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BordinaMarta@studiocerdaepanini.com

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Changes in engineering properties of natural stone
Yates, Richardson and Miglio

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Changes in engineering properties of natural stone

Tim Yates PhD, BTech
BA Technical Director, Building Technology Group, Building Research Establishment Ltd, Watford, UK

David Richardson MSc, CGeol, FGS
Group Director Building Technology, Building Research Establishment Ltd, Watford, UK

Bruno Miglio MSc, MBA, CGeol CEng
Director, Ove Arup & Partners, London, UK

It has long been known that some stone types show significant loss in strength, both in compression and bending during the service life of a building. These differences can include changes as a result of long-term changes in physical structure, for example as a result of thermal and moisture cycles, but there can also be shorter term changes between the wet and dry states. The European Standards that are now in place in the UK and the rest of the CEN countries require stones to be tested under ambient conditions and, if required to declare these values as part of the CE marking process. This has led to some concerns that the data available for structural calculations may not reflect the actual performance during use, and as the demands for more slender stone units increase so the margins for error become smaller. The present paper begins by examining a number of reported examples of changes in properties with time as a result of changes in structure in some marbles as well as changes in appearance and strength in limestone and sandstone as a result of salt damage and freeze-thaw action.

1. Introduction

It has long been known that some stone types show significant differences in strength, both in compression and bending during the service life of a building (Schaffer, 1932). These differences can include changes as a result of long-term changes in physical structure, for example as a result of thermal and moisture cycles, but there can also be shorter term changes between the wet and dry states. These changes have traditionally been accounted for in the design stage by the inclusion of 'factors of safety' and also by the 'over-engineering' of structures found in older codes and standards.

Originally, natural stone was used for load-bearing masonry walls, but more recently it has been used as thin cladding or as curtain walling for high rise buildings. In the years following World War II the thickness of natural stone facade cladding decreased from over 100 mm to typically 20–40 mm but it was only after 40–50 years of practice using thin cladding that an increased rate of issues relating to the long-term performance of the stone began to be detected. Present day pressures towards more innovative uses of stone, where safety margins become ever smaller, has resulted in a need to understand and quantify the changes that can occur to the properties of natural stone during its expected service life.

2. Bowing and warping of marble

Many well known examples of stone cladding failure in service now exist, often involving some type of marble but not exclusively so. Buildings such as the 'Amoco Building' (now known as the 'Aon Center') in Chicago and the 'Finlandia Hall' in Helsinki had their cladding replaced after less than 30 years. The marble cladding on the Amoco Building was replaced by granite at a cost of £40–55 million in 1989, whereas the marble cladding on the Finlandia Hall was replaced between 1998 and 2000 at a cost of £2·5 million, by a similar marble, which is already showing signs of severe deformation. In each of these now famous cases the problem was essentially one of 'bowing' or 'warping', namely essentially an expansion-based form of deterioration (Malaga *et al.*, 2004; Yates *et al.*, 2004).

Even though the vast majority of buildings with façade cladding of natural stone that exhibit durability problems in terms of bowing are clad with marble from Carrara region in Italy, the problem is clearly not restricted to this type of marble. Other marble types, such as those from Portugal, Norway and the USA are known to bow. It is equally important to recognise that there are many buildings where marble from the Carrara region has performed well as cladding.

The mechanism of the observed deterioration is still not clear. Several hypotheses have been proposed but as yet none explains all the observations from practice. Both temperature variations and moisture are known to be involved, and recent studies including the EU-funded TEAM project (http://www-v2.sp.se/building/team/PDF/TEAM%20Final%20Report.pdf) have recently acknowledged moisture as a key factor. Freezing will aggravate the deterioration, but is not a primary parameter. Studies of the mineral structure of marbles that are prone to bowing show that the microstructure seems to be the most important influencing intrinsic factor. The durability differences between the polygonal (granoblastic) (Figure 1(a)) and lobate to sutured structured (xenoblastic) (Figure 1(b)) marbles are obvious, the first ones being those most prone to bowing and strength loss.

The results of the deterioration show as bowing, warping or dishing, volume and porosity increase, brittleness, loss of strength and modulus of elasticity and ultimately in failure (Figure 2).

As part of the TEAM project more than 150 building projects around Europe have been recorded and about 50 of these were reported to have bowing problems. It is likely that many more examples remain unnoticed.

The research, inspection and testing from the TEAM project showed that a great many factors may influence the risk of bowing and expansion, for example, stone type, panel thickness, joint design, fixing methods as well as the environment. The inspections showed that an environment with temperature cycles and moisture, as in Northern and Central Europe, increased the risk of bowing and expansion.

The TEAM project identified, as part of its research, that probably the most important physical change observed on those structures exhibiting bowing was the loss of flexural strength. The results of changes in strength from both a literature review and the testing of samples from buildings are summarised in Table 1.

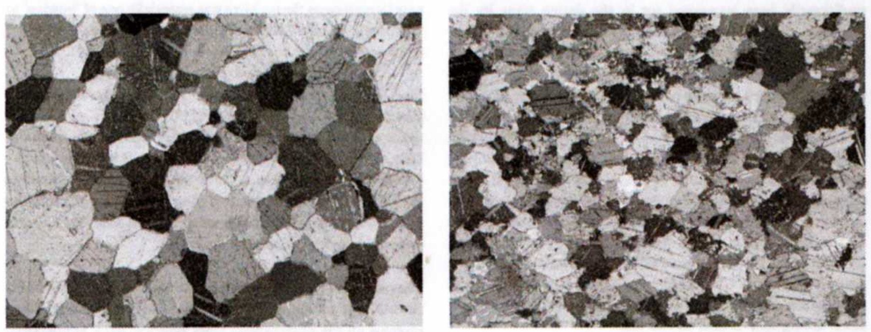
This type of problem has become increasingly important as the international trade has led to an increased use of new stone types in new environments, without the proper testing. The problem can be solved or reduced substantially by screening the selected stone types with, for example, a realistic, accelerated testing of the bowing and expansion potentials.

3. Changes in strength – short-and long-term

Winkler (1994) summarised a number of earlier studies on changes in compressive strength as a result of the stone being saturated with water at atmospheric pressure. Winkler (1994) suggested that the 'wet-to-dry' strength ratio could be used as 'a crude immediate indicator of the stone's durability' and suggested the following definitions, based on the wet-to-dry strength ratio

(*a*) 0·8–1·0: indicates an excellent durability
(*b*) 0·7–0·8: good to excellent durability
(*c*) 0·6–0·7: fair to poor durability
(*d*) 0·5–0·6: poor durability
(*e*) less than 0·5: very bad durability (as a result of too much clay).

The reasons for these changes are not at all clear. Work by Murphy *et al.* (1984) demonstrated a relationship between normalised vapour pressure and changes in modulus of elasticity. These changes are attributed to the differing adsorption of water found for a range of stone types. Murphy *et al.* (1984) put forward the idea that sandstones were more susceptible to changes in strength because the structure and cementing of the grains allowed for a greater adsorption of moisture. They were also able to demonstrate that even a small amount of moisture could effect the 'contact stiffness' between quartz grains. The more complete contact and cementation between grains in limestone was shown to result in a much smaller difference between wet and dry strength in comparison to sandstone.



(a) (b)

Figure 1. (a) Polygonal calcite grains in a granoblastic marble; (b) lobate to sutured grains in a xenoblastic marble

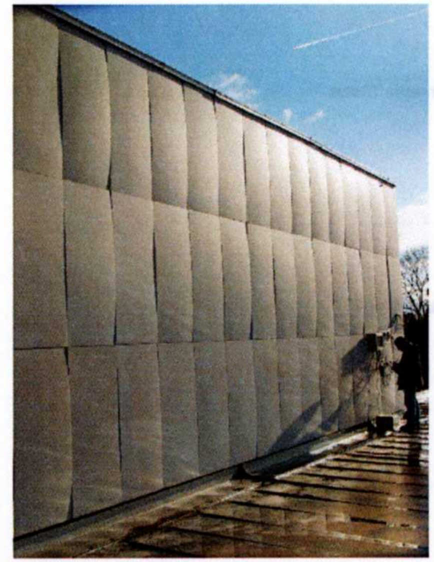


Figure 2. Marble cladding panels at Göttingen Univeristy, Germany showing warping and distortion as a result of changes in the mineral structure

In addition to the short-term effect of loss of strength when the stone is wet (where reductions which have been shown to be reversible), there are also longer term changes which are associated with the weathering of stone that result in longer term, but irreversible, changes in strength as a result of freeze–thaw and salt crystallisation cycles. The changes in strength can be expressed directly as changes in compressive or flexural strength (for example, Table B in Corbella *et al.* (1990)) or as changes in modulus of elasticity (for example the European Standard BSEN 12371 (BSI, 2010b).

4. Determination of wet and dry strengths for two UK stones

4.1 Rockingstone sandstone

Rockingstone sandstone is from the Millstone Grit of Carboniferous age. It is a medium to coarse-grained slightly micaceous sandstone, pale yellow buff in colour with red/brown veining. The quarry is located on Bolster Moor near Huddersfield, West Yorkshire. Block sizes of up to 3000 mm × 1500 mm × 1200 mm are available along with veneers and paving material up to a maximum of 3·5 m^2 ranging from 30 to 100 mm in thickness.

In December 1996 BRE were supplied with a sample of 50 specimens of this sandstone. The specimens were nominally 75 mm test cubes cut from a single stone block and the cubes

Building	Marble type (origin, country)	Age: years	Loss of flexural strength
Finlandia Hall, Helsinki, Finland	Bianco Carrara (Italy)	21	~ 85%
Amoco/Aon, Chicago, USA	Bianco Carrara (Italy)	15	~ 40%
Office building Nyköping, Sweden	Bianco Carrara (Italy)	31	~ 75%
Hospital Lünen, Germany	Trigaches/Escamado	14	~ 30%
	(Portugal)	28	~ 75%
Office building Switzerland	Bianco Carrara (Italy)	3	~ 40%
Bank building Copenhagen, Denmark	Porsgrunn (Norway)	23	~ 45%
Office building Copenhagen, Denmark	Porsgrunn (Norway)	41	~ 75%
Office building Lyngby, Denmark	Marmorilik (Denmark)	60	~ 45%
Office building Paris, France	Bianco Carrara (Italy)	11	~ 50%
Office building Malmö, Sweden	Bianco Carrara (Italy)	20	~ 10%

Table 1. Summary of some measured strength reductions for cladding panels (Yates *et al.*, 2004)

will aggravate the deterioration, but is not a primary parameter. Studies of the mineral structure of marbles that are prone to bowing show that the microstructure seems to be the most important influencing intrinsic factor. The durability differences between the polygonal (granoblastic) (Figure 1(a)) and lobate to sutured structured (xenoblastic) (Figure 1(b)) marbles are obvious, the first ones being those most prone to bowing and strength loss.

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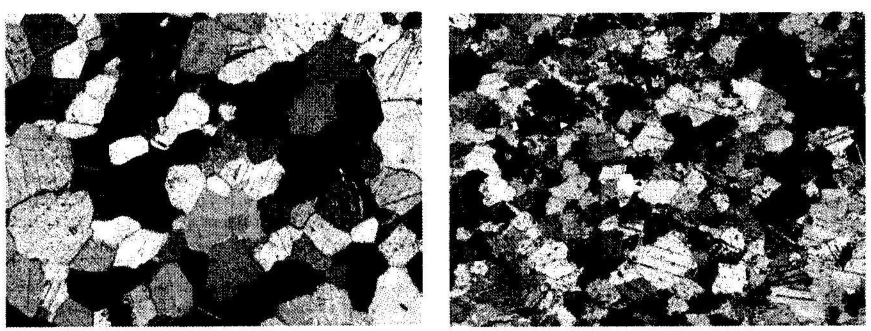

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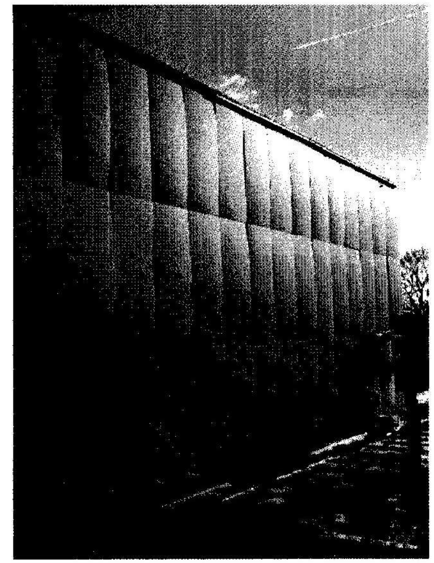

Figure 2. Marble cladding panels at Göttingen Univeristy, Germany showing warping and distortion as a result of changes in the mineral structure

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Table 1. Summary of some measured strength reductions for cladding panels (Yates *et al.*, 2004)

Reference no.	Compressive strength: MPa			Condition method*	Sample condition
	Mean	Minimum	Maximum		
E7095/11-20	115·7	101	122	EN, 24 h 70°C	Dry
E7095/21-30	113·7	105	124	ASTM 48 h 60°C	Dry
E7095/41-50	111·2	99	119	EN, 24 h drying 1 min in water	'Just' wet
E7095/31-40	79·7	69	88	EN, 24 h drying 60 min in water	'Partially' wet
E7095/1-10	65·7	53	77	ASTM 48 h in water	Saturated

ASTM and EN refer to standards used for testing.

Table 2. Test data from a single block of Rockingstone sandstone

were tested by loading perpendicular to bedding. The cubes were dried in accordance with currently available test methods and the sample conditioning in, then proposed, European Standard test methods. Samples were then placed in water for different lengths of time. This included soaking samples for 1 and 60 min in addition to saturating some samples. The compressive testing was then carried out in accordance with the provisional standard that is now BSEN 1342:2001 (BSI, 2001). Each sample contained 10 specimens. Table 2 and Figure 3 summarise these results.

The two 'dry' values are very similar – and certainly within the variation likely to be found within a natural material. The results show that overall there is a 43% reduction in the mean strength between the dry and saturated samples. It is interesting to note that there is a smaller reduction in the maximum values (37%) and a great reduction in the minimum values (48%) and so overall the spread of results increases as the specimens become saturated. The crucial time period appears to be between 1 min (3% reduction in the mean strength) and 60 min (30% reduction in the mean strength).

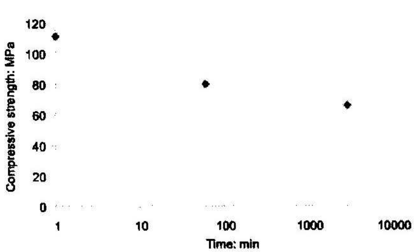

Figure 3. Changes in compressive strength with time of soaking in water

4.2 Portland limestone

Portland Whitbed limestone is an open-textured oolitic limestone from the Portlandian formation (Jurassic). The stone is formed from micrite (fine-grained calcium carbonate) ooids with a small quantity of micritc occurring as matrix. The shell fragments are elongated to rounded and are typically about 5 mm across. The Basebed is similar but in general contains fewer shell fragments, a greater degree of cementation, and a reduced voidage. The stones have been quarried on the Isle of Portland, Dorset for 1000 years and have been used across the UK since the early seventeenth century.

The extensive use of Portland limestone has led to the stone being tested on many occasions. Between 1988 and 2008 some 1136 tests were carried out in accordance with current European standards by the Building Research Establishment (BRE) on behalf of Albion Stone, or by other accredited testing houses. Albion Stone Quarries Ltd and BRE have collated these results and the data for strength tests are summarised in Table 3.

Although there are variations in the effect of water saturation on the strength between the different beds the most obvious difference is between the compressive and flexural strength tests with changes of around 25% in the compressive test and 45 to 50% when the flexural strength test is used. The increased cementation in the Basebed increases the overall strength but does not affect the percentage change in strength. If the 'lower expected value' is used in place of the mean then the percentage change remains very similar.

5. Managing changes in properties at the design stage

There has long been a sense that stone cladding systems are over-designed, particularly in the UK, where typical cladding thickness is 75 mm for limestone against perhaps 40 mm in Continental Europe. This discrepancy has been challenged in recent times and the revised version of BS8298 published in 2010 (BSI, 2010a) encourages thickness to be determined by

Stone type			Bowers Basebed	Independent Basebed	Bowers Whitbed	Independent Whitbed
Test	Orientation	Condition				
Compressive Mean: MPa	Perpendicular	Wet	41·9	42·0	26·1	39·5
		Dry	55·3	56·1	37·7	49·8
		% change	24%	25%	31%	21%
Compressive Mean: MPa	Parallel	Wet	35·0	–	–	32·8
		Dry	–	–	–	42·2
		% change	–	–	–	23%
Flexural (3 pt) Mean: MPa	Perpendicular	Wet	3·73	4·03	3·58	5·04
		Dry	7·16	8·30	5·57	7·65
		% change	48%	52%	36%	34%
Flexural (3 pt) Mean: MPa	Parallel	Wet	4·34	–	–	3·30
		Dry	8·08	–	–	7·38
		% change	46%	–	–	55%

Table 3. Comparison of wet and dry strength test results for four Portland limestones

engineering calculation rather than relying on the standard table given in the 1994 version (BSI, 1994).

The calculation method requires two very important pieces of information with regard to the stone. The first is the flexural strength of the material. This is usually taken as the mean strength of the stone, and in accordance with European Standards this would be the strength when tested dry. The second piece of information that is required is the appropriate factor of safety to be applied. This has commonly been taken to be 6 when applied to the mean flexural strength result. But this is UK practice; the USA, for example, has a different approach. Traditionally, in the USA stone has been designed using factors of safety that are grouped according to stone type. This is exemplified in ASTM C1242-10 (ASTM, 2010e) from which Table 4 is produced.

Clearly this seeks to attribute higher factors of safety to classes of stones which are thought to be more variable in their properties. These factors of safety have been used for decades

Stone type	Specification	Safety factor
Granite	C615 (ASTM, 2011a)	3
Limestone	C568 (ASTM, 2010a)	6
Group A marble	C503 (ASTM, 2010b)	5
Travertine	C1527 (ASTM, 2011b)	8
Sandstone	C616 (ASTM, 2010c)	6
Slate	C629 (ASTM, 2010d)	5

Table 4. Generally accepted safety factors for stone cladding by stone type

and changed and refined by various authors without very much justification for their selection. The factor of safety is supposed to take into account all aspects of the material variability; however, with some stone types this is unlikely to be the case. First, as has been demonstrated with some white marbles, the strength loss over time can be considerable, as shown in Table 1. However, the 'ageing' strength loss is not restricted to some types of marble. Recent testing involving several types of onyx for a major project indicated significant changes in strength for some of the stone types, but more commonly some sandstones and limestones can also lose strength both in the short term and in the long term. Obviously the mechanism of the strength losses will be different in each case but they can clearly be significant.

Strength loss through ageing however is only one aspect of the problem in choosing the correct strength value for input into the thickness calculation. The use of 'dry' testing in European standards is another potential problem, particularly with some sedimentary rocks but also some of the more weathered granites that are used in construction. The difference between dry and wet strength of some of these materials can be considerable and at least 50% lower in some cases (see section 4 above), and indeed as we have seen Winkler uses the ratio of wet and dry strength as a measure of durability (Winkler, 1994).

What are the options for the designer then? Work carried out by the Centre for Window and Cladding Technology (CWCT) has to some extent described a possible way forward (CWCT, 1997).

More recent guidance such as the CWCT seeks to capture the differences in stone performance by using the actual variability of test results on individual stones. Of course such an approach pre-supposes that recent test data are available for the stone or that project specific testing will be carried out. Given the move to CE marking and product verification we do not believe that this is an unreasonable approach.

5.1 Option 1

Where a stone is well known and has a proven in-service durability record then the prudent approach is to carry out flexural strength testing in both the wet and dry conditions. This does not present a problem as American Standard Test Methods (ASTMs) have always done this and have a standard method for conditioning the samples (ASTM, C880-09; C170-09; C99-09). The ratio of wet-to-dry values will also give some preliminary indication of potential durability problems.

Assuming that the stone type is well known and durability issues can be ignored on the basis of the knowledge of the in-service performance of the material then the wet values should be used in any calculation.

However, rather than use the mean, the modern British Standards allow for the calculation of the 'lower expected value' (LEV) which takes into account the statistical variation in the individual strength values obtained in the test. Under such circumstance, having taken a somewhat cautious approach with the testing by using the LEV rather than the mean, the flexural safety factor (FSF) that is typically adopted is 3 rather than 6. This can be adopted for all stone types as the material variability has been allowed for in the LEV calculation.

It should be noted that a different approach to the same problem is given in Camposinhos (2012, this issue) again where the materials variability is taken up in a statistical approach using the coefficient of variation. Their Table 4 shows a partial factor of safety for the highest risk class of 2.4. This is then subject to a multiplier depending on the coefficient of variation.

5.2 Option 2

Where the durability characteristics are not well known then a different approach is required. The objective is to determine a durability factor rather than rely on subjective factors of safety. So, in addition to wet and dry testing a number of flexural strength samples need to be subjected to a durability test, the precise nature of which will be dependent on the stone type.

The durability test selected will be dependent on the stone type and the anticipated exposure conditions. For marbles this will be a thermal cycling test, for some limestones or sandstones it may be a frost test (depending on the geographic location of the project) or it may be a combination of thermal cycling and wetting and drying. Once the samples have been subjected to the durability test they should then be tested for flexural strength and the LEV obtained. These values are then compared with the values obtained before durability testing. The durability factor is given in Table 5.

Obviously if any samples show indications of cracking, spalling or deformation then they should not be considered further. If they simply show a strength loss then the durability factor can be adopted. The calculation remains the same but the flexural safety factor is multiplied by the appropriate durability factor described above. Using the calculation methods given in Yates *et al.* (1998) the following examples are presented for assessing the suitability of the stone in bending.

5.2.1 For Option 1

It is assumed that the calculations for a particular thickness showed a flexural breaking load of 0·5 MPa (the calculated strength at which the stone would fail). This load is then multiplied by a FSF, which in this case is simply the basic factor of safety (FoS) of 3, giving a design flexural strength of 1·5 MPa.

The design flexural safety is then compared with the LEV obtained for the stone on the wet tests (or dry, whichever is the lower). If the LEV for the stone is greater than 1·5 MPa then the stone will be acceptable at that thickness.

5.2.2 For Option 2

The same method applies except that to obtain the FSF, the FoS would be multiplied by the durability factor (DF); that is, FSF = FoS × DF.

Assuming in this case a loss of strength in durability testing of say 30%, namely a fraction of 70% of the initial strength, then a durability factor of 1·5 would be adopted.

The calculated flexural breaking load of 0·5 MPa would be the same, but it would be multiplied by both the FoS of 3 and the DF of 1·5 giving a design flexural strength of 2·25 MPa.

This would then be compared with the LEV obtained for the stone after durability testing. If the LEV for the stone is greater than 2·25 MPa then the stone will be acceptable at that thickness.

As stated above, this is only applicable if the samples show no signs of distress; if any samples show indications of cracking, spalling or deformation then the stone should be considered unsuitable for use on the project.

Fraction of initial flexural strength: %	Durability factor
95 to 100%	1
75 to 95%	1·2
60 to 75%	1·5
Less than 60%	1·8

Table 5. Durability factors

6. Conclusion

The data presented in this paper have demonstrated that there can be significant changes in the physical properties of natural stone both in the short term (hours to days) and the longer term (years to decades). Many of the changes are associated with climate, and rainfall in particular, and so any trend towards an increase in the amount of rain falling on the façade may well change the structural properties of the stone.

These results have implications for the use of stone in construction, and particularly the assessment and design of cladding panels in which the loading is to be in bending. Any trend towards thinner panels needs to be accompanied by an understanding that the stones are likely to become saturated more easily and that loading in this condition needs to be taken into account in the dimensions of the panels, the fixing system and the overall expected service life.

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