2.1 Compressive Strength

Figure 2.1 shows that for a concrete to be durable it must, amongst other things, be able to resist abrasion attack. The diagram also indicates that two important factors that determine durability are w/b ratio and cement content. Since these factors also play an important role in both abrasion resistance and compressive strength, it is not surprising that trafficked surfaces are often considered to have adequate abrasion resistance if the concrete has a satisfactory compressive strength. This assumption however is not universally correct. Although abrasion resistance is related to compressive strength generally, by means of using special surface finishing processes or materials, it is possible to achieve a surface with a high abrasion resistance using a concrete that has a low compressive strength. Conversely, with poor construction practice, it is also possible to produce a poor quality surface that has a low abrasion resistance from a concrete that has a high compressive strength.

Given that compressive strength is still the most widely used means of assessing concrete quality, it will be useful to gain a better understanding of the underlying principles that govern the relationship between compressive strength and abrasion resistance.

2.1.1 Factors Affecting the Relationship between Compressive Strength and Abrasion Resistance

Abrasion resistance is a measure of the ability of the surface zone concrete to resist normal and tangential abrasive loads. Compressive strength on the other hand is a measure of the core concrete’s ability to resist compressive stresses. Therefore the extend to which the morphology of the ‘surface’ concrete resembles that of the ‘core’ concrete will determine how closely abrasion resistance is related to compressive strength.

There are construction processes and special materials that have a very profound influence on the microstructure of the surface zone concrete, and hence abrasion resistance, but which have either limited or virtually no influence on the compressive strength, as determined by a core taken of the slab. For example *curing* is very important in promoting abrasion resistance, less so for compressive strength. *Delayed power trowelling* produces an extremely dense/hard surface even in concrete with a relatively low cement content. It has the potential of completely undoing the detrimental effects that surface bleed water would otherwise have on w/b at the surface zone. Likewise a *dry shake* consisting of cement and a hard wearing aggregate such as quartz or even metal shavings makes for surfaces with exceptionally good abrasion resistance. Some *liquid surface treatments* are also very effective in improving abrasion resistance that will conceivably make no difference to compressive strength. Furthermore, abrasion resistance is far more sensitive to *aggregate hardness* than to compressive strength. In the longer term weathering effects that influence the integrity and hardness of the paste at the surface zone are unlikely to affect compressive strength. Examples include *carbonation;* and various forms of chemical attack that may result in *dissolution, leaching, expansion and alteration*. (Only if the concrete is relatively permeable will such weathering effects influence the core concrete).

On the other hand, any measure taken to improve compressive strength also has the potential to improve abrasion resistance, to a *greater or lesser* degree. Examples of this are minimising voids by thorough compaction, and reducing the w/b (water:binder ratio) in various ways; e.g. increasing binder content, using a superplasticizer to reduce water content, selecting fine aggregate with a low water demand.

Conversely, it is possible for a high grade concrete to have good compressive strength but poor abrasion resistance, if the surface was subjected to detrimental effects such as excessive bleeding (resulting in a thick surface laitance) or poor curing (e.g. early exposure to wind and heat).

The various statements made above are supported by the following authors:

Hilsdorf(1995) concluded that an abrasion-compressive strength relation depicts the effects of technological parameters such as water/cement ratio, grading of aggregates and to some extent duration of curing. However, it does not reflect variations of parameters that influence only the properties of the surface layer of a concrete member, such as effects of bleeding, or special surface treatments.

Alexander(1984) reviewed the factors that affect the abrasion resistance of concrete and concluded that 'in general, it is true to say that measures that improve strength, impermeability and density of the concrete will also improve the abrasion resistance'. These measurements include aspects such as w/c ratio, cement content, mix design, curing, drier mixes consistent with full compaction.

Ghafoori(1992) pointed out that it is often assumed that concrete pavers that have a certain minimum compressive strength will also have adequate abrasion resistance. Although such an assumption may be reasonable, is not always correct says Ghafoori, since surface finish and curing have more impact on abrasion resistance than on the bulk strength characteristics.

From these statements it is evident that depending on the presence or alternatively absence of factors and processes that only affect the surface, compressive strength and abrasion resistance may or may not be correlated.

2.1.2 Comparison of Failure Mechanisms

In chapter 3, consideration is given to the mechanisms that govern failure, both for crushing and for abrasion wear. At first glance they appear to be substantially different. Crushing failure is the result of crack initiation and progressive growth as the load is increased, Newman(1997a), while abrasion wear is generally the result of a gradual removal of microscopic asperities where relative movement occurs between two bodies in contact, Hutchings(1992).

Abrasion wear of this kind may be expected when the abrasive loads impose normal and tangential stresses that result in 'plastic deformation', rather than 'brittle fracture', resulting in '*mild*' abrasion wear. (These terms are fully discussed in chapter three).

However as the severity of the abrasive load increases, there is a shift towards small localised cracks of various kinds, resulting in '*severe*' abrasion wear. These may be quite similar to the small cracks that are initiated in the concrete's bulk during the middle stages of a compression test, Newman(1997a).

For purposes of discussion later on, it will be useful to say that 'mild' abrasion results in '*shallow*' abrasion-wear of a depth not exceeding 1mm, while 'severe' abrasion results in '*deep*' abrasion-wear exceeding a depth of 5mm.

In any comparison of compressive strength and abrasion resistance it must therefore be kept in mind that there are both differences and similarities in the respective failure mechanisms, depending on the severity of abrasion.

2.1.3 Choice of Abrasion Test Influences Relationship with Compression Test

The correlation between abrasion resistance and compressive strength may to a large extent depend on the type of abrasion test used. Some have a very severe action and remove large amounts of material to a substantial depth. *O*thers are purposely designed to evaluate the quality of the top mm and consequently remove very little material. As a general rule it may be said that abrasion tests that abrade to some considerable *depth* will correlate well with compressive strength. Clearly compressive strength measures a volume property of the overall concrete, and an abrasion test that penetrates deeply can be expected to do the same. To illustrate, some tumbler tests have the capacity of wearing away 80% of the material, reducing the original cube specimens to relatively small round balls by the end of the test. Likewise the heavy rolling/impacting steel balls of the rattler tests used by Abrams:1921 and Crepps:1920 was capable of removing 25mm of the specimen's surface.

Conversely the depth of wear of tests involving lightly loaded rolling steel wheels, such as that developed by the Cement and Concrete Association, UK, would generally be limited to 1mm, sometimes as little as 0,02 mm. This test therefore has the ability to give useful information on the quality of the topmost surface layer, a layer that may have a morphology that has been substantially modified by various surface processes, discussed in 2.1.1. Such a test used on such a surface has no ability to measure the quality of the core concrete if various surface processes have been used to modify the hardness of the surface, leading to a weak correlation between abrasion resistance and compressive strength.

On the other hand correlation between abrasion resistance and compressive strength may be good, even for an abrasion test that only has a slight penetration, if the surface concrete has not been subjected to hardening processes. For example, where both the surface and core concrete is soft, or where both are hard, correlation should be good.

Notwithstanding, the choice of abrasion test should ideally match the expected depth of penetration in the field. In dam spill-basins a depth of 25mm or more can easily still be considered the surface zone, while in some industrial floors where dust may be unacceptable, even abrasion wear less than 1mm may constitute failure.

2.1.4 'Core' and 'Surface' Abrasion Resistance

The various processes and factors affecting the correlation between abrasion resistance and compressive strength were discussed in 2.1.1. In 2.1.2 some related terminology was proposed such as 'mild' versus 'severe' abrasion, 'shallow' versus 'deep' abrasion wear. In 2.1.3 the influence that the type of abrasion test has on the correlation was discussed. The relationships between these various concepts are now illustrated in figures 2.3 through 2.5 and table 2.1.

Figure 2.3 shows the several factors that affect the morphology of the 'surface zone paste', and how this influences 'aggregate/paste bond' and 'paste hardness'. These, together with 'aggregate hardness' determine the 'surface' abrasion resistance. To measure this a 'mild' abrasion test that results in 'shallow' abrasion wear (less than 1mm) should be used.

Similarly, figure 2.4 shows the several factors that affect the morphology of the 'core' paste, and how this influences 'aggregate/paste bond' and the paste's 'hardness' and 'strength'. These, together with the aggregate's hardness and toughness determine the 'core' abrasion resistance. To measure this a 'severe' abrasion test that results in 'deep' abrasion wear (more than 5mm) should be used.

Figure 2.4 also indicates that the factors governing compressive strength are very similar to those for 'core' abrasion resistance, although 'strong' rather than 'hard' or 'tough' is the requirement for the aggregate. There is thus a strong likelihood of a good correlation between 'severe' abrasion resistance and compressive strength, as both essentially measure the quality of the core concrete, albeit in different ways.

Clearly there are four possible combinations of hardness for 'surface' and 'core' concrete, and these are shown in table 2.1, as type (a) through type (d). The table also shows that a 'mild' abrasion test gives different information relative to a 'severe' abrasion test, depending on the type of concrete. It also predicts that where a difference in hardness exists between 'surface' and 'core' concrete, the 'mild' abrasion test is unlikely to correlate well with compressive strength, whereas a 'severe' abrasion test may correlate 'reasonably' well, or even well, as it has the ability to penetrate the hard 'surface' concrete and measure the abrasion resistance of the core concrete below.

It is therefore essential to have a clear understanding of the degree of severity of the abrasion test *relative* to the hardness of the surface and core concrete. A concrete of average hardness may appear to be soft when subjected to an extremely severe test, while the same concrete may appear to be hard under the action of a mild test. Therefore when assessing abrasion resistance, both the severity of the test and the hardness of both the surface and the core concrete must be considered together, as has been done in table 2.1, and in a graphical format in figure 2.5. Both presentations show the eight possible outcomes for the correlation between compressive strength and abrasion resistance; in five cases the correlation is 'good', in two cases it is 'poor' while in the last instance it may either be 'reasonable' or 'good'.

Note that in figure 2.5 a change in the hardness of the concrete is indicated by a change in the gradient of the line, e.g. in figure 2.5(c) the gradient increases as the hardness changes from 'hard surface zone' to 'soft core'. Conversely a straight line, where the line crosses the transition line from the surface abrasion zone to the intermediate abrasion zone, indicates that there is no change in hardness, and in such cases the correlation between abrasion resistance and compressive strength should be good. This is seen to be the case in figure 2.5 (a) and 2.5 (b).

A consideration of the various combinations of 'surface' and 'core' hardness, together with the two degrees of severity of abrasion test, as illustrated by table 2.1 and figure 2.5, is essential when attempting to explain the correlation or absence of correlation between abrasion resistance and compressive strength.

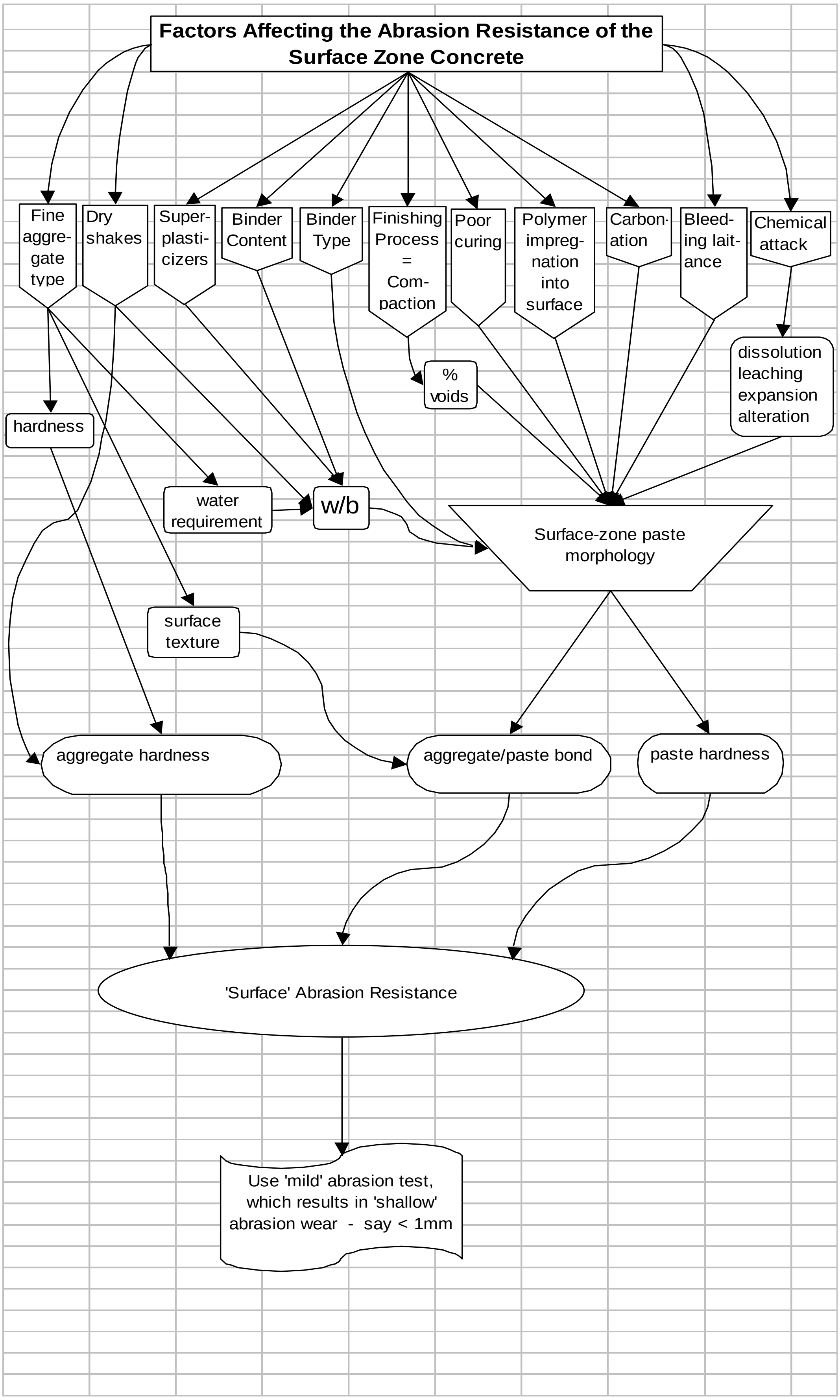

Figure 2.3 Factors affecting surface abrasion resistance

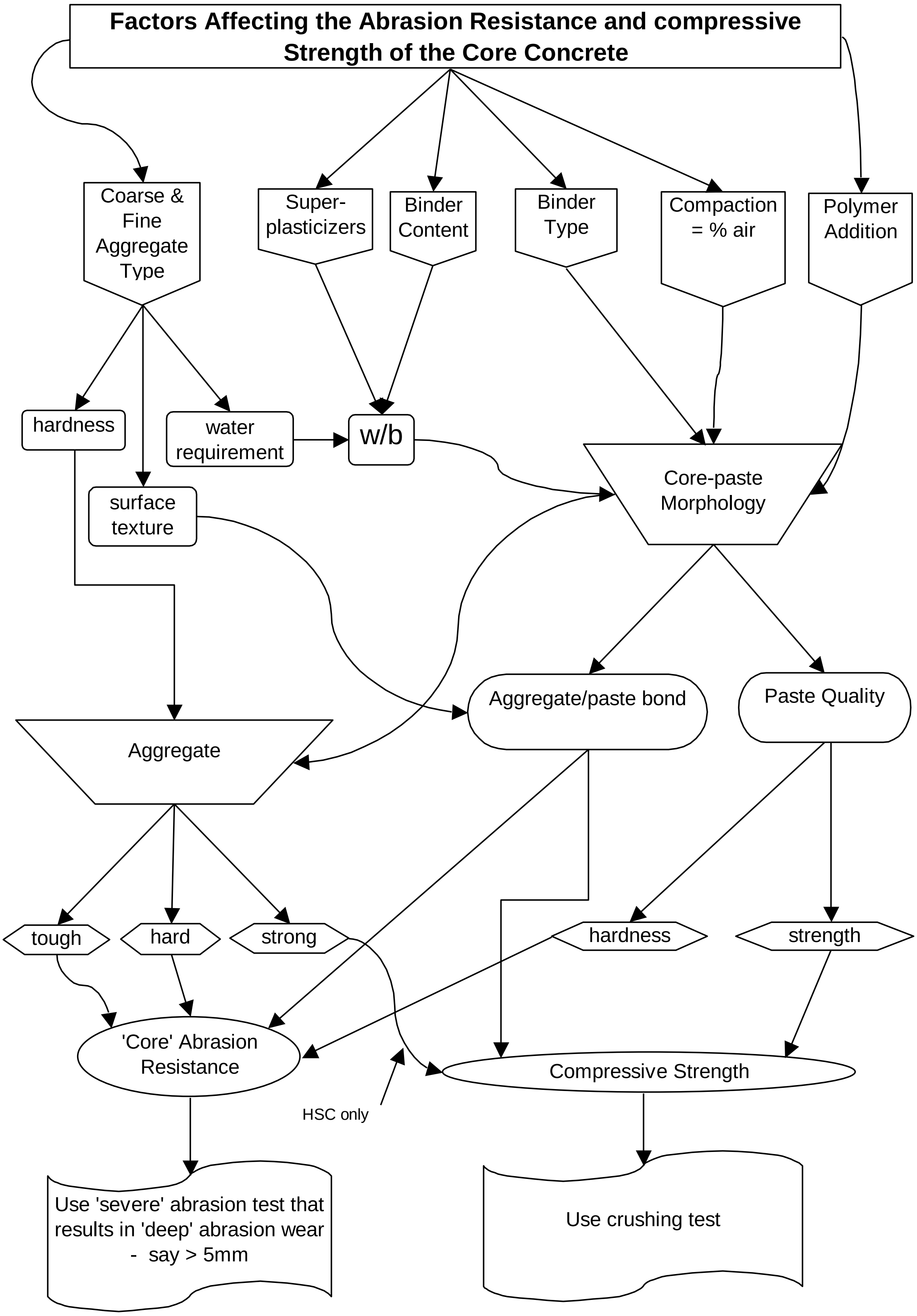

Figure 2.4 Factors affecting core abrasion resistance and compressive strength of the core concrete

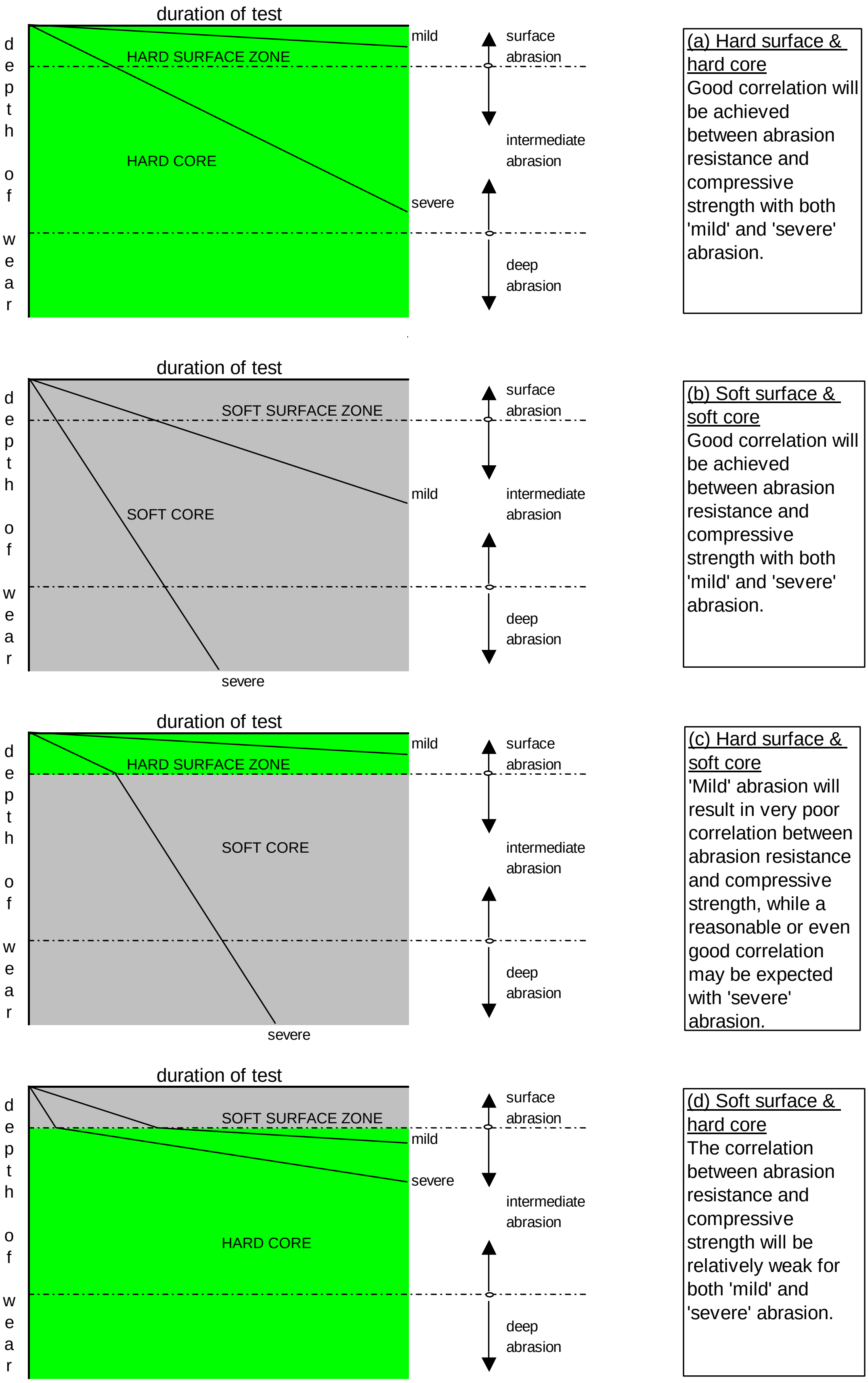

Figure 2.5 Relationship between depth of wear and duration of test, for variations in hardness with depth, as well as severity of abrasion.

Table 2.1 Interpreting Abrasion Wear in the Light of Concrete's Surface and Core Hardness, and Type of Abrasion Test

SIM-ULA-TION	HARDNESS OF -		ABRASIVE ACTION OF TEST		Anticipated Correlation* Between Abrasion Resistance ('Ia') and Compressive Strength ('MPa') for:	
	Surface	Core	'Mild' abrasion	'Severe' abrasion	'Mild' abrasion	'Severe' abrasion
A	Hard (1)	Hard (3)	Surface zone is not penetrated ⇒ a hard surface	Surface zone is penetrated, and core is abraded	Ia test ⇒ ↑ Ia MPa test ⇒ ↑ MPa ∴ Good Correlation	Ia test ⇒ ↑ Ia MPa test ⇒ ↑ MPa ∴ Good Correlation
B	Soft (2)	Soft (4)	Surface zone is penetrated, and core is abraded	Very deep core penetration	Ia test ⇒ ↓ Ia MPa test ⇒ ↓ MPa ∴ Good Correlation	Ia test ⇒ ↓ Ia MPa test ⇒ ↓ MPa ∴ Good Correlation
C	Hard (1)	Soft (4)	Surface zone is not penetrated ⇒ a hard surface	Surface zone is penetrated, and core abrades rapidly	Ia test ⇒ ↑ Ia MPa test ⇒ ↓ MPa ∴ Poor Correlation	Ia test ⇒ ↓ Ia MPa test ⇒ ↓ MPa ∴ Reasonable or Good Correlation
D	Soft (2)	Hard (3)	Surface zone is penetrated, but core abrades slowly	Surface zone is penetrated very quickly, and core abrades steadily	Ia test ⇒ ↓ Ia MPa test ⇒ ↑ MPa ∴ Poor Correlation	Ia test ⇒ ↑ Ia MPa test ⇒ ↑ MPa ∴ Good Correlation

PREFIX NOTES:

(1) A hard surface is achieved with the application of processes such as delayed power trowelling, curing; the application of liquid surface treatments or dry shakes, minimising w/b, using hard fine aggregate

(2) A soft surface is achieved by the absence or opposite of items 1 mentioned in (1), presence of bleeding laitance

(3) A hard core is achieved with a low w/b, using hard coarse and fine aggregates

(4) A soft core is obtained by the opposite of items mentioned in (3)

Note:

'Ia' = abrasion resistance

'MPa' = compressive strength

'Ia test' = abrasion test

'MPa test' = compressive strength test

Note : '*' It will not be possible to predict the correlation between abrasion resistance and compressive strength for abrasion tests that are not capable of attacking the paste constituents independently, e.g. the 'horizontal grinding table'.

2.1.5 Conflicting Views on the Relationship between Compressive strength and Abrasion Resistance

The foregoing sections will hopefully fore-arm the reader with some insight as to why the findings of different authors (reported hereafter) sometimes appear to be contradictory regarding the relationship between abrasion resistance and compressive strength.

In the references that follow, the various authors together with their arguments, are arranged in three groups according to their views. According to the first view, abrasion resistance is clearly related to compressive strength, the third view is that the relationship is weak or non existent, while the second view takes an intermediate position.

To assist the reader to apply the criteria set out in table 2.1 (or figure2.5) to the many findings referred to hereafter relating compressive strength with abrasion resistance, abrasion wear is arbitrarily classified either as <u>'shallow' for wear less than 1mm</u>, indicating a hard surface, <u>'deep' where wear exceeds 5mm</u>, indicating a soft core, or <u>'intermediate'</u>. Stating abrasion wear in this way (i.e. in terms of 'depth of wear'), necessitated some calculations where authors had reported their results in terms of volume or mass of lost material.

(a) View 1 : Abrasion resistance is related to compressive strength

The findings of a number of authors are given below who reported abrasion resistance to be related to compressive strength. It will be seen that a reasonably good relationship exists when the surface concrete has not been altered or modified relative to the core concrete by 'surface' processes shown in figure 2.3, such as power trowelling, impregnation of the surface by special polymers, carbonation, poor curing, chemical attack, bleeding laitance, etc.

Equivalence of 'surface' and 'core' concrete is modelled by both 'simulation A' and 'simulation B' of table 2.1, or alternatively by figure 2.5 (a) and 2.5 (b). In the references that follow the writer has invoked these models whenever applicable to demonstrate their ability to predict a good correlation between abrasion resistance and compressive strength.

Abrams(1921) did abrasion tests using a Talbot-Jones Rattler [= 95mm *impacting steel balls,* see appendix U.2.01; result = deep abrasion (i.e.>5mm)] and obtained <u>a very good correlation of quadratic form between depth of wear and compressive strength</u>. For example, *depth of wear* was 10mm for a compressive strength of 41MPa, and 50mm for a compressive strength of 24MPa.

[The good <u>correlation</u> that Abrams reported may be attributed to the significant depth of wear generated in this test. Relative to the severity of the test, the hardness of *both the 'surface' and 'core' may be classified as soft*, and the situation described in table 2.1 under 'simulation B' for severe abrasion applies, or alternatively the relationship that is shown as 'severe abrasion' in figure2.5(b)].

Ghafoori(1999) reported on abrasion tests according to ASTM C779 Proc C [= *rolling steel balls,* see appendix U.2.12; result = shallow abrasion mostly (<1mm)] to determine the effect of replacing a proportion of the fine aggregate with silica fume. Four levels of fine-aggregate replacement were considered i.e. 5%, 10%, 15% and 20%. He found that these replacements <u>improved abrasion resistance slightly more than compressive strength</u>, e.g. for a 10% silica fume replacement of fine aggregate, abrasion resistance improved by 57% while compressive strength improved by 49%.

[Ghafoori's good correlation between abrasion resistance and compressive strength may be attributed to the '*surface' concrete used in the abrasion test being equivalent to the 'core'* concrete used in the compression test. This was achieved by ensuring that all specimens (prisms and cylinders respectively) were cured by immersion in water for the full period prior to testing. Clearly there were no special finishing processes done to the surface concrete relative to the core concrete. 'Simulation A', 'mild abrasion' of table 2.1 best fits here].

Sukandar(1993) did abrasion tests according to ASTM C779 Proc C [= *rolling steel balls,* see appendix U.2.12; result = shallow abrasion mostly (<1mm)] on concrete pavers having a:c ratios varying from 3 to 9. The abrasion resistance and compressive strength were not affected equally by these variations. Blocks made with the higher cement contents had substantially more abrasion resistance, with a 300% improvement in the 455 kg/m^3 mix relative to the 153 kg/m^3 mix. The difference for compressive strength testing, done on the same mixes, was only 96%. However the relationship between compressive strength and abrasion resistance correlated well with a quadratic expression such that R^2 = 0,98.

[No special finishing processes were used that might differentiate the *core from the surface concrete.* The only possible difference between the bulk and the surface may have been that the core would have dried out somewhat slower than the surface, as 'air' curing was used. Evidently this did not make much difference, possibly because the blocks were relatively porous. The abrasion described here may be considered as 'mild', 'simulation A' for the low a:c mixes and 'mild', 'simulation B' for the high a:c mixes. Both indicate good correlation between abrasion resistance and compressive strength].

Dhir(1991a) did abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06; result = shallow/intermediate abrasion] and found that 28-day and 90-day abrasion resistance was proportional to compressive strength and this relationship held for all his variables. His variables were two curing regimes (RH = 55%, water immersion) and three fly ash substitutions (0%, 15%, 30%). Generally the transition between 'intermediate' and 'shallow' abrasion (i.e. at 1mm penetration) was in the region of 30MPa, while penetrations were approximately 2,5mm for the 20MPa mixes and 0,6mm for the 60MPa mixes.

[The good correlation between abrasion resistance and the high strength water cured specimens is expected, as there were no special surface effects to differentiate between 'surface' and 'core' concrete. Depth of abrasion was generally less than 1mm, and 'simulation A, mild abrasion' therefore best characterises these results. Similarly, good correlation was also obtained for the low strength air cured specimens, even though the air curing would have detrimentally affected the surface to a greater extent than the core. However, it appears that air curing did not dominate the relationship, and as there were no other special surface effects to differentiate between 'surface' and 'core' concrete, 'simulation B, mild abrasion' and figure 2.5(b) are the most likely representations].

deAlmeida(1994) did abrasion tests using the Dorry Hardness apparatus [=*sliding fine abrasive*, see appendix U.5.01; result = shallow abrasion]. His results show that a decrease in compressive strength is matched almost perfectly by a proportional increase in wear. For example, a decrease of 20% in compressive strength was followed by an increase of 21% in abrasive wear in the mix where no cement replacement was made. Similarly closely matched percentages were obtained for mixes with 10% substitutions with fly ash and silica fume. Note that in each case the author used only two points to establish the relationship between abrasion resistance and compressive strength.

[deAlmeida's samples sized 60mm x 60mm x 25mm, and were all obtained from the interior of 150mm concrete cubes by saw cutting, after the cubes had been water cured for 28 days. Both the saw cutting and the curing would have ensured that the 'surface' concrete was identical to the 'core' concrete, and this explains the good correlation between abrasion resistance and compressive strength. Furthermore, the maximum wear of 0,5mm (corresponding to a minimum compressive strength of 60 MPa) indicated that these 'surfaces' were hard, particularly in relation to the abrasion test used by deAlmeida, where the action is one of steady grinding with no impact effects. 'Simulation A, mild abrasion' from table 2.1, or figure 2.5(a) is appropriate here].

Fwa(1989) did abrasion tests using a LA machine [=*impacting steel drum*,, see appendix U.1.01; result = deep abrasion]. He achieved losses in mass up to 70%, and concluded that abrasion resistance is 'proportional' to compressive strength, regardless of the curing or weathering regime.

[Clearly, an abrasion test that can penetrate this deeply into the 'core' concrete is very likely to correlate strongly with compression testing. Given the very severe abrasive conditions relative to the abrasion resistance of the concrete, 'simulation B, severe abrasion' is appropriate here].

Gjorv(1990) did abrasion tests using NORCEM's abrasion machine [=*rolling studded tyres,* see appendix U.3.03; result = intermediate/deep abrasion] and found that abrasion resistance was proportional to compressive strength for high strength concretes between 50MPa and 100MPa. [The well cured specimens were not subject to any special surface finishes, so that the 'surface' concrete can be expected to be very similar to the 'core' concrete, and this accounts for the good correlation between abrasion resistance and compressive strength. The high compressive strengths and corresponding low rates of wear (approximately 1mm per 10000revolutions for dry 60 MPa concrete), but with deep eventual penetration suggests that 'simulation A, severe wear' is the most appropriate model].

Helland(1995) reports on 130 MPa concrete (cylinder compressive strength!) being used as a 35 mm deep inlay to repair an existing 55MPa concrete road in Norway that had been severely rutted by tyres with steel studs. Accelerated abrasion was by means of NORCEM's abrasion machine [=*rolling studded tyres,* see appendix U.3.03]. The abrasion resistance of the high strength concrete was comparable to that of massive granite. Following the tests it was possible to predict that the new surface would have four times the life of the old, [indicating that excellent abrasion resistance is achieved with very high strength concrete. This good correlation between high abrasion resistance and high strength concrete under severe abrasion is best modelled by 'simulation A, severe abrasion', and by figure 2.5(a) – see 'severe line'].

Liu(1981) did abrasion tests according to (the forerunner of) ASTM C1138 [=*impacting steel balls* (mild), see appendix U.2.07; result =intermediate abrasion] and found that for a given aggregate type, abrasion loss increased when compressive strength reduced. The rate of wear was a function of both aggregate type and compressive strength.

[This seems to indicate that both *aggregate/paste bond* (assuming that it is a function of compressive strength) and aggregate *hardness* are important. Figure 2.3 and 2.4 show that these two attributes affect both 'surface' and 'core' abrasion].

Liu(1991) stated that in the absence of special finishing abrasion resistance is proportional to compressive strength.

Meyer(1980) explains that in 1964 a standard for concrete pavers was introduced in Germany, DIN 18501. This standard required a compressive strength of 60 MPa and called for both freeze/thaw and abrasion testing. However, experience over the years showed that 'the high compressive strength also resulted in adequate resistance to freeze/thaw effects and abrasion resistance'. [He does not mention what the abrasion test was, presumably a sliding fine-abrasive test?]. Meyer advocated that the ' earlier requirement regarding wear should be dispensed with because it has been shown that all concrete paving stones with a compressive strength of over 60 MPa are sufficiently wear resistant '. In other words a good correlation exists between abrasion resistance and compressive strength for high strength concrete pavers. (This may be expected, as nearly all paving done in the early 1960s and earlier was done as a single layer, unlike paving today which is often manufactured with a separate 'face concrete' or topping, that may or may not have the same properties as the core concrete). Assuming the relatively mild sliding fine-abrasive abrasion test, 'simulation A, mild abrasion' from table 2.1, or figure 2.5(a) best models this situation.

Naik(1994) did abrasion tests according a modified version of ASTM C944 [=*sliding fine abrasive beneath* rolling dressing wheels, see appendix U.3.09; result = intermediate abrasion] on concretes with 0%, 50% and 70% fly ash replacement levels. He found that the abrasion resistance was proportional to compressive strength. Respectively 28-day compressive strengths were 43MPa, 32MPa, and 18MPa, while depth of wear was 2.1mm, 2.8 mm, and 3.6 mm.

[No special finishing materials or processes were used that could have made the surface harder than the 'core' concrete in the specimens. Curing was carried out at maximum humidity. Therefore given that the surface concrete was essentially the same as the core concrete, it is not surprising that the abrasion resistance (which may be defined as the inverse of depth of wear) decreases with decrease in compressive strength. Given that the abrasion depth went substantially beyond the shallow zone threshold into the intermediate zone, 'simulation A, severe abrasion' is an appropriate model for the 43MPa concrete, while 'simulation B, mild abrasion' is appropriate for the softer 18 MPa concrete].

Prior(1966) did a state of the art review of abrasion resistance, considering various tests including a reciprocating plate with abrasive, rolling dressing wheels, rotating discs with abrasive, rolling steel balls, and impacting fine abrasive. He concluded that adequate compressive strength was very important for abrasion resistance, and conversely that low compressive strength resulted in poor abrasion resistance.

Tangtermsirikul(1997) did abrasion tests using an apparatus similar in principle to ASTM C779 Proc C [= *rolling steel balls,* see appendix U.2.12; result =intermediate abrasion] on roller compacted concrete, with fly ash replacement levels ranging between 0% to 50%. He found that depth of wear was reduced with increasing compressive strength in the range 28Mpa (2,5mm) to 70Mpa (1,0 mm), a straight line relationship being a good representation of the data. The author does not give any indication of his experimental procedure, and it is therefore only assumed that no special finishing process was used that may have enhanced the surface relative to the core.

(b) View 2 : Abrasion resistance is poorly related to compressive strength

The findings of a number of authors are given below who reported a poor relationship between abrasion resistance and compressive strength. A poor relationship exists when the surface concrete has been altered or modified relative to the core concrete, by 'surface' processes mentioned in figure 2.3, such as power trowelling, impregnation of the surface by special polymers, carbonation, poor curing, chemical attack, bleeding laitance, etc. In the references that follow the writer has referred to the particular 'surface' process/processes, whenever applicable, to show how this has altered the surface. Non equivalence of 'surface' concrete relative to 'core' concrete is modelled by both 'simulation C' and 'simulation D' of table 2.1, or alternatively by figure 2.5 (c) and 2.5 (d), and reference to these models is made where applicable.

a'Court(1954) did abrasion tests using a reciprocating steel pan [=sliding fine-abrasive, see appendix U.5.17; result =shallow abrasion] and found that 'that there is no very clear relation between abrasion resistance and strength as measured by cube tests', since abrasion resistance is a '*surface characteristic'*. He suggests that the abrasion resistance deeper in the specimen may in fact be related to compressive strength. [a'Court's surfaces appear to have been hand floated, air cured, and made with a relatively high w/c, which may have resulted in some bleeding. All these processes would have had a far greater effect on the 'surface' relative to the 'core', resulting in a poor correlation between abrasion resistance and compressive strength. 'Simulation D, mild abrasion' appears to be the most appropriate model. or see figure 2.5(d).

Beningfield(1995) did abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06; result = shallow abrasion] on polymer modified concrete. He found 'little direct correlation' between the abrasion resistance and corresponding compressive strength of specimens incorporating *different polymer types, or slightly different w/c ratios within a polymer type.* In some cases these differences were as much as an order of magnitude. Beningfield does not offer any explanation for the substantial disparities between abrasion resistance and compressive strength.

Bettencourt Ribeiro(1998) did abrasion testing on roller compacted concrete using an apparatus similar to DIN 52108 [*sliding fine-abrasive,* appendix 5.02; result = intermediate abrasion]. He found virtually no relationship between abrasion resistance and compressive strength. He concluded that abrasion resistance was a function of aggregate type (hardness).

[Figure 2.3 shows that the hardness of the surface aggregate affects 'surface abrasion'. This is particularly the case with the abrasion test used by Bettencourt. It does not have the ability to dislodge the relatively hard aggregate particles by preferentially attacking the paste constituent, as is the case with most other abrasion tests, and hence aggregate hardness controls the rate of abrasion. On the other hand aggregate hardness plays no part in compressive strength (see figure 2.4), which is governed by aggregate/paste bond. The lack of correlation between abrasion resistance and compressive strength in Bettencourt's work is thus explained].

Dreijer(1980) reported on research and experience in the Netherlands, where strength testing [in this case flexural testing] was unable to identify an inferior surface, caused by such aspects as *poor curing, improper plasticity [resulting in air voids] of a weak topping mix*. [The 'topping' is a thin skin of mortar (often pigmented) which is placed over the base concrete in a paver prior to final compaction].

Figure 2.3 shows that curing and a high voids content in the topping affect 'surface abrasion', while their affect on compressive strength will be minimal, thus explaining the poor correlation.

Holland(1991) did abrasion testing on concrete pavers using rolling steel balls, sliding wire bristles, and impacting fine abrasive (see appendices U.2.15, U.6.02, U.5.21 respectively). The corresponding depths of abrasion may be classified as [=shallow abrasion/intermediate], [=shallow abrasion/intermediate] and [=intermediate abrasion]. The results of the abrasion tests were compared with compression tests on companion blocks. He concluded that the 'compressive strength test is not a good indicator of abrasion resistance'.

[It is important to appreciate that Holland used blocks from different suppliers made in different production environments. As such there would be variations in *% voids, aggregate* type, *cement type and quantity, production processes*, and *curing* regimes. Furthermore the blocks had been in service for different periods and were therefore of different *age*, with corresponding different degrees in *carbonation* and curing, from rain. It is therefore not surprising to see that his abrasion resistance values did not correlate well with compression testing. In particular, aspects such as curing, carbonation and the hardness of the fine aggregate would influence abrasion resistance far more than compressive strength, resulting in a poor correlation].

Humpola(1996b) did abrasion testing using rolling steel balls and impacting steel balls (see appendices U.2.13 and U2.03). He observed that whereas compressive strength is a measure of the strength of the whole unit, the abrasion resistance is only influenced by the top 3mm. He rationalised where dissimilar curing conditions exist there is little chance to establish any reasonable relationship between abrasion resistance and compressive strength.

Kettle(1987b) reported on many abrasion tests made using an apparatus developed by the C&CA [=rolling steel wheels, see appendix U.4.06; result = shallow abrasion] on both new and old industrial floor slabs. The results demonstrate that a higher specified concrete strength does not necessarily increase the abrasion resistance.

[Assuming that the floors Kettle investigated had been power trowelled (most 'industrial' floors are), his results are not surprising. Chaplin(1991), Kettle(1984) showed that even a low grade concrete that has been *power trowelled* will have a surface that has a high abrasion resistance. This would correspond to 'simulation C, mild abrasion' or figure 2.5(c). Moreover figure 2.3 shows that ***power trowelling*** affects 'surface abrasion', while figure 2.4 shows that it does not affect 'core abrasion' and compressive strength, accounting for the unpredictable correlation between abrasion resistance and compressive strength].

Papenfus(1993b) observed that in practice the quality of the surface may deviate significantly from the overall quality of the block as measured in a compression machine. Compression testing measures the bulk strength of the concrete, while an abrasion test measures the quality of the surface. In particular there are four aspects in the manufacturing process where compression testing is an ineffective means of quality control, but where an abrasion test would be most effective:

(i) Topping concrete

Topping concrete is increasingly being used in the production of concrete pavers. If the topping concrete was made from a dry mix, or insufficient binder content, or inadequate compaction, this would not register in a compression test. The 5mm to 10mm topping layer may fail prematurely, but because it is relatively thin and is trapped between the platen and the base concrete, it is largely held in position by frictional effects and the dial of the compression machine gauge will continue to rise until the base concrete fails. Thus a superior topping will not be recognised, while an inferior surface will 'pass' undetected.

(ii) Surface drying

Premature drying of the surface from inadequate curing, will affect the surface far more detrimentally than the bulk of the block. Once again the compression test may indicate that the quality is acceptable, whereas an abrasion test would reveal that the blocks had not been cured.

(iii) Aggregate hardness

The hardness of the aggregate is not readily measurable in a compression test, but plays an important part in the wear resistance of the surface.

(iv) Carbonation

The reaction between the CO_2 in the air and the $Ca(OH)_2$ in the pores of the surface concrete produces $CaCO_3$, a much harder and more stable compound. The accompanying hardness of a carbonated surface can be substantial and has been reported by Papenfus:1995. Again the benefits of this surface hardening effect benefit the abrasion resistance far more than the compressive strength.

Prior(1966) did a state of the art review of abrasion resistance, considering various tests including a reciprocating plate with abrasive, rolling dressing wheels, rotating discs, rolling steel balls, and impacting fine abrasive. He concluded that *adequate compressive strength was very important for abrasion resistance, and conversely that low compressive strength resulted in poor abrasion resistance* [reported earlier in section (a)]. There was however a critical compressive strength, ranging between 30 MPa to 40 MPa, which if attained resulted in high abrasion resistance. Any improvement beyond this critical MPa made little further improvement in abrasion resistance.

[The above findings indicate a threshold value for compressive strength, below which abrasion resistance seems to be related to compressive strength, while for strengths exceeding 40MPa the correlation is weak. This can be explained in terms of paste/aggregate bond and aggregate hardness. If the paste/aggregate bond is weak, then the aggregate particles are plucked out of the paste matrix, resulting in substantial abrasion wear. Conversely if the paste/aggregate bond is strong, the aggregate particles, that are generally much harder than the paste, are held in place, protecting the paste as they resist the abrasive forces. Therefore, *aggregate hardness* governs abrasion wear in relatively strong concretes, while aggregate/paste bond governs weaker concretes. Figure 2.3 shows that ***hardness*** of the fine aggregate affects 'surface abrasion', while figure 2.4 shows that it does not affect compressive strength, although ***aggregate/paste*** bond does. The different degrees of correlation between abrasion resistance and compressive strength for stronger and weaker concretes are thus explained].

Robertson(1991) analysed a series of tests done on concrete pavers and found reasonably good universal correlation between compressive strength and tensile splitting strength, for samples that were selected from different manufacturers to correspond with equivalent blocks installed at ten different sites. On the other hand only limited correlation was obtained between compression testing and the four different abrasion tests done respectively using *impacting fine abrasive* [see appendix U.5.21; result =intermediate abrasion], *rolling steel balls* [see appendix U.2.15; result = intermediate abrasion], *sliding wire bristles* [see appendix U.6.02; result = intermediate abrasion], *rolling dressing wheels* [see appendix U.3.09; result = intermediate abrasion]. He rationalised that compressive strength is a measure of the *shear strength*, aggregate interlock and *aggregate paste bond* of the body of the material in which the properties of the block surface (which determine abrasion resistance) will have little effect. Conversely variations in curing and *environmental conditions* affect the surface far more than the body, accounting for the variable correlation between abrasion resistance and compressive strength.

[This principle is further demonstrated by figure 2.3, which shows that ***environmental conditions*** and ***curing*** affect 'surface abrasion', while figure 2.4 shows that they do affect the 'core' concrete and hence the compressive strength to the same extent.]

Sadegzadeh(1986) did parallel testing on various concrete surfaces using an *impacting* rebound hammer [see appendix U7.02] and cube crushing tests from the same mixes. He stated that it is unwise to use the Impact rebound hammer to estimate the compressive strength of concrete, since this device is sensitive to aspects that mainly affected the surface of the concrete, such as power trowelling, the application of dry shakes, and curing. (By this he is inferring that it is possible for a high strength concrete to have a poorly finished and therefore inferior surface, and conversely for a relatively low strength concrete to have a hard and durable surface, given the correct *finishing*). He also referred to the case hardening effect of *carbonation* on *the surface, which will effect the rebound hammer and not the compressive strength. Given the rebound hammer's ability to discern surface effects, it appears that it will* correlate *better with abrasion resistance than compressive strength.*

Shackel(1993a) used rolling steel balls and impacting steel balls [see appendices U.2.13 and U2.03, respectfully obtaining shallow/intermediate abrasion and shallow abrasion] in several investigations, and found that abrasion resistance could be weakly correlated to compressive strength for moist cured specimens, but no correlation could be found for *dry cured* specimens.

[As before, figure 2.3 shows that ***curing*** is one of the factors that affects the 'surface' concrete, while the 'core' concrete, which determines compressive strength (see figure 2.4) is less affected by bad curing practices. Therefore abrasion resistance can be expected to correlate poorly with compressive strength where curing has been neglected].

Simon(1999) did abrasion testing using a drum sander [sliding fine-abrasive, see appendix U.5.18; result = intermediate abrasion] on high strength concrete. Results showed that whereas a definite relationship exists between compressive strength and w/c ratio, abrasion resistance was almost constant for all the mixes for the full range of compressive strength between 32 MPa through 71 MPa.

[Once again this may be explained by the particular characteristics of the abrasion test adopted, which gently grinds its way into the specimen rather than exert impact or high localised compressive loads, (that tend to dislodge the aggregate). The high strength *igneous aggregate*, which was the same for all mixes, therefore controlled the rate of abrasion, completely masking variations in paste strength. (Variations in strength paste, on the other hand, primarily determine compressive strength). As long as the binder is strong enough to adequately anchor the aggregate particles, and 32 MPa concrete (the lowest) certainly would be, the aggregate will remain bonded and control the rate of abrasion.

Note further that figure 2.3 shows that ***aggregate hardness*** affects 'surface abrasion', while figure 2.4 shows that it does not affect compressive strength, thus accounting for the poor correlation].

Webb(1996) made abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels*, see appendix U.4.06; result = shallow abrasion] to test the abrasion resistance of *power finished* concrete surfaces made of various coarse and fine magnesian limestone aggregates, and found no clear correlation with compressive strength.

[Clearly power finishing hardens the surface and not the core concrete. 'Simulation C, mild abrasion' models this situation and accounts for erratic correlation with abrasion wear].

(c) View 3 : Abrasion resistance approximately follows compressive strength

In this section a number of authors are quoted to show that while there may sometimes be a general trend between abrasion resistance and compressive strength, the relationship is not necessarily clear. This may therefore be considered a grey area in between the findings discussed under (a) and those of (c). Given the middle position of these findings, it is not always possible to identify the factors that led to the mediocre correlation between abrasion resistance and compressive strength. It may even be that some opposing factors from either side cancel each other out. The findings of the authors will therefore be quoted, mostly without comment.

Connell(1985) performed abrasion tests according to BS 812:Part 3:1975 clause 9 [=*sliding fine-abrasive*, see appendix U.5.06; result = intermediate abrasion]. He concluded that '*compressive strength does not appear to give consistent answers but generally an increase in compressive strength will result in an increase in abrasion resistance*'. A marked reduction in abrasion wear for concrete specimens with strengths above 45 MPa was evident.

[The most likely explanation here is that at higher compressive strengths (above 45MPa in this case) the aggregate/paste bond is sufficient to substantially limit the plucking out of the aggregate from the matrix, resulting in a much reduced rate of abrasion wear. This done, the aggregate takes the brunt of the abrasive forces and so doing *protects* the softer paste, allowing it in turn to fulfil its chief function of cementing the aggregate particles].

Doulgerous(1996) did abrasion tests on high strength concrete (>50MPa) according to PCI.TM.7.8 [25mm *impacting steel balls*, see appendix 2.02; result =intermediate abrasion]. Generally correlation with compressive strength was 'fair'. (Above 70MPa correlation was poor). [The same explanation given for Connell above is probably applicable here].

Naik(1995) did abrasion tests using a modified version of ASTM C944 [=*sliding fine abrasive* beneath rolling dressing wheels, see appendix U.3.09; result = intermediate abrasion] on concretes with 0%, 50% and 70% fly ash replacement levels and found abrasion resistance to be reasonably proportional to compressive strength across a fairly wide range of % air entrainment (R^2 = 0,82). The abrasion resistance specimens were uniformly finished and cured at 100% RH for 28 days, reducing the possibilities of other variables distorting the result.

Sawyer(1957) did abrasion tests according to DIN 51951 [*rolling steel balls*, see appendix U.2.09; result = intermediate/deep abrasion] and found that abrasion resistance was related to compressive strength in a quadratic fashion, such that the rate of *abrasion reduced with increasing compressive strength*. A *degree of scatter* was observable in the results used to plot this relationship. He noted however that compressive strength alone cannot take into account some other factors that affect the wear resistance of the surface.

Scripture(1954) used *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14; result = shallow abrasion] to do tests on specimens with different aggregates and found that the abrasion resistance was a function of compressive strength rather than aggregate type. Some scatter in the results was observable. [It is questionable to what extent the hardness of the aggregate contributed to the results, given that the depth of wear was of the order of 0,3mm, and at this depth it is doubtful that the aggregate would have been engaged to any significant degree].

Smith(1958) investigated the influence of various coarse and fine aggregates on the abrasion resistance of the corresponding concretes using three abrasion tests, (1) *rolling steel balls*, see appendix U.2.11, result = shallow/intermediate abrasion; (2) *impacting fine abrasive*, see appendix U.5.20, result = intermediate abrasion; (3) *rolling dressing wheels over fine abrasive*, see appendix U.3.08, result = intermediate abrasion. The correlation between abrasion resistance and compressive strength was reasonable for the rolling steel balls and rolling dressing wheels / sliding fine abrasive, and good for the impacting fine abrasive test. (Correlation between abrasion resistance and w/c however, was considerably better).

Horiguchi(1994) did crushing and abrasion testing on companion specimens using *rolling steel balls* - (BB - see appendix U.2.12), *rolling dressing wheels* – (DW – see appendix U.3.06, and also RC – see appendix U.3.09), and *bouncing steel balls* in water (SF - see appendix U2.06). His results are reproduced in figure 2.A below.

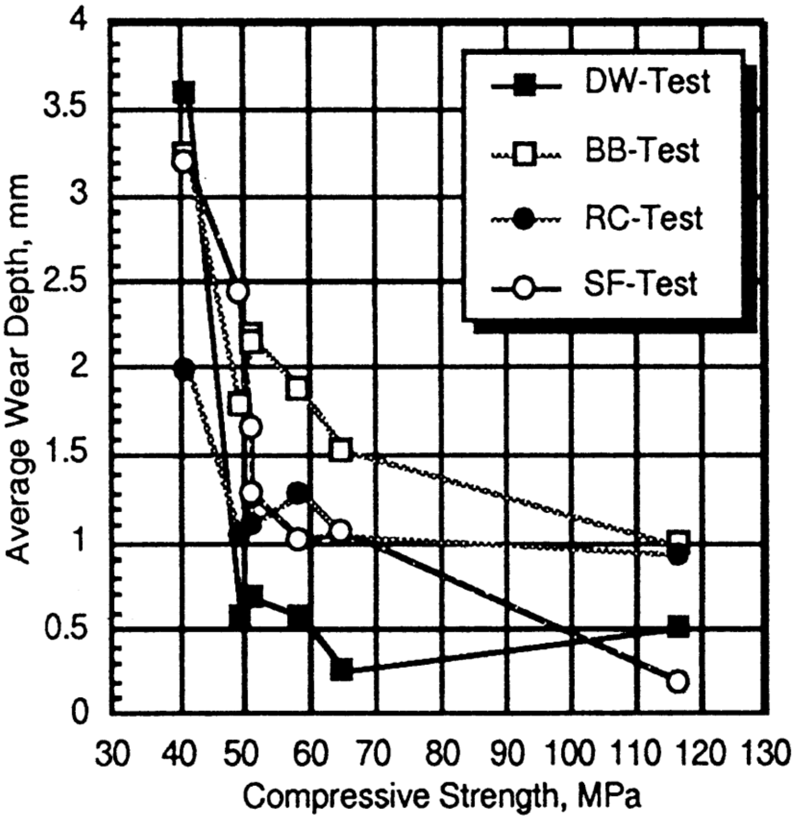

Figure 2.A Relationship between the average wear depth and compressive strength.

All the tests show a rapidly increasing depth of wear as compressive strength decreases below 50 MPa. On the other hand it may be seen that above compressive strengths of 50 MPa abrasion-wear remains essentially constant. This indicates an MPa threshold at which a sufficiently strong aggregate paste bond is achieved, preventing dislodgement, and therefore allowing the aggregate to dictate the rate of wear without regard to further increases in compressive strength. (Only the BB test has the ability to detect increases in abrasion resistance above 50 MPa).

2.1.6 Summary and Conclusions

It has been shown that the relationship between abrasion resistance and compressive strength can be strong, intermediate or weak. This serves as a warning that abrasion resistance may only be gauged from compressive strength in certain situations i.e. where the surface zone concrete has not been modified by any one of many possible processes.

Notwithstanding, the apparent controversy and complexity can be resolved by using of a number of concepts such as ‘hard’ and ‘soft’ ‘surface‘ and ‘core’ concrete, ‘deep’ and ‘shallow’ wear, ‘mild and ‘severe’ abrasion, and by understanding the various factors that affect both ‘surface’ and ‘core’ abrasion (see figure 2.3 and 2.4).

It has been shown that abrasion resistance is proportional to compressive strength when no differences exist between the ‘surface’ and the ‘core’ concrete, such as is the case in either ‘simulation A’ or ‘simulation B’ of table 2.1. This means that the curing will have been the same, no special treatment of the surface will have been effected, either in terms of paste densification by means of power trowelling, or the application of dry shakes or liquid surface treatments, or the occurrence of weathering effects such as carbonation. Furthermore the aggregate hardness should be very similar to that of the paste if good correlation is to be achieved, or if it is not, an abrasion test should be used that can abrade the paste preferentially, and thus nullify this aspect (e.g. sliding wire bristles test, sandblast test), so that the aggregate is neutralised in the abrasion test as it is in crushing tests. (In crushing tests, the physical hardness of the aggregate only plays a part where the concrete has very high strength. Normally it is the aggregate/paste bond that fails first).

On the other hand, where substantial differences occur between surface and core concrete, such as modelled by 'simulation C' or 'D', the correlation between compressive strength and abrasion resistance is relatively poor.

Several investigators found a transition in the correlation. At relatively low compressive strengths there was a good correlation, while at higher strengths if was much weaker. The explanation given is that at lower paste strength (corresponding to lower compressive strengths, the aggregate is plucked out relatively easily from the paste which in itself may be substantially abraded so as to be unable to adequately bond the aggregate. On the other hand, a stronger paste (corresponding to high compressive strength) has a strong aggregate/paste bond, so that the aggregate is effectively held and withstands the abrasive forces, and in so doing protects the paste. However, the implication of this is that the correlation with compressive strength will be much weaker since compressive strength is largely governed by paste strength and aggregate/paste bond.

Finally, to end this discussion and maintain perspective, we need again to remind ourselves that abrasion resistance is a measure of the ability of the *surface* zone concrete to resist normal and tangential loads. Compressive strength on the other hand is a measure of the *core* concrete's ability to resist compressive stresses. Therefore the extend to which the morphology of the surface concrete resembles that of the core concrete will determine how closely abrasion resistance is related to compressive strength.

The type of abrasion test used to measure abrasion resistance strongly influences the result, and in particular the depth of penetration is very dependant on the severity of the test.

2.2 Paste

Introduction

In this section the factors that affect the intrinsic strength of the paste will be considered, while in section 2.5, the strength of the bond between the paste and the aggregate is considered. The latter is clearly influenced by the former.

Several investigators (e.g. Powers, Abrams) have made a study of the laws governing the strength of the cement paste. They determined that the compressive strength of concrete is strongly related to the strength of the paste, especially as this relates to % voids and w/c. It was also shown in section 2.1 that abrasion resistance is often related to compressive strength, and where this is the case abrasion resistance will also be a function of paste strength.

Abrasion resistance is a surface phenomenon, and is therefore especially influenced by the quality of the concrete at the surface, consisting of aggregate and paste. It will be shown that the morphology and abrasion resistance of the paste in this zone is substantially influenced by finishing processes such as power finishing, application of dry coats or liquid surface treatments and curing. It is also influenced by environmental factors, resulting in processes such as carbonation, dissolution, leaching, expansion etc.

Basic Definitions

Concrete is made up of coarse aggregate, fine aggregate, binder and water.

Binder may consist of pure cement, but increasingly it is a blend of cement with one or more extenders, such as fly ash, ground granulated blast furnace slag, or silica fume. (In volume1 of this thesis all four of these materials have been considered).

Paste is a mixture of water and binder.

Function of Paste

The paste is there to coat each aggregate particle during the mixing stage, and becomes the matrix into which the aggregate is in effect submersed. When it hardens, it becomes the 'glue' that surrounds and joins the aggregate together.

Influence of Paste Content

The quantity of paste required will depend on the voids content of the aggregate, and this in turn is a function of the grading, shape and relative proportioning of the coarse and fine aggregate. Too little paste makes it difficult to achieve full compaction. The resultant air voids are very detrimental to strength (see section 2.3 Voids). On the other hand too much paste is uneconomic. Furthermore, from a performance point of view, excessive quantities should be avoided because paste tends to shrink on drying out and creep under load.

Influence of Paste Content in Concrete Pavers

In concrete pavers shrinkage and creep are inconsequential. In the first instance they are made from a semi-dry mixture that shrinks minimally. Secondly, even if the unit shrinkage were similar to that of conventional concrete, the individual pavers are too small for this to have any effect. Finally, since the joints between pavers are filled with fine sand, the pavers are therefore minimally restrained in tension laterally, and are thus not subject to tensile stress build-up.

Influence of paste strength on concrete pavers

Paste strength plays an important role in abrasion resistance. Accordingly pavers may be broadly classified into three groups:

1. *Pavers with a weak paste*: In this case the paste is relatively soft and is therefore easily abraded. Furthermore, the bond capability of a weak paste is weak. Therefore the coarse and fine aggregate are easily dislodged after the paste has been abraded to some critical depth.

2. *Pavers with an intermediate strength paste*: The paste still offers a lower resistance to abrasion attack than does the aggregate, but it is soon protected quite substantially by the aggregate particles as they begin to protrude, since in this case the paste has sufficient strength to adequately bond the aggregate particles as they take the brunt of the attack. Therefore the rate of wear will depend on the abrasion resistance of the aggregate.

3. *Pavers with a high strength paste*: A strong paste is also relatively hard. Although still softer than most aggregates, it is quite resistant to abrasion and consequently the aggregate will barely protrude above the paste. In this case aggregate type is not of great significance providing it is not excessively soft, e.g. soft limestone, decomposed sandstone.

Doulgerous (1995) stated that a relatively weak cementing matrix has been observed to be the main cause of failure of concrete pavers under foot, vehicular and other forms of abrasive traffic.

Papenfus(1989a) found that the strength of the paste influenced the aesthetic stability of the pavers. As the cement paste is abraded away, progressively more aggregate is exposed, resulting in a colour shift in the direction of the aggregate's colour. At a test site subject to bus and pedestrian traffic this was already noticeable after a few months, particularly on those pavers made with lower cement content and from drier mixes having a relatively higher percentage voids. Pavers made from mixes that had higher cement contents and which were sufficiently lubricated to achieve full compaction had less surface abrasion and colour shift was not readily apparent. High strength paste will also eventually experience a colour shift, given sufficient traffic, but this will not be accompanied by the objectionable change in surface texture that low strength pavers experience. The change in surface texture comes about as the relatively soft paste component is preferentially abraded, leaving the aggregate particles protruding, resembling mini cobbles. In extreme cases this texture deterioration is exacerbated by coarse aggregate being plucked out of the matrix.

In the final analysis, the strength of the paste is a function of its microstructure, and the extent that this structure has been modified by weathering. These two aspects are discussed in 2.2.1 and 2.2.2.

2.2.1 Micro-Structure of Hardened Paste

In the foregoing section the role that paste has in abrasion resistance was discussed and it was shown that this role changes depending on the strength of the paste. Logically it follows that paste strength is governed by its micro-structure, and this is a focus of this section. It is also shown that the microstructure of the paste has a strong influence on such well known factors as w/b and %voids.

The microstructure of hardened paste is defined by characteristics such as the structure and composition of the cement gel, pore size and distribution, and total pore volume.

When unhydrated cement particles come into contact with water, a series of chemical reactions are set in motion. The calcium and silica components that predominate in cement, combine with water molecules to form new compounds referred to as calcium silicate hydrates (CSH), or simply 'hydrates'. These hydrates make up the solid part of the 'gel'. Hydrates are sub-microscopic particles separated by sub-microscopic interstices, referred to as 'gel pores'. In appearance hydrates are fibrous particles with straight edges and bundles of these fibres form a cross-linked network. Thus the CSH and the gel pores together make up the 'gel'.

In addition to gel, paste contains crystals of calcium hydroxide, some minor components, residues of the original cement, and residues of the original water filled spaces from the fresh paste. These residues of water-filled space exist in the hardened paste as microscopic interconnected channels called capillary pores, or if the structure is dense enough, as capillary cavities interconnected only by sub microscopic gel-pores.

Thus two classes of pores exist within the microscopic structure of the paste:

1. sub-microscopic gel pores, which are a characteristic feature of the structure of the gel
2. microscopic capillary pores or cavities, representing space not filled by gel or other solid components of the system

Factors that influence the gel morphology

There are four factors that influence the gel structure:

First is the relative distance between the individual cement grains dispersed in the paste and separated by water, or more correctly, the ratio of the mass of the cement to the mass of the water making up the paste. The inverse of this ratio, the w/b, is commonly used throughout the industry as an indicator of paste quality, concrete strength, abrasion resistance, etc., with low w/b ratios superior to high ratios.

This means that for low w/b ratios the resultant cementitious gel structure will be very dense and the pore structures much more refined. With the cement particles so closely packed, there is insufficient water in the spaces between the cement grains to fully convert all the hydrated cement into gel. Therefore the inner cores of the cement particles remain unhydrated, but this is not detrimental to strength.

The second factor influencing the gel structure is the % air in the paste. Generally, this will only occur when very low w/c ratios are used, such that there is insufficient water to fill the voids between the individual cement grains, or if there is inadequate compaction. Sukandar(1993) used w/c ratios as low as 0,21 in his experimental pavers. It is possible to calculate that this equates to a thickness of water around the cement particles of less than one tenth of the diameter of the cement particle, assuming that it is a round particle. In such dry pastes it is not difficult to perceive of air spaces between the individual cement grains, which may or may not fill up with the products of hydration, depending on curing, permeability etc. If these spaces in the paste remain unfilled a proportion of the potential strength of the paste will be lost.

It can also be shown that the entrainment of air into the paste for such purposes as frost resistance, etc. also reduces strength and abrasion resistance. In this case the air bubbles are not so small that they occupy the spaces between the cement particles. Rather they are surrounded by paste. In fact their stable existence in the matrix is dependant on having a shell of paste.

Thirdly, the morphology of the gel/microstructure will also be affected by the type of the binder, its physical properties and chemical composition, and its proportioning relative to the other mix constituents. Increasingly cement is being ‘extended’ by partial substitution with materials which also have hydraulic properties e.g. ground granulated blast furnace slag, and/or alternatively by materials which have pozzolanic properties, e.g. fly ash, silica fume.

Fourthly the extent that the microstructure of the paste develops is influenced by curing, especially for more permeable pastes with higher w/b ratios. On the other hand, Addis(1991) showed that even ‘over-cemented’ mortars with very low w/c ratios and very low permeabilities benefited from ongoing hydration.

Conclusion

It has been shown that the microstructure of the hardened paste is a function of four factors:

1. w/b
2. % air (or air/b)
3. binder type
4. curing

Therefore, by virtue of the contribution that paste has towards the performance of the concrete of which it is a part, these factors will also influence the characteristics of the resultant concrete. The role that w/b, binder type, and curing have on concrete, as reported by various investigators, is now discussed in sections 2.2.1.1, 2.2.1.2 and 2.2.1.3, respectively, while that of % air is discussed in 2.3.

2.2.1.1 Water Binder Ratio

In this section we shall begin by examining Power's gel space ratio and some other concepts that are useful in explaining some of the fundamentals of how w/b influences strength. This is followed by a consideration, from the findings of several investigators, of how w/b influences abrasion resistance.

2.2.1.1(a) Influence of w/b on density of microstructure

Cement gel can only be produced in water-filled capillary cavities, and, according to Powers(1958), when all those cavities become full, no further hydration of cement can occur. (This begs the question: when are those cavities full, or how full is full? It would seem from Addis(1991) that there is ongoing densification of the cavities by continuing gel development, over a long period of time, even for mixes with very low w/c ratios).

Powers reported that the strength of paste, f_c, increases in direct proportion to the cube of the increased gel-space ratio, X, defined as the ratio of the *volume of gel* to the *volume of available space*, (where the maximum ratio should not be taken to exceed unity).

The compressive strength of the paste may therefore be expressed as:

$f_c = \text{constant} \,.\, (X)^3$ -------------- (1)

Where 'constant' = a numerical coefficient which depends principally on the intrinsic strength of the gel produced by the particular binder in question.

In effect gel-space ratio indicates the degree to which the gel structures have developed into the original water filled space.

From equation (1) it is evident that if there was too much water in the original mix such that the 'available space' can never be fully filled with gel, even after all the cement has hydrated, then the denominator is larger than it needs to be. The result is a smaller gel-space ratio, translating into a paste with a weaker microscopic structure.

Conversely, if there is too little water in the original mix such that the 'available space' is fully filled with gel before the individual cement particles have hydrated through to their innermost cores, then it may be seen that all the 'available space' is filled with gel, and the gel-space ratio is therefore equal to unity.

Addis(1991) confirms these concepts by introducing the concept of cement saturation. He speaks of a cement-saturated mix as one where volume of gel (solid gel volume + gel pore volume) is equal to the sum of the original absolute volumes of cement and water. He refers to a cement-undersaturated mix as having too little cement relative to initial water, so that the gel does not fill the original space, even when fully hydrated, resulting in some capillary pores with no gel. Conversely a cement-oversaturated mix is one where there is too much cement relative to the available water, so that gel development is terminated by all capillary pores being filled with gel before the cement is fully hydrated.

Furthermore, whereas the properties of cement under-saturated pastes are dominated by porosity, especially capillary porosity; those of cement-oversaturated pastes are determined by interface bond. In oversaturated pastes the outer cores of the cement grains hydrate until all the available capillary water is consumed in the formation of CSH gel structures. However the corresponding 'inner' core remains unhydrated, acting as a strong well bonded 'aggregate' particle.

Addis:1991 has shown that it is possible to calculate the w/b ratio corresponding to 'cement saturation' (i.e. w/b = 0.43) theoretically, but this will vary from one binder to the next depending on the chemical composition of the various constituents. (Normally a figure of 0,40 is assumed). What this means is that for the binder he considered, if the w/b is greater than 0,43, the ratio X will be less than unity, and because of the cube relationship of equation 1, strength drop off will be rapid as X decreases.

Buchner(1987) found that concrete paving, made as it is from semi-dry concrete, generally has a w/b ratio ranging between 0,34 to 0,42 with 0,36 considered ideal. These ranges in w/b ratio are within the range of those used in this research, reported in table 6.2, where the experimental blocks that were purposely make with as wide a variation of w/b as possible varied from a high of 0,54 to a low of 0,30. (The wetter mixes corresponding to the w/b of 0,54 should not be considered as representative of an upper limit, as these blocks were so wet that they slumped slightly, and would therefore not be useable in any commercial application).

Paving mixes can therefore be considered as slightly cement over-saturated and consequently pore capillaries will largely be filled with gel, given adequate compaction.

Sadegzadeh(1987) did mercury intrusion porosimetry determinations (MIP), which provides a useful means of comparing pore structures in materials of similar composition. He found that pores increased with increasing w/b, whereas finishing processes such as repeated power trowelling significantly reduced porosity. In effect this means that the negative effect of a high w/b can be corrected by a mechanical process that reduces the spaces between binder particles (i.e. reduces w/b in the surface zone).

Sadegzadeh also did microhardness determinations using a Vickers M41 photopan microscope fitted with a model M12 microhardness tester, to apply loads to produce indentations. The depth of the indentations is a measure of the hardness of the paste. As with MIP he found that microhardness is a function of porosity for hydrated cements and similar materials.

The microhardness profile of concrete floors which had been finished with repeated power trowelling was seen to change rapidly with depth. His results show that the first 0,5mm is very hard, then there is a rapid drop in hardness till about a depth of 2mm, at which depth most of the hardness is gone.

Using an abrasion test that uses rolling steel wheels he showed that abrasion resistance is inversely related to porosity and proportional to microhardness. Best-fit straight lines were well correlated in both cases.

2.2.1.1(b) Influences of w/b on Abrasion Resistance

The results of various investigators are not all together in agreement. Some found the abrasion resistance increased as w/c was reduced, while others found that it was virtually independent of w/c. These findings are presented below under two headings representing the two cases. This is followed by a discussion in which an attempt is made to reconcile the two positions.

Case 1 : Abrasion resistance improves with decreasing w/b

Starting with **Abrams(1921)** many researches have shown that a definite relationship exists between w/b and the abrasion resistance of concrete. According to their findings a decrease in w/b is accompanied by an increase in abrasion resistance. This may be explained by an improvement in the microstructure of the paste as the w/b decreases (see previous section).

Dhir(1991b) did abrasion tests [using an apparatus developed by the C&CA, =rolling steel wheels, see appendix U.4.06] and showed that a decrease in w/b is accompanied by an increase in abrasion resistance. However at lower w/b, returns in abrasion resistance were diminished.

DeAlmeida(1994) did abrasion tests using the Dorry Hardness apparatus [=*sliding fine abrasive*, see appendix U.5.01], to determine the abrasion resistance of high strength concretes with w/b ranging between 0,24 and 0,42. His results show that abrasion resistance varies inversely with w/b. 28-day compressive strengths ranged between 66 MPa through 106 MPa. Granite coarse aggregate was used.

Di Maio(1999) did abrasion tests using the Dorry Hardness apparatus [=*sliding fine abrasive*, see appendix U.5.01] and found that both the abrasion resistance and the compressive strength of two year old concrete increased as w/c decreased. However his testing was done on a is very limited scale, and the proportion of aggregate, as seen in cross section across the test surface, was very varied from one sample to the next. Given the dependence of the abrasion test he used on aggregate hardness, it is difficult to be sure of his findings with respect to abrasion resistance.

Fwa(1989) did abrasion testing using a LA machine [=*impacting steel drum*, see appendix U.1.01] and showed that for a given curing / weathering / conditioning regime a decrease in w/b is accompanied by an increase in abrasion resistance, although his loss of material was so much that the test can scarcely be considered a measure of the hardness of the surface.

Fwa(1990) did abrasion testing using a LA machine [=*impacting steel drum,* see appendix U.1.01, resulting in deep wear from impact, sliding, rubbing). He found that after 2000 revolutions the cubes made with a w/c of 0,65 had lost 80% of their original mass compared to a mass loss of 66% for cubes made with a w/c of 0,45. (This is obviously very deep abrasion).

Harashima(1998) did abrasion testing on concrete pavers [according to ASTM C779 ProcA, *sliding fine abrasive,* see appendix U.5.15] and found that at a w/c of 0,30 the depth of wear was 1/3 that of 'normal blocks'.

Helland(1991) reported on the use of high strength concrete (HSC) for concrete road and bridge surfaces in Norway, which had to have exceptionally good abrasion resistance to resist the very abrasive action of steel studded tyres on vehicles. In Norway this is generally achieved by means of using silica fume, together with a superplasticized very <u>low w/b ratio</u> mix. Examples of this are:

- the Valerenga tunnel, where a w/(c+s) ratio of 0.32 was used, resulting in drilled out cores with a characteristic strength of 92 MPa
- highway E-18 and E-6, where a w/(c+s) ratio of 0.36 was used, resulting in a characteristic 28-day cube strength of 90 MPa
- the Ranafoss bridge, where a w/(c+s) ratio of 0.34 was used, resulting in a characteristic 28-day cube strength of 110 MPa
- the Smestad tunnel, where a w/(c+s) ratio of 0.22 was used, resulting in a characteristic 28-day cylinder strength of 130 MPa. Specimens sawed out from the actual pavement and tested in an abrasion testing machine (NORCEM's road tester), resulted in 0.47mm/10000 revolutions-of-four-studded-truck-tyres-at-63km/hr, when tested dry, and 1.04mm/10000 revolutions for wet conditions. This is comparable to massive granite.

Laplante(1991) did abrasion tests [according to ASTM C779 Proc C; = *rolling steel balls,* see appendix U.2.12] and found that a reduction in w/b resulted in an increase in abrasion resistance. W/b ranged from 0,27 through 0,48.

Liu(1981) used an abrasion test [the forerunner of ASTM C1138, =*impacting steel balls* (mild), see appendix U.2.07; result = intermediate abrasion] and found that abrasion resistance increased as w/c decreased. W/b ranged between 0,4 and 0,72.

Liu(1991) made a state of the art appraisal on abrasion resistance and concluded that abrasion resistance increased with decreasing w/b.

Sawyer(1957) did abrasion tests according to DIN 51951 [*rolling steel balls*, see appendix U.2.09; result] and found that abrasion resistance increased with a decrease in w/c. For example the depth of wear in a lean mix of 4,5 sacks/cu yd ≡ 267 kg/m^3 (w/c=7,4 gal/sack) was 6,6mm, while that for a rich mix of 7,5 sacks/cu yd ≡ 444 kg/m^3 (w/c = 4,6 gal/sack) was 2,5mm. Both mixes were moist cured for 7 days.

Schuman(1939) did abrasion testing using *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] and found increased abrasion resistance for reduced w/b.

Shackel(1993b) considered w/b ratios of 0,47 , 0,49 , 0,52 , 0,54 and found that abrasion resistance [rolling steel balls and impacting steel balls, see appendices U.2.13 and U2.03] decreased with increasing w/b. (These w/b ratios are considerably higher than what could be expected in a block manufacturing environment, and are a result of the low cement contents selected for his experiments (9,2% to 11,2%)).

Smith(1958) investigated the influence of various coarse and fine aggregates on the abrasion resistance of the corresponding concretes using three abrasion tests [(1) *rolling steel balls*, see appendix U.2.11, (2) *impacting fine abrasive*, see appendix U.5.20, (3) *rolling dressing wheels over fine abrasive*, see appendix U.3.08]. He showed that abrasion resistance was inversely proportional to w/c ratio. Very good correlations were obtained for all three tests.

Wastlund(1946) made wear tests in concrete slabs [using steel wheels and leather covered wheels, revolving in a circle, see appendix U.4.07]. He considered many variables including fine aggregate size, workability, aggregate/cement ratio, cement replacement material, finishing technique, compaction, coarse aggregate size, and duration of curing. His results are not easy to interpret as he did not isolate one variable at a time against the control mix. What is clear is that the two mixes exhibiting the highest wear ‘were too dry to allow satisfactory compaction’ and therefore ‘to render possible wood floating and steel trowelling, water was added by sprinkling the surface’. Thus w/c was compromised (possibly also compaction), and stands out as the most important factor.

Case 2 : Abrasion resistance is not related to w/b

Bettencourt Ribeiro(1998) used an apparatus similar to DIN 52108 [*sliding fine-abrasive*, appendix 5.02] to measure the abrasion resistance of roller compacted concrete. He tested 90 day old specimens which incorporated approximately 40% of fly ash.

[His results show that whereas compressive strength improved quite dramatically as w/b ratios decreased from 1.68 to 0.55, the abrasion resistance stayed virtually constant, with a slight drop for the higher w/b ratios. This may be attributed to the nature of the test, where the mode of attack is tangential and of a gentle scratching nature. The fine-sand abrasive grinds away but imparts minimal shock to the specimen, and hence aggregate particles are not easily debonded/loosened from the matrix. Even if they should be loosened they cannot be removed as the loosened particles are likely to remain trapped in the matrix, unable to drop out, being too close to the base/wear-plate, given the narrow distance between sample and base/wear-plate (approximately equal to the thickness of the fine-sand abrasive)].

Simon(1999) did abrasion testing on high strength concrete using an apparatus similar to DIN 52108 [*sliding fine-abrasive*, appendix 5.02]. Results showed that whereas a definite relationship exists between compressive strength and w/c ratio, abrasion resistance was almost constant for all the mixes for the full range of w/c between 0.30 through 0.70. (Corresponding cement contents were 520 kg/m^3 and 270kg/m^3). [Once again this may be explained by the particular characteristics of the abrasion test adopted, which gently grinds its way into the specimen rather than exert impact or high compressive loads, (that tend to fracture or dislodge the aggregate). The high strength igneous aggregate, which was the same for all mixes, therefore controlled the rate of abrasion, completely masking variations in paste strength. As long as the binder is strong enough to adequately anchor the aggregate particles, and a 0.70 w/c concrete (the lowest) still would be, the aggregate will remain bonded and control the rate of abrasion].

Discussion

The two seemingly different findings of case 1 and case 2 can be explained by a consideration of the mix constituents and the test method used. Bettencourt's work was done on roller compacted concrete using very low to low cement contents (90kg/m^3, 140kg/m^3, 190kg/m^3) and the hardness of the aggregate completely overshadowed the effect of variations in w/c. Similarly the hard granite used by Simon(1999) also dominated the rate of wear, although here the compressive strength at the 90 day test dates ranged between 38 MPa through 80 MPa. These two cases illustrate the relative softness of paste when compared to certain aggregates, even for pastes with low w/b ratios.

However when the w/b ratios were further reduced to the range of 0,24 through 0,42 (66 MPa through 106 MPa) as in Simon(1999), abrasion resistance was seen to be inversely proportional to w/b. At these strengths the paste was hard enough to make a contribution.

On the other hand other apparatuses used *were* able to discriminate between mixes with w/c variation. These include the rolling steel wheels, rolling steel balls and sand-blasting used by Smith, the sliding fine-abrasive of Schuman(1939), the impacting steel balls (mild) of Liu(1981), the rolling steel wheels of Dihr(1991b) and the impacting steel drum of Fwa(1990).

At face value the sliding fine-abrasive of Schuman appear to have the same action as the sliding fine-abrasive of Bettencourt and Simon, and should therefore yield similar tends in results. Both amount to a grinding action applied to the test sample by an abrasive. However there is a notable difference. In the sliding fine-abrasive apparatus used by Bettencourt and Simon the aggregate particles are unable to escape once loosened, while they can be 'spun free' in the case of Schuman's apparatus. There is significant evidence of this in Schuman's photographs of the abraded test surfaces. Schuman mainly used hard aggregates, such as traprock and siliceous aggregates that would have protected the softer paste had they not been spun out. See the important note under 'Wear Mechanisms' in the appendix U.5.13 (Schuman) compared to that of U.5.02.

Concrete pavers have low w/b ratios

Some final comments on w/b are now made to point out the low w/b ratios used in manufacturing concrete pavers.

Komonen(1998) did abrasion tests on concrete pavers [using rolling studded tyres, see appendix U.3.04] with target w/c ratios of 0,34 for continuously graded aggregates and 0,36 for gap-graded mixes. Although the results were not available at the time that Komonen published, the low w/b is noted.

Clark(1980) did freeze thaw experiments on concrete paving with various a/c (aggregate/cement) ratios:

- For a/c = 3,0 w/c was 0.22 for a moisture content of 5.2%, and w/c was 0.30 for a moisture content of 6.9%
- For a/c = 5,0 w/c was 0.33 for a moisture content of 5,2%, and w/c was 0,45 for moisture content of 7.0%
- For a/c = 7,6 w/c was 0,36 for a moisture content of 4,2%.

[These results show that as a/c increases, so must w/c. In effect increasing aggregate relative to cement requires a higher w/c in the paste, as the paste needs to be more lubricating if there is less of it. It also indicates how narrow are the limits for water content, i.e. 4,2% to 7,0% in the above quoted mixes, since concrete pavers made with inadequate moisture content will have excessive air voids, while those made too wet will slump.]

Sukandar(1993) did abrasion tests [according to ASTM C779 Proc C, = *rolling steel balls,* see appendix U.2.12] on concrete pavers with a:c ratios varying from 3 to 9. Blocks made with the higher cements had substantially more abrasion resistance, with a 300% improvement in the 455 kg/m^3 mix relative to the 153 kg/m^3 mix. Corresponding w/c ratios were 0,21 and 0,34 respectively, very low even for concrete pavers! [Once again these results show that as a/c increases, so must w/c. Again it may be said that increasing aggregate relative to cement requires a higher w/c in the paste, as the paste needs to be more lubricating if there is less of it].

Sectional conclusion to 'Water/Binder Ratio'

Generally abrasion resistance increases as w/b decreases. However, when a 'sliding fine-abrasive' abrasion test is used, where relatively hard wearing aggregate can not be dislodged during the test, then this trend may be masked by the hardness of the aggregate, particularly for weaker binders.

It is has been shown that the semi-dry mixtures used to make concrete pavers result in very low w/b ratios.

It should be observed that reducing w/b will only improve abrasion resistance if density does not suffer. To ensure that it does not suffer, additional compactive effort is usually required in low w/b mixes, unless measures are taken to reduce the internal friction of the mix, such as introducing a superplasticizer. Using more paste relative to aggregate also reduces the internal friction of the mix, so allowing a lower w/c.

The balance of this section will be devoted to the findings of various authors reporting on improving abrasion resistance by reducing w/b. It is therefore evident that this can be achieved either by reducing 'w', the numerator, or by increasing 'b', the denominator. And whereas increasing 'b' is a simple matter of using more binder in the mix, there are many things that influence the water content, 'w'. In section 2.2.1.1.1 the various ways of reducing water content are considered.

2.2.1.1.1 Water Quantity

Given the major influence of the ratio w/b on strength and other properties of concrete, this section (and all its sub-sections) will consider the main factors governing the quantity of water added to the mix. These factors include the fine and coarse aggregate, water reducing agents, pigments, binder type selection, water extraction techniques and compaction.

Although the focus here will be on minimising the water, it is important to recognise that too little water can be more detrimental than too much, and thus, at the outset, consideration is given to the importance of optimising the water content both in conventional and semi-dry concrete.

Optimum Water Content

Lane(1986) stated that the two main purposes of the mixing water are to hydrate the cement and to lubricate the mix. A feature of semidry mixes is that they have no slump [essential for immediate demoulding/extrusion] and therefore have high internal friction. This makes compaction difficult, requiring specialized equipment. Therefore, according to Lane, the optimum water content is the maximum water content that can be used that will not adversely affect the manufacturing process. Under the action of the vibrators the paste fluidises, and acts as a lubricant between aggregate particles, and permits compaction of the concrete.

On the other hand, ideally there should be no more water in the mix than what is required to fully hydrate the cement. Excess water that will not combine chemically with the binder, that was added merely to speed up the compaction process, reduces the concrete's potential density and has a detrimental effect on the strength. In the manufacture of concrete pavers semi-dry mixes are generally made at moisture contents that are slightly too dry to allow full hydration. This is borne out by Buchner(1987), who reported that in block plants across Europe typical w/b ratios vary between 0,34 to 0,42, whereas Addis(1991) showed that, theoretically, a w/c ratio of 0,43 is required to fully hydrate the cement; (this fraction may vary slightly depending on the chemical composition of the cement/binder).

Abrams(1921) did abrasion tests [using a Talbot-Jones Rattler [= 95mm *impacting steel balls,* see appendix U.2.01] and showed that increasing water in the mix reduced compressive strength and abrasion resistance.

Abram's statement may appear to contradict that of Lane '… the optimum water content is the maximum water content …', but Lane is referring to semi-dry concrete, and Abrams to conventional concrete. <u>In semi-dry concrete the gains in reducing voids by having improved lubrication far outweigh the effect of a slightly higher w/b</u>. In conventional concrete on the other hand there generally already is sufficient water in the mix to achieve full compaction, particularly with the aid of superplasticizers, and increasing the water in the mix in this case results in an increased w/c with no improvement in reduced voids. It may therefore be postulated that a distinguishing characteristic between semi-dry concrete and conventional concrete is that in the former case the quantity of water is maximised for a *given* cement content, while in conventional concrete it should be minimised.

[While every effort should be made to maximise the water in cbp mixes, too much water has the following adverse effects:

- the concrete will tend to form clumps in the feed-wagon while it is moving forwards and backwards over the mould. This prevents the concrete from flowing into the mould compartments.

- the concrete adheres to the tamper shoes during vibration resulting in a tearing effect as the tamper moves upwards in the demoulding process, or the surface of the pavers end up with ripple marks
- the concrete may slump or deform during the post forming handling process].

If using a relatively moist mix is problematic, it is still possible to maintain the same degree of compactability at a lower w/b, by simply increasing the proportion of binder. Investigations into a/c ratios by Clark(1980) and Sukandar(1993) have shown conclusively that a drier paste, if there is more of it, can achieve the same degree of compaction as a wetter paste. This finding has also been proved in this investigation (see chapter 6 of volume 1). Furthermore an increased content of drier paste has the added benefit of increased strength, since it reduces w/b, but the greater binder content is a more costly solution.

Dowson(1980) stated that typically moisture contents varied between 5% to 7% for concrete pavers. Within these parameters the moisture level used is a function of the power of the moulding machine, with more powerful machines requiring less water, thus achieving a lower w/b consistent with minimal air voids.

This range of water content ties in very closely with the range achieved in this investigation. It may be seen in table 6.2 of volume 1 that a range of 4,2% through 7,6% was obtained. This greater range is expected as it was always the intention to vary the moisture content from ‘very wet’ to ‘very dry’ in this work.

Lane(1986) stated that ‘ the amount of water required is directly related to the quality and duration of vibration, less water being required for superior vibration.

Nanni(1989) found that a moisture content of 5,8% yielded optimum densities in roller compacted concrete (also semi-dry), with corresponding optimal compressive strength and abrasion resistance.

Schuman(1939) [used an abrasion test based on the principle of *sliding fine-abrasive* beneath revolving discs, see appendix U.5.14 and] found increased abrasion wear in the form of pitting as water content increased. Below 24% water, pitting was minimal, while it became excessive above 28%.

Summarising: The function of water in concrete is to hydrate the cement and lubricate the mix. Both too much and too little water ate detrimental For every application there is always an optimum quantity.

Following are five sub-sections (2.2.1.1.1.1 through 2.2.1.1.1.5) which discuss materials and processes that influence the water content of concrete mixes, and hence the w/b ratio.

2.2.1.1.1.1 Fine Aggregate

In this section only the influence that fine aggregate has on the total water input into the mix, and hence on w/b, is considered. Other aspects that also affect abrasion resistance, such as aggregate type, hardness etc. will be considered later in section 2.4.

The quality of the fine aggregate plays the major role in determining the water required for a given workability. Sand is called ‘thirsty’ or ‘good’ depending on how much water is required. Fulton(1986) states that water demand for South African aggregates varies from 180 l/m^3 to 250 l/m^3, ‘depending mainly on the quality of the sand used’. Terms such as ‘water requirement’ and ‘water demand’ are standard definitions in the industry and are defined by Fulton(1986) as:

- ‘Water requirement’ is the mass of water required to produce one cubic meter of concrete of the desired consistence for a given aggregate.

- ‘Water demand’ is the water requirement in the special case where 19mm coarse aggregate is used with a slump of 75mm.

Fine aggregate with a low water requirement will allow either an increase in the potential strength of the concrete (by means of a reduction of w/b), or a saving in binder content (and w/b remains constant).

There are four factors that influence the water requirement of a sand:

(a) Grading

Grading clearly influences the % voids that must be filled with the paste component. Given the fineness of cement, with its relatively high water requirement, a low percentage of voids in the sand will result in a correspondingly lower water demand. Fulton(1986) refers to tests carried out on a large number of South African sands by Davis(1975*)* that ‘gave an average void content of 36% and ranged between 28% to 44%. This range in percentage voids affects the water demand of a mix to the extent of about 70 l/m^3’.

To safeguard against excessive voids a standard grading envelope is used to specify concrete sands, and any sand with an acceptable particle shape and surface texture that falls within the standard envelope will usually work well.

From the perspective of abrasion resistance however, the most serious shortcoming in a poorly graded sand is not the high voids content (i.e. the natural void content of the sand), but rather a tendency towards gravity induced segregation of the relatively heavier aggregate in the fresh mix, the direct consequence being the upward displacement of water to the surface, known as ‘bleeding’. This has the effect of increasing the w/b ratio at the surface, resulting in a reduction in abrasion resistance.

According to Stokes’s law (explained hereafter) the rate of settlement of a solid in a liquid is related to its particle size and its relative specific gravity. Consequently larger particles have a substantially greater rate of settlement relative to smaller particles, and particles sink faster where the specific gravity relative to the surrounding matrix is greater. Accordingly the coarse aggregate displaces some of the mortar upwards. Similarly there is also a measure of separation in the mortar, resulting in the fine aggregate particles displacing some of the paste upwards. This means that there will be a relatively high concentration of paste at the surface. Finally the individual cement grains in this paste settle under the action of gravity, displacing water upwards. Therefore bleeding, which is referred to as water rising to the surface, is related to segregation associated with differentials in particle size and specific gravity.

Papenfus(1947) found that the rate of settlement of quartz aggregate in *water* was very dependant on the size of the particles. Carefully conducted experiments showed that the terminal velocity of various diameters were:

1mm/sec for diameters of 0,04mm (fly ash, MGBS and OPC have similar diameters)
10mm/sec for diameters of 0,2mm
100mm/s for diameters of 0,7mm
300mm/s for diameters of 3mm

He found that at velocities below 5mm/s the experimentally determined rate of settlement in water followed Stokes’s law very closely, even though the quartz particles used in his experiments were not perfect spheres. [Stokes’s law applies when (1) constant (terminal) velocity has been reached; (2) the particle is smooth and spherical; (3) the velocity is less than a certain critical value (laminar flow applies); (4) no slip occurs between the particle and the fluid; (5) the particle is great relative to the molecules of the fluid, (6) and the extent of the fluid is infinitely large relative to the size of the particle].

This law is represented by:

$V = 2/9(D1-D2)gr^2/\eta$

Where D1 = specific gravity of the solid
D2 = specific gravity of the liquid
g = gravitational acceleration
r = radius of the particle
η = viscosity of the fluid
V = terminal velocity

It is evident that not all the conditions of Stokes' Law apply to a cement grain settling in paste. One notable difference is that the spacing of the cement particles in cement paste is very small, and this represents a significant violation of condition (6). The close proximity of the cement particles will likely slow down the rate of settling very significantly.

It is clear from this formula that the velocity is proportional to the square of the particle size. Thus it might be expected that the rate of settlement of a grain of cement in water would be very much less than that of a particle of sand. However the particle of sand is settling in a binder paste that has a 'D' which is only slightly less (depending on how much water is in the paste), whereas the grain of cement is settling in water. Thus $[D_{aggregate}-D_{paste}]$ is much smaller relative to $[D_{cement}-D_{water}]$, substantially countering the effect of a higher r^2, thus preventing excessive differential segregation.

a'Court(1954) did abrasion tests [using a reciprocating steel pan, =*sliding fine-abrasive*, see appendix U.5.17] and found some variability in surface wear between mixes made from differently graded aggregates.

[These differences however should more correctly be attributed to variations in the w/c of the *surface* paste. The depth of this wear generally ranged between 0,1mm to 0,6mm and evidence suggests that the main cause was a higher w/c resulting from a paste dilution, i.e. blending of bleed water with the cement paste during the hand finishing process. Although he aimed at keeping the w/c of the different mixes constant, he shows that the mixes corresponding to the different aggregates had different workabilities. This would have resulted in different bleed characteristics, causing different degrees of paste dilution in the hand finishing process. <u>It is therefore principally the influence that grading had on *bleeding* that influenced the rate of surface abrasion.</u> Had he managed to keep the workabilities constant, the various aggregates may have contributed to similar wear. This view is further supported by other data presented, where he shows wear rates at a depth of 0,6mm to be very similar for the various aggregate types and gradings, and it should therefore be concluded that the variations closer to the surface were as a result of paste dilution, exacerbated further by air curing].

Fulton(1986) concurs that while the grading of a sand has a relatively minor effect on water demand, it has a major influence of the workability, cohesiveness and *bleeding* properties of concrete in its plastic state. In particular the proportions passing the 75, 150, and 300 μm sieves have the greatest effect on the properties of the mix. Sand particles larger than 300 μm have less effect on the characteristics of the mix, their presence mainly adding body to the sand. <u>The minus 75μm fraction assists in controlling *bleeding* in fresh concrete on the one hand, but on the other an over-abundance contributes to a higher *water requirement*.</u> The minus 150 μm and minus 300μm fractions play a major role in determining the cohesiveness and workability of the mix, and sands deficient in these sizes tend to produce harsh concrete liable to *segregation.*

On the other hand deficiencies in grading, whether this takes the form of excessive void content (i.e. its natural void content) and/or a tendency towards segregation and bleeding in the fresh mix, can be successfully combated by repeated power trowelling. This was demonstrated by Chaplin(1991), who did abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels*, see appendix U.4.06] on concrete made from sands with gradings lying both above and below the envelope, that nevertheless yielded equivalent abrasion resistances relative to sands lying within the envelope. This is because the power trowelling operation (1) negates the detrimental effects of bleeding by re-compacting the surface layer after bleed water has evaporated, ensuring a low w/b ratio, and (2), the high water content of the mix corresponding to a sand with excessive natural voids is also reduced by the re-compaction process.

(b) Particle shape

Round or cubical shaped sand is better than aggregate comprising flaky or elongated particles. Round particles move easily in the mix during placing, allowing compaction in a more viscous paste (lower w/b). Cubical particles, while not as mobile as round particles, also facilitate good compaction.

Schuman(1939) did abrasion tests using *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] and found that both shape and grading influenced abrasion resistance, by virtue of their influence on the w/c ratio.

Liu(1991) stated that water demand is a function of the shape of the fine aggregate. Poorly shaped fine aggregate, for example flaky aggregate, has a higher water demand, therefore a higher w/b, therefore a lower abrasion resistance. Angular shaped particles on the other hand, although not having as low a water demand as rounded aggregate, generally had good bonding properties which was favourable for the abrasion resistance.

Lane(1986) pointed out that round smooth aggregate particles permit easy flow and compaction in the moulds. On the other hand rough textured, elongated or flaky particles improved the green strength of the blocks.

[Generally the 'green ' strength is seldom an issue in paving, the blocks being *solid* and relatively squat. On the other hand products such as *hollow* building blocks have thin walls and are relatively tall and long. If the handling systems of such products are not carefully maintained, the product may be subjected to knocks and bumps whilst still in a green state, resulting in the appearance of cracks or even total collapse. The writer has witnesses the phenomenon on a number of occasions].

Certain investigators have found that crushed aggregates that are angular with sharp edges nevertheless have better abrasion resistance than relatively well shaped natural aggregates. This can usually be ascribed to a degree of weathering in the natural aggregates. Essentially this means that the hardness of the aggregate generally plays a more important role than w/c. This aspect is discussed more fully in 2.4.1.1.

(c) Surface texture

Aggregate particles with a smooth texture generally have lower surface areas and therefore a smaller volume of paste is required to fully coat the particles. Also particles can move past each other, with less friction, and again this allows a thinner lubricating coating of paste. In this case a more viscous paste (lower w/b) will still provide adequate lubrication to the mix.

Ghafoori(1997) did abrasion tests [according to ASTM C779 Proc C, =*rolling steel balls,* see appendix U.2.12] to measure the abrasion resistance of concrete prisms incorporating bottom ash as fine aggregate. An increased quantity of mixing water was necessary to overcome particle angularity, rough and very porous surface texture, and excess fines found in the bottom ash aggregates, such that the corresponding concrete was 40% worse than the control. (The control used a natural siliceous sand, having particles that were 'dense, relatively smooth in texture, and had a well rounded shape').

(d) Proportioning

Increasing sand content requires increased paste content, to coat the corresponding increased surface area. This means additional water in the mix as the water required by the very fine binder particles to achieve the desired fluidity in the paste is relatively high. Thus w/b is increased with increasing fine aggregate. Conversely, using more coarse aggregate reduces the total surface area of the combined aggregate, less paste is required, and hence a lower w/b is achieved, resulting in an improvement in abrasion resistance.

Schuman(1939) used an abrasion test that uses *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] and found that abrasion wear in the form of excessive pitting increased as the ratio fine-aggregate:cement increased.

Sectional Conclusion

Well <u>graded</u> fine aggregates that result in workable mixes at low water requirements will result in lower w/b ratios. Round or cubical <u>shaped</u> particles with smooth <u>surface textures</u> further promote workability and w/b reduction. Finally the mix should ideally be <u>proportioned</u> to maximise the coarse aggregate component relative to the fine aggregate, as this also lowers w/b.

All the principles discussed in this section apply equally well to concrete pavers.

2.2.1.1.1.2 Coarse Aggregate

Coarse aggregate also affects the water requirement of the mix, but to a much lesser degree, owing to a very much reduced surface area per unit mass. There does not appear to be any specific literature on how coarse aggregate's influence on the water requirement affects abrasion resistance. Therefore this section will be covered briefly by applying some general principles of concrete technology.

(a) Grading

Coarse aggregates are either gap graded or have a continuous grading (a blend of the various sieve sizes). Generally gap graded mixes will have a higher coarse aggregate : fine aggregate ratio, and this means less paste overall in the mix, and ultimately this has the potential for improved abrasion resistance as explained in 2.2.1.1.1.1. On the other hand continuously graded aggregates have better mobility, do not lock up, and are therefore used exclusively in pumped concrete.

(b) Size

Opting for as large a size of coarse aggregate as is practical will also reduce the water requirement. However, in the production of concrete pavers, the maximum particle size will be limited to 9,5 mm for two reasons:

- aggregate which is too large will cause damage to the mould as the feedwagon reverses on it's final backstroke; (this applies when such aggregate is trapped at the interface between the front plate of the feedwagon and the compartment plates of the mould).
- the surface texture of the top of a concrete paving block is substantially affected by the size of the aggregate. In the short cycle times of paving machines, even with very powerful vibration, there is not much time for the paste/mortar to wrap itself around the coarse aggregate particles which are at the top and in contact with the shoe of the press. This is aggravated by the semi-dry consistency of the mix. Therefore particles that are too large do not get covered, leaving a rough surface texture, unacceptable to most clients.

(c) Shape

As in the case of the fine aggregate, round or cubical shapes require slightly less water to lubricate the mix relative to flaky or elongated particles, thus reducing w/b to a small degree with a corresponding increase in abrasion resistance.

(d) Surface texture

Again, as with fine aggregate particles, smooth particles will have a lower water requirement.

(e) Proportions

The greater the proportion of coarse aggregate, the lower is the water requirement of the mix. Therefore, for a given quantity of cement, a lower w/b can be achieved, with a corresponding increase in paste strength and aggregate/paste bond. The stronger paste is harder, while the improved bond reduces aggregate loss. Generally this means an increase in abrasion resistance, providing compactability is not jeopardised.

In concrete pavers the proportion of coarse aggregate is often limited by aesthetic considerations, since the surface takes on more of a 'stoney' look, not generally preferred. However, where a topping concrete is used, this can be made to a very smooth texture, with the base concrete being substantially 'stonier'. In this way aesthetics and economy/strength respectively are combined.

Sectional Conclusion

Coarse aggregate does not affect the w/b ratio to the same extent as fine aggregate. On the other hand its hardness plays an important part in resisting *deep* abrasion, but this will be discussed in 2.4.2.1.

2.2.1.1.1.3 Superplasticizer

Superplasticizers are powerful water reducing admixtures and are widely used. They have the effect of substantially reducing the viscosity of the paste, with a corresponding increase in the lubrication of the mix. Generally this allows either a reduction in the binder content (without sacrificing strength), or an increase in strength (without increasing binder content), or increased workability (without increasing water content). Respectively the superplasticizer achieves this by reducing both the binder and water contents without altering the w/b ratio, decreasing the w/b ratio by reducing the water content while leaving the binder content unaltered, and by leaving both the binder and water contents unaltered.

Following are some references that show that superplasticers also have the potential to increase abrasion resistance:

Investigating high strength concrete **DeAlmeida(1994)** showed that the abrasive wear [as measured by the Dorry Hardness apparatus, =*sliding fine abrasive*, see appendix U.5.01] of a superplasticized concrete was reduced by as much as 50 % of that of the equivalent unsuperplasticized concrete. This work was done on binders consisting of pure cement, as well as 10% replacement levels of fly ash and silica fume.

Ghafoori(1997) did abrasion testing [according to ASTM C779 Proc C,= *rolling steel balls,* see appendix U.2.12] on concrete beams incorporating bottom ash as fine aggregate. An increased quantity of mixing water was necessary to overcome particle angularity, rough and very porous surface texture, and excess fines found in the bottom ash aggregates, such that the corresponding concrete was 40% worse than the control. (The control used a natural siliceous sand, having particles that were 'dense, relatively smooth in texture, and had a well rounded shape'). However when a powerful water reducing agent was used to lower the water requirement of the mix (and maintain equivalent slump), it was possible to achieve a lower w/c ratio for a 50/50 blend of bottom-ash/siliceous sand, resulting in a 13% increase in abrasion resistance relative to the control samples.

Superplasticizers will often reduce the w/c ratio to a level lower than 0.43, resulting in some of the cement remaining unhydrated. This is particularly true for concrete pavers, that by virtue of their zero slump consistency already have low w/b ratios, and are thus substantially ‘over-cemented’. Nevertheless special superplasticizers are available for semi-dry mixes. While these mixes will have improved strength, consistent with reducing w/b, more and more of the cement will behave as clinker aggregate. The bond between this ‘aggregate’ and the surrounding gel will be extremely good, but there is a cost implication!

Helland(1991) reported on the use of high strength concrete (HSC) for concrete road and bridge surfaces in Norway, which have to have exceptionally good abrasion resistance to resist the very abrasive action of steel studded tyres on vehicles. This is generally achieved by means of using silica fume, together with a superplasticizer to achieve a very low w/b ratio mix. Examples of this are:

- the Valerenga tunnel, where a lignosulphonate based water reducer and a melamine based plasticizer achieved a w/(c+s) ratio of 0.32, resulting in drilled out cores with a characteristic strength of 92 MPa
- the Ranafoss bridge, where a lignosulphonate based water reducer and a naphthalene based plasticizer achieved a w/(c+s) ratio of 0.34, resulting in a characteristic 28-day cube strength of 110 MPa
- the Smestad tunnel, where a lignosulphonate based water reducer and a naphthalene based plasticizer achieved a w/(c+s) ratio of 0.22, resulting in a characteristic 28-day cylinder strength of 130 MPa. Specimens sawn out from the actual pavement and tested in an abrasion testing machine (NORCEM’s road tester), resulted in 0.47mm/10000 revolutions-of-four-studded-truck-tyres-at-63km/hr, when tested dry, and 1.04mm/10000 revolutions for wet conditions. This is comparable to massive granite.

A number of companies in South Africa supply a variety of different superplasticisers / water reducing agents. A typical product range is included in appendix S.1, for use in normal concrete, flowable concrete, and mortars.

Sectional conclusion

Superplasticizers are widely used to lower w/b, with a corresponding increase in abrasion resistance.

2.2.1.1.1.4 Pigment

Following some backgound information, the effect that pigment has on the water requirement will be considered.

Sometimes the surface finish of a concrete floor is given a colour to create a special effect. (It would be uneconomic to colour the floor all the way through). This can be done by using a pigment in the finishing mix, usually a finely ground iron oxide.

Similarly, for aesthetic reasons, concrete pavers are frequently pigmented, particularly in applications such as shopping malls, domestic driveways etc. In these applications they are typically 50mm thick and through colouring adds substantially to the cost. Alternatively, by using a slightly more sophisticated machine, the pavers may be made with a surface skin that is typically 5 to 10mm thick.

It is the binder paste that is coloured by the finely particulate colour pigments. The aggregate itself cannot be coloured, its particles being merely surrounded by the coloured cement paste.

Pigments made from inorganic oxides are lightfast, weather-stable and unaffected by the highly alkaline environment of cement. Different oxides are used to produce different colours; red, brown, yellow and charcoal concrete are produced using iron oxides; green from chrome oxide; blue from cobalt; white from titanium dioxide, to be used with white cement.

Oxides may be considered as inert very fine fine-aggregate. **Puttbach(1987)** explains that their diameters lie in the order of the wavelength of light, i.e. well below 1 micron. The different colours are obtained by variations in shape and diameter.

Buchner(1987) showed that additions up to 10% of the weight of the cement had practically no adverse effect. In fact, in his tests there was a well defined increase in compressive strength for red, black and brown pigments, peaking at 6% by mass of cement. [The increase may be attributed to improved densification, just as it has been shown that the addition of an inert very fine material such as carbon black increases strength]. However yellow oxides have a greater water adsorption (they are extremely fine and needle-like in shape) and at 6% addition show no improvement relative to 0%, while at 10% addition Buchner found that compressive strength was noticeably reduced for this colour).

Shackel(1993a) did abrasion tests [using rolling steel balls and impacting steel balls, see appendices U.2.13 and U2.03] and found that using up to 7% pigmentation did not have a detrimental effect on abrasion resistance of 100mm concrete cubes, and in some cases results were significantly improved. (Presumably his work was not done using yellow oxide).

Generally the point of colour ‘saturation’ (the point at which colour intensity approaches a maximum) is reached at between three to five percent by mass of binder, and therefore iron oxide pigments, when proportioned in this way will not have a detrimental effect on compressive strength and abrasion resistance.

Sectional conclusion

It appears that providing dosage levels are not excessive, the void filler effect of incorporating iron oxides more than compensates foe the corresponding increase in surface area.

The dosage level of pigment used for identifying the different mix designs in the experimental phase of this thesis is given in table 3.1 of volume 1. It was generally 4% except for mix 8, which had 1 ½ %. From the evidence presented by Buchner and Shackel, it appears that the pigmentation would have enhanced the abrasion resistance of the experimental pavers to a slight degree.

2.2.1.1.1.5 Binder Type

Certain commonly used additives, referred to as cement extenders, are known to influence the water requirement of the mix.

(a) Fly ash

Fly ash particles are generally spherical in shape. This improves the lubricating properties of the paste towards ease of placement and consolidation during vibration. If this increased workability is not required, a reduction in the water and total binder content can be made whilst maintaining the same w/b. It is largely by virtue of this reduction in water that equivalent 28-day strengths are achievable for fly ash mixes with up to 30% replacement, relative to mixes consisting entirely of Portland cement.

However this was not the case in the experimental phase of volume 1, where the ‘Matla’ fly ash (see appendix A.4) retarded 28-day compressive strength and abrasion resistance at 14%, 21% and 28% substitution levels. (It is understood that Lethabo fly ash has a better 28-day performance)

Fly ash is discussed in more depth in 2.2.1.2.1.

(b) Silica fume

Silica fume particles are extremely fine, resulting in the material having a specific surface area of 20000/kg. At replacement levels of approximately 2%, the material acts as a very effective void filler, increasing density and strength. The silica fume particles are so fine (of the order of one hundredth of a cement particle) that they easily occupy the voids between other binder particles, whether particles of cement, fly ash or ggbs, and even penetrate the voids within floccs of these particles.

However replacement levels exceeding approximately 4% generally require additional water to maintain workability. This increases w/b and results in a decrease in 28-day strength, even when the beneficial pozzolanic effects are taken into account (see section for 2.2.1.2.3 for a fuller discussion on pozzolanic effects).

Khayat(1998) refers to work done by Jahren, P., 1983, which shows that the gains in strength from the ‘pozzolan-effect’ and ‘microparticle-effect’ are offset to some extent by the water ‘demand-effect’, and this impacts negatively on the w/c ratio. An increase in water demand follows from the very high surface area of silica fume (20000 m^2 / kg). It is therefore necessary to use superplasticizers to lower the w/c ratio. Jahren’s graph does however show a positive contribution at small dosages of silica fume. This may be the result of silica fume particles entering the spaces within cement floccs and displacing the water that would otherwise have occupied this space, which is released as free water to facilitate the fluidity of the mix.

Ramezanianpour(1999) did abrasion tests [using *impacting fine abrasive* in a water-sandblast test, see appendix U.5.22] on silica fume mixes. He found that it was possible to maintain a constant w/b ratio of 0,48 for silica fume replacement levels of up to 4%, but that at 6%, 8%, and 10% replacement it was necessary to use respectively 0.66%, 1,88% and 1,99% (by weight of total binder) superplasticizer.

Papenfus(1995) did abrasion tests on concrete pavers using *rolling steel balls*, *sliding wire bristles*, and *impacting fine abrasive* (see appendices U.2.15, U.6.02, U.5.21 respectively). The results indicated that 5% and 10% substitutions of fly ash resulted in very similar densities relative to the 0% control. Corresponding moisture contents in the respective mixes were also very similar, although the 5% silica fume mix achieved its density at a lower moisture content of 5.6%, relative to the other at 6.1%. Note that each result is the average of 30 determinations. The implication of this is that silica fume does not negatively influence the water requirement of semi-dry paving mixes, even at replacement levels of up to 10%.

ACI committee 234(1995) report that above 5% by mass of cement the water demand of concrete containing silica fume increases with increasing amounts of silica fume. This increase is due primarily to the high surface area of the silica fume, and is usually offset by use of a high-range water-reducing admixture (superplasticizer).

Other forms of silica may also be used as cement extenders. While not as fine as silica fume, ‘geosilica’, a mined and milled silica has been used successfully.

Chisholm(1997) did abrasion tests [using an apparatus developed by the C&CA, *=rolling steel wheels*, see appendix U.4.06] on concretes incorporating 7% and 10% geosilica, an amorphous silica mined and processed in New Zealand. In order to equalise the water requirement of the two geosilica mixes relative to the 0% geosilica control, he found that it was necessary to use 0,25 % of a high range water reducer. This indicates a higher water requirement for geosilica mixes, which would impact negatively on the w/b ratio in the absence of water reducing agents. (Geosilica has particle sizes ranging between 10 and 0,4 micron, somewhat larger than silica fume, where all particles are below 1 micron).

Sectional conclusion

The inclusion of fly ash up to replacement levels of 30% will generally yield equivalent 28-day compressive strengths (and presumably also abrasion resistance) whereas it appears that above 4%, silica fume requires the use of a superplasticizer to achieve equivalency in strength. To the extent that compressive strength is an indicator of paste quality, to that extend it will also influence abrasion resistance.

2.2.1.1.1.6 Water Extraction

In the foregoing sections a number of materials have been considered along with their various attributes, to show how these influence the water required in the mix, and hence the w/b ratio. By judicious consideration of these, the materials engineer is able to select those materials with the most favourable water requirement.

Notwithstanding the eventual choice of materials, Chaplin(1990) describes certain techniques and *finishing procedures* which are able to remove water from the concrete after it has been cast, and in so doing reduce the w/b ratio:

(a) **Finishing (Hand and Power)**

Concrete slabs may be finished using hand held wooden floats and hand held steel trowels, or by power floating and power trowelling.

Power floating is normally done by means of a machine powered large rotating disc, one to two hours after casting, once the mix has stiffened to a degree. It results in a level surface but not smooth and not very dense. After further delays (to allow further stiffening and any additional bleed water brought to the surface to evaporate), successive power trowelling operations are done by means of four machine-powered rotating blades.

Power finishing essentially reverses the harmful effects of bleeding. Bleeding occurs when water is displaced upwards through capillaries as the aggregate particles undergo gravity induced settlement. Where this process occurs relatively rapidly, the flow of water will be fast enough to take some relatively fine cement particles along, resulting in a loose layer of cement or ‘laitance’ at the surface that has a high w/b. Thus by allowing a period for the ponded bleed-water to evaporate, and then compacting this loose layer by power finishing, the cement particles at the surface are brought into close proximity. The final result is a smooth and very dense layer at the surface, consisting of a binder rich compacted mortar with a much reduced pore volume, a very low w/b, and consequently a high abrasion resistance. The negative effects of bleeding are thus reversed. Furthermore, in bringing additional cementitious material to the surface, bleeding actually increases the binder content at the surface, and if power trowelling is correctly applied, the end result may be a surface with an even higher abrasion resistance relative to a surface that experienced no bleeding. The increased abrasion resistance of a low w/b ratio may be ascribed to (1) increased hardness in the paste component, and (2) an improved aggregate/paste bond.

Kettle(1986), Kettle(1987b) found that repeated power finishing produced highly significant improvements in abrasion resistance [using an apparatus developed by the C&CA, =*rolling steel wheels*, see appendix U.4.06].

Kettle(1987a) offers this explanation: The repeated agitation of the blades after a period of rest causes 'additional consolidation through the disruption of the initial gel structure. This vibratory disturbance also allows further compaction by eliminating the voids that result from internal bleeding as the concrete hardens. The extent to which void content is reduced depends on the length of the delay before vibratory disturbance is applied and on the amount of vibration. The action of power trowelling (due to the application of high uniform pressure) tends to bring particles that are present in the surface matrix into more intimate contact, thus closing up surface pores and microcracks. This is evident by the smooth and close texture of the surface finish'.

Chaplin(1991) stated that in a well finished industrial concrete floor, abrasion wear is limited to the top 1mm. For this reason this depth has been used to designate 'shallow abrasion', which was frequently referred to in section 2.1).

In constructing these floors the action of power floating and power trowelling depresses the coarse aggregate particles that were at the surface, leaving a surface layer consisting of a rich mixture of binder and fine aggregate particles. (This is different to concrete pavers, where the nature of the semi-dry mix together with the short vibration cycle does not suppress the coarse aggregate to any appreciable degree).

The startling economic benefit of repeated delayed power trowelling finishing, as illustrated by Chaplin(1991), is that a very ordinary concrete mix of say 36MPa with a relatively high w/b to start with, can have its surface transformed into a very low w/b surface, giving equivalent abrasion resistance to an 85MPa power floated mix.

Chisholm(1994) did abrasion tests [using an apparatus developed by the C&CA, =*rolling steel wheels*, see appendix U.4.06] and found that the abrasion resistance of a 22MPa concrete slab finished by repeated power trowelling was superior to that of a 52MPa slab finished by single power trowelling.

C&CA:NZ(1999) have produced a guide based on abrasion tests done [using an apparatus developed by the C&CA, =*rolling steel wheels*, see appendix U.4.06]. The document states that for industrial floors repeated power trowelling, with delays to allow bleed water to evaporate between successive applications, has the greatest influence on abrasion resistance, followed by curing, then concrete mix proportions. This finishing technique can improve the abrasion resistance by a factor or three or four relative to hand trowelling.

Clearly, in shallow abrasion, it is the quality of the paste at the surface that is primarily responsible for the abrasion resistance of the floor. (Only in exceptional circumstances will there be sufficient wear to expose the coarse aggregate, in which case the hardness of the aggregate will influence subsequent abrasion performance).

Munn(1995) reported on two concrete floors at construction sites in Australia which employed double and triple headed power trowels. Double and triple headed power trowels have the added advantage of ensuring a flatter floor, and greater productivity. Very good abrasion resistance was achieved [using an apparatus developed by the C&CA, =*rolling steel wheels*, see appendix U.4.06].

Fentress(1973) showed that whereas power finishing correctly done greatly enhances the abrasion resistance of a concrete floor, incorrect timing of the various operations is problematic resulting in:

- Scaling: This is a loss of the top surface at an early age under traffic, caused by premature finishing operations while bleed water is still present. The bleed water is unable to reach the top surface through the semi-hard impermeable top surface zone. This practice leads to a high w/b in the zone below the top surface paste and consequently also a low paste/aggregate bond.
- Dusting: This is a loose powdery surface on hardened concrete, again the result of premature floating and finishing (done before all the bleed water has evaporated). It also may occur in poorly cured surfaces.
- Crazing: This consists of a pattern of fine surface cracks, formed by surface shrinkage during the early stages of hardening. It is usually seen on steel floated surfaces with a high paste content. It can be effectively contained by good curing.

Dahir(1981) did abrasion tests [according to CRD-C52-54, =*rolling dressing wheels*, similar to appendix U.3.09] and found that concrete pavements which had been rained upon prior to final set had reduced abrasion resistance. [This may easily be attributed to an increased w/b from paste dilution].

a'Court(1954) did abrasion tests [using a reciprocating steel pan, =*sliding fine-abrasive*, see appendix U.5.17] and found that finishing with hand floats and trowels resulted in 'a relatively easily abraded skin' compared to the 'harder inner core' concrete further down. The depth of this skin generally ranged between 0,1mm to 0,6mm. The softness of the skin may be attributed to a higher w/c resulting from a paste dilution, i.e. blending of bleed water with the cement paste during the hand finishing process.

Grieve(1993) found that by hand trowelling a small test surface after allowing bleed water to evaporate, very substantial improvements in abrasion resistance were possible relative to a cast face of the same mould. From this it is evident that carefully done hand trowelling can also be effective in densifying the surface relative to the core concrete.

Schuman(1939) did abrasion testing [using *sliding fine-abrasive* beneath revolving discs, see appendix U.5.14] and found increased abrasion resistance for specimens that were hand trowelled after delays. The greater the delay was the higher the abrasion resistance. For example a delay of five hours to the trowelling operation resulted in a greater abrasion resistance than a three and a half hour delay, which in turn was greater than a two hour delay, which was greater than immediate trowelling with no delay.

Schuman also found that excessive floating that brought water to the surface was very detrimental to the abrasion resistance if no further finishing was done. In these cases abrasion resistance at some depth was much greater than that at the surface.

Concrete pavers are made from semi-dry concrete, typically with a w/b in the region of 0,36. Such a low moisture content in the mix, coupled with the short vibration period of about 6 seconds, albeit with very powerful vibrators, is barely enough to expel the air filled voids. There certainly is inadequate plasticity/lubrication for any settlement and segregation to occur (such as takes place during the lengthy vibration/bleeding/power finishing cycle of industrial cast insitu floors). Nevertheless the writer has observed that pavers made upright (the wear face is vertical during manufacture) have superior abrasion resistance relative to pavers made with their wear faces horizontal. The most likely explanation for this is that there is a degree of finishing that takes place with the 'upright' pavers. Some explanation of the workings of a concrete paving machine is required to understand how this works, and the essential components are illustrated in figure 2.6.

Vibration systems in paving machines:

Most modern concrete paving machines have two sets of vibrators.

The *table vibrators* are by far the most powerful and are connected to the *vibrator table*, which is situated beneath the timber *pallet* upon which the *mould* rests. The function of these vibrators is to move the table up and down through a *neutral plane*. The pallet upon which the mould rests is supported by a set of independent and *fixed supports*, set exactly at the height of this neutral plane, so that when the vibrators are at rest, both the vibrator table and the fixed supports will be touching the underside of the pallet. At the same time the fixed supports are so designed that they do not interfere with the upward and downward oscillating motion of the vibrator table as it moves through the neutral plane. This means that the table will strike the pallet (and thus the mould) at its maximum speed in the upward direction, like a heavy hammer, lift it off the supports by a few mm, then begin with a downward acceleration to complete the cycle, and en-route drop the pallet (and mould) back onto the fixed supports as the table proceeds to its lowest position (under the influence of the vibrators), this time some mm below the neutral plane, whereupon it again starts its upward acceleration towards the neutral point to again strike the waiting pallet. In this way the pallet is being struck in both the upward and downward direction, first by the vibrator table, and then by the fixed supports in ‘hammer’ like fashion. The principle function of the tamper in this exercise is to prevent the mix from being knocked out of the mould when the table strikes the mould on its upward cycle. It is precisely this knocking/striking effect that produces most of the compaction. In this process there is a degree of high pressure trowelling that occurs between the vertical walls of the mould and the concrete, which takes place as the concrete is consolidated from its uncompacted state to its final density. The process of consolidation consists of the table striking the pallet, whereupon the concrete mix that was in the mould is accelerated away from the pallet, and in the process rubs against the vertical sides of the mould under ever increasing pressure. A similar trowelling effect takes place when the table strikes the pallet supports on its downward journey. Given that a typical frequency for the vibrators is 2880 revolutions per minute, and that there is an upward and downward strike with each revolution, this effectively means, that for a typical 6 second vibration period each batch of pavers is subjected to : 2 strikes per rev x 2880 revs per min / 60 seconds/min x 6 secs vibration = 576 strikes. This is a considerable amount of trowelling at a high pressure, induced by a substantial compactive effort, considering that the mass of the table including vibrators is of the order of 700kg, striking at high speed a mould (including mix) of say 350 kg.

Once the vibration cycle is complete, further trowelling occurs as the mould moves upwards leaving the product in its naked demoulded state on the pallet.

These relative motions between concrete and mould may be regarded as a form of trowelling, under considerable pressure. The end result of this high pressure towelling in the direction of the vertical axis is easily observable on the sides of the pavers as a faint smear, indicating higher concentration of mortar/paste in this zone, particularly for a mix

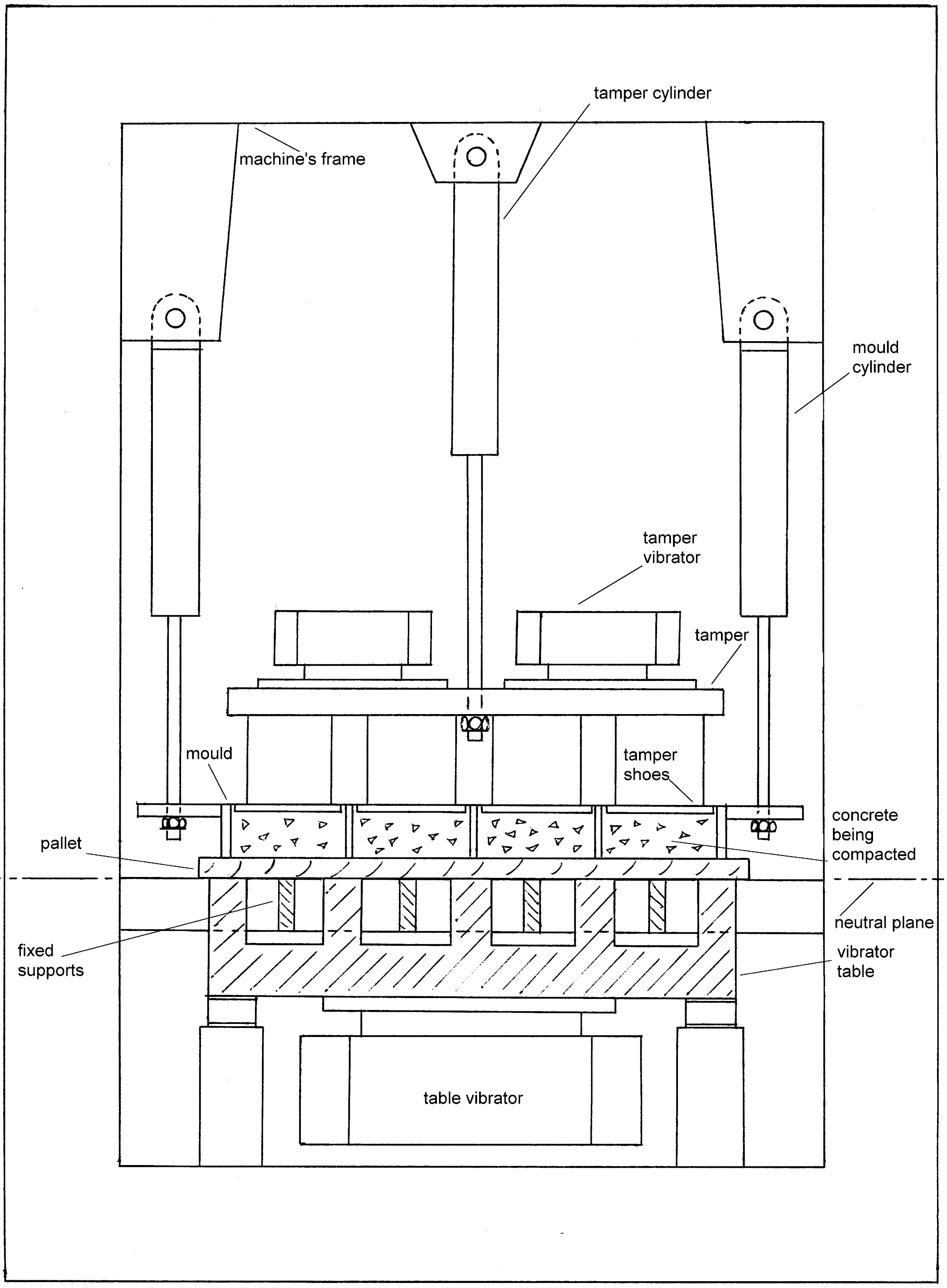

Figure 2.6 Schematic showing a section through a concrete paving machine. The components of the compaction and vibration system are emphasized.

of optimum water content. This trowelling effect tells apart the concrete in contact with the upright sides of the mould from that in contact with the horizontally orientated tamper shoes, as this latter concrete experiences no trowelling effects. On the contrary, it experiences the negative effects of a slight degree of bleeding which increases the w/b ratio at this interface.

Thus a paving block made upright will benefit from trowelling effects, rather than suffer the effects of bleeding, and together these two aspects account for its substantial improved abrasion resistance relative to a paver made with the wearing face horizontal.

Finally there is also a second set of vibrators fastened to the tamper, the *tamper vibrators*. These input a much lower level of energy into the system, with no hammering effect, and chiefly serve to consolidate the surface in contact with the tamper shoes. In practice these vibrators are not always required and are often switched off or even removed.

These principles also apply with some modification in the case of multi-layer machines. (These are machines that have no pallets. In one version of these machines, following the filling/compaction cycle, the machine moves away from the vibrator table, with the compacted blocks 'hanging' in the mould. The blocks are extruded on a standard forklift pallet in front of the vibrator table and are sequentially stacked in successive layers with a layer of sand separating them).

Fiola(1988) has explained the important role played by the vibrators in the quality of concrete pavers. He explains aspects such as direction of rotation and frequency, and gives mathematical expressions for amplitude, speed, and acceleration as they relate to the vibration equipment.

Holthaus(1988) reported on the latest development in vibration systems that can vary both the amplitude and frequency of vibration, such that it is possible to have different values for the pre-vibration and final vibration. It is even possible to vary these values from one pallet to the next to compensate for a variation in pallet stiffness.

Dowson(1998b) showed that variations in pallet stiffness accounted for significant variations in density. Densities were greater by approximately 50kg/m^3 for 'new boards' relative to 'old boards', and the corresponding compressive strengths were generally 5 to 10 MPa more.

It may be concluded that the selection/adjustment/maintenance of the equipment that is associated with the compaction and finishing in the pavers, is easily as important as achieving the optimal moisture content in the mix.

(b) Vacuum dewatering

In this process a 'vacuum blanket' is placed over the surface of the concrete. The material in contact with the surface is a fine filter sheet that allows water to pass through it but prevents cement, additions, or fine aggregate to enter under the action of a vacuum pump. This process both reduces the water/binder ratio and by virtue of the pressure on the surface of the concrete closes the bleed channels and capillaries, resulting in a dense surface. The time delay before power finishing can commence is also much reduced, and in some instances power floating and subsequent trowelling follow on immediately.

Kettle(1984), Kettle(1987b) found that vacuum dewatering together with a single application of power floating and trowelling was very effective in improving the abrasion resistance [as determined using the C&CA *rolling steel wheels* apparatus, see appendix U.4.06] of concrete floors relative to hand floated and trowelled floors, more effective than power floating and trowelling (but not as effective as delayed and repeated power trowelling).

(c) Re-Vibration:

This implies that the cast concrete is re-vibrated after some hours. This results in a disruption of early gel structure, releasing water trapped in gel cavities, as well as facilitating a measure of settlement of the solid components of the mix, as occurs with power finishing. Revibration is not specifically a surface hardening process, rather it improves the density of the core concrete. As such it is unlikely to make the kind of improvement to abrasion resistance that would be the case if a special finishing process were used.

Sectional conclusion

Different techniques of extracting water from the completed concrete slab have been considered. The most effective is delayed and repeated power trowelling. The action of power trowelling (due to the application of high uniform pressure) tends to bring the binder enriched surface matrix (a positive consequence of segregation and bleeding) into more intimate contact, thus closing up surface pores and microcracks. Although this technique only improves the top few millimetres of the slab, the resultant surface is so hard that it easily withstands the intended traffic.

In concrete pavers an equivalent form of power trowelling occurs, chiefly on that part of the paver that was in contact with the vertical walls of the mould, and surfaces of concrete pavers formed in this orientation exhibit a noticeably improved resistance to abrasion.

2.2.1.1.2 Binder Quantity

In the foregoing section various ways of increasing strength by reducing the water quantity in the mix were considered.

A second obvious way of reducing the w/b ratio is to increase the binder content.

However, the relationship between binder quantity and abrasion resistance, taken at face value, appears unclear from the literature. These inconsistencies can however be explained, and as a first step in this direction, the findings of the various authors are arranged under two headings to illustrate the different points of view:

(a) **Increasing cement content promotes increased abrasion resistance**

That increased binder content promotes increased abrasion resistance is implied in a number of codes of practice that specify minimum cement contents as a means of ensuring adequate strength and abrasion resistance. An example would be The Concrete Society – Technical Report No 34, which calls for various minimum cement contents ranging from 325kg/m^3 to 475kg/m^3, depending on the severity of abrasion.

Sukandar(1993) did abrasion tests [according to ASTM C779 Proc C, =*rolling steel balls*, see appendix U.2.12] on concrete pavers with a/c ratios varying from 3 to 9, and found that the abrasion resistance of concrete blocks is strongly affected by the aggregate-cement ratio. Blocks made with the higher cement contents had substantially more abrasion resistance, with a 300% improvement in the 455 kg/m^3 mix relative to the 153 kg/m^3 mix. Corresponding w/c ratios varied between 0,21 and 0,34. The relationship between cement content and depth of wear was shown to be a quadratic equation, with the wear for high cement contents (slight) accelerating at an ever increasing rate as the cement content was reduced. [It is worth noting that this acceleration in wear would have been even greater, was it not for the increasing bearing area between ball and concrete surface with increasing depth of wear. The ball test therefore has the inherent ability of inducing extremely high compressive stresses at the start of the test, reducing as the contact arc between ball and concrete increases with increasing penetration].

Sukandar explains that an increase in cement content resulted in a binder-rich and *dense* block surface, which, in turn, greatly enhanced the resistance to abrasion. He noted that a change in aggregate-cement ratio had a ‘much greater influence’ on abrasion resistance than on compressive or tensile splitting strength.

Sukandar further states that high cement content ‘enhances workability and compactability of the matrix’, [and the 9,15% difference in density between pavers made from highest and lowest cement contents proves this point. The respective densities were 2317kg/m^3 and 2108kg/m^3. Increased density was obtained for the high cement contents even though the w/c ratio of the lowest cement content at 0,34 was considerably more than that of the highest cement content at 0.21. An increase in cement content increases the thickness of the coating of paste around the aggregate particles. This thicker layer has greater mobility and lubricating ability than does in the presence of vibration. Accordingly it is possible to reduce the w/b ratio of the thicker pastes and still achieve adequate compactability. It may therefore be postulated that the volume of paste contributes more to the rheology of semi-dry concretes than the w/c of the paste. Or put another way, more of a viscous paste is better able to reduce voids than less of a more fluid paste. The greater quantity of the viscous paste is able to 'bed-in' the aggregate particles more easily with fewer voids. Although it is less fluid, the relative movements of adjacent cement particles can be less, as there are more of them at any given section, and cumulatively their capacity for shear related displacements is better. Thus adequate flow during compaction is achieved by the greater volume of paste in the case of the low a:c mix].

Ghafoori(1995) reported on Sukandar's work, which incorporated cement contents of 10.0%, 11.1%, 12.5%, 14.3%, 16.7%, 18.2%, 20.0%, 25%. He noted that there was a marked reduction in the penetration of the balls between 12,5% and 14,3% cement. (The further reduction in depth from 14,3% all the way through to 25% cement was no greater). This seems to tie in well with Ghafoori's observation that cement contents of 'about 16%, possibly higher due to freeze-thaw limitation, is a common practice in this industry' and that such pavers 'possess a satisfactory performance'.

[These results indicate that there is a critical binder content, below which there is a sharp drop in abrasion resistance. There are two likely explanations for this: (a) For a given compactive effort and aggregate there is a minimum binder content required to facilitate compaction and fill up the voids between the aggregate particles (b) For a given abrasive attack, there is a minimum aggregate:paste bond required to prevent plucking out of the aggregate. Clearly (b) is also dependant on (a)].

Ghafoori(1999) did abrasion tests [using ASTM C779 Proc C, =*rolling steel balls,* see appendix U.2.12] to determine the effect of replacing a proportion of the fine aggregate with silica fume. He found that increases in binder content produced increases in abrasion resistance, assuming continuous curing. At 28 days, increasing the binder content from 227 kg/m^3 to 295 kg/m^3 (10% fly ash replacement), resulted in an increase of 24%, while increasing binder content from 295 kg/m^3 to 363 kg/m^3 resulted in an increase of 29%.

Papenfus(1988) did abrasion tests with using *rolling steel balls*, *sliding wire bristles*, and *impacting fine abrasive* (see appendices U.2.15, U.6.02, U.5.21 respectively). He considered binder contents of 10%, 14% and 18% and found significant improvements in abrasion resistance with increasing cement content. This work is fully reported in volume 1.

This can be ascribed to two contributing factors:

- The 14% and 18% binder contents had substantially improved densities compared to the 10% mixes, (respectively 2203kg/m^3, 2219kg/m^3, 2090kg/m^3). The most probable explanation is that there was not sufficient paste to completely fill the voids in the fine aggregate with the given compactive effort.

- The best of the 18% mixes only had a marginally improved density compared to the best of the 14% mixes (0,4% improved), but the corresponding compressive strength was substantially improved by 27,2%. The difference comes in with w/b, where the 18% ratio was 0,342 compared to 0,486 for the 14%, a reduction of 29,6%. Clearly it is the paste that facilitates compaction of the aggregate particles, and these findings show that more lubricating material between aggregate particles (increase in paste content), allows a greater viscosity in the lubricating material (decrease in w/b). Essentially greater paste contents allow reductions in w/b without sacrificing compactability.

From the above it may be concluded that in the case of concrete paving increasing the cement content has a marked effect on the strength of the blocks, firstly by serving as a void filler, and secondly by reducing the w/b.

Papenfus(1993b) made the observation that there are two ways of decreasing w/b ratio; (1) by decreasing the water content or (2) increasing the binder content. However, unless very powerful vibration is available, reducing the water content makes it difficult to achieve full compaction, and the negative impact of this on abrasion resistance is far more serious than any gains achieved by the associated decrease in w/b. A decrease in w/b is therefore best achieved by an increase in cement content for products made from semi-dry concrete.

Shackel(1993a) considered cement contents of 9,2%, 10,2% and 11,2% in cubes and found that the abrasion resistance [as determined by *rolling steel balls* and *impacting steel balls,* see appendices U.2.13 and U2.03 respectfully] increased as cement content increased. [These cement contents are lower than may be expected in a typical paving manufacturing environment, and were such low binder contents to be used, density would be seriously compromised].

Smith(1958) investigated the influence of various coarse and fine aggregates on the abrasion resistance of the corresponding concretes [using three abrasion tests, (1) *rolling steel balls*, see appendix U.2.11; (2) *impacting fine abrasive*, see appendix U.5.20; (3) *rolling dressing wheels over fine abrasive*, see appendix U.3.08]. He showed that abrasion resistance was proportional to cement content for all three tests, (although the correlation between abrasion resistance and w/c was considerably better).

(b) **Increasing cement content does not increase abrasion resistance**

Newman(1997a) stressed that specifying minimum quantities of binder content in codes and specifications to ensure adequate strength (crushing strength, abrasion resistance etc.) was outdated and even misleading.

While it is true that for a given workability the mix with the higher cement content will exhibit the greater strength, it is equally true that high strength concretes can be made with relatively low binder contents if the w/b is kept low without sacrificing density.

The writer has designed many concrete mixes for use in prestressed concrete that have 28-day crushing strengths of 80MPa with as little as 300kg/m^3. A low w/b was achieved by judicious selection of aggregates, use of an efficient superplasticizer, and incorporation of fly ash as part of the binder. Similarly in concrete floors, power trowelling, curing, and liquid surface treatments are generally more efficient than binder content in achieving a surface with good abrasion resistance.

These statements are borne out by the findings of various authors:

Chaplin(1991) did abrasion tests [using an apparatus developed by the C&CA, =*rolling steel wheels*, see appendix U.4.06] and showed that a relatively low grade concrete (36MPa) which had been power trowelled thrice was as good as a high grade mix (85MPa) which had only been power floated. Therefore trowelling intensity has a large influence on abrasion resistance and may mask a low compressive strength / low cement content owing to changing the w/b ratio.

Effectively this shows that the w/b of the surface layer can be improved by a special form of mechanical compaction, and that cement content is not the sole criteria to be considered.

On the other hand a high binder content does offer a number of advantages:

- A measure of insurance against inferior aggregates or finishing processes or curing methods
- Improved cohesion
- Good rheology

The disadvantages are:
- Higher costs, although it may be possible to offset these costs by incorporating additives such as fly ash or ggbs.
- Greater shrinkage

Abrams(1921) did abrasion tests [using a Talbot-Jones Rattler [= 95mm *impacting steel balls,* see appendix U.2.01] and showed that increasing the cement content was accompanied by increased compressive strength and abrasion resistance. However at a cement content of about 60% by volume no further benefits were realized, and gains were relatively small after 30%. [Clearly these cement contents are very high, for experimental reasons. The principle demonstrated here notwithstanding is that too much cement undermines the relatively harder coarse aggregate's contribution in protecting the softer mortar/paste. (His coarse aggregate consisted of granite, slag, pebbles and limestone, with the granite mixes displaying the lowest abrasion wear)].

Dhir(1991b)*'s* results show that mixes with reduced paste content have increased abrasion resistance relative to mixes with higher paste contents, w/b being the same in both cases, (but the mixes with the greater paste content had the greater workability). Dhir reasons that lower paste content implies a higher proportion of harder wearing aggregate, and therefore greater abrasion resistance. [He used an apparatus developed by the C&CA, =*rolling steel wheels,* see appendix U.4.06].

Dowson(1980) recommended the use of a void meter to determine the lowest possible percentage of air voids when considering aggregate selection for concrete pavers. In this way paste content could be minimized for a given w/c. [In effect this is a recognition that paste content beyond that which is required for full compaction is uneconomic and results in a reduction in abrasion resistance].

Dreijer(1980) reported on experiments in the Netherlands on two sets of concrete pavers. Abrasion tests [made according to the NEN 7000 sandblast test, =*impacting fine-abrasive*, see appendix 5.19] showed no improvement in abrasion resistance for an increase of 10% in cement. [This supports the view that above a certain binder content additional binder may have no further value, or even be detrimental, given that hardened paste is softer than most aggregates. (Given that the Netherlands experiences some very severe winters pavers are already made with a relatively high cement content, and a 10% increase may therefore not add any value to abrasion resistance)].

Lane(1986) observed that with *increasing* vibration power the water content of the paste and the thickness of the paste layer can be reduced, thus improving strength and reducing shrinkage. [Paving manufacturing machines are generally made with very powerful vibrators, so enabling paste content (and hence cement content) to be minimised.

On the other hand, if voids are to be minimised, there is a limit to how far paste content can be reduced. Evidence suggests that for a *given* compactive effort, increasing cement content, rather than decreasing it improves abrasion resistance. Sukandar (1993), Papenfus (1988), Clark (1980).].

Rushing(1968) did abrasion tests [using ASTM C418, =*impacting fine-abrasive*, see appendix 5.21] and found that increasing the quantity of cement increased the abrasion resistance, up to 356 kg/m^3 (6 bags/yrd^3). However when increasing further to 415 kg/m^3 (7 bags/yrd3) there was sometimes a drop in abrasion resistance. Assuming he aimed at constant workability, 415 kg/m^3 would have had a lower w/c than 356 kg/m^3. [An explanation for the reduction in abrasion resistance of the mix corresponding to the higher proportion of cement is that the paste resistance to wear is not as high as that of the aggregate, and that having achieved a sufficiently low w/c to achieve adequate bond to the aggregate, any further increase in paste content merely exposes more of the less wear resistant paste component].

Bettencourt Ribeiro(1998) did abrasion testing on roller compacted concrete [using an apparatus similar to DIN 52108 *sliding fine-abrasive,* appendix 5.02]. He found that as binder content increased from 90kg/m^3 to 140kg/m^3 to 190kg/m^3, the abrasion resistance stayed virtually constant *for a given aggregate type*, with a slight drop for the 90kg/m^3 mix. In contrast compressive strength improved quiet dramatically. He attributes this to the nature of the test, where the mode of attack is tangential and of a gentle scratching nature. [This is a classic example of a test where a superior aggregate hardness controls the rate of abrasion. The fine-sand abrasive grinds away but imparts minimal shock to the specimen, and hence aggregate particles are not easily loosened]. Even if they should be loosened they cannot be 'desegregated' as the loosened particle is likely to remain trapped in the matrix, unable to drop out, being too close to the base/wear-plate, given the narrow distance between sample and base/wear-plate (approximately equal to the thickness of the fine-sand abrasive].

DeAlmeida(1994) did abrasion tests using the Dorry Hardness apparatus [=*sliding fine abrasive*, see appendix U.5.01]. Considering binder contents between 294 kg/m^3 and 378 kg/m^3 he found that abrasion resistance increased with decrease in binder paste content. [Here again, the higher proportion of paste relative to the harder granite coarse aggregate resulted in a lesser average resistance to wear.]

Conclusions on 'Binder Quantity'

The findings of the various authors may be reconciled, summarised, and applied to the manufacture of concrete pavers as follows:

In lean mixes, increased binder:

- reduces voids and therefore greatly improves abrasion resistance
- reduces the w/b and therefore improves abrasion resistance
- increases the bond between aggregate and paste, resulting in reduced loss of aggregate (this effect is generally a natural consequence of the first two effects)
- is not always necessary if w/b can be increased by means of surface densification, e.g. power trowelling
- appears to make no difference to abrasion resistance, if the abrasion test used does not permit the release of loosened aggregate

In rich mixes, increased binder:

- reduces the w/b, and therefore improves the abrasion resistance of the paste component of the mix
- reduces the proportion of aggregate, and this may lead to a reduction in abrasion resistance, given that the aggregate is harder than the paste

Clearly the above two effects give opposite results, and this accounts for the apparently unexpected findings of some authors. The findings are also dependent on the type of abrasion test that was employed.

In concrete pavers, the work done by Sukandar (and confirmed in chapter 6 of this thesis), indicates that semi-dry paving mixes with less than 14% binder contents may be classified as 'lean'.

2.2.1.2 Binder Type

Influence of binder type

Different binder types have different physical properties. For example fly ash contains round silica spheres, while a silica fume particle is two orders of magnitude smaller than a cement particle. Differences also exist in chemical composition. These differences influence the morphology of the gel.

In the foregoing sections the factors that influence the quantity of water and the quantity of binder used in the mix were studied. Clearly water and binder content determine w/b, and this in turn influences the paste structure. Essentially, the relative proportion of binder and water determine the spacing of the individual binder particles, which in turn determines the density of the gel, and hence the strength of the paste, and finally this impacts on abrasion resistance.

We shall now consider the various possible types of cementitious material that make up the binder, including fly ash, ground granulated blast furnace slag, silica fume, Portland cement, polymers and other additives. We shall see that binder type also affects the structure and strength of the paste, as well as abrasion resistance (to the extent that paste quality has an influence on abrasion resistance).

2.2.1.2.1 Fly Ash

(a) General benefits

PFA(1983) and **PFA(1985)**: Fly ash is reported to have many benefits including lower water demand / improved workability, more cohesion in the mix / less segregation, less bleeding, less shrinkage, better impermeability / durability, superior off shutter finish, higher resistance to chemical attack, better long-term strength development, resistance to alkali aggregate reaction etc.

PFA(1983) The physical and chemical characteristics of fly ash vary depending on the source of coal, the power station, the load on the power station and the type of equipment used to extract ash from the flues. Nevertheless, for a given power station using coal from a single source, tests have shown that fly ash is less variable than Portland cement, and may be regarded as the most consistent ingredient for the making of concrete. (The strength development of cement is known to vary between sources and also within a source over a period of time). Therefore substitution of cement with a well controlled source of fly ash leads to a reduction in variableness of results.

The paragraphs that follow will review the literature to examine some of these benefits, as well as other characteristics of fly ash, to see how they affect the abrasion resistance of concrete incorporating this material.

(b) Strength contributors

PFA(1983) Fly ash contributes towards the strength development (and therefore potential abrasion resistance) of concrete in a number of ways:

(i) by reducing the water content required for a given workability, thereby reducing the w/b ratio and so improving the quality of the paste. [This is achieved by virtue of the spherical shape of some of the fly ash particles, which increases the mobility of the paste, making it possible to use less water].

(ii) by increasing the quantity of the paste in the mix [If a proportion of the cement is replace on a 1kg for 1kg basis with fly ash, there will be a greater absolute volume of binder, since fly ash has a specific gravity of 2,2 compared to 3,14 for cement. Essentially this means that fly ash occupies 1,43 times more space (3,14/2,2). The effect of this, again, is to increase the mobility of the mix, as there is a greater proportion of binder/paste (volume sense) in the mix to lubricate the aggregate. This allows a further reduction in w/b].

(iii) By the pozzolanic reaction of pfa (i.e. the fly ash reacts with the free lime produced by the cement during the process of hydration to form additional cementitious compounds).

(iv) Pfa contributes to strength within the first 24 hours, but this reaction is not a pozzolanic one as chemical studies indicate that the pozzolanic reaction is only material after some weeks. This phenomenon is as yet unexplained.

(c) Mix design considerations

PFA(1985) The design of fly ash mixes is very similar to that of mixes with only Portland cement, after two aspects have been considered:

(i) Fly ash has a lower cementing efficiency than Portland cement, but this is almost offset by the lower water demand so that generally, 28-day strengths will be only slightly reduced.

(ii) Strengths at 90 days and beyond will be increased for replacement levels of up to 30%, owing to ongoing pozzolanic activity.

(iii) An increased stone content is possible because of the improved workability and the higher proportion of paste owing to the lower relative density of fly ash compared to the replaced OPC.

(d) Pozzolanic effect

Since failure of a pavement surface by wear is slow and progressive, the long term beneficial effect of ongoing pozzolanic reactions is likely to increase the abrasion resistance and extend the life of the pavement. [Gordon(1991)]

Paving: Following extensive abrasion resistance testing in the laboratory, 28-day old pavers showed the fly ash mixes to be inferior to the 50%ggbs/50%opc control [Papenfus(1988)]. However the same tests repeated after 7 years on equivalent blocks which had been exposed to the elements, showed that the fly ash paving incorporating up to 28% fly ash now had very good abrasion resistance and were noticeable superior to the ggbs and silica fume mixes [Papenfus(1995)]. This trend was also seen from wear measurements taken at the site after some six years of traffic. It is evident that the ongoing pozzolanic reactions associated with fly ash mixes substantially improve abrasion resistance.

(e) Source

Naik(1997) used the a modified version of ASTM C944 [=*sliding fine abrasive* beneath rolling dressing wheels, see appendix U.3.09] to assess the abrasion resistance of fly ash concretes with cement replacement levels of 40%, 50% and 60%, from three different sources. At 40% replacement only one source yielded equivalent abrasion resistance, and one source resulted in substantially more wear, generally twice as much. At 50% and 60% replacement levels abrasion resistance was progressively lower.

(f) Fineness

It is generally reported that the finer the fly ash the more reactive it is. For example the finer Letabo Field 2 PFA is widely reported to be more reactive than the Matla PFA referred to in PFA(1983).

Dhir(1998) used an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06] do abrasion tests on 35 MPa and 50 MPa fly ash concretes, where the fly ash constituted 30% of the binder. For each of the two strength categories six different fly ashes were used, each with its own unique grading. In particular the percentage of material retained on the 45 micron sieve varied from a low of 3,0% to a high of 41,5%. The results of the tests showed that there was a slight difference in abrasion resistance in the case of the 35MPa specimens, with the '41,5% retained' material registering the lowest wear, while the '3,0% retained' material had the greatest depth of penetration. This is somewhat surprising as the trend in the parallel compression testing was that compressive strength gradually but consistently improved with decreasing % 45 micron sieve retention. In the case of the 50 MPa specimens there was no observable trend, and all six fly ashes performed almost identically, with variation of depth of penetration limited to between 0,79mm and 0,85mm. Dhir concluded that the observed differences were slight and that 'equivalent performance in terms of durability' (including abrasion resistance) is achievable for the various fly ashes considered.

It may be concluded that the effect on abrasion resistance of fly ash particle size/fineness is not clear, but it seems reasonable to assume that as far as it increases or decreases the strength of the paste, it will have a similar effect on abrasion resistance.

(g) Acid attack

DeBielie(1997) considered the problem of acid attack on concrete in animal houses, where lactic and acetic acids are formed in meal – water mixtures, resulting in pH's as low as 3,8. On top of this the concrete is subject to abrasion wear in the form of cow hoofs, as well as the action of high velocity water from cleaning hoses. DeBielie subjected various concrete prisms to solutions of lactic and acetic acid, followed by mild abrasion in the form of brushing to remove loose material. He found that specimens incorporating fly ash had improved resistance to the acid solution. However the amount of material lost depended on the pH of the solution. At a pH of 2.0, fly ash mixes had 20% less loss of material. The improved resistance to acid attack of concrete incorporating fly ash can be attributed to changes in the paste as follows:

(a) a reduced content of Ca(OH)2
(b) a lower Ca/Si ratio of the C-S H
(c) and a refined pore structure

(h) Effect of replacement level

PFA(1983) Fly ash reacts with the free lime produced by the cement during the process of hydration to form additional cementitious compounds. This is referred to as the pozzolanic reaction of fly ash. Normally the pozzolanic reaction before age two weeks is minimal. However by the time the concrete is 28 days old, at fly ash replacement levels of between 15% to 25%, parity of strength with 100% Portland cement mixes is generally achieved. Thereafter fly ash should gain on 100% opc mixes.

The above stated guidelines on the strength development of fly ash i.e. (a) through (h) are now considered in the light of abrasion resistance, taking into account the degree of replacement. To simplify matters, the literature is divided into three categories, to categorise the performance relative to 100% Portland cement concrete:

(i) **Improved abrasion resistance**

Dhir(1991a) showed that fly ash mixes at 30% replacement levels had increased 28-day abrasion resistance relative to 15% replacements, which in turn were superior to 100% OPC mixes. This applied to compressive strengths ranging between 20 MPa and 60 MPa. However at 90 days the abrasion resistance for the three mixes was nearly the same. His tests were done using an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06]

Papenfus(1995) did abrasion tests [using *rolling steel balls,* see appendix U.2.15] on six year old concrete pavers subject to 'natural' curing, and found that the abrasion resistance of mixes incorporating 14%, 21% and 28% fly ash was significantly more than that mixes using a 50/50% blend of OPC and GGBS.

(ii) **Equivalent abrasion resistance**

Chaplin(1990) did abrasion tests [using an apparatus developed by the C&CA, =*rolling steel wheels*, see appendix U.4.06] and obtained comparable 28-day abrasion resistance with 35% fly ash substitution relative to 100% Portland cement mixes.

Liu(1991) referred to tests done by the US Army Cops of Engineers who found that replacement with fly ash (replacement level is not mentioned) results in mixes with equivalent abrasion resistance relative to 100% OPC mixes, for mixes of equivalent MPa with the same aggregate type.

Naik(1995), found parity in 28-day abrasion resistance up to 30% replacement with fly ash. This equivalency up to 30% replacement, also applied for 91 day tests, and 365 day tests. As expected, abrasion resistance generally improved with age. [He used a modified version of ASTM C944, =*sliding fine abrasive beneath rolling dressing wheels*, i.e. similar in principle to appendix U.3.08].

(iii) **Lower abrasion resistance**

Goncalves(1998) used PrEN 1338 [=*impacting fine-abrasive*, appendix 5.12] to determine the influence of 0%, 30% and 50% fly ash replacements on the 28-day and 180-day abrasion resistance of concrete. He found that abrasion resistance varied inversely with cement replacement, capillary absorption and oxygen permeability.

Capillary absorption in particular is highly affected by fly ash replacement, in that it increased by more than 100% for a fly ash replacement of 50%.

However although porosity results in reduced abrasion resistance, the reductions are comparatively small compared to the level of replacement. At 50% replacement, abrasion wear increase was limited to about 15%, while at 30% replacement the abrasion wear was up by as little as 4%. [The relatively insignificant changes in abrasion resistance relative to absorption may be ascribed to the nature of the abrasion test, which grinds the aggregate and paste at the same rate. The paste is therefore protected by the harder aggregate providing it has sufficient bond strength to hold the aggregate (and since the weakest compression test yielded 30 MPa, aggregate/paste bond would indeed have been adequate to prevent aggregate pluck-out). This test then does not yield very reliable information for determining variation in abrasion resistance owing to variations in binder/paste strength, it is a better test of aggregate hardness].

Naik(1994) found that the 28-day abrasion resistance [according to a modified version of ASTM C944, =*sliding fine abrasive beneath rolling dressing wheels*, similar in principle to appendix U.3.08] of a 50% fly ash replacement mix was less than the 100% cement equivalent but more than the 70% fly ash replacement mix. At 90 days the 100% mix was still superior, while the 50% and 70% mixes had equivalence. As could be expected the 90 day abrasion resistances exceeded the 28-day values.

Gordon(1991) did abrasion tests according to PCI.TM.7.11 [=*sliding wire bristles* , appendix U.6.02] and found that at 28 days the abrasion resistance of fly ash mixes at 15% replacement was similar to the 100% OPC mix, but was less at 30% and 50 % replacement. However by 90 days parity between the OPC and fly ash existed, regardless of replacement level.

Dhir(1991a) did abrasion tests using the C&CA apparatus[=*rolling steel wheels,* see appendix U.4.06] and obtained improved abrasion resistance at 28 days with replacement levels up to 30%, but offers no explanation why 91day and 365day results with full curing show a reduction in abrasion resistance relative to 28-day results. Experimental testing therefore does not always support the view that ongoing pozzolanic activity necessarily improves abrasion resistance.

Summary and Conclusion

It is clear from the above that the expected trends do not always prevail. Nevertheless certain general trends have been confirmed:

- for substitutions up to 30%, fly ash mixes have very similar 28-day abrasion resistance relative to 100% OPC
- at levels of 50% replacement and more, fly ash mixes have lower abrasion resistance
- fly ash mixes improve with age (i.e.at 90 days, 365 days) more than 100% Portland cement mixes
- fly ashes with finer gradings tend to contribute more towards abrasion resistance
- consideration must be given to the source since not all fly ashes are equally reactive in concrete. An example is Matla power station relative to Letabo power station, where ash from the latter source is known to be more reactive.
- fly ash performs relatively well in an acid environment

Finally, the results of this investigation, reported in volume 1, show conclusively that concrete surfaces incorporating fly ash perform very well in abrasion resistance. This is because abrasion wear is a slow process, and this suites fly ash, given its superior long term strength development relative to 100% OPC mixes. At the same time, for substitution levels of 30% and less, they are also approximately equivalent to 100% OPC mixes in terms of 28-day abrasion resistance.

2.2.1.2.2 Ground Granulated Blastfurnace Slag

According to **Fulton(1986):** ‘Blast furnace slag is the non-metallic mineral by-product formed when iron is produced in a blast furnace.

The chemical compounds in blast furnace slag are similar to those in Portland cement clinker, but the proportions of the constituents in the two materials are different. When the molten slag is cooled rapidly (usually by quenching with water) the material granulates into sand sized particles. The product obtained by grinding there granules is known as ground granulated blast furnace slag (GGBS), and has valuable cementitious properties when used with Portland cement.

On its own GGBS shows little hydraulicity. On initial contact with water, calcium ions are released into solution, but the concentration of Ca(OH)2 is very low compared to that obtained with Portland cement.

The hydration of a mixture of Portland cement and GGBS can be envisaged as the normal hydration process of the cement, resulting in a continuous release of calcium hydroxide which in turn activates (and is partly taken up in) the hydration of the slag'.

GGBS may be mixed with OPC in various proportions. Figure 2.7 gives a number of envelopes of expected strengths that may be expected in concrete with varying proportions of GGBS, from 0% through 70%. From this is may be seen that GGBS has the effect of:

- retarding early strengths. (Curing should therefore be extended for longer periods).
- giving parity of strength at 28 days
- advancing long term strength for substitutions of up to 50%
- narrowing the envelope at 28 and 90 days at higher substitution percentages, as its physical and chemical properties are relatively consistent.

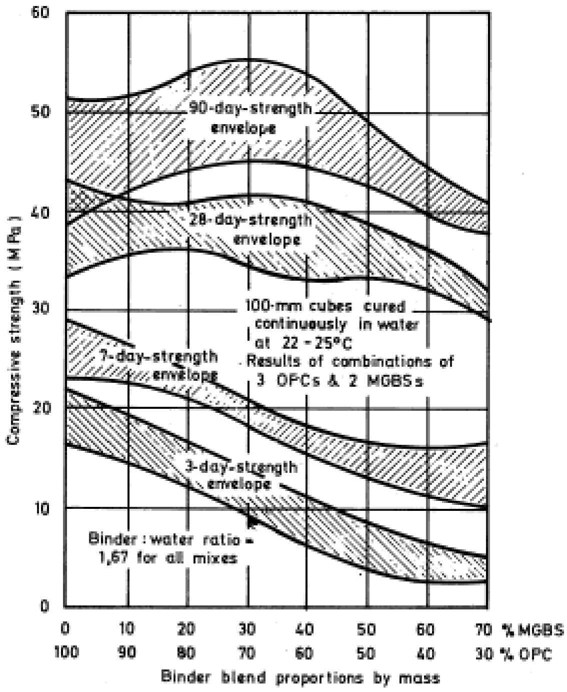

Figure 2.7 Effect on concrete strength of different blends of OPC and GGBS (results of tests done by PCI 1985/86) [Fulton 1986, pg 88]

Chaplin(1991) did abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels*, see appendix U.4.06] and showed that providing curing is carefully attended to, power finished floors made with a 50% GGBS substitution have comparable 28-day abrasion resistance. On the other hand inadequate curing resulted in relatively inferior abrasion resistance.

Papenfus(1995) used the MA20SA abrasion test [=*rolling steel balls,* see appendix U.2.15] to determine the abrasion resistance of 50% substitution GGBS concrete pavers that had been exposed to the elements for seven years. Although there was a dramatic improvement in abrasion resistance relative to 28-day tests made on the identical specimens, pavers incorporating substitutions with fly ash up to 28% had improved substantially more. While much of the improvement in abrasion resistance in both cases was probably the effects of carbonation, the fact remains that the fly ash mixes were clearly more resistant to the abrasion test.

Fernandez(1990) did abrasion tests using ASTM C779-82 Proc C [=rolling steel balls, see appendix U.2.12] and showed that the 365 day abrasion resistance of slag replacement mixes at levels of 25% and 50% was substantially less than mixes incorporating 100% Portland cement, at w/b ratios of 0,45 and 0,55. At w/b = 0,70 the three blends had approximately the same abrasion resistance. These trends were the same for compressive strength. Curing was done at RH = 100% till time of testing.

Conclusions and Recommendation

There appears to be some divergence in the literature. For example figure 2.7 suggests that at moderate GGBS substitutions (e.g. 25%) the long term abrasion resistance should improve. However Fernadez found that depending on the w/c it was either equivalent or substantially lower even when curing at RH=100%. Chapmin found parity at 28 days.

From these limited findings it seems that it is not possible to come to any definite conclusions, and further research is recommended.

2.2.1.2.3 Silica Fume

Before the effect that silica fume has on abrasion resistance is considered, it will be useful to consider some of its physical characteristics and the mechanisms whereby it enhances the mechanical properties of the concrete. A number of authoritative sources will be consulted.

(a) Physical characteristics

Silica fume has also been referred to as silica dust, condensed silica fume, microsilica, and fumed silica. **ACI committee 234(1995)** consider that the most appropriate term is silica fume.

As the name implies, silica fume originates from the oxidation of SiO exhaust gases that escape via overhead flues from molten Si reduction in submerged electric arc furnaces, to be captured in bag house filters. It consists primarily of very fine smooth spherical glassy particles (of the order of one tenth of a micron) with a surface area of approximately 20000 m^2/kg. It has been calculated that for a 15 % silica fume replacement of cement, there are approximately 2,000,000 particles of silica for each grain of Portland cement in a concrete mixture (ACI Committee 234:1995). While the chemical composition of silica fume will vary according to the type of silicon alloy that is being produced (e.g. ferrosilicon of various Si contents, calcium silicon, ferrochromium silicon, silicomanganese), the common denominator in all of these is the high content of SiO_2, from 85% upwards to 98%.

Khayat(1998) summarises by stating that the unique characteristics of silica fume that make it suitable as a supplementary cementitious material are its fineness, highly amorphous nature, and elevated content of SiO_2.

(b) Mechanisms of enhancement

From a survey of the literature Khayat outlines the following attributes that promote silica fume's performance in concrete:

- Filler effect: The small spheres act as fillers since they occupy some of the space between the relatively coarser cement grains that would otherwise be occupied by water. This results in a denser matrix with a better gradation of fine particles, and it has been shown (by Detweiler and Mehta (1989) who did tests with inert but very fine carbon black) that such densification leads to improved mechanical properties of cement pastes.
- Pozzolanic properties: The resulting reaction between silica fume and $Ca(OH)_2$ increases the volume of C-S-H structures. Alternatively, each silica fume particle

can act as a 'nucleation site' for precipitation of $Ca(OH)_2$. As a result numerous small crystals of $Ca(OH)_2$ can form rather than a few large ones. This absence of large and weak crystals of $Ca(OH)_2$ enhances the mechanical properties of concrete, which is similar to metallic alloys where grain size refinement improves strength.

- Pore refinement: The pozzolanic reactions result in a reduction of the total volume of capillary pores in the cement paste. In particular the volume of large capillary pores are reduced resulting in a finer and less interconnected pore structure.
- Transition Zone refinement: It follows that there will also be a reduction in the porosity of the transition zone between cement paste and aggregate which increases the strength and impermeability of the concrete. In a conventional concrete, the transition zone can have large and oriented $Ca(OH)_2$ crystals which form weak zones in the concrete. The thickness of the transition zone can be drastically reduced by adding silica fume to the concrete since silica fume reduces bleeding and hence the amount of water accumulated under aggregate.

ACI committee 234(1995) state that the presence of silica fume accelerates the hydration of cement during the early stages. Equal volumes of an inert filler (calcium carbonate) have produced the same effect. The mere presence of numerous fine particles-whether pozzolanic or not, has a catalytic effect on cement hydration. In hardened concrete, silica fume particles increase the packing of the solid materials by filling the spaces between the cement grains in much the same way as cement fills the spaces between aggregate particles in concrete.

In the presence of hydrating Portland cement, silica fume will react as any finely divided amorphous silica-rich constituent in the presence of calcium hydroxide – the calcium ion combines with the silica to form a calcium silicate hydrate through the pozzolanic reaction.

ACI committee 234(1995) report that concrete containing silica fume shows significantly reduced bleeding. This effect is caused primarily by the high surface area of the silica fume; in wetting it there is very little free water left in the mix for bleeding. Additionally, the silica fume reduces bleeding by physically blocking the pores in the fresh concrete. It has also been found by mercury intrusion porosimetry that silica fume makes the pore structure of paste and mortar more homogenous by decreasing the number of large pores. It also transforms continuous pores into discontinuous ones, reducing permeability.

Khayat(1998) refers to work done by Jahren, P. 1983, which shows that the gains in strength from the 'pozzolan-effect' and 'microparticle-effect' are offset to some extent by the water 'demand-effect', which of course impacts negatively on the w/c ratio. An increase in water demand follows from the very high surface area of silica fume (20000 m^2 / kg). It is therefore necessary to use superplasticizers to lower the w/c ratio. Jahren's graph does however show a positive contribution at small dosages of silica fume. This may be the result of silica fume particles entering the spaces within cement flocculations and displacing the water that would otherwise have occupied this space, which is now released as free water to facilitate the fluidity of the mix.

The above quotations have described the various mechanisms whereby silica fume enhances concrete. In practice this means that substantial compressive strength gains can be made by using SF as an addition to the cementitious content of the concrete. (Alternatively two to five parts of cement can be replaced by one part silica fume to achieve the same 28-day compressive strength).

Given the right blend of aggregate and cement these strength gains can be quite dramatic. For example **Microsilica South Africa (Pty) Ltd** speak of cases where a 10% replacement in cement has achieved a gain in compressive strength of up to 30%.

(c) Effect of silica fume on abrasion resistance

Various researchers have found that while compressive strength is substantially enhanced by silica fume, this is not always the case with abrasion resistance. Investigators have come to various conclusions. Some found it was very effective in improving abrasion resistance, others found modest improvement, while others, still, noticed a reduction. These findings are grouped and considered below:

(i) Substantial improvements to abrasion resistance

Shi(1997) found that 28-day abrasion resistance of mortar was very substantially improved by the addition of 15% silica fume. The wear from the ASTM C944-90a [=*rolling dressing wheels*, see appendix U3.09] was reduced from 1,17mm for plain mortar to 0,145mm for silica fume specimens. Silica fume addition was also found to be slightly more effective than latex (styrene butadiene copolymer) addition. Silica fume addition also significantly improved the tensile modulus, but not the tensile strength. He therefore concludes that silica fume's improved abrasion resistance is a result of improved resistance to deformation rather than resistance to fracture. [Given that he cured his test specimens in air at a RH of 33%, it is very probable that the good performance of the silica fume concrete is partly related to the greater impermeability of the silica fume mixes, whereas the surfaces incorporating only Portland cement as the binder would have been more porous and therefore dried out much faster, resulting in retardation of hydration, and consequently inferior abrasion resistance. The second reason for the enhanced abrasion resistance of the silica fume concrete is simply that it is known to accelerate strength development up till 28 days, the age of testing in this case. However after 28 days strength development is inferior relative to portland cement].

Ghafoori(1999) did abrasion tests using ASTM C779 Proc C [=*rolling steel balls,* see appendix U.2.12] to determine the effect of replacing a proportion of the fine aggregate with silica fume. Four levels of fine-aggregate replacement were considered i.e. 5%, 10%, 15% and 20%. Substantial improvement at 28 days was obtained at the 10% level (49%), with 15%, 5% and 20% showing progressively smaller gains.

Chisholm(1997) did abrasion tests using the C&CA apparatus [=*rolling steel wheels*, see appendix U.4.06] on power floated and trowelled concrete floors incorporating 7% and 10% geosilica, an amorphous silica mined and processed in New Zealand. Polythene sheeting was used initially to prevent drying out, followed by 7 days of ponding, followed by air curing. The 56 day abrasion resistance of the 7% geosilica mix was in the order of 50% improved relative to the control (0% geosilica), and even outperformed the 10% geosilica mix. Geosilica particles vary in size between 0,4 and 10 microns, and are thus considerably larger than silica fume particles, that are generally less than a micron.

Helland(1991) reported on the use of high strength concrete (HSC) for concrete road and bridge surfaces in Norway, which had to have exceptionally good abrasion resistance to resist the very abrasive action of steel studded tyres on vehicles. This is generally achieved by means of using silica fume, together with a superplasticized very low w/b ratio mix. Examples of this are:

- the Valerenga tunnel, where 7% silica fume in a w/(c+s) = 0.32 mix resulted in drilled out cores with a characteristic strength of 92 MPa
- highway E-18 and E-6, where 5% silica fume in a w/(c+s) = 0.36 mix resulted in a characteristic 28-day cube strength of 90 MPa
- the Ranafoss bridge, where 9% silica fume in a w/(c+s) = 0.34 mix resulted in a characteristic 28-day cube strength of 110 MPa
- the Smestad tunnel, where 14,3% silica fume in a w/(c+s) = 0.22 mix resulted in a characteristic 28-day cylinder strength of 130 MPa. Specimens sawed out from

the actual pavement and tested in an abrasion machine (NORCEM's road tester), resulted in 0.47mm/10000 revolutions-of-four-studded-truck-tyres-at-63km/hr, when tested dry, and 1.04mm/10000 revolutions for wet conditions. This is comparable to massive granite. Presumably this concrete was also 28 days old.

Tveter(1994) found that a 10% substitution with silica fume 'gives increase in wear-resistance compared to concrete with the same level of compressive strength. This effect is more obvious on wet wear properties than on dry'. He does however not give any indication of the degree of improvement. His abrasion tests were made with the NORCEM tester, (see appendix U.3.03), which subjects the specimens to *rolling studded tyres*.

(ii) **Modest improvements in abrasion resistance**

Horiguchi(1995) did abrasion tests using *impacting steel balls* in water (see appendix U2.06) and found that 15 % silica fume addition made modest but consistent improvements to the 'surface fatigue wear' for a range of mixes (of different coarse aggregate : fine aggregate). The curing regime and age of the test specimens is not stated.

Laplante(1991) did 28-day abrasion tests according to ASTM C779 Proc C [= *rolling steel balls,* see appendix U.2.12] and showed that where silica fume improved compressive strength from 32% to 53%,corresponding improvements in abrasion resistance were moderate. He found that variations in coarse aggregate type and w/b were more significant.

Ramezanianpour(1999) did 40 day abrasion tests on silica fume mixes using ASTM C418 modified to water-sandblasting [*impacting fine-abrasive*, appendix U5.22]. His replacement levels were 0%, 2%, 4%, 6%, 8% and 10%. Generally his results show a substantial improvement in compressive strength from 0% to 2%, and then a small increase for each increment thereafter. At 10% and at 120 days the 2% mix was 25,6% up on the control, while the 10% mix was up 41,4%. A similar pattern was observed for the abrasion resistance, but here the % gains relative to the control were much less. The 2% mix was up only 7,4% while the 10% mix was up only 12,5%.

Ramezanianpour attributes the increase in abrasion resistance 'to the compressive strength and the transition zone of the concretes incorporating silica fume', but makes no attempt to explain the much lower percentages.

[A possible explanation: In *compressive strength testing* cracks are likely to start at the interface of the two dissimilar materials, i.e. the paste/aggregate interface. This also happens to be the zone where the paste is weakest. The relatively large gains in compressive strength therefore can be attributed to the improvement that silica fume makes in this crucial zone, as explained by Khayat(1998). It is therefore the improved ability of the silica fume rich paste to resist crack initiation at the aggregate/paste interface and its subsequent propagation that results in the superior compressive strength for the silica fume mixes.

On the other hand, in the case of the 'water sand-blast' *abrasion test*, failure is not related to crack propagation in the interfacial transition zone, but rather to the ability of the paste to resist the cutting/shearing effects of the impacting sand particles. Furthermore, since the sand particles may typically be in the order of 0,5mm to 1,0mm (the author does not divulge this information) the sand will be unable to concentrate the attack on the 50 micron thick transition zone as it erodes the paste between two aggregate particles. The attack will be on the full width between the two adjacent aggregate particles. (The author does not give the relative proportions of coarse aggregate and fine aggregate). Furthermore the impact of a small piece of sand at a local area of paste will undoubtedly result in some microscopic crushing at the microscopic contact points, (over and above the cutting/shearing already referred to), but the nature of the load is too small to cause a sufficiently large stress build-up resulting in crack propagation, even in the relatively weaker transition zone.

It may therefore be postulated that whereas silica fume is known to be effective in improving the interfacial transition zone which plays an important role in compressive strength, it is not as effective in a sand blast test where the interfacial transition zone plays a lesser role].

(iii) **Reductions in abrasion resistance**

de Almeida(1994) did abrasion tests using the Dorry Hardness apparatus [=*sliding fine abrasive*, see appendix U.5.01] and showed that for high strength concretes substitutions of 10% of silica fume reduced the 28-day abrasion resistance considerably. For example at 84MPa the wear on the silica fume mix was 42% more than the 100% cement mix. This was against expectations, and deAlmeida has no certain explanation, although he attempts to link it with the effects of self-dessication. It should be noted that the testing was carried out on the face of a 100mm cube that had been sawn off 25mm from the top face, and thus surface effects should have been eliminated. Similar trends were noted for fly ash, but these anomalies were on a much smaller scale.

(iv) **Sectional Conclusion:**

Most investigators found that abrasion resistance is enhanced by silica fume substitution or addition. This is expected from its attributes, explained in '(b) Mechanisms of enhancement'. The type of abrasion test however does affect its performance relative to its compressive strength performance.

(d) Rate of strength development

Khayat(1998) refers to work done by Maage M et al, and shows that whereas a mix incorporating silica fume has superior early strength, after 28 days the control mix without silica fume becomes progressively stronger, so that at age one year, the control is 10 to 20 percent stronger. After one year the situation stabilises and the strength difference appears to remain almost constant, even as far as 10 years. [A possible explanation is that the refined pore structure in silica fume mixes inhibits ongoing hydration thus allowing a more permeable concrete to hydrate longer, but eventually this hydration also renders it relatively impermeable].

Papenfus(1995) used the MA20SA test [=*rolling steel balls,* see appendix U.2.15] to determine the six year abrasion resistance of pavers with silica fume binder replacements levels of 5% and 10%, the balance of the binder being a 50/50 blend of OPC and GGBS. The improvement in six year abrasion resistance relative to the 28-day abrasion resistances was substantially less than the same ratio for pavers incorporating fly ash.

These findings are very much in line with the comments of the **ACI Committee 234(1995)**: 'At 28 days the compressive strength of silica fume is always higher and in some instances significantly so. The contribution of silica fume to strength development after 28 days is minimal'. Although these comments apply to compressive strength testing, they confirm the similar trends seen to apply to abrasion resistance, as stated by Khayat(1998) and Papenfus(1995).

(e) Resistance to acid attack

DeBielie(1997) considered the problem of acid attack on concrete in animal houses, where lactic and acetic acids are formed in meal – water mixtures, resulting in pH's as low as 3,8. On top of this the concrete is subject to abrasion wear in the form of cow hoofs, as well as the action of high velocity water from cleaning hoses. DeBielie subjected various concrete prisms to solutions of lactic and acetic acid, followed by mild abrasion in the form of brushing to remove loose material. He found that specimens incorporating silica fume had improved resistance to the acid solution. However the amount of material lost depended on the pH of the solution. (At a pH of 2.0, fly ash mixes had 20% less loss of material relative to silica fume). The improved resistance to acid attack of concrete incorporating silica fume can be attributed to changes in the paste as follows:

- a reduced content of Ca(OH)2
- a lower Ca/Si ratio of the C-S-H
- and a refined pore structure

(f) Treatment with sulphuric acid

Li(1998) did abrasion testing using ASTM C944 [=*rolling dressing wheels, see appendix 3.09*] and found that abrasion resistance increased by 20% where the silica fume (15% by weight of cement) was surface treated/etched with sulphuric acid, relative to the equivalent untreated silica fume. This improvement is thought to be due to increased bond strength between silica fume and the cement matrix facilitated by the increased specific surface area of the etched silica fume.

Sectional conclusion

On balance the literature indicates that the inclusion of silica fume results in a moderate to substantial improvement in abrasion resistance. This improvement is most noticeable at 28 days, and contribution of silica fume to strength development and abrasion resistance after 28 days is minimal. It therefore appears that silica fume is not ideally suited to abrasion resistance, which is generally a long term process. However, in very arid regions, where no ongoing curing may be expected, the relatively high 28-day result, which may be boosted by effective curing techniques, could be advantageous. Other than this somewhat unusual application, pavers incorporating fly ash are generally more economic and will have superior long term performance in conditions of ‘natural’ curing.

2.2.1.2.4 Ordinary Portland Cement

Cement is made up principally from raw materials such as limestone, shale (or clay) and iron oxides. Although a great deal of effort goes into carefully homogenizing the material, there will inevitably be variations in the final blend. Every cement factory therefore produces a cement with a distinctive chemical composition, albeit somewhat variable, according to the restraints placed upon it at source of raw materials.

This variability is illustrated by comprehensive testing that was done on cements from eight different sources (**PFA:1983**). The results demonstrated that different sources of the same nominal grade cement had distinctly and not insignificantly different strength characteristics, both in terms of the rate of strength development as well as the eventual 28-day compressive strength, (28-day results ranged between 31,5MPa to 38,5MPa).

Explaining these variations in terms of gel/space ratio (which governs strength) it may be said that different cements require different hydration time in order to produce the same quantity of gel.

Although the European Standards, now adopted in South Africa, place an upper as well as a lower limit on the 28-day strengths, requiring cement producers to stay within a window of 20MPa, this window is so wide that variability in results is still a function of source and even time of purchase from that source, given that results at any one source also fluctuate.

The various producers also manufacture three different cements for three different grades of structural concrete i.e. 32,5MPa, 42,5MPa, 52,5MPa, the latter clearly in the domain of what was formally referred to as RHPC.

Therefore in so far as paste strength influences abrasion resistance, the physical and chemical characteristics of the cement will influence abrasion resistance. This assertion has yet to be confirmed for South African cements (in terms of abrasion resistance).

2.2.1.2.5 Rapid Hardening Portland Cement

RHPC is merely OPC that has been more finely ground. In terms of the newly adopted European standards it is classified as Cem type 1 Class 52,5. The finer composition of the material makes it more reactive, resulting in faster strength development and higher strength at any given age. Generally the use of RHPC can lead to gains in the strength of concrete from 10% to 15%, relative to OPC, and insofar as the strength of the paste affects such aspects as paste/aggregate bond, paste hardness, and other aspects, RHPC can be expected to increase abrasion resistance as well.

Dhir(1991a) did abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06] and showed that the abrasion resistance of specimens made with RHPC was greater than that for OPC, both at 28 days and 90 days, although the 28-day gain was proportionally greater.

Schuman(1939) did abrasion tests using *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] and found increased abrasion resistance for specimens that were made with RHPC relative to OPC.

Schuman also found that OPC was far more vulnerable to inadequate curing relative to RHPC. [RHPC would achieve proportionally more strength prior to drying out].

Sectional conclusion

The hardness of the paste and its ability to bond the aggregate particles are both important contributors to abrasion resistance. RHPC, being finer and more reactive than OPC has been found to do better than OPC on both these counts.

2.2.1.2.6 Polymers

Polymers are thought to function by:

- facilitating compaction
- improving dispersion of cement particles
- reducing capillary pores by means of a secondary polymer-matrix within the primary cementitious system

In densifying and improving the paste structure it would appear that polymers perform a similar function to silica fume. For best performance they should always be added to the mix in their liquid form.

Various researchers have studied the effect of polymer addition on abrasion resistance. Certain polymers, such as certain types of styrene-butadiene rubber (SBR) and some pure acrylic polymers, are reported to dramatically improve abrasion resistance:

Liu(1991) stated that polymer mixes had the potential to greatly improve abrasion resistance. In decreasing order of effectiveness he ranked them as follows:

- vinyl ester
- methyl methacrylate
- polymer impregnation
- polymer Portland cement

Shaker(1997) did abrasion testing on latex modified concrete (LMC) using ASTM C779-89 Proc A [*sliding fine-abrasive* beneath revolving discs, see appendix U.5.15] and found the resistance to abrasion to be much improved compared to the equivalent conventional concrete without polymers. The penetration of the steel disc was reduced from 9,6mm to 5,6mm at age 28 days, and from 8mm to 4mm at age 90 days.

Shaker gives insights into LMC's improved performance:

Scanning electron microscopy of the LMC's microstructure showed that its improved performance relative to conventional concrete may be attributed to a more dense microstructure. The polymer latex was observed to form a polymer film that is well dispersed throughout the cement matrix forming a dense network which is interwoven with the cement matrix. Furthermore, a great reduction was observed in the size of the crystals formed throughout the polymer cement matrix, such as calcium hydroxide crystals, indicating a more dense microstucture than that of the conventional concrete.

The LMC was observed to have a pore size that is much smaller than that of conventional concrete. No crystal deposition, nor microcracks were found inside the pores. Most of the pores were filled with the polymer latex (leading to greatly improved impermeability).

The microcracks in the LMC were found to be bridged with the polymer film and also exhibited a smaller width compared to those microcracks found in the conventional concrete. Observations of the transition zone of the LMC showed a lack of the well formed crystals of calcium hydroxide in the vicinity of the aggregate indicating a less porous zone, and thus overcoming the weakest link in the conventional concrete. The micrographs indicated a well developed bond between the aggregate and the polymer-cement co-matrix. The polymer film seems to form a connecting phase between the aggregate and the cement matrix. This is evidenced by the 60% improvement in tensile strength over that of the equivalent conventional concrete.

Summarizing, LMC was found to have a dense microstructure, smaller discontinuous pores, less porous transition zone, better bond between the aggregate and the cement matrix, and bridged microcracks with respect to conventional concrete.

Shi(1997) found that 28-day abrasion resistance of mortar was very substantially improved by the addition of 20% latex (styrene butadiene copolymer). The wear from the ASTM C944-90a *rolling dressing wheels* [appendix U.3.09] was reduced from 1,17mm for plain mortar to 0,161mm for latex specimens. Latex addition also significantly improved the tensile strength (from 0,88Mpa to 3,03Mpa). He therefore concludes that latex's improved abrasion resistance is a result of improved resistance to fracture. [Given that the prevailing RH during the curing period was 33% it is quite likely that the improved impermeability afforded by the latex accounted for improved curing and hence improved abrasion resistance].

Beningfield(1995) did 28-day abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06] and found that increasing polymer concentration substantially improved abrasion resistance. For example, increasing the SBR (styrene-butadiene rubber) content from 5% to 10% of cement content (calculated by dry polymer weight of cement) improved the abrasion resistance by 100%.

Polymers are known to be very sensitive to w/b, with performance rapidly dropping off on either side of an optimum. Beningfield determined that by moving the w/b by as little as 0,04 off the ideal of 0,284 there was a seven-fold decrease in abrasion resistance. It appears from his work that polymers make the greatest impact on low w/c concretes, in the range of 0,27 to 0,29.

Horiguchi(1995) did tests with *impacting steel balls* in water (see appendix U2.06) and found that 15 % polymer (BTR) addition made modest but consistent improvements in the ‘surface fatigue wear’ for a range of mixes (of different coarse aggregate : fine aggregate). Polymer addition was slightly more effective than silica fume addition. This test is more of an impact test than an abrasion test, which may explain why the results are ‘modest’ relative to the findings of the other investigators. The curing regime and age of the test specimens is not stated.

Justnes(1994) concluded that the abrasion resistance of latex-modified mortar and concrete (PCC) depends on the type of polymer added, polymer-cement ratio and abrasion or wear conditions. In general, the abrasion resistance is considerably increased with an increase in polymer-cement ratio. Figure 2.B illustrates the abrasion wear of typical latex-modified mortar [from Ohama(1981)] tested according to Taber’s abraser. It may be seen that the abrasion wear of a PCC with a polymer-cement ratio of 20% decreases by 20 – 50 times compared with an unmodified mortar. However, it should be noted that the depth of penetration in figure 2.B is very shallow, and therefore these findings do not indicate whether or not the abrasion resistance at say a depth of 0,5 mm would also be improved by as much.

Even more dramatic improvements have been noted by other researchers. Justnes quotes Teichman(1976)’s results whereby PAE (i.e. polyacrylic ester) modified mortar with a polymer-cement ratio of 20% had an abrasion resistance 200 times higher than conventional mortar.

Paving: Given the ability of polymers to improve the paste structure of low w/c ratio concretes it seems likely they may have useful application in improving the abrasion resistance of concrete pavers. From the literature this does not appear to have been attempted as yet.

Sectional conclusion

The findings given above indicate that polymers can be very effective in improving abrasion resistance. There are a number of different types on the market, which appear to vary in performance. Some appear to be very sensitive to w/c, making their widespread use in the general market place difficult. There appears to be scope for the use of polymers in concrete pavers.

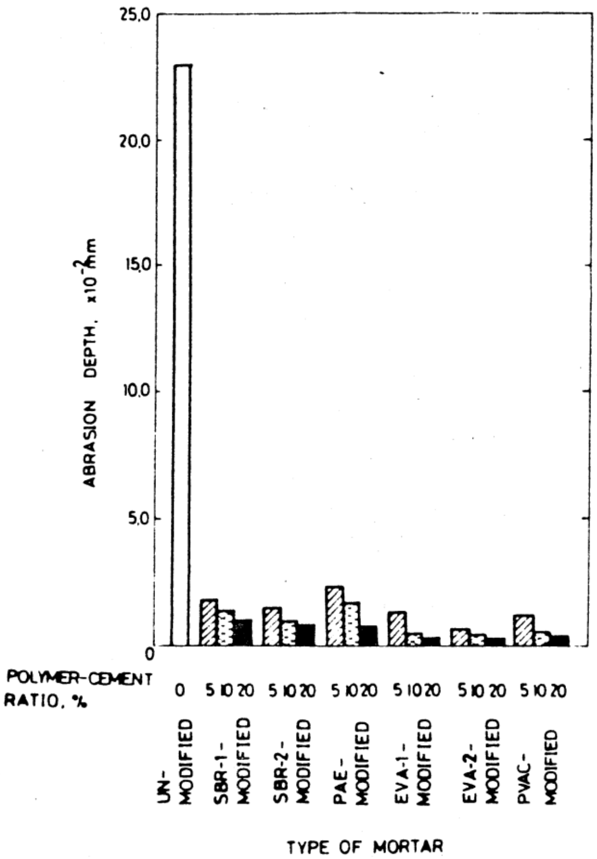

Figure 2.B Abrasion wear of typical latex-modified mortars (PCC) with different dosages and types of polymer (SBB = styrene-butadiene rubber, PAE = Polyacrylic ester, EVA = ethylene-vinyl acetate copolymer, PVAC = poly vinyl acetate). From Ohama(1981), quoted by Justnes(1994).

2.2.1.2.7 Hydrated Lime

Early researchers in abrasion resistance found that the addition of lime yielded satisfactory results:

Abrams(1921) did abrasion tests using a Talbot-Jones Rattler [= 95mm *impacting steel balls,* see appendix U.2.01] and found that for up to 20% addition of hydrated lime ‘and other powdered admixtures’ the wear of concrete was not sensibly increased.

Jackson(1924) did abrasion tests using *rolling chained tyres* (see appendix 3.02) and found that a 12% substitution of cement (by mass) with hydrated lime made little difference to the abrasion resistance.

These findings have limited application today given the widespread availability and low cost of extenders which are by-products from other processes, such as fly ash and slag. Furthermore these materials respectively have beneficial pozzolanic and hydraulic properties, whereas the hydraulic properties of hydrated lime are minimal in comparison and are very slow.

Sometimes milled limestone is used as a filler in cement. Although it has no hydraulic properties, it may be cost effective relative to other extenders, and also serve to give the desired strength versus time characteristic.

2.2.1.2.8 Calcium Aluminate Cement

Scrivener(1999) mentions in a review of calcium aluminate cement that it has three outstanding uses:

- Excellent resistance to acids
- High temperature performance
- Resistance to abrasion.

Scrivener explains that even at equivalent compressive strength, calcium aluminate cement concretes show better resistance to abrasion than do Portland cement concretes, and are particularly favoured in hydraulic situations where there is fast flowing water. The full benefit of this resistance to abrasion is realised in concrete made with synthetic aluminate aggregate (alag). In this case there is a slight reaction of the surface of the aggregate resulting in a dense interfacial transition zone (ITZ). The ITZ of conventional and even silica fume concretes is vulnerable to erosion and the aggregate particles can be torn away, leading to more rapid wear. The similarity between paste and aggregate in Alag concrete leads to even wear and longer service life.

Subramanian(1999) states that aluminate cements having a low calcium content, together with hard aggregates like corundum, are used in applications subjected to high abrasion.

2.2.1.2.9 Summary and Conclusion on ‘Binder Type’

In this section the influence of a number of cement extenders and cement types on abrasion resistance were considered. These included fly ash, ground granulated blast furnace slag, silica fume, polymers and hydrated lime. The type of cement used clearly will also affects abrasion resistance, e.g. OPC, RHPC, CAC. Clearly the influence of these materials is limited to modifying the properties of the paste, and therefore the influence that the paste has on abrasion resistance. (This will include its ability to bond with aggregate, as well as the actual hardness of the paste). Furthermore, results achieved are dependant on the type of abrasion test used, where some are more appropriate in measuring the contribution of the paste than others. Notwithstanding, the various results have shown that binder type has a significant influence on abrasion resistance.

2.2.1.3 Curing

It is well established that proper curing has a significant effect on the strength of concrete, as hydration can only proceed in the presence of water, or water vapour with a relative humidity exceeding 90%. Therefore premature surface drying leads to a termination of the hydration of the cement in that zone that arrests further gain in abrasion resistance.

The factors that are known to impact negatively on the rate at which a newly cast concrete floor will lose its free water through evaporation, leading to plastic shrinkage and cracking, are low relative humidity, high wind speed, high ambient temperature and hot fresh concrete (resulting from sun warmed aggregates, water etc).

Curing at increased temperatures accelerates the rate of hydration, but results in a coarser gel/pore structure, with a corresponding reduction in potential strength.

It is generally left in the hands of the main contractor to attend to curing. With all the other operations he needs to attend to, the contractor sometimes does not give curing sufficient attention, or even overlooks it altogether. The consequences of this in the case of industrial concrete floors can be very serious. **Chaplin(1991)** has proposed that curing should be priced as a separate item on the tender documents, to be done by specialist and independent sub-contractors.

Importance of curing

Curing affects abrasion resistance far more than compressive strength. An uncured surface will dry out quickly, bringing hydration to a halt, while drying out of the bulk concrete takes much longer.

Beningfield(1995) did abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06] and found that uncured mortar had 250% greater abrasion wear compared to the equivalent cured mortar.

Shallow Curing

Schuman(1939) did abrasion tests using *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] found that abrasion resistance increased appreciably for specimens that were cured at a RH of 95% relative to ‘air curing’ (RH unspecified). The surface zone was far more sensitive to curing than deeper in the specimen, where the absence of curing hardly affected abrasion resistance. Starting the curing process as soon as possible was very important.

Deep Curing

Abrams(1921) did abrasion tests using a Talbot-Jones Rattler [= 95mm *impacting steel balls,* see appendix U.2.01] and showed that 10 days of curing reduced wear from 36mm to 20mm. [These specimens had 28-day crushing strengths below 30MPa and were therefore sufficiently permeable for water to penetrate the pore structure, thus making a difference to abrasion resistance to some depth].

2.2.1.3.1 Curing Systems

Cement grains develop into gel, comprising strong needle like structures of calcium silicate hydrates (CSH), as long as water is present. If the water carrying capillaries around the cement particles dry out, hydration ceases. The various methods of curing described hereafter are therefore aimed at preventing water from escaping from the system.

(a) **Polythene sheeting**

According to this system a thin plastic sheeting is placed over the surface as soon as finishing (floating, trowelling) is complete.

Some producers in South Africa use a shrink-on-with-heat plastic shroud to create a sealed cocoon-effect around the pack of newly stacked pavers (24hours old), while others merely strap the blocks together with steel straps.

Sukandar(1993) did abrasion tests according to ASTM C779 Proc C [=*rolling steel balls*, see appendix U.2.12] on concrete pavers with a/c ratios varying from 3 (25% cement) to 9 (10% cement). Abrasion testing on pavers revealed that at age 24 hours sealed pavers had 75,5% more abrasion resistance than air-cured specimens, while 'moist-cured' specimens had 23,5% more.

[The fact that the air cured specimens had the lowest abrasion resistance comes as no surprise, and is no doubt the result of inhibiting the process of hydration by allowing drying out. What is perhaps more surprising is the superiority of the sealed specimens relative to the moist-cured specimens. In this regard an examination of the methods used for these two curing regimes is revealing. The specimens with the higher abrasion resistance were 'wrapped' in black plastic polyethylene, then stored at 73 deg C (in a temperature controlled room). This means that the heat of hydration would have increased the temperature even further, particularly as the many samples were stored closely together, on the production pallets in their configurations, and thereafter stacked three pallets high. The elevated temperatures would have accelerated the development of strength and abrasion resistance.

On the other hand the 'moist-cured' specimens were probably left exposed to the atmosphere for a few hours (unless they were temporarily covered in polythene, it is not clear) and then 'frequently sprayed' with tap water to ensure that their surfaces were kept wet. This method has some potential problems. The spraying with water would have cooled off the specimens, particularly if this took place on a cold-day. Next the subsequent partial evaporation of this water would also have had a cooling effect, exacerbated by possible cold drafts or winds. Furthermore some surface drying would have taken place in the delay period prior to commencing the spraying. On the other hand if this delay period was too short, so that spraying commenced before the blocks had achieved final set, the w/c ratio in the upper surface would have been negatively affected. Thus these cooling effects coupled with some possible drying out effects would have retarded strength development, particularly at the surface. However, the most likely reason for the superior 24-hour performance of the sealed specimens is that their surface capillaries would no longer be saturated, with ongoing hydration processes drawing water from these capillaries. Therefore, relative to the sprayed blocks, they would in a relatively dry state at the time of the 24-hour abrasion test, and it is well known that testing in a dry state yields significantly higher results].

Papenfus(1992) In South Africa pavers are typically made on wooden pallets which are sometimes stacked one on top of another in rows, in the open. It is important that these rows are covered with plastic tarpaulins as soon as the row is full to promote good curing. Failure to do so has resulted in problems leading to high abrasion wear. (The first 24 hours of curing is always the most critical!)

On the other hand this system, if correctly applied can be very advantageous and energy efficient. Temperatures under the covers will generally be elevated, thus promoting accelerated strength development as well as good curing. (The writer has personally measured temperatures as high as 50 deg C).

(b) **Wet burlap**

Burlap mats are laid over the finished surface and kept wet. This system does provide a high relative humidity environment, but care must be taken to delay the wetting of the mats until the concrete has set sufficiently, and thereafter keep the mats continuously wet. Furthermore evaporation can lead to cooling of the surface, retarding strength development slightly. These are possible reasons why this curing system resulted in a lower abrasion resistance relative to polythene sheeting in the work done by Sukandar(1993).

(c) **Ponding**

Ponding is done by building small grout-walls around the periphery of the floor followed by flooding. The major advantage of this method is that it feeds the surface with as much water as it requires for ongoing hydration. Self-desiccation is thus avoided and the density of the surface microstructure is maximized.

Alternatively, relatively small test specimens can be directly submerged in water, and this practice is usual on construction sites and concrete labs.

Dhir(1991a) did abrasion tests using the C&CA machine [=*rolling steel wheels,* see appendix U.4.06] and found that the abrasion resistance of all water immersed specimens was considerably greater than the corresponding air cured samples.

Nanni(1989) did abrasion tests using ASTM C779-82 Proc C [=*rolling steel balls*, see appendix U.2.12] and found that concrete that had been water cured for one day had 35% of the 28-day abrasion resistance of equivalent specimens which had been cured for 25 days. In the case of compressive strength the one day specimen was 70% of its 25 day companion. At five days of curing these ratios had respectively increased to 62% and 90%. The importance of prolonged curing if concrete is to achieve its potential abrasion resistance is thus clearly demonstrated.

(d) Curing compounds

The most dramatic reduction in abrasion wear occurs on surfaces treated with 90% efficient resin curing compounds. Although clearly these materials produce high efficiency curing, part of the effect is due to the impregnation of the surface pores by the resin which supports and reinforces the paste's microstructure at the surface zone, preventing the crushing breakdown which is the typical mode of wear for floors (Chaplin:1987).

Resin curing compounds should not be used where surface treatments are to be applied later on. In this case polythene sheeting should be used.

Munn(1995) reported on concrete floors at two construction sites in Australia that employed chlorinated rubber curing compounds, resulting in very good abrasion resistance.

This has the double benefit of accelerating strength development at the surface as well as inhibiting drying out.

Samson:2000 Various types of curing compounds are available commercially in South Africa. Examples include 'Cureseal F' a chlorinated rubber compound and sealer, 'Resincure H' a resin based compound, 'Polycure' an acrylic emulsion, 'Curing concentrate' a wax emulsion (see appendix S).

(e) Air

This method of curing really amounts to no curing at all, and seriously retards the potential abrasion resistance.

a'Court(1954), did tests with a reciprocating steel pan [=*sliding fine-abrasive*, see appendix U.5.17] and found that the difference between equivalent air cured and water cured specimens was confined to a very thin skin at the top. His data indicates that after an average wear depth of approximately 0,4mm, air cured samples wear at the same rate as water cured samples. [An examination of a'Court's photographs makes it clear that there is a significant proportion of 'gravel' and 'granite' coarse aggregate that is fully exposed. There can be little doubt that it was this aggregate that governed the rate of abrasion once the thin skin of paste/fine aggregate had been removed].

Humpola(1996b) did abrasion testing using *impacting steel balls* (see appendix 2.03) and found a 26% greater loss for air cured specimens relative to specimens that had received 15 hours of mist curing during the first 15 hours***.***

Meyer(1980) recommended that in ' summer the moistening of the stones (concrete pavers) should not be dispensed with if sanding of the surface is to be avoided. The concrete, with a low w/c ratio, used for concrete paving stones, is sensitive to premature drying ' [but far less sensitive than a concrete with a high w/c].

Shackel(1993a) concludes from a number of investigations that the abrasion resistance of moist cured paving is superior to that of air cured paving.

However, using *impacting steel balls* (see appendix 2.03) he found that air cured specimens that were about 100 days old in fact had a higher abrasion resistance than moist cured specimens. This however is likely to be the effect of carbonation, which would have occurred only in the air cured specimen.

(f) **Heat and steam**

Precast products such as concrete pavers are sometimes cured with heat to accelerate their strength development. This may be in the form of an open steam system which supplies moisture to the product, enhancing hydration by preventing drying out.

Accelerating the strength development by supplying heat means a guaranteed minimum abrasion resistance can be achieved, but the products long term potential is sacrificed to a degree, reducing strength by as much as 20% according to **Dowson:1980**. [A coarser pore and gel structure results from accelerated curing].

'Dry' heat should never be used with paving, where clearly abrasion resistance is an important characteristic.

Alexanderson(1973) : 'When the temperature of concrete is raised immediately after it has been placed, the expansion is much greater than for set or hardened concrete. The reason appears to be that in the first few hours concrete is too weak to resist the pressure set up in the pores by the rising temperature. In horizontal directions there is some restraint to this expansion, but usually the top surface of the concrete is exposed and so there is little restraint to vertical expansion. After the concrete cools, that portion of the early expansion caused by the expansion of the pores is retained and becomes permanent, giving a weaker and more porous product than one cured at normal temperature. The higher the temperature at the time the concrete sets, the more adverse the effect of permanent expansion. If the temperature is raised to the boiling point of water a further large increase in pressure occurs, with a consequent increase in volume if the concrete is not strong enough to resist it'.

The concept of a surface made porous by heat induced expansion as described above clearly has implications for concrete pavers and will reduce abrasion resistance.

Verbeek(1972) : This research shows a tendency towards improving compressive strength for decreasing curing temperature, till 60^{o}C. Below this temperatures threshold, loss in strength is relatively insignificant. The trend for abrasion resistance is like to be the same.

Dreijer(1980) was involved with 'very extensive research' from 1970 through 1978 and found that steam curing could have the effect of 'burning'.

[It is not certain what Dreijer means by 'burning', nor does he say how early the steam was applied.]

Fulton(1986) refers to Alexanderson(1973) to explain the detrimental effect of excessive heating prior to concrete setting, and refers to research by Verbeek and Copeland(1972) to show the adverse effect of over-heating concrete after it has hardened (after final set).

Humpola(1996b) did abrasion testing using *impacting steel balls* (see appendix 2.03) and found a 26% greater loss for specimens that had received 15 hours of steam curing (peaking at 85 degree C) during the first 15 hours relative to specimens that were subject to mist curing for the first 15 hours.

Sectional conclusion: From the above it is evident that overheating concrete, even if the RH remains at 100%, results in loss of strength, and this loss is greater for higher temperatures. In the case of very early high heat, the density of the surface zone is substantially reduced, leading to significant reductions in abrasion resistance].

2.2.1.3.1.1 Comparison and relative performance of different curing systems

Kettle(1987a) studied different curing systems including wet burlap/hessian polythene sheeting, 75% efficient resin, 90% efficient resin, a wax based curing compound, and direct exposure to air. He found that the wax and 90% curing compounds gave the best abrasion resistance (*rolling steel wheels*, see appendix U.4.06), followed closely by the 75% resin. Wet burlap treatment was 4th, polythene sheeting 5th, and as may be expected air curing rendered the lowest abrasion resistance. His results show that depending on the curing regime and finishing technique employed, cured surfaces have wear ranging between 15% and 50% of uncured surfaces.

Kettle(1984): Wet burlap may result in increased or reduced abrasion resistance relative to plastic sheeting depending on whether or not it is kept in a moist condition. Replenishing the moisture on a continuous basis limits the extent of self-desiccation in binder rich surfaces and the additional hydration promotes increased density of the microstructure.

Fentress(1973)*:* found that the best abrasion resistance results were obtained from applying a curing compound immediately after final trowelling. Second best was moist burlap curing, third was no curing, while the worst was a delayed application of the curing compound.

Dihr(1991a) did abrasion tests with (*rolling steel wheels*, see appendix U.4.06) and found that relative to air curing, membrane and hessian curing halves abrasion wear.

TR34(1994) states that curing makes a dramatic difference to the abrasion resistance of a concrete floor. While polythene sheeting curing is regarded as acceptable, it is recognised that very high abrasion resistance can be achieved by spray applied resin compounds that penetrate the surface pore structure.

In the manufacture of concrete paving a number of methods can be used to achieve good curing. A resin spray can be applied to the surface of the units as they emerge from the machine, or before they are stacked into packs. A plastic shroud can be used to seal the blocks in a water-tight compartment during the packing process. Overhead water sprays can also be deployed.

It is however unfortunate, that these measures are mostly neglected in favour of saving a few cents per square meter. A common practice is to bind the blocks together with steel strapping thus forming a pack suitable for transportation at a later date. However the outer blocks in such packs are likely to dry out and good curing will not be achieved in these blocks.

2.2.1.3.1.2 Unexpected results

Dhir(1991a) and **Chaplin(1991)** did abrasion tests using rolling steel wheels (see appendix U4.06), and report some surprising results on fly ash concretes, showing lower abrasion resistance at 90 days compared to 28 days, but offer no explanation. In both cases the pavers were cured at 100% RD.

Similarly **Naik(1995)** did abrasion tests according to a modified version of ASTM C944 [=*sliding fine abrasive beneath rolling dressing wheels* similar in principle to appendix U.3.08]. He studied fly ash replacement and found no improvement at 91 and 365 days relative to 28 days for up to 30% replacement. Specimens were submersed in water for the full duration.

2.2.1.3.1.3 Curing of low w/b mixes relative to high w/b mixes

Curing is not so important where the surface concrete has a low w/b. In low w/b mixes evaporable water is soon bound up into calcium silicate hydrates. Therefore if evaporation can be kept at bay for a few days, for example by resin or polythene sheeting, virtually all free water will be hydrated, resulting in no possible further loss.

Furthermore in very low w/b mixes, there is limited space for the development of gel, with no further hydration possible once this space has been filled. Investigators concur that ongoing curing in low w/b mixes yields minimal improvement in abrasion resistance. (However, Addis(1991) has reported some ongoing hydration in ‘over-cemented’ mixes after many months of soaking, albeit slowly).

Conversely curing makes a substantial improvement in concrete surfaces that have a high w/b. The open pore structure caused by relatively large distances between the individual cement grains allows full conversion of the cement into gel, but only if these spaces remain water filled.

Following are the findings of investigators that confirm this:

Chisholm(1994) showed that curing was the second most important factor in controlling abrasion wear, second only to the finishing technique. Curing resulted in greater wear reductions than variations in w/b, dry shake applications or liquid treatments. However whereas curing of high w/b mixes was vital, low w/b ratios were not nearly as affected by poor curing.

Dihr(1991a) did abrasion tests with rolling steel wheels (see appendix U4.06) and found that curing was as significant as w/b, and was very necessary at higher w/b.

Kettle(1986) found that curing was important in controlling abrasion resistance, and of prime importance with the higher w/c ratio mixes.

Ghafoori(1992) reported on abrasion tests on concrete pavers according to ASTM C779 Proc C [=*rolling steel balls,* see appendix U.2.12] and found that the improvement in abrasion resistance was very noticeable for cement contents of 10% 11,1% and 12,5%.

From 14,3 % onwards the benefits of curing decreased, and at a cement content of 20% the benefits of curing were minimal.

Sawyer (1957) did abrasion tests according to DIN 51951 [*rolling steel balls*, see appendix U.2.09] and found that abrasion resistance increased with good curing. Curing benefited lean mixes substantially more than rich mixes. The depth of wear in a lean mix (267 kg/m^3 = 4,5 sacks/yrd^3) reduced from 7,5mm to 4mm as duration of moist curing increased from 3 days to 28 days. On the other hand the depth of wear in a rich mix (444 kg/m^3 = 7,5 sacks/yrd^3) only reduced from 3,0 mm to 2,1 mm.

Paving: The same rule applies as for conventional in-situ cast floors. Paving made with w/b below 0,4 (i.e. richer mixes and good quality sand) will be less affected by poor curing, whereas paving made with higher w/b ratios (i.e. leaner mixes and thirstier sand) will be severely affected.

Sectional Conclusion:

Many curing systems are used to cure concrete, some more effective than others. Concretes with high w/b ratios are very dependent on good curing if they are to reach their potential abrasion resistance, whereas concretes with low w/b ratios are much less affected.

2.2.1.3.2 Binder Content

The concept of either ‘over-cemented’ or ‘under-cemented’ concrete has already been discussed. Over-cemented mixes, given favourable curing conditions, soon hydrate to the point where no further space is available for further gel development. On the other hand, given poor curing conditions, over-cemented mixes will self-desiccate, whereby the process of hydration causes the relative humidity within the pores to drop to a level where no further hydration can continue. In substantially over-cemented mixes, most of the free water in the pores hydrates before it can evaporate, thus allowing a reasonable level of hydration, and it is this phenomenon that makes binder-rich mixes less vulnerable to poor curing.

In such instances hydration will continue if an external source of water is applied at some future date, but the concrete will never achieve its full potential (i.e. in terms of strength, abrasion resistance, impermeability etc.). Some examples are given of this further on under the heading, 2.2.1.3.4.

2.2.1.3.3 Binder Type

The additional fineness of *RHC* relative to *OPC* means that for a given curing regime, the former will hydrate at a greater rate and to a greater extent and therefore develop more strength. Given the sensitivity of abrasion resistance to curing this is a factor to be considered when selecting cement type.

By the same token, additives that initially retard strength development (e.g. ggbs) are likely to yield reduced abrasion resistance, given limited curing. Conversely, pozzolanically active additives such as *fly ash* may improve abrasion resistance, given prolonged curing, even if this is in the form of natural curing after installation.

Following are two examples of how the curing regime impacts on abrasion resistance where different binder types are used:

Grieve(1993) did abrasion tests according to PCI.TM.7.11 [=*sliding wire bristles* , appendix U.6.02] on 28-day old cubes, and showed that the abrasion resistance of air cured concrete in an outside environment at ambient temperatures was dependant on the type of binder used. The OPC mix had the least wear, 15% replacement with fly ash was 2nd, 30%

replacement with fly ash was 3rd, while 50% replacement with slag was worst affected by the air curing. [Given continuous moist curing to 28 days, experience has shown that these mixes would have achieved very similar results].

Papenfus(1995) observed that while the abrasion resistance of fly ash mixes was retarded at 28 days, they had significantly improved abrasion resistance at 7 years relative to the 50:50 OPC:MGBS control. Conversely silica fume mixes had superior 28-day abrasion resistance, but only achieve parity with the control after 7 years.

These findings agree with the widely reported long term beneficial effects of pozzalanic activity associated with fly ash, after a slow start up, even under 'natural' curing. Conversely, the rapid pre 28-day strength acceleration effect of silica fume is now known to be followed by minimal post 28-day strength development.

It may therefore be stated that whereas pre 28-day curing is always important, fly ash mixes benefit the most from long term curing.

2.2.1.3.4 Age

In this section, the relationship between curing and age will be considered. Three time related effects will be considered; i.e. the effect of the duration of curing, the effect of early curing, the effect of late curing.

(a) Duration of curing

It is generally assumed that the compressive strength and abrasion resistance of concrete are a function of their age, for any given curing regime. This hypothesis is tested below in the light of the available literature on abrasion resistance, and the various points of view are grouped together under three headings:

(i) **Abrasion resistance is not related to curing duration**

Sukandar(1993) did abrasion tests according to ASTM C779 Proc C [=*rolling steel balls*, see appendix U.2.12] on concrete pavers with a/c ratios varying from 3 (25% cement) to 9 (10% cement). Compression testing on sealed specimens revealed that one-day strengths were generally slightly superior to 3 day and 7 day results.

Similarly abrasion testing on companion blocks using the same sealed curing conditions indicated that curing age had no significant effect on abrasion resistance.

A possible explanation for this unexpected trend is that all available moisture in the pavers was used up in the hydration process before the blocks had reached the age of one day old. Considering the limited free water available within the semi-dry mix, given the very low w/c ratios of 0,21 through 0,34, this may in fact be the case. (Note that corresponding 1day compressive strengths ranged between 83 MPa through 34 MPa).

(ii) **Abrasion resistance improves with increased curing duration**

Kellerman(1994) cautions that the period of curing cannot be simply prescribed; curing of concrete should be continued until the properties of a particular concrete are developed to the required extent. For normal conditions (temperatures around 20 degrees, a RH around 65% and low wind speeds), a minimum moist curing period of 5 days has been suggested.

Figure 2.8, from Price(1951), illustrates the affect on compressive strength of terminating curing prematurely. Given that abrasion resistance is far more sensitive to curing than compressive strength, the trends shown below can be expected to be magnified, with the possible exception of concrete that is very dense, and that has a low w/b. (In this case self desiccation is likely to negatively affect both surface and core concrete).

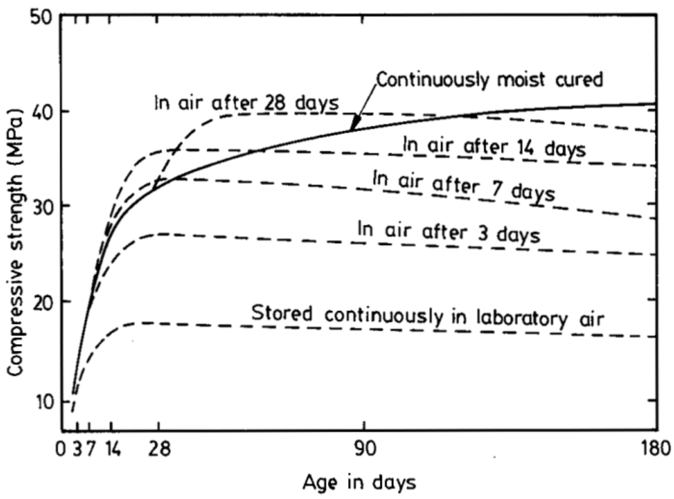

Figure 2.8 Compressive strength of concrete dried in laboratory air after preliminary moist curing [Kellerman(1994) published in Fulton's Concrete Technology 7th edition, pg 232, originally from Price(1951)]

Liu(1991) stated that curing should be done for a minimum of one week to positively impact on abrasion resistance.

Naik(1997) used the ASTM C994 *rolling dressing wheels* abrasion test [see appendix U.3.09] to assess the abrasion resistance of fly ash concretes with cement replacement levels of 40%, 50% and 60%. He found that generally fly ash mixes improved at a greater rate than did the equivalent 0% control mix (41Mpa at 28d), considering 28, 91, and 365 day results. The point here is that all concretes improved with age, given good curing (ASTM C192).

Rourke(1986) reported on abrasion testing using MA20 [=*rolling steel balls*, see appendix 2.13] and found some dramatic reductions in the penetration of the balls with increase in age. Penetrations of 1.8mm, 1.15mm, 0.5mm, 0.26mm, 0.19mm are shown for corresponding ages 1 day, 4 days, 11 days, 18 days, 32 days. (It is not known how the blocks were cured).

Goncalves(1998) used PrEN 1338 [=*impacting fine-abrasive*, appendix 5.12] to determine the influence of 0%, 30% and 50% fly ash replacement on the abrasion resistance of concrete. Generally, continuous immersion curing increased the abrasion resistance by 17% from 28 days to 180 days.

Ghafoori (1999) did abrasion tests using ASTM C779 Proc C [=*rolling steel balls,* see appendix U.2.12] to determine the effect of replacing a proportion of the fine aggregate with silica fume. Four levels of fine-aggregate replacement were considered i.e. 5%, 10%, 15% and 20%. A curing (by immersion in water) period of three days resulted in negligible improvement to abrasion resistance relative to the 0% (silica fume) concrete, while that at seven days was slight. However, after 28 days curing, a substantial increase of 24% in abrasion resistance was noted for the 10% fine-aggregate replacement level, improving only slightly more by 91 days.

Naik(1995) did abrasion tests using a modified version of ASTM C944 [=*sliding fine abrasive* beneath rolling dressing wheels, see appendix U.3.09] and found that abrasion resistance increased progressively with age, at 1 day, 3 days, 7 days, 28 days, 91 days, although at 365 days there was generally a drop in abrasion resistance relative to the 91 day results. This drop off applied to replacement levels of 70%, 50%, 40%, 15%, while the 30% and 0% fly ash replacement levels showed the more expected trend of increase with age at 365 days .

(iii) **Abrasion resistance reduces with increased curing duration**

Dhir(1991a): found that the abrasion resistance (C&CA rolling steel wheels) of specimens cured in water for 28 days was noticeably superior to equivalent specimens cured for 90 days. This unexpected result was consistent for mixes with 100% Portland cement as well as mixes with 15% and 30% fly ash substitutions. Prior to testing the blocks were conditioned in air in the laboratory for 20 days, effectively ruling out pore pressure as a contributory factor to early failure in the case of the 90 day-in-water specimens. He offers no explanation for this unusual trend.

(b) Early curing is the most important

It is well known that early curing is important in the development of *compressive* strength as illustrated in figure 2.8. [This is particularly the case in weak to medium strength concretes that are porous enough to allow continued hydration in the core concrete. Conversely curing, and especially early curing, will only marginally increase the compressive strength of well compacted cement over-saturated concrete with a low w/b ratio, since in this case it will take a long time, possibly years, for water to reach the heart of the core concrete].

Abrasion resistance differs from compressive strength, in that curing will always be beneficial, particularly in regard to shallow abrasion.

Dhir(1991a) found that the first four days of curing were the most important for abrasion resistance, whereas the effects of curing between 7 and 28 days was relatively insignificant.

Dreijer(1980) was involved with 'very extensive research' from 1970 through 1978 using the NEN 7000 sandblast abrasion test [=*impacting fine-abrasive*, see appendix 5.19] and concluded that curing in the first 48 hours was one of the two crucial factors for concrete pavers to develop adequate abrasion and frost resistance. (The other factor he mentions is adequate plasticity in the mix, [which ensures a low voids content]).

(c) Delayed curing

Where good curing procedures have been neglected, the application of water at a later date will make some improvement to the abrasion resistance of the concrete. However the surface will have lost irrecoverably some of its potential abrasion resistance.

The mechanism whereby the original potential abrasion resistance is permanently impaired may be described as follows. A residue of $Ca(OH)_2$ crystals is deposited in the entrances of the capillaries as the pore water evaporates. In the presence of CO_2 from the atmosphere, the $Ca(OH)_2$ converts to $CaCO_3$, calcium carbonate, a stable and nearly insoluble compound, resulting in a substantial sealing off of the capillaries. This makes it very difficult to get water back into the concrete to replace the evaporated water, so that hydration can continue.

Clearly the degree to which the pores are blocked off is a function of how old the concrete is. In the examples cited below, both Chaplin and Humpola found that delayed curing when the pavers were five and seven days old was still beneficial.

Humpola(1996b) did abrasion tests using *impacting steel balls* (appendix U.2.03) and showed that air cured pavers that were air cured for 28 days had 48% greater abrasion-wear relative to air cured pavers that were immersed in water after five days.

Chaplin(1990) presented results of test panels that were subjected to air curing for seven days before commencement of water curing. Generally they had lost about 50% of their potential abrasion resistance (C&CA *rolling steel wheels*) compared to equivalent specimens that had been water cured from day one, but abrasion wear was down to about a third relative to panels that were exclusively air cured.

Abrams(1921) pointed out that whereas concrete used on outside pavements would recover to a degree from moisture supplied by rain, this was not the case for floors in warehouses.

(d) Summary and Conclusion of ‘Age’

With one or two exceptions, the main body of evidence is that abrasion resistance improves with duration of curing, that curing is most important during the early life of the concrete, and that delayed curing will only recover *some* of the original potential abrasion resistance.

2.2.1.3.5 Conclusion of Curing

This section has considered various aspects of curing as they relate to abrasion resistance.

Starting with the various curing systems that were considered, in all of them the common thread is an attempt to either retain the existing water, or to make additional water available.

It has been observed that lean concrete is far more dependant on good curing than is binder rich concrete. Lean concrete especially benefits from ‘water addition’ curing.

All binder types are dependent on good curing. Concrete incorporating fly ash appears to gain in abrasion resistance/strength over many years even from natural curing.. However, some researchers come up with unexpected results that showed no strength gain for fly ash beyond 28 days, even after 91 days or 365 days of immersion in water. For silica fume good curing in the first 28 days is decisive, and not much development occurs thereafter. Finally it was seen that abrasion resistance improves with duration of curing, that curing is most important in the early life of the concrete, and that delayed curing is only partially effective.

2.2.2 Weathering and Corrosion

Weathering is defined here as a *gradual alteration* of a concrete's surface from non abrasive physical or chemical processes such as freeze/thaw exposure, early frost exposure, soft water dissolution, acid attack, carbonation, and wetting and drying cycles. Usually it results in a loss in integrity, so that any subsequent abrasion wear on the weathered surface will occur at an increased rate. In other words weathering is the result of exposure of a concrete surface to such natural elements as temperature, water vapour, carbon dioxide, or to man made substances such as acid.

In this section the various processes that result in weathering are considered, with due reference to the literature. The findings of various researchers are analysed to determine whether abrasion processes, if superimposed on such weathered surfaces, will accelerate or retard these processes.

2.2.2.1 Freeze/Thaw Damage

Both concrete and the aggregate used to manufacture the concrete is to a larger or lesser extent permeable by virtue of microscopic capillaries and pores. In moist and wet conditions these pores take in water. As temperatures drop below zero deg C, this water begins to freeze, and in so doing expands. Hydraulic and osmotic pressures develop in the pores of the cement paste and aggregate.

Hydraulic pressures occur when ice crystals initially form and expand in the pores and capillaries nearest the cold front, forcing the unfrozen pore water into the surrounding paste. If the distance to the nearest escape void is too great, the expelled water can cause increased dilation stresses within the capillaries.

Osmotic pressures are generated over time as unfrozen water from the smaller pores migrates to the larger cavities. Continued diffusion of water from ongoing crystal growth produce capillary expansion and increased pressure in the pore structure.

The application of calcium chloride and sodium chloride de-icing salts to remove snow and ice from concrete roads and bridges, tends to magnify these hydraulic and osmotic pressures. These pressures may result in rupturing within the gel/paste structure if the tensile strength of the gel/paste is exceeded. The now weakened paste structure is vulnerable to accelerated abrasion-wear and surface scaling. Freezing and thawing, aggravated by abrasive forces (the loose scaling becomes a grinding paste under the action of traffic), can lead to severe deterioration of a cementitious surface.

Several standardised procedures have been developed in order to assess the de-icer salt scaling resistance in concrete. These include ASTM C672, Swedish Standard SS 13 72 44 and Canadian Standard CAN3-A231.2-M85 (for concrete pavers).

Ghafoori(1997) tested concrete pavers using ASTM C672. At a *cement content* of 296 kg/m^3 (500 lb/yd^3) the pavers tended to be too porous, resulting in deep penetration of the de-icing salts, which led to the total disintegration of the test specimens after only 25 freeze/thaw cycles. However, with 356 kg/m^3 (600lb/yd^3) there was no sign of surface scaling or loss after 50 cycles.

[In a semi-dry mix, increasing the cement content has the effect of decreasing the w/c ratio, which amounts to a closer packing of the cement particles and therefore a denser gel structure. Furthermore the additional paste improves compactability thereby reducing air voids. The end result is a more refined cavity/capillary/pore structure, and this leads to a reduction in magnitude of the hydraulically induced splitting/rupturing forces within the microstructure of the paste. Furthermore, these forces are *more easily contained without* *damage owing to the* increase in the tensile strength of the gel structure for the richer mixes].

Tests also exist to test the likelihood of freeze thaw deterioration in aggregate, such as **ASTM C88 – 99a**. In this test the aggregate is repeatedly immersed in saturated solutions of sodium or magnesium sulphate followed by oven drying to partially or completely dehydrate the salt precipitated in permeable pore spaces. The internal expansive force, derived from the re-hydration of the salt upon re-immersion, simulates the expansion of water upon freezing.

Specifications for paved surfaces exist which go beyond considering only traffic-induced abrasion, and consider weathering effects as well, e.g. **ASTM C902-92** (Standard specification for Pedestrian and light traffic brick). This document classifies fired-clay paving brick according to two environments: (1) weather and (2) traffic. The main consideration in (1) is the possibility of freezing while saturated, while (2) considers the degree of abrasion due to different volumes of traffic.

Dreijer(1980) chaired a national committee in the Netherlands commissioned to investigate the problem of freeze/thaw damage and finally concluded: 'With the very extensive research between 1970 and 1978 co-related with field tests, it is possible to declare: 'If concrete paving blocks meet the requirements for loss of weight due to the blasting of 1000g of sand (with no more than 10 grams loss of material in the sand-blast test)…. it can be assumed with great certainty that these blocks are resistant to frost and de-icing salts' '.

[The limits of the ASTM C418 sandblast test are 15cm^3 in 8 holes of diameters 28,7mm using 8x600 grams of silica sand. It can be shown that the NEN 7000, 1980 revision requirement is 2,27 times less stringent than ASTM C418 (see 11.7.11 of volume 1 for the conversion between ASTM and NEN). In spite of this, from much observation and debate Dreijer was able to claim 'with great certainty' that this level of abrasion resistance ensures freeze thaw resistance. This would imply that ASTM C418 is rather conservative].

Dreijer(1980) reported on freeze/thaw testing (25 cycles) on specimens subjected to abrasion resisting testing using sliding wire bristles test (Amsler apparatus modified by fastening wire brushes to the steel bed, see appendix U.6.03). He found that the weight loss during the brushing tests on the frozen block surfaces hardly deviated from the losses of companion specimens subjected to the sliding wire bristles test that had not received the freezing cycles.

[It appears from this that deterioration resulting from freeze/thaw has a negligible effect on abrasion resistance for high strength concretes. That the Dutch pavers had high strength is confirmed by the average flexural strength of 6,0 MPa (61 kgf/cm^2) for the weakest batch tested].

Clark(1980) did freeze thaw experiments on concrete paving where the w/c ratios varied from 0.22 (for a moisture content of the mix of 5.2%) to a w/c as high as 0,62 (for a moisture content of 4,9%). Surface deterioration increased with increasing w/c. [It is evident from this that w/c is an important aspect of freeze thaw resistance, as it is known to be in abrasion resistance].

von Szadkowski(1987) explains that the combined effect of weathering (e.g. from freeze/thaw) and abrasion-wear results in a colour shift in the direction of the aggregate.

This comes about in the following way: In the process of manufacturing the paver, the mechanical vibration in the compaction operation results in a thin covering of paste on the aggregate particles that are immediately beneath the steel tamping shoes. Thus the surface of the paver appears to have a uniform colouring when new, but as weathering and abrasion take their course, and this thin skin of paste is weathered/abraded, the colour of the aggregate starts influencing the overall colour of the paving. This colour shift is therefore an indication of the degree of weathering/abrasion that has occurred. (It stands to reason that the degree of this colour shift is more or less noticeable depending on the colour contrast between the pigmented paste and that of the aggregate).

Conclusion to 'Freeze/Thaw Damage'

Freeze thaw deterioration is much reduced for pavers made from rich mixes, that inevitably have low w/c ratios. Pavers with good abrasion resistance (sandblast test) also have good freeze/thaw resistance, but the sliding wire bristles test is not sufficiently severe to detect potential freeze/thaw problems.

2.2.2.2 Early Frost Damage

The deleterious effects described above (from expansive pressures as freezing of the water in the pores/capillaries occurs) are much more of a problem for newly cast concrete floors, because the gel structure has not had time to gain strength. The volume of unhydrated water is also much more, with the corresponding potential for greater expansive forces relative to mature concrete. Therefore if good abrasion resistance is to be achieved, great care should be taken to prevent the fresh concrete from freezing when working in cold conditions (e.g. use thermal blankets).

2.2.2.3 Water Corrosion

The application of water to the surface of concrete is an effective means of curing, thus enhancing strength development and abrasion resistance.

However, usually over a period of some years, water can result in severe corrosion to concrete surfaces in certain situations.

Basson(1989) explains that the two properties of water that contribute most towards its high corrosive state are its extreme solvency and its ability to dissociate dissolved salts and enable them to participate in ion-exchange and ion-addition reactions.

Solvency: Calcium is leached out of concrete surfaces exposed to water until such time as it is saturated. The softer the water (i.e. less dissolved salts), the faster is the rate of leaching. Leaching is also accelerated when the water flows over the concrete. (Loss of calcium in stagnant water becomes diffusion dependant and proceeds much more slowly). Given that calcium is the most common element in concrete, leaching out effectively amounts to 'corrosion' of the surface, and in time can lead to progressive disintegration.

Ion-exchange reactions: Salts dissolved in water are almost invariably dissociated into their component ions, which are then free to participate independently of their original partners in various chemical reactions. Certain ions in the liquid phase will be exchanged for ones present in the solid phase, creating new and deleterious ingredients in the solid phase.

Ion-addition reactions: This occurs when ions in the liquid phase react directly with certain of the calcium compounds in the solid phase, creating modified compounds with increased molecular volumes, and this in turn sets up expansive stresses leading to cracking and spalling.

Ammonium ions, magnesium ions, and sulphate ions are amongst the most harmful. Chloride ions are capable of rapid penetration into concrete. On reaching the reinforcing they behave as a powerful catalyst in the corrosion of steel reinforcing. Since oxidised steel has over twice its original volume, the accompanying expansive forces lead to spalling of the cover concrete. Chloride ions however are no danger to concrete per se, and therefore are of no consequence in the case of paving.

Malhotra(2000) did abrasion tests according to ASTM C779 Proc C [= *rolling steel balls,* see appendix U.2.12] to determine the seven year abrasion resistance of concrete slabs in arctic tidal zones. He found greatly increased porosity for concrete near the surface for several of the panels at the tidal zone, and attributed this to chemical dissolution of concrete constituents together with their physical removal by ice abrasion and impact, and wave action. However at a depth of 5mm, the Ca(OH)2 was found to have been preserved, even in the concrete without any pozzolan or slag.

[This indicates that ion-exchange/addition reactions from the aggressive ions of the sea water had not caused much harm, and the leaching out of calcium was relatively superficial. This may be attributed to low permeabilities achieved by low w/c ratios (0,37 through 0,42), and possibly also to the very low prevailing ambient temperatures (mean annual temperature = -14 deg C). In support of this **Basson(1989)** states that generally chemical reaction rates double for every 10 degree temperature rise].

Summary and Conclusion

The two properties of water that contribute most towards its high corrosively are its extreme solvency and its ability to dissociate dissolved salts and enable them to participate in ion-exchange and ion-addition reactions. Concrete that has been thus corroded will have a reduced abrasion resistance.

2.2.2.4 Chemical Attack

The resistance of concrete to chemical attack is mainly determined by its permeability, its alkalinity and the chemical composition of the cement paste.

Chisholm(1997) made concrete specimens with 0%, 7% and 10% geosilica (a mined product, coarser than silica fume), and subjected them to various chemicals, to simulate typical industrial situations:

- 3% sulphuric acid sewers and geothermal environments
- 5% acetic acid food industry
- 15% ammonium sulphate fertilizer industry
- 5% lactic acid dairy industry
- 40% caustic soda industrial cleaning agent
- 2% nitric acid cleaning agent and fertilizer

After three months exposure the four acids had caused the greatest weight loss, with the 3% sulphuric acid causing the most and 5% acetic the least. Resistance to acid attack increased with increasing geosilica content. This is as a result of the reduction in concrete permeability and a reduction in the amount of calcium hydroxide in the concrete, partly owing to partial replacement of the cement with silica fume resulting in less Ca(OH)2, and partly because of some Ca(OH)2 is bound up in calcium silicate hydrate structures following pozzolanic activity.

Weight loss from ammonium sulphate and caustic soda was low for all mixes, with little improvement from geosilica addition.

Clearly the simultaneous application of abrasive effects would more easily remove that part of the surface that had been weakened by the attacking chemical.

In acidic waters or solutions more material can be dissolved relative to non-acidic water before saturation point is reached, if indeed it is reached. The rate of corrosion accelerates with increasing acidity. Acids react with the free lime (Ca(OH)2) in concrete, producing very soluble calcium salts. When these salts are leached the concrete porosity increases and the pH in the pores decreases. This pH shift destabilizes the calcium silicate hydrates, resulting in a significant loss of binding power. The hydrates begin to decompose, and the concrete disintegrates as the cement paste is consumed.

DeBielie(1997) considered the problem of acid attack on concrete in animal houses, where lactic and acetic acids are formed in meal – water mixtures, resulting in pH's as low as 3,8. (Although these acids are not as strong as others, they nevertheless react aggressively with concrete). On top of this the concrete is subject to abrasion wear in the form of cow hoofs, as well as the action of high velocity water from cleaning hoses. DeBielie subjected various concrete prisms to solutions of lactic and acetic acid, followed by mild abrasion in the form of brushing to remove loose material. He found that specimens incorporating fly ash had improved resistance to the acid solution, while silica fume mixes were better still. However the amount of material lost also depended on the pH of the solution. At a pH of 2.0, fly ash mixes had 20% less loss of material while silica fume mixes had 40% less. For a much less aggressive pH of 5,5 the mass loss was much lower, of the order of a sixth of that for a pH of 2,0, and in this case the addition of fly ash or silica fume seemed to make no improvement.

Clearly the abrasion resistance of concrete is seriously impaired by acid's weakening of the cement paste.

Fruchtbaum(1938) did abrasion tests using a steel wheel apparatus (see appendix U.4.05) on specimens that had been exposed to various acids (H_2SO_4, HNO_3, HCl) 'of the greatest concentrations commercially available'. He found that specimens made with OPC were superior to equivalent specimens made with high alumina cement, that the silica aggregate was unaffected by the acids, that HCl was by far the most severe, and that HNO_3 was more corrosive than H_2SO_4.

Paving: Gauteng's polluted atmosphere gives rise to acidic rain, and recorded pHs as low as 5 are not uncommon. It follows that concrete surfaces exposed to this rain will be attacked. The writer has observed that paving installed in places where it is exposed to rain but nevertheless inaccessible to traffic (e.g. on the verges of some buildings) is noticeably rougher than equivalent paving under a ledge where no rain can get to it (undercover-out-of-traffic pavers). The general appearance of these rain washed surfaces is not very unlike that of the adjacent areas subjected to abrasion and rain, in that both can be seen to have rougher surfaces relative to the undercover-out-of-traffic pavers. However, the distinctive feature of the trafficked blocks is that the exposed aggregate particles are relatively polished, while the untrafficked-rained-on pavers have aggregate particles with a rougher texture, and some paste adhesion.

Generally these effects start becoming noticeable (depending on paver quality, traffic intensities and climatic conditions) within five years.

Conclusion to 'Chemical Attack'

Certain chemicals have a deleterious effect on concrete, and acids are amongst the most severe. Partial cement replacement with fly ash or silica fume reduces the potential for corrosion. Clearly a paste weakened by extended exposure to harmful chemicals will abrade relatively easily. Furthermore, its ability to bond with the aggregate will be weakened, leading to accelerated abrasion wear in the concrete.

2.2.2.5 Carbonation

Basson(1994) explains that CO_2 from the atmosphere enters the pores and combines with $Ca(OH)_2$ and free CaO to form $CaCO_3$. The principle reaction proceeds as follows:

$CO_2 + Ca(OH)_2$ ► $CaCO_3 + H_2O$

The process of converting $Ca(OH)_2$ into $CaCO_3$ also results in a reduction in the alkalinity of the hardened cement paste. This is potentially dangerous in reinforced concrete, as a highly alkaline environment safeguards against corrosion of the reinforcement. Carbonation generally moves through concrete as a front and eventually reaches the reinforcing. Fortunately, in high quality, well compacted (i.e. dense and nearly impermeable) concrete the carbonation front moves so slowly that it is unlikely to ever reach the reinforcing, given the concrete cover. In this regard finishing techniques such as power trowelling, which greatly densify the surface concrete, also serve to resist the movement of the carbonation front.

The rate of carbonation is strongly affected by the relative humidity in the concrete structure. No carbonation takes place when the pores are completely dry, or when they are fully saturated. On the other hand the rate of carbonation is at a maximum when the relative humidity is between 40% and 60%, and also increases with increasing ambient temperature. Johannesburg's relative humidity meets this RH criteria most of the year, and certainly the daytime temperature of any outside paved surface will be substantially elevated from September through April.

As $CaCO_3$ is chemically stable and much harder than $Ca(OH)_2$, the net effect in concrete surfaces is a surface hardening. If the carbonation front has moved a few millimetres into the concrete this surface skin may greatly improve the abrasion resistance.

On the other hand, carbonation may close the surface pores off to such an extent that further curing from rain etc. is seriously inhibited. In this case the incompletely hydrated cement paste may never reach its potential strength or abrasion resistance, although evidence is presented below that suggests that the increased hardness from carbonation more than compensates for the loss in abrasion resistance from reduced ongoing long term curing.

Papenfus(1995) did splitting tensile strength tests on paving blocks that had been exposed to natural curing for seven years. Having split the blocks they were sprayed with phenolphthalene to measure the depth of carbonation. Blocks that were relatively porous had carbonated throughout, while very dense blocks had only been penetrated to a depth of a few millimetres. It may therefore be said that the carbonation front advances very slowly in well compacted dense concrete. The inability of the CO_2 to penetrate deep in well compacted concrete is an indication of pores that have become blocked by $CaCO_3$, and this probably inhibits the ingress of rain water as well, preventing continuing hydration deeper in.

Papenfus(1995) did abrasion tests according to MA20SA [=*rolling steel balls,* see appendix U.2.15] and found that the abrasion resistance of concrete pavers that had been exposed to the elements for seven years had increased by up to 400% relative to their 28-day values. Note that the seven year test was performed on the same block used for the 28-day test in each case. While some of this improvement in abrasion resistance may be attributed to long term pozzolanic effects from long term natural curing, assuming that the pores were not fully closed off by carbonation, the magnitude of the post 28-day improvement is considered far too much to be attributed to curing alone, and that carbonation is the major contributor to the improvement.

Kettle(1987b) reported on abrasion tests (C&CA *rolling steel wheels)* on both new and old industrial floor slabs. The results demonstrate that old slabs have a higher abrasion resistance, which Kettle ascribes to carbonation. [It is unlikely that the increase in abrasion resistance of *indoor* slabs for warehouses etc. are the result of ongoing hydration, since curing ceases below a RH of 85%. The improvement in abrasion resistance is therefore best explained by carbonation.

Dahir(1981) did abrasion tests using *rolling dressing wheels* (similar to appendix U.3.09) and reported an increased abrasion resistance of the order of a magnitude on pavements that had been in service for a few years relative to a 'new' pavement. It is likely that increased coarse aggregate exposure and increased curing of the older pavements contributed to this improved abrasion resistance. On the other hand carbonation may be the main reason for the improvement; certainly it is a contributor.

Agron(1998) did testing according to ASTM C779 Proc B [=*rolling dressing wheels* , see appendix U.3.06] which may be considered to be a severe test, given that it was performed on paving made from lightweight fine aggregate (SG of 1,2 and water absorption of 26%) of volcanic origin. Some pavers were exposed to 360 days of natural weathering at prevailing temperatures. In addition to the natural exposure of the sun and prevailing climate, the blocks were frequently wetted. Other pavers received accelerated curing over a period of 50 days consisting of 4 hours in water and 20 hours in a drying oven set at 50 degrees C. Some deterioration in both compressive strength and abrasion resistance was noted between 180 days and 365 days for the naturally cured blocks, while abrasion resistance increased throughout for the pavers subject to accelerated curing. Agron offers no explanation for the reduced abrasion resistance in the case of the naturally cured blocks.

[A possible explanation for these results may be that by 180 days the pores and capillaries were fully blocked by carbonation, such that no further curing could take place. In addition a measure of fatigue may have come about from continual thermally induced expansion / contraction movements in the aggregate. On the other hand the continued gain in abrasion resistance of the pavers subject to accelerated curing may be the result of carbonation and curing. At 50 days, carbonation may not have closed the pores off, and thus the pavers benefited from both carbonation and ongoing curing].

Siro(1991) did wear tests using revolving *rolling steel wheels* (see appendix U.4.01) under load. He found that wear resistance improved after four weeks 'more than the change in compression strength would lead one to believe. It is probably the effect of drying of the concrete and the carbonation of the surface'.

Shackel(1993b) did tests on cubes using the SSC test (*impacting steel balls*, forerunner of test in appendix 2.03), a sensitive test capable of discerning surface variations. He found that air cured specimens that were about 100 days old in fact had a higher abrasion resistance than companion moist cured cubes, and considered this to be the result of carbonation.

Conclusion to 'Carbonation'

There appears to be substantial agreement in the literature that carbonation improves abrasion resistance.

2.2.2.6 Wetting and Drying

The issue here is how frequently and how soon? If wetting is so frequent that the pores do not have time to dry out then the surface as well as the bulk is likely to benefit from ongoing *curing*, providing the relative humidity exceeds 85%.

If on the other hand the pores are able to dry out, then according to **Kellerman (1994)**, 'calcium hydroxide will be deposited in the entrances of the capillaries by the evaporating water. This calcium hydroxide is then *carbonated* by carbon dioxide in the air, sealing the capillaries, and making it very difficult to get water back into the concrete to replace the evaporated water. This reduces the degree to which cementitious materials can react. Curing by intermittent wetting has a similar effect'.

Although this carbonation is not beneficial for ongoing curing of the bulk concrete, it increases the hardness of the surface and thus improves abrasion resistance.

Fwa(1989) subjected cubes which had been though 150 wetting and drying cycles to the tumbling / rolling / sliding action of a LA machine (=*impacting steel drum*, see appendix U1.01). His results in effect show that weathering stops further strength development, or more correctly reveals that any strength development that would normally occur as a result of ongoing curing from the wetting cycles is negated by a weathering effect.

Conclusion to 'Wetting and Drying'

Frequent wetting and drying may promote carbonation if it leads to conditions where the relative humidity in the pores is approximately 50%. In relatively dense concrete carbonation may completely block off the surface pores, preventing any further long term curing from rain. However, carbonation often more than compensates for such diminished curing and results in improved abrasion resistance.

2.2.2.7 Influence of Abrasion Resistance on Weathering

A number of physical and chemical processes that cause weathering in concrete surfaces have been considered, such as freeze/thaw attack, acid attack, carbonation etc.

The opening statement of section 2.2 defined weathering as a 'gradual alteration' of a concrete's surface, that usually leads to a loss of surface integrity. The assumption was made that 'abrasion wear on a weathered surface usually occurs at an increased rate'. In the next few paragraphs the validity of this statement is considered. In other words the question is asked, 'Do abrasion processes, if superimposed on weathered surfaces, accelerate or retard weathering?' This question will be answered by considering the processes reviewed in 2.2.2.1 through 2.2.2.6, although in a different order.

(a) **Acid attack** results in a weakened and more porous paste structure. Nevertheless, there is a gradual increase in the pH within the pores of the weathered paste in the direction of the unweathered paste. In effect the weathered paste constitutes a buffer zone between the acid solution and the alkaline environment of the unweathered concrete. It has the same pH potential as the acid on one side, and similarly the same pH as the unweathered alkaline concrete on the other side. Therefore abrading away the weathered layer will have the effect of opening up new horizons of paste that have high concentrations of calcium hydroxide in their pores, and the high difference in pH potential between this alkaline solution and the acid solution will result in rapid dissolution, and hence rapid weathering.

In short, abrasion *accelerates* acid attack.

Similarly, acid attack is likely to *accelerate* abrasion. The weathered paste is itself easily removed by abrasive loads, and furthermore having lost much of its ability to bond with sand particles, releases the fine aggregate. Being a lot harder than the paste, even the unweathered paste, the released aggregate will act as an abrasive and cause gouging and localised crushing as traffic presses such particles into the surface.

(b) **Water,** it was explained, has the potential for leaching out calcium and promoting ion-exchanges and ion-additions, and these processes are invariably deleterious for the paste. The weathered layer, although it has lost some of its calcium etc., still serves as a buffer, preventing free access to the aggressive water. In removing this layer, abrasion has the effect of opening up a new front for the water, one that has high concentrations of the chemicals that the water seeks relative to the weathered layer that was removed by the abrasion process.

In short abrasion wear *accelerates* corrosion by water.

Similarly, corrosion (as in acid attack) is likely to *accelerate* abrasion. Firstly it weakens and softens the paste, as well as its bond capabilities with the aggregate. Secondly, the harder sand particles released in the disintegration of the paste become additional abrasive material and will therefore scratch/gouge/impact the surface accelerating the abrasion process.

(c) **Freeze/thaw** cycles, as with acid attack, weaken the integrity of the paste near the surface. Although this paste is weathered, it has the potential of retaining moisture, and in so doing maintain a moist environment at the interface of the unweathered paste. It has the potential of preventing the unweathered paste from drying out during the day and in so doing shedding some of its pore water which would have the effect of reducing freeze thaw damage. (Saturated pores give rise to hydraulic pressures during a freezing cycle that ruptures the gel structure). It may therefore be said that abrasion wear, in taking away the unweathered paste essentially removes a ‘wet blanket’, allowing the pores of the unweathered paste to dry out, thus reducing the potential for freeze/thaw damage.

In short, abrasion wear *retards* the rate of freeze/thaw weathering.

Conversely freeze/thaw weathering is likely to *accelerate* abrasion wear on two counts. Firstly the weathered paste, being partially ruptured by hydraulic effects, has lost some of its strength, and secondly the sand particles released in the disintegration of the paste are generally a lot harder than even the unweathered paste, and will therefore cause gouging and localised crushing when trafficked.

(d) **Carbonation**, as far as abrasion resistance is concerned, improves the hardness of the surface very significantly in the long term. It is however self regulating in dense surfaces, in that once the surface pores have been blocked by calcium carbonate, further carbonation will virtually stop. Therefore abrasion wear, in removing some of the paste, and hence some of these blocked pores, and opening up new pores which have not as yet been carbonated, promotes carbonation to start up again on a new front.

In short, abrasion wear *accelerates* carbonation.

Conversely, quite clearly, carbonation ‘weathering’ hardens the surface and thus *reduces* abrasion wear.

(e) **Wetting and Drying:** Generally this results in carbonation, and the effects are given above.

Alternatively if the wetting cycle is so dominant that the pores remain at a RH that is too high for carbonation, in this case the surface and bulk concrete benefit from ongoing curing. Furthermore abrasive actions will access more capillaries and pores and thereby promote further curing and hence lead to an increase in abrasion resistance of the newly exposed surface. On the other hand, if the water is substantially pure (corrosive), or acidic, then abrasion will lead to increased weathering, as discussed earlier.

In short, abrasion wear *accelerates* wetting and drying induced weathering, and here there are two possibilities. On the one hand new fronts are opened up for further curing and carbonation, but increased rates of corrosion will occur if the water is acidic or pure.

Finally, frequent wetting *accelerates* the development of abrasion resistance, providing that the water is not too soft or acidic.

2.2.2.8 Conclusion to ‘Weathering and Corrosion’

It is evident that while abrasion wear generally accelerates weathering, we have seen from the above discussions that this is not always the case. Likewise weathering normally accelerates abrasion wear, except for carbonation and possibly frequent wetting/drying.

2.3 Voids

2.3.1 Introduction

In section 2.2.1 it was shown that sub-microscopic spaces and channels, referred to as capillaries / pores exist within the structure of paste. With favourable curing these capillaries will remain filled with water, and gel will continue to develop into these spaces with a dramatic increase in strength according to the gel-space ratio rule [Powers(1958)].

On the other hand if the original water filled spaces in the paste are excessive, as with cement undersaturated mixes, the capillary-voids will be permanent, resulting in a loss in paste strength.

There are however also other cavities within the structure of concrete, ranging from microscopic to as much as 10mm in diameter, consisting of trapped *air* that was introduced during the mixing and placing operation. For a given workability, such voids will decrease with increasing compactive effort. Another form of void comes about from deliberate air entrainment, consisting of very small bubbles of air within the paste, introduced by the addition of specially formulated chemicals.

Many studies have shown that, regardless of the source of the voids, they have a detrimental effect on the strength of the concrete, and this has been confirmed in this research. Generally for every one percent of voids there is a five percent loss in strength/abrasion resistance.

Lane(1978) quoted Mielenz who stated that the abrasion resistance (and compressive strength) of concrete could better be stated in terms of voids/cement ratio rather than water/cement ratio.

Addis(1994) stated that air filled voids contribute to overall porosity and have a significant effect on strength. In fact, strength is affected equally by capillary porosity and air-void porosity.

2.3.2 An Expression for Strength Including Influence of Air Voids

Section 2.2.1.1 concentrated on the effect of w/c on the microstructure of paste, considering it a useful predictor of the strength and abrasion resistance of concrete and concrete pavers. However an important underlying assumption was that the concrete is fully compacted, and thus free of air voids. While this may be a reasonable assumption for most well compacted conventional concrete floors, it is not necessarily the case with concrete paving. For example Sukandar(1993) made pavers for abrasion testing where the aggregate:cement ratio (a/c) was varied between 3 and 9. The corresponding % voids resulting from entrapped air varied from 8,6% through 17,4%.

This evidence clearly suggests that entrapped air may contribute more to the eventual voids in the pavers than capillaries and cavities originating from the mix water. In fact, given that typical w/b ratios used in pavers are of the order of 0,36, so that these mixes are cement oversaturated, the products of hydration should, curing permitting, completely occupy the voids that were originally water filled.

Given the above considerations, it would be useful to find an expression that related strength not only to w/b, but also took account of air voids, be they from entrapped air, or entrained air.

Tam(1987) considered the influence of air voids on the compressive strength of cellular concretes of densities of 1300kg/m3, 1600kg/m3 and 1900kg/m3, as well as a 2250 kg/m3 mortar. Three models for predicting compressive strength from the air, water and cement absolute volumes were considered:

1. Abram's 1919 formula:

   Strength = $K1/(K2)^{w/c}$. K1 and K2 are empirical constants, and w and c represent the water and cement volumes. This formula does not consider air voids, and is therefore not a good model for cellular concrete, or even pavers that may have a relatively high voids content.

2. Feret's 1896 formula:

   Strength = $K[1/(1+w/c+air/c)]^n$, where 'air' = volume of air voids. The strength of concrete is thus seen to be a function of both the w/c ratio and the air/c ratio, decreasing as these ratios increase.

3. Power's modified gel/space ratio formula [**Powers(1958)**] may be stated as:

   Strength = $K[Ks/(1+w/\alpha.c+air/\alpha.c)]^n$, where Ks is a factor representing the increased volume of the hydrated gel relative to the volume of cement that has become hydrated, and α is the degree of hydration. When full hydration has taken place, i.e. when α = 1, then this expression is very similar to that of Feret's. Tam's experimental strength values were best predicted by this model.

Application of Ferret's formula to concrete pavers

In this section Feret's formula is tested against the experimental data of two investigators, Sukandar(1993) and Papenfus (volume 1). In both cases sufficient data is available to construct tables (i.e. Table 2.2 and Table 2.3) from which graphs are plotted showing straight line regression trend lines and R^2 correlation coefficients. It may be seen that the parameters K and n were obtained by trial and error until the least squares variance between Feret's compressive strength and the corresponding experimental result was minimised.

The respective R^2 coefficients of 0,9924 and 0,9864 indicate that Feret's formula can accurately predict compressive strength. This confirms the importance of considering the air/b ratio together with w/c in compressive strength determinations.

It is acknowledged that Power's formula would yield more accurate results, but the additional sophistication of determining the degree of hydration does not seem warranted in the light of the good correlation achieved with Feret's formula.

Table 2.2

Table 2.3

2.3.3 The Influence of Paste on Entrapped Voids

It is well known that for a given compactive effort, entrapped air voids in concrete decease with increasing workability. The workability of the mix is improved if aggregates are well graded with rounded particles, if the mix is superplasticized, if fly ash is used as a cement extender. These aspects are well documented in the literature and are considered under the appropriate headings elsewhere in this chapter. The focus here will be to consider the effect of the paste on the workability of the mix, and hence entrapped air voids.

In the mixer, a layer of paste is made to coat the various aggregate particles. During the compaction process the paste behaves as a lubricant in bringing the aggregate together. Two characteristics of the paste that have an important bearing on its lubricating ability are its consistency, and its proportioning relative to the other mix constituents. In this regard the findings of various investigators are now considered:

2.3.3.1 Effect of Consistency on Entrapped Voids

Just as the paste is the lubricating medium of the aggregate, so is water the lubricating medium within the paste. Therefore for a given quantity of paste, a high water content paste (implies a high w/c) has better lubricating properties resulting in fewer entrapped air voids.

The writer's experience of many years of producing concrete pavers has convinced him that <u>optimising the moisture</u> in the mix is the surest way of reducing voids and <u>improving abrasion resistance</u>. It should be noted that the optimum moisture content is close to the point at which the product begins to slump, and a recognised method of determining the optimum moisture content is to continue adding more and more water to the mix until it does slump, then cut back by a few litres!

Table 6.2 in this thesis shows that regardless of the binder quantity or type, '<u>wet' mixes have superior 28-day abrasion resistance relative to 'dry' mixes,</u> as determined by the abrasion tests used (sandblast, sliding wire bristles, rolling steel balls). This rule also applied to a range of binder combinations, incorporating various ratios of opc with mgbs, fly ash, and silica fume. Wear measurements taken on site after six years showed the same trend.

Komonen(1998) explained that concrete blocks are very sensitive to the optimum water content of the concrete mix. The mixture acquires a continuous cohesive structure that has sufficient strength to support its own weight upon being extruded seconds after vibration/compaction. If the water content is too low the block crumbles from lack of cohesion, while it slumps from too great a plasticity if too much water is present. The mixture therefore has to be both non-plastic and cohesive.

[While these statements may appear to be a statement of the obvious, what is not so apparent is that the limits corresponding to these extremes are very close together, with crumbling becoming a problem below a moisture content of 4%, and slumping generally occurring above 7%. Furthermore, between these limits, the degree of compaction that is possible from a given block machine is very sensitive to the moisture content, and <u>adequate compaction soon becomes very problematic for mixes that have less than the optimum moisture content</u>].

Dreijer(1980) reported that blocks that had been provided with a top layer of 'face concrete' suffered very badly in the very severe winter of 1962/63 experienced in the Netherlands. Later investigations using a sandblast abrasion apparatus revealed that production techniques in widespread use at that time resulted in this layer being substantially more porous [i.e. more entrapped air voids] than the base concrete. Consequently much care was placed on <u>maximising the plasticity of the mixes, in order to</u>

improve abrasion resistance. The measure taken had the effect of reducing the voids content in the face concrete. [Increased plasticity is generally achieved with more fines in the form of a higher cement content, and more water. It will be seen from the next section that increasing paste proportion also has the effect of reducing voids]. At any rate these measures proved effective as very few problems were experienced in the equally severe winter of 1978/79 in product supplied by producers who had implemented the recommendations of maximum plasticity (and 48 hours of curing).

Thus increasing water content is a very effective means of improving the lubricating powers of the paste. The corresponding increase in density (e.g. see figure 6.1 and 6.2 of volume 1) more than compensates for the increase in w/b.

A second means of improving compactability, which does not rely on increasing the w/b, is simply to increase the proportion of paste in the mix, and this is considered below.

2.3.3.2 Effect of Relative Proportioning of Paste on Entrapped Voids

Increasing binder content will improve the plasticity and hence compactability of the mix, and semi-dry mixes are no exception. The additional cementitious material also reduces the w/c ratio and increases the relative proportion of cementing material. This increasing binder content has a very beneficial effect on strength/abrasion resistance.

Sukandar(1993) did abrasion tests on concrete pavers according to ASTM C779 Proc C [=*rolling steel balls*, see appendix U.2.12] with a:c ratios varying from 3 to 9. Blocks made with the higher cement contents had substantially more abrasion resistance, with a 300% improvement in the 455 kg/m3 mix relative to the 153 kg/m3 mix. Sukandar rationalises that the ‘higher cement content produces a stronger bond between the fine particles of the top surface’.

An analysis of Sukandar’s results however (see appendix ‘R’), shows that the main reason for the improved strength is a reduction in % voids, rather than a lower w/c. A regression analysis shows that R^2 for compressive strength vs % voids is 0.98, while that for compressive strength vs w/c is only 0.82, confirming the stronger influence of % voids on strength.

In altering the aggregate/cement ratio from 9 to 3, the compressive strength increased from 40.4 Mpa to 79.3Mpa, the % void dropped from 17.3% to 8.6%, while the w/c ratio decreased from 0.34 to 0.21.

Although the w/c is substantially lower for the highest cement content, more importantly, the void content of this cement content is much lower relative to the mix made from the lowest cement content. He reports an improvement of 9,15% in unit weight for the highest cement content relative to the lowest.

Sukandar correctly states that high cement content ‘enhances workability and compactability of the matrix’, and the 9,15% difference in density between pavers made from highest and lowest cement contents proves this point. Increased density was obtained for the high cement contents even though they had the lowest w/c. An explanation must be found why a paste with a relatively high fluidity has less ability to facilitate compaction than a more viscous paste. It may therefore be postulated that the volume (i.e. relative proportion) of paste contributes more to the rheology of the mix than the w/c of the paste. Or put another way, *more of a viscous paste is better able to reduce voids than less of a more fluid paste*. Furthermore the greater quantity of the viscous paste is better able to ‘bed-in’ the aggregate particles with fewer voids. Although it is less fluid, the relative movements of adjacent cement particles can be less, as there are more of them at any given section, and *cumulatively their capacity for relative displacement is better*. Thus adequate flow is achieved by virtue of the greater volume of paste in the case of the low a/c mix.

This illustrates an important principle in the rheology of semi-dry concretes. An increase in aggregate proportion, which is equivalent to a decrease in paste content, reduces the compactability of the mix (even though it may have a higher w/c ratio), resulting in a reduction in density. In effect the air voids in the low paste content mix are not as easily expelled for the compactive effort applied, and the abrasion resistance decreases.

2.3.4 The Influence of Compactive Effort on Entrapped Voids

In the foregoing section it was shown that increasing the paste content reduces the entrapped voids and vice versa. Clearly, it is equally true that for a given paste with a particular lubricating capability, the proportion of air voids left in the finished product is a function of the compactive effort.

The findings of a number of investigators are now considered, for various means of compaction. The chief aim of vibration is to so agitate the concrete that particles move downwards in shearing motions relative to each other, moving into a closer packing, and in this process air voids escape to the surface, thus allowing the desired density to develop. The effect on the abrasion resistance is highlighted.

Chisholm(1994) showed that floor slabs made by means of fixed screed rails in conjunction with a vibrating beam have improved abrasion resistance relative to floors done with ‘free screeding’. In the former the nature of the mechanical equipment allows for a low slump / low w/b, while in the latter, the hand operated lightweight screed-bar requires a higher slump / relatively high w/b.

Nanni(1989) studied roller compacted concrete that had been compacted by means of two passes of a rubber tired roller, and three passes of a steel wheel roller, two of which were with vibration. In this process some of the paste and fine aggregate made its way to the surface. Thus under the action of the compaction and vibration the RCC was substantially densified. This resulted in a considerable improvement of surface abrasion (rolling steel balls test) relative to that lower down.

Kettle(1987a) did abrasion testing using an apparatus developed by the C&CA [=*rolling steel wheels*, see appendix U.4.06] and found that power finishing significantly increased the abrasion resistance of concrete slabs. This is mainly due to the surface compaction and to reduction of the w/b of the surface matrix. This is achieved by delayed and repeated vibratory disturbances by the power.

Paving Manufacturing Process: In the process of filling the mould with semi-dry concrete, between 10 and 25 percent air is introduced, depending on the degree of pre-vibration opted for. (Pre-vibration is the amount of vibration used whilst the feed-wagon is traversing to and fro over the mould. More pre-vibration means less air.) Once the feedwagon is in its final reversed position, this air is expelled by the joint action of pressure from an overhead tamper whilst vibration is introduced from below and sometimes even from the mould and the tamper as well, where these are also fitted with vibrators. (Since semi-dry concrete has a relatively high internal friction, compaction in concrete pavers requires high-energy vibration). Providing that the paste quantity and consistency is adequate to sufficiently lubricate the mix, most of the air is expelled in a matter of a few seconds by the action of the powerful vibrators.

2.3.5 Air Entrained Voids

It is sometimes desirable to entrain air into concrete. For example air entrained concrete is effective in resisting freeze thaw attack. The entrained bubbles also serve to improve the plasticity of the mix, reducing the harshness of a particular mix etc.

In section 2.3.2 it was shown that both Feret and Powers developed formuli that predict a reduction in strength for increased air voids. This applies equally to entrapped air and entrained air. Following are the findings of investigators that confirm this:

Connell(1985) did abrasion tests on air entrained concrete using an abrasion test described in BS 812:Part 3:1975 clause 9 [=*sliding fine-abrasive*, see appendix U.5.06]. He confirmed the well-established rule that a loss in compressive strength of 5% was observed for every 1% increase in air content.

When % air was plotted against abrasion mass loss, there appeared to be no increase in the rate of abrasion with increase in air content. However, when abrasion was calculated in terms of volume loss, <u>abrasion loss increased with % air entrainment.</u> [This may be explained by the presence of the aggregate, which is much harder than the paste, and compensates for the weaker abrasion resistance pastes as follows; the pastes with the higher percentages of air are indeed intrinsically weaker, but because they are also lighter, to register the same abrasion *mass* loss as stronger pastes deeper penetration is required, and in the process more of the highly abrasion resistant aggregate has to be abraded. This led him to observe that 'the entrainment of air appeared to have reduced the abrasion loss'].

Lane(1978) stated that <u>air entrainment</u>, by virtue of introducing <u>voids, reduces the abrasion resistance</u> of concrete.

Liu(1991) stated that increased air content resulted in reduced abrasion resistance (at constant w/b).

Naik(1995) did abrasion tests using a modified version of ASTM C944 [=*sliding fine abrasive* beneath rolling dressing wheels, see appendix U.3.09] and found that whereas increased air entrainment generally reduced compressive strength, for a given replacement level, its effect on abrasion resistance for the various mixes with fly ash replacement levels of 0%, 15%, 30%, 40%, 50%, 70% was 'insignificant'. [The writer believes that it should not be said that the higher air entrainment is not detrimental to abrasion resistance and vice verse. It should be noted that Naik achieved penetrations of between 1,5mm and 4mm. At these depths the cutter would be resisted to a degree by the 25mm coarse aggregate limestone particles, which are much harder than the air-entrained paste. Note that the tests were done on the bottom moulded sides, so that with some settlement during the making of the cube the coarse aggregate would be relatively close to the test surface and more concentrated. With such a large particle size even the highest air-entrained/weakest-paste would easily bond the course aggregate particles so that they could effectively resist the abrading cutter. The same argument applies to the 'gravel' fine aggregate, maximum size of 6,7mm, although here some of the smaller fine aggregate particles would have been dislodged, especially at a depth of 4mm.

Prior(1966) found that <u>up to 6% air entrainment did not seriously reduce abrasion resistance</u>, providing the mix was suitably reproportioned. [The experimental data that he uses to come to this conclusion was not presented. It is possible, as in the argument postulated above that the aggregate protected the relatively soft air entrained paste. It is not possible to come to any definite conclusions on his statement].

2.3.6 A Model Linking Abrasion Resistance to Voids

Thus far in volume 2, various influences that affect abrasion resistance have been considered. These include the quantity of water (see 2.2.1.1.1), the quality of binder (see 2.2.1.1.2), the w/b (see 2.2.1.1) and now the voids (see 2.3).

Arising out of these discussions a simple model is now proposed, showing the inter-relationships of these influences. Accordingly four expressions are given below that show how the quantity of water, w, and the quantity of binder, b, determine the voids content and w/b, and these in turn determine the abrasion resistance, Ia.

↑b + ↓w	=	↓↓entrapped voids	+	↓↓capillary voids	= ↑↑Ia	(2.3-1)
↑b + ↑w	=	↓↓entrapped voids	+	? capillary voids	= ? Ia	(2.3-2)
↓b + ↑w	=	? entrapped voids	+	↑ capillary voids	= ↓ Ia	(2.3-3)
↓b + ↓w	=	↑↑ entrapped voids	+	? capillary voids	= ↓↓Ia	(2.3-4)

Note that a single arrow signifies low or high, double arrow signifies very low or high, while a question mark indicates an uncertain or middle-of-the-road position.

Expression 2.3-1: A high binder content equates to a high paste content, and this improves the rheology of the mix, minimising ‘entrapped voids’. The favourable rheology also allows a reduction in the water content, which equates to a low w/b with a dense pore and fine capillary structure, i.e. reduced ‘capillary voids’. Therefore the combination of high binder content and low water content reduces both ‘capillary’ and ‘entrapped’ voids, resulting in high abrasion resistance.

Expression 2.3-2: Here some of the advantages of a high binder content are compromised by a high water content. Once again a high binder content equates to a high paste content, and together with a high water content this translates into a mix with excellent rheology, thus minimising entrapped voids. On the other hand the high water content also means a higher w/b that equates to a less dense pore structure and larger capillaries. The combination of minimal entrapped voids, and a compromised capillary structure, will result in a surface that is likely to have an intermediate abrasion resistance.

Expression 2.3-3: Lowering the binder content means a lower paste content, and this negatively affects the rheology of the mix. On the other hand increasing the water content impoves workability. The position relating to ‘entrapped voids’ is thus uncertain. What is certain is that the combination of high water + low binder leads to a most unsatisfactory w/b ratio characterised by high ‘capillary voids’, resulting in a low abrasion resistance.

Expression 2.3-4: The combination of too little water and too little binder is very detrimental as far as mix workability goes, resulting in a high proportion of ‘entrapped voids’. This will very significantly reduce abrasion resistance, even without any deficiencies in the morphology of the paste.

Note that for a given compactive effort, the ‘entrapped voids’ will be a function of w and b. As a general rule both b and w may need to be increased to ensure the condition ‘↓entrapped voids’, which may be regarded as the most critical requirement for increasing abrasion resistance. Alternatively, it may be possible to leave w and b unchanged and establish the condition ’↓entrapped voids’ by increasing the compactive effort.

2.3.7 Summary and Conclusion

There are three types of voids in concrete. Firstly there are microscopic capillaries and cavities that are especially prevalent in concrete made from high w/c ratios. Secondly there is entrapped air, the result of insufficient paste content, or too little mix water resulting in too viscous a paste and/or (too little of it), or inadequate compactive effort. Thirdly air may be entrained into the paste to serve a specific purpose.

Allowance is made in Feret's/Power's modified gel/space ratio formulae for the effect of voids on compressive strength. Consideration should be given to using this expression in concrete pavers that may have significant levels of either entrained or entrapped air.

A simple model is discussed that shows how abrasion resistance is related to the volume of 'entrapped' and 'capillary' voids, which in turn are governed by the binder and water content.

All voids in concrete are detrimental to compressive strength. However abrasion resistance will not be as seriously affected by voids if the aggregate is sufficiently bonded to take the brunt of the attack, shielding the softer paste. The important role aggregate plays in abrasion resistance is therefore considered in the next section.

2.4 Aggregates

Introduction

The influence of various aggregate characteristics on w/b was fully discussed in 2.2.1.1.1.1 and 2.2.1.1.1.2. It was seen that aggregates that are well graded, have a rounded or cubical shape, and a smooth surface texture, are ideal for the production of concrete. Conversely aggregates with excessive fine material, especially material passing the 150 micron sieve, and aggregates which are angular in shape, or aggregates with a rough surface texture will impact negatively on the w/b.

In this section the hardness and other related characteristics of the aggregates will be the main consideration.

Aggregates are placed in concrete primarily to reduce the cost of the mix and to restrain the high volumetric changes associated with cement paste.

They must also be sufficiently robust to resist the stresses associated with the mixing process.

In most applications aggregates must bond with surrounding cement paste to permit transfer of stress through the aggregate particles, without breakdown or excessive deformation. Furthermore, at the surface of pavements and hydraulic structures, they must resist abrasive and impact stresses.

Generally most aggregates will have superior compressive and tensile strength relative to the resultant concrete. There is however a significant spread depending on aggregate type, with some limestones having compressive strengths as low as 105MPa, while basalt can be as high as 300MPa (**Meininger(1991)** from Iyer et al (1975)).

Bettencourt Ribeiro(1998) observed that most of the concrete volume is occupied by aggregate, and therefore it is the main component that opposes the deterioration of concrete by abrasion. Thus provided that the strength of the cement paste is enough to secure the aggregate in the test sample during the abrasion test, its influence is relatively small, given its reduced proportion. In effect the binder connects the various aggregate particles, and if the bond capacity of those connections is adequate, aggregate will not be plucked out. He showed this to be the case where the mode of attack is tangential and of a gentle scratching nature, using a horizontal-grinding-machine that is virtually free from impact and high compression loads. His apparatus is similar to DIN 52108 [*sliding fine-abrasive,* appendix 5.02].

In conventional concrete floors and pavements the compaction and finishing processes tend to cause a measure of segregation, with the coarse aggregate subsiding somewhat relative to the mortar, and the fine aggregate subsiding somewhat relative to the paste. In this process any excess water is displaced upwards. The result is a layer of bleed water on the surface. Processes such as power floating are specifically designed to depress the coarse aggregate. The end result is a layer that varies between 2mm and 15mm, free of coarse aggregate, with the top 2 mm rich in paste. It is therefore clear that the somewhat submerged coarse aggregate will play virtually no role in surface wear, only in deep wear. ‘Surface wear’ is here arbitrarily defined as being restricted to the first one mm, after which wear may be considered as ‘intermediate’ to a depth of 5mm. There after it may be termed ’deep wear’. Chaplin(1991) considers the surface in an industrial warehouse to have failed once wear exceeds one mm, since dust prevention and smooth travelling with lifting equipment are both very important.

Paving: Unlike conventional floors, virtually no segregation takes place during compaction owing to the short duration of the vibration cycle and the semi-dry nature of the mix, which has relatively high inter-particle friction. Therefore in paving the coarse aggregate is also present at the surface and so plays an important role in abrasion resistance right from the beginning.

Meininger(1991) points out that abrasion resistance in conventional concrete is initially a function of the hardness of the fine aggregate, and that as wear progresses to deeper levels the hardness of the coarse aggregate comes into play. Their respective contributions are considered in sections 2.4.1 and 2.4.2.

2.4.1 Fine Aggregate

In this section fine aggregate is considered in terms of hardness/type, soundness, grading, shape and texture, and finally volume fraction relative to paste. Remarks and conclusions will be made where appropriate.

2.4.1.1 Hardness and Type

The literature on abrasion resistance indicates that there are two positions. The first states that the type of aggregate and its hardness *are related* to abrasion resistance. The second position is that there is *no clear relationship*. These positions are considered sequentially, and although they appear to be contradictory, the difference is easily explained.

(a) Position 1: Aggregate hardness/softness influences abrasion resistance

Addis(1989) subjected a number of mortar mixes with various aggregate types to the NBRI test, which applies an abrasive (silicon carbide C6/36 grit) below a rotating disc (=sliding fine-abrasive). Ranking the abrasion wear of the respective mortars yielded the following trends:

- Corundum and andesite crusher sands had the least abrasion wear
- Dolerite, dolomite, quartzite and granite followed, their wear closely banded together
- Decomposed sandstone and decomposed granite formed the next band
- Pure cement paste had the highest wear. It generally had 3 to 4 times more wear than the corundum and andesite mortars, approximately twice that of dolerite, quartzite, dolomite and granite, and about 20% more than the mortars made of decomposed materials. These ratios applied to corresponding compressive strengths ranging from 10MPa through 70MPa.

The NBRI test, is a good measure of the contribution made by the hardness of an aggregate towards the abrasion resistance of a mortar. It grinds the aggregate particles and paste matrix evenly, so that at a given paste quality, meaningful comparisons can be made between the various aggregates. The test is essentially a measure of scratch/gouge/shear resistance.

Jackson(1924) found that abrasion resistance of a mortar as measured by *rolling chained tyres* (see appendix 3.02) depended on the hardness of the fine aggregate. For example:

- lead and copper slags were very wear resistant
- sands of pure quartz led to high abrasion resistance
- carbonaceous shale impurities led to poor abrasion resistance.

Doulgerous(1995) did abrasion testing according to PCI.TM.7.11(wirebrush) [=*sliding wire bristles*, see appendix U.6.02] and found that a highly <u>weathered decomposed granite</u> aggregate had <u>140% of the wear</u> of an equivalent granite aggregate, even thought the compressive strengths were the same at all strength levels.

Chaplin(1991) did abrasion tests using the C&CA apparatus [=*rolling steel wheels*, see appendix U.4.06] and found that <u>limestone fine aggregate</u> substantially reduces abrasion resistance. He explains that the relatively soft material does not stand up to the crushing effects of the steel wheels

Grayston(1994) reported that in Norway hardwearing pavers had been developed from high strength concrete (75MPa) incorporating ‘special aggregates’ to withstand the effects of studded tyres. They were found to outlast ‘good quality asphalts’ by a factor of 3 to 4.

Hitotsuya(1988) did abrasion tests using a steel studded tyre (see appendix U.3.5). Pavers made with ferronickel slag as the aggregate (and a low w/c) had an average wear of 1,1mm after 117000 revolutions of the horizontal table compared with 2,6mm for pavers with 'normal' aggregate, and 4,8mm for asphalt. Field tests had similar ratios.

Ishai(1984) did abrasion tests on concrete pavers using DIN 52108 [=*sliding fine abrasive,* see appendix U.5.02] and found that when a layer of quartz was incorporated in the topping the abrasion resistance of the pavers increased between 25% to 50% relative to blocks made only with dolomite and basalt. Site tests consisting of tanks with steel caterpillar tracks skidding to a halt, resulted in metal marks on the pavement surface indicating the greater hardness of the quartz aggregate.

Lane(1978) concluded that well graded hard mineral aggregates resulted in concrete with good abrasion resistance. (This statement also applies to coarse aggregate).

Liu(1991) stated a hard fine-aggregate increased abrasion resistance.

Hutchings(1992) said that 'it is observed experimentally that abrasive grit particles of any shape will cause plastic scratching only if Ha/Hs>1,2, where Ha is the hardness of the abrasive particle, while Hs is the hardness of the surface being abraded. Accordingly *soft* abrasion occurs where Ha/Hs<1,2, while *hard* abrasion corresponds to Ha/Hs>1,2. Hard abrasion proceeds *much* faster than soft abrasion'.

The observation that a minimum ratio of hardness is needed for one material to scratch another provides the physical basis for the scale of hardness derived by the Austrian mineralogist Mohs in 1824. Mohs derived an integer hardness number to a sequence of ten minerals, each of which would scratch all those, but only those, below it in the scale:

- Talc
- Gypson
- Calcite
- Flourite
- Apatite
- Orthoclase
- Quartz
- Topaz
- Corundum
- Diamond

Meininger(1991) (from Stiffler K) stated that hardness is the single most important factor that controls aggregate wear. Abrasion-wear results from movement under pressure of an abrasive relative to the surface, resulting in scratching and surface pitting as mineral grains and particles are pulled from the matrix. [This statement is well thought out as it embraces two main mechanisms of wear, (1) the harder abrasive scratches out the softer material and (2) hard materials are plucked out of a paste matrix that has inadequate bonding ability].

Webb(1996) used the C&CA apparatus [=*rolling steel wheels*, see appendix U.4.06] to test the abrasion resistance of concretes made of various coarse and fine magnesian limestone aggregates, and found that the different fine aggregates tested had a significant influence on the abrasion resistance of a power finished floor. The magnesian limestone surfaces were penetrated to depths of 0,38mm and more, while the Trent valley fine aggregate surface (control) was only worn to a depth of 0,06mm.

Kettle(2000) states that, in its guidelines, the new standard, BS 8204: Part 2: 1999 places particular emphasis on the fine aggregate. The characteristics of the sand are vitally important as abrasion resistance is controlled by the properties of the surface layer, especially the top millimetre. He emphasises that concretes containing inferior sand are inappropriate for all concrete floors, even those exposed to very light traffic.

Ozkul(1996) did abrasion tests using DIN 52108 [=*sliding fine abrasive,* see appendix U.5.02] to determine the abrasion resistance of concretes made with coarse and fine aggregate slag sourced from an electrical arc furnace (34MPa at 28-days). The abrasion resistance results were approximately 25% more than a mix made entirely from limestone coarse and fine aggregates (reported by Ozturan:1987).

(b) Position 2: There is no clear relationship between aggregate hardness and abrasion resistance

Abrams(1921) found that fine aggregate type did not materially influence abrasion resistance providing it was sound, clean, with not too much fines.

[It should be noted that Abrams derived his conclusions by using a Talbot-Jones Rattler (= 95mm *impacting steel balls,* see appendix U.2.01), which resulted in severe impact that led to 'deep penetrations', so deep in fact that this test is much more a measure of the concrete's 'core' strength. (Typically, loss of material amounted to depths ranging between 10mm and 30mm). Impact tests can also be misleading in other ways. Smith(1958) found that soft aggregates, which may be more resilient, tend to resist impact type loads better than hard brittle aggregates. It is therefore not advisable to use an impact test, and especially a severe one such as Abrams's when attempting to understand surface wear subject to normal abrasive loads. On the other hand his test is ideal when studying deep abrasion as would apply in stilling basins].

Lane(1978) explained that aggregate hardness (or lack of hardness) did not affect abrasion resistance of strong concretes, in excess of 55 MPa.

[This phenomenon has been reported by a number of investigators, directly and indirectly, but with no real attempt to explain it. One possibility is that an aggregate particle - even a fine aggregate particle - under a concentrated load e.g. a small steel wheel of a tine-trolley carrying a load in a warehouse, will experience very high compressive stresses arising from this load, and it can easily be calculated that these stresses, if acting upon a small enough area, may exceed the expected crushing strength of the aggregate particle in question. (Siro(1991) refers to compressive stresses varying between 70MPa and 100MPa beneath the small steel wheels used in abrasion testing equipment in Norway, Sweden and Finland). The aggregate particle however is not in a simple state of unconfined uniaxial compression, but will experience lateral restraint in the direction of the two minor axes as it attempts to dilate in proportion to Poisson's ratio. A strong and stiff support environment from the surrounding concrete along these axes, which may be associated with high strength concrete, has the potential to increase the normal crushing threshold very substantially. (Newman:1997b explains that the mode of failure in unconfined compression testing is that of crack initiation followed by progressive crack propagation. However as the state of stress tends from unconfined uniaxial compression towards a truly 'triaxial compression, the mode of failure changes from 'brittle' to 'ductile', and no clear fracture mechanism is exhibited since the cracks are extremely small and localised').

The effect of this horizontal buttressing on an aggregate particle under load, explains why some aggregates that are known to be relatively soft appear to resist the crushing effects of small steel wheels so well when used in stiff and high strength concretes.

Sectional conclusion

The evidence strongly indicates that the hardness of the fine aggregate plays an important, if not the leading part, in abrasion resistance. The two exceptions were seen to be:

- the case of very severe impact where aggregate and paste are shattered, irrespective of their hardness, and indeed hardness may exacerbate this effect. Once shattered the aggregate particle is easily dislodged from the matrix.
- the case of concretes that have a high compressive strength and stiffness. Here abrasive forces that would normally lead to localised crushing effects in both aggregate and paste, are resisted by concrete that locally has enhanced compressive strength owing to significant lateral support.

2.4.1.2 Soundness

Materials which easily break up under mechanical loads or degenerate on weathering, should be considered as unsuitable as aggregate for any concrete which is to resist abrasion.

Partially decomposed aggregates, such as some 'river sands', should be used with caution. The author has observed that paving made from decomposed granites from a nearby commercial quarry break up under pedestrian and vehicular traffic. This material has a highly flawed internal structure.

A very simple way for checking for internal flaws is to apply the 'Papenfus tea cup' test. In this test an individual aggregate particle is placed on a table, and then lightly tapped with an empty porcelain teacup. If it starts disintegrating after a few taps, going from say 4mm in nominal size to virtual dust with continued tapping, the material should not be used in concrete that is to resist even relatively mild abrasive loads. Although the test is crude, it is nevertheless a quite dramatic illustration of the presence of internal flaws, or the absence thereof, and has over the years helped to educate a number of suppliers of 'river sand'.

Addis(1989) found that mortars made from decomposed sandstones and granites were only marginally more wear resistant than pure concrete paste, and had significantly increased abrasion wear relative to mortars made of hard/sound crusher sands. His tests were made using the NBRI test, which applies an abrasive (silicon carbide C6/36 grit) below a rotating disc (=sliding fine-abrasive).

Lightweight materials are another area of concern and should be carefully evaluated before using.

Agron(1998) reported on a lightweight volcanic material (lahar), used as a fine aggregate for paving. It had an SG of 1.2 and water absorption of 26%. Borderline compression testing values between 20MPa and 25MPa were achieved and may just be acceptable in non-freeze thaw environments. [Agron appeared satisfied with the abrasion resistance values ASTM C779 Proc B [=*rolling dressing wheels*, see appendix U.3.06], but his results showed an increase in abrasion wear from 1.4% for a 30:50 blend of the lahar sand: pea gravel to 2.2% when the ratio increased to 70:30. Clearly this demonstrates the relative softness of the lahar sands].

Sectional conclusion

Unsound aggregates should be avoided in applications where abrasive loads are expected.

2.4.1.3 Grading and Particle Size

The findings of the various investigators have been grouped under appropriate sub headings, and a general conclusion is made at the end of the section.

(a) Fine sands have higher abrasion resistance

Fwa(1990) did 'abrasion' tests by loading 100mm cubes into a LA abrasion machine [=*impacting steel drum*, see appendix U1.01], resulting in deep wear, mainly from impact, but there would also be some shearing from sliding, and mild crushing effects from rolling. Keeping w/c and a/c constant, he found that after 2000 revolutions the cubes made with a siliceous finely graded fine aggregate had lost 40% of their mass compared to 75% for cubes made with a siliceous coarsely graded fine-aggregate. [It should be noted that the grading changes described above also resulted in a change of compressive strength, from 47MPa for the fine aggregate to 37MPa for the coarse aggregate, even though w/c was constant! It may be that the siliceous sand had such poor interfacial zone bonding capabilities, that reducing the particle size also minimised this deficiency, i.e the stress concentrations that initiate crack growth and subsequent propagation (Newman(1997a)) would be smaller for the smaller 'inclusions' (finer aggregate)].

Schuman(1939) did abrasion tests using *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] and found that at equal w/c, abrasion resistance of concrete made from fine sand was slightly greater than that made from coarse sand.

(b) Coarse sands have higher abrasion resistance

Jackson(1924) did abrasion tests with rolling chained tyres (see appendix 3.02) and found that coarse grading led to higher abrasion resistance relative to finer sand. [A possible explanation is that the coarse aggregate particles were better embedded in the paste matrix, and were therefore better able to resist being dislodged by the abrasive forces].

(c) Coarse and fine sands are equally good

Chaplin(1991) did abrasion tests with rolling steel wheels. He varied the grading of fine aggregate from very fine to very coarse and found this made very little difference to the abrasion resistance.

(d) Sands must be well graded

Liu(1991) stated that well graded fine aggregates increased abrasion resistance.

Jackson(1924) did abrasion tests using rolling chained tyres and found that blast furnace slag fine aggregate was harsh, resulted in excessive voids and was therefore a poor substitute for natural sands. [The reference to the 'excessive voids' indicates a deficiency in the grading].

Lane(1986) recommended that grading for concrete paving should conform to the following sizes:

Recommended grading limits for aggregates for concrete masonry units:

Sieve size (mm)	Cumulative percentage passing
19,0	100
13,2	90 - 100
9,5	75 - 100
4,75	60 - 85
2,36	40 - 65
0,3	10 - 25
0,15	5 - 15
0,075	2 - 10

Aggregates falling outside these limits could be used but should be tested in full scale production.

(e) There should not be excessive fine material

Lane(1978) recommended that for good abrasion resistance the –300 micon and –150 micron fractions should be minimized.

(f) Some sub 75 micron material promotes fine filler effect

Dumitru(1999) varied the percentage of minus 75 micron material in the fine aggregate from 0% to 25%, and found optimum compressive strength in the range 5%, with 10% equivalent to 0%, both of which were only very slightly less than the 5% values. This may be ascribed to the filler effect, and may be expected to improve abrasion resistance as well, insofar as improvement to the mortar constituent affects abrasion resistance.

Sectional conclusion

No clear trend has emerged regarding the coarseness or fineness of the sand. Perhaps this should come as no surprise considering the divergence of test methods used; ranging from severe impact/tumbling effects, to a steady grinding process with an abrasive, to a heavily loaded chained wheel, and finally the relatively gentle rolling of lightly loaded small steel wheels. Wear depths ranged from very deep, to half a millimetre. From the limited number of findings given here, and considering the diversities in materials and test methods, all that can be said is that for certain tests coarser sands appear to perform better than finer sands, in other tests this situation is reversed, and in the case of the small steel wheels, it appears that coarse and fine sands perform equally well.

Sands should notwithstanding be well graded, with no excessive amounts passing the 300 micron and 150 micron sieves. Finally it would seem that approximately 5% in sub 75 micron zone is not detrimental and may even improve abrasion resistance as a result of the ‘fine filler effect’.

2.4.1.4 Shape and Texture

Shape and surface texture are known to have an effect on the water requirement and therefore abrasion resistance of concrete. This was discussed in section 2.2.1.1.1.1 (b) and (c). In this section the other effects of shape and texture on abrasion resistance are considered, except that the role played by shape and texture on aggregate/paste bond is discussed in 2.5.3.

Gjorv(1990) did abrasion tests using truck tyres with steel studs (=*rolling studded tyres*, see appendix 3.03). He found that there was a distinct improvement in abrasion resistance when substituting the 2mm to 4mm component of natural jasper aggregate with the same fraction of crushed jasper aggregate, even though this was accompanied by an increase in the water demand and consequently a reduction in compressive strength.

Shackel(1994) reported on the findings of MA20 abrasion tests (=rolling steel balls, see appendix U.2.13]. Mixes incorporating crushed aggregate tended to exhibit higher abrasion indices than those manufactured using river gravel.

Dumitru(1999) did abrasion tests using the C&CA apparatus [=*rolling steel wheels*, see appendix U.4.06] and found that the abrasion resistance of concrete specimens tended to increase slightly as the quantity of 'natural' fine aggregate was progressively replaced with a 'manufactured' basalt and crushed river gravel fine aggregate.

Sectional conclusion

Crushed aggregates generally have sharp corners and are often angular in shape. Although these attributes give rise to higher water demand and hence lower compressive strength, they are substantially hard and sound. This more than compensates, as there appears to be good agreement that crushed aggregates promote good abrasion resistance relative to natural aggregates of equivalent type. Crushed materials are not decomposed to any extend and therefore have fewer internal flaws. Furthermore the virgin rock corresponding to a quarrying and crushing operation is generally carefully selected to be suitable for concrete manufacture, given the expense of installing and managing a crushing operation. Summarizing it may be said that the hardness and soundness aspect of aggregate can contribute more to abrasion resistance than its shape and texture.

2.4.1.5 Volume Fraction Relative to Paste

Given that cement paste is much softer than most fine aggregates, it seems reasonable to assume that an increase in the proportion of the harder constituent will also lead to an increase in abrasion resistance. On the other hand, it also seems logical that there will be a point beyond which further increase in sand proportion relative to the paste will lead to a reduction in compactability, necessitating an increase in w/b if optimum density is to be maintained. In fact, evidence presented in 2.3.2.2 states that voids are unavoidable at high a/c ratios, even where w/c has been increased. Finally, it seems reasonable to expect that there will be a middle of the road zone where paste/aggregate ratios will neither be too high or too low to fall in either camp, and thus good abrasion resistance may be expected over a range of paste/aggregate ratios in this zone.

These three positions are illustrated in figure 2.9 and may be summarized as follows:

Zone 1: High proportions of cement paste. In this zone abrasion resistance goes from low to high as aggregate proportion increases.
Zone 2: Intermediate proportions of paste. In this zone abrasion resistance remains high and is essentially constant over a middle range of a/c values.
Zone 3: Low proportions of cement paste. In this zone abrasion resistance goes from high to zero as paste proportion decreases

The findings of a number of investigators are now considered with reference to these positions:

Addis(1989) did abrasion testing with sliding fine abrasive grit on mortars with varying aggregate types. He found that an equivalent pure cement paste has the lowest abrasion resistance.

This corresponds to the start of Zone 1.

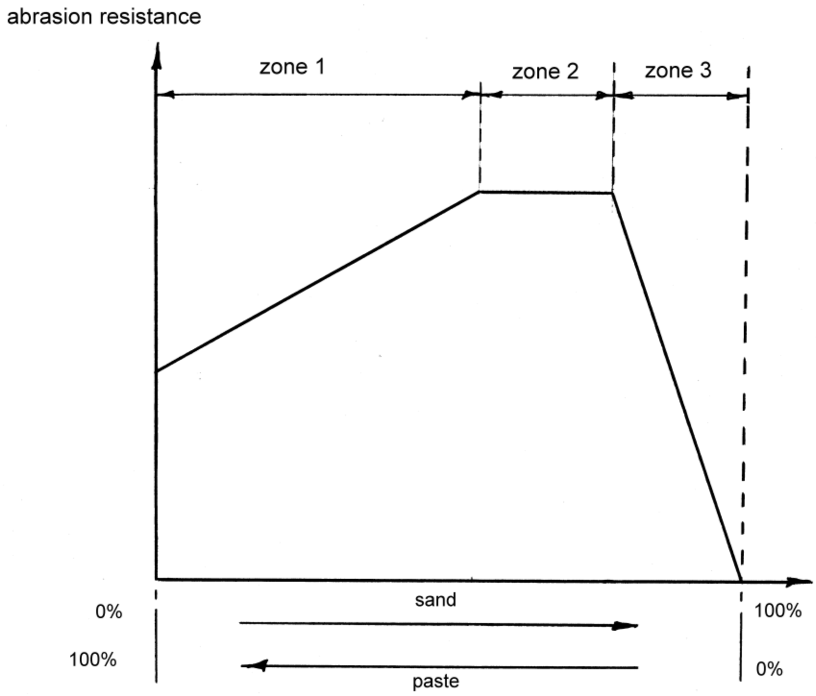

Figure 2.9 Relationship between paste/sand proportion and abrasion resistance

Fruchtbaum(1938) did abrasion tests using *rolling steel wheels* (see appendix U.4.05) on floors with various sand:cement ratios, (with some minor variations in water:cement ratios). He confirmed earlier work done by L F Fairchild in 1933, that the mix with the highest sand content also had the best abrasion resistance. The reduction in wear as a/c (volume basis) increased from 2,32 to 2,67 to 3,1 was of the order of 20%, comparing worst with best.

From this it may be seen, that providing w/c is not compromised, increased aggregate proportion increases abrasion resistance. This amounts to saying that for a given paste quality, abrasion resistance increases if a greater proportion of wear resistant material is used.

Note that an a:c of 3.1 still represents a rich paste, and clearly these findings fall within Zone 1.

Schuman(1939) did abrasion tests using *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] and found that at equal w/c, abrasion resistance sometimes increased considerably with increasing proportions of fine aggregate up to a certain point, after which abrasion resistance decreased again.

These findings correspond to Zone 1 and Zone 3, with a very narrow Zone 2.

Again it may be said that abrasion resistance increases with an increasing amount of wear resistant material, for a given paste quality. There is however a point at which aggregate proportion becomes so great, that air voids are introduced, which reduces aggregate paste bond. This seems a likely explanation for reduced abrasion resistance where sand/cement ratios were excessive.

Jackson(1924) did abrasion tests with *rolling chained tyres* (see appendix 3.02) and noted that 'cement in itself has very little resistance to abrasion, but acts simply as a binder, so that for any mix in which the cement content is high enough to produce a strong bond, the actual wear will be proportional to the hardness of the aggregates'. Jackson found that an a/c ratio of two was something of a watershed. There was no improvement to abrasion resistance in reducing the a/c below 2, but if this ratio was increased above 2 there would be an increase in wear. For example a mortar surface of a/c = 3 had very much more wear than a surface with a/c = 1,5 or 2.

In the a/c ratios given here, Zone 2 would be characterised by a/c = 1.5, while a/c = 3 falls in Zone 3. It is difficult to explain why his a/c ratio corresponding to the Zone1/Zone 2 interface was so low, unless 'a' in the ratio a/c represents both coarse and fine aggregate.

Smith(1958) used three abrasion tests, (1) *rolling steel balls*, see appendix U.2.11, (2) *impacting fine abrasive*, see appendix U.5.20, (3) *rolling dressing wheels over fine abrasive*, see appendix U.3.08, and showed that abrasion resistance did not follow any trend for variations in sand content between 38% and 60%, (w/c was constant). This trend was observed for all three abrasion tests.

The lack of a clear incline followed by a subsequent decline in his abrasion resistance values (rather there was a general scatter), may be an indication that his range of paste/sand ratios all fall within Zone 2. This does not seem an unreasonable explanation given that the stated percentages for sand content are neither unreasonably high nor low.

Clark(1980) did freeze thaw experiments on concrete paving on a/c = 3, and achieved w/c = 0.30 for a moisture content of 6.9%. At an a/c = 5 a very similar moisture content of 7.0% resulted in a w/c = 0,45.

[These results show as a/c increases so must w/c if voids are to be avoided. Essentially this means that if the quantity of the lubrication medium (i.e. the paste) is reduced, its lubricating power needs to be increased, i.e. if a/c is increased towards Zone 3, w/c must be increased to keep it in Zone 2. If this is not done, then the voids will increase dramatically (a characteristic of Zone 3), and this is far more detrimental to strength and abrasion resistance than the effect of the increase in w/c. Summarising it can be said that by increasing the lubricating power of the paste (i.e. higher w/b), it is possible to stay in Zone 2, even though a greater proportion of aggregate is used].

Sukandar(1993) did abrasion tests on concrete pavers according to ASTM C779 Proc C [=*rolling steel balls*, see appendix U.2.12] with a:c ratios varying from 3 to 9. Blocks made with the higher cement contents had substantially more abrasion resistance, with a 300% improvement in the 455 kg/m3 mix relative to the 153 kg/m3 mix. [A careful analysis of Sukandar's results however (see appendix R), shows that the main reason for the improved strength is a reduction in % voids, rather than a lower w/c. Although the w/c is substantially lower for the high cement contents, more importantly, the void content of the high cement content is much lower relative to the mixes made from low cement contents. He reports an improvement of 9,15% in unit weight for the highest cement content relative to the lowest.

Increased density was obtained for the high cement contents even though the w/c ratio of the lowest cement content at 0,34 was considerably more than the highest at 0.21. It is therefore evident that <u>*more of a viscous paste is better able to reduce voids than less of a more fluid paste*</u>. Adequate flow is achieved by virtue of the greater volume of paste in the case of the low a/c mix].

Sukandar's low a/c ratios may be identified with Zone 2, while the higher ratios are clearly Zone 3.

Dowson(1980) found that typically <u>aggregate / binder ratios</u> in pavers ranged between <u>3 and 6</u>.He recommended the use of a void meter to determine the lowest possible percentage of air voids when considering aggregate selection. In this way paste content can be minimized for a given w/c.

In effect Dowson's recommendation recognises that every fine aggregate will have its own unique limits for Zones 1 to 3. Using the void meter helps to find the transition between

Zone 2 and 3, which may be regarded as the ideal, since at this point a/c is as high as possible consistent with no appreciable drop in abrasion resistance.

Sectional conclusion

The findings of the various investigators appear to fit well into a model defined by zones that characterise abrasion resistance in terms of the relative proportions of paste and fine aggregate.

2.4.2 Coarse Aggregate

As was the case with fine aggregate, coarse aggregate is considered in this section in terms of type/hardness, grading, size, and finally volume fraction relative to *sand*.

2.4.2.1 Hardness and Type

Most investigators have found that abrasion resistance of concrete improves with increasing hardness of the coarse aggregate. Their findings are reported immediately below. This is followed by a section where abrasion resistance does not seem to be dependant on aggregate hardness. A third position in effect states that the hardness of the aggregate is of no concern. An attempt is made to reconcile these apparently opposite positions.

(a) Position 1 : Abrasion resistance is a function of the coarse aggregate's hardness

Mohs scale of hardness calibrates various minerals in order of their ability to resist being scratched. Scratching and gouging are a major form of abrasion wear. There is thus a correlation between the type of mineral, or in the context of concrete the type of aggregate, and the abrasion resistance of concrete made with that aggregate. In the findings reported below there is often an underlying assumption that different aggregate types have different hardnesses leading to different rates of abrasion-wear.

Alexander(1984) concluded from a literature study that generally hard aggregates give better abrasion resistance than do soft aggregates, but poor correlation often exists between aggregate hardness and the strength of the concrete that contains these aggregates. He reported on a case involving a terrazzo tile where the hard aggregate resulted in low abrasion-wear even though a low cement content had been used. The tiles however did not perform satisfactorily in practice. [Certain abrasion tests have a relatively gently grinding action, e.g. DIN 52108. This explains how a hard aggregate can remain anchored in the matrix even though the paste is weak. However, once subjected to the rigors of traffic the aggregate particles are easily plucked out, particularly if sand/grit is present on the surface to gauge out the supporting paste/mortar].

Conversely, Alexander continues, a good quality matrix will not yield good wear properties if a 'soft' aggregate is used. [This is confirmed by Addis(1989) who showed that even a 70 MPa paste was slightly less wear resistant than an equivalent 70MPa mortar incorporating decomposed aggregates. By incorporating hard aggregates, wear was reduced to approximately ⅓ - using the same paste.]

Bettencourt Ribeiro(1998) did abrasion testing on roller compacted concrete using an Amsler-Laffon machine [=*sliding fine-abrasive,* similar to appendix 5.02]. He tested specimens with approximately 40% fly ash at age 90 days and found that whereas compressive strength improved quite dramatically as binder content increased from 90kg/m3 to 140kg/m3 to 190kg/m3, the abrasion resistance stayed virtually constant, for a given aggregate type. However a change in aggregate type produced a remarkable change in the abrasion depth. Average depths of wear were as follows:

- For schist aggregate 4.1 mm
- For limestone aggregate 2.2 mm (the fine aggregate was siliceous sand)
- For granite aggregate 0.8 mm

He attributes this to the nature of the test, where the mode of attack is tangential and of a gentle scratching nature. [The fine-sand abrasive grinds away but imparts minimal shock to the specimen, and hence aggregate particles are not easily loosened]. Even if they should be loosened they cannot be dislodged as the loosened particle is likely to remain trapped in the matrix, unable to drop out, being too close to the base/wear-plate, given the narrow distance between sample and base/wear-plate (approximately equal to the thickness of the fine-sand abrasive).

Therefore the ability of the aggregate to resist a scratching type of abrasion without the possibility of particles being dislodged and ejected during the test means that this test is essentially exclusively a measure of the hardness of the aggregate.

Dihr(1991a) did abrasion tests using an apparatus developed by the C&CA [=*rolling steel wheels,* see appendix U.4.06] and found that lightweight aggregate (12mm) resulted in lower abrasion resistance, even for equivalent compressive strength.

Doulgeris(1995) observed that aggregate derived from softer rock (such as limestones and dolomites) or containing softer constituents (such as feldspars in decomposed granite sand) will be abraded more rapidly. Most commercially available aggregate meeting the requirement of SABS 1083 should prove acceptable for general use as they will always be harder than the cement matrix. For extreme conditions the harder the aggregate the better the concrete performance is likely to be, although some aggregates bond poorly while others may shatter under impact loads.

Jackson(1924) did abrasion tests using *rolling chained tyres* (see appendix 3.02) and found that the abrasion resistance of concrete made of crushed coarse aggregate is related to the wear properties of the parent rock. Expressing wear as a percentage of volume of abraded material he found that different aggregate sources profoundly affected the resultant abrasion wear as follows:

Quartzite	= 1,8%
Altered diabase	= 2,3%
Granite porphyry	= 2,4%
Calcareous slate	= 4,0%
Argillaceous dolomite	= 4,0%
Sandstone	= 5,3%
Argillaceous limestone	= 6,3%
Limestone	= 9,5%
Sandstone	= 13,8%
Dolomite	= 14,5% (presumably of inferior hardness)

Khalid(1993) used the ‘flat bed polishing machine, i.e. FBPM’ [=rolling fine-abrasive, see appendix 5.10] to do testing on concrete flags surfaced with various types of exposed aggregate and found that limestone coarse aggregate, a relatively soft aggregate, had the lowest resistance to abrasion. [The FBPM is generally used to test for skid resistance, but can effectively simulate ‘shoe sole rubber’ or ‘tyre rubber’, the main means of abrasion in concrete pavers, and may therefore be regarded as a useful test, although it may be too gentle to abrade hard aggregate].

Komonen(1998) did abrasion tests using *rolling studded tyres* (see appendix U.3.04) and found that the ‘quality’ of the coarse aggregate (as assessed by % mass loss Los Angeles, % Impact value, abrasion value) determined the abrasion resistance of concrete paving blocks. In descending order of performance these aggregates were vulcanite/granite, white gabbro, and quartzite. Aggregate quality was more significant than compressive, flexural or splitting tensile strength. It even masked the type of cement used, such that the Cem I 42,5 R mix with the harder granite/vulcanite aggregate was more abrasion resistant than the softer quartzite employing Cem I 52,5 cement.

Komonen found that the different aggregate types had different wear mechanisms:

1. The continuously graded high strength *granite* mix formed an entirely homogenous system that had no weakness zones. The surface looked like pure granite after the test.
2. Aggregate crushing was the abrasion mechanism of the *quartzite* aggregates, which did not endure the impact type loading of the studs and were rapidly crushed and abraded. The broken aggregate was then dislodged, resulting in voids with a maximum depth of 7 mm below the surface of the paver. The abrasion thereafter depended on the strength of the cement matrix.
3. Bond failure between aggregate and cement paste, followed by loss of material, was the predominant mechanism of wear in the case of the angular shaped *gabbro* aggregate. [This bond failure is likely to have been aggravated by a substantially lower cement paste strength, which may be deduced from the substantially lower densities and compressive strength results].

The test that Komonen used applies a very high compressive stress to the local area beneath and around the steel stud, and the corresponding wear mechanisms could easily be various forms of sub-surface cracking (see chapter 3), rather than the milder compression failure of a few asperities which is the usual form of abrasion wear. The severity of the test perfectly matches the actual application, since the same wheels are used in the test that are used on the roads, where rutting may be as deep as 35mm after two years of traffic.

Laplante(1991) found that the type of coarse aggregate made a large contribution to the abrasion resistance of the concrete. This is best illustrated by the penetration of the rolling steel balls of the ASTM C779 Proc C test:

- Trap rock = 0,18 mm
- Granite = 0,22 mm
- Dolomite = 0,35 mm
- Limestone = 1,3mm

The penetration in the softest material relative to the hardest was thus over seven hundred percent. By way of contrast the difference, considering the four compressive strengths from the same mixes was less than 10%. Aggregate type is thus most important in abrasion resistance.

Laplante(1991) showed that the abrasion resistance (according to ASTM C779 Proc C) of concrete was the average of its two constituent materials, mortar and coarse aggregate. A mortar of high wear resistance with a relatively soft coarse aggregate (e.g. limestone) resulted in a concrete with an abrasion resistance that was less than the mortar's. Conversely a mortar of relatively low wear resistance with a hard coarse aggregate (e.g. granite) resulted in a concrete with an abrasion resistance that was more than the mortar's.

Laplante showed that the coarse aggregate could influence the abrasion resistance of the concrete substantially more than silica fume could. Aggregate could even make a greater impact than w/b ratio. He found that as long as the w/c ratio was maintained at about 0,30, the concrete's abrasion resistance almost equals that of the coarse aggregate source rock.

Laplante also found that uniform wear was achieved when the coarse aggregate and the mortar had approximately the same abrasion resistance. Gravels performed as well as crushed stone providing excessively soft materials were discarded. Siliceous materials had better wear characteristics relative to calcareous materials. The characteristic round shape of the gravels was not detrimental to abrasion resistance.

Liu(1981) did abrasion tests according to (the forerunner of) ASTM C1138 [=*impacting steel balls* (mild), see appendix U.2.07] and found that for a given aggregate type, abrasion loss increased when compressive strength reduced. However abrasion losses were 'slight' relative to changes in compressive strength whereas substantial changes in abrasion resistance occurred for different aggregate types. For example the wear in limestone was twice that in chert. The aggregates tested in his programme had the following order of performance (in decreasing order of abrasion/erosion resistance): chert, trap rock, quartzite, limestone, granite, slag.

He also observed that the initial wear took place at a greater rate, indicating that the outer skin of *mortar* is more susceptible to wear compared to the combination of coarse aggregate and mortar deeper in. (There certainly would have been some settlement of the coarse aggregate in the 102mm deep sample, although steel trowelling by hand would have overcome some of the negative effects on the w/b at the surface associated with bleeding. However, given the relatively deep abrasions of between 5mm through 9mm, depending on aggregate type, the coarse aggregates would have been engaged at these depths allowing the comparisons in the different aggregate types).

Liu found a highly significant correlation between Moh's hardness of the various aggregates and abrasion resistance of the corresponding concrete.

Van der Klugt(1989) describes various abrasion tests used to determine the abrasion resistance of floor tiles. Using the P.E.I. test (=small *rolling steel balls*, with no load, see appendix U.2.08), he found a fine-corundum abrasive (working with the steel balls) was so hard that it always resulted in some abrasion wear. A softer abrasive resulted in less material being removed. However, certain tiles were so hard than when subjected to the 'Turning Test with Sand' no scratch marks could be seen. In spite of this he found that even the softest abrading materials, e.g. leather soles, eventually resulted in some slight wear, even if this only amounted to some discolouration. He therefore concluded that a zero scratch mark from a particular mineral on the Mohs scale did not mean that softer materials would not in time cause some wear. [Although this work applies to ceramic tiles, it nevertheless illustrates that there is a relationship between hardness and abrasion wear, with harder abrasives resulting in more wear, on the other hand, soft abrasives would in time, also result in wear, however slight].

Liu(1991*)* made a review of several abrasion tests and the factors that promote abrasion resistance and stated that abrasion resistance is a function of the hardness of the coarse aggregate (amongst other things).

Malhotra(2000) used the *rolling steel balls* (ASTM C779 Proc C, see appendix 2.12) test to determine the seven year abrasion resistance of concrete slabs subject to ice abrasion in arctic tidal zones. He found that concretes made with normal weight aggregate (crushed limestone, SG = 2.7, absorption = 0,6%) had superior abrasion resistance relative to lightweight aggregate (expanded shale, SG = 1.8, absorption = 10.1%). [The corresponding compressive strengths were essentially equivalent, indicating the abrasion resistance is influenced by the hardness of the coarse aggregate to a much greater extend than is compressive strength. (However in very high strength concrete the physical strength of the aggregate does play a significant role in compressive strength)].

Ozturan(1987) did abrasion tests according to DIN 52108 [=sliding fine abrasive, see appendix U.5.02] and found that different types of coarse aggregate resulted in concretes with different abrasion resistances. Natural quartz gravel gave the highest, followed by granite, while the limestone concrete had a substantially lower abrasion resistance.

Paving: In chapter 8 of volume 1 there are a number of photographs depicting coarse aggregate that has become very prominent as a result of abrasion wear of the mortar (corresponding to 3rd and 4th degree abrasion). The coarse aggregate therefore offers a measure of protection to the softer mortar component. In such advanced stage of wear, hardness of the aggregate exclusively governs the rate of abrasion, unless the aggregate particle is dislodged. Generally examples of hard aggregates (in descending order of hardness) include corundum, andesite, dolerite, dolomite, quartzite, granite and sandstone.

Examples of soft aggregates would be decomposed sand-stones and decomposed granites, and should be avoided in surface concretes.

Rushing(1968) used the ASTM C418 – 58 sandblast test (=impacting fine-abrasive) to determine the effect of various coarse aggregates on abrasion resistance. He found that natural coarse aggregates had greater abrasion resistance relative to expanded clay coarse aggregate.
He also found that the LA abrasion resistance values of the aggregates did not necessarily indicate the abrasion resistance of the resultant concrete (it is more of an impact test). On the other hand, he found that aggregate type (source, hardness) determines abrasion resistance rather than grading.

Sustesic(1996) did abrasion testing according to DIN 52108 [=*sliding fine-abrasive*, appendix 5.02] on concretes made with FeCr carbure slag aggregate (for both coarse and fine aggregate), a waste product from ferro chromium production, as well as with natural aggregates consisting of crushed river sand and crushed quartzose amphibolite coarse aggregate. The abrasion wear on the very hard FeCr slag aggregate was approximately 50% of that of the natural aggregate.

Yates(2000) used the EN 1342:1999 test, ‘slabs of natural stone for external paving’ ((=sliding fine abrasive), to measure the abrasion resistance of 35 sources of natural rock, either limestone, sandstone or slate. The highest abrasion resistance, a sandstone from Rockingstone had a wide wheel abrasion resistance of 15,7mm, whereas the lowest result of 34,9mm was also a sandstone, from Red St Bees 2. Aggregate source therefore is seen to play an important role in the abrasion resistance of natural stone slabs. By extension the abrasion resistance of the concretes made with these aggregates will also be influenced by the source. Clearly the source of the aggregate, not just the type, influences its hardness.

Wastlund(1946) did wear tests on various concretes using steel wheels and leather covered wheels that revolved in a circle [see appendix U.4.07]. He found that blocks made of limestone abraded at a greater rate than concrete blocks made with harder aggregates such as ‘sand’ and ‘gravel’.

Surface Abrasion and Coarse Aggregate

There are several reasons why abrasion resistance is often *not* related to the hardness of the *coarse* aggregate, and these will be considered in the following two sections. Perhaps the most common reason is that the coarse aggregate tends to settle slightly while the concrete is still in a fresh state, as it is generally heavier than the mortar matrix, or it is mechanically depressed by a floating/trowelling operation. Therefore in stating that abrasion resistance *is* related to aggregate hardness, as has been said in this section, this implies that the aggregate was at or very near the surface. This begs the question, ‘How can this be, given the processes of settling and finishing etc?’. By carefully considering the test each author used in this section, and especially considering the preparation of the test specimens (in the instances where this aspect was disclosed) it has been possible to put forward various arguments, and to group the various authors appropriately:

(i) Sawed test face: It appears that Ozturan(1987), Bettencourt(1998) and LaPlante(1991) all did tests on sawed concrete faces, thus ensuring that the proportion of coarse aggregate at the ‘surface’ was already representative of that deeper into the section.
(ii) Deep abrasion: The tests that Jackson(1921) and Liu(1981) did tended to abrade to levels of at least 5mm, going up to 25mm in the case of Jackson and 9mm for Liu. At these depths, even with some settlement, the coarse aggregate would be engaged, unless the consistency of the mixes was very fluid.
(iii) Exposed aggregate: Khalid did tests on slabs with exposed aggregate, while Yates(2000) tested slabs made of natural aggregate, and van der Klugt tested

ceramics. Clearly the 'coarse aggregate' would immediately contribute 100% to abrasion resistance in these instances.

(iv) Near surface aggregate: In concrete pavers the consistency of the mix is such that there is minimal settling of the heavier coarse aggregate. Therefore in all but the mildest abrasion tests the contribution of the coarse aggregate would be reflected almost immediately, and would be essentially the same with ongoing depth of wear. This explains the contribution of the coarse aggregate in Komonen(1998).

(v) Unexplained: Authors Rushing(1968), Siro(1991), Malhotra(2000)and Sustesic(1996) did not give sufficient details on the preparation of their samples. So for example, it was not possible to determine whether Rushing did his abrasion testing on the top face of his prism, in which case the coarse aggregate would have settled somewhat, or if he tested the prisms on their sides, in which case the proportion of the 'surface' coarse aggregate would have been more representative with aggregate deeper in.

Position 1 conclusion

The hardness of the coarse aggregate has been shown to be an important factor in the abrasion resistance of the surfaces discussed in this section. *A necessary proviso is that the abrasion test employed is able to penetrate deeply enough to reach the coarse aggregate. Where abrasion tests with a mild abrasion action are intended to simulate the abrasion resistance of the coarse aggregate, the test sample should be prepared in such a way as to expose the coarse aggregate, or otherwise the coarse aggregate should lie very near the surface.*

(b) Position 2 : Abrasion Resistance does not depend on aggregate hardness/type

The findings of the various authors quoted in this section may be explained in terms of three or four arguments. These are given as conclusions at the end of this section. The relevant conclusion is therefore referred with each author's findings, and in this way unnecessary repetition is avoided.

Alexander(1984) reported that it was possible to measure <u>similar LA abrasion values for aggregates of very different hardnesses</u>. In such instances the abrasion resistance of the corresponding concretes would not correlate well with the LA values.

[Essentially two aggregates with 'very different hardnesses' can have similar LA abrasion values if they also have proportionally different toughnesses according to the ratio $(\text{toughness})^4/(\text{hardness})^3$, since this *ratio* determines fracture resistance. (See **conclusion 1** at the end of this section for a discussion of the importance of toughness relative to hardness). Note that the LA abrasion test imparts a significant degree of impact from the heavy tumbling balls. The quality required in aggregate for resisting impact is *toughness*, not hardness. On the contrary, an increase in hardness results in a reduction in impact resistance, unless accompanied by an increase in toughness. But since it is hardness that primarily determines abrasion resistance, aggregates that are both hard and tough will also perform well both in abrasion tests and the LA test].

Abrams(1921) studied the effect of various types of aggregate on abrasion resistance, including limestone, granite, trap, sandstone, slag, flint, marble, larva rock, tufa, firebrick, cement clinker, and boiler cinders. He found that <u>aggregate type did not make much difference to the abrasion resistance</u> of the corresponding concretes, except in the case of the poorly structured boiler cinders.

[The heavy tumbling steel balls of Abram's tests (see appendix U.2.01) result in impacting effects. Wear depths of half an inch were typical, which would sufficiently expose the coarse aggregate to the action of the tumbling balls, and the Talbot Jones rattler is therefore a very good test for assessing the contribution of the various coarse aggregates relative to *impact* resistance (as the LA abrasion test does), which is not necessarily proportional to abrasion resistance. Hard wearing aggregates are often very brittle, and easily fracture under impact. Softer more resilient materials tend to absorb this impact better, but will nevertheless be abraded (from rolling, sliding, impact) under the action of the balls. This explains why most of the aggregate types seemed to perform similarly. Note that these observations apply only to 'deep abrasion' as course aggregate plays virtually no part in 'surface abrasion'. See also **conclusion 1** at the end of this section for a discussion of the importance of toughness relative to hardness].

Connell(1985) found no sensible correlation between the abrasion resistance of concrete and the impact value of the corresponding aggregate. The abrasion tests were done in accordance with BS 812:Part 3:1975 clause 9 [=*sliding fine-abrasive*, see appendix U.5.06] while the impact value was obtained from the aggregate impact value test (AIV – BS 882:1983). The 20mm coarse aggregates used were all suitable for 'heavy duty concrete for floor finishes', i.e 'Stock Thames Valley Gravel', 'Gritstone', 'Limestone', 'Granite', and 'Quartzite Gravel'. Corresponding concrete strengths in the test specimens were in the region of 50MPa.

[The explanation for the poor correlation is threefold. Firstly, the BS 812 test operates on the Böhme principle, and may be classified as a mild abrasion test, practically free of any impact, whereas the AIV test is most definitely a measure of impact resistance (see **conclusion 1**). Secondly, the former test measures the combined abrasion resistance of the mortar and the aggregate, while the latter considers only the aggregate. Thirdly, the influence of aggregate type on the abrasion resistance of high strength concrete is relatively much less than for low to medium strength concretes (see **conclusion 3**)].

Prior(1966) made reference to Collins and Waters who found that the abrasion resistance of concrete was not necessarily related to the 'abrasion' resistance of the aggregate as determined by the LA abrasion machine. [See **conclusion 1** at the end of this section for a discussion of the importance of toughness relative to hardness].

They also found that aggregate type was important for weak to medium range concretes (14MPa to 28MPa), and conversely not important for compressive strengths above 42 MPa. [See **conclusion 2** at the end of this section for a discussion of the importance with respect to abrasion resistance of coarse aggregate type in low/medium strength concretes, and conversely see **conclusion 3** for a discussion of the relative unimportance of coarse aggregate type in high strength concretes].

Smith(1958) did abrasion tests using three abrasion tests, (1) *rolling steel balls*, see appendix U.2.11, (2) *impacting fine abrasive*, see appendix U.5.20, (3) *rolling dressing wheels over fine abrasive*, see appendix U.3.08. He reports mainly on the steel ball test results, showing that for *high strength* concretes with low w/c ratios, abrasion resistance was virtually independent of the hardness of the aggregate, and this applied to both the course and fine aggregate. [See **conclusion 3** at the end of this section for a discussion of the relative unimportance of coarse aggregate type on the abrasion resistance of *high strength* concretes]. More specifically, the considerable differences obtained in the LA abrasion machine for the various coarse aggregates had an insignificant effect on abrasion resistance (see **conclusion 1**), unless the aggregate was substantially softer, i.e. the limestone was substantially softer than the basalt, granite and quartz, and therefore had high abrasion-wear relative to the Los Angeles Abrasion value.

On the other hand Smith showed that the hardness of the aggregate strongly influenced abrasion resistance for weak to medium strength concretes (i.e. between 20MPa and 35MPa), and in this instance the coarse aggregate had the greater influence. [See **conclusion 2** at the end of this section for a discussion of the importance of coarse aggregate type in low/medium strength concretes].

Olorunsogo(1999) did abrasion tests in accordance with PCI.TM.7.11(wirebrush) [=*sliding wire bristles*, see appendix U.6.02] to determine abrasion resistance of 30 MPa concrete made from: (1) natural fine aggregate and various combinations of recycled coarse aggregate (RA) 'processed to 26,5mm', and (2) natural 26mm coarse aggregate (NA). NA was substituted with RA as follows 0%, 30%, 50%, 70%, 100%. Note that the same mortar was used for all the test specimens. He found that the depth of penetration of the brush was not a function of the type of coarse aggregate.

[This may appear to contradict **conclusion 2**, which states that aggregate type *is* important in low/medium strength concretes. The reason it *is not* in this case is because the sliding wire bristles test is a relatively mild test. Crushing and impact effects are insignificant in this test. Moreover according to Addis(1989) in high strength mortars the relatively flexible bristles hardly scratch the surface. Therefore assuming that both the NA and the RA were at least as hard as a high strength mortar, the test would not be expected to yield different results for the two aggregate types, nor the corresponding concrete specimens, since the mortar was a constant. Furthermore the aggregate would substantially protect the mortar.

On the other hand a consistent trend was observable in the compressive strength results whereby increased substitution with RA resulted in lower compressive strengths (e.g. 100% RA mix was approx. 14% lower than 100% NA mix). Given that the mortar was the same for all specimens, this reduction in compressive strength can most probably be attributed to a reduction in coarse-aggregate/paste bond. Such a bond reduction would not be a factor in the abrasion tests, where the maximum depth of wear recorded of 2,26mm would not dislodge the 26mm aggregate particles, nor would the relatively hard aggregate particles themselves have any appreciable wear.

These arguments explain why the different aggregate types did not in this case affect the abrasion resistance of what amounted to medium strength concretes.

Gjorv(1990) did abrasion tests using truck tyres with steel studs (=*rolling studded tyres*, see appendix 3.03) and found that their was no clear relationship between *abrasion* resistance and aggregate type for very high strength concretes of 150 MPa. However at strength levels of between 50MPa to 100MPa, concretes incorporating quartzdiorite were inferior to those of equivalent concrete made of hornfels, syeniteporphyr and jasper. (Note that some of the busy roads in Norway are so rutted after 2 years that they require resurfacing, hence the search for better wearing surfaces).

Normally the abrasion behaviour of concretes in the region of 50MPa and higher may be described as per conclusion 3, while weaker/medium concretes (20MPa/35MPa) behave according to conclusion 2. However, it is apparent that for a very severe test the transition between the two types of behaviour will move upwards. The steel studs beneath a loaded truck wheel will grind and crush all but the hardest surfaces, and this explains why the 50 to 100 MPa concretes are sensitive to aggregate type (**conclusion 2** behaviour), while the 150 MPa concrete is not sensitive to aggregate type **(conclusion 3)**. This is further explained in **conclusion 4**.

Position 2 conclusions

Conclusion 1: Toughness is important in impact tests

Hutchings(1992) speaks of the onset of lateral cracks in brittle materials developing when the normal load exceeds a critical value w*. The value of w* depends on the fracture toughness of the material, Kc, and on its hardness, H. According to one theory:

$w^* \propto (K_c/H)^3.K_c$

where H is the Vickers indentation hardness, and K_c is usually taken to be the mode I plane strain fracture toughness, K_{Ic}, as measured in conventional notched tensile or bending tests.

It may be seen from this relationship that cracking is very sensitive to both toughness and hardness, but whereas resistance to cracking will dramatically increase with increase in toughness (i.e. 4th power), it also reduces very rapidly with increase in hardness (3rd power). It may therefore be said that aggregates that are tough, and at the same time not too hard, will perform well in aggregate impact tests such as the LA abrasion machine. Conversely, aggregates that are very hard but not tough will crack and break down very rapidly. It follows that concretes made with these aggregates will perform in like fashion when subject to abrasion tests with severe impact, such as the Talbot Jones Rattler test.

Conclusion 2: Aggregate type is important in *low/medium strength concrete* (concrete with relatively weak mortar).

A concentrated load like the heel of a shoe placed upon a grain of sand situated on a particular coarse-aggregate particle does not induce very high bond stresses between the coarse aggregate particle and the surrounding mortar, so no debonding occurs. However the same concentrated load applied to a small fine aggregate particle is likely to result in very high fine-aggregate/paste bond stresses that may lead to bond failure. Although the magnitude of the load and the contact area of the load is the same in both cases, the bond interface is very much smaller in the case of the fine aggregate particle, leading to relatively high bond stresses. For example a man weighing 80kg stepping on a grain of sand as his heel takes his weight applies a force of approximately 800N. If the sand grain is situated on a fine-aggregate particle that has a surface contact with surrounding paste of 8mm^2, then the applied stress amounts to 100 MPa. This is likely to lead to fracturing effects in the relatively weak interfacial zone, leading in time to a complete bond failure, followed by a loss of the fine aggregate particle as abrasive forces pry the debonded particle loose.

A second mechanism of wear in the mortar constituent will be direct attack on the paste, given that a certain percentage of the mortar is paste. Clearly weak pastes are far more susceptible to scratching and crushing effects from sand particles sliding or rolling over the surface under pressure.

By the two mechanisms described above the inferior mortar is abraded rapidly, until the coarse aggregate is prominent relative to the mortar. The coarse aggregate now provides a significant measure of protection against abrasive forces. Clearly the much larger contact area between coarse-aggregate and mortar leads to relatively small bond stresses compared to that between paste and fine aggregate. In this situation, both the hardness and toughness of the substantially exposed coarse aggregate is important, the former for normal abrasion type loads, the latter where impact is a factor. Hence aggregate type is important where mortar strength is inferior.

Conclusion 3: Aggregate type is less important in *high strength concrete*

It should not be inferred from this that the paste associated with a high compressive strength is extremely abrasion resistant. Addis(1989) showed that, compared to aggregate, it is still relatively soft, although clearly, the paste of a 60MPa concrete *is* substantially harder than that of a 20MPa concrete. However, the principal reason for improved abrasion resistance is not the improved abrasion resistance of the paste but the higher paste/aggregate bond. In other words there is point where the substantial bonding capability of the high strength paste allows the relatively high abrasion resistance of the fine aggregate to resist the abrasion loads, without being debonded and then plucked out. In this case the hardness of the coarse aggregate is far less important. No longer is material (fine aggregate) being pulled out, resulting in the demise of the surrounding paste, and a rapid loss of mortar relative to coarse aggregate. No longer is the coarse aggregate left to stand out and face the abrasion attack on its own. In fact, a degree of reverse protection may take place, particularly where a hard siliceous fine aggregate is used in conjunction with a softer calcareous coarse aggregate. In terms of Mohs hardness, it may be said that the abrading medium, on a road or sidewalk, say a 10 mm siliceous pebble under load, in effect rolls/scrapes over the surface resulting in minimal damage to the mortar consisting of strongly bonded fine aggregate particles of similar hardness. [Hutchings(1992) has shown that wear only becomes significant when one material is more than 1,2 times harder than the other].

Conclusion 4: The <u>transition</u> between the differing behaviours described in 2 and 3 above will depend on the severity of the abrasive attack. The greater the crushing/impact/gouging effects the higher will be the transition point where aggregate type plays a minor role in abrasion resistance.

(c) Position 3 : The hardness of the coarse aggregate is of no concern

It is clear from the literature (or it may be deduced) that there are instances where the coarse aggregate does not contribute to abrasion resistance. However it soon becomes apparent that in these instances the abrasion tests all generally focus on the top 1 mm, or less. Given that the coarse aggregate generally undergoes some settlement induced segregation in concretes made with a positive slump, as well as some suppression by the action of the power float, in these instances there is virtually no coarse aggregate in the top 5mm to 20mm, and this explains what would otherwise seem to contradict some of statements made earlier in positions (a) and (b).

C&CaofNZ(1997) have produced a guide based on abrasion tests done using an apparatus developed by the C&CA [=*rolling steel wheels*, see appendix U.4.06]. The document states that for industrial floors <u>the hardness of the coarse aggregate only becomes significant under exceptionally abrasive conditions</u> i.e. when the surface matrix has been worn away. Although coarse aggregates should be free from soft sandstones or limestones, they need only be specially selected for conditions of exceptionally heavy wear.

TR34(1994) Power finishing has the effect of pressing the coarse aggregate down slightly, and creating a very hard surface skin of binder rich mortar. It is evident that abrasion-wear in this surface layer should be minimal to limit dusting, which is most undesirable in industrial warehouses. TR34 therefore considers a floor to have failed if it has worn as much as one millimeter, and certainly any surface that has worn as much will release increasing volumes of dust when trafficked. It is therefore evident that <u>the hardness of the deeply submerged coarse aggregate in these applications is largely irrelevant</u>.

Webb(1996) did abrasion testing using an apparatus developed by the C&CA [=*rolling steel wheels*, see appendix U.4.06] to test the abrasion resistance of concretes made of various coarse and fine magnesian limestone aggregates, and found that <u>the different coarse aggregates tested did not have a significant influence on the abrasion resistance</u> of a power finished floor, explaining that the slight penetration of the steel wheels of 0,2mm or less did not reach down to the coarse aggregate. He concluded that concretes incorporating magnesian limestone coarse aggregates are suitable for power finished concrete floors.

Scripture(1954) did abrasion tests using *sliding fine-abrasive* beneath revolving discs [see appendix U.5.14] and found that <u>abrasion resistance was not a function of the hardness of the coarse aggregate.</u>

[The depth of penetration that Scripture abraded to was generally of the order of 0,3mm, and at this depth it is unlikely that the testing apparatus would have engaged much coarse aggregate. Furthermore, he used a slump of one to two inches, and this would also have resulted in a degree of settlement of the coarse aggregate].

Wu(1999) reviewed the five test procedures currently in use in USA to evaluate abrasion / impact capability of aggregate for surfacing roads:

- ASTM 131. Resistance to Degradation of Small Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (LA)
- British Standard 812:Part 3.6 Aggregate Impact Value (AIV)
- British Standard 812:Part 3.7 Aggregate Crushing Value (ACV)
- British Standard 812:Part 3.9 Aggregate Abrasion Value (AAV)
- Ontorio Test Method: LS-618 Micro-Deval Abrasion Test

[Aside: The rationale for using impact tests for evaluating the abrasion resistance of a surface that is to be subjected to rubber wheeled traffic should however be questioned. A wheel has almost no velocity in the vertical direction at the point that it makes contact with the ground (if it did not have a slight flat where it makes contact with the ground, it would have zero velocity at the point of contact) and hence virtually no impact is imparted to the surface (see chapter 3). Jackson(1924) showed that rubber lined wheels resulted in virtually no abrasion wear].

Sixteen aggregate sources were tested including carbonates, granites, traprock, siltstone. sandstone, basalt and slag. The greatest wear was obtained with S Carolina Granite, with LA = 49%, AIV = 32%, ACV = 33%. However, even the roads constructed with this aggregate were considered by the authors to have acceptable wear characteristics, indicating that <u>the abrasion resistance of a concrete road is not meaningfully related to aggregate impact tests</u>, and even the aggregate abrasion value (AAV) has no relevance if the coarse aggregate is somewhat submerged beneath a thin skin of very hard wearing mortar.

Position 3 conclusion

Some surfaces have virtually no coarse aggregate in the first few millimetres. Clearly, in these instances coarse aggregates plays no role in the abrasion resistance of the topmost surface. In this case a clear distinction should be made between ‘shallow’ and ‘deep’ abrasion resistance.

The only time that coarse aggregate is likely to play a role in abrasion resistance, is where it is used as the wearing surface of an *asphalt* road – in well designed and constructed concrete roads the coarse aggregate will be sufficiently submerged so as not to come into contact with traffic.

(d) Overall Conclusion to Type and Hardness

The hardness of the coarse aggregate is only relevant where the depth of the penetration of the abrading medium brings this aggregate into play. This understood, severe abrasion tests will correlate well with the hardness of the coarse aggregate.

If mild abrasion tests are used, this will require special preparation of the test specimens if a concrete assessment of the coarse aggregate’s hardness is required.

Hardness of coarse aggregate is more relevant in weak/medium strength concretes, where deeper penetrations are possible. Where abrasion is accompanied by impact, aggregate should be both hard and tough.

2.4.2.2 Grading

The grading of the coarse aggregate does influence the w/b to a degree, and hence abrasion resistance. This was considered in 2.2.1.1.1.2(a) and will therefore not be repeated here.

Not much else has been said regarding the effect of grading on abrasion resistance. The findings of three authors will be considered below, followed by the conclusion.

Tveter(1994) stated that whereas traditionally gap graded mixes were recommended for road paving, there was no indication in his abrasion tests (rolling studded tyres, see appendix U3.03) that gap graded mixes had superior abrasion resistance.

Helland(1991) reported on the use of high strength concrete (HSC) for concrete road and bridge surfaces in Norway, which had to have exceptionally high abrasion resistance to resist the very abrasive action of steel studded tyres on vehicles. In order to utilize the hardwearing characteristics of the coarse aggregate, <u>a continuously graded mix was used rather than a gap graded mix</u>, which (according to Helland) results in segregation. It may be inferred from this that the coarse aggregate of a continuously graded mix is relatively near the surface .

Komonen(1998) did abrasion tests on concrete *pavers* using rolling studded tyres [see appendix U.3.04, =*rolling studded tyres*] and found that gap graded mixes lost the mortar constituent until the coarse aggregate was sufficiently prominent and afforded a measure of protection. Thereafter <u>the abrasion resistance of the pavement was related to the wear resistance of the coarse aggregate</u>.

Abrams(1921) found that 'grading of the coarse aggregate may vary over a wide range without materially affecting the wear'.

Sectional conclusion

From the comments made by Helland and Komonen it is evident that coarse aggregate is generally harder than mortar. Therefore to gain most from the hardness of the coarse aggregate, continuously graded mixes, which settle less than gap graded mixes, should be used in concrete that is made to a slump. This however has no relevance to power finished industrial floors where the coarse aggregate is mechanically suppressed and abrasion is generally limited to the top 1mm.

The proportion of coarse aggregate should be maximised in semi-dry concrete (given that it is not abraded as easily as the mortar), providing this does not adversely affect the surface texture.

Adam's finding relates to a very severe form of wear. Slight variations in surface mortar thickness would be disguised by the severity of the wear. Of more importance would be the overall proportion of the relatively hard coarse aggregate, rather than its grading.

2.4.2.3 Size

The findings of various authors are given below, followed by a conclusion.

Dihr(1991b) did abrasion tests with the *rolling steel wheels* of the C&CA machine and found that 10mm and 20 mm aggregate gave improved abrasion resistance relative to 5mm, all at a w/b of 0,55. The wear of the 5mm aggregate was more than 50% greater than that of the two larger sizes.

Doulgeris(1996) did abrasion tests according to PCI.TM.7.8 [=25mm *impacting steel balls*, see appendix 2.02] and found that concrete made of a graded 19mm aggregate performed slightly better than the equivalent with 13mm aggregate. He attributes this to better embedment on the part of the larger size. This finding was more noticeable in mixes of relatively low compressive strength.

Jackson(1924) did abrasion tests using *rolling chained tyres* (see appendix U.3.12) and found that the abrasion resistance of concretes made with ¾' slag aggregate was greater than that made with 1½' material, which in turn exceeded that of concrete made with 2½' slag aggregate.

He found that the abrasion resistance of normal aggregates in the ¾'– ½' range was marginally more than that in the ½'- ¼' range.

Lane(1978) referred to ACI Floors (302) to state that the abrasion resistance of concrete was superior to that of mortar. A grading of 19mm to 39mm was suitable for light traffic, but 10mm to 13mm was preferable for heavy traffic.

Komonen(1998) did abrasion tests on concrete pavers using *rolling studded tyres* (see appendix U.3.05) and found that a 'good surface quality' could be achieved with aggregate as large as 16mm. [However, in the writer's experience, such large aggregate leads to a rougher surface texture, and damage to the mould on final return of the feed wagon].

Sectional conclusion

Two opposing principles come into effect here, which may be stated as:

(a) Large aggregate particles are anchored relatively deeply into the mortar, and are therefore not easily plucked out. However this does not always imply increasing abrasion resistance with increasing aggregate size. The very large aggregate sizes will also have relatively large open mortar filled spaces in between them, that have the potential to abrade relatively easily.

(b) Small aggregate particles have a greater surface area relative to their bulk, and therefore have more paste surrounding them to anchor them (relative to their bulk). They also yield a more homogenous concrete. If the size is reduced by too much however, then they can be plucked out of the matrix relatively easily.

The ideal size in any given application will depend on the mode of attack. Large protruding aggregate particles may not always be advantageous. Such protrusions may be very vulnerable to impact type abrasion. On the other hand large particles are well anchored and 'protect the weaker mortar constituent'.

It appears from the authors quoted above that 13mm to 19mm has worked well in their experience, but clearly tests will be required that simulate the actual type of abrasion before a final decision can be made on the best size.

2.4.2.4 Volume Fraction Relative to Sand

A high percentage of coarse aggregate provides a considerable measure of protection for the softer mortar/paste against abrasive forces. The findings of the following investigators confirm this, with one exception. (The only exception is Ghafoori:1995 who found that the surface of the paver, without coarse aggregate, was more abrasion resistant to a test face midway down with coarse aggregate. An explanation is given).

Abrams(1921) did tests with large *impacting steel balls* (very severe, see appendix U2.01) and found that increasing the fineness modulus of the aggregate in the mix (i.e. this amounts to a <u>increase in coarse aggregate content of the concrete) increased abrasion resistance</u>. This was true up to a fineness modulus of 6, after which further increases in fineness modulus resulted in a reduction in abrasion resistance (possibly because of voids increasing).

Crepps(1920) used a modified Talbot-Jones rattler (=impacting steel balls) and found that abrasion wear decreased with increase in fineness modulus (defined as 1/100 x sum of the mass of percents retained on the sieves, including coarse aggregate). [This amounts to an increase in abrasion resistance with increase in coarse aggregate proportion].

Doulgeris(1996) states that if the paste strength is sufficient to firmly embed / bond the coarser aggregate particles within the matrix, and especially if these particles are spaced relatively close together, then wear of the surface concrete can only proceed if the aggregate itself is worn away. Clearly, increasing the proportion of hard aggregate (consistent with full compaction) increases abrasion resistance.

Ghafoori(1995) did abrasion tests according to ASTM C779 Proc C [=*rolling steel balls,* see appendix U.2.12] and found abrasion resistance of pavers tested at their surface to be greater than sawed pavers tested at their centres, and that this was related to the presence of coarse aggregate at the centre.

He explains that there was a greater proportion of siliceous sand/paste at the surface, as a result of the vibration. (The blocks were vibrated for 18 seconds, which is excessive, and this would indeed have resulted in a higher proportion of siliceous sand/paste at the surface). On the other hand the blocks tested at their centres had a proportion of relatively soft limestone coarse aggregate. Therefore increasing the proportion of a relatively soft coarse aggregate reduces abrasion resistance.

Horiguchi(1995) found substantial reductions in wear (*impacting steel balls* in liquid, see appendix U.2.06) as the coarse aggregate : fine aggregate ratio was increased from 50:50 to 70:30. (This trend was consistent for plain concrete, as well as concretes with 15% additions of silica fume and polymer respectively.)

Horiguchi(1994) found that the above trend was reversed when testing with rolling steel balls (appendix U.2.12), while the abrasion resistance seemed to be independent of the s/a ratio (the ratio of the fine aggregate to the total aggregate) when tests were made using rolling dressing wheels. It appears that s/a ratio is a test sensitive characteristic. Horiguchi does not offer an explanation for this.

Komonen(1998) made concrete pavers for abrasion testing and found that a ‘good surface quality’ could be achieved with as much as 70% of the course aggregate exceeding 8mm and 20% exceeding 16mm. His abrasion tests were made with a studded tyre machine, see appendix U.3.05. He found that initially there was a high rate of abrasion, but that this soon reduced as penetration increased. He explains that this reduction in wear is due to a higher proportion of course aggregate at deeper levels, relative to the higher proportion of fine-aggregate/paste in the immediate vicinity of the surface. He refers to work carried out by Lampinen(1988) stating that it has been well established that the abrasion resistance of concrete depends on its compressive strength and the *volume* and hardness of the course aggregate.

[Komonen’s argument is logical considering that fine aggregate which predominates near the surface is plucked out relatively easily, and furthermore the high concentration of mortar at the surface is relatively soft compared to the concrete further down which is made more wear resistant by a higher total aggregate percentage. However, the coarse aggregate comes into play very soon, since a semi-dry mix has a great deal of internal friction that makes it difficult for the course aggregate to segregate and sink relative to the lighter mortar. Furthermore geometry dictates that rounded and irregular shaped course aggregate particles will have a low percentage of their mass at the immediate surface, a situation that soon changes as abrasion proceeds.

It is however possible that the high rate of early penetration that he reported should partly be attributed to surface drying from poor curing].

Omata(1997) did abrasion tests with pneumatic rolling studded tyres made to roll over specimens, and found that the initial rate of wear of methyl methacrylate (MMA) resin mortar and concrete was far greater than later on. He attributes this to a degree of segregation caused by some settlement of the relatively hard aggregate, at the time of construction, resulting in a layer of the relatively softer resin at the top. [Clearly a hard aggregate in a relatively soft matrix has greater abrasion resistance than the matrix by itself].

Ozturan(1987) found that different aggregates had different critical volume fractions, V_{cr}. V_{cr} is very close to the compactability of the coarse aggregate. Any further increase in volume of coarse aggregate above Vcr results in voids. V_{cr} for granite was found to be 0,50 while V_{cr} for gravel was 0,62. Since the presence of voids seriously reduces the strength of concrete (and abrasion resistance), abrasion wear will increase dramatically if the ratio of coarse aggregate to fine aggregate is too high.

Murakami(1998) did abrasion wear testing using a rolling studded wheel (see appendix U.3.05) and found that the ideal ratio s/a, representing the proportion of fine aggregate (sand) relative to total aggregate was 30% to 40%, where the coarse aggregate was a 50:50 blend of 5mm to 13mm and 2,5mm to 5mm. Outside these parameters wear increased steeply on both sides. On the one hand, reducing the fine aggregate too much results in a decrease in workability, tending to increased voids. Conversely too much fine aggregate means that w/c must be increased to maintain workability.

Ozturan(1987) found that concretes with different coarse aggregate types had different abrasion resistances, as determined by DIN 52108 [=*sliding fine abrasive*, see appendix U.5.02]. In descending order these were found to be gravel, granite, limestone. The abrasion resistances of the corresponding concretes were directly proportional to both the abrasion resistance of the coarse aggregate and the mortar, depending on the volume fractions used. It may therefore be said that the rate that abrasion resistance increases with increasing coarse aggregate proportion depends on the type of coarse aggregate.

Jackson(1924) did abrasion tests using rolling chained tyres (see appendix U.3.02) and found that the proportion of coarse aggregate relative to fine aggregate could be increased with minimal reduction in abrasion resistance even though this implied a lower cement content. He obtained very similar abrasion wear for mixes proportioned 1 : 2 : 3 and 1 : 2 : 4.

Papenfus(1993a) made wear measurements on concrete pavers that had experienced various degrees of abrasion. The wear patterns show that as abrasion wear increases, the coarse aggregate plays an increasingly important role in abrasion resistance. This indicates that abrasion wear will be reduced as the proportion of coarse aggregate increases, providing compactability is not compromised, and providing that it is harder than the mortar.

Simons(1992) did abrasion tests with a bouncing rotating drum covered with abrasive (=impacting fine-abrasive, see appendix U.5.18). He found that specimens with higher w/b ratios of 0,40 were abraded at a greater rate relative to those with w/b = 0,30, but only to a certain depth. Thereafter the two mixes abraded at approximately the same rates.

[The most likely explanation is that the fine aggregate is more easily dislodged in the higher w/b mixes, whereas the increased bonding capability of the low w/b mixes reduces this tendency (i.e. in the low w/b mixes the fine aggregate contributes to wear resistance rather than being removed).

The fact that at greater depth the rates of abrasion are the same for both w/b ratios may be explained by the greater contribution of the coarse aggregate at this depth. Owing to its uneven shape and some settlement, it is only fully in contact with the abrading drum at greater depths. However, with further abrasion wear full contact with the coarse aggregate is eventually made. Effectively, the 'increased' proportion of coarse aggregate relative to mortar at this depth increases abrasion resistance, thus accounting for the same rates of abrasion for the higher and lower w/b ratios. Owing to the greater contact area of the relatively large coarse aggregate with surrounding paste, the bond stresses are sufficiently low to prevent these particles from being dislodged, even for the w/b = 0,4 mix].

Suda(2000) did abrasion tests on concrete pavers using rolling studded tyres (see appendix U.3.05) and found that abrasion-wear reduced with increasing fineness modulus. Pavers made with an aggregate having a fineness modulus of 2,0 had 50% greater wear relative to where the aggregate had a fineness modulus of 4,5. Maxmum particle sizes considered in the respective mixes were 13mm, 10mm, 5mm, 2.5mm, and 1.2mm. Suda concluded that the bigger the aggregate size is, the better is the wearing resistance. [This may be expected since for a given abrasive load, the shear stresses at the aggregate paste interface will be lower as the aggregate particles increase in size].

Siro(1991) did wear tests on concrete using revolving *rolling steel wheels* under load (see appendix U.4.01). The first 50 revolutions typically resulted in 0,5 mm of wear, but by 1000 revolutions the wear rate had stabilized to 0,05mm/50 revolutions.

[While this may partly be attributed to better paste quality once a depth is reached where the effects of bleeding and poor curing are minimal, the contribution of the relatively hard coarse aggregate, fully engaged at greater depth, is probably the main reason for improved abrasion resistance].

Sectional conclusion

The improvement in abrasion resistance with increasing coarse aggregate proportion relative to the mortar constituent may be attributed to several factors:

- it increases the hardness of the concrete, depending on the hardness of the aggregate
- it generally results in a lower water demand and hence allows a decrease in w/b
- the larger particles are better imbedded

It may be concluded that increasing the proportion of coarse aggregate increases abrasion resistance, subject to three considerations:

(a) There is a point beyond which further increases in coarse aggregate will result in a reduction in abrasion resistance. This point is related to the volume of the voids between the packed aggregate particles. Once the mortar can no longer fill them, abrasion resistance will decrease.
(b) If the coarse aggregate is substantially softer than the fine aggregate, then increasing its proportion may result in a reduction in abrasion resistance.
(c) In some cases the abrasion wear is more related to the type of test than the relative proportion of fine aggregate and coarse aggregate. It should however be observed that in these instances the 'coarse' aggregate was relatively small (maximum size 13mm), as is dictated by the paving manufacturing process.

Conclusion to Coarse Aggregate

Of the various attributes considered in this section hardness, grading, size and proportion-relative-to-mortar, the hardness is the most important. This being the case, it follows that the proportion relative to the mortar should be maximised.

However, there are many applications where the coarse aggregate is submerged, and moreover abrasion is unlikely to ever reach such depths - clearly in these cases the hardness of the coarse aggregate is of no concern.