged from readings taken over a 29 year period (Met.ie, 2010)

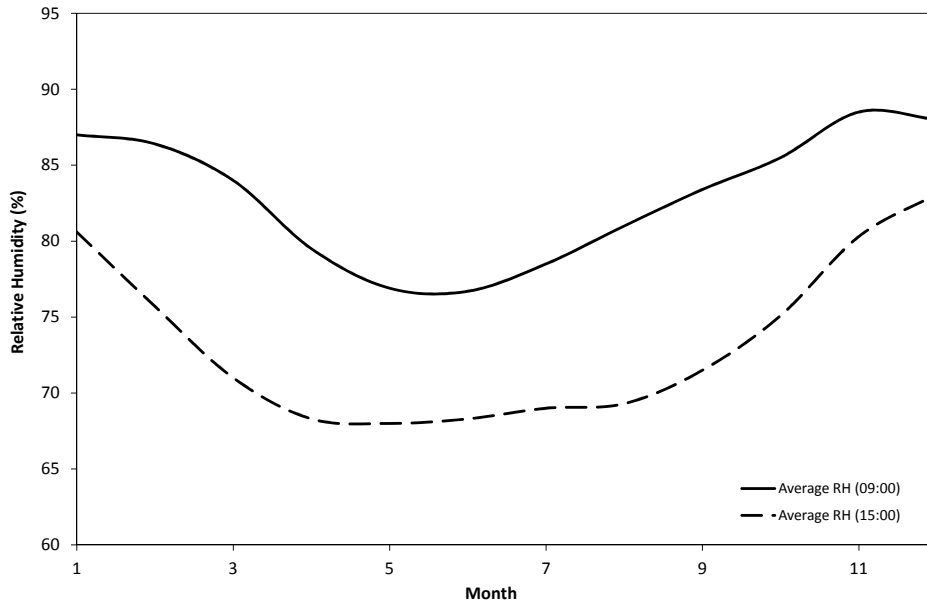


Figure 8.20: Variation of the average relative humidity in Dublin over the course of a year (Met.ie, 2010).

The peak average humidity shown in this data is 88.5% whilst the minimum is 68%. Both of these values are larger than the peak value of 56% obtained from readings in the laboratory, and the lowest site value is almost double the laboratory average value.

It is therefore assumed that the decrease in reaction within the tests is associated with drying out of the samples after removal of the water. The control sample discussed by Sutton et al. (2013) that reacted even without a separate water supply was in an environment that had a relative humidity that was likely between 68 – 88%, which would have provided additional moisture for the reaction process.

In the case of the Sheffield tests, the humidity only ever reached a maximum of 56% and had an average of around 36%. This provides much less moisture to help maintain the reaction process, leading to a slower rate of expansion.

In order to study this effect further, it is planned to remove some tests from the water environment and monitor their expansion whilst away from the tank of water that would provide an increase in local humidity. Of those tests remaining within the tank, the lower air and water access holes of some of the tests will be sealed, such that they only have access to the increased local humidity, whilst the remaining tests will have full access to

water restored.

In order to account for the differing expansion rates that the tests showed before the water was removed, the tests will be sorted such that there are comparisons in expansion rate possible between those with access to water, access to humidity or neither. For example, Tests 1 & 11 showed expansion rates of 1.11 and 1.12 mm/year before the water was removed, and so could be compared with access to just water or just humidity once the planned additional testing begins.

### **8.5.2 Sample Restriction**

The literature offers several values for the amount of pressure induced upon a given system by the oxidation process and subsequent precipitation of gypsum. These values are based, in most cases, upon back calculation from data available in case studies (Table 2.1, Chapter 2) and the implied value obtained by Maher et al. (2011) discussed in Section 8.2.1 above.

These values range from as low as 28 kPa (Berube et al., 1986) to as high as 600 kPa (Maher et al., 2011), and as such give comprehensive lower and upper estimates of the pressure that would be expected to be recorded within material undergoing pyrite related heave.

It was therefore decided to try and determine the amount of pressure induced by the expansion of the material in the swell tests.

Since the swell tests were already restrained on all except the upper surface, restraining this free surface and measuring the pressure against it was the quickest way to determine whether the pressure could be measured or not.

To this extent, two of the swell tests were removed from the tub of water and placed into triaxial frames, with the upper surface restrained by the frame and any movement registered by a proving ring, as shown in Figure 8.21.



Figure 8.21: Placement of swell test 10 within the triaxial frame.

The two swell tests that were used were one with increased specific surface and one with increased compaction. These were chosen as they had previously been showing a relatively high expansion rate and as such were anticipated to continue expanding once restrained in the triaxial frame. The proving rings used initially were 5 kN and 10 kN rings, with the 5 kN ring on test 2 with increased specific surface, and the 10 kN ring on test 10 with increased compaction, with 0.002 mm scale dial gauges on both the proving ring and the surface of the material in order to measure both movement of the ring and

any expansion that occurred.

The tests were monitored over several weeks, with the 10 kN ring showing no movement and the 5 kN ring showing a movement of 0.07 mm which gave a reading of 71 N on the proving ring. However, this reading was achieved within the first 2 weeks with no movement being monitored after this point.

Considering that no movement was recorded on the dial gauge set on top of the fill, it is unlikely that this reading on the proving ring was a steady state pressure, but was caused by some movement, most likely someone knocking the tube and/or frame. In order to confirm this, the tests were left in place and the dial gauges monitored. When no further movement was recorded on either gauge, it was concluded that either the restriction imposed upon the surface of the material by the proving rings was either too large, thus inhibiting the expansion of the fill, or, more likely, the lack of water supply had significantly slowed the reaction as discussed in Section 8.5.1.

At the time of writing, work was ongoing to safely modify the set up in the triaxial frame in order to re-introduce water to the tests. It was also decided that once the tests are re-started, they will initially be monitored with just the dial gauge on their upper surface for the first week or so to ensure that the material is swelling before replacing the proving rings.

## **8.6 Discussion**

The swell tests carried out at Sheffield built upon work carried out in Ireland to further consider the effect of different variables upon the amount and rate of expansion of pyritic mudrocks.

In this case, the effect of an increased density was confirmed, with the two tests compacted at high bulk density (Tests 10 & 11) showing more expansion than the other tests with changed variables.

The other tests were used to study the effect of grading upon the expansion amount and rate.

The reaction within the fine material progresses quicker due to its larger specific surface allowing more access of air and moisture to the framboidal pyrite. Similarly, the pyrite within this fine material is oxidised more rapidly than that contained in the coarser material, meaning that although the reaction progresses quicker, it will also not last as long. This is shown in Tests 2 & 3, where the reaction rates are some of the highest in material that was not compacted to a specifically high bulk density.

The pyrite in the coarse material, which has a lower specific surface, oxidises slower and over a longer period of time. The action of the precipitation of gypsum along the laminations in the larger material particles opens the laminations and so increases the

specific surface and encourages the reaction process. This is seen in Tests 4 & 5, the latter in particular, where there are signs of an increase in reaction rate in the latter portion of the test period.

Most of the tests show an initial period of rapid expansion, in which the rate of expansion of the material is frequently more than double that of the ‘all in’ rate. Considering the small amount of movement likely to occur in either the aluminium frame or the mudstone fill with a maximum temperature range of 5°C, and that all the tests were allowed to equalise to the laboratory temperature before the start of the test, this initial expansion period is believed to be a part of the reaction process in this case.

Modification of the tests by removing the water supply from the tank at the base of the tubes caused a significant decrease in the reaction rate of all of the tests. This was determined to be due to the drying out of the material caused by the low natural humidity in the air conditioned laboratory.

This implies that the relative humidity of the environment plays a crucial factor in the reaction process, and is supported by work by Sutton et al. (2013). In those tests the control sample was allowed to react without an external water supply, but while being stored at the natural humidity that was likely to be at least 68% compared to that of the Sheffield laboratory which averaged at 36%.

Further work is planned to further study this link between the humidity and the reaction process, as detailed in Sections 8.5.1 and 9.3, with some samples having their water supply re-introduced and others being sealed off from the water but in an area of increased local humidity.

This re-introduction of the water supply will also be used to monitor the swell pressure of the tests currently restrained within the triaxial rig as detailed in Section 8.5.2, as currently neither of these tests are expanding enough for a swell pressure to be measured. This is again detailed further in Section 9.3.



## 9 Discussion and Conclusions

### 9.1 Discussion

Pyrite is a naturally occurring mineral found in many different rock types, formed in many different environments. Of particular interest in this case is the presence of fine-grained framboidal pyrite – this has a larger specific surface due to it being formed of clusters of small grains, and is so more reactive than the cubic form.

This increased specific surface means that the pyrite oxidises easily in the presence of air and water, leading to the production of sulphuric acid. This has the dual effect of making the local groundwater more acidic, and leads to the reaction of the sulphuric acid with other minerals present in the local rock.

Where calcareous mudstone is present, either as local bedrock or as fill material, the acidic groundwater will react with calcite and lead to the production of gypsum. This gypsum will precipitate out of solution into space within the parent material – this includes void spaces created during compaction of fill, and along laminations that may occur both in natural bedrock and in larger pieces of fill material.

Eventually the amount of precipitation will exceed the capability of the environment to constrain it, which leads to expansion of the parent material. Where this occurs in a natural outcrop or in a quarry face, expansion is more likely to occur in the surface layers of the material, where there is better access to air and water, and the material is unconstrained.

However, when this expansion occurs in material below a floor slab, there is generally better access to air and moisture to more of the material. Either this is caused by the placement of damp fill material, or due to the opening of fractures and laminations in bedrock as part of the construction process. Additionally, the system below the floor slab is constrained laterally by the foundation walls and vertically by the floor slab above and the natural material below (either the unreacted bedrock material, or the material upon which the foundations are sited).

This restriction of the material means that the expansion induces pressure upon the floor slab and foundation walls, which leads to the damage seen in pyritic heave cases. Estimates of the amount of pressure caused by the precipitation of gypsum, based on laboratory and field data, range from a low of 28kPa (Berube et al., 1986) to a high of 600 kPa (Maher et al., 2011).

## 9 Discussion and Conclusions

The amount and rate of the expansion caused by the precipitation of gypsum is dependent on several environmental factors that include temperature, density of fill, depth of fill and water supply. In order to better understand the effect of variation of these factors, tests have been carried out both in Ireland and as a part of this study, and are considered further below.

These factors can vary just as much as the composition of the material might in the space below a floor slab, and it is this variation of the material and the associated variation in amount of heave leads to differential uplift of the floor slab. This in turn causes the typical stellate cracking seen in many cases of pyritic damage, and damage to fixtures and fittings that are seated on the floor slab. Attempts to address the link between the chemical factors and the uplift of the slab are discussed below in reference to Chapter 7.

As mentioned, the uplift of the floor slab leads to damage of the structure, with some of the damage being key in determining that pyritic heave was the cause. The damage seen in the Irish cases has been studied as part of this thesis and is discussed further below.

However, there are multiple instances in the literature in which cases of pyritic heave in other areas of the world have been recorded and studied. This thesis aims to present a single resource for those historical examples of damage caused by pyritic heave across Europe and North America, including those in Canada and Ireland. These case studies cover cases both where structures have been built on pyritic bedrock and those where pyritic fill material from a secondary location has been used below the ground floor slab.

Although the amount of information available varies from case to case, the majority of them detail important factors such as whether the source of the pyrite is local bedrock or imported fill, details regarding the chemistry of the material, and the rate/amount of heave seen at a given site.

This information, particularly the comparison of material chemistry and expansion amount/rate, is discussed in Chapters 3 and 8, and is summarised in Table 9.1 for ease of reference.

## 9 Discussion and Conclusions

Source	Pyrite level (%)	Rate of heave
Sheffield tests	1.4	0.34 – 1.47 mm/year
Maher et al. (2011)	1.91	1.8 mm/year
Sutton et al. (2013)	2.1 – 3.6	2 – 3 mm/year
McCabe et al. (2015)	1.36	2 – 4.5 mm/year
Penner et al (1970)	1.3 – 1.6	18.9 mm/year
Grattan-Bellew & Eden (1975)	4.25	1.4 mm/year
Berube et al. (1986)	1 – 5	up to 20 mm/year
Wilson (1987)	2.2	0.6 mm/year

Table 9.1: Comparison of expansion rates from both laboratory tests and case studies with pyrite levels. N.B. Pyrite levels for laboratory tests are those given at the start of the test period. For field data the value given here is that provided by the relevant paper, and is assumed to be the value at the time of sampling.

This further confirms the idea put forward both in the case studies, and in the standards that reference problems associated with pyrite and other sulphides, that even a low level of pyrite can lead to enough expansion of the material to cause damage to the structure.

Also of note is the variation in expansion rates that is highlighted in Table 9.1. This variation does not seem to be directly linked to the amount of pyrite, nor to the classification if defined by whether the material is bedrock or fill.

There has been some speculation in the literature that this difference in rate can be linked, in part, to the local climate, and in particular to the average temperature.

The reasoning for this difference is that the *Thiobacillus* bacteria that catalyses the reaction process is known to operate best between temperatures of 25 – 40°C (Holt, 1977). This implies that with a lower average temperature, the bacteria would not be as active during the coldest periods and this would lead to a slower reaction rate.

This assumption is supported when considering the average temperatures in locations that are both known to have suffered pyritic heave, and for which there is a certain amount of evidence. In this case, that comprises cases in Canada (Section ??) and in Ireland (Chapter 4).

The average temperature in Ireland is cited as being around 10°C (Tuohy et al., 2012), whilst that in Canada averages lower at around 4°C (CTQ-M200, 2001). This is reflected in the average time between construction of a property and the first noticeable appearance of damage: 3–5 years in Ireland and 10–15 years in Canada.

The inverse is also implied when considering the damage seen at Rideau health centre (Quigley and Vogan, 1970), at which damage was believed to have been increased by the ideal conditions created below the floor slab. In this case the foundation style led to increased average temperatures in a plug of bedrock material that had ample access to air and moisture.

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This theory seemed to be supported by initial work carried out at the University of Galway, Ireland, where swell tests (discussed in more detail below), in which tests stored in an uninsulated shed showed a decrease in expansion activity during periods in which the temperature also dropped (Section Sutton et al., 2013)).

In order to clarify this, further tests were carried out at Galway, in which the tests were held in a climate-controlled room in which the temperature was carefully regulated (McCabe et al., 2015). In these tests, the behaviour of the material (the expansion amount and rate) was monitored over a period during which the temperature was changed at regular intervals, ranging from 10–20°C.

Although initial results appeared to confirm an increase in the amount of expansion with the temperature increases, there was less indication of a corresponding decrease in the amount of expansion when the temperature dropped. Additionally, the overall rate of expansion was not affected, leading to further investigations. It was determined that the short increase in expansion after the temperature change was caused by thermal expansion of both the aluminium frame holding the dial gauges, and of the fill material itself. There was a corresponding shrinkage of the frame and material when the temperature was decreased, although this seemed to be smaller.

Although this seems to indicate that the temperature changes have little or no effect on the overall amount and rate of expansion, the situation is not actually that clear. The temperatures used during the test were 10, 15 and 20°C, on the basis that this was believed to be the range of temperatures that were likely to be present below a domestic floor slab.

In order to determine the range of temperatures likely to be present, the Arduino system was designed to monitor temperature changes in the fill material below a domestic floor slab in comparison to those in the room above the slab. This is discussed further below in relation to the Arduino system.

Although it is unlikely that the temperatures below a domestic floor slab will dramatically exceed the 20°C maximum used in the Galway tests due to the influence of external temperature variations, factors such as central heating or under-floor heating will play their part in affecting the temperature in the fill. A similar effect to this is seen at both Llandough Hospital and Rideau Health Centre (Chapter 3), where service ducts raised the temperature below the slab considerably above the external or even internal building temperature.

As stated above, temperatures between 25–40°C are where the bacteria are most active. So if the below-slab temperature were to rise above 25°C due to the influence of heating systems, it could cause an increase in the reaction process that was not seen in the Galway tests. However, this cannot be stated with certainty without further testing. It is also possible that any increase in the reaction rate due to higher temperatures would be overshadowed by the influence of other factors, as is discussed below.

Other factors that affect the amount/rate of expansion were considered by the Galway

## 9 Discussion and Conclusions

tests, including the thickness of the fill material, the depth of the water and the density of the material. The results from these tests were more conclusive than those relating to the effect of temperature changes.

It was determined from the initial set of tests (Sutton et al., 2013) that the expansion rate was roughly proportional to the thickness of the fill. This was inferred by some of the case studies in the literature, and confirmed by the tests, since with a greater amount of material available and reacting at one time, more precipitation of gypsum and so more expansion will occur.

Conversely, the amount of water present in the test was found to have no effect on the amount or rate of expansion. In the initial tests (Sutton et al., 2013) the control sample, which had no additional water supply other than the moisture present when it was compacted, showed the highest reaction rate. This is due to the fact that water is produced by the chemical reaction itself, meaning that it is actually self-sustaining when no external moisture is present.

Additionally, when a large amount of liquid water is present in the system, it actually inhibits the reaction process. When the fill material is saturated, the air cannot react with the pyrite and so the reaction process stalls. This is why, as discussed below, one of the remediation options considered when dealing with cases of pyritic heave is to raise the local groundwater table.

So, while a lower level of water will have little or no effect on the reaction process, a higher level of water can reduce the activity of the material and lead to less expansion of the material.

The other factor that was considered in the Galway tests was the compaction density of the material.

Since the gypsum precipitates into void space within the material, either voids caused during compaction or laminations within the material, less void space should lead to a faster reaction rate. In order to consider this point, three pairs of tests were set up with respectively low, moderate and high compaction levels (McCabe et al., 2015).

As predicted, the tests with the highest level of compaction showed both the greatest amount of expansion and also the highest expansion rates. The tests with low compaction levels did not appear to react initially, although they did begin to expand after some time had passed. In these tests, the initial precipitation filled the larger amount of void space present in the material before precipitating in ways that would lead to expansion of the test.

This has been confirmed in tests carried out as part of this work, as is discussed further below with the other swell test information.

One of the other key points indicated in the Case Studies and discussed in Chapter 4, is that they highlight that the presence of framboidal pyrite can lead to expansion that

## 9 Discussion and Conclusions

will damage a property, and that the level of pyrite does not need to be particularly high for this to occur.

This is further emphasised by the mention of pyrite, or more often sulphides, throughout various standards related to construction that are in place now and, more importantly, were in place at the time of the Irish housing boom. All of the standards discussed below were in use and relevant during the Celtic Tiger housing boom, the peak of which existed between 2000 and 2007 (Tuohy et al., 2012).

Many of the standards focus initially on the size of particles for use as fill, with both the Homebond Manual and the Irish Building Regulations aiming to specify a material that would compact well. Interestingly both these sources make reference to potential contaminants in the material, with the Homebond Manual referring to potential problems due to the presence of sulphates, and the Building Regulations saying that fill material should be free of “matter likely to cause damage to concrete”.

Although in both these instances, the references are intended to filter out material likely to cause sulphate attack on concrete, one of the minerals likely to lead to sulphate attack is also the one that leads to the precipitation of gypsum – pyrite.

With this consideration of substances likely to cause sulphate attack on concrete, many standards refer to the maximum level of water soluble sulphate (WSS) present in material to be used within a specified distance of buried concrete. In some of the earlier standards, this distance is for material to be used within 1 metre of buried concrete, by the early 2000’s this distance had been decreased to 0.5 metres.

Some standards also specified a limit for the amount of acid soluble sulphate that could be present in material intended for use within 0.5 metres of buried concrete. The Highways Agency (UK) and National Roads Authority (Ireland) both revised their documentation in 2000 to give limiting values of 0.24%  $\text{SO}_4$  for acid soluble sulphate levels in material intended for use within 0.5 metres of buried concrete.

Either inferred or outright stated in these standards is a requirement for testing material before its use. Whether for placement as sub-base for roads or as domestic fill, when in use within 0.5 metres of any buried concrete, the values for the water and acid soluble sulphate of the fill material should be known.

Both EN13242:2002 (BSI, 2002) and EN13285:2003 (BSI, 2003) place the onus for this testing onto the producer or supplier of the material, although it is reasonable to say that it would also be on the part of the purchaser to ensure that testing of the material had occurred.

The production of TRL Report 447 in 2001 (Reid et al., 2005) and BRE Special Digest 1 in 2005 (Establishment, 1995) detailed further the limits required for what the BRE termed “aggressive ground” with respect to its placement near buried concrete. More importantly, these two documents revised the limits for sulphate values downwards.

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TRL Report 447 (Reid et al., 2005) gave initial limits for material within 0.5 metres of concrete of:

- 2.3 g/l SO<sub>4</sub> for WSS
- 0.6% SO<sub>4</sub> for TPS (total potential sulphate)
- 0.46% SO<sub>4</sub> for OS (oxidisable sulphides)

These were reduced even further in the 2005 version of the report:

- 1.5 g/l SO<sub>4</sub> for WSS
- 0.3% SO<sub>4</sub> for OS

Figure 5.13 (Chapter 5) shows the range of chemical values from material sampled in Development Alpha along with the relevant limits for each value at the time of writing. It should be noted that for the graph of WSS values, all samples exceeded the 500 mg/l limit specified in IS 398-1 (NSAI, 2013).

Although these values represent the state of the fill material at the time of sampling for remediation rather than at the time of placement, it is considered to be indicative that the material is not suitable for purpose. It can be argued that the values may have increased over the reaction process, as the pyrite oxidises and produces an acidic environment, and so the values at the time of placement might have been within the levels of the standards. However, this is the purpose of considering more than one chemical value, since as the pyrite is oxidised and “used up”, other values will decrease.

It can be argued that some of these standards or reports, such as the TRL Report or the BRE Special Digest, were not necessarily relevant in Ireland in the early to mid 2000’s, being produced by primarily British organisations. However, standards such as the Homebond Manual, Irish Building Regulations, National Roads Authority and EN13232:2002 (with Clause SR21, the Irish documentation) were all relevant in Ireland at the peak of the housing boom. Table 4.1 summarises these standards and the limitations they present.

That material was used that does not and did not meet these standards seems evident from the available data. Even if it could be proven that the material met the applicable standards at the time of placement in terms of chemical limits, the high mudstone component and the additional breakdown of particles that has occurred due to the precipitation of gypsum, calls into doubt the suitability of the material due to its strength.

It cannot be doubted that the fill material in the Irish cases contains pyrite, this is evident in the SEM and this section images produced as part of the insurance claim process. It is also shown through these tests and in the literature that gypsum is present both surrounding the larger rock particles and along laminations in the mudstone particles. In Ireland, both pyrite and gypsum have been proven to be present in fill material below properties where uplift of the floor slab and associated damage has occurred.

## 9 Discussion and Conclusions

If the damage to a property has been caused by uplift of the floor slab, which has in turn been caused by expansion of the fill material, then surely the fill material can be classified as defective and not fit for purpose.

Nevertheless, there is now a situation in Ireland in which potentially as many as 12,250 domestic properties had or currently have pyritic fill material situated below their floor slab (Tuohy et al., 2012). The reason for the expansion of this pyritic material is well known as detailed in Chapter ?? and discussed above, the degree of influence that various factors have upon the amount and rate of expansion is not as well understood.

As discussed above, tests at the University of Galway, carried out on Irish mudrocks known to contain pyrite and to be reactive, identified that the thickness and the level of compaction of the fill material had a direct effect on the amount/rate of expansion. A deeper layer of fill, compacted to a higher density would lead to more expansion at a faster rate than a thin layer compacted to a low density.

These tests also implied that temperature variations had little effect on the reaction process, seemingly contradicting theories put forward by evidence in the literature in which cases such as the Rideau Health Centre implied the presence of heating elements below the slab speeded the reaction process.

In order to better understand the conditions below a domestic floor slab, a monitoring system was devised to look at how the temperature and moisture of the fill varied with depth. This system is intended to consider what the expected temperature ranges might be along with how domestic activity, such as the use of central heating in colder months, affects the temperature, and if there is a temperature gradient present in the material. The moisture sensors are intended to determine whether there is a significant input of water into the fill system from the natural clay material below the fill. The success of this system is discussed further below.

Additionally, there are other factors that influence the reaction process that have not yet been considered in detail through laboratory testing. To that end, laboratory tests were carried out as part of this work, duplicating the tests considering increased density to confirm those results, and considering the effect of grading upon the reaction rate. This is also discussed in more detail below.

Initially however, it was decided to try and classify the damage seen in various Irish cases in order to compare this with the chemical properties of the fill material. The aim of this was to determine whether one aspect of the material chemistry has more of an effect upon the damage than others.

Classification systems for the pyritic damage have been used before – as mentioned above, the work carried out in Montreal by Ballivy et al. (2002) was used to determine whether the damage seen in a given property was likely to have been caused by pyritic expansion. Certain types of damage, such as the stellate cracking of the floor slab or cracking of the plinth at DPC level are more indicative of pyritic heave below the floor slab than simple cracking at wall/ceiling interfaces.



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These case studies, combined with the recent development of IS 398-1, presents a unique way to study the damage to the Irish properties, since IS 398-1 was specifically developed to determine whether the damage seen in new Irish cases is likely to be caused by pyritic heave.

In this study, the classification system presented in IS 398-1 was applied to previous cases from a large housing development in County Dublin, with structural reports and photographs taken before and during remediation used to determine the level of damage present. This data was then compared with the chemical data available for each property in order to determine whether there were any correlations present, i.e. whether there were any key factors that could be seen to have a direct influence on the amount of damage seen in a property.

However, it soon became clear that a combination of the limited information available and the influence of factors such as poor construction practices meant that the expected correlations between factors such as the damage level and the amount of pyrite were not present in the data.

Part of the problem, is also that IS 398-1 is intended as a tool to analyse new cases. In these instances, the survey would be carried out with the questions asked by IS 398-1 in mind, whereas the initial surveys did not always look directly into aspects that would provide more information within the context of IS 398-1.

As a structural assessment method IS 398-1 accurately identifies the damage that is most likely to be caused by pyritic heave, and the chemical analysis method provides a structure through which to confirm that the fill for a property contains pyrite and is subject to pyritic heave. However, the number of factors that can influence the rate and amount of heave, combined with factors such as non-pyritic structural defects that may exacerbate progressive damage, are not accounted for within the structure of the standard at present.

In order to consider whether this lack of correlation was endemic to the system being used, the damage level was compared with factors such as the uplift of the slab which are known to be directly linked to the damage. Although the uplift of the slab is accounted for within IS 398, this is in a generic manner, with weighted points being awarded in that section of the classification if the uplift is above a certain level. Since the specific maximum point of uplift was not accounted for within the classification, and since this was being used to see what correlation would be possible, the two were compared.

This still presented a lower than anticipated level of correlation between the damage and the uplift. This is again believed to be due to the influence of factors such as density, which cannot yet be measured accurately in situ, and the other structural defects present in a given property.

However, this does not mean that IS 398-1 and its damage classification scheme should be dismissed as a tool by which to study the damage. It is anticipated that further refinement of the system may present clearer links between the data, whether this is via

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better data analysis or via more advanced statistical modelling.

As mentioned above, in order to better understand the environment below a domestic floor slab, and therefore to be able to further consider what, if any, effect temperature and moisture changes might have on the reaction process, an Arduino-based monitoring system was developed.

The Arduino system is a small, comparatively cost effective, easy to install method of data gathering. With an uninterrupted power supply, it could conceivably be left in place for 12 months with no need for regular monitoring. The addition of an external battery or solar cell could also be included to make the system independent of the power supply, and the possibility to add factors such as remote monitoring further improve its use and range.

This system was designed to record the temperature and moisture of the fill at three different depths, along with a record of the temperature in the room above the slab for comparison. The use of the Arduino system allows for a small, reasonably priced system that gives multiple customisation options, and that can be left in situ for months at a time with minimal maintenance.

Initial work considered the use of combined temperature and humidity sensors, as these had been in use to monitor the humidity in the laboratory during the initial stages of the swell tests. However, the sensors were determined to be unsuitable since they returned a value of 100% humidity at a moisture content of just 8%. Since the moisture content in the Irish fill cases has been recorded as high as 12%, another system was investigated.

Instead, the system was adapted to use separate temperature and dielectric moisture sensors. This system allows more flexibility, as the dielectric sensors can be site directly into the fill material, whilst the temperature sensors would be held in place in the centre of the sampling hole by the backfilled material.

The only problem that was encountered with the dielectric sensors, is that the value returned is affected by the grading of the material. As the sensors return a numerical value that must then be converted to moisture content via a calibration factor, this is not considered to be too significant a problem. In the case of the sensors used as part of this study, the sensors were calibrated using material taken from the fill in which they were situated.

At the time of writing, the sensors have been in place beneath the slab of a domestic property in County Dublin for 8 weeks, with associated data available for that time.

Initial results show that the temperature in the fill is influenced by both the external (i.e. diurnal/seasonal changes) and internal (i.e. room/heating changes) temperature variations. Over the 8 week period, there was a noticeable overall decrease in temperature, in both the fill and the room temperatures.

Of particular interest, is that this data can be compared to the temperature gathered by the Irish Meteorological Service at Dublin Airport that is located a few miles from

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the monitored property. This data gives air temperatures and ground temperatures at 10 cm depth for the same time period as the data collected for the fill.

The soil temperature at Dublin airport has an annual average range of 4–16°C, although this is a 10 year average, with individual months occasionally exceeding this amount. The temperatures at Dublin Airport over the monitoring period averaged out to 15.9°C in August and 13.2°C in September. In the fill over this time, the temperatures ranged between 16 and 17°C, values higher than the annual maximum at the airport even heading into the colder calendar months.

At present it seems likely that the higher temperature range within the fill is caused by the insulating effect of the house and foundation, with the human influence becoming clearer as the external temperature drops further and factors such as central heating come into play. Since the sensors are still in place at the time of monitoring, as more data becomes available it will be possible to determine what influence human activity has on the temperature and if this makes a more suitable environment for the reaction process

Also of note is that the range of temperatures observed match the range of temperatures under which the laboratory samples are being tested, confirming the viability of this aspect of the laboratory tests.

The moisture data currently returned by the Arduino and dielectric sensors is, at present, not as useful as the temperature data. This is partly caused by the limited backfill material available when the sensors were placed – this had a much lower moisture content than the fill material and so there was an initial period in which the fill material alongside the excavation dried out to match the moisture content of the backfill.

This can possibly be improved in the case of future placement of sensors by using backfill material with a higher moisture content, however, the drop in recorded moisture content does mean that it should be easier to determine if there is any upward seepage of moisture from the natural material below the fill.

Additionally, it may be worthwhile to install, if possible, both dielectric sensors and humidity sensors, with the latter providing additional information at very low moisture contents.

The swell tests carried out at Sheffield were designed to build upon the work carried out previously in Ireland. Two tests were used to confirm the link between reaction rate and the density of the material proposed by McCabe et al. (2015), whilst the other tests explored the potential link between reaction rate and the grading of the material.

Given that an increased specific surface would aid the access of air and water to the pyrite and in turn speed the reaction rate, the proportion of ‘fine’ and ‘coarse’ material was varied. This gave two sets of tests with an increased ‘fine’ component, and two sets of tests with an increased ‘coarse’ component. The material in these tests was well mixed before being placed into the test system, in order to see if any accidental layering

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of the material would have an effect – this resulted in two tests in which the material gradually fined upwards, and two in which it fined downward.

The tests with the overall increased specific surface, i.e. the greater amount of fine material, initially showed more expansion and a faster overall expansion rate than the tests with a lower specific surface. This is due to the increased availability of pyrite and an improved access to air and water within the finer fraction of the material.

However, this also means that the pyrite in this material will be used up quicker, meaning that the reaction will not be sustained for as long in the fine material.

Conversely, the tests with a lower specific surface showed a lower reaction rate over the early stages of the test period, with some indication of an increase in expansion rate towards the end of the recording period. Although the initial rate is longer, it is expected that the coarser material would react slower over a longer period of time, as air and water take longer to fully access the larger particles. This access generally occurs as the precipitation of gypsum along laminations in the larger particles cause them to break down, thus increasing the specific surface and exposing fresh material for oxidation. Tests are ongoing to study how this material behaves over time.

Laboratory tests carried out whilst preparing samples for the swell tests also confirmed that although the material might have met Clause 804 limits at the time of purchase, the mudrock material is so weak that the compaction process breaks down the larger particles and increases the fines content. This weakness of the material itself implies that despite the PSD appearing to fit the Clause 804 limits, its physical properties should have discounted the material from being used.

Initial modification of the swell tests involved the removal of the water from the swell tests in order to determine whether the reaction rate would change once they were reacting solely with the water already in the material. However, once the water supply was removed, the reaction process slowed significantly.

Although this is touched upon in Section 9.3 below, it raises the question of whether the higher relative humidity in Ireland, compared to that in Canada, is a more significant driving factor than temperature when considering the faster reaction rates observed in Ireland to those in Canada.

It is thought that the dry environment in the laboratory, which has a low relative humidity of around 36%, caused the tests to dry out beyond the point at which the reaction could support itself. Thus the relative humidity of the environment plays a significant part in the reaction process. These are the first tests to be studied while reacting without a nearby water supply in a climate controlled laboratory. The tests carried out at Galway were either not in a climate controlled space, or had a their own water supply. The control sample at Sheffield, although not directly in contact with the water in the tank, was in the more humid environment that the water tank created.

## 9.2 Conclusions

The initial aim of this study was to gather together key information from the literature relating to both the process of the oxidation of pyrite and the precipitation of gypsum, and to the ways in which this process behaves when in situ beneath a ground floor slab.

To that end, this thesis contains within it a compilation of the available information from within the literature relating to the reaction process, the factors that affect the reaction process, the likely pressures caused by expansion of the material, and the variations in the amount and rate of heave. Following on from this, the work also combines into one resource the details of case studies from across the world that show the effects of the expansion of pyritic material upon various types of structures. This includes locations wherein the material was the natural bedrock and fill material from a secondary location, as is the case in the affected Irish properties.

This consideration of the case studies available in the literature also considers the attempts of other authors to classify the damage caused by pyritic heave and looks at the variations seen in the rate and amount of expansion across the sites where this information is available. Both of these influenced the work carried out in this study, with respect to the work on IS 398-1 and the swell tests.

The consideration of the available literature confirms the influence, or lack thereof, of certain factors upon the rate and amount of expansion. Factors such as the thickness and the density of the fill are confirmed by both information in the literature, and the swell tests carried out as part of this study, to have a direct influence on the amount/rate of expansion, with a higher density and thicker fill layer leading to more expansion. Other factors, such as the amount of water present in the system are confirmed by the swell tests to have less of an influence, with tests that contain no external water supply also expanding since the reaction process itself produces additional water.

With some factors, however, the link was less clear. Work throughout the literature implies that there is a direct influence on the amount/rate of expansion caused by an increase in temperature within the system. This is thought to be caused by the increased activity of the bacteria that catalyse the reaction process at higher temperatures. Conversely, the tests carried out at Galway proved that the influence of the temperature was minimal, although they did not progress to temperatures higher than 25°C that would be expected to have more of an effect.

In order to further consider the temperature influence, the Arduino system was developed. Although at the time of writing this was still in situ and the whole data set was unavailable, initial data shows that the temperature below the floor slab is higher than expected and is influenced by both external air temperature changes and the changes in the temperature of the room above the slab. It is unclear at this time whether the temperature in the fill will rise significantly above 20°C, although it is anticipated that the use of central heating over the winter months will have an effect.

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There have been, in some of the cases seen in Canada, attempts to classify the damage seen in cases of pyritic expansion. In Ireland, these are succeeded by IS 398-1, which is designed for use in new cases where pyritic expansion is thought to be at fault for damage seen in domestic properties. When utilising IS 398-1 to compare the damage seen in the Irish properties to the chemical properties of the fill material, it was determined that a direct comparison was difficult. This was largely caused by the influence of factors that were not accounted for within the data and the natural variation of the fill material. The factors that were not accounted for included poor construction practices that exacerbate the amount of damage seen but were not always mentioned in the structural data, and the density of the fill material, which is not currently measured as part of the standard testing process.

The natural variation of the material, was considered in a case where samples were taken on a grid system from a property undergoing remediation. The variation in chemical values obtained from these samples was compared to the variations in floor level at the property. Again this showed that the natural variation in the material is significant, and it is difficult on a point by point basis to compare the expansion to the chemical properties. Rather it is the behaviour of the fill material across the whole of the foundation that contributes to the uplift. For example, oxidation of pyrite in one location may lead to precipitation of gypsum at that location, but also in other locations close by, depending upon movement of moisture through the fill.

With the initial use of IS 398-1, the influence of these factors was sufficient that there was less correlation than would be expected between the level of damage and certain chemical factors that are known to directly affect the amount of heave. It is to that end, that the author believes that further refinement of the system, to account for these factors, would be worthwhile. This refinement may be via more advanced data analysis, or by statistical modelling.

The swell tests carried out at Sheffield as a part of this work confirmed work by Galway University, Ireland that shows the link between density and reaction rate, and that shows the reaction can proceed with no outside water source.

Additionally, the Sheffield tests confirmed that the presence of an increased fine content of the fill material led to an initial period of increased reaction rate that slowed once the pyrite in the fines content had been used up. Accordingly, the material with an increased coarse content was slower to react, although the reaction process continued for much longer than that with the increased fines content. Therefore, in the typical situation wherein well graded material is used as fill, the fine content of this material will fuel the initial reaction process, with the coarser material sustaining the reaction in the long term.

Removal of the water from the base of the Sheffield swell tests was intended to consider the way in which the reaction would proceed under those conditions – it was expected that the reaction would continue, given the evidence from the Galway tests and from the control test at Sheffield. However, once the water was removed, the reactions in all

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of the tests, including the control sample, slowed dramatically. This was determined to be caused by the decrease in the humidity of the environment around the samples.

The laboratory is air conditioned in order to maintain a temperature of around 20°C, however, this reduces the humidity of the laboratory down to around 36%. The presence of the water in the tank below the samples had increased the humidity around the samples such that the reaction process would continue in those tests without direct access to the water. With the tank emptied, the humidity around the samples returned to that of the laboratory as a whole and the samples dried out leading to the slowing of the reaction process. The tests at Galway that had reacted without an external water supply were not in a climate controlled environment, and so the natural humidity was higher, allowing the reaction to continue. This is a potential cause for the difference in reaction periods between Canada and Ireland, with Ireland having a higher relative humidity and pyrite cases that show a faster reaction rate.

Thus the major findings of this work can be summarised as follows:

- Consideration of the literature and historical case studies presents a single reference for future use. This includes details of those factors known to have a direct influence on the amount/rate of reaction and those with less or no influence, and a comparison of reaction rates from across the world.
- Development of an in situ monitoring system using an Arduino<sup>TM</sup> base to allow for flexibility. This is currently monitoring the temperature and moisture conditions below the ground floor slab of a property undergoing pyritic heave. Initial data shows variation in the fill temperature influenced by seasonal variations and internal temperature changes.
- The application of IS 398-1 to compare damage levels to chemical properties showed less correlation than expected. This is believed to be caused by factors not being fully accounted for in the data, such as structural defects, and by natural variations in the fill material. It is hoped that this can be modified by the use of more detailed analysis or by statistical modelling.
- Swell tests carried out at Sheffield confirmed the following:
  - Density has a direct influence on the amount of expansion, with a higher compaction density leading to more expansion of the material. This is due to the little available void space being filled quickly, leading to precipitation of gypsum in places likely to cause expansion of the material.
  - Grading of the material also has a direct effect on the amount and rate of expansion. The fine material reacts quickest, as it allows the best access of air and water to the pyrite, however, this pyrite will also be used up quicker. Conversely, the pyrite in the coarse material is harder to access, requiring some breakdown of the rock particles to allow access of air and water, meaning that this pyrite will react slower but over a longer period of time. Thus the practice

of using well graded material means that the reaction will be sustained over a period of time until the pyrite is consumed.

- Although tests at Sheffield and Galway show that the material can react using only the natural moisture content, i.e. not requiring an external water supply, tests at Sheffield show that the local humidity of the testing environment must also be accounted for. The presence of the water tank raised the local humidity around the tests and allowed the material to behave more like it would in an in situ environment. Upon removal of the external water supply, there was a decrease in expansion rates of all of the tests as the material returned to the low humidity of the climate controlled laboratory.

### 9.3 Further work

Work is currently ongoing at the University of Galway with a scale model of a foundation and floor slab in order to better understand how the foundation, fill and slab interact during the expansion process. With the method of expansion giving such a characteristic form of damage – the stellate cracking seen in cases both in Ireland and abroad – understanding this interaction would give a better understanding of the process as a whole.

It is the author's opinion that numerical modelling would be the best way to achieve this understanding. In addition to modelling the slab and fill properties, of particular interest is the interaction of the slab with both the fill below and the foundations walls along its edges. It is thought that the friction present between the edges of the slab and the foundation walls leads to the typical crack patterns seen in pyrite cases, although this is unproven at present.

A better understanding of how the expansion of the fill translates to the uplift to the slab; what restrains this movement and what aids it, will not only give a better understanding of the damage process, but will also aid the potential development of other remediation methods.

In addition to the interaction of the fill with the foundation and the floor slab, it would be of interest to determine whether the type of structure of the dwelling itself has any effect upon the way the damage manifests. Data is available across several developments that contain different types of structure, allowing for such an analysis to be completed from available data.

With the introduction of IS 398-1, and with the large amount of data currently being recorded at properties in Ireland, it is anticipated that future datasets should be more complete than the one considered in Chapter 5. With those datasets, and a record of the other structural defects present at the properties, it is likely that it will be possible to better refine the analysis method in order to modify the weightings of the various



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categories. This would ideally remove the influence of factors such as structural defects, and so improve the correlation between certain factors and the damage rating.

It is also anticipated that the application of more robust statistical model, and the consideration of the interaction of more than two variables at a given time, would present a better analysis of the available information. Although that was outside the scope of this study, it is hoped that this will be possible as a part of any future work on the subject.

It is hoped that it will, in future, be possible to install the Arduino system into more properties. Ideally this would be co-ordinated so as to have overlapping datasets that cover a full year so as to consider the influence of human habitation on the environment under the ground floor slab, as well as measuring responses to changes in climatic conditions and water table effects. This installation would involve increasing the number of sensors, with a view to comparing the internal and external temperatures, as well as considering how water is introduced into the sub-floor environment.

It would also be beneficial to consider detailed methods by which the system can be adapted for remote monitoring and be provided with its own power supply. This system would then be completely self sufficient, and could be used to measure any number of environments both inside and outside.

The initial modifications to the swell tests proved that the low natural humidity within the laboratory was enough to significantly decrease the rate of expansion. In order to confirm this, it is planned to selectively re-introduce water to some of the tests and allow the others to rest within the humid environment created in the area around the water tank. In order to determine the extent of the changes in the humidity within the immediate vicinity of the water tank, and so around the swell tests, humidity sensors will be used to monitor the changes over the time during which the water is added and progressing from that point.

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# Appendix A

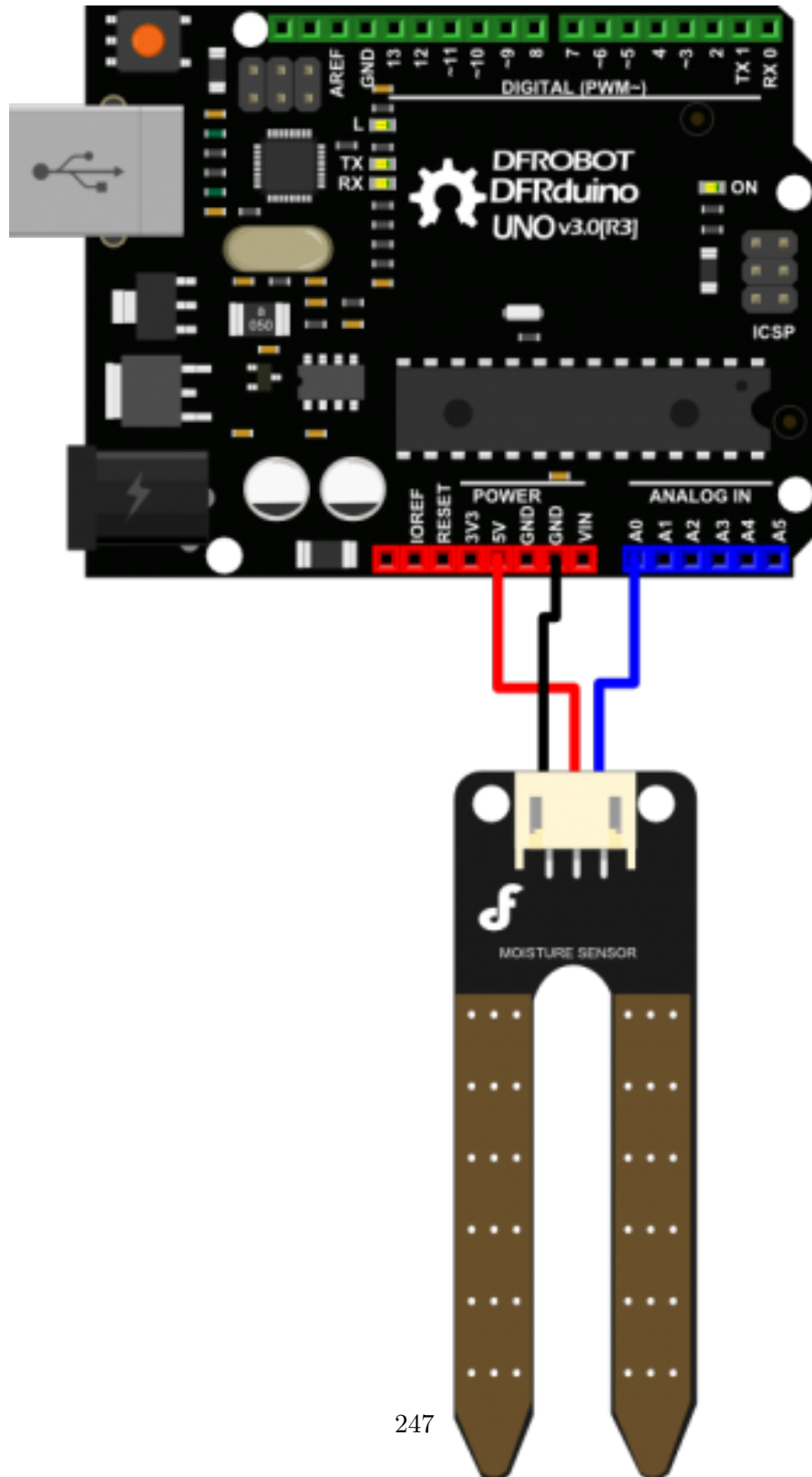


Figure 1: Circuit layout for the dielectric sensors used for the Arduino in Chapter 6 (SKU:SEN0114).

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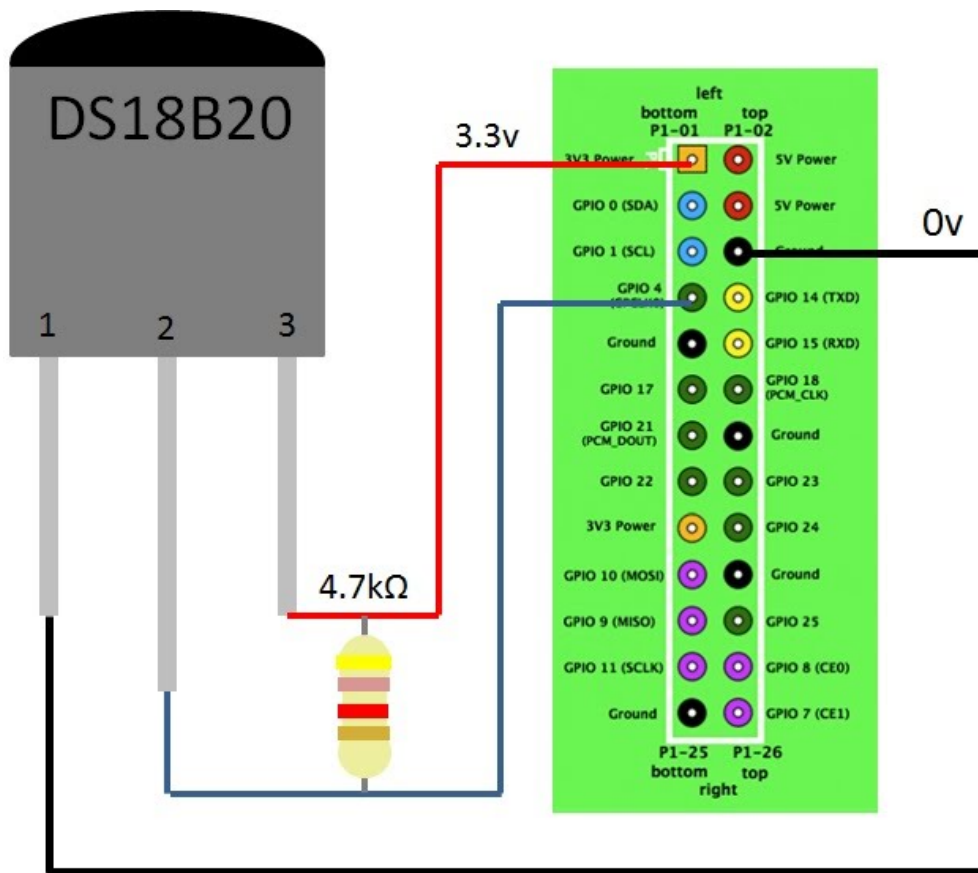


Figure 2: Circuit layout for the temperature sensors used for the Arduino in Chapter 6 (DS18B20).



## Appendix A

```
# Version      : 1.0
# Connect the sensor to the A0(Analog 0) pin on the Arduino board

# the sensor value description
# 0 ~300      dry soil
# 300~700     humid soil
# 700~950     in water
*/

#include <SD.h>
#include <Wire.h>
#include "RTClib.h"

// how many milliseconds between grabbing data and logging it. 1000 ms is once a
second
#define LOG_INTERVAL 5000 // mills between entries (reduce to take more/faster
data)

// how many milliseconds before writing the logged data permanently to disk
// set it to the LOG_INTERVAL to write each time (safest)
// set it to 10*LOG_INTERVAL to write all data every 10 datareads, you could lose
up to
// the last 10 reads if power is lost but it uses less power and is much faster!
#define SYNC_INTERVAL 5000 // mills between calls to flush() - to write data to
the card
uint32_t syncTime = 0; // time of last sync()

uint32_t LogTime = 0; // time of this log
uint32_t NextLogTime = 0; // time for the next log

#define ECHO_TO_SERIAL 1 // echo data to serial port
#define WAIT_TO_START 0 // Wait for serial input in setup()

// the digital pins that connect to the LEDs
// #define redLEDPin 8
// #define greenLEDPin 9

#define humPin0 0 // analog 0
#define humPin1 1 // analog 1
#define humPin2 2 // analog 2

RTC_DS1307 RTC; // define the Real Time Clock object
```

Figure 3: Code for the for the Arduino setup detailed in Chapter 6 part 1/5.

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```
// for the data logging shield, we use digital pin 10 for the SD cs line
const int chipSelect = 10;

// the logging file
File logfile;

void error(char *str)
{
  Serial.print("error: ");
  Serial.println(str);

  while(1);
}

void setup(void)
{
  Serial.begin(57600);

#if WAIT_TO_START
  Serial.println("Type any character to start");
  while (!Serial.available());
#endif //WAIT_TO_START

  // initialize the SD card
  Serial.print("Initializing SD card...");
  // make sure that the default chip select pin is set to
  // output, even if you don't use it:
  pinMode(10, OUTPUT);

  // see if the card is present and can be initialized:
  if (!SD.begin(chipSelect)) {
    error("Card failed, or not present");
  }
  Serial.println("card initialized.");

  // create a new file
  char filename[] = "LOGGER00.CSV";
  for (uint8_t i = 0; i < 100; i++) {
    filename[6] = i/10 + '0';
    filename[7] = i%10 + '0';
    if (!SD.exists(filename)) {
      // only open a new file if it doesn't exist
      logfile = SD.open(filename, FILE_WRITE);
      break; // leave the loop!
    }
  }
}
```

Figure 4: Code for the for the Arduino setup detailed in Chapter 6 part 2/5.

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```
if (! logfile) {
    error("couldnt create file");
}

Serial.print("Logging to: ");
Serial.println(filename);

// connect to RTC
Wire.begin();
if (!RTC.begin()) {
    logfile.println("RTC failed");
#ifdef ECHO_TO_SERIAL
    Serial.println("RTC failed");
#endif //ECHO_TO_SERIAL
}
// RTC.adjust(DateTime(__DATE__, __TIME__));

logfile.println("timestamp/ms,datetime,H1,H2,H3");
#ifdef ECHO_TO_SERIAL
Serial.println("timestamp/ms,datetime,H1,H2,H3");
#endif //ECHO_TO_SERIAL

// If you want to set the aref to something other than 5v
//analogReference(EXTERNAL);
}

void loop(void)
{
    delay(50);

    DateTime now;

    // delay for the amount of time we want between readings
    //delay((LOG_INTERVAL -1) - (millis() % LOG_INTERVAL));

    LogTime = millis(); //get the millisecond time now

    if ((NextLogTime - LogTime) >= LOG_INTERVAL) // check the NextLogTime
    {
        NextLogTime = (LogTime + LOG_INTERVAL); //increment the NextLogTime variable
        // log milliseconds since starting
        uint32_t m = millis();
        logfile.print(m);           // milliseconds since start
        logfile.print(", ");
        #if ECHO_TO_SERIAL
            Serial.print(m);       // milliseconds since start
            Serial.print(", ");
        #endif
    }
}
```

Figure 5: Code for the for the Arduino setup detailed in Chapter 6 part 3/5.

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```
// fetch the time
now = RTC.now();
//log time
//logfile.print(now.unixtime()); // seconds since 1/1/1970
//logfile.print(", ");
//logfile.print('');
logfile.print(now.year(), DEC);
logfile.print("/");
logfile.print(now.month(), DEC);
logfile.print("/");
logfile.print(now.day(), DEC);
logfile.print(" ");
logfile.print(now.hour(), DEC);
logfile.print(":");
logfile.print(now.minute(), DEC);
logfile.print(":");
logfile.print(now.second(), DEC);
//logfile.print('');
#if ECHO_TO_SERIAL
//Serial.print(now.unixtime()); // seconds since 1/1/1970
//Serial.print(", ");
//Serial.print('');
Serial.print(now.year(), DEC);
Serial.print("/");
Serial.print(now.month(), DEC);
Serial.print("/");
Serial.print(now.day(), DEC);
Serial.print(" ");
Serial.print(now.hour(), DEC);
Serial.print(":");
Serial.print(now.minute(), DEC);
Serial.print(":");
Serial.print(now.second(), DEC);
//Serial.print('');
#endif //ECHO_TO_SERIAL

// Reading temperature or humidity takes about 250 milliseconds!
// Sensor readings may also be up to 2 seconds 'old' (its a very slow sensor)

int humReading0 = analogRead(humPin0);
int humReading1 = analogRead(humPin1);
int humReading2 = analogRead(humPin2);

logfile.print(", ");
logfile.print(humReading0);
logfile.print(", ");
logfile.print(humReading1);
logfile.print(", ");
```

Figure 6: Code for the for the Arduino setup detailed in Chapter 6 part 4/5.

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```
logfile.print(humReading2);

#if ECHO_TO_SERIAL
  Serial.print(", ");
  Serial.print(humReading0);
  Serial.print(", ");
  Serial.print(humReading1);
  Serial.print(", ");
  Serial.print(humReading2);

#endif //ECHO_TO_SERIAL

logfile.println();
#if ECHO_TO_SERIAL
  Serial.println();
#endif // ECHO_TO_SERIAL

  // Now we write data to disk! Don't sync too often - requires 2048 bytes of
I/O to SD card
  // which uses a bunch of power and takes time
  if ((millis() - syncTime) < SYNC_INTERVAL) return;
  syncTime = millis();

  // blink the red LED to show we are syncing data to the card & updating FAT!
  logfile.flush();
  delay(50);

}
else
{
  delay(1000); //if not taking a reading then wait 500ms so we get a nice
flash
}
}
```

Figure 7: Code for the for the Arduino setup detailed in Chapter 6 part 5/5.