The Canadian Pyrite Experience and Comparisons with the Irish Problems

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Introduction

The phenomenon of swelling shale bedrock affecting building performance was first recognized in Canada in the late 1960s. The therapy treatment building at the Rideau Health and Occupation Centre (later known as the Rideau Veteran's Home) on Smyth Street in Ottawa experienced heave in one section of the building (Quigley and Vogan 1969). The portion of the building that developed a problem was a two-storey section with a service tunnel and no basement. This portion of the building was adjacent to a building section that had a basement area and swimming pool (Fig. 1).

The building was founded in the Lorraine Shales that overlie the Billings Shale in the area. Because the service tunnel and basement were at a lower elevation, the ground water was lowered, which allowed the unexcavated shale beneath the ground floor in the two-storey section of the building to drain and facilitated the oxidation of the fine grained pyrite within the shale. The resulting heave occurred in the ground floor slab as well as in the second floor slab which was also supported on column footings within the Lorraine Shale.

Another well-published early case study is the Bell Canada Building on Albert Street in downtown Ottawa. This building, which was originally built in 1929, had an addition constructed in 1961. The first problems were noted in the basement floor (Penner et al. 1970) in the form of two rounded domes. Based on heave monitoring commenced in 1967, it was estimated by Penner et al. that the average rate of heave was 22 mm per year and the maximum total heave recorded was 120 mm. The Bell Canada Building is still functioning today (Fig. 2) after remedial works were undertaken. The Rideau Veteran's Home was demolished in 1999.

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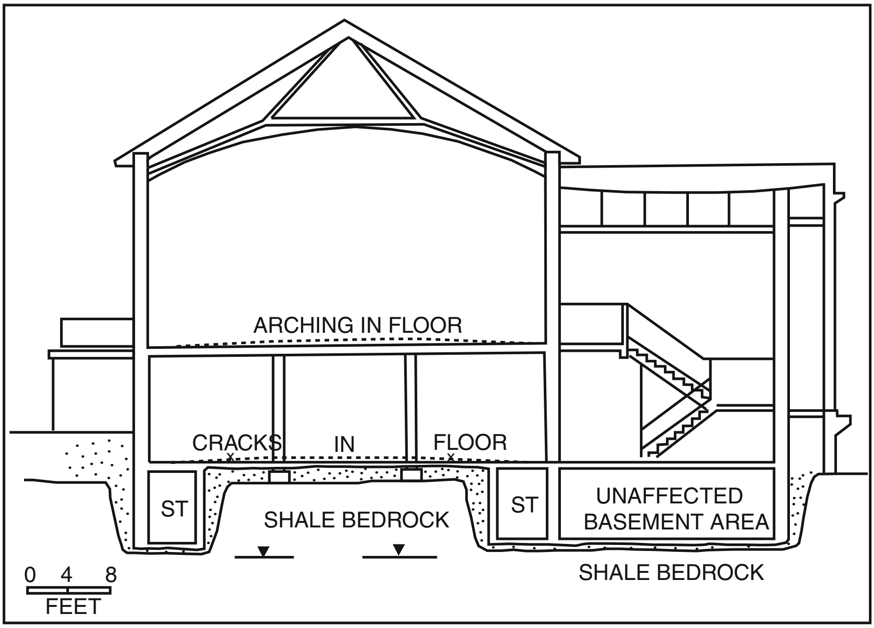

Fig. 1 Cross-section of Rideau Veteran's Home illustrating heave in the ground and second floor slabs (from Quigley and Vogan 1969)

The first case of suspected pyrite-induced floor heave identified in Ireland was in late 2006. The second recorded case was a house in a housing estate in north Dublin which began construction in 2003 (Fig. 3). In March 2007 it was confirmed that the cause of the cracked and heaved floors and cracked partition walls reported in the house was swelling of the underlying crushed rock fill beneath the floor slabs. The damage was confined to the floors and internal partition walls with no visible damage on the exterior of the house. Testing on the fill material confirmed it to be a crushed calcareous mudstone (Fig. 4), with estimated original pyrite concentrations of about 2.6 %. The Petrographic Number (CSA A23.2-15A, 2004 and ASTM C295, 2011) for a sample of the underfloor aggregate was found to be 250, confirming very low physical–mechanical quality.

Geological Setting

Canada

The first identification of pyrite-related problems in buildings in Canada was in the Ottawa area in the 1960s. Subsequently, in the 1970s similar problems were recorded in Sainte-Foy near Québec (Bérubé et al. 1986). By the mid-1980s,

Fig. 2 Bell Canada Building, Albert Street, Ottawa

widespread pyrite-related problems, on the scale of an epidemic, were reported in the Montréal area. As can be seen from Fig. 5, these areas trend in a northeasterly direction parallel to the St. Lawrence River.

When the geological mapping for these areas is examined (Fig. 6), it is seen that the affected areas are characterized by shale deposits. This is consistent with the published literature on pyrite-related problems around the world where they are associated with areas of shale and mudstone, i.e., fine grained sedimentary rocks with significant clay contents.

It should be noted that pyrite-related problems in buildings have come about in two different ways. The first is due to heave of building foundations and floor slabs supported on shale bedrock, and the second is where crushed shale or mudstone has been used as compacted underfloor fill or backfill against foundation walls.

In the Ottawa area, the heave-related problems have been mainly associated with the Billings Formation of Ordovician age. As seen from the schematic in Fig. 7, the Billings Formation is associated with the Carlsbad and Queenston Formations which are also shales.

Fig. 3 The second reported house in Ireland with pyrite-induced floor heave, Dublin

Fig. 4 Trial pit through kitchen floor exposing problematic underfloor fill

Fig. 5 Key plan showing Ottawa, Montréal and Québec City

The Billings Shales are typically described as dark grey, black and brown shales, interbedded with calcareous siltstone and silty limestone of the Carlsbad Formation. They are also frequently interbedded with the red to greenish grey siltstone and shale of the Queenston Formation. These shales were formed in an anoxic marine environment some 450 million years ago. Typical of these rocks originating from fine marine muds, they generally contain widely disseminated fine grained pyrite.

Along the St. Lawrence River, between Montréal and Québec City, are a series of shale deposits comprising Utica Shales and Nicolet and Pontgrave Formation

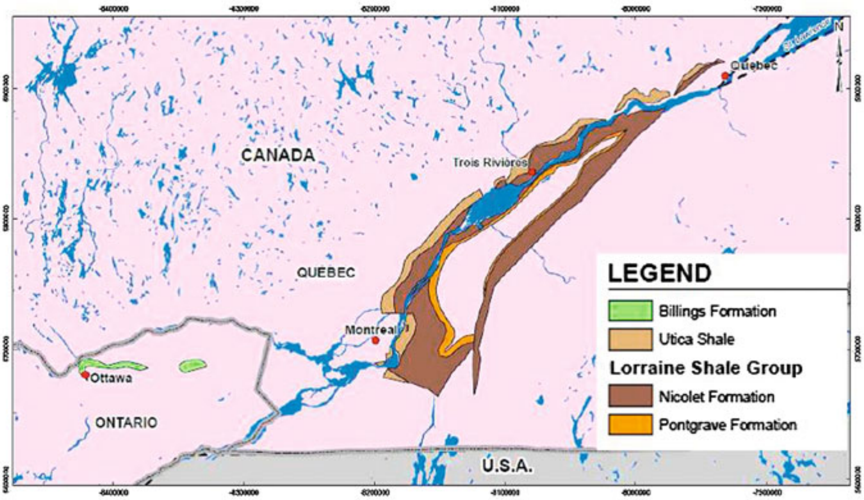

Fig. 6 Areas of shale bedrock in areas reporting pyrite-related problems (from MNR NRVIS, 2004, CANMAP 2006 & Ontario Ministry of Natural Resources, 2008)

Fig. 7 Ordovician sequence in the Ottawa area

PERIOD	GROUP	FORMATION
ORDOVICIAN	QUEENSTON	QUEENSTON
	CARLSBAD	CARLSBAD
	BILLINGS	BILLINGS
	OTTAWA	Lindsay
		Verulam
		Bobcaygeon
		Gull River
		Shadow Lake
	ROCKLIFFE	ROCKLIFFE
	BEEKMANTOWN	Oxford
		March
CAMBRIAN	POTSDAM	Nepean
PRECAMBRIAN		

shales of the Lorraine Shale Group. The relationship between these shales in terms of depositional history is shown in Fig. 8.

The Utica Shales are 50–300 m thick and were deposited during a period of rapid sea level rise in a poorly oxygenated environment. They comprise siliciclastic and carbonate muds with moderate to high calcite contents. Because of this calcite content, many of these rocks could be classified as silty limestones and so were frequently used in the 1970s, 1980s and 1990s as processed crushed rock in residential and commercial construction.

The overlying Lorraine Shales are more argillaceous and with lower calcite contents, and comprise mudstones and siltstones as well as true shales. The Lorraine Group shales also have high concentrations of disseminated pyrite.

Ireland

At least four quarries have been identified as supplying aggregate that has given rise to pyrite-induced heave when used as crushed aggregate beneath floor slabs in

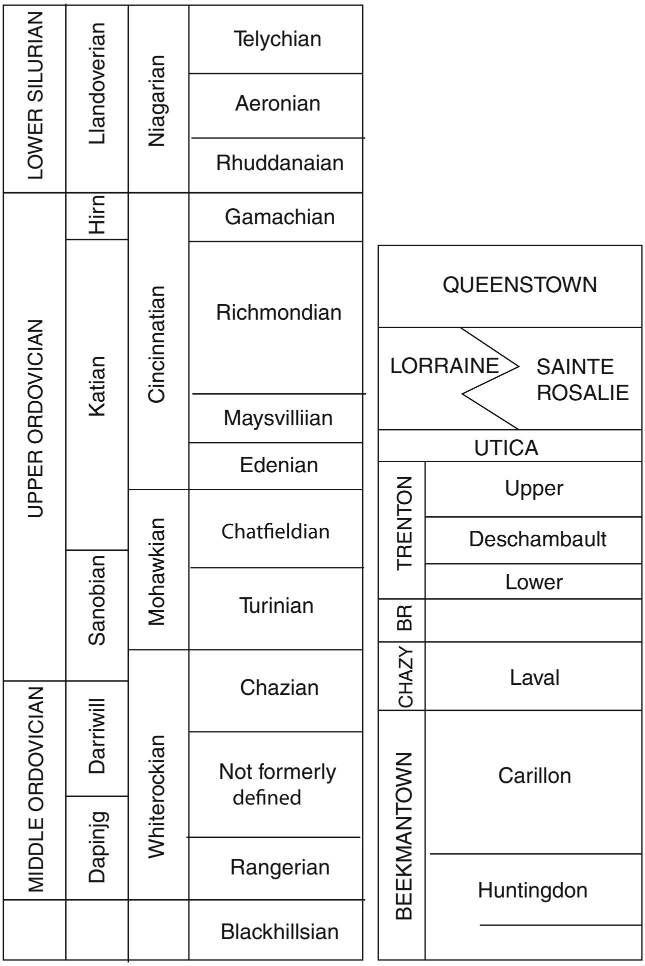

Fig. 8 Geological sequence of the shale formations of Upper Ordovician age along the St. Lawrence River (from Lavoie et al. 2011)

Ireland. Because of ongoing legal actions, these quarries are identified as Quarry B (North Dublin), Quarry M (North Dublin), Quarry R (Co. Meath) and Quarry T (Co. Meath).

The quarry that supplied the largest quantity of crushed rock to the booming development market between 2003 and 2008 was Quarry B. This quarry, located just west of Dublin Airport, obtained planning permission to operate a commercial quarry in early 2003. For the five year period between 2003 and 2008, this quarry produced just over 2 million tonnes of crushed aggregate that was sold as unbound material for residential and commercial construction, generally in the northern part of Dublin. While it was assumed that the quarry could supply aggregate conforming to Clause 804 (NRA 2000), i.e., the National Roads Authority granular road base material, no contemporary independent sampling and test results have been identified that verify this conformance.

Extensive testing carried out on samples of the material recovered from buildings during remedial works demonstrate that the aggregate from Quarry B failed to meet the Liquid Limit, Coarse Aggregate Water Absorption, Magnesium Sulphate Soundness and Ten Percent Fines Value for Clause 804. This is not surprising when it is considered that the quarry is located within the Tober Colleen Formation that is described as dark grey, calcareous, commonly biorturbated mudstones and subordinate thin micritic limestones (GSI 2001). The Tober Colleen Formation is of Lower Carboniferous age and is the lowest unit within the Calp Limestone. Quarry M, located further north of Quarry B, is within the Loughshinny Formation. This rock is described as argillaceous, pyritic, locally cherty micrites and graded calcarenites, interbedded with dark grey to black shale (GSI 2001).

The two Co. Meath quarries that have supplied problematic pyritic aggregates are from the Lucan Formation and are described as dark grey, well bedded, cherty, graded limestones and calcareous shales. The geological succession of these various formations is illustrated in Fig. 9 and examples of some of the materials are shown in Figs. 10 and 11.

Thus the questionable quality of these rock formations for the production of construction aggregates was documented; however, this information was either not consulted or ignored when these quarries were developed. It is worth noting that these quarries did not supply aggregates to major road construction projects and hence no comprehensive independent testing of aggregate was undertaken.

Case Studies

Case Study 1, Commercial Warehouse, Ottawa

This case study relates to a two-storey warehouse building in the City of Ottawa. The building was constructed in the 1960s and damage in the form of floor cracking and unevenness was first noted in the 1980s. There was a relatively high water table at the site so a sump and pump had been installed some 3.6 m below the floor slab. In addition, there was an elevator shaft that extended approximately 2 m below the floor slab. Damage was first noted in the area of the building near the sump (refer to Figs. 12 and 13). Over the intervening years, the floor heave continued to progress and was estimated to be approaching 100 mm in total heave displacement. Some temporary repairs were undertaken but eventually more extensive rehabilitation was deemed necessary as the operations of the fork lift trucks were being affected.

Golder Associates were retained to undertake an investigation of the extent of the problem and to advise on remedial measures. The scope of the investigation comprised a desk top study, one floor cut-out and one corehole to a depth of 3 m.

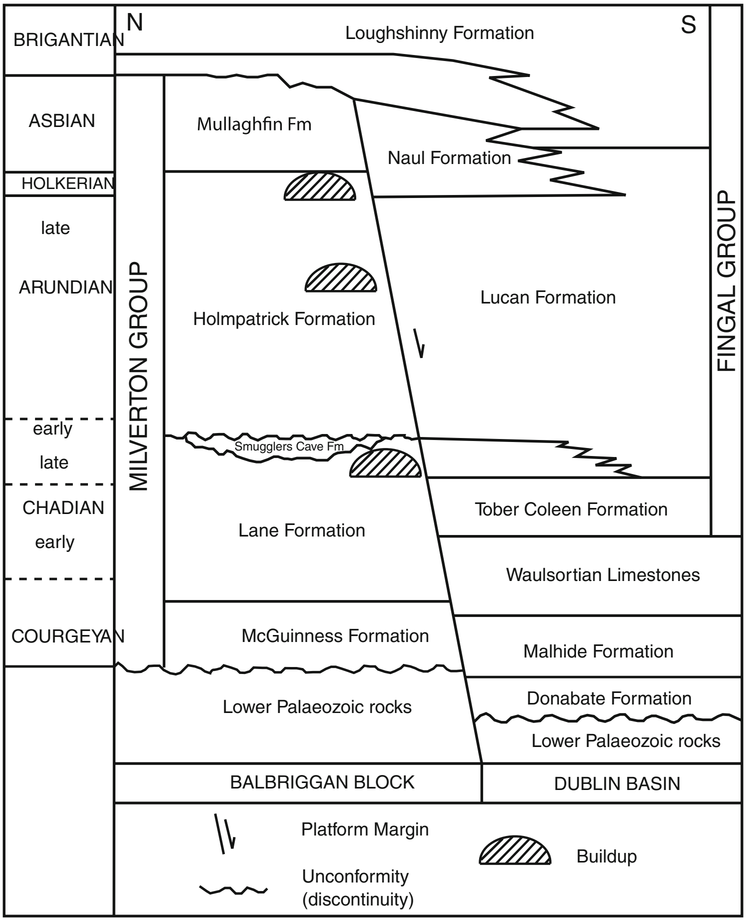

Fig. 9 The Dinantian successions in north Co. Dublin (from GSI 2001)

The investigation confirmed (Figs. 14 and 15) that only a thin layer (<100 mm) of compacted granular base separated the underside of the floor slab from the underlying bedrock surface. The lowering of the local water table in the vicinity of the sump had allowed the pyrite in the upper portions of the shale bedrock to oxidize and for gypsum to develop between laminations (Fig. 16). It could be clearly seen from visual examination that the laminations within the shale were being prised apart by the growth of gypsum.

The remedial options involve removal of affected areas of the floor slab, excavation to remove the upper weathered areas of shale and sealing the surface, typically using shotcrete. New granular fill is then installed and the floor slab reinstated.

Fig. 10 Particle from Quarry B opened on a lamination reveals gypsum clusters

Fig. 11 Aggregate particle from Quarry T with laminations propped open with gypsum

Fig. 12 Intersecting floor cracks indicative of upward heave of the slab

Fig. 13 Cracking in internal block wall supported on floor slab

Fig. 14 Cut-out through floor slab

Case Study 2, Extension to the Light Rail Transit, Ottawa

The City of Ottawa has a well established public transit system but it needs to be upgraded to cater for growing ridership. The upgrade will involve the construction of Ottawa's Light Rail Transit (OLRT) system. This will require converting a portion of the existing Bus Rapid Transit (BRT) system into an LRT system. In addition to upgrades at some existing stations and construction of new stations, the project will

Fig. 15 Chunks of shale bedrock extracted from beneath floor slab

Fig. 16 Close-up views of chunk of brown Billings Shale with clusters of gypsum crystals on lamination surface

also include a Downtown Area Underground Section. The Downtown Area Underground Section will be approximately 2.5 km long and will include three new stations (Fig. 17). Construction of the OLRT is scheduled to commence in 2013.

A geotechnical investigation was carried out to characterize the subsurface (overburden and rock) conditions along the proposed alignment (Golder 2011). Shale of the Billings Formation was encountered at a number of locations, including at the Downtown East Station and at the Maintenance Yard. The Billings Shale comprised fresh, thinly laminated to laminated, dark brown to black, and weak to medium strong shale with grey limestone seams. Testing on samples of the recovered rock core (Golder 2011) yielded results for the Billings Shale as summarized in Table 1.

Shale formations in Southern Ontario and in other areas are known to exhibit swelling behaviour upon stress relief and when in contact with fresh water (Lo and Micic 2010). Since very limited data on the swelling potential of the Billings Shale is available, a preliminary laboratory testing program was undertaken to determine the potential swelling characteristics of this rock formation.

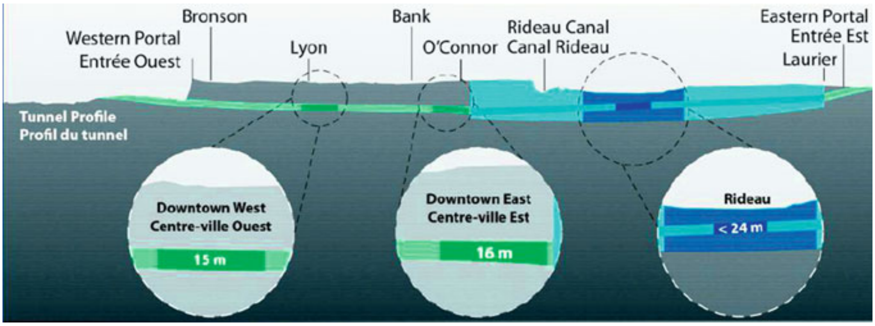

Fig. 17 Proposed tunnel section of extension to OLRT (from OLR 2012)

Table 1 Test results on recovered core from Billings Shale (from Golder 2011)

Test	Test standard	Average	Range
Unconfined compressive strength (MPa)	D 2938	22	17–26
Density (Mg/m^3)		2.62	2.50–2.78
Point load index—axial (MPa)	D 5731	5	2–11
Point load index -diametral (MPa)	D 5731	2	0–4
Mohs hardness		2.5	2.5–2.5
Slake durability index	D 4644	98.8	98.0–99.2
Calcite content		2.6	2.0–3.5

The swell testing was carried out at the University of Western Ontario by K.Y. Lo Inc. The core tested came from about 20 m deep and was subjected to three different test procedures. These specialized tests had been developed by Prof. K.Y. Lo, University of Western Ontario, London (Lo et al. 1978). The tests are the Free Swell Test (FST), the Semi-Confined Swell Test (SCST) and the Null Swell Test (NST); Fig. 18. Each of these tests is run for 100 days and during the tests the samples are immersed in fresh water.

During the course of the tests the following parameters are checked; water content, calcite content, salinity and pore water salinity. In general, the salinity of the rock samples tested was fairly uniform, ranging from 1.67 to 2.55 mg/g. The water content was generally low, ranging from 1.6 to 3.0 %. The calcite content of the test samples was also generally low, ranging from 2.0 to 3.5 %. The salinity of the pore water was generally high, ranging from about 69 to 120 g/L (note: salinity of sea water is around 35 g/L). The high salinity of the pore water is consistent with the marine depositional environment of the Shale and is also consistent with the range of pore water salinity of other shales that exhibit swelling behaviour.

The data obtained from the FSTs and the SCSTs are analysed by plotting the swelling strain versus the logarithm (to the base of 10) of elapsed time (in days). After the initial stabilization period, the swelling strain versus logarithm of time plot approximates a linear relationship. Following the method used by Lo et al. (1978), the average slope of such a plot (taking the approximately linear portion of

Fig. 18 Null swell test apparatus at the University of Western Ontario

the graph) gives an indication of the tendency of the rock to swell. The amount of swelling strain measured within one log cycle of time (generally between 10 and 100 days or extrapolated for such period) is taken as an index of the "swelling potential" of the rock being tested. The swelling potential for the samples of Billings Shale tested (based on the FSTs) ranges from 0.31 to 0.35 % in the vertical direction (z direction), and is higher than those in the horizontal directions (x and y directions) which range from 0.18 to 0.23 %.

Figure 19 shows the relationship between the swelling potential and calcite content of various shale formations in Canada (Lo and Micic 2010). Data from the Billings Shale are also included in the same plot, for comparison. The swelling potential of the shales increases with decreasing calcite contents. It may be noted that the calcite contents of the Billings Shale are within the range of calcite contents in other shales which exhibit swelling behaviour. The swelling potentials of Billings Shale are also consistent with those of other shales with comparable calcite contents.

The stabilized pressure achieved during the NSTs is termed as the "suppression pressure" at which the swelling of the test sample is fully suppressed. The "suppression pressure" in the horizontal direction was between 0.22 and 0.28 MPa, which was much lower than that in the vertical direction which ranged from 0.96 to 1.08 MPa.

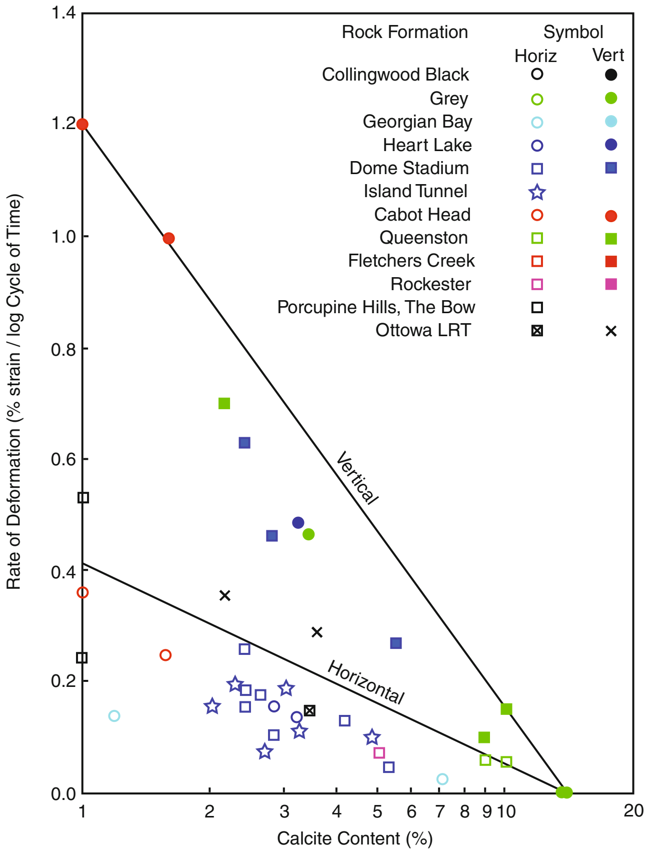

Fig. 19 Relationship between swelling potential and calcite content of various shale formations in Canada (from Lo and Micic 2010)

The swelling tests on Billings Shale confirmed that all the test samples underwent swelling when immersed in fresh water and that this form of swelling could be suppressed under an applied "suppression pressure". Based on the results of swelling tests on various shales in Southern Ontario and North America, Lo and Micic (2010) postulated that the mechanism of this form of swelling is a result of the dilution of pore water salt concentrations which causes the space between the clay particles to expand. The osmosis and diffusion processes constitute the mechanism for the dilution of the salt concentration in the pore fluid if the rock is surrounded by an ambient fluid of lower salt concentration.

In practice, the conditions for swelling to occur are (a) accessibility to fresh water (b) an outward gradient of salt concentration in the pore fluid to the ambient fluid of lower salt concentration, and (c) the relief of in situ stresses which serves as the initiating mechanism. Swelling is also affected by the interparticle bonding. If the bond strength is higher than the osmotic pressure, no swelling will occur and vice versa. The bond strength is dependent on the calcite content which acts as a cementing agent between the clay particles.

Based on the field conditions pertinent to the OLRT, the rock surrounding the underground structures, including the access and ventilation shafts, will have accessibility to fresh water as the structures will be located below the ground water table. The high salinity of the pore water fluid in the rock will generate a high outward gradient of salt concentration in the pore fluid to the ambient ground water that has a much lower salt concentration. In addition, in situ stress measurements carried out at the OLRT site indicate that high initial in situ stresses exist in the rock formations. As a result, stress relief in the rock formation will occur upon excavation. The low calcite content of the rock also constitutes weak bonding strength between the clay particles. Therefore, these field conditions provide the attributes for swelling to occur in the Billings Shale.

A study of the heaving of black shale in the Ottawa area by Quigley and Vogan (1969) indicated that the black shale (Billings Shale) in the Ottawa area comprises an abundance of illite and mica and minor pyrite (iron sulphide of between 0.7 and 3.9 %) in thin laminae and disseminated form. No significant amount of swelling clay minerals was noted. Heaving of these black shales will occur in areas above the water table where an oxidizing environment (drained, warm and humid) for the pyrite is present. As the proposed underground structures for the OLRT will be located permanently below the groundwater table during its operation and measures could be undertaken to seal the rock formation from oxidation during construction, heaving of the black shale due to the oxidation of the pyrite in the shale will not likely impact the design of the underground structures.

The swelling characteristics of the Billings Shale due to dilution of the salt concentration, as determined from the laboratory swell tests, will need to be taken into consideration in the design of the underground structures located within this rock formation. If an underground structure is constructed directly in contact with the rock, the time-dependent deformation (swelling) of the rock will cause pressure to build up with time at the rock-structure interface. The magnitude of the pressure will depend on the rigidity of the structure, the time schedule of construction, the swelling characteristics of the rock and the initial in situ stresses in the rock formation. These swelling pressures will be allowed for in design.

Case Study 3, Pyrrhotite/Pyrite in Concrete Aggregates, Trois-Rivières, Québec

In the early 2000s, 30 owners of relatively new homes in the Trois-Rivières area, 140 km east of Montréal (Fig. 5), began experiencing problems with cracking and deterioration of the exposed concrete in their basement walls (Roy 2011). Following investigation, it was established that the coarse aggregate used in the making of the concrete was an anorthositic gabbro that contained typically

Fig. 20 Typical damage in exposed house concrete, Trois-Rivières, Québec

between 5 and 10 % pyrrhotite, pyrite and chalcopyrite, all sulphides of iron. Domestic house construction typically uses concrete with compressive strengths of 20–25 MPa for foundation and basement wall construction. The source quarry for the coarse aggregate was a small local quarry that only operated for a few years.

Subsequently, in 2009, several new cases of early deterioration of concrete in houses were reported, also in the Trois-Rivières region (Fig. 20). The aggregate used was predominantly an anorthositic gabbro or norite containing calcic plagioclase feldspars, biotite, and pyroxene with various proportions of pyrite, chalcopyrite and pyrrhotite (Duchesne and Fournier 2011). The pyrite and pyrrhotite contents vary significantly and are reported to be as high as 5–7 %. The houses were generally 4–6 years old at the time damage occurred.

By 2011, the Garantie des Maisons Neuves (GMN), a government sponsored mandatory home warranty programme in the province of Québec, had received over 600 claims, with potential repair costs estimated at $65 million (Nantel 2011).

Internal sulphate attack on concrete has been well documented in the literature, although mainly relating to the oxidation of pyrite in aggregates of sedimentary origin. The 'mundic' problem in Cornwall, England is a well documented example of early deterioration of concrete blocks due to the presence of pyrite in the aggregate (Bromley 2000). In that region it became common to process local mine waste rock for use as aggregate in the production of concrete and concrete blocks used in residential and commercial construction (Lugg and Probert 1996).

The rocks in question were fine grained sedimentary rocks containing high levels of reactive sulphide minerals primarily in the form of pyrite (FeS_2), but also including chalcopyrite (CuFeS), arsenopyrite (FeAsS) and sphalerite (ZnS). The reactions generated from the presence of these minerals in the mine waste aggregate resulted in deterioration and cracking of concrete. The resulting damage to the houses was attributed to the oxidation of the sulphides in the presence of moisture and oxygen. The production of limonite, or more commonly goethite, and the formation of gypsum result in an expansion in solid volume with an associated

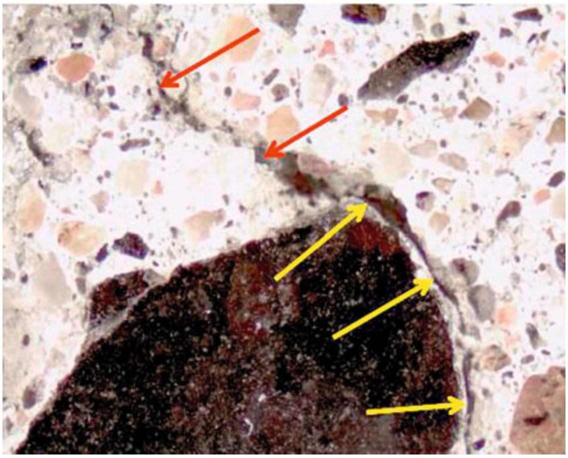

Fig. 21 Photomicrograph showing crack in concrete at coarse aggregate and paste interface

breaking of the bond between the aggregate and the cement paste and the propagation of cracks to accommodate the expansive products.

A paper published in 2005 (Lee et al. 2005) studied the premature deterioration of ten highway bridges in the State of Iowa due to secondary ettringite formation as a result of pyrite oxidation in the aggregates. Concrete degradation problems related to pyrrhotite were identified in Sweden in the 1940s (Hagerman and Roosaar 1955). Concrete damage was noted at the Norrforsen hydroelectric plant. Investigation attributed the damage to the presence of pyrrhotite in the crushed gneiss and granite aggregate. In the recent problems in Trois-Rivières, which came to light in 2009, the suspect aggregate is from a different (although nearby) source quarry from that which gave rise to similar problems in 2000. The aggregate in question was of extremely high quality in terms of physical properties. The oxidation of the pyrrhotite resulted in the formation of ettringite and thaumasite and the development, over time of extensive internal cracking in exposed concrete (Fig. 21). For example, Micro-Deval loss values were about 7 %. However, the Canadian Standards (CSA A23.1 and .2 1994) for concrete aggregate do not provide a limit for sulphur in aggregate for concrete. Since 1994, the Standard contained a warning that the use of aggregates that produce excessive expansion in concrete through cement-aggregate reaction other than alkali reactivity shall not be used for concrete unless preventive measures acceptable to the Owner are applied. In contrast, the European Standard (EN 12620 2002) suggests a limit on the total sulphur content of concrete aggregates to 0.1 % where the sulphide is in the form of pyrrhotite.

Some Characteristics of Problematic Shales and Mudstones

Shales and mudstones, in general, have always been acknowledged as not suited for the production of good quality construction aggregates. Due to their high clay contents, low strength and laminated structure, they tend to have high water

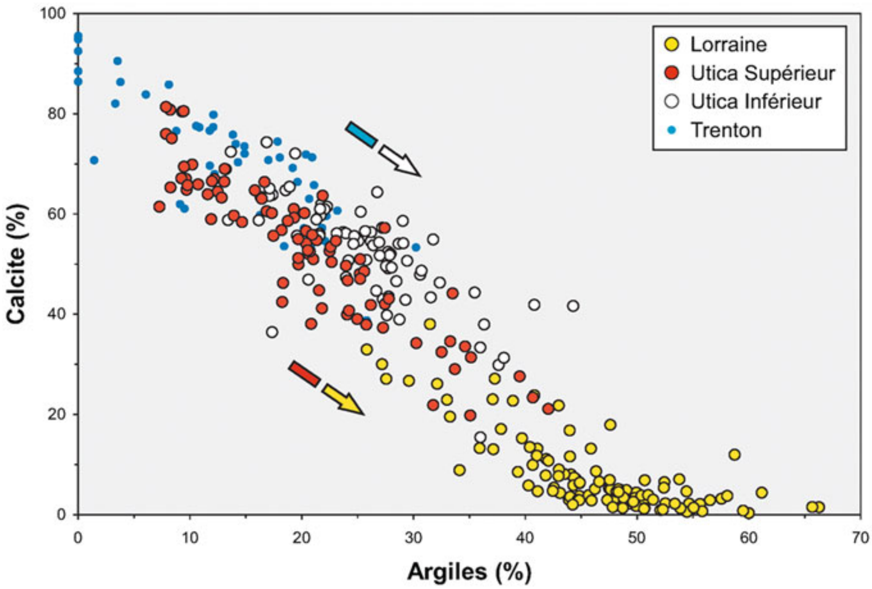

Fig. 22 Diagram showing calcite and clay (argiles) contents for Québec shales (from Thériault 2012)

absorption characteristics and low resistance to abrasion and impact. With these characteristics, these rock types are very vulnerable to chemical weathering. The easy access to moisture and the low internal bonds between laminations allow the very fine disseminated pyrite to readily oxidize. Since many of these shales may also contain small amounts of calcite, all the ingredients for the formation of calcium sulphate dihydrate, or gypsum, are present. It is generally accepted that the higher the clay contents in shales and mudstones, the poorer the quality of the rock in terms of durability and mechanical properties. It is for these reasons that the test standard developed in Québec (BNQ 2560-500 2003) to assess susceptibility of an aggregate to pyrite-induced heave, is the Swelling Index Test which is based on establishing the clay content of the sample.

A recent study of the shales of the St. Lawrence Lowlands of Québec (Thériault 2012) has compiled extensive mineralogical and other data on these materials. Figure 22 shows the relative calcite and clay (argiles) contents in Québec shales. It can be seen that the Lorraine Shales have very high clay contents and very low calcite contents. These shales are highly prone to pyrite-induced heave but because of their extremely poor quality were generally not used in construction.

On the other hand, the Utica Shales were used extensively as crushed rock fill throughout the 1970s and 1980s. These results span quite a range extending into lithologies with over 60 % calcite and less than 10 % clay. Generally rocks comprised of greater than 50 % calcite can be described as limestones, sometimes

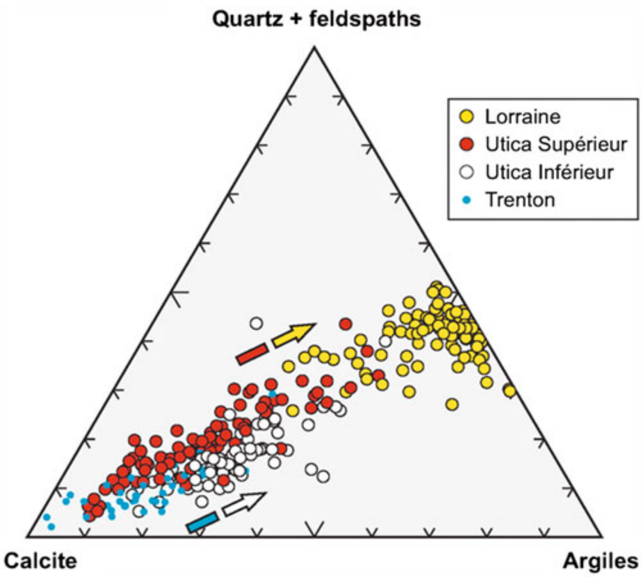

Fig. 23 Trilinear diagram showing composition of Québec shales in terms of quartz and feldspars (feldspaths) calcite and clay (from Thériault 2012)

with description modifiers of "muddy" or "silty". As reported in Table 1, the Billings Shale tested for the OLRT project had up to 3.5 % calcite content.

To allow comparisons with the Irish problematic mudstones, the data for the Utica and Lorraine Shales (Fig. 23) have been re-plotted with similar mineralogical data for the Irish mudstones. The combined results are shown on Fig. 24. It can be seen that Quarry B plots close to the centre of the triangle and with close to 30 % calcite in some instances. However, it also has generally greater than 20 % clay content and about 2.5 % pyrite content.

Figure 25 compares the estimated pyrite contents in some problematic mudstones and shales. However, as an indicator of the potential for pyrite-induced heave, clay content is likely better than pyrite content. This is the approach taken in Québec which is discussed later in this chapter.

In our experience with these problematic shales and mudstones, the pyrite content is not the controlling factor since there is typically more than sufficient sulphides available to sustain the reactions if the other enabling conditions are present.

The low quality of the aggregates prone to pyrite-induced heave can be confirmed from conventional aggregate suitability testing. The most indicative tests are coarse aggregate absorption/water absorption, Micro-Deval and Magnesium Sulphate Soundness. Figure 26 shows typical test results for these problematic aggregates with the red line indicating the test limits generally exceeded by these

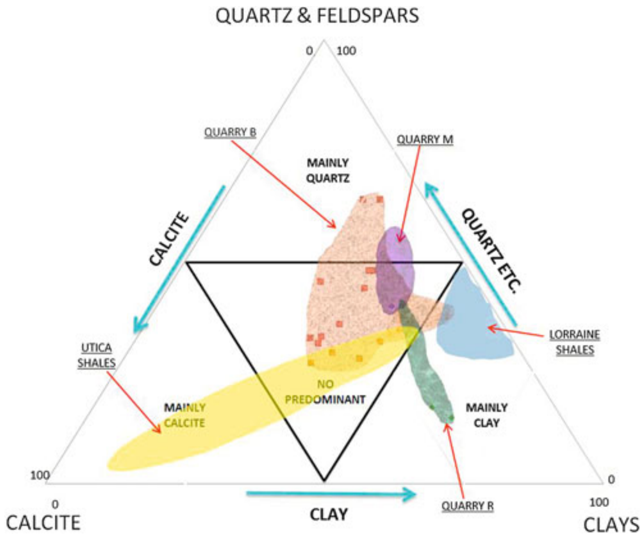

Fig. 24 Comparison between Utica and Lorraine Shales and Irish mudstones that cause pyrite-induced heave

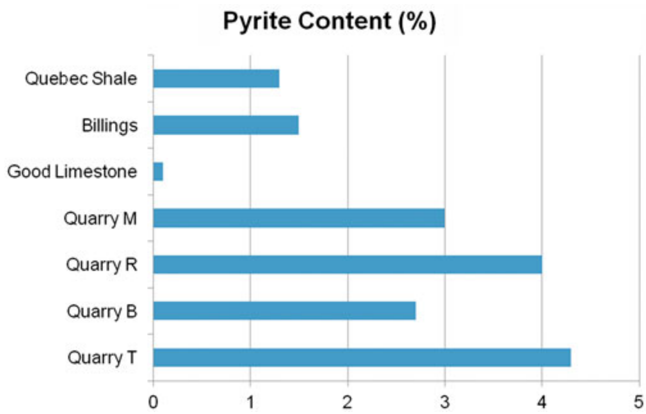

Fig. 25 Comparison of estimated pyrite contents in problematic mudstones and shales

materials. Physical testing undertaken as part of a research study for the Canada Mortgage and Housing Corporation (Bellaloui et al 2003) obtained Micro-Deval results for Québec shales ranging from 90 to 97 % and coarse aggregate water absorption ranging from 2.8 to 3.7 %.

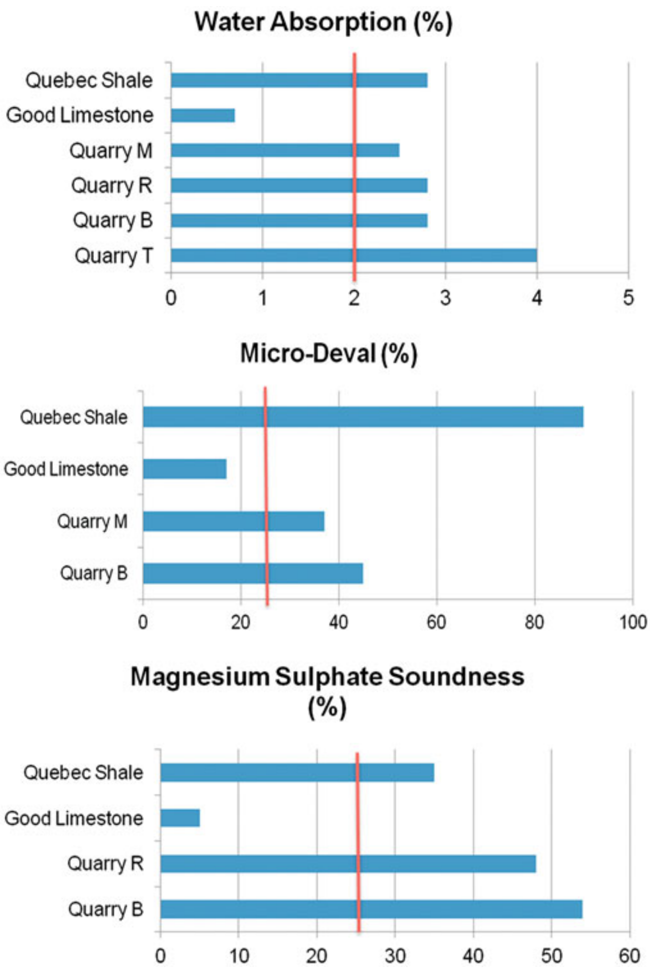

Fig. 26 Typical results for water absorption, micro-deval and magnesium sulphate soundness for problematic shales and mudstones. *Red line* indicates the values generally exceeded with these materials

Rates of Expansion

The rates of pyrite-induced expansion within buildings will vary significantly depending on a wide range of influence factors. Among others, these will include the following:

- Whether it is a foundation on shale bedrock or a floor slab placed on a compacted shale or mudstone fill;
- Quality of the source rock and proportion of mudstone/shale versus limestone;
- Distribution and form of pyrite within the aggregate;

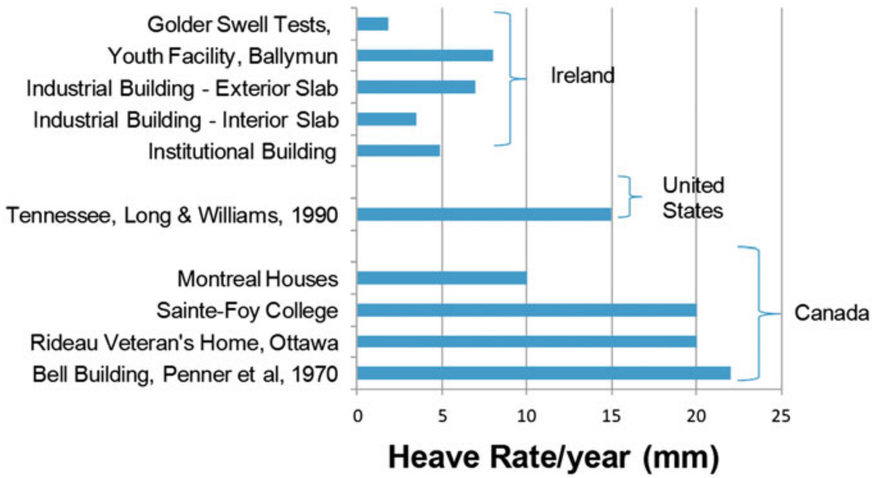

Fig. 27 Recorded heave rates in buildings due to pyrite-induced heave

- The in situ density of the infill (based on the degree of compaction and applied loads);
- The extent of moisture within the infill and access to capillary water;
- Ambient temperature and pH;
- The particle size distribution of the infill (the fine fraction will oxidize faster than the coarse fraction);
- Trace element concentrations;
- The thickness of the fill; and
- The presence of bacteria.

A study of houses undergoing pyritic heave in the Montréal area demonstrated that the total heave in garages was significantly larger than in the adjoining basement and this was attributed to the fact that the garages are underlain by one meter or more of compacted fill while the basement slabs typically have less than 200 mm (Pépin 2000).

Despite these variables, some orders of magnitude for rates of heave can be established from a review of monitoring in buildings that has been reported in the literature and from the results of forensic engineering investigations. Some recorded heave data is plotted in Fig. 27. For North America the reported heave ranges are from 10 to 22 mm per year.

From a number of buildings in the Dublin area where monitoring has been undertaken for periods of up to three years, the heave rates are lower and generally in the range of 4–8 mm per year. From laboratory swell tests undertaken using crushed mudstone from the Dublin area, heave rates were measured at about 2 mm per year in a 300 mm thick layer of compacted fill (Maher et al. 2011).

Laboratory swell testing has shown that compacted mudstone can attain a relatively uniform rate of heave in a controlled laboratory environment. In

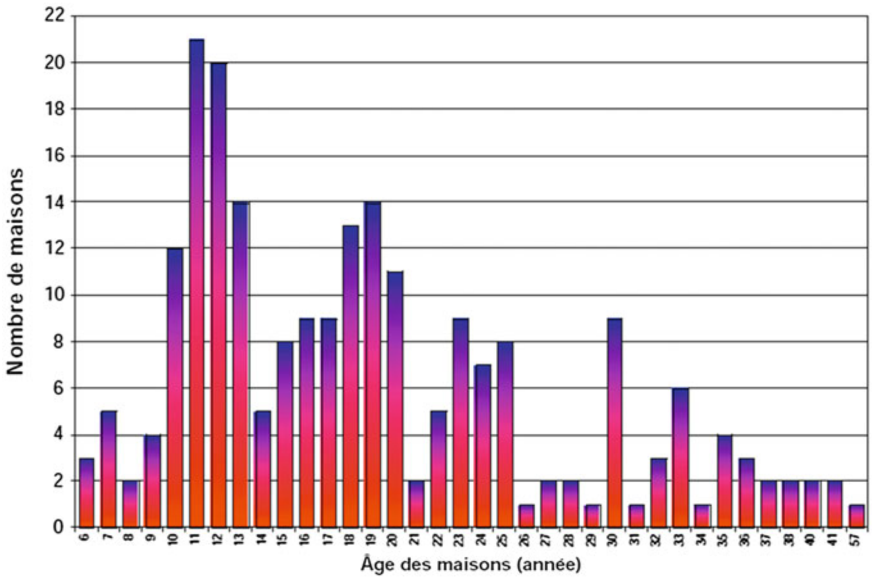

Fig. 28 Age distribution of houses with pyrite-induced damage from the Montréal area study (from Pépin 2000)

practice, the length of time from placement of compacted fill to the development of damage in a building is extremely variable. The main variables will relate to the nature of the compacted fill as listed above, the climatic conditions and local groundwater and temperature regime in the building, and most importantly, the nature of the structure, its rigidity and the fill confinement conditions. Anecdotal evidence suggests that in Québec, damage in houses related to pyrite took typically 10 years to occur. This is corroborated by a study of 224 houses in the Montréal area (Pépin 2000). Figure 28 shows the distribution of houses by age from that study which indicates that houses in the 10–13 year old category were the most frequent.

Over the past six years and based on investigations of thousands of houses and other buildings experiencing pyrite-induced heave in Ireland, the typical time span for initial damage to manifest itself is in the range of 2–6 years. Even after the swelling of the infill starts, the rate is quite small, in the order of 3 or 4 mm per year.

Most buildings can tolerate a certain amount of minor swelling of infill before damage will occur in the structure. The initial heave will be 'absorbed' within the building (e.g., within the insulation layer below the floor slab) and will take time to lead to cracking. The underlying subgrade soils can also compress when load is applied to it. This would reduce the upward pressure on the floor slabs until such time as consolidation of the underlying soil had occurred and then the pressure would be applied upward.

The generally faster times to building damage in Ireland can be attributed mainly to differences in climatic conditions and building design practices.

Expansion Pressures

In connection with legal proceedings in Ireland, Golder Associates developed a large scale laboratory swell test to study pyrite-induced heave in a controlled laboratory environment. The initial experiment with a large scale laboratory swell test began in December 2007 (Maher et al. 2011). The test was performed in a 1.2 m internal diameter concrete manhole ring with a compacted sample height of 1.02 m. The concrete pipe section was placed vertically within a heavy duty steel trough and the aggregate (removed from a house being remediated), with a maximum particle size of 75 mm, was compacted within the cylinder (Fig. 29).

There was no base installed in the pipe as it was assumed that the mass of the concrete pipe and fill would be adequate to force the heave to occur at the top of the fill. This assumption proved wrong. Within seven weeks of commencement of the test it was noted that the entire test cylinder had tilted slightly on the base, due to the expansion of the fill and the friction acting on the inside of the concrete pipe. After 17 weeks, a vertical hairline crack developed in the side of the concrete pipe. After 50 weeks, the vertical crack had expanded to 12 mm (Fig. 30).

Using analytical and numerical techniques, the pressure produced inside the pipe at rupture was back-calculated and established to be about 600 kPa. Figure 31 illustrates various estimates of pressures generated from pyrite expansion. It can be seen that the back analyzed value from the Golder swell test of 600 kPa is in line with estimates made by Spanovich (1969).

Fig. 29 Initial large scale laboratory swell test

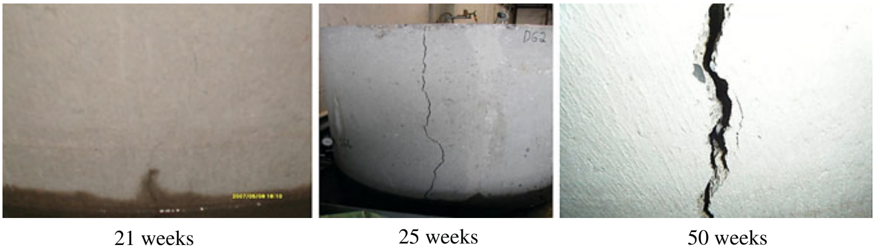

Fig. 30 Progression of vertical crack in concrete manhole ring in 21, 25 and 50 weeks. At 33 weeks it was 12 mm wide

Solution Developed for Pyrite-Induced Heave Problems in Québec

As a result of the extent of the problem in the Québec area, a technical committee (*Comité Technique Québécois D'étude Des Problèmes De Gonflement Associés à La Pyrite*) was created in 1997 to examine the problems caused by the swelling of granular backfill under concrete floor slabs. It was established that the swelling occurs as a result of chemical reactions involving some of the minerals present in the aggregates used in that area of Québec. Due to the extent of the problem, the committee developed standard procedures for investigating and evaluating the problem. These procedures are known as Procedure CTQ-M200, and entitled, "Appraisal procedure for existing residential buildings" (Comité Technique 2001).

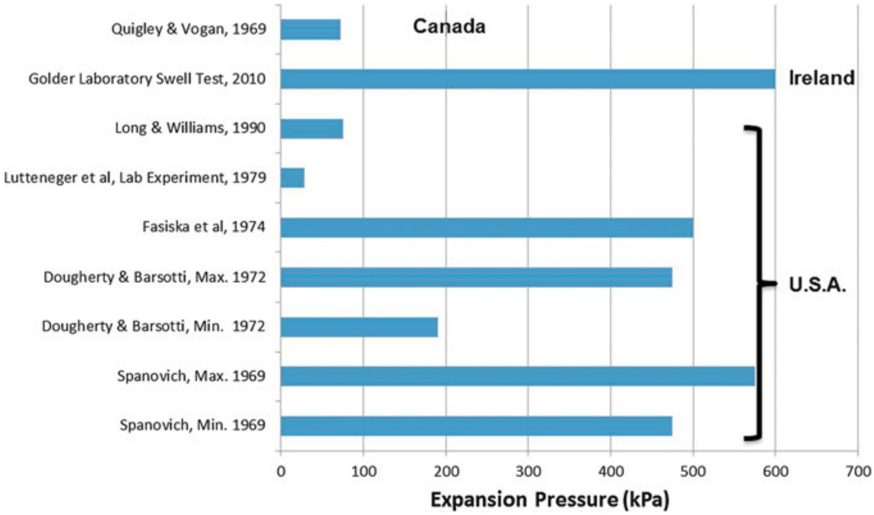

Fig. 31 Comparison of estimated pyrite heave pressures from a literature review

One of the key aspects of CTQ-M200 is the performance of a Swelling Index Test. This petrographically-based test procedure was developed in 1999 by the Québec Technical Committee and was referred to as CTQ-M100/2000. This was subsequently adopted as a Québec Standard (BNQ 2003) and designated as BNQ 2560-500. The laboratory test procedures are divided into two stages.

Stage 1 comprises:

- Sieve analysis
- IPPG (indice pétrographique du potential de gonflement)—Swelling Index Test
- Absorption test
- Micro-Deval testing

Essentially, the Stage 1 evaluation focuses on establishing the clay content of the crushed rock material (i.e., Swelling Index Test) and two key physical quality parameters. What is interesting is that it does not involve the determination of pyrite contents. In the Swelling Index Test, also referred to as the SPPI test (Swelling Potential Petrographic Index), the sample is divided into 20, 14, 10 and 5 mm size fractions. These are then examined by the petrographer who evaluates the clay content within each of the sample particles. Depending on the clay content, a weighting factor ranging from 0 to 1 is applied. If no clay is present in the particle, a granite or basalt, for example, it is scored as 0. If it is a pure shale it is scored as a 1. On this scale a clayey limestone would be assigned a score of 0.5 and a limestone with some clay veneer would be assigned 0.1. A final weighted score is then determined, taking into account the original grading of the sample. This final score, or Swelling Index, will fall between 0 and 100.

An Index of 0 would indicate a crushed rock product containing no clay and thus with no likelihood of causing pyrite-induced heave. Crushed rock products with Swelling Index values of 10 or less are considered suitable for use in construction.

Stage 2 of the BNQ testing protocol is generally undertaken when the results of the Stage 1 testing are not conclusive. Stage 2 involves:

- Total sulphur content (S)
- Sulphate content (SO_4)
- Alumina content (clay content)
- Carbon dioxide content (carbonate content)

BNQ 2560-500 has been adopted as the basis for certifying aggregates for use in building construction. Since 1999, most of the aggregate producers observed this certification requirement and purchasers look for it. The process is referred to as "DB" (dalle de béton or concrete slab) certification. The process for certification requires the sampling and testing to be performed by an approved laboratory and must be performed for every 10,000 tonnes of material for which the Swelling Index must be 10 or less. Each approved stockpile is then assigned a certification number to ensure future traceability.

The École Polytechnique de Montréal undertook a statistical analysis of reports prepared according to the CTQ-M200 protocol for crushed rock fill from 1,700 houses in 26 towns and cities in Québec in 1999. The results were reported by Cormier in 2000 (Cormier 2000). This study concluded that the Swelling Index protocol was a very good predictor of damage. In only 3 % of cases where an index of 10 or less was obtained was damage detected in a house. For Swelling Index values greater than 20, there was a one in three chance of the house having damage. It should be noted that non-representativeness of the sample taken could also account for some of the 3 % of inaccurate assessments.

The vital role played by petrography in identifying the problematic aggregates in Ireland was confirmed in a recent paper by Shrimer and Bromley (2012). The paper concluded that,

> Petrography provided the proof positive that the aggregates were of substandard quality in the sense that they were demonstrably not chemically inert, and that gypsum and other secondary minerals had developed both within the aggregate particles as well as on their surfaces.

In Situ Treatments

Grouting Solutions

While crushed rock products are relatively inexpensive, the costs associated with their removal and replacement after they have been incorporated into a building are huge. In Québec, the problems affected both the basement floor as well as the garage floor. In general, the heave in the garage floor was the most severe, since the underlying fill was quite thick, usually greater than one meter, and a lower grade fill with high fines content was used. The basement slab was typically underlain by a clean stone and was usually less than 200 mm thick. Nevertheless, the repair work generally did not require the residents to vacate their homes. In Ireland, where houses do not have basements, the remedial works are more costly and much more disruptive with the residents having to vacate their homes for up to 12 weeks. In addition, the exterior backfill against rising walls and under footpaths and driveways also needs to be replaced. The typical remedial costs, even for a small house, are at least €30,000.

Clearly, the development of an effective in situ remedial technology would represent a major advance in the treatment of pyrite-induced heave problems in buildings. Over the years, in situ treatments have been considered. They have generally focused on various grouting options. An experimental study to evaluate grouting options was commissioned by the Canada Mortgage and Housing Corporation (Bellaloui et al. 2003). The approach followed in this research was to cut off the supply of oxygen and humidity to the aggregates. Therefore, to be successful, the process needed to completely coat the aggregates with cement grout. The tests showed that with well compacted, graded aggregates (i.e., 0–20 mm), grout penetration could not be achieved, regardless of the type of grout applied. It was concluded that a grouting solution could only work in the case of thin layers of clean stone without fines.

It is our view that grouting solutions are inherently the wrong approach for an in situ treatment for this problem. The basic principle for grouting solutions is that every particle needs to be encapsulated so as to seal it from access to oxygen and moisture—in essence, this process attempts to recreate the condition that the rock had, prior to being quarried, for millions of years. This is extremely difficult to achieve in a well compacted crushed rock, frequently with up to 15 % fines content. The documented cases of concrete undergoing internal sulphate attack suggests that incorporating reactive aggregates into concrete mixes is not a permanent solution.

Grout requires injection under some pressure to achieve penetration. Even modest pressures applied under a floor slab would have a tendency to lift the slab. Encapsulation requires a coating of grout around every particle which necessitates a volume increase. Further, the compatibility and durability of cement-based grout in conjunction with highly reactive pyritic mudstone is an overriding concern with these treatments. A further complication is the solidification of the compacted fill. Thus, if the treatment was unsuccessful, the costs to remove the cemented fill would be significantly increased.

"pHoam" Solution

Modern mining activities, both coal and hard rock, generate acid rock drainage (or "ARD"). The formation of ARD is a natural process. In the presence of air, water, and bacteria, sulfide minerals such as pyrite oxidize and produce sulphuric acid. To have ARD, one needs air, water, and a metallic sulphide mineral source and the bacteria to speed reactions that would otherwise occur slowly.

Based on this, Gusek el al (2008) have proposed a new approach to in situ remediation for problems caused by pyrite oxidation. We can visualize each requirement for ARD generation being positioned at a corner of a tetrahedron (Fig. 32). If any of these primary ingredients are missing, isolated, or chemically

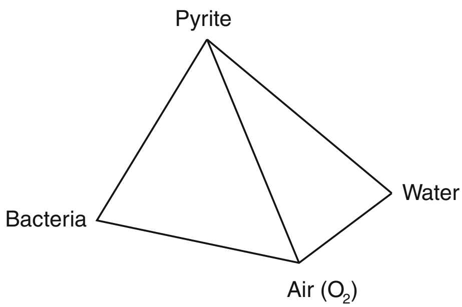

Fig. 32 The ARD tetrahedron (from Gusek 1994)

neutralized, ARD will not occur. The active ingredients to disrupt the tetrahedron and thereby prevent ARD, include liquids, solid particles, gases, and living microbes.

Pyrite-induced heave in buildings requires the same key ingredients as ARD. Current research efforts are underway to apply these principles as the basis for in situ treatment to stop the oxidation process and thereby stop the heave.

By fully understanding the chemical processes that lead to the production of gypsum in compacted shale and mudstone fills, we can devise chemical treatments that prevent the oxidation from occurring. These treatments can comprise liquids as well as solids, as described by Gusek et al. (2012).

These treatments directly control one or more of the four essential elements in pyrite-induced heave, in a similar approach to the use of pharmaceuticals in treating disease. Some examples are provided below:

- Air: Treatments that consume or displace air such as carbon dioxide or sawdust or vegetable protein.
- Water: Coat reactive surfaces with chemical treatments such as potassium permanganate solutions, bentonite clay etc.
- Sulphides: Neutralize acid such as with lime solutions or flyash.
- Bacteria: Bactericides such as sodium lauryl sulphate or phosphate solution.

While a cocktail of chemical treatments could be delivered by way of a liquid solution, in the case of compacted fills within buildings, this is impractical. The solution would drain to the bottom and escape through joints in walls. The solution devised is to utilize foam technology. Foam is a two-phase fluid consisting of a gas component surrounded by a thin fluid phase that is developed with a soluble surfactant or soap (Gusek et al. 2012). The pyrite oxidation-suppressing active ingredients can be entrained in, or are a part of, the foam structure. In addition, solid phase ingredients such as limestone, lime, biosolids, or cement kiln dust can be entrained and suspended in the foam structure. Such mixtures comprised of ARD-suppressing or pyrite oxidation-suppressing components are hereafter referred to as pHoam™ to distinguish it from common foams used in other industrial applications, including fire-fighting.

The proposed pHoam™ method solves the active ingredient delivery problem by increasing the mobility and surface area of solutions or mixtures of active ingredients without sacrificing hydrologic control. Active ingredients suspended or contained in a pHoam™ of predetermined "stability", can flow omni-directionally or bi-directionally from a single injection point as an advancing front. It is also considered possible to introduce agents to dissolve gypsum by way of pHoam™.

Laboratory experiments are currently underway to develop the optimum treatments for suppressing the oxidation of pyrite in shales and mudstones and to confirm the effectiveness of pHoam™ as the delivery mechanism. The basis of the testing program is to utilize a series of Golder laboratory swell tests. Once the steady state condition of pyrite-generated heave is attained, these test cells can be injected with pHoam™ containing different ingredients either singly or in combinations.

Fig. 33 Series of six Golder swell tests, five containing compacted swelling mudstone and one containing inert limestone

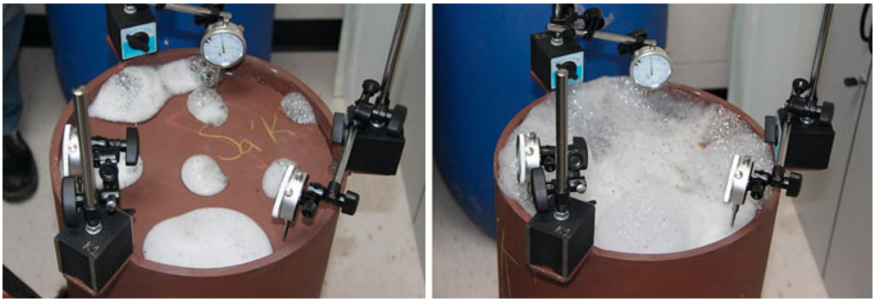

Fig. 34 Foaming trial showing foam injected from the *bottom* of the sample escaping through and around the *top* plate

The test cylinders contain samples that are 336 mm in diameter and 302 mm high (Fig. 33). The bases of the cylinders are perforated to allow moisture to enter the sample from the water trough in which it stands.

The first foaming trial included a bactericide. This was introduced into Test Cell K by way of a foam injection from the bottom of the test cylinder (Fig. 34). The injection was performed at Week 15 and the results at Week 20 demonstrate that the introduction of the bactericide interrupted the steady state pyrite-induced heave (Fig. 35).

While this testing program is ongoing, the early results are very encouraging. We have confirmed that the Golder swell tests develop a steady state rate of heave within 5–10 weeks in compacted mudstone. We have also demonstrated that pHoam^{TM} is effective in permeating compacted crushed mudstone with dry densities as high as 1.90 Mg/m^3 and fines contents (passing 75 μm) of 11 %. We have

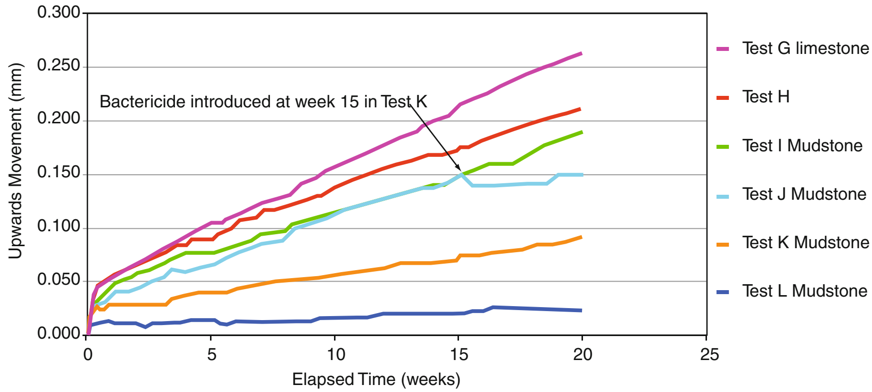

Fig. 35 Results of swell tests showing effect of introducing bactericide into swelling mudstone

also demonstrated that the introduction of a bactericide suppresses pyrite-induced heave for at least a period of five weeks. This indicates the importance of bacteria in supporting the reactions.

Conclusions

This chapter presents an overview of problems associated with sulphide minerals in rocks used in construction in Canada and compares them to similar problems experienced in Ireland, which were first identified in 2006. The first problems with pyrite-induced heave in shales in Canada were identified in the Ottawa area in the 1960s. These related to foundations supported on Billings Shale. The problems were appropriately investigated and the causes identified. These early investigators were accurate in the mechanisms they ascribed to the problem and they recognized the key characteristics of these problematic fine grained sedimentary rocks.

Despite this clear identification of problems associated with shales containing disseminated pyrite, a widespread 'epidemic' of pyrite-induced problems developed in the Montréal area in the mid 1980s. In this case, the problems came about from the use of pyritic shales as compacted fills beneath floor slabs in residential and commercial construction. While it could be argued that the Utica Formation shales of the Montréal area were generally of better quality than the shales of the Billings Formation, it points to an overall lack of quality and testing standards for construction aggregates used in residential construction.

It is worth noting that these problematic aggregates do not appear to have made their way into municipal infrastructure works, which suggests that the industry applies different quality standards to aggregates used in low rise building construction. It is observed that tests for total sulphur have not been historically performed in Canada for construction aggregates. These problematic aggregates could have been identified as being of extremely poor quality and low durability by simply performing the standard aggregate source acceptance testing protocols.

This chapter also presents three contemporary pyrite-related problems in relation to building and infrastructure developments. The first case deals with an industrial building partly supported on Billings Shale where the water table was lowered, triggering oxidation of the underlying shales. These are legacy problems from a time when this problem was not fully understood and the appropriate mitigative measures were not instituted. The case of the Ottawa Light Rail Transit system demonstrates that potential problems with pyritic shales can be avoided when they are identified by investigation at the pre-design stage of a project. It also demonstrates that shales are not just problematic because they contain reactive pyrite. They also contain high clay contents and are subject to swelling when exposed to fresh water. Pioneering work undertaken at the University of Western Ontario has developed test procedures to quantify these expansive pressures so that they can be fully accommodated in construction works in shales.

The third case study describes a current problem with aggregates used in concrete mixes mainly in residential construction in Québec within the last eight years. These aggregates were of extremely high quality from a mechanical-physical perspective, but contained reactive pyrrhotite, pyrite and chalcopyrite. There was a failure to appreciate at the time the risks associated with the incorporation of aggregates containing pyrrhotite into concrete mixes, even in cases where the aggregates were very dense and of low permeability.

In the light of the Canadian experience, which has been well documented in the scientific and engineering literature since the 1960s, it is difficult to understand how such widespread problems could have arisen in Ireland in the early part of the 2000s. Although much of the Canadian literature on the topic refers to shales while in the Irish cases the problematic aggregates are generally classified as mudstones, the similarities are quite stark—fine grained sedimentary rocks deposited in a marine environment and containing disseminated very fine grained and framboidal pyrite.

This chapter also presents some mineralogical data on the problematic Lorraine and Utica Shales. While the Lorraine Shales contain extremely high clay contents and very low calcite contents, the Utica Shales are quite variable and many have relatively high calcite contents. Nevertheless, due to their high porosity they are prone to ingress of humidity and oxygen and allow the growth of gypsum with consequential damage to buildings in which they are used. From Fig. 24 it is clear that some of the Utica Shales are of similar mineralogical composition to the problematic Irish mudstones.

Since the introduction of the DB aggregate certification program in Québec in 2003, it is our understanding that no buildings using certified fills have experienced pyrite-induced heave. This confirms the effectiveness of the protocols developed which are based primarily on petrographic examination rather than simply quantifying the pyrite content. However, the concrete problems in Trois-Rivières demonstrate that each aggregate application must be assessed separately, as the problematic aggregates that have given rise to extensive concrete deterioration would have had a Swelling Index of 0.

This chapter also reviews some current research work aimed at developing in situ remedial treatments for compacted fills undergoing pyrite-induced heave. Given the history of this problem, it is reasonable to assume that related problems will continue to occur in the future in other parts of the world. Whilst any in situ treatment of this nature has inherent complications, not least of which is confirming that it will provide long term relief from future damage, nevertheless, we need to continue to progress our understanding of this problem which has devastating impacts on both people's lives and on commercial enterprises.

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