Ken W. Day

Concrete Mix Design, Quality Control and Specification

3rd Edition

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Third Edition

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Acknowledgements

My third edition builds upon the shoulders of the work done for the first two and I do not wish those I thanked then to be forgotten now. Therefore the acknowledgements in the second edition are reprinted in full following those for the current edition.

My company Concrete Advice Pty Ltd was sold in 2001 to Maricopa Readymix, my first US client, at the instigation of Dave Hudder, at that time Managing Director of Maricopa. I have him to thank for his recognition of the value of ConAd in USA and for providing me with the means to enjoy my semi-retirement and to travel the world preaching my concepts.

On Dave leaving Maricopa, Concrete Advice was on-sold to Command Alkon. I was very pleased about this because ConAd is a perfect fit for a major, worldwide, batching system provider. I thank them for continuing my part-time consultancy until the end of 2004, even though I have had little influence on the new version of ConAd.

I will never forget the part played by Don Bain, Technical Manager of Maricopa, in all this. It was he who recommended the initial purchase of the ConAd system to Dave Hudder back in early 2000, he who used ConAd to enable the expansion of Maricopa and build the US reputation of ConAd, and he who left Maricopa for a time to help Command Alkon with initial marketing of ConAd. His written contribution to this text is appreciated, but it is negligible compared to his contribution to the reputation of the ConAd system.

Andrew Travers continues to labour prodigiously as CEO of ConAd. Its future now depends on him as he rushes around the world promoting and installing it. Unfortunately he has been far too busy to write a section of this book, much as he wanted to, and much as it would have been appreciated. Perhaps he will write the next edition.

Two other stalwarts, having contributed greatly, are no longer able to do so. Dan Leacy, the Australian equivalent of Don Bain, unfortunately passed away at an early age and Michael Shallard retired at an even earlier age after a severe illness, depriving the system of its major source of computer expertise. I shall remember them.

My email directory overflows with large numbers of people substituting for my lack of field experience in recent years. Several names appear as contributing sections of the text: Dr Alex Leshchinsky and his father Dr Marat Lesinskij, Mark Mackenzie, Dr Norwood Harrison, Dr Grant Lukey, John Harrison and Tracy Goldsworthy – and Dr Joe Dewar whose contribution to the previous edition is repeated here.

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Justin Smyth (delphian@smythconsulting.net) operates my website (www.kenday.id.au) and has amended the free programs on that site.

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There are three individuals without whom this book could not have happened and four more without whom it may have been very different. The first group comprises: O Jan Masterman, Technical Director, Unit Construction Co, London in the 1950s, who somehow inspired and guided me to originate in my first two years of employment the greater part of the philosophy and concepts herein recorded; John J. Peyton, John Connell & Associates (now Connell Wagner), Melbourne, without whose encouragement I would never have started my company Concrete Advice Pty Ltd in 1973 and so the nascent control techniques would never have developed to fruition; John Wallis, formerly Singapore Director of Raymond International (of Houston, Texas) without whom my Singapore venture would have foundered in 1980, leaving me without computerization and without the broad international proving grounds for the mix design system.

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A third kind of indebtedness is to those who assisted in the actual production of the book. They have become too numerous to list all of them by name but Hasan Ay and Andrew Travers are especially thanked for their work on figures and tables.

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A new kind of indebtedness is to those individuals in my major client companies who have not only enabled my company (Concrete Advice Pty Ltd) to survive and prosper but have also contributed in no small measure to improvements in the system. They include Peter Denham and Dan Leacy of CSR Readymix, Paul Moses of Boral and Mark Mackenzie of Alpha, South Africa.

The Conad computer program has come a long way since the first edition and thanks are due to my staff at Concrete Advice. Michael Shallard and Lloyd Smiley wrote the latest program and Andrew Travers, now Manager of the company, knows how to use it better than I.

Finally I must thank my younger son, John Day, now Technical Manager of Pioneer Malaysia, for using these techniques so effectively as to make the world's tallest building, Petronas Towers, the best example yet of low variability, high strength concrete.

Introduction

Between writing this third edition and its actual publication, there have been some possibly quite significant developments in my situation. The assumption during writing the book was that I was fully retired and might contribute further to the future of concrete technology only via this book and my website, if at all. A change arose initially from the intention of Canmet/ACI to award me recognition for my contributions to concrete quality control at their symposium in May 2006. This has led to an arrangement for me to present a whole day seminar in Silver Spring, Washington, in June 2006, under the auspices of NRMCA and others, in an attempt to bring about a change in American practice from quality control by the purchaser to control by the producer, as advocated in this book.

A further development has been an invitation to join a partnership of the Canadian company Contek with the Shilstones (father and son, perhaps the best known names in concrete technology in USA) in an endeavour to produce world-leading software and to market and support it throughout the world.

You, the reader, will have to consult my website www.kenday.id.au for information as to the outcome of the above. I have not made changes in the rest of this introduction or in the text as a whole as a result of these developments but considered that readers should be made aware of them, and may be interested to see my views and intentions prior to their occurrence.

Original introduction

In this third and final edition my objectives differ from those of the previous editions.

Rather than promoting the commercially available program ‘ConAd’, I am now concerned to spread my developments as widely as possible in the world and in the concrete industry and professions and also to look as far as possible into the future of mix design and quality control. This is very much *not* a handbook on how to comply with established standards but rather a view on how those standards, and other entrenched attitudes, should be revised to more nearly reflect reality and enable progress.

I have a different attitude to the three topics of mix design, quality control and specification:

Of my Specific Surface Mix Design I say it has worked well for me for the last 50 years, and is simpler and more versatile than anything else I know of, but there are alternatives that are at least as accurate and not too much harder to use.

Of Multigrade, Multivariable, Cusum Quality Control I claim this to be the ultimate in QC. It is simple to use and more powerful than any alternatives. The whole world will eventually use such a technique with little if any modification.

Of Specification I say the battle is essentially won. There are still substantial numbers of people specifying minimum cement contents and the like, but they are just the last old, ill-informed diehards. There is just the one aspect of cash penalty specifications that does not seem acceptable to others, and yet in my opinion is an invaluable tool that must be adopted before the supply of concrete can be regulated with complete equity.

The future of the ConAd program is now in the hands of others and will hopefully achieve widespread acceptance amongst those major players who do not steal the concepts and write their own similar software.

My own attempt to spread my developments to even the smallest and least advanced producer rests on this book and the free programs on my website www.kenday.id.au. This website will hopefully enable readers to be kept up to date with new developments for years to come, perhaps even by others after my own final full retirement.

In looking to the future, self-compacting concrete, cement-replacement materials, crusher fines and geopolymer concrete are the main items to address, along with just-in-time mix design, early age maturity, portable rheometry and internet order placing. As ever in the concrete industry, what will be commonplace in 50 or 100 years time is probably already here, but struggling to achieve acceptance – the likely exception being improved chemical admixtures.

A new crusade is the advocacy of a requirement that concrete specifications should only be written by, or under the guidance of, qualified concrete technologists. This is a reaction to the increasing complexity of the material and its production and control process and the realization of how much progress has been delayed, especially in the USA, by antiquated and deleterious specifications.

Having belaboured the US concrete industry for technological lag (caused by inept specifications) for many years, I now have to admit that double tee units of almost 200 MPa, self-compacting, fibre-reinforced concrete are being made there (in one area) and are well in advance of anything with which I have personally been involved.

Ken Day
Nunawading (Melbourne), Australia

Chapter 1

Advice to specifiers

Concrete technology is in a period of rapid development. The use of self-compacting concrete is increasing, strengths are increasing, and knowledge of the factors affecting durability is increasing. Admixtures and cement replacement materials are undergoing rapid development. Natural sand supplies are being exhausted in many areas, focussing more attention on crusher fines ('manufactured sands').

Computers now have an essentially infinite capacity to store data and programs, permitting far closer control of the production process, including mix design, quality control and production control.

Even specialist concrete technologists have difficulty in keeping up with all developments, having to specialize in selected aspects and maintain a network of contacts to supplement their own knowledge.

In the first two editions of this book, the chapter on specification was placed after those on mix design and quality control. This was on the philosophy that it is necessary to understand the production process before being in a position to regulate it. This is now seen to be a flawed philosophy. Most specification writers are not going to learn enough about mix design and quality control to put them in a position to have a valid opinion as to how the production process should be regulated – and history teaches that a little knowledge is a dangerous thing. What is required is to advise specification writers how to get what they want (and what it is that they should want); and to convince them that the advice given is correct. It may also be necessary to convince them of the extent to which inappropriate specifications in the past have delayed the introduction of more effective control; and favoured unscrupulous or ignorant concrete producers over those who are competent and genuinely wish to produce good concrete.

1.1 Mix selection

Concrete has now become a potentially high-tech material capable of very high strength, high durability and excellent appearance and of being easy to pump and even self-compacting. It is perhaps in some danger of losing its other traditional desirable property of being comparatively inexpensive. The specifier needs to consider very carefully which of these properties are needed, and to what extent.

This consideration should be assisted by discussion with an expert – if you knew what was the best answer yesterday, that does not necessarily mean it is still the best answer today. Can I really have 100 MPa? Will it be excessively expensive? Will it generate too much heat? Will 20 MPa concrete give me a good warehouse floor? What sort of concrete do I need to last 100 years? Can concrete really be made without cement? I just want to be sure I have 20 MPa, how do I make sure I get it without exceeding the minimum cost?

A word of caution is that a completely independent 'expert' may nevertheless not be unbiased. It would not be surprising, especially if a fee is involved, for such a person to feel a pressure to recommend some restrictions on the concrete to be supplied rather than saying 'what you need is ordinary concrete, just specify the strength grade required'. Especially where substantial quantities of concrete are involved, it may also be worthwhile to solicit the advice of one or more potential suppliers of the concrete. This is particularly the case in countries where there is good overall regulation of the industry, for example in Australia, where the Standards Association provides such regulation, and in the UK, where the combination of EN 206 and the QSRMC (Quality Scheme for Ready Mixed Concrete) is in operation.

1.2 Quality control

The specifier, having decided – or been advised – what kind of concrete to specify, needs to find a way of ensuring that it is continuously provided throughout the supply period. The old concepts of minimum cement contents and adherence to a fixed set of proportions are absolute anathema. A low w/c ratio still means better concrete in most respects but it is now clear that more water is more deleterious than less cement. So, at a given w/c ratio (equals a given strength), the mix with the lowest cement content is the best concrete, since it has the lowest total water content.

It is clear that concrete quality is subject both to unavoidable (but reducible) random variation and to unintended changes in average quality from time to time. So it is important to distinguish between the two and to detect any real change (and its cause) at the earliest possible opportunity in order to restore the quality to the specified level. It is easier (and quicker) to detect change when overall variability is low. This is a self-intensifying situation in that quicker rectification of quality changes will reduce overall variability. Modern batching plants may be good at detecting any change in batch quantities of solid ingredients but the problem may not be a result of such changes. It is more likely to be due to either a change in cement or admixture quality or, more likely still, to a change in water quantity. Even the most obvious and well-established fact that water content at a given slump increases as concrete temperature increases, is unknown or disregarded by typical specifications, so that a higher slump concrete on a cold morning may be rejected when a lower strength concrete may be accepted on a warmer afternoon.

We shall see later that strength at any particular age is not necessarily a measure of overall quality. Strength may be needed early for demoulding precast units or for prestressing. Some concretes may gain little strength after 28 days, while the strength of others may double after this age. The use of substantial proportions of fly-ash or blast furnace slag may be highly desirable to improve durability while reducing strength at early ages. What strength will do is to reveal change, whether in w/c ratio or in cement quality. It will also be the best measure of variability and the best measure of testing error. In this respect it is important to realise that test data is not necessarily accurate.

The specifier should also realise that concrete is a variable material for which an absolute limit may be largely inappropriate. There is no exact point, in strength or otherwise, at which concrete suddenly becomes unusable but only a range over which it gradually becomes less acceptable. On the other hand it is necessary to provide a very exact basis for the acceptability of concrete under a contract. This is necessary in fairness to alternative suppliers who failed to obtain the contract because their price allowed for providing a fully acceptable material. It is also necessary in order to avoid a gradual deterioration in quality that may be experienced if there is no retribution for the supply of sub-standard material.

So what a specifier needs to do, after establishing what kind of concrete is required, is to require that the concrete be produced under an effective control system. The situation varies substantially in different countries around the world. There is an international standard (ISO 9001, ISO being the International Standards Organisation) which covers the implementation of quality control in any organization, not only relating to concrete. Many concrete producers in many countries have their ISO 9001 accreditation and, in areas where most are so accredited, it may be appropriate to require that only such producers be permitted to quote.

1.3 ISO 9001

ISO 9001 relates to an administrative procedure for certifying that an organization is operating effective quality control. It requires that the organization produce a 'Quality Manual' detailing its QC system but cannot itself provide such a manual since it is not industry specific. So an organization can have an effective control system without having it certified, and an organization can be certified to be operating its documented QC system efficiently without that necessarily being an ideal system. For example the system might provide for the detection and rectification of non-conformance in the product but it may not do so in the earliest or most effective manner.

The procedure to become ISO 9001 certified is lengthy and detailed and can be quite expensive. It is normal to appoint a QC consultant to assist in the process but it is possible to be guided through it by a textbook or other documentation. For example http://www.the9000store.com provides a great deal of free information on the subject and offers for sale comprehensive documentation supported by enthusiastic customer recommendations.

It is apparent that some small producers, especially ‘Ma and Pa’ single plant producers, may not be able to devote the necessary time and expenditure to attain ISO 9001 certification. Such producers may or may not have an effective QC system. The specifier needs to make a careful choice in the local circumstances. For a major project in an area where certified ISO 9001 plants are offering economical quotations, the answer is fairly obvious. In other circumstances, the specifier should understand that in permitting non-certified suppliers, more care is needed in specifying what is required and it may be desirable to require an inspection of past records and of the control process in operation. However such caution may also not be out of place with certified producers.

1.4 Testing

For the moment we will concentrate on compression testing but we should not forget that there are many other types of test and many important properties other than compressive strength. The reason for the concentration on compressive strength is that it is, in most cases, the most suitable and effective tool for the control of concrete quality – even when compressive strength is not the most important quality to be controlled. What is needed is to detect a change, not only in average concrete quality but also in variability and in testing error – and this is what compression testing normally does better than other tests. For example flexural strength or permeability may be the most important properties for some uses but are less suitable as a control mechanism. Such properties (and tests) may be used to decide the initial mix composition but compressive strength should then be used to detect change, even if, when change is detected and strength restored, it is then necessary to check that other properties are still satisfactory.

Although the preferred option, compression testing is still far from a perfect answer. The best attainable within sample testing error is a standard deviation of around 0.5 MPa and a figure of more than 1.0 MPa is not unusual. (This can be accurately evaluated from the average pair difference of two specimens.) So, leaving aside actual faulty tests, the test result from a single cylinder can be as much as ±/3 MPa from the true strength.

The standard deviation of true concrete strength can range from a little under 2 MPa to more than 6 MPa. So the true strength of a sample of concrete can differ from the mean of the grade in question by more than ±/10 MPa. This is why it is impractical to specify an absolute minimum strength. The solution adopted is to specify a level below which not more than a specified percentage of tests may fall. In most countries the selected percentage is 5% but in USA 10% is used. The use of a higher percentage essentially puts a lower financial value on the attainment of low variability.

The question then is how to establish when the specified limit has been breached. This subject is discussed in more detail in Chapter 4, but comes down to the error involved in small samples and the cost of obtaining sufficient data to detect change in an acceptable time. The author’s technique of multigrade, multivariable, cusum analysis (see chapter on QC) offers significantly faster detection.

An important philosophical aspect to grasp is that there are two quite separate requirements for control and attempts to combine them lead to neither being satisfied. One of these requirements is that the quality of concrete must be accurately assessed. The other is that if the quality becomes unsatisfactory, this must be detected and rectified at the earliest possible moment.

The first requirement has no timescale and need not reference any particular amount of concrete so long as any merited penalty is short of demolition. So it can be based on a statistical analysis of say the last 30, 28-day results. This yields a quite reliable assessment but any change in quality would not be detected for a substantial time.

The second requirement has no reference to accuracy but only to urgency. Action to improve quality cannot be demanded by a purchaser unless it is possible to establish that quality is unsatisfactory, but if the producer even suspects that quality has reduced sufficiently to invoke a penalty, he is likely to take action without waiting for proof. This, of course, is assuming that the specification does not contain a ridiculous requirement that mixes may not be changed without waiting for the results of prior new trial mixes.

1.5 Cash penalty specifications

Generally, the author likes to feel that, while he is presenting knowledge and proposals that may be new to the reader, most of his proposals are acceptable to at least a few leading edge practitioners. The reader should be warned that there is just one item of the author's strongly held beliefs that has so far proved almost universally unacceptable. This means that any specifier adopting the following recommendation can expect to meet with substantial opposition. The belief in question is that the regulation of concrete quality cannot be completely fairly achieved without some form of penalty or bonus clause. It is relatively easy to establish in retrospect the exact amount by which the concrete represented by 30 or more test results has fallen below the specified level. It is also easy to establish fairly exactly what increase in cement content would have been necessary to raise the mean strength by this amount. The imposition of a cash penalty of twice the cost of this amount of cement would be quite sufficient to ensure that no producer could make a profit by deliberately supplying under-strength concrete. Of course we are talking of minor shortfalls of 1, 2 or 3 MPa – say not more than 5 MPa at most. An important by-product of such a specification would be that the supplier would be just as keen as the purchaser to react promptly to a downturn in early age strength, whereas action could only be demanded on the basis of 28 day strength (see Chapter 12).

1.6 Originality

The specifier needs to approach the question of originality with caution. Where practice in an area is unsatisfactory and producers are disorganized (or worse still well organized in an unsatisfactory way!) it may be appropriate to produce a specification requiring a change in practice (especially for a major project).

However practice may be deeply ingrained and reasonably satisfactory. Requiring a change may upset an existing control system and produce an initial period of instability. It may also result in suppliers making a large allowance in their price for unexpected contingencies. Obviously the author has introduced original specifications on many occasions, and in many countries, but it needs to be seen as a win-win situation and carefully explained to potential suppliers. It is perhaps unlikely that the average specifier will feel competent, or at least motivated, to undertake such a task, unless with solid backing from others or willing acceptance by the concrete supplier.

While the author sees some points at which the operation of the UK QSRMC (for example) is less than ideal, any criticism is in the nature of a suggestion for future improvement to the QSRMC organizers, rather than advice to purchasers that they should consider an alternative. Only in the case of a major project with a dedicated supplying plant might such an alternative be considered where QSRMC is in operation.

1.7 Conclusion

It is hoped that the above brief dissertation will be of assistance in helping specifiers to understand the nature of the problems involved and at least to avoid being a negative influence. Over the past several decades, specifiers, particularly in USA, have held back the development of improved control by writing specifications that deny producers any financial benefit from reducing variability or improving mix design. This has resulted in American control practice lagging behind that of many other countries and so concrete in the US tending to be more variable, more expensive and of higher shrinkage.

1.8 P2P

There is now a move by the US NRMCA (National Ready Mixed Concrete Assn: – a producer's body) under the title 'P2P'. This is an acronym for changing from Prescription to Property specification (see paper on my website). The NRMCA is to be commended for this initiative, resulting from irritation at decades of being essentially prevented from improving practice. An interesting point is that the eventual main beneficiaries from the initiative will be the purchasers of concrete. There will be a financial benefit from being allowed to install control systems that will reduce variability and so permit lower mean strengths in addition to a further saving through improved mix design. However when all or many producers achieve these same savings, competition will no doubt result in lower prices. This will be after a period in which there will be an advantage to those producers who respond most rapidly to the new situation. It should not be thought that this will necessarily exclude smaller producers, indeed one of the aims of this book and its associated website is to enable such producers to implement advanced control systems. Moreover such producers may be in a position to implement change more rapidly than larger producers and so gain an advantage over them. One such producer, Maricopa Readymix, has derived very substantial financial benefit and prestige from being the first US operator of the author's ConAd control system.

So this chapter ends with a plea to specifiers to carefully consider the foregoing. Of course it is your function to ensure that your client receives satisfactory concrete at minimum cost. It should also be a consideration to avoid impeding the development of improved techniques in your local readymix industry. Please realise that in many countries, and especially in USA, inept specifiers have fulfilled neither of these requirements over many decades. Specifications have favoured low-tech suppliers producing high variability, oversanded, high shrinkage mixes failing to take advantage of cement replacement materials that would improve durability in addition to reducing cost. If your project requires special concrete, then you need expert advice. Whether or not, a good first step, if in USA, would be to talk to the NRMCA or ACI about your local situation and whether you should seek expert advice. Every country probably has its equivalent to ACI. It would be nice if specifiers in general could be a positive influence, leading their local industry to improved practices and improving the image of concrete worldwide, rather than being obstructionist.

Chapter 2

Properties of concrete

Before starting to design (or specify) concrete, it is necessary to consider what properties we want the concrete to have, and also what properties we do not want it to have.

Some properties may come under both headings, for example heat generation, but generally undesirable properties are simply a lack of desirable properties.

Desirable properties:

- Durability
- Strength
- Impermeability
- Workability
- Dimensional stability
- Good appearance
- Economy.

2.1 Durability

Durability must come first on our list because if our concrete does not survive for the required period, it cannot be displaying any of the other desirable properties (not even economy because the most expensive concrete you can get is that which has to be replaced!).

However there is a difference between durability for a few years, a few decades or a few centuries, between durability at any price and 'reasonable' durability of economical concrete, and durability in benign or aggressive conditions. More particularly there is a difference between the durability of plain concrete and the durability of reinforced concrete.

In an excellent and well researched paper (containing five times as many references as its 15 pages, many cited as examples of incorrect thinking) (the paper is unpublished but available on the website) A. Leshchinsky and M. Lesinskij raise some interesting and important points about the durability of concrete. Everyone knows that the current lack of durability is a serious problem worldwide and the nature of modern cements is often blamed. The paper advances the view that modern ingredients are in fact superior to those of both the distant and quite recent past, and the real problem is that specifiers do not understand that they are different to those of the past and require different usage.

A major point is that the greater strength efficiency of new cements should lead to the increased use of supplementary cementitious materials rather than an increased *water/cementitious* ratio. The excellent potential of fly-ash and blast furnace slag are under-rated and they are even not permitted by some specifications. On the other hand silica fume is described as an important ingredient and a strong 'remedy' for concrete, which is often 'over-prescribed'.

The area of chemical admixtures for concrete is also seen as one in which excellent new products are misunderstood and misused.

Regarding fine aggregate, the authors see a failure to understand the difference between deleterious and non-deleterious fines. Specifications tend to limit the percentage of fines rather than the proportion of those fines that is deleterious material (clay and silt). In these times of approaching shortages of fine aggregate it is important neither to reject or unnecessarily treat satisfactory material nor to permit the use of unsatisfactory material. The real assessment of acceptability should involve the methylene blue test for superfine material rather than, or in addition to, sieve analysis (I question whether the settling test, crude as it is, might perform this function, requiring less expertise and cost).

There is a move in some quarters to use demolition waste or steel slag as coarse aggregate for concrete in an attempt to conserve natural materials. The authors reluctantly disapprove of the use of these materials due to uncertain durability and inevitably higher variability, and suggest that there may be more conservation value in using concrete of more certain, higher, durability.

Adding substantially to the problems caused by a failure of knowledge and understanding on the part of those specifying and producing concrete is a failure to carry out concrete work in a satisfactory manner. Many examples are given, particularly in respect of curing. It is made clear that curing is even more important from the viewpoint of durability and impermeability than it is from the strength viewpoint. The remedy is seen as more onerous specification requirements and harsher enforcement of those requirements, along with increased supervision. It is pointed out that the increased cost involved in following these recommendations will be small compared to the costs of future remediation or replacement.

Water is the worst component of concrete from the permeability viewpoint. When the excess water over that required for hydration of the cement evaporates, it leaves voids permitting water penetration. The best easy measure of resistance to permeation is the water/cementitious materials ratio, but in fact more water is more deleterious than less cementitious materials.

One item distinctly worse than pure water is seawater, or any water containing chlorides. This is because chlorides increase the electrical conductivity of the concrete, promoting the mechanism of steel corrosion.

Sulphate attack is the main risk of deterioration in the concrete itself. Sulphates react with the tricalcium aluminate in normal cement to cause a disruptive expansion. In addition the cement paste itself is weakened. Sulphate resisting cement is cement in which the tricalcium aluminate content has been limited. Low heat Portland cement also has its tricalcium aluminate content limited for the different reason that it generates more heat. So low heat cement is sulphate resisting but sulphate resisting cement is not necessarily low heat because there is no limitation on the proportion of tricalcium silicate in sulphate resisting cement, and this is an even more important generator of heat. However both of these cements have lower than normal resistance to penetration by chlorides, so neither should be used in marine situations because seawater contains both sulphates and chlorides. A better solution is to use fly-ash or blast furnace slag substitution. The latter is particularly suitable for marine use but there must be at least 70% of slag to be effective, whereas 20–50% of fly-ash would be used. Silica fume is also very effective in reducing permeability.

The other notable cause of deterioration in concrete is alkali-aggregate reaction. This is another kind of disruptive expansion but caused internally rather than by external penetration. Alkali-silica reaction is a disruptive expansion of the cement matrix arising from the combination of alkalis (usually, but not necessarily solely, from the cement) and reactive silica (usually in the coarse aggregate). While relatively rare, the phenomenon can be totally disastrous when it does occur. There are three possible strategies to limit its occurrence. One is to avoid total alkalis (sodium and potassium) in the cement exceeding 0.6% calculated as Na_2O. Another is to test the aggregate for reactivity. A third possibility is to provide an excess of reactive silica in the form of fly-ash, silica fume, or natural pozzolan so as to consume any alkali present in a non-expansive surface reaction product.

Concrete is also not resistant to acid although low-permeability concrete will not be rapidly attacked. Interestingly, geopolymer concrete is highly resistant to acid attack.

2.2 Rusting

Reinforcement is in fact the Achilles heel of concrete. We are all familiar with cracked, rust-stained concrete caused by the expansion of steel when it is converted into iron oxide. Roman concrete was not reinforced and this is a major reason for its survival for centuries. However, as we shall see, there are other reasons and unreinforced concrete soon disintegrates if subjected to movement of its foundations.

The major factor in the corrosion of reinforcing steel is the cover. Without adequate cover no ordinary concrete can protect the reinforcement. However excessive cover means that the surface concrete is essentially unreinforced and can crack under shrinkage.

With reasonable cover, the next factor is the permeability of the concrete. Steel will not rust unless water and oxygen can reach it and carbon dioxide has depassivated the steel. Since it is the cement that provides passivation of the steel, and since it reduces permeability, it used to be thought that a high cement content was the way to achieve durability and that the substitution of a proportion of fly-ash or blast-furnace slag for some of the cement would reduce durability. It is now realised that substitution of such materials, and especially of finer materials such as silica fume, reduces permeability and is an important positive factor in reducing corrosion. However to be effective in reducing permeability, good curing is even more essential with concrete containing fly-ash or slag.

2.3 Strength

Strength is well established as the primary criterion of concrete quality. Mix design has generally meant designing a mix to provide a given strength. While strength is often not the most important requirement, the reason for its use as a criterion is clearly shown by the step following its selection in most mix design procedures. This is to convert the strength requirement into a water/cement ratio. The relationship between strength and w/c ratio is generally attributed to Abrams, (USA, 1919) (Neville, 1995). Actually Feret (France, 1896) (Neville, 1995) preceded him and proposed a more accurate proportionality, that between strength and the ratio of cement to water *plus voids*. It may be that accuracy was not the important thing, partly because the w/c ratio itself was arguably more important than the strength it was assumed to represent, and partly because the simplicity of the concept was as important as its accuracy.

While the concept of water/cement ratio is simple, and its *approximate* implementation is also simple, it would be a difficult criterion to enforce by testing. A case could be made that the most accurate way of establishing the w/c ratio of a given sample of production concrete (of which the w/c ratio v strength relationship has already been established) is to test its strength. It is perhaps unfortunate that w/c ratio rather than c/w ratio came to be the popular parameter since, over a substantial range, strength has an almost linear relationship with c/w ratio.

So much of the importance of strength is as a test method and a means of specification for w/c ratio.

A primitive way of designing a mix, assuming that only one fine and one coarse aggregate were involved, would be to make a mix of any reasonable proportions (say 1:2:4) and fairly high slump (say 100 mm). If a sample of this concrete were heavily vibrated for several (say 15) minutes in a sturdy container (such as a bucket, not as small as a cylinder mould) then any excess of either coarse aggregate or mortar would be left on top. If the top half were discarded, then the proportions of the bottom half would be a reasonable guide to the desirable sand percentage to use. This is a useful exercise for students since it illustrates the concept of filling the voids in the coarse aggregate with mortar and demonstrates that an ideal mix cannot be over-vibrated once it is fully compacted in place (in that the remaining concrete will not further segregate however long it is vibrated).

Very high strength depends on a number of other things besides w/c ratio. These include the strength of the coarse aggregate and the bond between the matrix and the coarse aggregate. It used to be very difficult to achieve a strength much in excess of 90 MPa (13,000 psi). Strengths of double this amount are now easy to obtain given a strong coarse aggregate, silica fume and a superplasticising admixture. The author recalls carrying out trial mixes for 60 MPa concrete in the late 1970s before either silica fume or superplasticiser were available. Of the two coarse aggregates tried, the stronger one gave unsatisfactory results. This was because it was such a hard impermeable material that the matrix did not bond to it sufficiently. With silica fume and superplasticising admixtures now available, excellent bond was developed and the stronger coarse aggregate gives better results than the other and both can easily exceed 100 MPa.

There are two words of caution about using very high concrete strengths. One is that concrete in a structure cannot be saturated with water as can test cylinders or cubes in a water bath. It will have a w/c insufficient to provide full hydration and will therefore self-desiccate and not develop the full strength of the test specimens. At best it may be possible to prevent the loss of any of the mixing water by polythene wrapping immediately on demoulding or placing the concrete in permanent formwork such as a steel pipe column. So perhaps high strength test specimens should be polythene wrapped rather than water-bath cured, although this should probably be restricted to a few comparison tests, since it may be undesirable for quality control from the viewpoint of introducing variability into the results.

A very interesting development is the suggestion of using saturated lightweight particles in a mix to provide internally the water for curing (Bentz *et al.*, 2005). Another suggestion has been to use a proportion of reactive magnesia to perform a similar function (see Section 5.4 TecEco).

The other problem with very high strength concrete (actually very low permeability concrete) is that of explosive failure in a fire situation. The theory is that water vapour from the interior will be unable to escape and will cause explosive spalling. This may seem unlikely considering the self-desiccation referred to above, but in fact chemically combined water can be driven off. Here again an interesting new development is proposed, this is that nylon or polythene fibres be introduced to the mix so that, in a fire, they would melt and provide an escape path for moisture. Generally, structures fail in a fire more due to a failure to protect the steel than from deterioration of the concrete, so lightweight aggregate concrete, providing better thermal insulation, will show an improved result. A thought for the future is that geopolymer concrete actually gains strength when heated to a high temperature. It has even been suggested that maybe the ‘New York Twin Towers’ would not have collapsed had the columns been of geopolymer concrete!

A remaining bone of contention about high strength concrete is whether it still requires air-entrainment for frost resistance. There is no question that test cylinders cured in a water tank and frozen while saturated will show a benefit from air entrainment in even very high strength concrete. However the self-desiccation referred to above, plus the virtual impossibility of re-saturation, seem to suggest that air entrainment would be unnecessary.

2.4 Impermeability

This aspect has been extensively covered above but there remain a few points worth making.

One is that curing is much more critical for impermeability than it is for strength.

There are three avenues by which water can penetrate concrete:

1 Gross voids arising from incomplete compaction, often resulting from segregation.
2 Micro (or macro) cracks resulting from drying shrinkage, thermal stresses or bleeding settlement.
3 Pores or capillaries resulting from mixing water in excess of that which can combine with the cement. That is water in excess of 0. 38 by mass of cement.

Gross voids may be regarded as too obvious a cause to be included. However they are worth mentioning because they may be made more likely by action which would otherwise reduce porosity, that is, a harsh, low slump mix will have a low water content and a richer mortar (higher cement/sand ratio) than a sandier mix of equal strength. Obviously a low permeability concrete must be such that it will be fully compacted by the means available. It must not depend on unrealistic expectations of workmanship. Of course the development of self-compacting concrete is an excellent answer to permeability since it is inherently of low permeability and, at least theoretically, cannot suffer from a lack of compaction.

Water occupies 15–20% of the total volume of fresh concrete and, when the w/c ratio exceeds 0.38 by mass, not all of this water can be consumed in the hydration of the cement. To the extent to which the voids left by the excess water are discontinuous, they will not provide easy passage for water. This explains the tendency for graphs of permeability against water content, water/cement ratio etc. to rise slowly for a while and then suddenly sweep upwards almost asymptotically at the point at which the voids became interconnected (see Fig. 2.1).

The latest packing theories of mix design have demonstrated that close attention to the packing of fine material of cement size and smaller can reduce total void space in the paste fraction, especially when accompanied by superplasticisers.

The total amount of pore space is not the only factor determining permeability. Another important factor is the distribution of the pores and their discontinuity. Bleeding is a source of continuous or semi-continuous pores. Bleeding is initiated by the settlement of cement particles in the surrounding mixing water, after compaction in place. This tends to leave minute pockets of water under fine aggregate grains. There may be enough water to allow the fine aggregate grains to settle slightly and the water to escape around them and rise up through the concrete. The process occurs on a larger scale under the coarse aggregate particles and eventually the whole mass of the concrete settles slightly, leaving a film of water on the surface. The process can happen very gently without having a great deal of effect

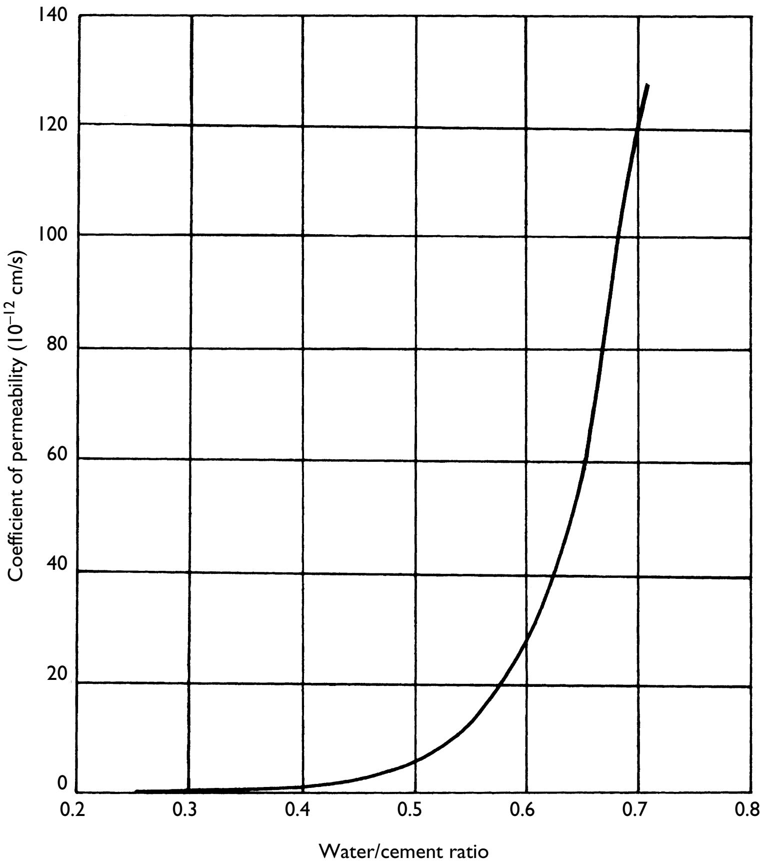


Figure 2.1 Relation between w/c ratio and permeability.

on the concrete properties. If bleeding is severe the rising water tends to leave well-defined capillary passages and it is then known as channel bleeding. Water penetration of the hardened concrete is obviously greatly facilitated by both the vertical channels and the voids formed under the coarse aggregate and even fine aggregate particles.

Reduction of permeability can be effected either by avoiding bleeding in the first place or by blocking the channels after formation. Pore blocking after they have formed takes place as cement continues to hydrate and extends gel formation into the pores. This requires the concrete to be well cured and is greatly affected by w/c ratio. See Table 2.1 and Fig. 2.2. Another means is to line the pores in the concrete with hydrophobic material. Such materials are marketed as 'waterproofing admixtures' and may be soapy materials such as stearates or

Table 2.1 Time taken to achieve discontinuity of voids

Water/cement ratio	*Age of concrete at which capillary pores become blocked*
0.40	3 days
0.45	7 days
0.50	14 days
0.60	6 months
0.70	1 year
over 0.70	infinity

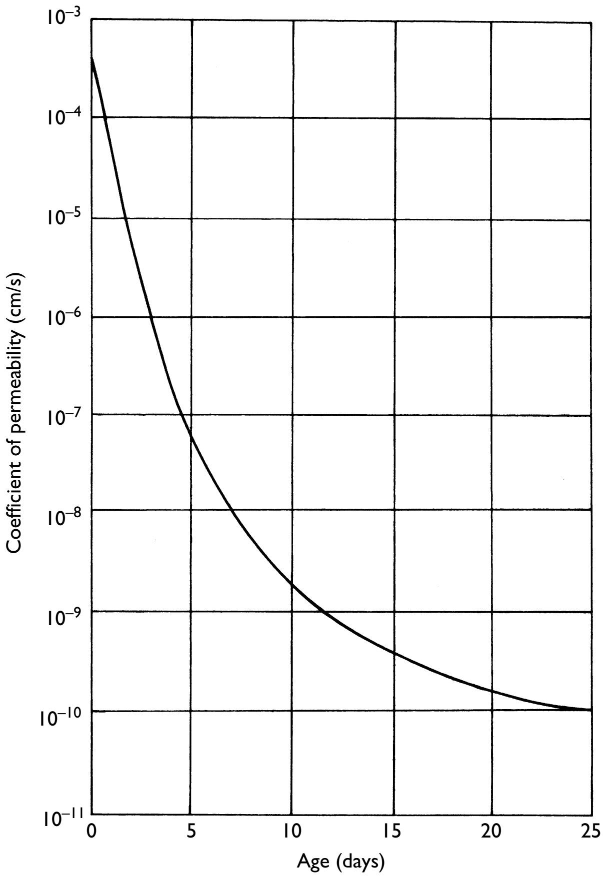


Figure 2.2 Reduction of permeability with curing.

materials such as silicones. Some hydrophobic material may provide an initial benefit but lose its effectiveness in the longer term.

Factors affecting bleeding are:

1 Amount of fine material (including cement, slag, fly-ash, silica fume and natural pozzolans)
2 Air entrainment
3 Water reduction through admixtures or lower slump
4 Continuity of grading (especially including fine aggregate grading)
5 The use of methyl cellulose or other gel-forming admixtures (mainly in grouts) now referred to as VMAs (Viscosity Modifying Agents)
6 Retardation, whether due to low temperature or chemical retarders, delays gel formation and so extends the period of bleeding

Essentially the mortar in concrete consists of a mass of particles saturated with water that is trying to escape: the more water there is, the more will escape by bleeding.

The better the particles pack together and the more difficult it will be for water to pass through the mass. Cement, slag, fly-ash, entrained air, rice hull ash and silica fume (in increasing order of effectiveness) are good inhibitors of bleeding. Very fine calcium carbonate (limestone) is a recent development and the superfine material in manufactured sand (crusher fines) is now considered very desirable in some circumstances. Silica fume is the most effective inhibitor of bleeding. It is many times finer than cement and particles of it fill the interstices between the cement particles. Small amounts (as little as 10–30 kg per cubic metre) are sufficient to prevent bleeding almost completely. It should be noted that the effectiveness of the fume is greatly reduced if it is incompletely dispersed. Essentially this means that silica fume should always be either batched as a slurry, or used in conjunction with a superplasticising admixture and given adequate mixing time.

It should be noted that eliminating or greatly reducing bleeding can create problems with evaporation cracking. Such concrete may require careful attention to preventative measures such as the use of liquid aliphatic alcohol evaporation retardant (Confilm) or polythene sheeting, mist sprays etc.

2.5 Workability

Workability is a critical feature of most concrete and there is much more to this property than is revealed by the still widely used slump test. Essentially we are considering the entire question of the fresh properties of concrete. Workability testing is more extensively dealt with under Testing (11.7) and Aggregates (7.1) contains much relevant information. The subject is only briefly covered here.

Apart from slump, workability may include some or all of mobility, fluidity, pumpability, compactability and, negatively, segregation and bleeding. A factor other than water content is clearly involved and this is best described as cohesion. Cohesion may be physically evaluated in terms of resistance to segregation and bleeding but a numerical measure is needed for use in mix design. The author uses a term he calls MSF (Mix Suitability Factor). This factor is derived from the overall mix specific surface adjusted for the content of cementitious material and entrained air, all of which increase cohesion.

The use of rheometers to measure the yield strength and plastic viscosity of concrete is taking over from traditional testing and traditional characterization in the laboratory but their use rarely extends to the field and these are measured parameters rather than something calculable from gradings and mix proportions. So they are to date a means of establishing whether or not the desired concrete properties have been achieved, rather than a means of calculating how to achieve them, although this may change in future.

MSF is certainly a big advance on characterizing mixes only by slump and a verbal description such as pump, structural, or paving mix. However it is not sufficient alone to cope with the 'new' material, self-compacting or flowing concrete. Even normal pumped concrete needs a measure of grading continuity and bleeding resistance. The latter is a matter of having sufficient fine material (at least passing a 200 sieve) or using a suitable chemical admixture.

Grading continuity can be regulated by nominating an ideal grading curve with limits. It is desirable that there should be two envelopes for the ideal curve, one being considered ideal and the other unacceptable. The acceptability of a grading can then be assessed by the cumulative percentage on each sieve outside the ideal limit, while a grading with any point outside the outer limit is simply unacceptable. It is also possible to nominate particular sieves as more important than the rest and to multiply the 'percentage defective' on those sieves by a factor. The author does not personally use this approach since there are too many unknowns in the shape of the ideal curve, the spread of the limits, and the particular sieve factors. However the ConAd system permits clients to enter their own opinions of these items and the author does like to look at a grading against one or other of these frameworks when pumpability decisions are marginal or disputed.

The author prefers to use a 'Gap Index' to measure the departure of a combined grading from a straight line. The cumulative percentage individually retained on the six finest sieves (0.15, 0.3, 0.6, 1.2, 2.36, 4.75 mm) normally account for almost 50% of total aggregate. So in a straight line grading each would have around 7% retained on it. So the author uses the sum of the squared differences between 7% and the amount actually retained on each of these sieves as a Gap Index. The advantage of this technique is that it can be incorporated in the author's 'Mixtable' system of mix design and also in the Mix Optimise free program on the website. When a mix is designed to provide a given strength, slump and MSF, the system displays a GI (Gap Index). If the range of mixes is required to be pumpable, or flowing, or requires high cohesion for any other reason, a limiting value of the GI can be specified. The program will then design a range of mixes in accordance with the instruction and will display the resulting increase in cost. Users can then decide how much improvement in grading continuity they are prepared to pay for. Some of the author's design systems also display a graph of individual % retained so that the user can see the effect of changing the GI.

There is a weak spot in this GI technique in that it would not distinguish between successive sieves each having 10% (= 7 + 3) retained and one having 10% and the next 4% (= 7 − 3). A second type of GI would total the squared differences between successive sieves. So the assessing system could output both GI 1 and GI 2, alerting the user to either situation.

Further detail on the slump test is given in Section 11.7 and its treatment in specifications is discussed in Chapter 6.

2.6 Pumpability

It has been a rule of thumb for many years that concrete which bleeds will not pump (although it does not follow that concrete which does not bleed will necessarily pump). In a definitive recent paper (Kaplan *et al.*, 2005), this theory is fully investigated and proven in full-scale tests. The paper describes a specifically developed new test for bleeding that is suitable for site use and provides definite guidelines for limits on bleeding that must be observed. It also gives useful advice on the operation of pumping. However it does not provide any advice on the design of pump mixes.

2.7 Slump

The mix design and quality control chapters have used slump as a measure of relative workability. It is important to realize that this is a matter of convenience and that the slump test is a very poor measure of the relative workability of *different* mixes. One reason for retaining slump as a criterion is that it is so deeply ingrained in the theory and practice of concrete technology. Another is that slump in combination with the author's MSF (mix suitability factor) does have a little more validity as an absolute criterion than slump alone. A third, probably the most important, is that it is a sensitive detector of a change in of water content between successive deliveries of the *same* concrete mix.

What is important is not to stop using the slump test but to realize and allow for its limitations. For example a limiting slump value is often included in a job specification. With few exceptions, this is not the best way to achieve the specifier's objective. First of all there should be an objective for the specification of anything, rather than it having been included in a previous specification and so mindlessly continued in the current document. The objectives may be to avoid high shrinkage, segregation and bleeding or to avoid an excessive w/c ratio leading to inadequate strength or durability. However any of these faults can be encountered at almost any slump, however low, and avoided at any slump, however high. It is also easy to detect from a theoretical mix submission which mixes will be subject to one or other of these problems. The contractor should therefore be permitted to submit his mix for approval at whatever slump he chooses providing it is designed to accommodate his own slump limit without detriment. It is quite possible to produce fully flowing (250 mm slump or more) concrete having none of the potential faults noted and to produce almost all these faults in a 50 mm slump mix.

Further detail on the slump test is given in Section 11.7 and its treatment in specifications is discussed in Chapter 6.

2.8 Self-compacting concrete

A whole new ball game in workability has been opened up with the concept of self-compacting concrete. This is a relatively new concept, having originated in Japan in the 1980s and originally met with a degree of skepticism in most of the rest of the world. It now, in early 2005, seems quite possible that it will become one of the most widely used kinds of concrete in the not too distant future. A whole section (5.3) has been devoted to this subject, even though the author has very little personal physical experience of it.

2.9 Dimensional stability

Dimensional stability may include undesirable degrees of thermal expansion and also disruptive expansion due to alkali-aggregate reaction or sulfate attack but essentially the problem is shrinkage. The major type of shrinkage is drying shrinkage but there are also autogenous or chemical shrinkage, carbonation shrinkage, elastic defection, and creep under load.

Autogenous shrinkage is relatively recently recognized as a phenomenon as it relates to concretes of very low w/c ratios which shrink as a result of self-desiccation. It occurs much more rapidly than normal drying shrinkage.

Drying shrinkage is a result of contraction of the cement paste as the uncombined excess water evaporates. This shrinkage is restrained by the aggregates, especially the coarse aggregates. From this it is obvious that shrinkage will be higher if there is more water and cement and more sand. Some coarse aggregates have an appreciable moisture movement that will directly contribute to shrinkage and apart from this, a higher elastic modulus of the coarse aggregate will reduce shrinkage.

2.10 Good appearance

A good appearance requires that concrete be fully compacted and free from 'bug holes' Actually the type of formwork and the mould oil used may have a considerable effect on this aspect.

However SCC demonstrates the importance of fair-faced concrete being non-bleeding. A tendency to bleed allows water to travel up the face of the formwork or towards any slightly leaking joints. This can produce very unsightly results including 'sand streaks' and 'hydration staining'. In its most severe form the later can result in black areas adjoining joints, caused by the bleed water washing the usual grey dust coating from the cement grains, which are actually black. Since true SCC does not bleed at all, it is free from such defects and can even be cast against inward sloping mould faces without defects being caused.

2.11 Heat generation

Heat generation is largely a matter of the type and quantity of cementitious material. Low Heat cement may or may not be economically available but in any case it is usually preferable to use a proportion of fly-ash to reduce generated heat. Where fly-ash is not available, some projects have used silica fume. Weight for weight this may generate even more heat (by speeding up the reaction) but the argument is that it permits more than enough cement reduction to leave a lower total heat generation.

Blast furnace slag cement calls for careful consideration. It actually generates more heat than normal cement but it does so more slowly. So in a typical situation the heat is able to escape and the peak temperature is reduced, but in massive sections, such as slabs more than one metre thick, the heat cannot escape quickly enough and the peak temperature is increased.

2.12 Economy

The most expensive concrete is that which has to be replaced due to being either initially unsatisfactory or inadequately durable. The cost of a higher quality grade of the concrete itself is, in most cases, a relatively small proportion of the total cost of the final structure. The costs of reinforcement, transportation, placing, finishing, curing, and especially of the formwork, often exceed the basic cost of the concrete. However it should be bourne in mind that the additional cost of a slightly higher quality concrete can be a significant proportion of the concrete producer's profit margin.

The message here is that you should not expect to get any higher quality than you have specified but that it may be worth specifying a quality that is a little higher than the absolute minimum quality you need (see 'What is economical concrete?' in Section 12.2). 'Quality' will generally mean a strength grade but shrinkage, bleeding, and resistance to deterioration may need consideration.

Contrary to past practice, the inclusion of cement replacement materials will generally give concrete of improved performance and is often worth specifying rather than merely permitting. At a given strength, the concrete with the *lowest* cement content will be preferable since it will also have the lowest water content.

Chapter 3

Mix design

Introduction

To some extent the science of mix design has moved on since the second edition of this work. The question really exercising advanced mix investigators relates to material passing the 100# (150 micron) and even the 200# (75 micron) sieves and the concretes of particular interest are often self-compacting, sometimes fibre-containing, and sometimes of strengths in excess of 150 MPa. Crusher fines ('Msand', manufactured sand) and superfine limestone are being looked at in a new light, alongside the now well-accepted fly-ash, silica fume and metakaolin.

So if you, the reader, wish to be an innovator and researcher, these (along with more powerful chemical admixtures) are some of the areas you should be considering.

However there are many small producers in every part of the world who will not encounter such techniques in their working lives and still need to know how to compete in the supply of ordinary concrete for ordinary projects. While the author is concerned to try to predict the future and assist in new advances in concrete design and control technology, he is also concerned to simplify the task of the small producer. Free programs for mix design and quality control now appear on the website www.kenday.id.au and this chapter attempts to guide such persons (and new student entrants to the field) to a fuller understanding of the situation and use of simple techniques that have enabled the author's clients to achieve unprecedented control and economy.

3.1 Simple mix design

The basic concept of a mix design is to select and proportion suitable materials so as to provide a required strength and workability. Strength is normally assumed to be proportional to w/c ratio and workability to slump and cohesion or sandiness. The more sand and the higher the water requirement (and therefore the cement requirement and cost) but the 'softer', more cohesive and easier to handle the concrete at a given slump. To a large extent, using a finer sand has the same effect as using a larger percentage of sand. Many mix design systems recommend an 'ideal grading' and some recommend a range of such curves for different purposes, but it has been left to this author to provide an actual numerical factor to represent the degree of sandiness. The factor is the MSF or 'Mix Suitability Factor' (some prefer to call it a mix sandiness factor) and it is derived from the specific surface (SS) of the combined aggregates.

The system was originally used in the early 1950s when not even calculators (let alone computers) were available. The previous edition of this book described how to use the system manually but now a simple free program has been provided on the website www.kenday.id.au

Details are provided later of the full ConAd mix design system developed by the author but this is now only available as part of the ConAd package marketed by Command Alkon. While this package is strongly recommended to any substantial concrete producer, it may be beyond the means of smaller producers. The basic technique of specific surface mix design is widely applicable and the author has therefore made the simple implementation program available free of charge on the website. Use of this program is easy, but users should realise that, as explained in a following section, there are limits to its applicability.

First, download the programs KensMix and KensQC from the website www.kenday.id.au following the instructions on the site (and ensuring that you have first downloaded the 'firebird' database program). Then go to KensMix, which will now be on your start menu. The screen Fig. 3.1 appears (but initially without data).

A previous mix can be recalled by clicking on the down arrow and selecting one from the list – DEMOMIX is provided to enable you to follow the explanation but, to be useful, you need to enter your own mixes. The first step is to enter aggregate gradings. To do this click on the 'Select Aggregates' button to get the screen in Fig. 3.2.

The gradings of fine and coarse aggregates are to be entered successively as per cent passing in the first column, giving the material a name under 'Current Aggregate File' and keying 'Save'. Subsequently the grading can be recalled at any time by keying 'Load' and selecting from the list displayed by highlighting it and keying 'Select' in the bottom right-hand corner. It is important that, for the system to work, the cost and SG of the aggregate must also be entered (the units of the cost are immaterial, just cost relative to the other materials is required). The Specific Surface (SS) is automatically calculated by the system. A large number of alternative materials can be entered.

Now recall a selected fine aggregate and click 'Use for' under fine aggregate and similarly for a coarse aggregate. The SG and SS of each are automatically displayed on the main mix screen when you return to it. When a mix has been recalled on the main screen, clicking 'Show' on the aggregate screen will display the properties of the material involved in that mix.

Where more than a single fine and coarse aggregate are to be used, they must first be combined using the combine screen in Fig. 3.3. In this screen, clicking on

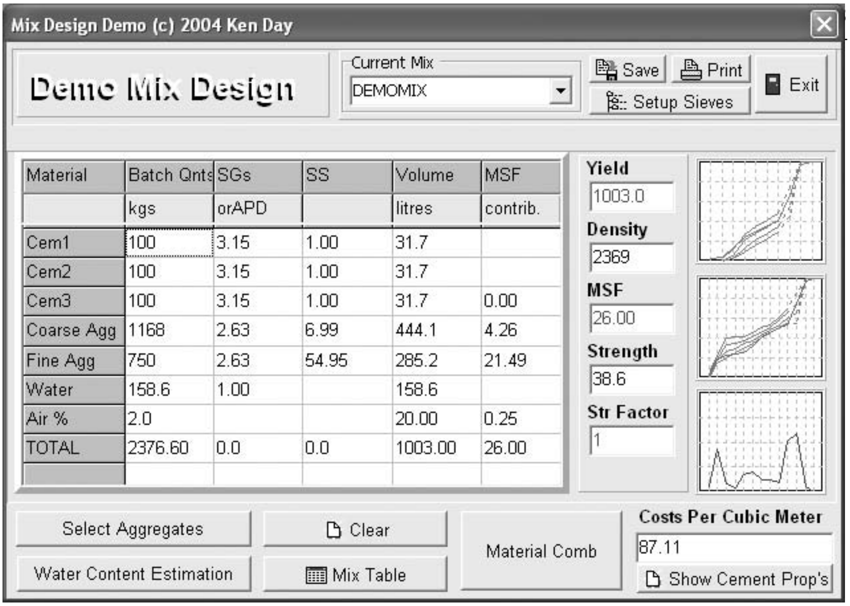


Figure 3.1 Simple mix design screen.

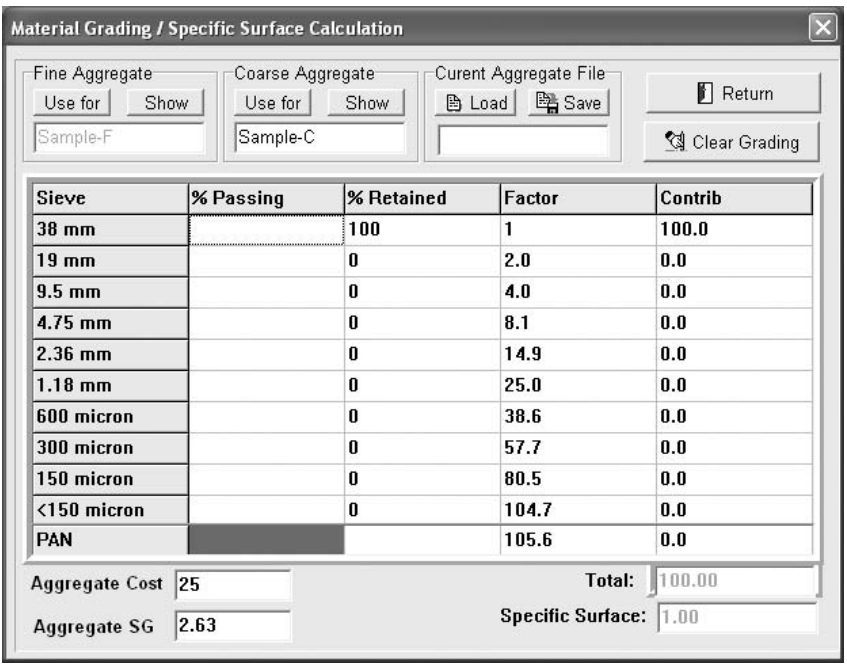


Figure 3.2 Specific surface calculation.

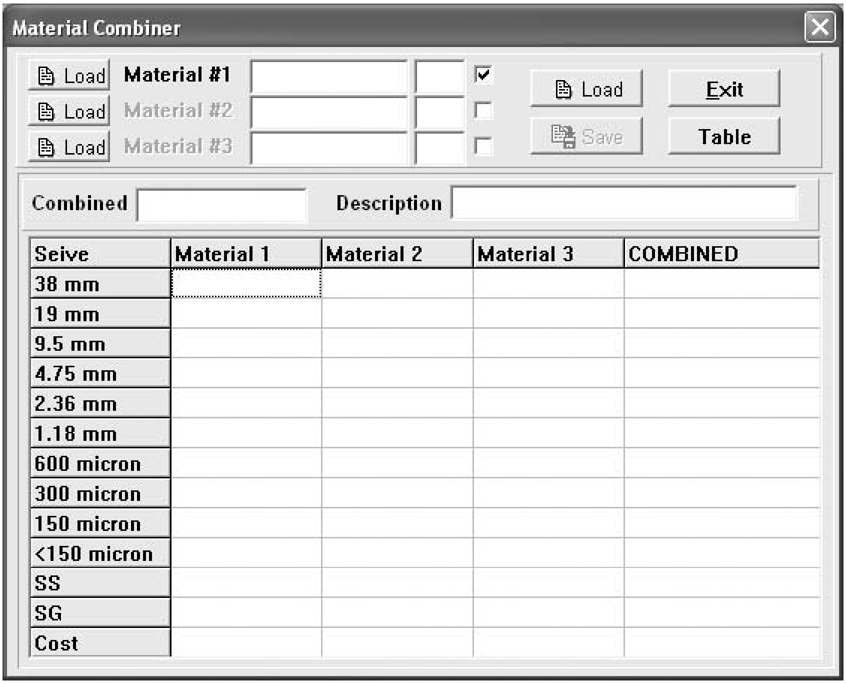


Figure 3.3 Material combiner.

'Load' at the LHS will display a list of all the aggregates you have entered. Highlight one of these and click Select and the name of the material appears in the first box at the top of the screen and its grading appears below. The second, smaller box is for the proportion of that material in the combination and the third small box is ticked to say 'use this material' or cleared to exclude it from the current combination so you can load three materials, coarse or fine, and try different relative proportions of either two or three, seeing the combined grading in the RH column. Having decided on a combination, give it a name next to Combined and a description. Then key Save. The combination will now appear as if a single material on your list of materials (and could, if desired, be brought into this screen as a single material and combined with other materials).

The main program is concerned only with combining one coarse with one fine aggregate. You have to nominate the relative proportions of two or more coarse or fine aggregates to each other. Having selected a combination, this is then saved as though it were a single aggregate. It is simple to recall this combined aggregate and use it in the mix system. (See Section 3.3 for guidance on desirable relative proportions.)

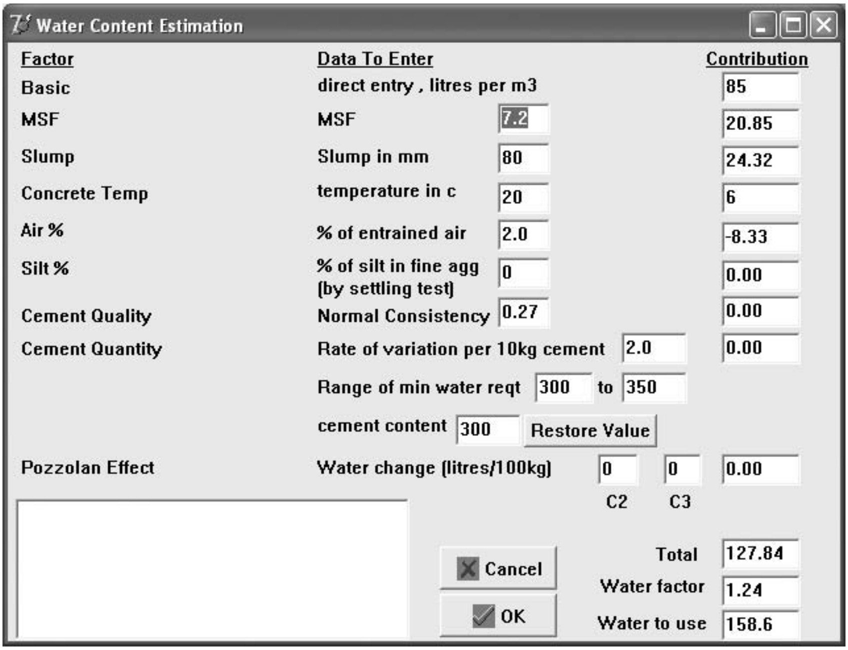


Figure 3.4 Water content estimation.

You can now return to the main screen and click 'Show Cement Props' in the bottom RH corner if more than a single cementitious material is to be involved. A small screen appears alongside the three rows allocated to cement. A strength factor and a cost must be entered for each material to be used. A suitable strength factor for fly-ash might be in the range of 0.5–0.9 and for silica fume 3.0–4.0, 1.0 having been entered for the basic cement. The SG of the cementitious materials is also to be entered in the appropriate column on the first screen. The 'equivalent cement' for workability purposes is not necessarily the same as that for strength. That for silica fume may be similar but that for fly-ash may be 1.0 or higher. These factors are to be directly entered in the 'SS' column on the main mix screen.

If you have a particular mix you wish to enter as trial, this can now be done in the first column. If you know the strength of this mix, the strength factor in the lower RHS should be adjusted until the program's prediction is correct.

If you do not know the water content of the mix, or if this is a new mix, click now on 'Water Content Estimation' in the lower LH corner to get the screen in Fig. 3.4.

The figures in the RH column are not to be entered or amended by the user except for the 'Basic' figure at the top of the column and the 'Water factor' or

'Water to use' at the bottom of the column. The latter two are inter-active, you may have an opinion about either and entering it will cause a change in the other to be in agreement with the total of all the contributions. Entering or amending a number as the water content on the main screen will cause it to appear here as 'Water to use' and the water factor will be automatically amended. Keying OK on this screen will cause the 'Water to use' figure to overwrite whatever is entered on the main screen.

In the central column, the MSF figure entered on the main screen will automatically appear here. The user can amend this figure to see the effect but the amended figure will NOT automatically transfer to the main screen. Slump, temperature, air content, silt content (per cent by settling test) and normal consistency of the cement are to be entered by the user and will cause changes in the Contribution column. If you do not know the values, guesses can be guided by seeing their resultant contribution. The effect of air content is of course to reduce water requirement.

'Cement Quantity' is a little complicated. There is an optimum range of cement content from the viewpoint of water requirement. Either more or less cement than this range will cause an increased water requirement. The user is to enter the range (of the order of 300–350 kg or possibly 250–400 kg) and the amount of water content change per 10 kg more or less than the range (1 to 2 litres/10 kg may be an appropriate figure). The cement content itself will be automatically transferred from the main screen. The user can amend this figure to see the effect, or in a case where the cement content on the main screen is expected to change. But again the figure entered will NOT automatically transfer back to the main screen and keying 'Restore Value' will cause it to revert to the figure that is on the main screen.

This leaves 'Pozzolan Effect'. The use of fly-ash is likely to cause a reduction in water requirement of the order of 15 litres/100 kg so – 15 (or your own alternative opinion) should be entered in the first of the two boxes (C2). Silica fume, on the other hand, may be considered to increase water requirement so perhaps 5 (+5 is not necessary) may be entered in the second box (C3).

The final entry is of the 'Water factor'. Three things need to be considered in selecting this. First is the effect of admixtures. Here you should remember that the effect of entrained air has already been allowed for and this accounts for a substantial part of the reduction achieved by a typical ordinary water reducer. So the additional effect may only be about 5%, giving a water factor of 0.95. On the other hand a 'High Range' or 'Superplasticising' admixture may give as much as 20% or more reduction and a factor of 0.80 or less.

A second effect to be included in this figure is that of fine aggregate particle shape. A badly shaped ('sharp') sand may increase water requirement by 2 to 4% and a manufactured sand or crusher fines can increase it by 7 to 10% or even more. The appropriate figure can be determined by establishing the per cent voids in the material. Every 1% of voids in excess of 35% may increase water requirement by 5 litres/cu metre or 2 to 3%.

Table 3.1 MSF values

MSF	*Slump range*		*Remarks*
	mm	*in*	
16			Unusable, too harsh
16–20			Harsh mixes, only suitable for zero slump concrete under heavy vibration
20–22	0–50	0–2	Hard wearing floor slabs, precast products under good external vibration
22–25	50–90	2–3.5	Good structural concrete
25–27	80–100	3–4	Good pumpable concrete. Fine surface finish. Heavily reinforced sections
26–28	90–120	4–5	Pumpable lightweight concrete
27–31	200+	8+	Flowing superplasticized concrete

So, with a normal water reducer and well-shaped crusher fines the water factor may be $1.00 - 0.05 + 0.07 = 1.02$. However the factor should finally be adjusted to accord with the user's own experience.

You are now in a position to return to the main screen and make final adjustments. If the purpose of the current entry is to evaluate an existing mix, then the evaluation is in terms of whether the MSF is suitable for the purpose in hand (Guidance is provided in Table 3.1), whether the yield is correct, and the strength, density and cost of the mix. At the RHS of the screen are three small graphs. Right clicking on them will expand these. If the cursor is moved off the graph while it is still expanded then the graph will remain expanded until it is again clicked on. The top graph is a traditional grading curve for aggregates only, shown against the old UK Road Note 4 type grading curves. Below this is a more useful curve showing all constituents including cement, air and water shown against curves derived from the UK curves at an average strength level. The lower curve is of percentage retained on individual sieves. This enables a critical examination of gaps in the grading that may affect pumpability, segregation or bleeding.

If the purpose is to design a new mix, a rough guess as to proportions is entered initially. Adjustments may be made in any order and repeated adjustments may be needed as one adjustment affects a previous one.

Start by entering 1,000 in the 'Yield' box in the upper RHS. The quantities will automatically adjust to give correct yield, but without changing the entered cementitious materials content.

Now enter the desired MSF value. The relative proportions of coarse and fine aggregates will change to provide this without affecting correct yield.

Next the strength can be considered. Direct adjustments can be made to any or all of the three cementitious materials and will be seen to change the predicted strength. They will also affect yield and probably water content so it may be necessary to revisit the water content screen before re-entering 1,000 in the yield box.

The predicted strength is of course dependent on both the main strength factor on this screen and the strength factors for the individual cementitious materials on the adjunct screen produced when ‘Show cement properties’ is keyed. The correct values of all of these depend upon the properties of your local materials and should be adjusted on the basis of prior or subsequent experience with one or more mixes. However they should NOT change when the mix is changed.

The density figure is NOT subject to error or opinion in the way that strength and water content predictions are. If the correct SGs and water and air contents have been entered, the density figure will be correct. If your test cylinders do not have this density then the concrete has not been fully compacted, the water or air content is incorrect, or you have entered incorrect SGs or batch quantities.

When satisfied with the mix it should be given a name and saved. A substantial number of mixes can be designed and saved to give ranges of strength, mix types, and cementitious combinations in addition to different aggregates. The cost comparison provided by this exercise should be of interest if different strengths, slumps, MSF values, cementitious combinations and aggregate sources are entered. The entered mixes are displayed in the screen shown in Fig. 3.5 when ‘Mix Table’ is keyed on the main screen (Fig. 3.1).

The provided system does not display it, but users may find it of interest to construct a spreadsheet table showing the variation of water content with the various factors listed in the water content estimation screen. The relative variations may be more interesting and more accurate than the individual predictions. By selecting a typical mix, it would be possible to put a cost alongside the various water content factors.

Having mastered the basics of SS Mix Design, the reader now needs to continue to examine the origins and limitations of the system and how it can be employed to originate commercially competitive mixes given a range of materials from which to choose.

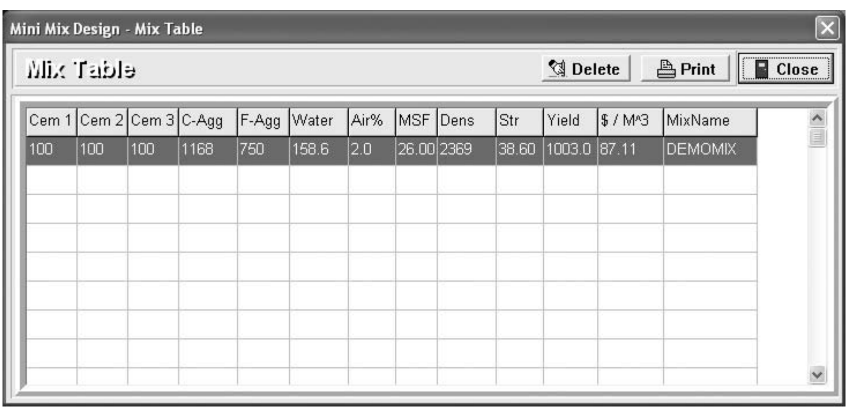


Figure 3.5 Table of mixes.

Trial mixes

In the second edition of this book, the author to some extent denigrated the practice of laboratory trial mixes. This was on the grounds that it may be more effective to start with an over-strength mix and adjust it under production conditions. The author is still confident that he can design a mix ‘over the telephone’ that will have suitable workability and a strength within $\pm$ 5 MPa using only an OPC (Ordinary Portland Cement). This claim always excluded the possibilities of chemical impurities or susceptibility to alkali-aggregate reaction, but a new risk has become very much apparent. As reported in Section 3.6, the author took place in a mix design competition in another country and was advised that the test cylinders of his mix ‘fell apart on demoulding’. The problem was an interaction between the admixture selected, the type and amount of fly-ash used, and the properties of the particular cement. The advice therefore has to be that, if new materials, or substantially unusual proportions of current materials, are involved, either a laboratory trial mix, or at least a Vicat or other setting time test using the proportions of cement, other cementitious materials, and admixtures envisaged, is advisable. For this purpose, laboratory facilities are not essential; a very rough trial would suffice to eliminate the possibility of such a problem.

Manual design

The basis of specific surface mix design can best be explained by an example of its use prior to the availability of computers, or even calculators. The author’s system was in use for many years (by himself only) prior to computerization. It is not necessary to forego the more precise assessment provided by modified specific surface just because a computer is not available.

Calculation of the modified specific surface of each aggregate using a calculator is little more arduous than fineness modulus calculation. The designer can select an MSF value from Table 3.1 and cement and water contents from previous experience or published data.

The required overall aggregate specific surface can then be calculated as

$$\text{MSF} - 0.025\text{EC} - 0.25\ (\text{air per cent} - 1) + 7.5$$

where EC = ‘equivalent cement content’ (see later).

The fine aggregate percentage is then calculated as:

$$\frac{\text{Desired combined SS} - \text{coarse aggregate SS}}{\text{Fine aggregate SS} - \text{coarse aggregate SS}} \times 100 \qquad (1)$$

Where more than two aggregates are to be used, the combined specific surface is given by:

$$\text{Combined SS} = \frac{\text{SSagg1} \times \text{per cent agg1} + \text{SSagg2} \times \text{per cent agg2} + \cdots}{100} \qquad (2)$$

All aggregates may be directly combined by trial and error in this way or all coarse aggregates may be combined in arbitrary proportions and all fine aggregates treated similarly. Equation (1) may then be used to determine the relative percentage of combined fine aggregates to that of combined coarse aggregates.

Before the advent of even pocket calculators, the author has designed many mixes in the field, literally on the back of an envelope, from no more information than a sand grading. The process took about five minutes. Coarse aggregate SS was usually guessed at 4 or 5 (it has only a small effect) and it was necessary to have in mind either a cement content or a w/c ratio.

The process is now made easy by the free program described earlier. All the features now incorporated in the author's computerised mix design system improve accuracy. The point is that the basic concept already provides as much or more accuracy and much more flexibility than most other mix design systems and only a direct assumption, such as a 1:2:4 mix, is quicker to use without (or even with?) a computer.

Example of manual approximate design

Desired characteristic strength	40 MPa
Allow for standard deviation (range 3 to 6)	say $+ 1.65 \times 4$
Required mean strength $40 + 1.65 \times 4$	$= 46.6$ MPa
Water requirement (160 to 200)	say 180 litres/m^3
Required w/c ratio (using strength $= 25/(\text{w/c}) - 8$)	$= 0.458$
Cement requirement $= 180/0.458$	$= 393$ kg/m^3
Required specific surface (22 to 30)	say 25

Sand specific surface[1]

Grading	*Sieve*	*% Pass*	*% Rtd*	*Factor*	*Total*
	4.75	100	0	8	0
	2.36	90	10	16	160
	1.18	80	10	27	270
	600	60	20	39	780
	300	30	30	58	1,740
	150	10	20	81	1,620
	0	0	10	105	1,050
					5,620

1 A very fine (zone 4) sand would have an SS of about 64 and a very coarse (zone 1) sand one of about 40.

Sand specific surface	= 5,620/100 = 56.2
Say coarse aggregate specific surface approx. 5	
Required sand per cent	= [(25 − 5)/(56.2 − 5)] ×100
	= 2,000/51.5
	= 39%
Cement paste volume	= water + cement + air
	= 180 + 393/3.15 + 2.0 × 10
	= 324.8 litres/m^3
So aggregate volume	= 1,000 – 324.8 = 675.2
Say SG of fine aggregate	= 2.6
and SG of coarse aggregate	= 2.8
Then	
Wt of fine aggregate	= 675.2 × 0.39 × 2.6 = 684 kg/m^3
Wt of coarse aggregate	= 675.2 × 0.61 × 2.8 = 1,153 kg/m^3

The approximations in this design are in selecting the water content and the strength formula. A more accurate way of estimating water content and a more accurate strength formula are given in the free program, or tabulated values can be selected from other systems.

The required specific surface is not an estimate but a selection by the designer to suit the particular job conditions. If desired, selection can be via the tabulated values of mix suitability factor in Fig. 3.1 (with no entrained air and a cement content of 250 kg/m^3, specific surface and mix suitability factor are identical).

The above process is simpler than most published systems whilst still providing accurately for the effect of varying fine aggregate grading and permitting the designer to select the type of concrete desired.

If the effect of varying cement and entrained air contents are to be neglected, as in most mix design systems, the determination of the desirable fine aggregate percentage is extremely simple. The designer may have a particular combined grading curve in mind. For example specific surface of the desired grading can be determined in exactly the same way as for an individual aggregate. With experience, what will be in mind will be a direct value of combined specific surface taking into account all circumstances (including desired slump, cement content, air content, etc).

All aggregates may be directly combined by trial and error in this way or all coarse aggregates may be combined in arbitrary proportions and all fine aggregates treated similarly. Equation (1) may then be used to determine the relative percentage of combined fine aggregates to that of combined coarse aggregates.

3.2 Origins and limitations of specific surface mix design

The basic concept of specific surface mix design is extremely simple but requires modification to work effectively. The simple basis is that a given degree of

Table 3.2 Modified specific surface values

Sieve fraction	*Author's modified SS values*	*Approx. true specific surface (cm²/gm)*[a]	*Surface modulus*
> 20 mm	2	1	1
20–10	4	2	2
10–4.75	8	4	4
4.75–2.36	16	8	8
2.36–1.18	27	16	16
1.18–0.600	39	35	32
0.600–0.300	58	65	64
0.300–0.150	81	128	128
< 0.150	105	260	256

Note
a According to B. G. Singh (1958).

workability will require an appropriate specific surface to avoid segregation, the higher the workability, the higher the required specific surface. Knowing the individual specific surfaces of the coarse aggregate and the fine aggregate, the required sand percentage can be calculated.

It is well known that a finer sand will have a higher water requirement than the same amount of a coarser sand, but specific surface theory says that, within wide limits, if the proportion of fine sand is reduced so that the specific surface of the combined aggregates is the same as with the coarser sand, the same water requirement and the same degree of cohesion will result.

The original SS theory did not work in practice because it was found to over-estimate the effect of very fine particles. The surface area of a sphere doubles as its diameter halves, giving rise to the second column of figures in Table 3.2 (neglecting particle shape). The author's modification recognises that, as diameter reduces, a point is reached where it takes less water to fill the voids in the material than it does to coat its surface. On a purely empirical basis, the first column in Table 3.1, 'modified specific surface' was originated by the author in the 1950s to implement this concept. It was assumed at the time they were originated that these values would require subsequent refinement but, in spite of attempts to improve them in the laboratory, and by their use for production concrete in many countries, the figures have remained substantially unchanged for 50 years.

It would be more correct to use surface area per unit solid volume than per unit weight but the weight basis was been retained because the actual numbers were familiar to users of the original SS theory. For the same reason, the author's original modified figures have been doubled so that the overall combined aggregate SS is of the same order as the original. However where there is a large difference between the SG (particle density) of coarse and fine aggregates an adjustment is desirable.

Modification of the basic SS values is not the only adjustment required to make SS mix design work. Other factors to be taken into account include:

1 The effect of cementitious materials and entrained air.
2 The effect of particle shape.
3 A requirement for continuity of grading.
4 Limitation of fineness and coarseness of sand grading.

Before discussing these points, some of the objectives of mix design should be reviewed. Generally a sandier mix will have a higher degree of cohesion and be easier handle and place. However it will have a higher water requirement. Traditionally, water/cement ratio has been regarded as the best criterion of quality, so that a sandier mix will require more cement and so be more expensive. Further investigation has shown that additional water is more deleterious than less cement at a given w/c ratio, increasing the desirability of minimizing water requirement. So the objective of mix design is to achieve acceptable fresh concrete properties at minimum water content. With the advent of self-compacting concrete, the task becomes even more critical.

Turning now to the above points:

The effect of cementitious materials and entrained air

These materials increase cohesion and so reduce the required SS of the aggregates. The author has coined a term MSF (Mix Suitability Factor) to represent the combined effect of all constituents on cohesion. The formula is:

$$\text{MSF} = \text{SS} + 0.025\text{EC} + 0.25\ (\text{air\%} - 1) - 7.5$$

where

SS = modified specific surface of combined coarse and fine aggregates
EC = ‘equivalent cement content’ (see later).

The effect of particle shape

An intrinsic assumption in SS mix proportioning is that a finer sand will cause less disruption to the packing of the coarse aggregate, permitting a reduction in sand percentage. It is not necessarily obvious that this reduction is exactly the same as the reduction needed to maintain the same combined specific surface of the combined aggregates but this seems to work in practice.

A more angular particle shape of the coarse aggregate also causes an increased requirement for sand, since it increases the percentage voids in the coarse aggregate to be filled by mortar. An increase of up to 3 in the appropriate MSF may be needed depending on the degree of angularity (which has a larger effect than flakiness or elongation).

The actual surface area of both coarse and fine aggregates is obviously increased by a more angular particle shape at a given grading. However whereas an increased fineness of a sand can be fully compensated by reducing its percentage (so there is no increase in water requirement), this is not so for a more angular fine aggregate, especially crusher fines used as a fine aggregate, since it does not reduce the interference with coarse aggregate packing, and may even increase it. So the angularity of the fine aggregate is neglected in determining the percentage to be used, but the predicted water requirement may increase by 5 to 15%.

Specific surface cannot be the only criterion for mix proportioning because it does not take into account particle shape and provides no assurance of continuity in the grading, which may be needed to avoid segregation and achieve pumpability. This is the aspect better covered by the void-filling theories, but the author believes he achieves a simpler and more workable solution by using crude, semi-empirical, corrections for these purposes.

Grading continuity

In the past, a great deal of research effort has gone into the search for an ideal aggregate grading. This has been to some extent pointless because, even if it exists, such a grading may be impossible or too expensive to attain with the materials available. One still sees requirements for sand grading to be within certain limits (particularly in USA) but the move to abolish them is gaining momentum.

However it is undeniable that gaps in an aggregate grading, while they may make the concrete easier to compact under vibration, increase the tendency of the concrete to segregate. Resistance to segregation is vital in higher slump and/or pumped concrete. The author has therefore added a ‘Gap Index’ to his mix design system. This is discussed in Chapter 2 ‘workability’.

Limitation of fineness and coarseness of sand grading

A wide range of sand fineness can be accommodated by appropriate adjustment of sand percentage to give a desired combined aggregate specific surface, but there are limits.

Upper limit of coarseness

A sand reaches the upper limit of coarseness when there is insufficient paste (cement, water and entrained air) in the mortar to provide adequate lubrication. This occurs not so much due to the coarser sand requiring more paste per unit quantity of sand, but rather because more sand must be used to provide the desired surface area if it is coarser. If the sand quantity is not increased, the overall mix will be too harsh, and will segregate unless of very low slump. If it is increased beyond the limit, the water requirement rises to provide the required total paste volume required. Strength will be reduced, the concrete will almost certainly bleed severely, and workability will suffer in a different way that is, it will have unsatisfactory mortar quality rather than an inadequate amount of mortar. A comprehensive mathematical treatment of this problem is given by Dewar in his latest book (Dewar, 1999) but here we will deal only with a few rules of thumb. What is important is that users should recognize the problem when they encounter it. As noted above, this will not occur at a particular sand percentage for all mixes but will depend on several other factors. Some rules of thumb to indicate when the problem should be considered are:

1 Sand percentages in the range of 50% of total aggregates (in low cement mixes) to 65% (in high cement mixes) (very rough guide).
2 Solid volume of sand exceeding about 5 times the solid volume of cementitious material. With normal sand and cement this can be taken as a sand/cement ratio of about 4 by weight. When fly-ash or very heavy or light sands are involved, the volume figure applies. This guide is still not invariably accurate because the limit is affected by the particle shape and grading of both the sand and coarse aggregate and by the use of air entrainment.
3 From a different viewpoint, the problem may arise when the FM (fineness modulus) of the sand exceeds 3.0 in low cement content mixes or 3.5 in high cement content mixes. In ConAd specific surface terms the danger signals may be around 40 for high cement contents and 45 for low cement contents.

Upper limit of fineness

The fine limit for a sand is reached when a further reduction in sand proportion will leave insufficient mortar (sand plus cement paste) to provide adequate lubrication to the coarse aggregate. With a very fine sand it is possible to get quite close to using a cubic foot of coarse aggregate by loose volume in a cubic foot of concrete and the shape and grading of the coarse aggregate makes a substantial difference to where the limit is. The limit will certainly be close however when the coarse aggregate approaches 60% by solid volume of the total concrete. Again from the other point of view, the problem is likely to arise with sands of FM around 1.5 (with a high cement content) to 1.8 (with a low cement content) or, in ConAd SS terms, in excess of 90 with any cement content. It is also possible that a high cement/sand ratio is intrinsically undesirable in the same way that a heavily oversanded mix is undesirable (e.g. higher shrinkage?). A sand weight less than the weight of cementitious materials should be viewed with suspicion and avoided if possible.

Coping with extreme sand gradings

The important point is rarely the establishment of the exact limit, rather it is the fact that within these quite wide limits, grading is not the problem that most typical specifications would suggest. It is of course necessary to accurately determine what proportion of sand should be used in each particular case and this is the main strength of the method of mix design evolved by the author.

A recent example of the coarse limit was encountered in Indonesia. The local sand on occasions had less than 3% passing a 300 micron sieve. Its Fineness Modulus was only of the order of 3.0, which did not seem an excessively high figure. However its Specific Surface of 40–42 was clearly excessively low. Increasing the proportion of this sand did not solve the problem, which was excessive bleeding. Eventually a choice had to be made between a proportion of finer sand, even though not locally available and so very expensive, and the use of additional cement purely for bleeding suppression. Another alternative would have been air entrainment but this was rejected, again due to non-availability locally, but also because the production personnel were unfamiliar with it and had no test experience or equipment. There have been very coarse sands in Singapore and in Australia requiring 48–55% of sand but these have all occurred when relatively high cement contents were required. In an extreme case, where the sand is very coarse and only a low strength and therefore a low cement content is required, the following possibilities should be considered:

1 Use of a small proportion of a second fine sand (even if quite expensive).
2 Use of a small proportion of crusher fines with a high ‘fines’ content.
3 Use of fly-ash, which has 37% greater volume than an equal weight of cement (if in an area where fly-ash is inexpensive, more might be used than strictly necessary for strength).
4 Use of air entrainment (as valuable, volume for volume, as cement for this purpose).
5 If no alternative is less expensive, the use of more cement than necessary on strength grounds would certainly solve the problem since it both reduces the sand percentage required for a given MSF and provides more paste to fill the sand voids.

Extreme testing of the fine limit has also occurred. In 1956 (Day, 1959) a case was encountered where the sand percentage calculated by the author’s system came to 15% (virtually all the sand passed the 300 micron [No. 50 ASTM] sieve). It proved possible to obtain a 1/4″ (7 mm) single sized crushed rock and the concrete was made with 10% of this material and 15% of sand (the balance being 75% of an almost single sized 20 mm [3/4 inch] crushed rock).

During the early development of the system (in the early 1950’s in England) sand percentages of 22–23% were used but, although the sand was purchased as ‘plastering sand’ rather than ‘concreting sand’, this was an example of the use of a very low ‘MSF’ on earth dry concrete rather than the use of a very fine sand. It should always be possible to use a proportion of crushed fines (choosing a coarse variety) when the natural sand is too fine for use alone. However the particle shape of the crushed fines will increase water requirement, and therefore increase cement requirement, at least somewhat.

3.3 Cost-competitive mix design

Overall economics

The economics of concrete production are extremely important, it is a competitive business and an uncompetitive producer will not survive. Certainly reliability and reputation are also important, but costs must be contained.

The main cost factors are:

1. Unit costs of materials
2. Ability to design economical mixes
3. Control margin (necessary difference between specified and mean strength)
4. Expenditure on staff, equipment and software
5. Efficiency of operation.

A producer must be able to make the correct choice of materials, taking into account the variability of those materials, which can increase costs by increasing concrete variability and therefore the necessary control margin. The ability to determine the relative proportions of available aggregates without extensive trial mixes is a substantial factor in making the correct decision.

A higher order of ability is called for when SCC or HPC, and the availability of multiple cement replacement materials and admixtures is involved.

Where free to choose, a balance between control costs, including personnel, equipment, software and testing frequency, and the additional cement cost of a higher margin must be sought. Standard deviation can range between 2 and 4 MPa, which, on a 5% defective criterion, means a control margin of between 3.3 and 6.6 MPa (Table 3.3). The difference between these two, 3.3 MPa, is probably worth about 25 kg of cement per cubic metre.

The use of a multigrade control system can easily justify halving the amount of testing and, even so, can dramatically reduce the time delay in reacting to change (if the optimum system).

For ‘ordinary concrete’ the free ‘Optimize’ spreadsheet program on the author’s website (further described below) provides a rapid way of assessing the relative merits of a number of aggregates.

Table 3.3 Percentage of results outside statistical limits

A (%)	*k*
0.1	3.09
1.0	2.33
2.5	1.96
5.0	1.65
10	1.28

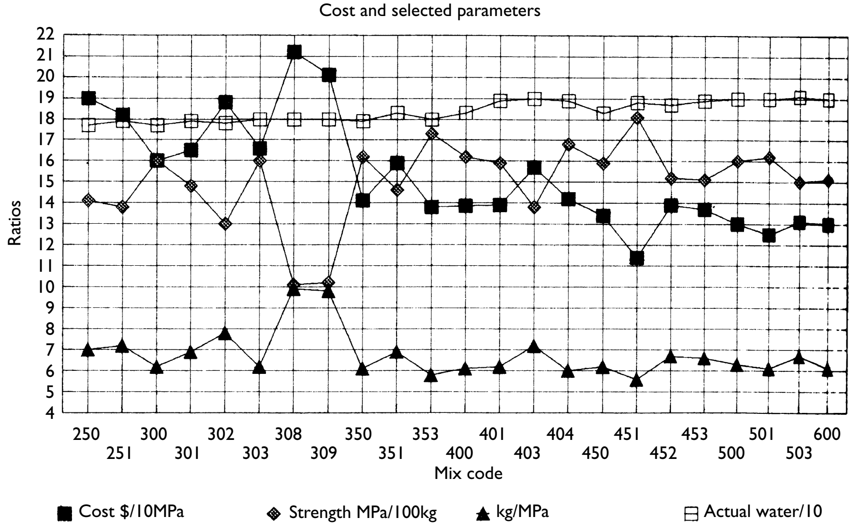


Figure 3.6 Variation of selected parameters over entire range of mixes.

A simple chart (Fig. 3.6) with all mixes strung out along the X-axis and ordinates of Cost ($/MPa); Strength (MPa/100 kg cementitious); and water content/10 will show the relative economy of all mixes. The best and worst of these can then be examined to indicate desirable changes. Alternatively the same data, with the same ordinates, can be plotted with the X-axis being strength. This will indicate whether some types of mix are economically more successful than others in particular strength ranges. The old favorite of simply plotting cement content per MPa against strength is no longer useful alone, because the relationship is distorted by the use of admixtures and cement replacement materials.

Of course even cost per MPa is not a fair comparison for all mixes. Higher slump, pumpable, and SCC mixes will show lower economy and it is useful to group such mixes together. However it is still useful to have them all on the same graph to see just how much the higher workability performance is costing.

Selecting aggregates for maximum economy

Combining two sands

It should be realized that cement content is not the only criterion of cost. There is often a quite wide difference between the price of sand and that of coarse aggregate. This can occur in either direction, but where sand is more expensive than coarse aggregate, use will normally be made of a proportion of crusher fines. Where all sands are cheap and there is a choice, the coarsest usable sand will be selected to maximize sand proportion.

The author has always been conditioned to think that higher cement content mixes were necessarily more expensive and therefore that pump mixes, which usually contain more sand and therefore need more water and cement, were more expensive than 'structural mixes' (jargon meaning mixes quite useable with skip placing but not pumpable). In Singapore, in 1980, he found that sand was so much cheaper than coarse aggregate that the sandier mix, in spite of the extra cement, was less expensive (or would have been except for its high clay content). The natural reaction to this is to use the pump mix even if it was not to be pumped, unless low shrinkage is an essential.

Where a coarse and a fine sand are combined in mixes, their relative proportions require careful logic. The assumption is that there is no such thing as an ideal sand grading so that a fairly wide range of relative proportions may give similar concrete quality. The relative proportions will therefore be biased to one extreme or the other of this range according to economic considerations. Surprisingly the relative cost of the two sands makes very little difference because increasing the proportion of a cheap fine sand simply results in using less of the sand combination and more coarse aggregate, so very little extra fine sand is used. What matters is the relative cost of the coarsest sand and the coarse aggregate. If coarse aggregate is more expensive, then the minimum amount of fine sand will be used to give the greatest total sand quantity.

Two formulas to give a guide to the values of combined SS that might be selected when combining two sands are:

$$\text{Maximum SS} = 66 - 0.02\text{C} - 0.024\text{F}$$
$$\text{Minimum SS} = 55 - 0.02\text{C} - 0.024\text{F}$$

where

SS = the Author's modified specific surface for the combined sands (see below)
C = the cement content in kg/m^3
F = the fly-ash content

It is not suggested that these values are actually limits. The reader should feel free to work outside them if driven by circumstances but they give some guidance to conservative values for inexperienced users. Another rule of thumb is that sand should not exceed four times the mass of cement (or cementitious materials).

Selecting from a range of available coarse aggregates

An XL spreadsheet originally devised by Michael Shallard is freely available on the author's website. It worked well in its original simple form but trying to take into account all the above factors appears to exceed its calculating capacity. There is now an improved version on the website, contributed by John Harrison and Pierre Perrault. This version is no longer permanently free but may be used for a demonstration period and purchased, if desired, from John Harrison.

The spreadsheet has a section in which up to eight coarse and five fine aggregates can be entered together with their gradings (as per cent passing), specific gravities, and costs.

The spreadsheet converts per cent passing to individual per cent retained and determines the specific surface of each aggregate.

In its simplest form, the user also inputs cement content and SG, water content, air per cent and a required MSF. The program uses MS Solver to determine the most economical combination of the thirteen materials. The obvious constraints on the system are:

1 The combined per cent of fine aggregates (as a per cent of total fine aggs) must equal 100, similarly the coarse aggs, and the per cent of total sand as a per cent of total aggs plus the per cent of coarse aggs must equal 100.
2 The SS (specific surface) of the combined aggs must equal the specified MSF minus the contribution of the cement and entrained air.
3 The solid volume of the combined aggs must equal 1,000 litres minus the solid volumes of cement, air and water.

So far, so good, the program is well able to do this. It can also output graphs of the overall grading as cumulative per cent passing, individual per cent retained and contribution of each sieve fraction to the Gap Index as defined above. Of course the Gap Index itself is also output, together with total cost of materials, sand per cent and strength if a formula has been provided.

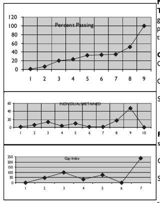
The author was then tempted to introduce the following further features:

1 A maximum and minimum limit on each aggregate.
2 A limit on the gap Index.
3 Optionally replacing the cement content by an input strength.
4 Optionally replacing the water content by a formula taking account of MSF, air and cement content, plus input slump, temperature.
5 Shape and silt content of aggregates affecting water content.

With all the above, the spreadsheet does not work so well at times. It may produce answers that are obviously not optimum (i.e. they will be optimum within the range considered but the range considered by the program may not be wide enough to obtain a true optimum). However it is still a useful adjunct to evaluating materials.

The user can input the grading, SG, and cost, of up to 8 coarse and 5 fine aggregates in columns G, H and L to T of the spreadsheet (Figs 3.7A, B). The maximum per cent in column E of the spreadsheet can be set to zero for rows left blank or materials to be excluded from a particular trial run.

A



NOTES: This version does not include limitation on Gap Index or make use of silt% or shape factor
TO USE first move cursor to a blank cell, then go to Tools in top row, select solver,key Solve in top RHS of Solver pop-up
gradings are to be entered as % passing in columns L to T, rows 8 to 21, costs and SGs in cols G & H
particular materials can be switched off by entering 0 in column E, feel free to make entries in col F to change starting point
they need not add up to 100%, system will correct **INPUT VALUES IN COLS F 29-34 &G 35-37, DO NOT OVERWRITE ANY FORMU**

SlumpWTR	-4.48		
AIR% WTR	-16.70		
Conc Tmp WTR	0.00	Agg Vol	824
		Sand%	33

Reqd EWF	23.8
Reqd SS	27.5
Reqd sand p	0.33

		% by Mass					ShapeFctr			Grading,%passing									Volume of	Propn of	Propn by Volume of	Total	Total	
COARSE		**Min%**	**Max%**	**USED**	**SG**	**COST**	>36%voids	Silt%	SS	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**	CAorFA	CAorFA	Total aggs	SILT	RWDCA	COST
CA%	1	0	0	**0.0**	2.75	28	0	0	5.22	1	1	1	1	1	1	1	5	95	0.00	0.00	0			0
67.0	2	0	0	**0.0**	2.65	30	0	5	7.34	1	1	1	1	1	1	5	50	95	0.00	0.00	0			0
Check total	3	0	100	**0.0**	2.65	44	0	0	10.01	1	1	1	1	1	1	15	98	100	0.00	0.00	0			0
100.0	4	0	100	**100.0**	2.65	11	0	0	6.25	1	1	1	1	1	1	3	28	100	37.74	1.00	0.669967			16.1
SS	5	0	0	**0.0**	2.65	20	0	0	7.00	1	1	1	1	1	2	10	30	100	0.00	0.00	0			0
6.8	6	0	0	**0.0**	2.65	30	0	0	10.26	1	1	1	1	5	20	20	30	100	0.00	0.00	0			0
	7	0	0	**0.0**	2.65	444	0	5	12.66	1	1	1	1	5	20	35	60	100	0.00	0.00	0			0
	8	0	0	**0.0**	2.65	111	3	0	42.40	1	10	20	35	50	65	95	100	100	0.00	0.00	0			0
FINE				AVG>			RWD			**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**	37.74					
sand%	1	0	100	**0.0**	2.65	111	1	20	42.40	1	10	20	35	50	65	95	100	100	0.00	0.00	0			0
33.0	2	0	100	**0.0**	2.65	11	1	6	59.20	1	5	35	70	85	95	100	100	100	0.00	0.00	1.21E-11			3E-10
Check total	3	0	100	**0.0**	2.65	111	1	6	65.58	1	15	40	80	90	98	100	100	100	0.00	0.00	0			0
100.0	4	0	100	**100.0**	2.65	25	1	6	70.66	1	20	60	70	98	100	100	100	100	37.74	1.00	0.330033			18.02
SS	5	0	100	**0.0**	2.65	55	1.1	20	81.07	2	33	70	95	100	100	100	100	100	0.00	0.00	0			0
70.7			TOTAL	100.0															37.74		1		0.00	34.12
			TOTAL	100.0																				

Grading,Individual % retained										Grading,contribution to % retained, by vol										SS Cont	Grading,contribution to combined % retained by vol										AVG SG	
PAN	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**	**PAN**	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**		**PAN**	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**	2.65	
1	0	0	0	0	0	0	4	90	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	0	0	0	0	0	4	45	45	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	0	0	0	0	0	14	83	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	0	0	0	0	0	2	25	72	0	1	0	0	0	0	0	2	25	72	0	6.25	0.67	0	0	0	0	0	1.34	16.7	48.2	0	178	
1	0	0	0	0	1	8	20	70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	0	0	0	4	15	0	10	70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	0	0	0	4	15	15	25	40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	9	10	15	15	15	30	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
							TOTALS		6.8	1	0	0	0	0	0	2	25	72	0.0	6.25												
1	9	10	15	15	15	30	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	4	30	35	15	10	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1E-11	0	0	0	0	0	0	0	0	0	0	
1	14	25	40	10	8	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1	19	40	10	28	2	0	0	0	0	1	19	40	10	28	2	0	0	0	0	70.7	0.33	6.27	13.2	3.3	9.24	0.66	0	0	0	0	87.5	
2	31	37	25	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
PAN	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**	**PAN**	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**	70.7	**PAN**	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**		CHECK
										3	19	40	10	28	2	4	50	144	%RETD		1	6.27	13.2	3.3	9.24	0.66	1.34	16.7	48.2	0		100
									70.7	1	19	40	10	28	2	0	0	0	SS/%PASS		27.51	1	7.27	20.5	23.8	33	33.7	35	51.8	100	100	SS>
																			GAP INDEX			0.07	48	98	35.3	73.6	0.46	237				493
																			Sieves:		**PAN**	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	**9.5**	**19**		
																						10.5	3.55	13.4	7.51	16.1	15.4	0				66.5
																			Desirable new agg grading,%retd:			15.8	5.34	20.2	11.3	24.2	23.2	0				100
																			Desirable new agg grading,%pass:				15.8	21.1	41.3	52.6	76.8	100				

%RETD	6.3	13.2	3.3	9.2	0.7	1.3	16.7	48.2	0.0	%RETD	
%PASS	1.0	7.3	20.5	23.8	33.0	33.7	35.0	51.8	100.0	SS/%PASS	
GAP INDEX	0.1	48.0	98.0	35.3	73.6	0.5	237.4	493	GAP INDEX		Sand%
Sieves:	**75**	**150**	**300**	**600**	**1.16**	**2.36**	**4.75**	Sieves:			33

OUTPUT			**INPUT**
COST	56.57	20	STRENGTH
SAND%	33	24	MSF
GAP INDEX	493	80	SLUMP
CEMENT Q	149.7	20	TEMP
WATER Q	128.3	0.95	WATER FACTOR
		2	AIR%
	CEMENT> COST	150	
	SG	3.15	
	STRENGTH FACTOR	1	

Figure 3.7A The Shallard spreadsheet.

B

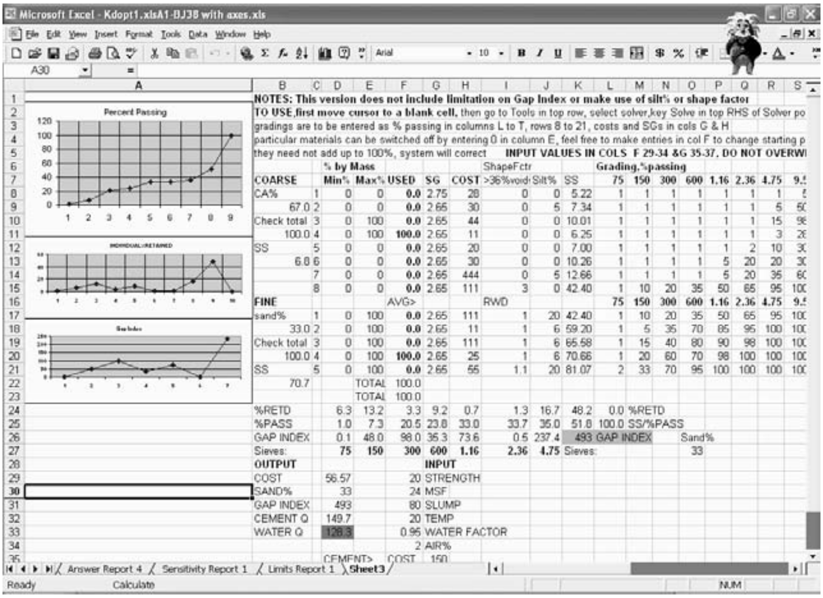


Figure 3.7B Entry and output section of spreadsheet in Fig. 3.7A.

The required strength, MSF, slump, temperature, water factor (covering use of admixtures and effect of fine aggregate particle shape) and airpercent are entered in rows 29–34 of column F.

The cost, SG, and strength factor of the cement are entered in rows 35–37 of column G (this allows for the use of blended cements).

The system is activated by going to ‘tools’ in the top row of the spreadsheet and selecting ‘Solver’ (obviously your Excel spreadsheet must have the Solver add-in). This causes the screen shown in Fig. 3.8 to pop up and Solve can be keyed in the top RHS corner, causing the answers to appear in rows 29–33 of column D and the selected proportions in rows 8–21 of column F of the main spreadsheet.

Even though the spreadsheet shown does not include Gap Index limitation, or the automatic inclusion of silt content and particle shape in the water estimation, it still does not find the optimum selection of materials and set of proportions in many cases. Nevertheless it is a very useful adjunct to manual consideration.

The user can look at the spreadsheet and conclude that a more economical answer might be obtained by a different selection of materials. This opinion can be tested by entering a large number (even 100) as the percentage of the favoured materials in column F and either entering zero in column F for a non-favoured material or actually excluding it by entering zero in column E.

The program will adjust any input percentages so that they add up to 100 % for each of coarse and fine combined materials and also so that the nominated

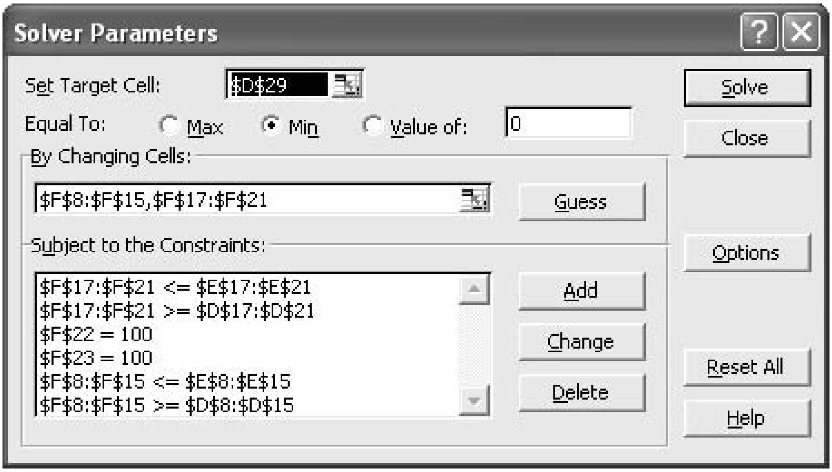


Figure 3.8 Solver set-up for spreadsheet.

MSF is provided. It does this very reliably and also accurately calculates the cost of the combination. So, although the program does not reliably provide the best answer, it certainly saves a great deal of time and effort in testing the user's opinions.

When the program is working well (e.g. in its original form), it is fascinating to alter costs and/or gradings of some of the materials to see how the program reacts. To some extent it is possible to see exactly how much the price of a particular material would have to be reduced to cause the program to select it in preference to its current choice – or conversely, how much the price of a material it has chosen to include could increase before the program would reject it, or use it in reduced proportion. Costs of course can be in any units, relative costs are what is required. If desired they can be distorted to include an 'inconvenience cost' for some aggregates. For example it may be that an additional fine sand would certainly involve an extra bin, perhaps it could be used if highly beneficial, but double or triple the true price may be entered to ensure that it would not be selected by the program unless really necessary.

To avoid further complication, if a cement replacement material such as fly-ash were to be used, it should be entered as a blended cement with an appropriate SG and strength factor.

The snag is that, for the solutions to be really valid, all the additional factors really need to be taken into account – and when they are the program is over-extended. However playing with this program is a good way for a novice to gain an understanding of what economically fine-tuning a mix really involves.

We are still some distance from designing a mix since the cementitious content, water content and admixtures remain to be discussed, but the above enables a selection of the available aggregates to be addressed.

Actual design of mixes – I

The above relates to the preliminary selection of materials where a substantial choice is available, rather than to actual design of mixes. The following design methods could be used repetitively with different materials to see which answer is most economical but this would be a much lengthier process than the above. However, where there is a close finish between different solutions, it may be worth carrying out the full design process with more than one group of materials.

The second edition reported five programs. Of these ‘The Basic System’ (described as ‘One Mix Fine Tune’ in the ConAd suite of programs) has proved too complex and limited for anyone to use and is not included here. Two more ‘Cement Margins’ and ‘Benchmark’ are now regarded as QC rather than mix design and reported in chapter 4. This leaves ‘Automix’ and the ‘Mixtable’ programs. To these can now be added ‘Relational Mix Maintenance’ and ‘Just-in-Time’ mix design in addition to the free program described earlier. The latter three are now reported in Section 5.1, Integrated Mix Design and QC.

All of these mix design techniques have things in common:

1 A database is required of all materials to be available. This includes aggregates and cementitious materials.
2 A database of created mixes is to be retained for future analysis.
3 A database of every batch of concrete produced is required.
4 A database of all tests on the resulting concrete is required and is to be integrated with the actual batch quantities in (3) above.
5 A formula is required to determine the w/c ratio necessary to provide an input strength. It is desirable that this should include a feedback factor to improve its accuracy as test data becomes available.
6 A formula is desirable to predict the water requirement of a designed mix. The formula will certainly require a feedback or adjustment factor.

It is now easy to obtain accurate cement contents for every mix batched and to link this with strength and workability test data. Accurate records of batched and subsequently added water can be obtained and both moisture probes and accurate physical tests for moisture content of aggregates are available. It seems that it should be possible to obtain an accurate water content from a sample of fresh concrete by either a volumetric analysis from water displacement or directly by microwave drying and the author has done substantial work on the former. Nevertheless it remains difficult to obtain accurate and reliable water content data and this remains the biggest difficulty in assessing the accuracy of mix design programs.

Materials database: aggregates

All constituent materials test data is entered, preferably as it is produced. For example ConAd allows actual sieve masses to be entered and automatically calculates percentages passing and retained, specific surface (Figs 3.9, 3.10 and 3.11) (and fineness modulus and logarithmic mean size, although ConAd does not currently use them). Flow value and bulk density from a flow cone test on sand (see Section 7.1) can also be entered since the author considers them to be of likely future significance. Past entries can be viewed graphically or in a table (Fig. 3.10 or 3.11) and the computer can produce the latest grading on any nominated date, or the average over any nominated period etc.

It is also possible to include a cusum graph of any entered property on the same screen as strength and other cusums – which is one place where flow values and/or bulk density could already be used if available.

Materials database: cementitious

All data appearing on cement test certificates should appear as dated records in the cement database. As with aggregates, any item in the database can be selected for cusum graphing along with strength test data in a search for change point correlation.

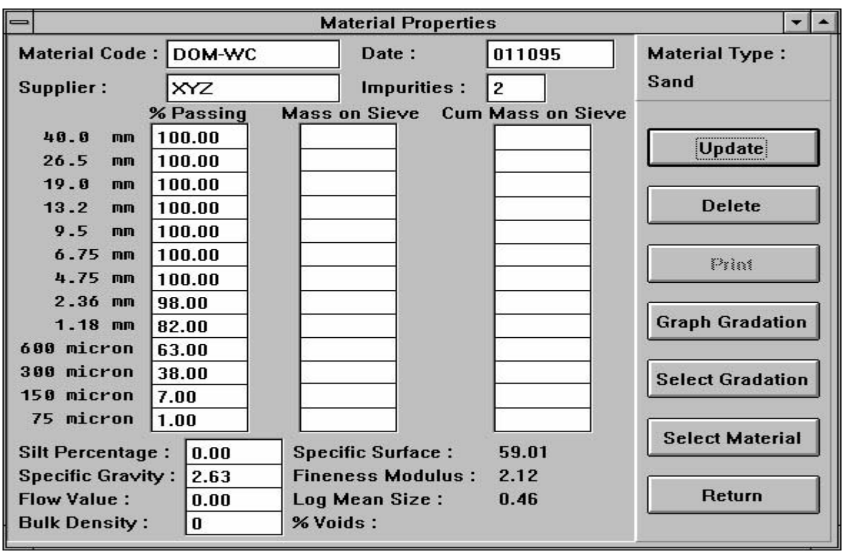


Figure 3.9 Material gradings.

Gradation Selection

Material	DOM-WC	DOM-WC	DOM-WC	DOM-WC	DOM-WC	DOM-WC	DOM-WC
Date	01/12/94	15/12/94	01/01/95	01/04/95	01/06/95	01/07/95	01/08/95
Supplier	XYZ	XYZ	XYZ	XYZ	XYZ	XYZ	XYZ
40.0 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
26.5 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
19.0 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
13.2 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
9.5 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
6.75 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
4.75 mm	100.00	100.00	100.00	100.00	100.00	100.00	100.00
2.63 mm	97.00	97.00	97.00	98.00	97.00	97.00	98.00
1.18 mm	81.00	82.00	80.00	82.00	82.00	82.00	82.00
600 micron	68.00	68.00	65.00	63.00	63.00	63.00	63.00
300 micron	38.00	38.00	38.00	38.00	38.00	37.00	38.00
150 micron	6.00	7.00	6.00	7.00	8.00	7.00	7.00
75 micron	1.50	1.50	1.00	1.00	1.00	1.00	1.00
Silt %	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Spec Grav	2.63	2.63	2.63	2.63	2.63	2.63	2.63
Spec Surf	59.49	59.85	58.80	59.01	59.14	58.67	59.01
Fine Mod	2.10	2.08	2.14	2.12	2.12	2.14	2.12
Flow(sec)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Bulk Dens.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
% Voids	0.00	0.00	0.00	0.00	0.00	0.00	0.00
LM Size	0.45	0.44	0.46	0.46	0.46	0.46	0.46

○ Cementitous ○ Coarse Aggregates ◉ Sands ○ Admixtures

Cancel

Figure 3.10 Material gradings listing.

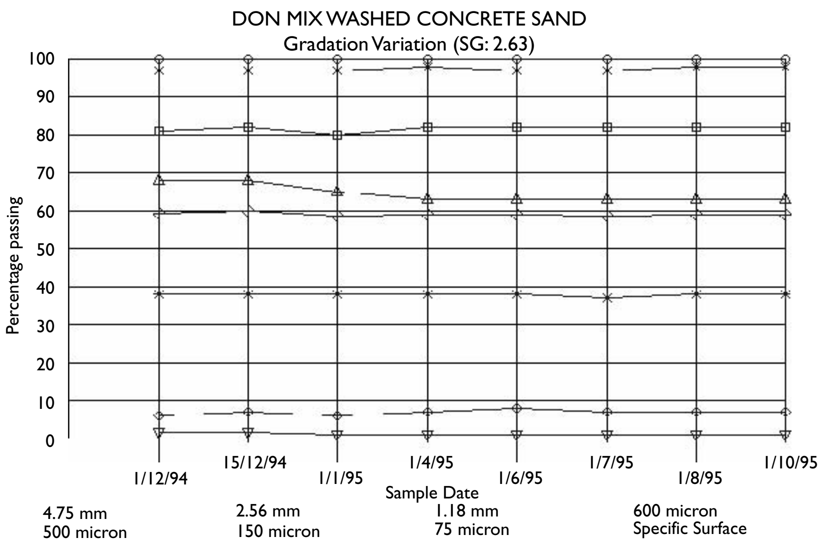


Figure 3.11 Sand grading variation over time.

The only items actually used by the mix design system are likely to be a strength factor, a water factor and a cohesion factor. These are more likely to be opinions rather than test data although the user may choose to automatically relate these to actual test data by a formula of his own devising. The system will be more concerned with relativities than absolute values because the QC system will feed back correction factors. So if only a single cement and no cement replacement materials were used, the values could all be left at one. Where alternative cements, and especially materials such as fly-ash, slag and silica fume are in use, relative factors are required.

In the second edition the assumption was that such factors would remain constant over a range of proportionate additions, and this is built into the Automix and Mixtable programs. Analysis of production data has shown that this is not the case. It is a reasonable assumption for a simple mix design but can cause too much inaccuracy when trying to base feedback correction on a limited number of early age tests covering a wide variety of cementitious combinations – as for Just-in-Time mix design. The solution adopted has been to regard each combination as a separate cementitious material – so that cement plus 20% fly-ash will be analysed as a separate cement to cement plus 30% fly-ash. A 'wide variety' of cementitious combinations does not necessarily mean an unworkably large number of them.

The Just-in-Time mix design system is the author's latest effort. A paper of this title was presented in Cancun, Mexico (Day, 2002) in December 2002 and more details appear in Section 5.1.

Water requirement

The most difficult aspect of mix design is the prediction of water requirement. So many factors are involved that there is a temptation to nominate a likely value and simply adjust this from time to time as experience dictates. However water content is directly proportional to cement content for a given required strength, and so to the economy of the mix. Also water content variation is usually the largest factor involved in the variability of test results, again impacting on the required mean strength and therefore the economy of the mix. So it is necessary to be as aware as possible of all the factors affecting water demand from both the initial design and the quality control viewpoints.

The author's approach is to list as many of the influences as possible, to provide an empirical correction for each, and then to provide an overall adjustment factor. The user should be prepared to make a correction (as a percentage or otherwise) to any of the individual terms that appear to over or under estimate the effect but it is essential that the overall correction factor be adjusted by feedback from test data.

Any change in water content will certainly be reflected in both the strength and density of the resulting concrete. A change in slump or other workability measurement may or may not reflect a change in water content for example an increase in temperature may cause a reduction in workability at a given water

Table 3.4 Factors affecting water content

Source of effect	*Effect on water requirement (ls/m^3)*
Basic water content	85
Grading effect	$+ 3 \times$ EWF
Slump effect	$+ 0.36 \times$ (slump) $- 0.0007 \times$ (slump)^2
Entrained air effect	$- 5A \times 250$/total cementitious content
Concrete temperature (°C) effect	$- 0.1 \times$ (temp) $+ 0.02 \times$ (temp)^2
Silt content effect (combined sands)	$+$ [(silt % $-$ 6) $\times$ (wt sand)]/300
2nd cement/pozzolan	$-$ factor $k_2 \times$ wt of material
3rd cement/pozzolan	$-$ factor $k_3 \times$ wt of material
Quantity of cement	$+$ entered factor $\times$ amount out of entered range

=SUM

SUM × Water factor = Total water requirement (excluding absorbtion)

content or an increased water requirement at constant workability. This is why it is so important to use a cusum of density along with strength cusums in quality control graphing.

The author's corrections are tabulated in Table 3.4 and discussed below:

Basic Water content – derived from experience, feel free to adjust.

Grading effect – this may exaggerate the effect a little.

Slump effect – a figure is needed for other means of measuring workability but is not currently available.

Entrained air effect – it is important to note that this effect, which can be as much as 10% of total water, is included in their claims of water reduction by admixture suppliers. So where 13% reduction is claimed, it may only be 3 or 4% more than already allowed for in this expression. The author's experience has been mainly in countries where frost resistance is not a problem and air contents (for workability improvement only) therefore low. It is possible that the effect of high air contents is exaggerated by this term.

Concrete temperature effect – Strictly speaking, higher temperature does not increase immediate water requirement (as can be demonstrated by making a mix with fly-ash only – it will not set, but neither will it require more water for a given slump when hot) rather it increases the rate of hydration, especially in the first few minutes. So water requirement for a given workability a short time after mixing does increase with temperature. It is interesting to note that this occurs in the first few minutes after the cement and water come into contact, essentially during mixing. As the author has pointed out (Day, 1996b) the subsequent slump loss during delivery is *not* higher in hot weather because most of it has already occurred by the time of dispatch. Of course the stiffening due to further hydration does occur earlier in hotter concrete, unless suitably retarded.

Silt content effect – in the author's MSF, fine silt or clay only counts along with other material passing the 150 micron sieve. It could perhaps be included with cement as regards its effect on water requirement but a different treatment is preferred. A small amount of silt actually appears to be slightly beneficial if anything, but beyond this, water demand increases by about 1 litre per 1% of silt. It is important to note that the silt percentage is that derived from a field-settling test. This percentage will be the same as that by weight for crusher fines dust but may be three times the amount by weight for clay in natural sands.

Quantity of cementitious material – if the amount of cement differs from the amount needed to fill the voids in the sand, additional water will be needed. If the difference is a shortfall, the water will be needed to fill the remaining space. If the difference is an excess, the water will be needed to lubricate the additional cement. I call this ‘the Dewar correction’ since the effect (but not my crude compensation for it) was pointed out by Dr J. D. Dewar. Generally a range of 250–350 kg of cement will not require extra water and the additional amount may be one, or even two, litres per 10 kg outside this range.

Normal consistency of cement – if a cement requires more water for normal consistency, it will require more water in the concrete.

Pozzolanic materials – fly-ash in particular will normally reduce water requirement. A figure of 15 litres reduction per 100 kg may be obtained but some fly ashes (especially coarser varieties) may even increase water requirement.

Particle shape, coarse – a crushed coarse aggregate will have a larger percentage voids and therefore require a higher fines content. However the extra water requirement arising from this will be taken care of by the increasing MSF, which is how the additional fines content is implemented. Depending on how badly shaped the aggregate is, an increase of up to 3 in MSF may be required. A very smooth rounded gravel on the other hand may merit a reduction of 1 or even 2 in MSF. Note that the sharpness or angularity of the particles, rather than their shape appears to be the main factor.

The effect of smaller or larger coarse aggregates is similar to that of particle shapes. The standard is taken as 20 mm. A reduction in MSF would be made for larger aggregates and an increase for smaller. The variation may be 1, or rarely up to 2, in MSF value.

Particle shape, fine – a crusher fines (‘manufactured sand’) or a sharp pit sand will increase water requirement. This is the same effect as with the coarse aggregate for example an increase in void space, but in this case the additional void space is filled with water. The effect of angularity in fine aggregate can range up to 10% water increase for a badly shaped crusher dust (with the actual dust content still separately allowed for by the settling test and the grading by its specific surface). A figure of 7% may be more normal for good crusher fines and 2 to 4% for a very angular (‘sharp’) natural sand.

A very rounded fine sand, such as a wind-blown dune sand, can act like ball-bearings, effectively lubricating a mix whereas its grading may suggest a substantial water requirement. Such a (relatively rare) situation may be better handled by an arbitrary reduction of the order of 5% in the specific surface value calculated from the grading. This will cause a higher proportion of such a sand to be permissible according to the system. It would only be done if the fine rounded ('dune') sand was cheaply available, but this is normally the case with such sands. The figures quoted are from the author's own experience. It should be noted that hearsay evidence from experimenters with the sand flow cone (see Fig. 7.3, p. 189) suggests that angularity and surface roughness can add up to 15% to water requirement.

Admixtures – the effect of admixtures is directly taken into account in the overall adjustment factor, along with fine aggregate particle shape. Warning has already been given that percentage water reduction claims by admixture producers usually include the reduction caused by entrained air. So it may be appropriate to anticipate a reduction of around 5% rather than something over 10% for a normal water reducer. However there are certainly high range water reducers capable of giving a 25% or more reduction.

For a normal water reducer and a crushed fine aggregate the water factor could be $1.00 - 0.05 + 0.07 = 1.02$. With a natural sand, the figure may be 0.95.

The use of air entrainment is particularly beneficial with crushed fine aggregates.

Actual design of mixes – II

The above extensive dissertation is an attempt to have the reader understand the factors involved in designing a concrete mix as much as to actually assist in the design. The author, and many others, have now produced computer programs that will design a mix from a few input figures. However if you do not start by selecting the most economical materials (not necessarily either the cheapest, nor the best quality) your concrete may not be competitive, and if you do not understand all the factors involved, you may have difficulty interpreting the output of your quality control system.

Automix

The simplest of the author's commercial computer programs (now marketed by Command Alkon) is Automix.

This computer program is aimed at providing a very user-friendly design program at the cost of some loss of features compared to the basic ConAd program. The main features lost are feedback of production data and shape correction, however feedback of test data is achieved in a different way through transference to the following Mixtables program.

The program goes some way towards being based on ideal gradings for those who do not feel comfortable with complete freedom to nominate the relative proportions of several coarse aggregates or two sands to each other. However it still uses specific surface to determine the ratio of total sand to total coarse aggregate.

In one concept, an ideal sand grading is one whose grading is normally distributed on a logarithmic scale. For example when plotted in terms of percentage retained on the normal sieve size X-axis, the result is a normally distributed histogram. No numerical penalty is known to be incurred if the grading is not normally distributed, but there may be a greater risk of segregation, bleeding or increased water requirement. The question of suitability may be better assessed in terms of percentage voids or flow time (see Section 7.1) but the normal distribution concept may be of some assistance in assessing the optimum combination of two sands where neither of these tests is available. For any given mean size (i.e. logarithmic mean size) it is possible to nominate a desired percentage passing the 75 mm sieve or alternatively to nominate standard deviation or coefficient of variation. Any of these permits calculation of a family of normally distributed gradings, one for each mean size.

Another alternative criterion is the old UK sand grading zones (illustrated in Fig. 3.12).

The Automix program is able to cycle through these alternative sets of criteria while retaining the input individual gradings and the current combination on screen. For any set of guidance curves Automix, on keying 'Calc' will cycle through each curve and every integer combination of the two sands from 4 to 1 to

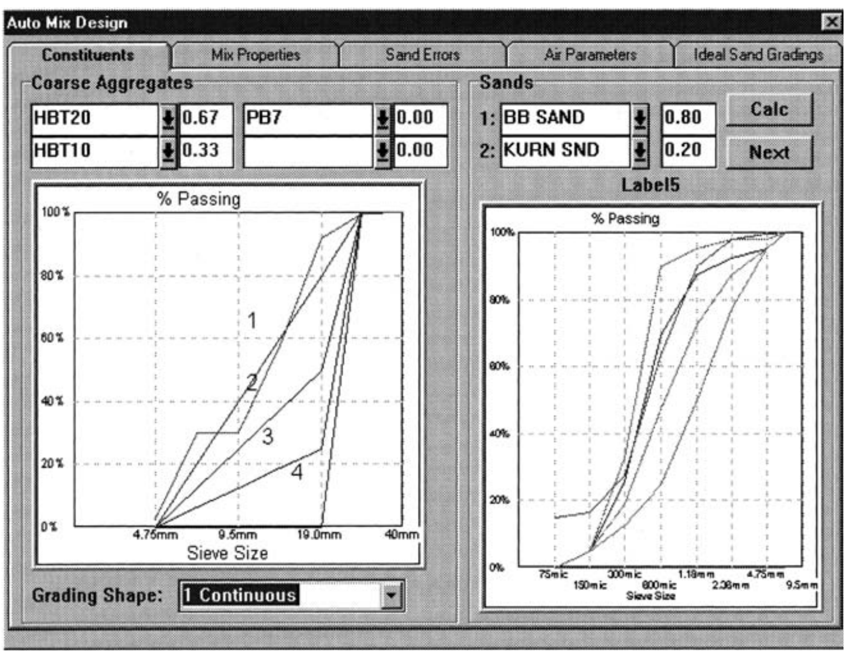


Figure 3.12 Automix constituents screen.

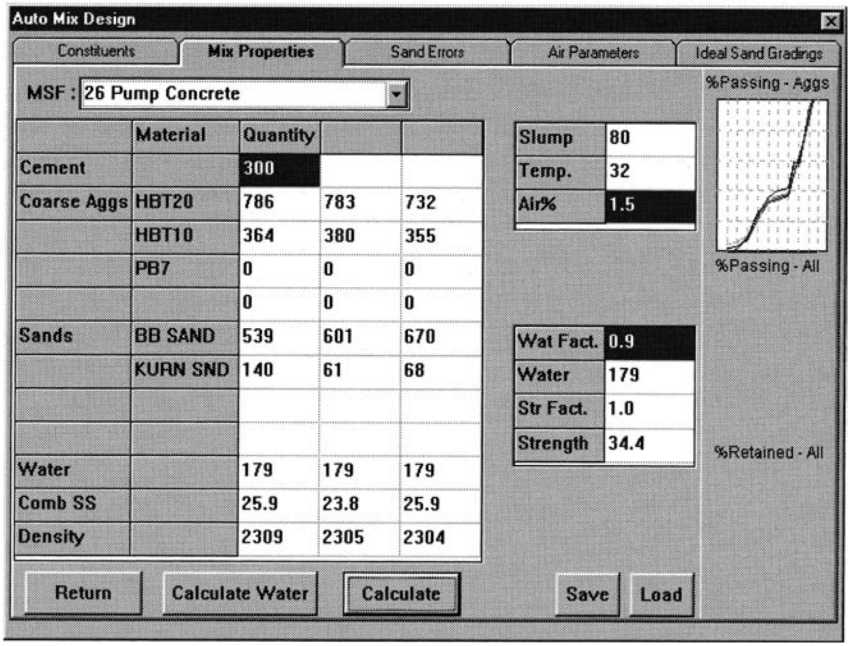


Figure 3.13 Automix mix properties screen.

1 to 4 to find which gives the closest match to one of the curves. However the user can input any desired combination and cycle through the background curves to form an independent opinion of its suitability.

For coarse aggregates no theory is advanced and the user merely selects from the available four options of continuous, semi-continuous, semi-gap and gap. The curve resulting from the combination selected is shown superimposed on the four optional curves. Again the user is able to input an alternative combination.

The user now goes to a second screen (Fig. 3.13) where the mix will actually be designed. The desired type of concrete is specified in terms of its MSF from a pull-down menu. This menu describes the type of concrete, which will be produced alongside each MSF number (e.g. Harsh Mix for low slump precast at 22 and Sandy Flowing at 30). However these are to some extent matters of opinion and users should feel free to nominate their own preference of MSF number for the particular work in hand once they become familiar with the fresh concrete properties to be anticipated from a given MSF number (any desired number can be keyed in rather than selecting one from the table and the descriptions in the table can be edited by the user).

Appropriate values are entered for slump, airpercent, and concrete temperature. The default figure for water factor is 0.95 (i.e. a 5% water reduction appropriate to the use of a normal water reducer), this value may be overwritten as desired on the basis of the user's own experience with the proposed materials.

As explained earlier the program makes an assumption that the water requirement calculated will apply over a limited range of cement contents, which the user is able to specify. A conservative range is 300 to 350 kg but a wider range may apply. The user is able to specify a rate per 10 kg of cement outside this range at which water content will increase, but a default value of 2 litres per 10 kg is suggested.

When air entrainment is employed, it is assumed that, in addition to reducing water content generally, the air will assist in filling the sand voids and so will avoid the increased water content otherwise to be anticipated in the low cement case (but not in the high cement case).

It now remains only to key 'calculate water' followed by 'calculate' for the program to proportion the mix. It does so by calculating the proportion of combined sands to combined coarse aggregates so as to yield the specified MSF. The program compares this result graphically with the grading resulting from combining the two curves the program was trying to match. The gradings resulting from both the calculated mix and the target grading are shown on three thumbnail graphs: per cent passing, aggregates only; per cent passing all materials; and individual per cent retained, all materials. Each thumbnail may be expanded by right clicking on it. The expansion will revert on releasing the right click key but may be retained on screen by moving the cursor off the expanded graph before releasing the key.

This system is intended to provide guidance and simplicity of operation for new or inexpert users. The mix designed can be saved in a database and recalled into the following 'Mixtables' program for expansion into a whole range of mixes. However with practice and expertise, the user may go straight to the Mixtables program (see Chapter 5).

Commercial mix design (by Dr Alex Leshchinsky)

Alex is a former associate of the author at his former company Concrete Advice Pty Ltd, and subsequently has experience of presenting courses and acting as a consultant to readymix producers. Here he offers advice based on that experience. It introduces aspects not considered by typical mix designers and might be regarded as a cautionary tale of errors in logic by those who should know better:

The objective of the commercial mix design is to maximize profitability of a ready-mixed concrete producer while delivering concrete of specified quality. In order to achieve this objective, the following recommendations should be taken into consideration in designing of mixes.

1 *To design mixes to customer satisfaction*: Concrete specifications stipulate only requirements set up by designers. In the majority of cases, customers

of ready-mixed concrete producers are concreters, which have their own appreciation of concrete quality. This relates to concrete appearance, its pumpability, finishability, setting time, etc. Meeting of these requirements is as important as the specification ones, otherwise a ready-mixed concrete producer will lose customers or will be forced to sell concrete at lower prices. For permanent customers, who use techniques different from the rest, sometimes a ready-mixed concrete producer should even set up special customer mixes.

2 *To use market prices for concrete ingredients produced by associate companies*: The ownership of ingredients for concrete determines the main goals of ready-mixed concrete companies in this business set-up, which are as follows

- To efficiently utilise materials produced by other group's divisions and
- To generate profit from the sales of concrete.

There is another situation, where ready-mixed concrete companies do not have their own ingredients for concrete and buy these ingredients from others. The profit for these companies comes only from the sales of ready-mixed concrete.

The differences in these business situations, and in the sources of their profits, determine their different strategies in relation to selection of concrete ingredients. Ready-mixed concrete companies, which do not have their own ingredient sources, usually can choose concrete ingredients at their own discretion. Ready-mixed companies, which have their own ingredient sources, are bound by a necessity not only to buy ingredients from their own sources but also to utilize those of their own ingredients (aggregates), which are in surplus.

The profitability of ready-mixed concrete companies with own ingredient sources depends not only on their performance but also on the prices for ingredients (so called internal prices or transferred prices), which they pay other divisions of the group. If these internal prices are in line with market prices for the ingredients, then the profitability of a ready-mixed concrete company realistically reflects its performance. Otherwise, the picture of profitability could be distorted. For example, a group produces its own cement, which is used by a consortium's ready-mixed concrete company. This cement is less efficient than cement of a rival cement company; that is more cement is needed for the same strength in concrete. However, internally the group sells its cement to its concrete division at the same price as the market price for the rival cement. This means that a cubic meter of concrete produced by the ready-mixed company, which is a part of the group, is dearer than the one produced by their competitor(s) using the cement of the rival company. In this case, the profitability of this ready-mixed concrete division, which is a part of the consortium, is artificially underrated. Some believe that it does not matter since money anyway will stay within the group. This thesis could be disputed, since on the basis of the performance of different divisions, consortia make their decisions on future directions, including investments. Such a distorted picture of the actual performance of ready-mixed concrete companies frequently leads to incorrect (costly) decisions.

3 *To investigate the performance of ingredients for the specific concrete application*: It is very important to know and to understand the performance of every concrete ingredient, since the same concrete ingredient could perform differently when used:

- For different strength grades. For example, some interground slag cements do not perform well in concrete above 40 MPa. Another example – effect of very fine fly-ashes (as cement replacement in terms of strength) usually increases with an increase in strength grade.
- At a different content level. For instance, the same water-reducing admixture could act as an accelerator (when used at 50% of a normal rate), a neutral set one and a retarder (at high dosages). Another example – a different performance of fly-ash depending on its content. The first 20–30 kg/m^3 replaces cement in one-to-one ratio (in terms of strength) regardless of fly-ash quality. Above this initial content, the quality of fly-ash as cement replacement becomes important.
- In combination with other ingredients. The use of silica fume suppresses pozzolanic reactions of other supplementary cementitious materials, GGBFS (Aïtcin and Neville, 2003) and fly-ash (Montes, 2005).

Ignorance of these matters often results in unnecessarily increased concrete cost and/or problems with concrete quality, which also incur further costs.

4 *To rationalize the selection of ingredients and the use of plant storage*: The cheapest ingredients, which provide the required concrete performance, should be used in mixes. For instance, there are circumstances when concrete with only one aggregate size, for example 14 mm, is the cheapest option. A quarry has an excessive stock of this size and heavily discounts it. This low aggregate price offsets an increase in cement content for concrete with this single-size aggregate.

10-mm aggregate is almost always in high demand. Therefore, quarries often offer 20/14 and 7-mm aggregates to ready-mixed concrete producers. But they also need some quantities of 10 mm for special jobs. Some plants have only four ground bins but have to carry three coarse aggregates and two sands (coarse and fine). The solution is to ask the quarry to supply combined 20/14/7-mm aggregate, which is used for 20-mm mixes as a single graded aggregate. A lot of quarries can blend 20/14 and 7 mm through their crushing and screening facilities (with no additional cost) not as a separate additional blending.

For front-end-loader plants, the use of graded aggregates (20/14/7 in lieu of 20/14 and 7 mm) minimizes a number of the ingredients in concrete, which determines speed of a plant (cubic metres per hour) and cost of batching (Aïtcin P.-C. and Neville A., 2003).

A lot of plants still have only two silos and when there are three cementitious materials available (say, Portland cement, GGBFS and fly-ash), there is a need to find out the most economical option.

In other words, selection of concrete ingredients should be determined by

- Cost of concrete with these ingredients
- Type of concrete plant (gravity, front-end-loader, etc.)
- Storage facilities of a particular plant.

In concluding this section, it should be pointed out that experience shows that compliance with the above recommendations allows the production of concrete, which

- Complies with project specifications
- Satisfies the actual customer's requirements
- Has minimum cost.

Often divisions of the same group in some regions are buying its group's ingredients and in other regions buying ingredients (some or even all of them), from other suppliers. Quite frequently, ready-mixed concrete companies buy such aggregates and binders at so called 'transfer' prices, which are higher than market prices. Even in the groups that do not have their own raw materials (aggregates and binders) sources, quite often there are shelf-companies, which buy raw materials and then resell them to their concrete operations at higher prices.

It should be stressed that mix cost optimization should be done on the basis of actual market delivered material prices. The use of the transferred, or any other way artificially adjusted prices, could, and often does, result in losses for the overall business. The thesis that internal prices do not make any difference for the total group performance is incorrect. The internal prices must be market prices otherwise groups of companies lose money and the larger is the business, the larger the loss.

Let's support this statement with an example.

Scenario 1: A quarry division of a group of companies sells sand (500 Kt pa) to its concrete division at a price of \$25/t. This quarry division sells the same sand to external concrete producers at \$17/t, although each of them is buying far less than the concrete division. There has occurred the shortage of sands on the market and the external concrete producers would like to buy more sand at approximately the same price, \$17/t. The other option for external concrete producers would be to look at other sand suppliers. The quarry division can't produce more sand, but has a stock of crusher fines at one of its quarries, which the quarry division can't sell and these crusher fines could replace the sand. The concrete division is prepared to take the crusher fines at \$5/t, which is the current market price for this product. By taking the crusher fines, the sand will be released for the external sales. However, this pretty logical solution does not interest the quarry division, since the sand, which is currently sold at $25/t (although internally) will be sold only for $17/t (although externally). Even the additional revenue from the crusher fines sales will not offset this loss of revenue for the quarry division. So, as a result of it, the external concrete producers will be buying sands from other sand suppliers, which is a strictly speaking a market share loss for the quarry division.

Scenario 2: The situation is completely different if the concrete division buys the sand at the market price, which is let's say $15.5/t, since its purchase (500 Kt pa) is much higher than that of the external concrete producers. With the shortage of the sand, the concrete division starts buying the crusher fines at $5/t, releasing the sand at $15.5/t, which is sold at $17/t to the external concrete suppliers, bringing $1.5/t profit improvement for the quarry division. Under this scenario the quarry division also benefits from

- the sales of crusher fines ($5/t) and
- the reduction in the quarry cost due to elimination of a necessity to move the crusher fines from the crushing and screening plant to a stockpile. This cost is of the order of $0.5–1/t of the moved quarry product.

It should be stressed here again that usually one of the main objectives of concrete divisions that are a part of raw materials' groups of companies, is to consume raw materials (concrete ingredients), which groups' divisions (quarry, cement, etc) can't sell to an external market. In addition to the above advice, that all concrete ingredients shall be sold to concrete divisions at market prices, concrete divisions have to produce concrete at the lowest possible cost obviously without compromising concrete quality. This means that for instance, a concrete division does not have to increase its cement consumption only because its cement company can't sell the cement; the cement company (the group) should look at other markets including an option of acquiring more concrete operations.

Another important point should be mentioned. The optimization of concrete mixes in terms of their costs shall not be done at the expense of a failure of meeting any customer requirements. First, a ready-mixed concrete producer has to meet customer requirements and only then, to reduce concrete cost maintaining the achieved concrete performance.

The reasons for blending different cementitious materials together are as follows:

- To enhance concrete properties
- To reduce concrete cost.

The majority of ready-mixed companies in the world are parts of consortium, which also produce concrete ingredients, mainly, aggregates and binders (like cement, GGBFS, fly-ash).

The objective of this original course is to present both properties of concrete ingredients and concrete from the commercial standpoint in other words how they influence concrete profitability for the specified quality. This should help to choose most effective ways of concrete cost reduction.

Commercial concrete technology considers factors, that are outside the scope of traditional concrete technology, for instance,

- Prices for concrete ingredients
- Ownership of the ingredients' sources
- Plant's storage capacity
- Plant's location and the range of mixes produced
- Customers perception etc.

3.4 The ConAd system

The ConAd system, originally developed by the author but now owned by Command Alkon Inc., takes the technique well beyond the basic specific surface/MSF principle reported earlier in this chapter. It incorporates the Automix program described in Section 3.3 and the integrated mix design and QC described in Section 5.1. It is especially strong on QC and on the use of huge quantities of automatically acquired batch plant and QC data for management and production engineering purposes. The system was extensively described in the second edition of this book and in several papers by the author (available on his website www.kenday.id.au).

Development of the program continues under the new owners, who took over the author's entire staff in acquiring rights to the program. It is anticipated that this will eventually incorporate the 'Just-in-Time' option described in Section 5.1. The author sees it as particularly appropriate and desirable that ConAd should be owned by, and integrated with the products of, a leading batch plant system developer. However he has very recently joined a new partnership with Shilstone and Contex to further develop his concepts.

A feature of the system for many years has been feedback from major system users. The latest such joint development, 'Relational Mix Maintenance' is described in Section 5.2 by Mark Mackenzie of Hanson. Mark has had a long association with ConAd, starting with Alpha in South Africa in the 1990s and continuing with Pioneer in Australia prior to their takeover by Hanson.

3.5 Alternative methods of mix design

It is clear that strength is at least approximately governed by w/c ratio and that sand fineness and proportion affect both water requirement and ease of handling. A system is required to decide how much of a particular sand is required (and whether more than one sand or coarse aggregate is required) and what the water content, and therefore the cement content, will be.

Leaving aside published tables, there are four approaches to the problem of aggregate combinations:

1 Try to match a published grading curve, considered to be an ideal grading.
2 Use a proportion of sand selected by fineness modulus and the bulk density of the coarse aggregate. (ACI Method)
3 Use a computer program based on packing density using mean particle size and percentage voids (or conduct trials with varying percentages of fine to coarse aggregate to find maximum density experimentally). (Dewar or de Larrard method)
4 Use a method based on specific surface, which may or may not be computerized.

1:2:4 Mixes

At one time it was common to nominate concrete as one part cement, two parts sand, four parts coarse aggregate (or 1:1:2 when stronger concrete was needed). As will become apparent later, this mix would be satisfactory only with a particular sand grading and therefore led to the specification (in the UK) of ‘Class A’ sand, a restricted grading envelope of sand which made good 1:2:4 concrete. On both sides of the Class A envelope was a further envelope called ‘Class B’ sand – sand which made reasonable but not good concrete if the 1:2:4 proportions were retained (Fig. 2.2).

In 1954 Newman and Teychenne (Newman and Teychenne, 1954) showed that equally good concrete could be produced from the Class B sand providing the relative proportion of sand to coarse aggregate was adjusted appropriately. They proposed the division of sands into four grading zones instead of two classes. Sand as a percentage of total aggregates was to range from 40% with the coarsest (Zone 1) sand to 22% with the finest (Zone 4) sand. Zone 2 at 33% is the old 2:1 ratio and Zone 3 would require 25% of sand (Fig. 3.15).

Although Newman and Teychenné allocated sands to the four zones on the basis of percentage passing the No25 BSS sieve (ASTM 30, Metric 600 μm) they did indicate that specific surface would have been a preferable basis except for the difficulty of measurement (see Sections 2.5 and 3.4).

The author’s system owes a great deal to this paper. The grading zone concept has now been dropped in favour of the BRE System (see below).

Ideal grading curves

Many investigators have put forward ‘ideal’ grading curves, either as actual curves or as mathematical formulas. Prominent amongst them were Fuller and Thompson (USA) (Fuller and Thomson, 1907). Bolomey (France) (Bolomey, 1926) modified the Fuller and Thompson formula to include cement and to vary

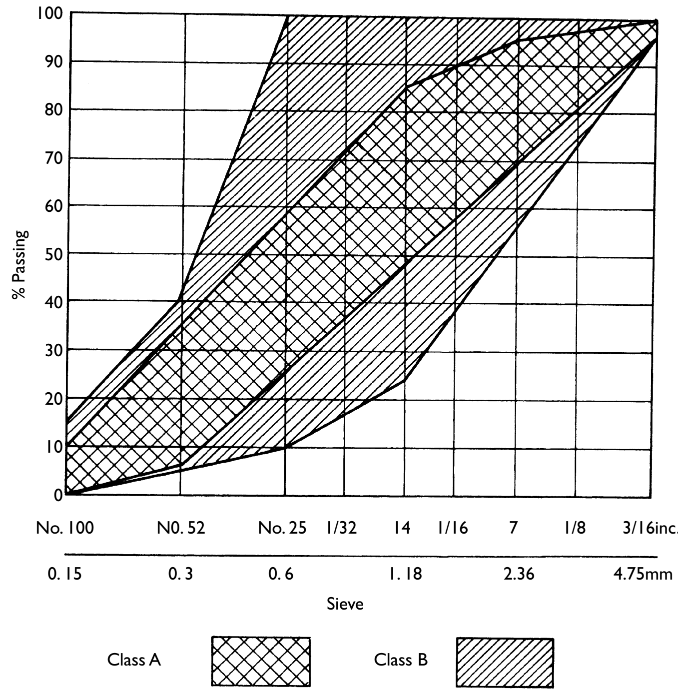


Figure 3.14 Class A and B grading zones (B.S. 882/1944 concreting sands).

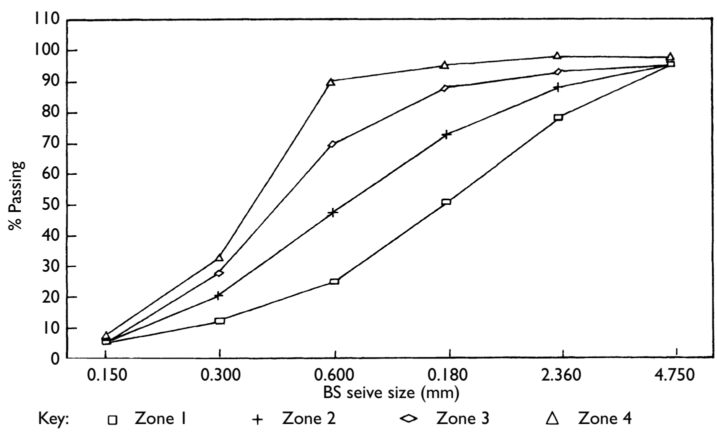


Figure 3.15 British sand grading zones (mean values).

the grading according to the desired workability and the aggregate particle shape (see Section 7.1 for further details).

The weakness of the ideal grading approach is that it is rarely possible (or economical) to replicate exactly the ideal grading in the field. Also the grading may be ideal for one use but could not simultaneously be ideal for all uses.

Gap gradings

There have also been many proponents of the use of gap gradings, for example D. A. Stewart (Stewart, 1951). The technique is to use a large, often single sized, coarse aggregate (often 40 mm) and a relatively fine sand. With such a combination it becomes valid to measure the voids in the coarse aggregate and provide just sufficient mortar to fill them, with a small surplus.

There is no doubt that gap-graded concrete compacts more rapidly under vibration (Plowman, 1956) and a given strength can usually be obtained more economically (at least if cement content is the only cost criterion) with a low slump, gap-graded mix. However several factors often militate against such mixes. The first, as with ideal continuous gradings, is that suitable aggregates may not be economically available. The second is that gap-graded mixes have a strong tendency to segregate at anything more than low (say 50 mm) slump. Although such concrete is easier to consolidate than a continuously graded mix of similar slump, it is sometimes difficult to convince workmen of this and water is frequently added with disastrous effects.

In short, gap-graded mixes can be unbeatable when used by those familiar with such mixes, and in suitable conditions, but are not to be recommended for general use.

Another property of gap-graded mixes is that, with a very stable coarse aggregate, very low drying shrinkage is attainable. This is taken to the ultimate in 'pre-packed' concrete. This technique involves filling the formwork to be concreted with a large single-sized aggregate and then pumping in an appropriate mortar from the bottom up. Since the coarse aggregate is everywhere in contact, shrinkage is not possible except as aggregate moisture movement. Such concrete is very suitable for use as a foundation block for large pieces of machinery; the concrete often being placed after the machine has been set in position (vibration being unnecessary).

Exposed aggregate finishes are a matter of taste but in the author's opinion there is no more attractive finish than that obtained with heavily gap-graded concrete, for example a concrete with a high proportion of a large, single-sized coarse aggregate and a small proportion of a relatively fine mortar.

Road note no. 4

For many years, in the 1940s, 50s and beyond, this was the accepted UK system. It offered tabulated data based on an extensive trial mix series at the Harmondsworth Road Research Laboratory (RRL, 1950).

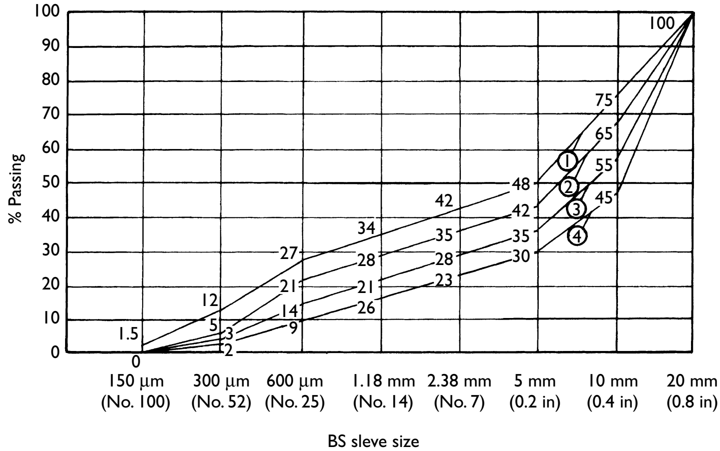


Figure 3.16 Road Note 4 reference gradings for 0.75 in (20 mm) maximum size aggregate.

Four alternative gradings were included so that the user could choose to use a harsher or sandier mix. These 'type grading curves' are still used as noted below.

The tabulated data not only covered four gradings but also three different maximum sizes of aggregate (40 mm, 20 mm and 10 mm) and two different particle shapes. The system was purely empirical and so could not be readily adapted when admixtures came into use and cement properties changed. As coarse sand became less readily available it became harder to match the grading curves. The fact that the system dealt with aggregate/cement ratio rather than batch quantities per cubic metre (or per cubic yard) became inconvenient with the rise of ready-mixed concrete.

However the tabulated or graphed gradings (Fig. 3.16) have long survived the demise of the actual system, being generally used (including by the author) as a frame of reference as to what constitutes harsh and soft gradings.

See Chapter 7 for further detail of sand grading zones.

BRE/DOE system

The British replacement for Road Note 4 was 'Design of normal concrete mixes', published in 1975 by the UK Department of the Environment (DOE) (i.e. the Building Research Establishment and the Transport and Road Research Laboratory). The system is attributed to D. C. Teychenne, R. E. Franklin, and H. C. Erntroy and clearly owes much to Teychenne's work on specific surface. It relates the percentage of a fine aggregate to its grading and the w/c ratio and accurately copes with a very wide range of fine aggregates. It is also up to date in terms of the relationship between water/cement ratio and strength and copes well with adjustments to this relationship and to water requirement on the basis of trial mixes. The latest (1988) version (DOE, 1988) does allow for air entrainment and the use of fly-ash and ggbfs but does not provide a choice of harsher or softer mixes or readily give an accurate yield or density. This version bears the BRE logo on the cover so the system may be found described as either the DOE or the BRE system.

The basis of this system in concrete technology is almost identical to that of the author's ConAd Mixtune system, even though the design process is completely different. It is, therefore, interesting to examine the techniques used in some detail and assess the relative advantages and disadvantages of the two approaches.

The most obvious and major difference is that the DOE system is presented for manual operation using tabulated and graphical data whilst the author's system is computerized. However there is no reason why the DOE system should not be computerized and the author's system could be presented manually. If these changes were made, the DOE system would work a little more accurately than it now does, in interpolating values from graphs and tables. The author's system, as seen in Chapter 3, would require a substantial amount of calculation or the provision of design aids in the form of graphs or tables. This clearly illustrates the point that computerization allows an elaboration of the technological basis without detriment to the ease of use.

It is possible that, given a brief to produce a computerized system, the DOE team would have produced something very similar to the author's system. However, if the author were required to produce a new manual system, he would graft the specific surface technique onto the ACI bulk density system (see 2.7 below) and would still have a more elaborate water prediction system.

D. C. Teychenné (together with A. J. Newman) (Newman and Teychenné, 1954) was essentially the person from whom the author learned the specific surface theory. However, although the theory is still the fundamental basis of both systems, the author and the DOE team have gone in different directions from using exact specific surface. The 1975 DOE system used sand grading zones and the 1988 version substitutes percentage passing the 600 micron sieve as their simplified approximation. (Obviously this cannot be as accurate as true specific surface but was selected as a balance between simplicity and accuracy.)

The author found that even true specific surface did not give a sufficiently accurate prediction of water requirement and therefore originated his 'modified specific surface' (MSF, see Chapter 7). Even in a manual system, the additional effort involved is minuscule and certainly does not justify the DOE simplification. It may be concluded that the DOE simplification was considered worthwhile because true specific surface still did not provide great accuracy so that little was lost by the simplification. It may also be that the simplification was attractive in terms of avoiding the need to promote the concept of specific surface, which has a long history of rejection and disbelief over the last century (see Section 3.2).

The mechanism of selection of fine aggregate percentage is illustrated in Fig. 3.17. This figure is for 20 mm maximum aggregate size.

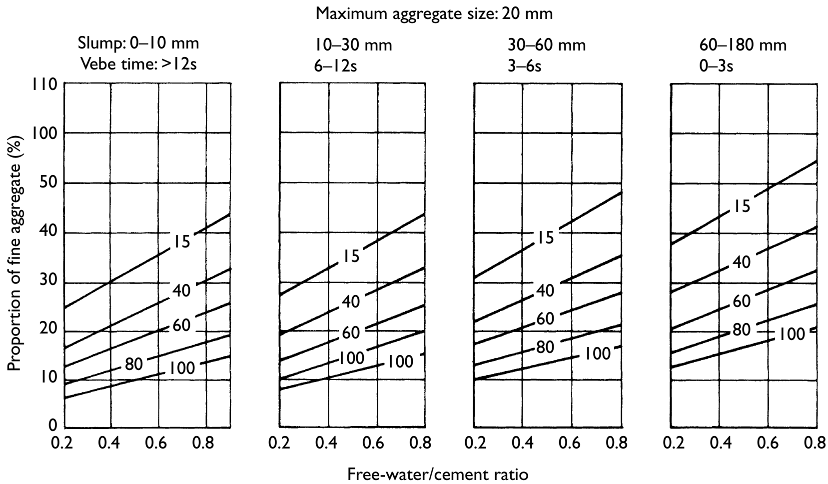


Figure 3.17 Selection of fine aggregate per cent.

Table 3.5 Required water content (BRE)

Slump *Vebe time (s)*		*0–10* *>12*	*10–30* *6–12*	*30–60* *3–6*	*60–180* *0–3*
Maximum size of aggregate (mm)	*Type of aggregate*	*Water content (kg/m^3)*			
Part A Portland cement concrete					
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205
Part B Portland cement/pfa concrete					
Proportion 'p' of pfa to cement plus pfa (%)		*Reduction in water content (kg/m^3)*			
10		5	5	5	10
20		10	10	10	15
30		15	15	20	20
40		20	20	25	25
50		25	25	30	30

The BRE booklet also provides similar charts for 10 mm and 40 mm maximum sizes. The difference between the recommended percentages of a given fine aggregate differs more between the different maximum sizes than this author would consider desirable.

It can be seen that a higher fine aggregate per cent (and therefore a higher surface area, giving greater cohesion) is used for higher slumps. At one time, the author's system automatically related specific surface to slump in the same way, but this was found to be too rigid, even though normally desirable. Fine aggregate per cent is also related to cement content, that is to w/c ratio at a given water content. This is the same result as obtained by inclusion of cement as the author's EWF and MSF (see Chapter 3).

The tabulated water contents are shown in Table 3.5. This is partly of interest for comparison purposes and partly to show the treatment of pfa (fly-ash).

The remaining interesting technique used is that of combining the tabulated strength data with a dimensionless series of w/c – strength curves (see Fig. 3.18).

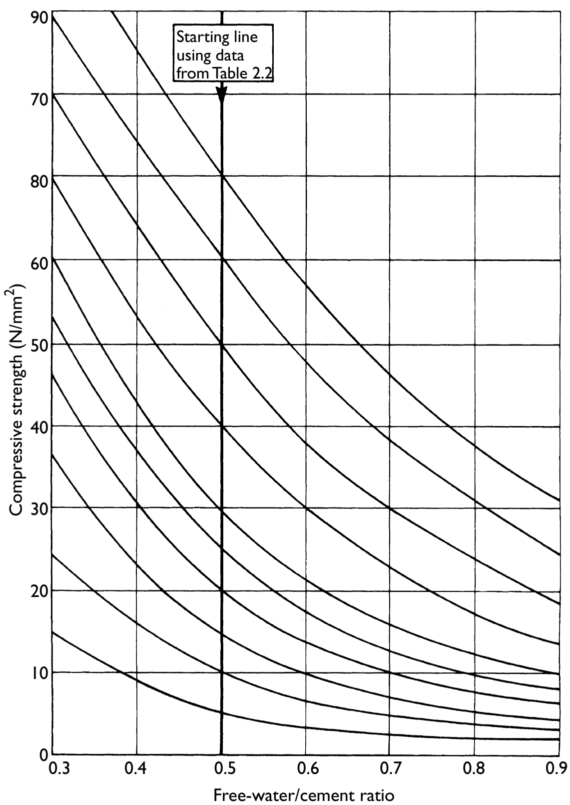


Figure 3.18 Strength – w/c curves.

The technique is to enter the graph on the 0.5 w/c line with the appropriate tabulated strength value. An adjustment to any other strength or w/c value can be made by moving parallel to the printed curves. The same graph can also be used for adjusting values in accordance with actual test results.

The table provided for cement/pfa mixes gives identical 28 day strengths but substitutes w/(c + 0.3f) for w/c ratio, that is the fly-ash is discounted to 30% of the cement strength value. This is excessive in this author's experience with Australian fly ashes. The table offers no opinion on strengths at earlier or later ages than 28 days but presumably these would be lesser and greater respectively, than those for normal Portland cement.

The ACI system

The American Concrete Institute (ACI) (ACI 211, 1991) system is no doubt the most widely used system in the world and has a number of good features. The principal such feature is the use of the bulk density or unit weight of the coarse aggregate as a starting point. This very neatly allows, in one number, for the combined effect of grading, specific gravity (particle density) and particle shape of the coarse aggregate on the desirable sand content (Table 3.6). The sand content is further varied on the basis of the sand fineness modulus of the sand (see Chapter 7) and the absolute volume of cement, water and entrained air. In effect the volume of all other ingredients is established and the balance is taken as sand.

The system does not provide for selection, at the user's choice, of other than the tabulated proportion of coarse aggregate but it is not invalidated by this being done. Water content prediction takes into account only slump, maximum aggregate size and whether or not air is entrained. The *tabulated strength* v *w/c ratio figures* are very conservative indeed. Given accurate specific gravity figures, yield is automatically exact by this system.

Table 3.6 ACI table for proportioning of coarse aggregate

Nominal maximum size of aggregate (mm)	*Volume of dry-rodded coarse aggregate[a] per unit volume of concrete for different fineness moduli of fine aggregate*			
	2.40	*2.60*	*2.80*	*3.00*
9.5	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
37.5	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.80	0.78	0.76
150	0.87	0.85	0.83	0.81

Note

a Volumes are based on aggregates in dry-rodded condition as described in ASTM C29.

The system can be quite readily computerised and the author (as a former member of ACI Committee 211, the revising committee for the document) has been advocating for several years that the committee do this officially. What is missing from the system is a recognition that different degrees of sandiness (cohesion) are appropriate for different uses. This could readily be provided in the form of a multiplying factor for the tabulated values of proportion of coarse aggregate, which could be called a ‘cohesion factor’.

The other weak aspects of the system are the tabulated water requirements (Table 3.7) and the assumption that strength is solely dependent on w/c ratio (Table 3.8). If these defects were remedied and the system computerized, it would be a strong competitor to the author’s system. There would be no difficulty in replacing the fineness modulus of the fine aggregate by specific surface in deciding upon (i.e. calculating) the proportion of the bulk density (or unit weight) of the coarse aggregate to be used. It should also be noted that the latest version of ACI 363 (high strength mixture proportioning) contains an adjustment for predicted water requirement based on per cent voids in the fine aggregate. This has yet to flow through to ACI 211 (normal mixture proportioning) but could be an important improvement.

Trial mix methods

The most widely used formal trial mix system is that used in the UK by the British Ready Mixed Concrete Association (BRMCA).

The initial trial mix uses an a/c ratio typical of the range likely to be supplied in practice. The fine to coarse aggregate ratio is adjusted by eye until optimum plastic properties are obtained. A range of mixes with varying cement contents is then prepared, and water requirements and strength obtained, at a given slump are determined. The data is then plotted to enable interpolation of properties at 5 or 10 kg increments of cement content.

While the above sounds crude, the actual detailed process is very carefully specified and has been found to give repeatable results. Drawbacks of the process are:

1 The need for laboratory facilities and, more importantly, expert personnel.
2 The time and cost involved.
3 While the system is very flexible in coping with strength variations (the scale already exists up or down which the cement content can be varied) it cannot cope with changes in aggregate properties (if the sand grading changes, the whole process must be repeated).
4 The only way of considering the relative merits of alternative aggregates is to carry out the whole process with both sets of aggregates. It would be very tedious and expensive to consider all possible permutations of several coarse and fine aggregates in this way.

Dr J. D. Dewar has devised a computerised simulation of this process.

Table 3.7 ACI 211 water requirement tabulation. Appropriate mixing water and air content requirements for different slumps and nominal maximum sizes of aggregates (SI)

Slump (mm)	*Water (kg/m^3) of concrete for indicated nominal maximum sizes of aggregate*							
	9.5	*12.5*	*19*	*25*	*37.5*	*50*	*75*	*150*
	Non-air entrained concrete							
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	—
Approximate amount of entrapped air in non-air-entrained concrete (%)	3	2.5	2	1.5	1	0.5	0.3	0.2
	Air-entrained concrete							
25 to 50	181	175	168	160	150	142	122	107
75 to 100	202	193	184	175	165	157	133	119
150 to 175	216	205	197	184	174	166	154	—
Recommended average total air content, % for level exposure:								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
Extreme exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5	4.0

Table 3.8 ACI strength versus w/c ratio

Comprehensive strength at 28 days (MPa)[a]	*Water/cement ratio by mass*	
	Non-air enterained concrete	*Air-entrained concrete*
40	0.42	—
35	0.47	0.39
30	0.54	0.45
25	0.61	0.52
20	0.69	0.60
15	0.79	0.70

Note

a Values are estimated average strengths for concrete containing not more than 2% air for non-air entrained concrete and 6% total air content for air-entrained concrete. For a constant water/cement ratio, the strength of concrete is reduced as the air content is increased.

Dewar – particle interference and void filling

Dewar (Dewar, 1986, 1988, 1999) has developed a comprehensive theory and an associated mathematical model of particle mixtures that has been validated for powders, aggregates, mortars and concretes. The theory is a development of ideas generated by Powers (Powers, 1968), in particular the use of the parameter voids volume per unit solid volume of particles.

The essence of the theory is that when particles of two different sizes are mixed together the benefit of reduction in voids caused by the smaller particles filling the voids between the larger particles is partially offset by interference in the packing of both sizes. Dewar has been able to model both effects mathematically into a comprehensive system that has been computerised by Questjay Ltd and SP Computing in the UK.

The operation of the system for concrete requires knowledge of only three parameters for each solid component. These are:

Particle density
- For aggregates – SSD basis
- For powders – Modified kerosene value.

Mean size (on log basis)
- For aggregates – from grading tests
- For powders – from particle size distribution or from fineness test.

Voids ratio
- For aggregates – from loose bulk density tests (in SSD condition) and from particle density
- For powders – from Vicat tests for standard consistence and from particle density.

Knowledge of the mean size of each material enables effects of size ratio to be computed. Influences of the range of sizes about the mean size together with effects of shape and texture are accounted for by measuring the voids ratio of each material.

For a simple mixture of only three components, for example cement, sand and gravel, the computer programme first blends the two finest materials, cement and sand, into the full range of mortars and then blends the mortars with gravel, selecting only those blends that will have adequate cohesion at the selected slump. The resulting concretes cover the complete range of all possible mixtures enabling selection of the most appropriate mixture for any purpose, for example. strength, durability. The results obtained by Dewar from theory correlate well with practice in the UK ready-mixed concrete industry, which needs to have a wide range of economic mixtures always available for instant use.

Fig. 3.19 shows, for several different sets of materials, how the variation of water content of concrete with cement content can be modified considerably by the properties of the materials used. With such variation it is important to know the relationship applicable to each set of materials to be combined together. It will be noted that some relationships are essentially constant over a wide central band but others are far from constant.

Dewar suggests that one of the uses for his programme can be to examine other methods to determine their range of applicability. This could be particularly useful when extending beyond the original range of a method.

By way of example, Dewar examined cursorily a number of methods including an early version of the ConAd system (Table 3.9).

Dewar was able to show generally good agreement between theory and the ConAd system over most of the range. However, Dewar's Fig 3.19 is a warning to

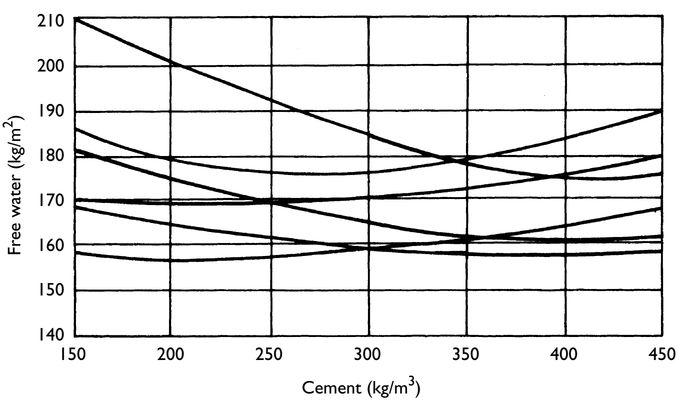


Figure 3.19 Examples of relationships between free water demand and cement content for six sets of materials.

Table 3.9 Comparison of ConAd and Dewar predictions

Parameter	*Method*	*Cement content (kg/m³)*			
		120	*230*	*310*	*420*
Free water (l/m^3)	Theory	186	162	162	176
	ConAd	158	160	161	163
% fines	Theory	51	46	41	32
	ConAd	53	48	44	38

all developers of systems to identify the range of applicability to reduce the risk of significant error.

On the assumption of the validity of the theory developed by Dewar, the question can be raised as to whether different methods can be equally valid at least within a particular range. Private discussions have concluded that although different terminology may be used there may be a hidden common basis in many systems.

For example, the concept of mean size ratio used by Dewar with regard to particle interference, that of specific surface index, and that of fineness modulus, are not identical but they do have common links. Specific surface has the dimensions of m^2/m^3 that is, 1/m, and is thus the reciprocal of linear size. Fineness modulus is determined from grading and is thus also size related. Thus when other factors are constant or are not dominant, apparently different concepts may lead to similar results.

Dewar's main contention against surface area concepts when used on their own is that they do not account for variation in grading about the mean size or for shape or texture, all of which have an influence on water demand because of their influence on voids between particles. Expanding on this, his contention is that water demand has three components (Fig. 3.20);

(a) Water to fill voids close to a particular surface.
(b) Water to fill voids between particles at normalized workability (50 mm slump).
(c) Additional or reduced water for selected workability.

The reason for the differentiation between (a) and (b) is that particle interference reduces the ability of smaller particles to fill voids close to the surface of larger particles compared with their ability to fill voids in 'open' space. There is a close but not identical analogy between (a) and the specific surface concept of water to coat the surface of particles.

Size ratio and thus other size related factors, affect both (b) and (a).

Particle interference and voids have been traditionally minimized by employing a large differential between the sizes of cement, fine aggregate and coarse aggregate, the respective mean sizes being in the order of say 0.015 mm, 0.4 mm and 12 mm that is relative size ratios of about 30.

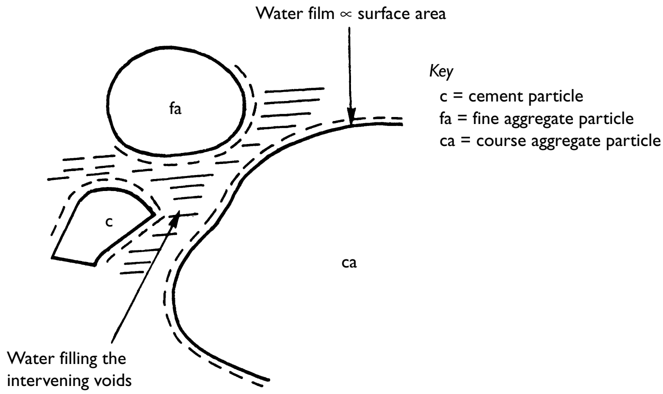


Figure 3.20 Functions of water in filling voids in concrete.

However, even this differential is not sufficient to reduce particle interference to zero and the coarser particles are required to maintain a dilated structure to accommodate the finer particles with consequent increased voidage and water demand.

The above section was kindly contributed by Dr Dewar. It is difficult for the author to compare the results of the two systems because they use different data, for example the author has extensive data on mix designs and their performance and constituent materials but his data do not include the bulk density data used by the Dewar system.

The ConAd system now recognizes that water content will increase outside an ideal range of cement content of the order of 300 to 350 kg/m^3. However ConAd is less concerned with an initial estimate of water requirement and more concerned with its variation as slump, temperature, sandiness, and air content vary. Also ConAd is designed to accept feedback of production data including water contents and to amend predictions accordingly.

Readers are strongly advised to read Dr Dewar's latest book (Dewar, 1999) for an in-depth account of his PhD thesis on mix design. Also S P Computing have produced a user-friendly Windows version of Dewar's system entitled Mixsim. Both book and software are described at www.mixsim.net.

de Larrard: void filling and maximum paste thickness

Francois de Larrard, working at the Ponts et Chaussees (Bridges and Roads) laboratory in Paris, originated a theory basically very similar to that of Dewar, but favoring aggregate void measurement under vibration rather than loose poured as with Dewar. He has also introduced a concept he calls MPT or maximum paste thickness, which appears to account well for the strength reduction (at any given w/c ratio) for mixes with a higher proportion of cement paste. His work includes very extensive mathematical coverage of many types of concrete and many pozzolanic and chemical admixtures and is available in an advanced computerized system.

Dr de Larrard has written a very comprehensive book (de Larrard, 1999), also published by Spon/Routledge, which is highly recommended to readers interested in precise mathematical mix design. He was offered a few pages in this volume (as accorded to Dewar) to briefly summarize his theories but preferred to have the author express his own views.

de Larrard has had the advantage not only of excellent facilities and assistance at the Ponts et Chaussees laboratory but also of collaboration with extensive actual project work, equipment fabrication, material supplies etc. His work has included the origination of the BTRHEOM, a parallel plate viscometer which has been of considerable assistance in his mix design work.

3.6 Mix design competitions

One would imagine that thousands, if not hundreds of thousands, of persons the world over (including many students who have yet to make any actual concrete) would be able to use a standard mix design system (such as ACI or DOE) to design a mix with a compressive strength within ± 50% of a specified target and without using trial mix facilities. It is therefore quite amazing to find entries in an international mix design competition having strengths ranging from 0–220% of the specified strength.

This latest competition (Hanley-Wood's, see the author's website) reinforces the results of the RILEM competition reported in the previous edition of this book (where cement contents for a given strength target differed by as much as 300 kg/m^3) in demonstrating how limited is the knowledge of mix design.

The latest competition was far more interesting than the RILEM one in that mix entries were actually made up in a laboratory and a wide range of aggregates and admixtures, and three fly-ashes (but only a single cement) were available.

One unfortunate feature of the contest rules was that the specified strength was an absolute minimum with mixes eliminated if they did not reach it and with no penalty however much it was exceeded by. This may have been a substantial factor in the average strength of the submitted mixes being approximately 50% higher than that specified (it would have been far more interesting had there been a penalty of say 1% of the points per 1% above or below the specified strength).

Another unsatisfactory feature was that points were awarded for attaining the target slump (of 6″) but the made up mixes were not brought up to this slump and the average slump of all mixes was only 3.6″. This would distort the relative strength results and may partly explain the high strengths. Slump would be affected by mix temperature and mixer and materials preparation (dry drum of mixer and aggregates dryer than SSD?).

A further problem was that the contest rules were changed after the submission of some mixes (including the author's) so that cost of materials as a criterion went from 45% of total points to only 10% and a rheology criterion was introduced at a value of 30% of total points. This was probably a further reason for the over-strength mixes as it was then worthwhile getting zero points for an economical mix in order to score well in strength and workability.

The competition was not intended to be a fair contest, in that US company contestants were able to purchase samples of the materials, carry out multiple trial mixes, and submit multiple entries, whereas this was impractical for individual entrants from overseas. However the objective of the contest should have been to gain knowledge, and this may have been furthered by permitting the trial mixes.

The worst feature of all was that, so far as the author is aware, there has never been a publication of the detailed results of all submissions and the objective of gaining knowledge was therefore subverted. This was in spite of the fact that Hanley-Wood stated, in charging a $250 entry fee, 'Each company entering will receive a complimentary report listing test results at the end of the review process. The retail value of this report will be $500' – Mr Hanley-Wood, I am still waiting and would sue you if you were in Australia!

However the author has managed to obtain at least a partial and anonymous set of results and these appear on his website (www.kenday.id.au), along with his more detailed commentary and assessment. It will be seen there that the author's mix was one that failed to set, which was certainly a learning experience! This was a consequence of interaction between the particular cement, admixture and fly-ash chosen and has to cause a revision of the author's long-standing practice of designing mixes internationally by telephone and telling clients it is OK to put the first truck of concrete into the structure without a preliminary trial. It does appear that, with a different admixture (or even no admixture), or a different fly-ash, or a different cement, the mix would have reached the specified strength and it did have the lowest material cost. A big lesson is that admixture technology is now so complex that even many of the technical representatives of major admixture suppliers are not aware of all the potential problems and their website information cannot be relied upon. However one man did provide the answer, unfortunately after the event, and without permission to quote details, but see (Roberts, 2005) for some idea of the situation.

So the field is still wide open for some organization to run a properly organized and well-reported mix design competition. It might be reasonable for there to be two sections of the competition, one for entries by individuals on a purely theoretical basis, and one for organizations having the facilities, and able to afford the cost, of obtaining samples and carrying out trial mixes. It would be really fascinating if one of the former managed to top the latter!

Chapter 4

Quality control

The author concedes that there are other methods of mix design based on more rigorous theory than his own specific surface technique. The latter is advocated for ease of use and flexibility. However it is not accepted that there is any rational alternative to the use of multigrade, multivariable, cusum quality control as developed by the author. It may be objected that the full system, as embodied in the ConAd software now owned and marketed by Command Alkon, is too expensive for smaller producers, or where specifications do not allow any financial advantage (in the form of reduced cement content and general freedom to adjust mixes) for improved mix design and quality control. However the author has now made a free program providing the basic features of MMCQC (multigrade, multivariable, cusum quality control) available on his website.

The cusum technique was first applied to concrete QC in the UK and the methods in use there do include a multigrade technique but, as explained later, it is both substantially less powerful and more difficult to establish and use. It also does not embody the multivariable features of even the simple free system on the web.

Experience has shown that it is worthwhile to explain the basics and philosophy of the system very thoroughly, rather than merely teaching the simple mechanics of its use.

'Quality control' sounds simple enough but its highest attainment requires that all the nuances of the concept be fully understood and that there is a willingness to discard decades of misconceptions and deleterious practices enshrined in obsolete specifications.

Quality control has nothing to do with setting a high or increased level of quality. The required minimum quality level should be set by the specification, and quality control or quality assurance are concerned with so regulating production that the required quality level is attained at minimum cost.

A common mistake is (or was in the past and still is in some areas) to confuse quality control with check testing. The two have little in common.

A basic quality control concept, as promulgated in the early 1950s by Prof. Juran, is to 'control the mass and not the piece'. It is far more economical to ensure that no significantly defective concrete is produced at a plant than to ensure that no defective concrete is accepted at particular delivery points (but the samples are still taken at delivery points).

As an example, take the City of New York's Dept of Environmental Protection ('CNYDEP'). In the 1990s (and perhaps still?) they were proud of their system to organize testing on all their many sites at short notice and they also employed inspectors to monitor batching. None of this data was analysed, the concept being to accept or reject. And since the producer was not allowed to change the mix, there was no benefit to him in installing an analysis system. The result was a control cost they were proud to have reduced to around $5 per cubic yard.

A typical cost of QC by an Australian concrete producer (based on plant control by the producer) would be much less than half the CNYDEP figure – and the concrete would be less costly in cement since it would be of lower variability and therefore requiring a lower margin between average and specified strength.

We now need to consider the nature of concrete variability, the factors affecting it, and the means available for detecting and controlling it.

4.1 The nature of concrete variability

The distribution pattern

Most investigators agree that strength is at least approximately a 'normally distributed variable'. This means that it can be completely described by a mean strength and a standard deviation, that is, that the percentage of results lying

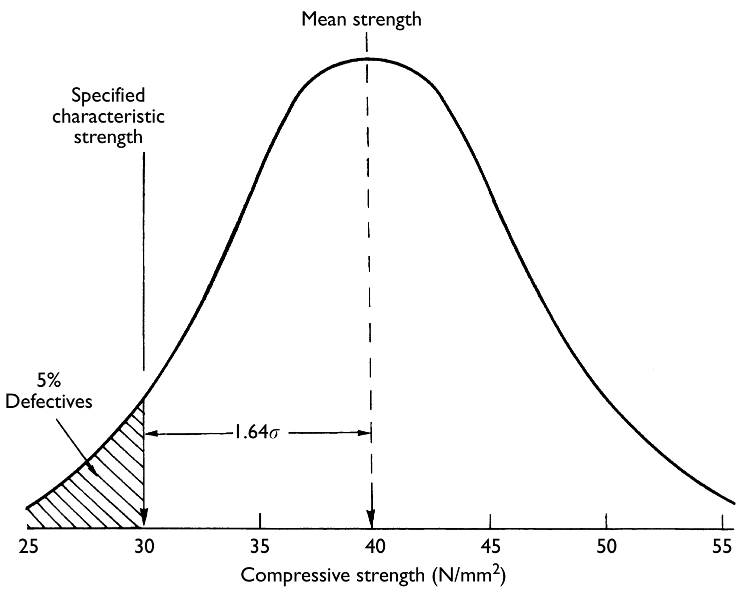


Figure 4.1 The normal distribution.

above or below any particular value can be calculated from the mean strength, the standard deviation and a table of values from a statistical textbook.

The author has found this assumption to be well justified in practice except that only about half the results theoretically expected to be below the mean minus 1.64 σ usually occur in practice.

The formula used is

$$X = F + k\sigma$$

where:

X = required average strength
F = specified strength
σ = standard deviation
k = a constant depending on the proportion of results permitted to be below F.

In USA the 'permissible percentage defective' is usually 10% giving a k value of 1.28.

In most of the rest of the world the percentage is 5% giving a k value of 1.645 (which in UK is rounded to 1.64 and in Australia to 1.65). This gives 28% more benefit from reducing the standard deviation than in USA.

Values of 'σ', the standard deviation, can range from less than 2.0 MPa (290 psi) to more than 6.0 MPa (870 psi) so that the required target average strength can vary by 6 MPa (870 psi) or more according to the degree of control achieved.

Theoretically, results can be expected to spread 3 SDs above and below the mean value (with 1 in 1,000 outside each of these limits). In practice this means that a result 3 SD below the mean has only one chance in 1,000 of not having an abnormal cause.

Some quite experienced persons, including a number of ACI committees, believe that *coefficient of variation*, which is standard deviation divided by average strength, is a more appropriate measure of variability than standard deviation itself. There is certainly a tendency to an increase in testing error at higher strengths, which adds to apparent variability. However having personally produced very high strength concrete at very low variability, the author is not in favour of coefficient of variation and believes that those who favour it are deluding themselves as to the degree of control achieved on their high strength concrete. The truth lies somewhere between constant standard deviation and constant coefficient of variation for high strength concrete and everyone is therefore entitled to their own choice. However the author has routinely analysed, month by month, many thousands of test results from many different suppliers, on many different projects, and in several countries. These results, from any one plant, almost invariably show very little difference in standard deviation in grades from, and including, 20–40 MPa whereas, according to coefficient of variation proponents, those from 40 MPa should be double those from 20 MPa. Some increase is often experienced in grades over 50 MPa but can be avoided by really good testing techniques. No one has yet succeeded in continuously producing 20 MPa concrete at an SD as low as 1.5 MPa but concrete of 100 MPa mean strength has been produced in large quantities at an SD of 3 MPa.

See Chapter 10 for further detail on statistics.

Safety margin

The concrete producer would face a 50% likelihood of his concrete being adjudged at least marginally defective if it was exactly of the intended mean strength and was perfectly assessed (see Chapter 10 for further discussion). Therefore he may decide to add a safety margin of say 1 or 2 MPa (150–300 psi) to avoid such problems. However the cost of such an additional margin would reduce his competitiveness and some of the expenditure may be more usefully directed to reducing variability. In the UK it is normal to use a target strength two standard deviations above the specified strength. This is all the more onerous since standard deviations of 4–6 MPa are apparently normal there, compared to 2–3 MPa for normal strength concrete in Australia. Thus mean strengths are typically 10 MPa above specified strength in the UK and only 5 MPa higher in Australia.

The cost of a safety margin may be unattractive to the producer, as being a large proportion of his profit margin. However the cost of such a margin may be close to negligible compared to the total cost of the structure and the owner of the structure may be well advised to allow a margin by specifying a higher grade of concrete than strictly required (see Section 12.2 'What is economical concrete?'). In the UK all premix suppliers have joined together in QSRMC (Quality Scheme for Ready Mixed Concrete). Amongst other advantages this avoids any competitive disadvantage in the use of a high strength margin.

I have noted that, almost invariably, the percentage lying 1.65 times the standard deviation below the mean is 2–3% rather than the 5% indicated by statistical tables. Why this should be so is not of any importance (perhaps through various kinds of control action such as rejecting overslump concrete or badly compacted test specimens, or perhaps to rapid reaction to any downturn) but it is fortunate that it is, because it reduces the amount of unnecessary concern occasioned by the inevitable lower end of the distribution. Interestingly a UK concrete technologist states that his experience is opposite to this and that he typically obtains *more* than 5% of results below the 1.65 SD level. If correct, this may be a result of the slow reaction of UK cusum to downturns (but see Fig. 4.1 and last paragraph of Section 4.2).

When the spread of results lying above and below the mean value (strictly speaking the Mode' or most frequently occurring value) are unequal, the distribution is said to be 'skew'. This is not a frequent occurrence and if it is encountered, a reason should be sought. If the spread of results is wider on the low side of the mean, some factor is probably truncating the spread of results to the high side. The cause may be genuine, such as a coarse aggregate of low crushing strength or having a smooth, non-absorbent surface leading to bond failure. On the other hand it may be non-genuine such as a defective testing machine or an operator who is afraid of explosive failures. Similarly when results are truncated on the low side (to a greater extent than the 2–3% replacing the theoretical 5% mentioned earlier) the cause should be investigated to ensure that malpractices or extraneous factors are not leading to an incorrect or deliberately biased assessment of the true situation. Another type of abnormal distribution sometimes seen is a double-peaked distribution. This is the result of two separate distributions being combined. It may be that the concrete comes from two readymix plants operating to different mean strengths. It is possible that there is a difference between morning and afternoon shifts (e.g. temperature, slump preference). It is also possible that different testing officers or testing machines give different results. (See the chapters on testing and statistics for more detail.)

When a large number (say 100 or more) results obtained over a period of several weeks are analysed, the assumption is that the mean strength remains unchanged and that variability about it is completely random. If the same number of test specimens were obtained from a single day's concreting, or even more so from a single truck of concrete, it would not be surprising if the variability (standard deviation) were much less. This is because not all the factors causing variability over a period are operative over the single day. In the case of specimens made from the same truck, the variability could be described as the 'testing error' since all the concrete is essentially the same if the specimens have been made from properly remixed multiple samples of concrete spaced during the discharge of the truck.

It is helpful to consider the types of variation that may be encountered:

1. Random variation with no assignable cause. As control improves, the extent of such variation diminishes and an assignable cause is anticipated for any substantial variation.
2. Isolated or non-sustained changes having an assignable cause – for example, an isolated high slump producing a reduced strength.
3. Sustained changes in mean strength.
4. Changes due to testing procedures (i.e. false changes), which again can be either sustained or isolated.

Strictly speaking, statistics only applies to random variations but what matters is not whether or not the statistics are valid but whether the techniques used enable improved control of concrete. The author's experience is that most sets of results over a period can be broken down into sub-periods of consistent mean strength and of lower variability than the overall set. The variability in the sub-period is the basic random variability caused by such factors as batching inaccuracy (including water) and testing inaccuracy. The overall variability is the combination of this basic variability with the variation in mean strength between the sub-periods. As explained earlier, the latter variations almost certainly do have an assignable cause, whether or not the control system is good enough to detect it. The points between sub-periods, at which mean strength shows a sudden or 'step' change, are known as '*change points*' (see Fig. 4.2). The typical extent of a change is of the order of 2–5 MPa or 300–700 psi (which probably only means that changes of much less than 2 MPa are not generally detectable) and it will be seen that their early detection is the basic objective of a control system.

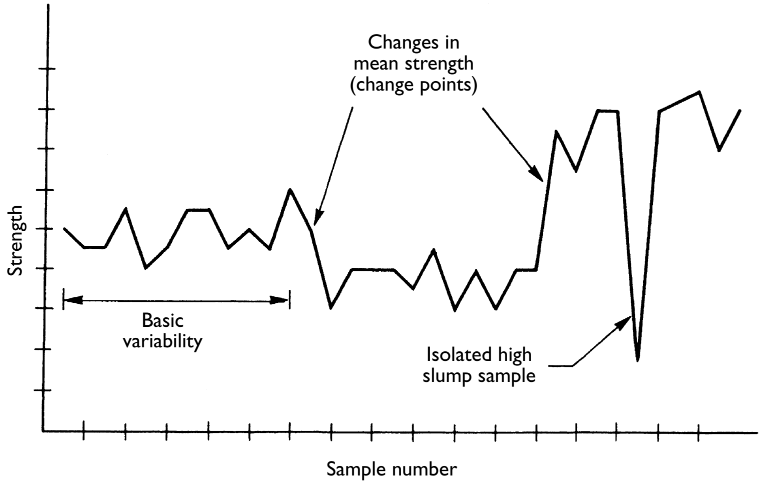


Figure 4.2 Change points and basic variability.

4.2 The objectives of quality control and quality assurance

There are two aspects to controlling concrete quality. One of these is the avoidance of failures and the other the attainment of low variability. Obviously low variability will be of assistance in avoiding failures and vice-versa but it helps to consider the two separately. Equally obviously there will be no failures if there is an adequate margin between the average quality level and the specified minimum.

What is useful is to consider separately those factors acting continuously and those acting intermittently. It is even possible that some of the same factors can fit into both categories, for example, sand grading is unlikely to be identical from truck to truck but there may be a more substantial change from time to time as extraction location or conditions change. It is a difference between variability about the same mean value and a change in mean value. If a change in mean value remains undetected it causes an apparent increase in basic variability.

The continuous basic variability can be thought of as a feature of the production process. It can only be improved by improving that process or the uniformity of the materials supplied to it. The early detection and reversal of occasional change in mean is a feature of the control system. So the control system *measures* the basic variability and *detects* change points. It also *contributes* to the overall variability to the extent that it fails to detect and correct changes immediately.

Apparent overall variability is also increased by error in testing or recording data. This also affects real overall variability in that, by partially obscuring change points, it slows their detection.

The analysis system adopted will similarly have an effect on overall variability through the speed at which it is able to react to change points. It may also have a substantial effect on basic variability to the extent that it is able to highlight the causes of that variability in such a way as to enable them to be reduced through maintenance or production system improvement.

The details are worth bearing in mind when comparing and contrasting quality control (QC) and quality assurance (QA). Insofar as they differ, QA is concerned with avoiding problems by pre-inspection of materials and certification of implementation of control and production procedures. It could be considered to be aimed more at eliminating change points rather than at their early detection. This could be counter-productive if it does not prove possible to eliminate change points and results in their slower detection. However this argument could be over-pedantic. QA has also been described as documented QC, suggesting that the main difference is only one of record keeping.

The control function consists of monitoring the situation so as to detect, at the earliest possible moment, when either the average quality or the variability of that quality changes or becomes likely to change. The system should then to go on to rectify, or take advantage of, the detected change (depending whether it was for the worse or the better).

The control system should monitor not only the quality of the resulting product but also the input materials, the production processes, the ambient conditions and the accuracy of the testing process.

The above was all being done as quality control by the author decades before the term quality assurance came into vogue. To some extent 'a rose by any other name would smell as sweet' but in so far as there is a difference between QC and QA, it is that QA is necessarily pre-planned and documented as to both procedures and their execution. QA provides an assurance, in the form of certified records, that the established QC procedures have been carried out in full. It is intended that the system should be sufficiently comprehensive to *necessarily* ensure the acceptable quality of the output. While QC *may* also include the same procedures, this is not necessarily the case.

The days of controlling by reacting to whether or not failures are being experienced are hopefully long past (although of course failures cannot be ignored). Statistical analysis is used to establish whether production has been satisfactory over some period of time. However, we need to bear in mind the importance of change points. There are mathematical methods of detecting these but they are less effective than graphical methods.

The problem is to distinguish between genuine change and random variation. A decade ago the author conducted an extensive investigation by computer-generating thousands of random (but normally distributed) sets of data to a series of selected standard deviations, containing a change point of selected magnitude, and analysing these by many different systems. These were extensively reported in the first edition and substantially summarized in the second. Here it suffices to report some of the conclusions:

1 Any system that, on average, detected a change point within 15 results would, again on average, produce a false detection within 100 results.
2 Detection efficiency was directly proportional to the variability of the results and it is impractical to detect a drop in mean strength of less than 0.5 standard deviations.
3 Detection efficiency can be improved by suppressing the influence of random variation. This can be done by analyzing the results in groups of 3, 4 or 5 (in increasing effectiveness) or by using the cusum technique. However not using individual result analysis sacrifices the possibility of identifying the cause of individual variations.
4 As explained later, the clear choice for an analysis system is cusum analysis. Just using a straightforward cusum graph is already three times as effective as normal (Shewhart) graphing but cusum also opens the way to multigrade, multivariable treatment that further greatly enhances its power.

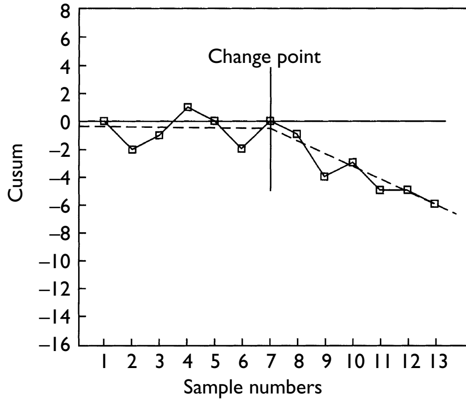
The details are little disappointing. We really do not want *either* to wait 15 results to detect a change *or* to have false detections every 100 results. The solution is to use 'related variables' to establish whether or not an apparent change is genuine. Thus if density, temperature, slump or the like, provide an explanation of a downturn, then it genuine, whether or not statistical analysis confirms this. The author has been using Multivariable' graphs for QC since the early 1950s and writing papers about them for over 40 years, yet the UK cusum system does not incorporate them.

4.3 Cusum charts

'Cusum' is a contraction of *'Cumulative Sum'* (of the difference between each successive result and a target value, preferably the previous mean). By definition the cumulative sum of differences from the mean is zero. So if the previous mean continues to be the mean, a graph of the cusum will have temporary divergences (the extent depending upon the variability of the concrete) but will remain basically horizontal.

However if the mean changes, even by a very small amount, each successive point on the graph will, on average, differ from the previous point by this amount. The graph will still show the same temporary divergences about a straight line, but the line will now make an angle with the horizontal and the *angle* will be an accurate measure of the change in the mean, and the point of intersection of the best straight line before and after the change will pinpoint the time of occurrence (Fig. 4.3).

The cusum technique was developed in the chemical industry (Woodward, 1964) and was first used for concrete QC in the UK in the 1970s (Testing Services Ltd, 1970).



Result	−50	Cusum	
50	0	0	Mean 50
48	−2	−2	
51	+1	−1	
52	+2	+1	
49	−1	0	
48	−2	−2	
52	+2	0	
49	−1	−1	Mean 49
47	−3	−4	
51	+1	−3	
48	−2	−5	
50	0	−5	
49	−1	−6	

Figure 4.3 Simple cusum control chart.

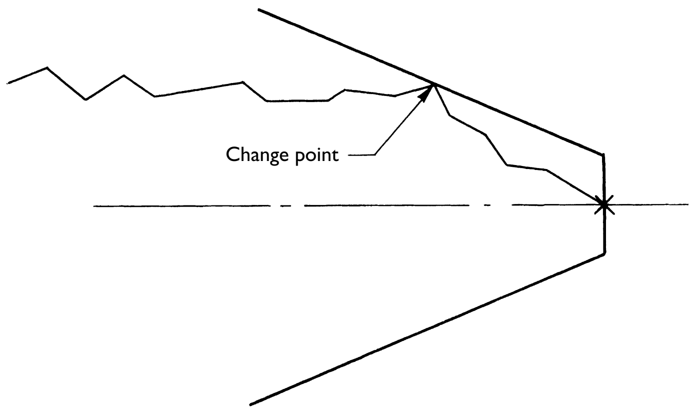


Figure 4.4 Use of V-mask on cusum chart.

The mathematical significance of any particular change of slope can be accurately and simply assessed by the use of a ‘V-mask’ (Fig. 4.4).

The lead point of the V is placed over the last point on the graph and if the graph cuts the V, a significant change has occurred. The V-mask can be a sheet of transparent material carrying a whole family of Vs, each indicating a different degree of significance.

The system was originally adopted as the basis for control by BRMCA and QSRMC although an alternative system involving a countback of the actual number of results above and below the target value of strength is now also permitted (Barber, 1983). A run of nine consecutive results above or below the target value is taken to establish that a change has occurred. Dewar and Anderson state that the alternative is simpler to operate but is 'slightly less sensitive than the cusum method'.

Whether the cusum technique is effective or not depends on a number of factors:

1 The most basic factor is whether changes in mean tend to be isolated 'step' changes or to gradually increase in magnitude. The author's experience is that the more important changes do tend to be step changes, although not invariably and uniquely so. If you draw cusum graphs, you will soon see for yourself the extent to which this is true for your concrete.
2 The change points will be much more clearly visible if the general scatter of points is reduced. It will also become clearly visible from a much smaller number of results after the change, if the scatter is low. However the efficiency of all kinds of control systems are greatly affected by the extent of scatter and in fact the Cusum technique, although substantially affected, is better able to function under high scatter conditions than any other.
3 A significant change, as previously explained, results in a change of slope. An isolated error, or non-significant change, appears as an offset to the slope and can usually be readily discounted by eye examination (Fig. 4.5).

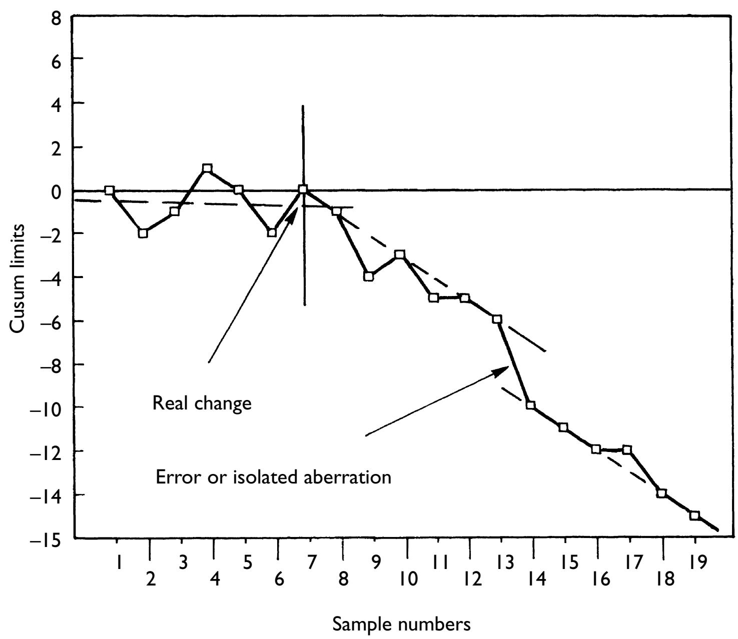


Figure 4.5 Cusum graph exhibiting both real and non-significant changes.

Such offsets may invalidate the use of V-masks on an automatic basis (i.e. assuming an unintelligent operator) but do little to upset the judgement of a skilled interpreter.

4 Obviously the number or frequency of tests and the time delay between testing and interpretation are also significant factors in the rapid detection of change. The author has substantially increased their efficiency through his techniques of combining separate grades into a single graph (Multigrade') and forecasting 28 day strength from early age results.

5 Finally, as previously explained, the confirmation of change detection need not rely on the mathematical analysis of results on the variable in question (as assumed by the UK use of a V-mask). If cusum graphs of 'related variables' such as density, slump and temperature are also drawn, and show a change explaining the change in the primary variable at the same point in time, then the change is certainly confirmed.

The difference made by the author's innovations of multigrade, multivariable cusums and predicted 28 day strength, is substantial. There may be dozens of grades of concrete included on a multigrade cusum, and detection that may, on a statistical basis, require 15 or more results after the change point, is often confirmed after only 4 or 5 results by consideration of related variable cusums.

UK cusum also uses multigrade data for cusum graphing, but it is on a different basis. A correction factor is established for each grade of concrete (mainly based on cement content) so that results converted by this factor can be analysed as though from a basic grade. The problems with this approach are first the difficulty in establishing accurate conversion factors and keeping them up to date and second that, even so, it is found necessary to restrict the grades to be included in a group for this purpose. EN 206 envisages groups of 28 day results spread over more than a year in the extreme, and extensive trial mixes are required.

Essentially the UK system is aimed at quality assurance enabling mixes to be established and remain unchanged over long periods of time. In contrast, the author's system is aimed at feedback quality control enabling day-to-day, and even hour-to-hour, adjustment to reduce variability of results (see Section 4.13 for comparison of EN206 and author's system).

Cusum graphing appears to overcome reported difficulties (Shilstone, 1987) in correlating related variables such as strength and slump. This is because coincident change points on cusum graphs display an instantaneous correlation unaffected by extraneous influences that may interfere with correlation over a period, as in a regression analysis.

The system originated by the author differs substantially from the original RMC version. The system calculates an average value of all recorded properties of each grade of concrete (of which there may be several hundreds). It then calculates a cusum of the difference between these average values and current results in any selected period. The average value of the same property (e.g. strength, density, slump) may differ very substantially from one grade to another but what is being examined is change. Therefore all the differences are cumulatively summed as though they were from the same target.

4.4 The significance of control action requirements

The basic variability in the period between change points is usually a matter of batching accuracy, especially water batching or slump control, made more difficult by minor fluctuations in sand moisture and grading and temperature variations. Thus it is essentially a property of the production process. The extent of additional variability added by the changes themselves is more a property of the input materials and the control system.

In the first place it may be possible to detect and allow in advance for substantial changes in the properties of input materials and the effect of temperature variations. To do this requires both that these changes are detected and that their quantitative effect on concrete strength be known.

In the second place, even if the cause of a mean strength change is unknown, its occurrence can be detected, and be compensated for, by a change in cement content. Time is of the essence in making such adjustments. This is first because the longer the mean strength remains away from its desired value, the greater will be the effect on overall (longer term) variability. However, there is another aspect to the urgency of making the adjustment, which is often overlooked. If adjustments were made on the basis of actual 28 day strength, this would obviously mean that the adjustment could only be made something more than 28 days after the need for it arose. It is quite possible that a further change could occur during the period between the occurrence of the first change and its detection. It is equally likely that a subsequent change could be in either direction. If the second change is in the opposite direction to the first, then the adjustment being made for the first change could reinforce the second change. In this way it is possible that delayed control action could accentuate rather than reduce variability.

It used to be accepted QC dogma in any industry that changes should only be made after the current situation was clearly established, in terms of concrete strength this would mean after at least 30, 28 day results were available. As noted earlier, the author takes almost exactly the opposite viewpoint. However this is only tenable if every effort is made to ensure that only genuine change points are acted on, and assuming a batching system that is easily and accurately adjusted and records exactly what it has done and what it was instructed to do. Obviously it also requires a mix design system that automatically corrects for yield and MSF (see Section 3.2) when cement content is varied.

It is useful at this stage to set down two basic requirements for a control system derived from the foregoing:

1 The system must react as quickly as possible to discrete changes in mean strength.

2 The system should as far as possible detect the cause of the change. If this can be done quantitatively, it will be valuable in confirming whether the detected factor is the sole cause.

It is apparent that the first of these requirements is best satisfied by a cusum system able to incorporate the results of all grades of concrete in a single analysis (so multigrade, to a greater extent than the EN206 system) and that operates on predicted rather than actual 28 day results.

The second requirement involves a multivariable cusum system, which also assists in the first requirement, since a change whose cause can be seen does not rely on statistical justification.

4.5 Who should control?

In the past, a contractor produced concrete in accordance with a specification (which often specified the mix proportions) and a supervisor representing the eventual owner arranged for testing and demanded or negotiated changes (depending on the nature of the contract) if necessary.

It is relatively recently (under their new standard AS3600–1989) that Australian consulting engineers have recognized that control should be carried out by the producer. However, for many years most major Australian producers have operated their own testing and control systems regardless of other testing by their clients.

It is instructive to consider the evolution of the situation in Australia. Strength specification was introduced in 1958 and was based on testing by independent NATA (National Association of Testing Authorities) registered laboratories. In 1973 a new code (AS1480–1973) attempted to hand over control to individual producers using their own registered laboratories. However an alternative was permitted of continuing independent testing and control. The producer's control alternative not only entirely handed control to each individual producer (not to a joint scheme as in the UK) but it did not make reasonable provision for notifying concrete users of the situation or require that early age testing be used. In effect it would have been possible for a supplier to produce defective concrete for almost two months (i.e. until a month's defective 28 day results were in hand) and then only required that the mixes be amended to restore the minimum strength to its specified value. They were not even required to advise purchasers of what had happened, except for the ones who actually received concrete on which tests giving results below the specified strength were obtained. Showing good sense, the consulting engineers *en masse* totally ignored this alternative.

In 1989 a new code (AS3600–1989) made a more definite attempt to introduce producer's control. This code *required* that producers operate a quality control system *whether or not independent testing was also in use*. It also contained reasonable provision for reporting, and required an early age prediction system to be in use. Consulting engineers were slow to accept even this code, tending to still require control by the previous code. The new code does contain optional provisions for independent testing in addition to the producer's control, but the provision is such that the independent results would have to be very low indeed before they outweighed the producer's own assessment.

In 1991 the author devised a compromise solution that worked well. The problem was seen as being that there were too few of the independent results to provide a reliable independent assessment of concrete quality and it would be too expensive to increase the testing frequency. The author's solution was to *use the independent testing to assess the concrete supplier's testing rather than his concrete*. There had always been a problem with concrete producers claiming that independent samples were not correctly taken or the specimens not properly cast or cured. This was overcome, and the cost of the independent testing reduced, by requiring the concrete producer's laboratory personnel to cast a double set of test specimens at a specified frequency. This varied from once every 3 or 4 samples to once per day or even once per week. Even a very low frequency will expose the existence of any problem, and the duplication frequency can be increased should a problem be encountered. The second set of specimens was required to be delivered by the producer to an independent laboratory for curing and testing. In addition the producer was required to fax his own test results on the day of test to an independent consultant for analysis. It was interesting to note that when more intensive investigation (by increased sampling) of a discrepancy was commenced, the result was that the discrepancy almost invariably immediately reduced (i.e. everyone tried harder!). However some significant genuine differences were unearthed (Day, 1981).

It will be apparent that this system not only provided a good knowledge of the concrete, but also of the quality of work of both the producer's laboratory and the independent laboratory. It should be remembered that Australia has had its NATA laboratory assessment scheme since the 1940s (far longer than either the UK or USA) and that it has such a good reputation that it has been used as a role model by many other countries in setting up their own systems. Details are given in Section 10.5, but it should be noted that the results of these comparisons certainly justified the duplication procedure. It should also be made very clear that the system did not reveal any hint of dishonesty, only evidence that it is quite difficult to avoid occasional unmerited low test results. Several times the independent results were used to show a producer that his own laboratory was producing lower results than merited by the concrete. However more often the opposite was the case. This is not to suggest that the supplier's results are often biased but arises because, when results differ, the higher result is more often the nearer to the true value. This being so, it clearly does not pay the concrete supplier to skimp on the quality of his testing, the costs of which can be subsidized by his savings in concrete cost. However *the independent laboratory can only operate profitably at a cost level that the market will bear and at a quality level which the market is able to distinguish*. Where supervising engineers believe that *any* registered laboratory *always* produces an accurate result, and specify independent testing but allow the contractor to choose the cheapest laboratory, a pressure to reduce standards to the lowest able to obtain registration is created.

By the late 1990s, concern by consulting engineers about control by the producer had largely disappeared and independent testing had become infrequent. This situation was locked in by the wide introduction of quality assurance.

4.6 Quality assurance

If it is not obvious from the aforementioned that control should be by the producer (even though limited monitoring by others may be desirable) the question is settled by the worldwide trend to Quality Assurance in concrete as in many other fields. QA requires monitoring of all incoming materials and all production processes as well as ultimate results. This is really only financially practicable if done by the producer himself.

The international standard ISO 8402 defines a 'Quality system' as *'The organisational structure, responsibilities, procedures, activities, capabilities and resources that together aim to ensure that products, processes or services will satisfy stated or implied needs.'* Clearly this is a much more comprehensive matter than techniques of testing or mathematical analysis. It may be taken as both advice and a warning. The advice is that such a formal and comprehensive pre-planned structure has been found to be necessary to achieve a full assurance of quality. The warning is that care is needed to avoid being submerged in form-filling and administration at great expense, and possibly to the exclusion of effective quality control.

It is also necessary to be careful to avoid the 'new player's' assumption that quality assurance and quality control are necessarily different, with the latter being old fashioned and superseded. It may be helpful to think of quality assurance as simply '*documented* quality control'. As an example, consider the specification and control of a sand. It may seem like the correct quality assurance approach to specify grading limits and reject sands not complying. This may be contrasted with the author's approach of saying that almost any grading is useable, providing that the mix is adjusted. The particular technique of 'feedback quality control' may seem the antithesis of quality assurance, since it reacts to the effect of a characteristic of an input material. However there is nothing wrong, or contrary to the principles of quality assurance, in writing in the quality manual that the mix shall be adjusted if the sand grading changes, or that water content shall be adjusted if slump is found to vary. What makes these actions quality assurance is that they *are* written down, and probably that a nominated individual has to make the change and document it.

4.7 Pareto's principle

Vilfredo Pareto was an Italian economist (1848–1923) engaged in traveling from town to town in an attempt to identify the country's sources of wealth. He came to realize that the 4 or 5 wealthiest men in a town almost invariably controlled over half its wealth. Therefore his survey could most efficiently be conducted by first seeking out the right men and then asking his questions, rather than attempting a random survey of a few per cent of the population.

This principle is of great value in QC of all kinds, certainly including concrete QC, that is, while there may be 100 or more factors causing variability in concrete strength, 70 or 80% of the total variability is often caused by only 2 or 3 of the 100 possible causes. Often only one single factor will cause more than half of the total variability. It will not be the same principal factor in all cases, nor even the same 'short list' of 2 or 3, but the following list is likely to include the major factors in most cases:

1. Slump (misjudgment or deliberate variation)
2. Temperature
3. Air content
4. Fine aggregate silt content
5. Fine aggregate organic impurity
6. Fine aggregate grading
7. Coarse aggregate dust content
8. Coarse aggregate bonding characteristics
9. Cement quality
10. Admixture quality, dosage or interaction
11. Fly-ash quality (especially carbon content)
12. Time delays
13. Coarse aggregate strength
14. Fine aggregate grain quality
15. Sampling and testing procedures (namely: segregation, compaction, curing, capping, centering in testing machine, lubrication of spherical seating, planeness of platens, stiffness of machine frame, alignment of ram and spherical seating, rate of loading, operator fear of explosive failure or desire to maintain specimen in one piece).

The mechanism of the effect on strength is via increased water requirement in many cases, specifically in items 1, 2, 4, 6, 7 and 12.

Finding the principal causes of variability

It may be fairly obvious in some cases which of these causes is likely to predominate, but often this is not the case. Rather than make a guess, or spread control either too thinly or too expensively over too many factors, it is better to follow the advice of the master of QC, J. M. Juran (Juran, 1951) and 'ask the process'. This can be done in two distinct stages:

1. Compare actual and predicted strength and if there is a discrepancy, track it down. This may provide a firm lead on what is most likely to affect strength on the particular project.
2. Monitor strength and a selected number of 'related variables' using Cusum analysis.

The selected variables will usually include slump, air content and concrete temperature. If a reasonably reliable water content is available from any source, this is certainly very important. The strength results will be particularly examined for pair differences and 7 to 28 day gain as a kind of internal consistency test. It is important to realize that low strengths do not ‘just happen’ they are usually caused by either high water content, low cement content, incomplete compaction, defective curing and testing, or reduced cement quality. The art or science of QC is to establish which of these is the cause by a logical examination of the pattern of results, for example difference in cement quality from one delivery to another is not a reasonable explanation for isolated low results or for a period of low strength extending for a shorter or longer period than that between the two deliveries. High water content will not explain low 28 day results if 7 day results from the same sample were normal. Certainly the possibility that the concrete is normal and the testing defective, should be adequately considered as it is frequently encountered.

4.8 Related variables

Strength results alone are certainly inadequate for the operation of a control system. We cannot be totally *unconcerned* with whether particular isolated results or isolated groups of results satisfy specification requirements. Nevertheless an examination of the *pattern* of results and their correlation or otherwise with ‘related variables’ (e.g. slump, density etc.) is far more rewarding. The primary aim should be firstly to establish the overall situation as exactly as possible, and only then to consider the significance of particular results.

Monitoring day to day performance variations

Experience has shown that it is not enough to set up an excellent laboratory and thoroughly train suitable staff, any more than it is enough to set up a good ready mix plant and supply it with reputable materials. In both cases it is necessary to monitor the actual performance. In the case of testing the criterion is the repeatability of the test (although the relevance of the result may also be in question, especially in workability testing but also for example in early age strength testing). The best measure of repeatability is the average pair difference when tests, for example cylinder compression tests, are carried out in pairs. Another useful indicator is the average gain from an earlier to a later age, this can be varied by other factors, such as cement composition and curing conditions, but such variation often arises from testing problems. For example when a strength drop is experienced on both 7 and 28 day results from the same *testing* date rather than the same *casting* date, the testing process would be highly suspect (the author has experienced this, but with flexural not compression testing).

Examining correlation

The classic method of examining correlation is to use a computer statistics package to provide a regression analysis (slope and intersect values, and a coefficient of correlation in a linear equation between two possibly related variables). This approach ought to work but disappointing results from it have been reported by Shilstone (Shilstone, 1987). The author has found that better results are obtained by cusum graphing.

The problem is that many factors are involved. An increased slump may be the result of an increased water content, in which case a strength reduction would be anticipated. However it may equally have resulted from a lower concrete temperature, or a coarser sand, in which case no strength reduction would result. So a regression analysis of slump against strength may give the disappointing results reported by Shilstone. However if there is a change point in strength coinciding with a change point in slump there is correlation at that point, even though a few results further on there may be another change point in one or the other without a corresponding change in the other.

4.9 Practical use of a cusum analysis

So cusum analyses can be set up for a selected number of variables for every mix in use. If manually, or using a spreadsheet, the number of variables may have to be quite limited and very carefully selected. Also it may only be practicable to include a limited selection of the more important mixes. If using a system such as the ConAd system, developed by the author and now marketed by Command Alkon, every mix and every recorded variable will be automatically cusummed. One solution is to use the free program on the website www.kenday.id.au and Figs 4.6, 4.7 and 4.8, but this restricts variables to 8 (early and late strength, 7–28 day strength gain, predicted strength on two alternative bases, slump, density and temperature) as opposed to more than 50 that can be entered in the full ConAd system (see Section 4.12).

The primary cusum is of early age strength (7 day or earlier) used to predict 28 day strength by adding the current average gain from the early age to 28 days for the mix in question. The differences being cumulatively summed are differences from the current average 28 day strength of the grade in question. All these differences are included in a single cusum (in date/time sequence), initially regardless of whether they are from 20 MPa or 100 MPa, whether from flowing or no-slump concrete, blended or straight Portland cement, and whether from dense or lightweight concrete. This cusum should be inspected every day as soon as the day's results have been entered. If it does not show a change point there is no problem and further consideration can be deferred for a week or a month depending on your situation.

If the results do show a change point it is necessary to decide whether it is real or only a statistical aberration and, if real, what caused it. The possible causes can be usefully separated into water content or non-water content. If excess water is the cause, then density will be reduced and so the next most important cusum is of density – and yes, you can cusum densities from dense and lightweight mixes on the same graph. If a change point is present on the density graph, coinciding with

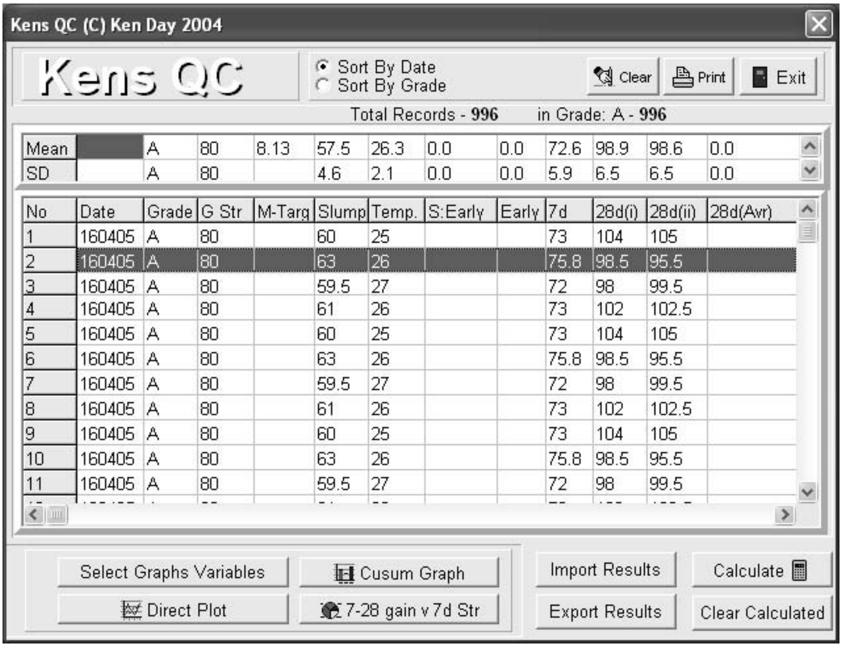


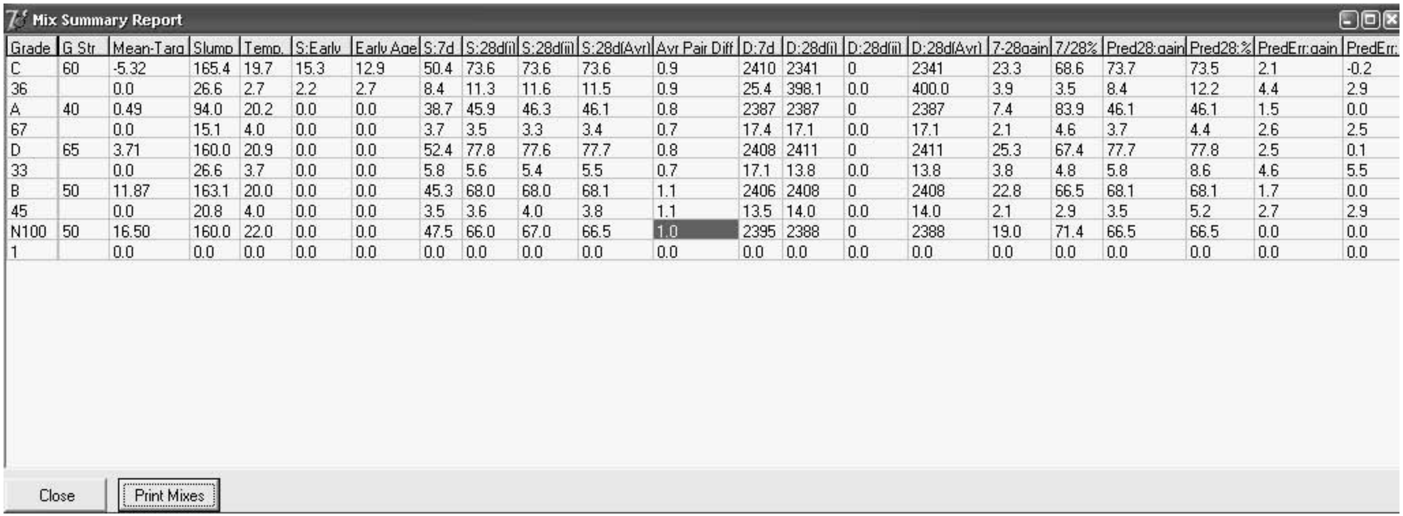
Figure 4.6 QC program (free download from author's website).

that on the strength graph, two things follow. One is that it is a real change point and the other that it is caused by excess water (or just possibly entrained air). An important point is that densities are hopefully density on receipt at the laboratory rather than at test, and have hopefully been entered on the day of receipt. In this case the density cusum will be running 6 days ahead of the 7 day strength cusum and so be a more certain confirmation that the detected change is genuine.

The next cusum is that of temperature. Past records brought for analysis by new clients almost invariably show a reverse correlation between temperature and strength. So strengths are higher in winter than in summer (except at early ages). Once aware of this, seasonal adjustment is made, since it makes no sense to carry a greater risk of failure in summer, or to use excessive cement contents in winter (unless early age strength is a problem).

A fourth cusum will usually be of slump, although this is not so likely to correlate for all mixes and all clients. Often a change point will be found to correlate with a change of personnel on a particular site, or correlation will be on an individual truck basis.

If data is available on sand grading, a cusum of sand specific surface is very likely to show a correlation with water-influenced strength (unless grading



Mix Summary Report

Grade	G Str	Mean-Targ	Slump	Temp.	S:Early	Early Age	S:7d	S:28d(i)	S:28d(ii)	S:28d(Avr)	Avr Pair Diff	D:7d	D:28d(i)	D:28d(ii)	D:28d(Avr)	7-28gain	7/28%	Pred28:gain	Pred28:%	PredErr:gain	PredErr:
C	60	-5.32	165.4	19.7	15.3	12.9	50.4	73.6	73.6	73.6	0.9	2410	2341	0	2341	23.3	68.6	73.7	73.5	2.1	-0.2
36		0.0	26.6	2.7	2.2	2.7	8.4	11.3	11.6	11.5	0.9	25.4	398.1	0.0	400.0	3.9	3.5	8.4	12.2	4.4	2.9
A	40	0.49	94.0	20.2	0.0	0.0	38.7	45.9	46.3	46.1	0.8	2387	2387	0	2387	7.4	83.9	46.1	46.1	1.5	0.0
67		0.0	15.1	4.0	0.0	0.0	3.7	3.5	3.3	3.4	0.7	17.4	17.1	0.0	17.1	2.1	4.6	3.7	4.4	2.6	2.5
D	65	3.71	160.0	20.9	0.0	0.0	52.4	77.8	77.6	77.7	0.8	2408	2411	0	2411	25.3	67.4	77.7	77.8	2.5	0.1
33		0.0	26.6	3.7	0.0	0.0	5.8	5.6	5.4	5.5	0.7	17.1	13.8	0.0	13.8	3.8	4.8	5.8	8.6	4.6	5.5
B	50	11.87	163.1	20.0	0.0	0.0	45.3	68.0	68.0	68.1	1.1	2406	2408	0	2408	22.8	66.5	68.1	68.1	1.7	0.0
45		0.0	20.8	4.0	0.0	0.0	3.5	3.6	4.0	3.8	1.1	13.5	14.0	0.0	14.0	2.1	2.9	3.5	5.2	2.7	2.9
N100	50	16.50	160.0	22.0	0.0	0.0	47.5	66.0	67.0	66.5	1.0	2395	2388	0	2388	19.0	71.4	66.5	66.5	0.0	0.0
1		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Figure 4.7 Export of analysed data from Kens QC to spreadsheet or notepad.

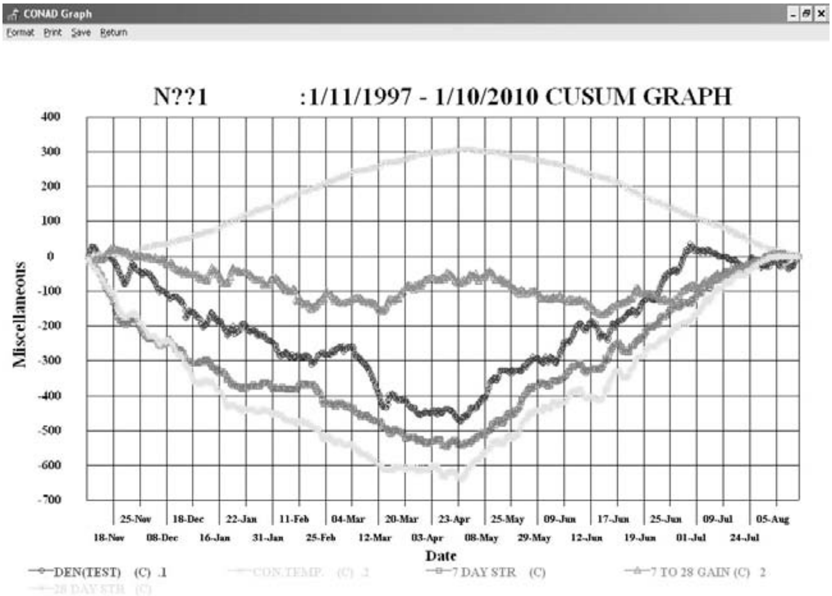


Figure 4.8 Typical output of cusum graphs (in colour in actual use).

variation is adjusted for using the author's MSF concept). Unfortunately it is only in very high volume situations that enough sand testing is done and even then there is a tendency to test during stockpiling rather than use, so often losing correlation.

If a strength cusum change point is not related to water content (i.e. is not reflected in density), then cement quality becomes suspect. The cement producer should be running cusum analysis on fineness, normal consistency, chemical composition, and early and late age strength and should be keeping their clients advised of changes (however some of the author's concrete producer clients have been known to advise the cement producer!)

Testing error is sometimes a significant factor and a cusum of pair differences will sometimes show a clear change point as a new operator is introduced. As explained later, recorded mean strength tends to be artificially depressed by at least as much as the 28 day pair difference. There is a significant additional cost in raising the mean strength 1 or 2 MPa to compensate for this, in addition to a likely effect on SD. A cusum of pair difference is therefore useful in revealing problems to be further investigated.

A predicted 28 day result predicted from an early age test is like a third 28 day result and a variable 7–28 day gain is equivalent to a testing error in the same way as a pair difference, although it may be caused by variable curing or variable cement characteristics. So a cusum of 7–28 day strength gain is certainly of interest, but change points may require careful consideration.

Depending on local knowledge or individual observation, at some point in the evaluation process a more limited selection of grades to include in the analysis should be made. If more than one cement or cementitious combination or admixture is in use then separating them in different groups will show whether one of them is responsible for the change. Likewise groups using particular aggregates can be set up.

The possibility certainly exists that a particular grade represented by very few results will develop a problem that will be swamped by the mass of other results. However the ConAd program, and even the free program on the website, automatically displays a list of every grade in order of departure from target strength. So any defective grades will appear prominently at the top of this list. This should be an essential feature of all such analysis programs.

The analysis program should produce an overall SD derived from the difference between successive results in the same grade. This is essentially the SD that would be obtained if there were no change points in the results. The list of grades will show the conventionally determined SD for each grade. In many cases there may not be enough results for this to be of much significance, but where there are say at least 20 results in the grade and the SD is substantially higher than the overall SD, this is an indication that there is probably a change point in the run of results for that grade. The individual cusum and also a direct plot of the grade in question should be examined.

4.10 Direct plots

The author's strong advocacy for the use of cusum analysis should not be taken to imply that direct plots of results are useless. Cusum works well in part because it is little affected by individual chatter, but individual results are also important.

One form of direct plot that can also be multigrade is a direct plot of result minus one of target strength, mean strength or specified strength. It is particularly useful to have graphs of one of these variables for each of 28 days and early age. Where both graphs show a low result, it is probably a genuine low result. Where only one of them shows it, then testing is suspect (Fig. 4.10).

It is also useful to show these direct plots along with the strength cusum. Where the cusum shows a low period (downturn) it is not surprising to see individual low results. However individual low results in a period when average results were satisfactory are more interesting. Such results are one of: testing error, miss-batches, added water on site, delayed discharge, or a particular problem grade. Obviously such results must be further investigated (Fig. 4.9).

A newly recorded high slump or low density may be a cause for concern and it may be possible to look back along a direct plot to see what strength resulted from the last such result.

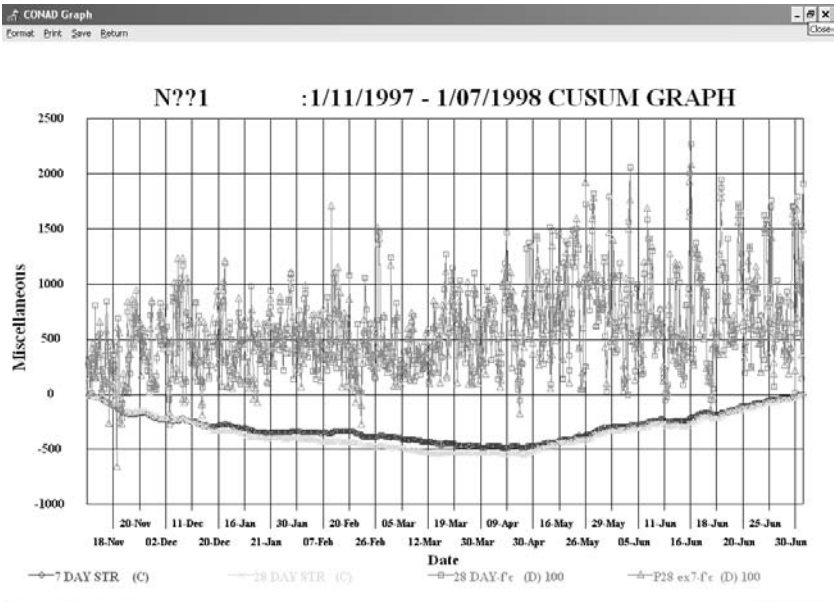


Figure 4.9 Strength cusum combined with direct plot of strength minus reqd. strength on multigrade data.

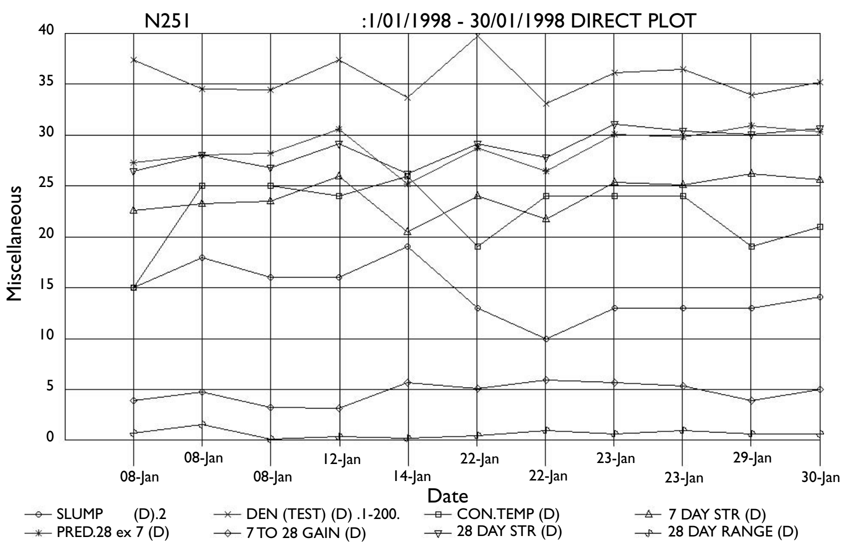


Figure 4.10 Direct plot of multivariable (but single grade) data.

4.11 Rejection, penalization or bonus?

In 1958 the author wrote a series of articles on 'Statistical Quality Control of Concrete and Concrete Products' (Day, 1959) that contained the following:

> The only rational objective for any but 100% testing is not to discover and reject faulty products but to ascertain the minimum quality level of the production. A moment's thought will show that if 10% of total production is tested, then for every faulty unit discovered and rejected, nine faulty ones will be accepted. This applies not only to final tests on products but also to each individual batch of concrete produced.
>
> If any reject units or concrete specimens whatever are discovered, a serious situation exists which cannot be met by the rejection of the tested defective units alone and should lead to extra testing on a scale that would normally dislocate the entire production system.
>
> A distinction should be drawn between unsatisfactory and unusable products. No useable product should be rejected and no unsatisfactory product should be paid for as first quality. The specified minimum strength may be 4,000 psi but clearly 3,950 psi would not constitute unusable concrete. If the absolute rejection limit be maintained at 4,000 psi and a large concrete unit contains nine batches stronger than 4,000 psi and one at 3,950 psi, then it would have to be rejected. This is clearly undesirable.
>
> If 10% of the tests made are below strength, then probably all units made contain defective concrete although on average only one in ten units would show defective on test cylinder results. It would not be satisfactory to reject this tenth unit. If however the results were statistically analysed and it was shown that 10% of results were below specification (but not dangerously so) a cash penalty could be imposed and all units accepted. If one dangerously low result were obtained then probably nine previous units contain dangerously defective concrete and acceptance testing of all production should be carried out.
>
> This underlines the desirability of a zone of useable though unsatisfactory concrete since, in its absence, we have either to regard 3,950 psi concrete as dangerously weak, or to allow a manufacturer to produce poor concrete with impunity on occasions.

The author is still of the opinion that a distinction should be drawn between *structurally* defective concrete and *contractually* defective concrete defined as follows:

Structurally defective concrete is that which is unable to serve its intended purpose and must be removed from the structure or supplemented in some way. It is absolutely imperative that no such concrete whatever be produced (it is not practicable to allow some to be produced and then attempt to ensure its exclusion from the structure).

Contractually defective concrete is that which, while capable of serving its intended purpose, is not quite of the specified quality. A small proportion of such concrete may be incorporated in the structure with little detriment.

There is usually a substantial margin between the two and the author's experience is that if no contractually defective concrete is accepted without some penalty, or substantial expense and inconvenience to the contractor, no structurally defective concrete is produced. However if contractually defective concrete is supplied with impunity, structurally defective concrete is likely to follow.

4.12 Data retrieval and analysis/ConAd system

Coping with data

A basic challenge in the quality control of concrete is to cope with the availability of possibly excessive amounts of data. There is no doubt that facts can be harder rather than easier to deduce if included in more data than a person can cope with. It should not be forgotten that quality control is an exercise in cost reduction and that cost includes the cost of the quality control. A better quality concrete can be purchased at a higher price, but the task of quality control is to deliver concrete of a chosen quality at the minimum cost.

So the value of given data should be considered alongside the cost of acquiring, storing, analysing and employing those data. In particular no substantial cost should be incurred in acquiring and storing data that will definitely not be used. On the other hand storage of huge amounts of data is no longer a problem, providing it can be acquired at negligible cost and effort and the precise data needed can be automatically retrieved with little effort.

An example of inadequate cost/benefit was previously referred to in New York where inspectors were employed to manually write down batch quantities at substantial cost, but no analysis of the acquired data was carried out. In contrast batch quantities (intended and actual) are automatically acquired electronically by the ConAd system, are automatically matched with test data on tested loads, and errors can be automatically displayed either numerically or graphically. In the latest development, the system can automatically email or telephone selected personnel to advise of errors, and can predict the strength of a miss-batched load. Long-term trends in inaccuracy can be precisely displayed graphically. Of course these facilities require both suitable batching equipment and a suitable analysis program.

Other data that can be automatically acquired include details of the original order, so that a field testing officer only needs to record a batch number and his actual measurements. Also many laboratory testing machines are able to output test results direct to a laboratory computer. This not only saves time but also avoids the possibility of error in transference and the necessity to check for such errors. Not only crushing loads but also weights and dimensions of compression specimens are often automatically recorded.

The other end of the process is the retrieval of data from storage. If large amounts of data are recorded, then retrieval must be substantially automated. The largest amount of data is usually batching data. This is required in full so that cumulative errors and the variability of the process can be studied. It is not enough to have all the information tabulated so that the analyst can run their eye down the column to look for exceptions. It is not even enough to graph the data, revealing exceptions many times more quickly. It is necessary for the system to be able to retrieve those items, and only those items, having an error in excess of any nominated amount. It is also necessary to have cumulative error graphs, showing whether consumption averages that planned.

The ConAd system

The ConAd system comes with an instruction manual several hundred pages in length and obviously cannot be reproduced in this book. The objective of this book is not to demonstrate the ConAd system but to make the reader aware of the existence of various QC techniques. Some of these are now available elsewhere, including the free programs on the author's website and the Labsys program of Contek, the author's new partner.

The huge manual makes it obvious that a comprehensive QC system is not something that can be committed to memory in a short training course. A good system should have a simple but very comprehensive data entry and an essentially automatic detection of the existence of any significant problem with concrete quality. A relatively junior or secretarial person should be able to learn these features in a single day and will not forget them as they are used every day. A more senior and technical person can then gradually become familiar with more and more of the program's capability as they use them to investigate the causes of revealed problems or to reduce variability and achieve greater economy.

So the ConAd system (version 2) is used to illustrate techniques here, even though Command Alkon have now superceded it by version 3.

A first screen (Fig. 4.11) allows selection by date period, docket number range, or sample number range. Data can be restricted to that for a particular client, project, producing plant or supplier (this last for use of the system by a major project purchasing concrete from more than one supplier). There are options to use batch plant data in the analysis or not, and to restrict batch data to only that from trucks which have been tested. Data can be restricted to a particular cement or aggregate source group. The right hand side fields allow adaptation to suit different countries, running means of 3 or 4 and 10 or 20 (or anything else you enter), 'k' values of 1.65 for 5% below or 1.28 for 10% below.

At the top left is Product Code entry. Clicking on the arrow of this brings up a list (Fig. 4.12) of all product codes in use (there may be many hundreds in the largest organizations). The facility is provided to make these into multigrade groups. Even though this only has to be done once, it may still be an onerous task so the facility to use wildcards has been added. Depending on how carefully

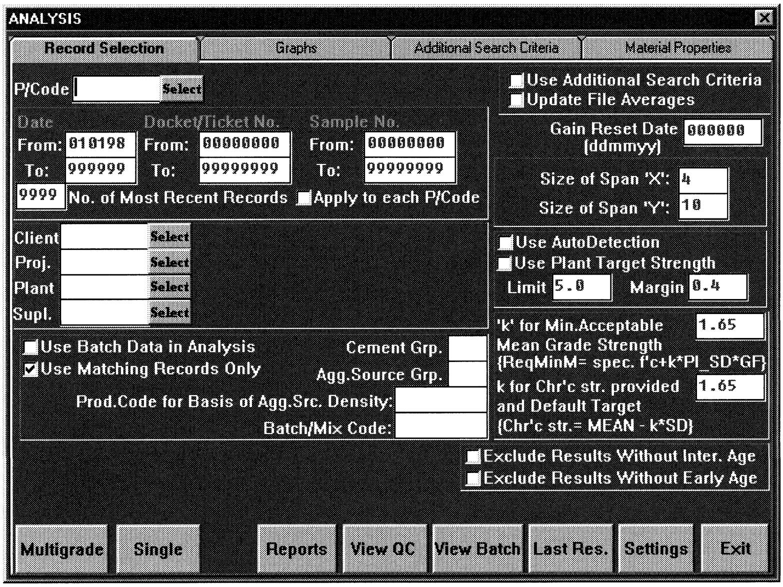


Figure 4.11 Record selection screen.

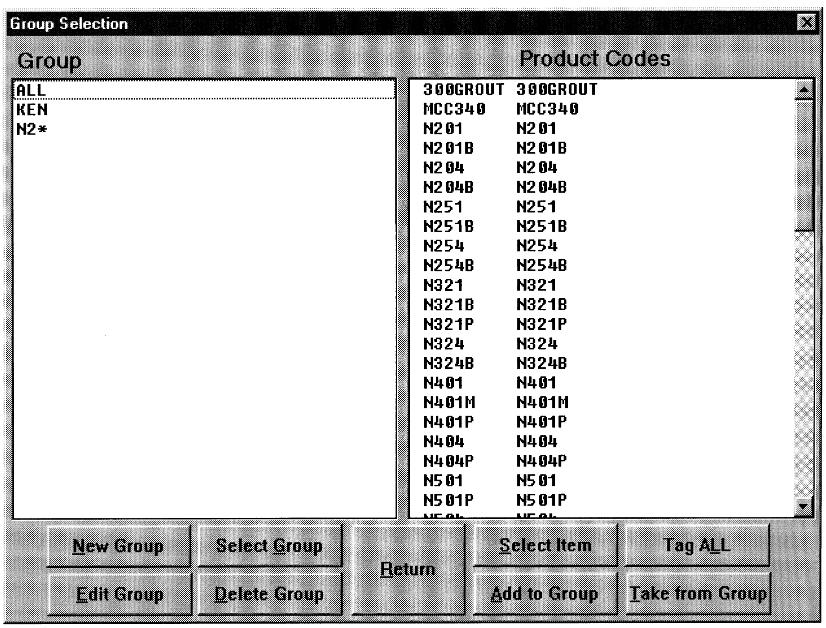


Figure 4.12 Grade/group selection screen.

product codes have been chosen, this can make life much easier. Maybe product codes start with an N or an S (for normal or special, as they do in Australia). Maybe the second and third, or fourth and fifth, give the grade strength. Maybe the sixth or seventh tell which are pump mixes, or which have 14 mm maximum sized aggregates. So N* will give all standard mixes, ???1 all pump mixes, ????F all mixes with fly-ash etc.

Then there are the two check boxes in the top right-hand corner of the screen in Fig. 4.11. These refer to a second screen (Fig. 4.13) which offers an extensive choice.

Using the top two sections, it is possible to segregate data that has any number of batch ingredient quantities above, below or between any nominated limits; or which has given test results at any age above, below or between any nominated limits.

The bottom section of this screen offers even more interesting possibilities. The average pair difference (in 28 day strength tests) of each testing officer in turn, over any selected period, can be examined, or the average difference between ordered and tested slump for each individual truck. Even the average difference between target and actual strength for each individual test specimen mould could be examined. Although this screen offers a very large range of possibilities, rarely would more than one of them be selected at a time. It is not suggested that extensive use should necessarily be made of this screen but, in the spirit of the rest of the program, if you have a use for the facility it is certainly available in a very comprehensive way.

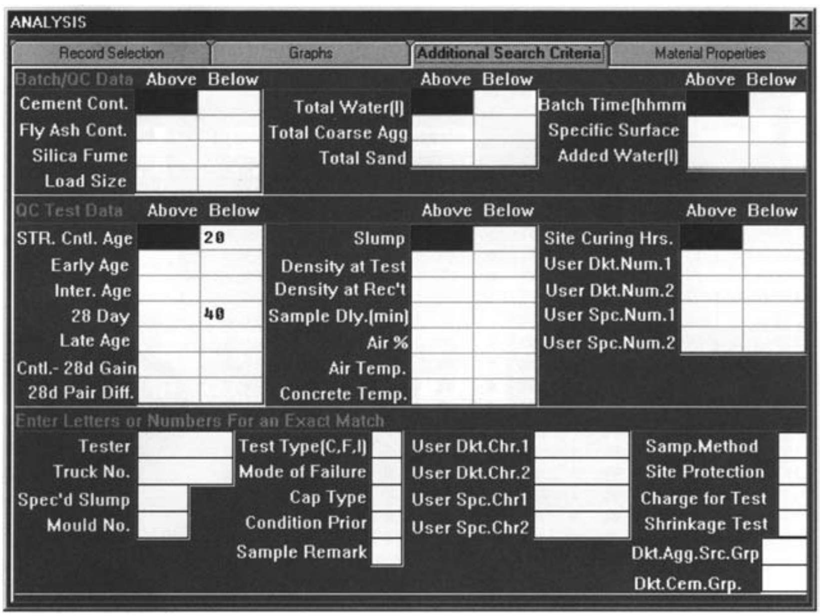


Figure 4.13 Second screen criteria.

The check box for 'Use Additional Search Criteria' on the main screen has been found to be very necessary as some clients may forget they have made entries and inadvertently use biased data in an analysis.

Similarly, when performing a restricted analysis, the running averages maintained by the system must not be updated.

Finally on the main selection screen, 'Gain Reset Date' requires explanation. The system automatically maintains an average gain for every grade of concrete for which results are entered. This enables the system to give correct predictions of 28 day strength whatever the characteristics of the particular cement or concrete mix in use. However if a sharp change in cement characteristics (or admixture usage) takes place, it can take some time for the average to adjust to the correct value. Therefore if such a change is detected, its date should be entered in the box shown so that all results prior to that date will be excluded from the average.

It may not be practicable to store all batching records for many months, nor is this usually necessary, but the system must be able to match up specimen test data with full detail of the batch from which it was taken. It is then possible to archive more than 90% of batch data after several months, while still retaining batch data from all tested trucks, which may be separately stored along with the test data on that concrete.

Strength cusum target types

It is necessary to distinguish different purposes for which cusum graphs may be required, and to further consider what target values may be appropriate for each purpose.

Type 1: Adherence to pre-set target – Obviously in this case the pre-set target value will be used as the cusum target. When the cusum exceeds some pre-set limiting value it will not necessarily indicate that any change has taken place but only that the mean value in practice is different from the target value at some pre-selected level of certainty. Unless the production mean has settled down to be very close to the target value, this type of cusum may be relatively poor at displaying correlation with other variables on a multivariable graph.

The slope of this type of cusum positively shows whether the results are above or below target since they are level when on target, sloping down when below, and sloping up when above. They may therefore be more suitable for use by relatively untrained or inexperienced personnel.

Type 2: Change (and cause of change) detection – For this purpose it is better to use the actual mean of the results being analysed as the target value. Such graphs will always start and finish at zero and will only display change. Since this applies to all variables on a multivariable graph, correlation is much easier to detect, however adherence to a pre-set target is not necessarily clear from such a graph.

Type 3: Un-monitored factor problem detection – If a cusum is drawn of either actual strength divided by calculated strength (i.e. 'strength factor') or actual minus calculated strength, what it will reveal will depend on the sophistication of the calculated strength. If the formula accurately allows for the effects of temperature, slump and haul time, these would no longer cause change points on the graph. Change points due to such causes as cement quality and sand grading or silt content might then be more clearly displayed. If these items also are being effectively monitored and included in the formula for calculated strength, then the analysis would be very sensitive to sub-standard testing (including sampling and care of specimens), inadequate mixing, etc.

The most likely situation is that such an analysis (if very comprehensive) will show inadequacies of either or both of the calculation formula and the testing regime (for both concrete and input materials).

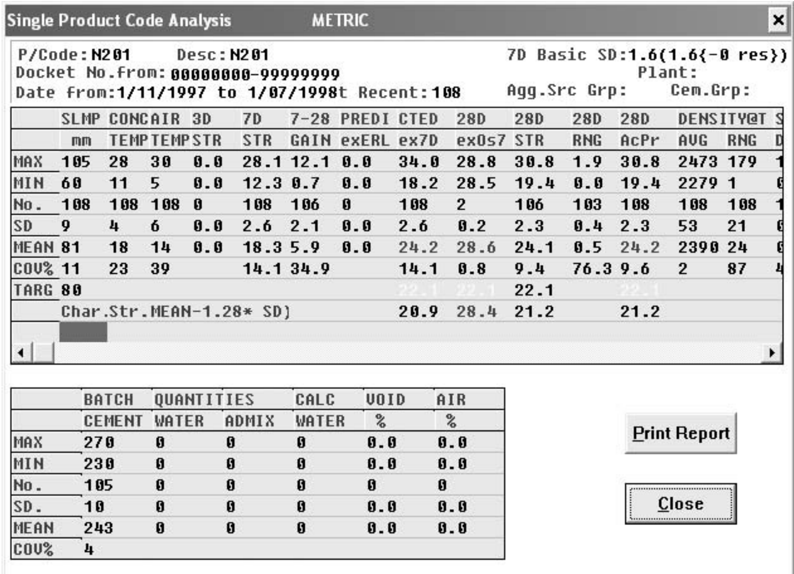
The target value for such a cusum should usually be one but if there were some reason to leave the strength factor at some other value (e.g. to indicate a higher or lower than average cement quality) the mean value over the analysis period could be used.

Control age basic SD

If a set of results contains a change in mean strength, the basic SD will be increased. As explained in the Statistics Chapter 10, it is useful for the system to highlight this. A 'basic SD' can be calculated from the average pair difference of successive results in the same grade. This figure gives a picture of the typical variability that can be averaged over hundreds, possibly thousands, of results in dozens or hundreds of different grades. A figure of 2.0–2.5 should be possible and if the figure is much over 3.0, concrete production is distinctly variable. Some of the variability may be due to isolated results from bad testing or batching errors. In the ConAd program the user can nominate a range outside which results are to be discarded for the purpose of this calculation. However the possibility exists that a large number of results will be discarded and the user will be fooled into thinking variability is low when it is not. So the system advises how many results it has discarded.

The presentation to the user is shown in the top right of Fig. 4.14 and the calculation process is shown in Fig. 4.15. The conventional SD shown in the statistical summary screen is 3.3. The next column is the difference between consecutive results, three are in bold font since they are higher than the limit of 5 set in the middle section of Fig. 4.13. The first 7-day basic SD (2.61) is shown is at the bottom of the 4th column and is calculated using the formula shown in Section 5.3, that is (Average difference between successive control age results) / 1.128. The final column has the three difference values over 5 removed, the second 7-day basic SD (2.06) is calculated at the bottom of this column.

The calculation for the first and second basic SD for a multigrade analysis is essentially the same as for a single grade analysis. The only difference is that



Single Product Code Analysis METRIC

P/Code:N201 Desc:N201 7D Basic SD:1.6(1.6{-0 res})
Docket No.from:00000000-99999999 Plant:
Date from:1/11/1997 to 1/07/1998t Recent:108 Agg.Src Grp: Cem.Grp:

	SLMP	CONC	AIR	3D	7D	7-28	PREDI	CTED	28D	28D	28D	28D	DENSITY@T		S
	mm	TEMP	TEMP	STR	STR	GAIN	exERL	ex7D	ex0s7	STR	RNG	AcPr	AVG	RNG	D
MAX	105	28	30	0.0	28.1	12.1	0.0	34.0	28.8	30.8	1.9	30.8	2473	179	1
MIN	60	11	5	0.0	12.3	0.7	0.0	18.2	28.5	19.4	0.0	19.4	2279	1	0
No.	108	108	108	0	108	106	0	108	2	106	103	108	108	108	1
SD	9	4	6	0.0	2.6	2.1	0.0	2.6	0.2	2.3	0.4	2.3	53	21	0
MEAN	81	18	14	0.0	18.3	5.9	0.0	24.2	28.6	24.1	0.5	24.2	2390	24	0
COV%	11	23	39		14.1	34.9		14.1	0.8	9.4	76.3	9.6	2	87	4
TARG	80							22.1	22.1	22.1		22.1			
	Char.Str.MEAN-1.28* SD)							20.9	28.4	21.2		21.2			

	BATCH	QUANTITIES		CALC	VOID	AIR
	CEMENT	WATER	ADMIX	WATER	%	%
MAX	270	0	0	0	0.0	0.0
MIN	230	0	0	0	0.0	0.0
No.	105	0	0	0	0	0
SD.	10	0	0	0	0.0	0.0
MEAN	243	0	0	0	0.0	0.0
COV%	4					

Print Report

Close

Figure 4.14 Statistical summary screen.

Date	Docket	All Results 7-Day	Included Difference	3 Results Excluded as >5MPa Difference
98-06-02	38994	25		
98-06-02	38996	27.6	2.6	2.6
98-11-11	42176	20.1	**7.5**	
98-11-12	42205	23.1	3	3
98-11-23	42389	22.5	0.6	0.6
98-11-23	42394	21.9	0.6	0.6
98-11-24	42433	25.2	3.3	3.3
98-11-25	42476	26.2	1	1
99-01-08	42892	29.2	3	3
99-01-12	42899	33.7	4.5	4.5
99-01-15	42934	25.1	**8.6**	
99-01-21	43040	27.4	2.3	2.3
99-01-26	43122	27.1	0.3	0.3
99-01-26	43126	23.6	3.5	3.5
99-01-27	43145	25.4	1.8	1.8
99-02-02	43294	21.9	3.5	3.5
99-02-03	43328	23.1	1.2	1.2
99-02-03	43329	22.2	0.9	0.9
99-02-09	43494	26.1	3.9	3.9
99-02-09	43509	22.9	3.2	3.2
99-02-10	42542	27.6	4.7	4.7
99-02-15	43658	27.9	0.3	0.3
99-02-22	43857	24.4	3.5	3.5
99-03-02	44059	30.0	**5.6**	
99-03-02	44077	31.2	1.2	1.2
		3.25 **SD**	**2.61** **1st BSD**	**2.06** **2nd BSD**

Figure 4.15 Calculation sheet.

when it is calculating the successive differences between results the difference between the last result of one product and the first results of the next product code is not calculated. The control age results are considered one product code at a time, in a date then docket number order.

Explanation of graphing options

ConAd Graphs are of two types, direct plots and cusum graphs. The former work better on a limited number of results (say up to 50 or perhaps 100). They are especially useful in establishing correlation between strength and other variables on isolated abnormal individual results (e.g. low strength and high slump).

Cusum graphs work well on large numbers of results. They are much more efficient at detecting a change in mean value (and the precise time of that change) and correlating change in one variable with change in other variables.

There are over 90 items, which can be selected for graphing, and any eight can appear on one set of axes. Direct and cusum graphs can appear on the same screen. Generally the x-axis is for a sequence of batches but the label is date. A difficulty arises in the case of material properties. All other items are matched by docket/ticket number so that all points in a vertical line are data on the same batch of concrete. Material properties are not peculiar to a particular batch of concrete but can still be matched by date. A material property graph will display a castellated form, only changing its level on the first date after a new measurement of the property has been entered.

We at ConAd have noted that only a very small selection of the 90+ available options is used by most of our clients. Certainly the intention is to provide all possible options and the expectation is that each client will have a favourite (small) set of options. However it may be that many clients do not understand the intention behind many of the options. This guide attempts to remedy that situation by listing each option and stating the intention behind its inclusion. Following this, advice is provided as to how to proceed in analyzing results.

Grade strength: Strength is taken as the best indication of concrete quality and as a measure of the variability of the concrete. Two basic factors affecting strength are water/cement ratio and cement quality. Temperature (in the form of maturity) also has a significant effect on early age strength. When strength shows a downturn a major first question to ask is whether or not it correlates with a change in water content (it usually does).

Docket no.: This is the basic connector of all other variables except material properties.

Slump: Slump is not a good measure of true workability but it is an excellent measure of relative water content between successive deliveries of supposedly identical concrete. However slump is also affected by temperature, air content, sand fineness, silt content, time since batching etc. An increase in slump between two successive truckloads (i.e. without the opportunity for any other variables to change) will always produce a reduction in strength. However changes in other variables often blur the relationship between slump and strength over a period. The converse of a reduction in strength always being associated with an increase in slump is certainly not necessarily true.

Slump minus specified slump: This is useful as a measure of the skill of the truck driver if he is allowed to add water. If the operation is sufficiently sophisticated to adjust cement content when a different slump is specified, this variable may be more significant than actual slump.

Total water content: If the total water content is accurately known, it is the most important of all variables other than strength. Unfortunately it is rarely accurately known, however a best estimate may still be very useful and interesting. It is possible to measure total water content by drying (oven or microwave) or by water displacement. The greatest inaccuracy in such measurements is usually an unrepresentative proportion of coarse aggregate to mortar. It may be desirable to carry out tests of extracted mortar and convert to concrete assuming correct aggregate proportions.

Water content ex slump and temperature: The water content is not determinable solely by slump and temperature but the program calculates the change anticipated from a change in these variables. It is often very useful to compare this graph with the previous one and to see which of the two best correlates with changes in strength and density.

Density @ test minus average: In most countries, regulations call for density to be measured at test. This is far better than not measuring it but there is no reason for not obtaining the data earlier by measuring at receipt. 'Average' refers to the average of the group of specimens all made from the same sample of concrete. The lightest two ingredients of concrete are entrained air and water. The heaviest is cement. Therefore if a sample of concrete is lighter than previous samples it will almost invariably be weaker. If a sample is weaker without being lighter, then it is time to consider cement quality, curing temperature or contamination OR testing error.

Range of density at test: This is the difference between the heaviest and lightest specimens in a group from the same sample of concrete. Obviously it will be more significant if it includes all such specimens, i.e. if it is measured on receipt. This variable is a measure of consistency in compaction (or perhaps in specimen dimension measurement). If a cusum of this variable shows a change point, it points to a change in the quality of testing.

Density @ receipt – See earlier. Testing on receipt is greatly preferred to testing at the time of compression testing because it enables earlier detection of problems.

Density @ test minus density @ receipt: Concrete will gain weight if properly cured. A change point in a cusum of this difference would indicate some change in compaction or curing. It is probably not worth testing twice to obtain this value but if you are unfortunate enough to have someone insisting on density at test, then it is still worthwhile to test at receipt and if the data is there, it may be of interest to see what it can tell you.

Density @ receipt minus that calc ex actual batch quantities: The program automatically calculates density from actual batch quantities if these are obtained. A change in this difference probably means water is being added after leaving the plant, or possibly that test specimens are being allowed to dry out before collection, or are incompletely compacted. The figure should usually be negative. If it is positive it may mean that specimens are not being measured.

Plastic density: This is infrequently measured in Australian and European practice but quite frequently measured in USA. In principle it is a very significant property obtained at a commendably early stage. In practice it is rarely obtained with sufficient accuracy to do more than detect gross faults (except in lightweight concrete). It is possible that this view is now too pessimistic as accurate and robust weighing devices, suitable for field use are now available. A rigid container of known volume is no problem and can be combined with a glass top plate ('striking off level' is not good enough).

Concrete temperature: This has a significant effect on water requirement and should always be measured and recorded since it involves almost no cost in time or equipment (a metal dial thermometer should be used to avoid breakage but should be calibrated, as they are often substantially inaccurate). The temperature of the concrete at the time the slump is measured is the reading required.

Air temperatures: Generally air temperature is irrelevant but may possibly explain some surprising strengths if very high or very low.

Early Age minus average age and strength and predicted strengths at control age and 28 days: There may be more than one specimen at an early age and recording the actual average values may help to explain any errors in prediction. What really matters is the predicted 28 day values, but any tendency to error in prediction should be detected and its source tracked down. The *K* value is the slope of the line on the strength v log equivalent age graph prior to the control age (Section 11.4). This value should not change. If it does it may be that the selected control age is too late or that cement or admixture properties have changed. I have suggested using this in Singapore as a means or detecting a change in source of cement clinker.

Intermediate Age minus average strength, range, and predicted 28 day strength: There will not often be more than one intermediate age specimen but if there are, the average is used and the range is available. Unlike Early Age, Intermediate Age is a fixed (by the user) age and so statistics at this age are valid. Predicted 28 day strength is automatically available after the first 28 day result is entered (together with an Intermediate Age result). When no 28 day results are available it may be worthwhile to enter one estimated value to enable graphing of predicted 28 for the first 21 days.

Gain to 28 days is to some extent a property of the cement and a change in it may indicate a different source of clinker or fineness of grind. It can also be caused by a change in admixture or admixture properties or a difference in concrete specimen curing.

Control Age minus average strength, range and predicted 28 day strength: As for intermediate age above. Called Control Age because control decisions cannot be left to 28 days. We have established that it is more accurate to add the average gain from control to 28 day to the control age result rather than to regard this as some percentage of the 28 day result. This also applies to intermediate ages in excess of 2 to 3 days, but not to earlier age results.

28 day minus average strength, range, running means: 28 day strength is assumed to be the main quality criterion, but it is too late to be used for control purposes. Average Range over a period between the usual pair of results, sometimes three, is the main criterion of testing quality. For pairs it should be between 0.5 and 1.0 MPa for good testing. Over 1.5 MPa is unacceptable. Individual pair differences of up to 2 MPa are not unusual but above this a cause should be sought and the lower of the pair probably discarded. Cusum graphs are best to show when there has been a significant change in average strength or testing quality but direct plots show better when individual results are influenced by testing quality, slump, temperature etc.

The program permits automatic calculation and graphing of any two different running means. Running means of three are used by ACI and of four by UK for specification purposes, five may be an even better choice from the control viewpoint. The second running mean may be of 10, 20 or even 30 to give a more stable value.

28 day minus actual or predicted: Some ConAd clients prefer this graph. It plots 28 day results where available and then continues to plot 28 day predicted from control age so far as that is available, and then from intermediate age and finally early age to give the best current estimate of the situation on a single graph. The author prefers to look at these graphs separately.

28 day minus predicted ex slump, temperature and density: A complete prediction based on the above is not possible but what is evaluated is the difference from average explainable by changes in the above. If a change point on the actual strength cusum is mirrored by this graph, the cause is obviously clear. If it is not so mirrored then the cause may be testing error or a change in the quality of cement, admixture or aggregates etc.

Late age: Worth plotting if worth testing.

Actual 28 day strength (and predictions of it) minus required mean, F'c, or target strength: Either of these are very useful variables as direct plots on the same axes as a cusum of actual strength. It is particularly useful to plot both actual and predicted differences on the same graph as it tends to make errors stand out. If 'failures' are being shown by both actual and predicted result graphs at a time when the cusum shows a downturn, that is to be expected. However if the two results do not agree and there is no cusum downturn, testing or sampling error is indicated. If the two results are in agreement about a low result but there is no cusum downturn, the problem is with an individual truckload. It may be overslump, an excessive time on site, inadequately mixed or badly sampled, cast or tested. These are some of the very few direct plots that are useful on a multigrade basis.

Calculated strength ex plant water or ex calculated water: It is very useful to plot both these two together with actual strength as either cusums or direct plots. Again what is being calculated is a difference from average rather than an independently evaluated strength. The interest is in establishing the reliability of data by seeing which variables change together.

Total cement divided by actual or predicted strength: As a direct plot, this may be useful in showing the 'cement efficiency' (in kg/MPa) of various mixes. In circumstances where this data is available (i.e. for computer batching plants) it is not very likely that cement content will be the major cause or variability, so a good correlation is not often obtained. These days cement replacement materials are frequently involved, making a good correlation even more unlikely.

Sample delay: This is not often the problem but it is useful to be able to plot it when it is under suspicion. An increase in sample delay is equivalent to an increase in slump since slump reduces with time (unless water has been added of course).

Air percentage: Where being tested, it is interesting to compare this with strength, particularly to see at what point air content causes strength loss. (2–4% of air can reduce water content enough to avoid strength loss in lower strength mixes.)

Percentage voids (in a concrete test specimen): Percentage voids is calculable from the mix design, material specific gravities, and actual concrete density. Incomplete compaction is generally considered to cause a loss of 4 to 5% of strength per 1% loss of density. Also the heaviest component of concrete is cement and the lightest is water (or air) so lighter concrete can be expected to be weaker.

Specific surface: This is the surface area of the sand. An increase in surface area will cause an increase in water requirement and therefore a strength reduction if not compensated for. Alternatively the program can calculate the SS of combined aggregates taking into account batching errors also.

Yield: The program calculates this for every batch of concrete. Over or under yielding affects cement content per actual cubic meter and therefore strength. Of course it also has intrinsic economic importance but this should be dealt with under Production Analysis.

Cement strength: There is provision to enter a figure (approximately 1) for a cement in the Materials section. The basis on which this figure is established is up to the user but if meaningful variations are recorded (e.g. from cement test records) they can be graphed alongside concrete strength.

Cement table: Some clients read mixes from tables and vary the level in the table (effectively vary the cement content) according to current test data. If this is being done it is obviously important to take it into account in assessing current results.

Cement type: It would not be very usual to change the cement type in use for a particular mix, but in a multigrade analysis some grades may use a different cement than others and it may be important to check if downturns or upturns are associated with a particular cement.

Cement, fly-ash and silica fume contents: Cement and Cementitious contents are available if batch data is being obtained and it is obviously important to take this into account in assessing the performance of a single grade. Equally obviously it will not be helpful to include cement content in a multigrade analysis.

Batched aggregates and admixtures: As with cementitious materials.

User defined docket or specimen data: This allows any desired item recorded about each batch (in Test Data Entry) to be graphed. There are obviously too many alternatives to include all but any item can be selected.

Slump minus (slump after SP): The slump difference obtained by using a Superplasticiser (= High Range Water Reducer) is of interest. If the above is what is actually calculated it will be negative.

Specified slump: Where a grade of concrete is being supplied to a variable slump requirement (perhaps to different purchasers or to different parts of a structure) it is useful to graph this whether or not the cement content is being adjusted in an attempt to keep strength constant.

Total cementitious: This is an available option for graphing but it may be better to graph the individual materials if separately recorded.

Grade strength: It may be useful to have this as a horizontal reference line in a single grade analysis or in conjunction with a multigrade cusum analysis to see if change points are particularly associated with high or low strength grades.

DIN Flow (or nowadays slump flow) test: This can be recorded in place of slump and will be equally useful in an analysis if for very high workability concrete.

Volume of permeable voids: This is a test particularly used by the Victorian Roads Authority (VicRoads). It may correlate well with durability and, if being entered, if would certainly be of interest to see what correlates with it.

Average of last 4 divided by target strength: This is a useful check on the situation, particularly in UK where running mean of four is a specification figure. It may permit multigrade data to be usefully analysed in this way although we would prefer to see cusum analysis for multigrade data.

Advice on selection of graphing options

Order of priority of actions

- The first priority is to avoid producing excessive future failures.
- The second priority is to detect any future downturn in results at the earliest possible moment.
- The third priority is to take advantage of any readily obtainable savings.
- The fourth priority is to optimize all mixes.

AVOID PRODUCING EXCESSIVE FUTURE FAILURES

On completing entry of the day's results (or more frequently if desired, for example, if some results are indicating failures as they are being entered) a Multigrade Analysis of all results should be carried out. It may be desirable to exclude some results from very specialized uses ('not really concrete') but the consequence of not excluding even tests on mortar or no-fines concrete specimens are unlikely to be serious. The analysis would typically cover the last two months so as to include some reliable 28 day data.

The program displays a list of every grade of concrete present in the analysed data and should be set to list these in order of departure from target strength. The items listed along each row are at the user's discretion and should be set to include at least number of results, number and percentage of failures and number and percentage of predicted failures. It may also be useful to include mean strength and standard deviation at both control age and 28 days plus gain from control age to 28 days. Early age and Intermediate age results may also be useful but if available have probably already been utilized in specialized control.

From the above listing it should be easy to see if any grades require immediate intervention. Generally the approach to intervention should be to over-correct for under-performance. Later it will be seen that under-correction is advised for over-performance. For example, it may be assumed that 8 to 10 kg of cement should be added for every 1 MPa of strength shortfall but only 4 to 5 kg should be removed per 1 MPa excess strength.

If some grades with excessive failure rates have very few results, it may be worth looking at such grades individually as direct plots before making changes. This is provided the further investigation will take place quickly.

Any grades showing standard deviations well in excess of the 'Basic Standard Deviation' shown at the top of the screen probably include a distinct change point and should be scheduled for early investigation as single grade. Grades showing substantially higher or lower strength gain to 28 days than other grades of similar strength may also merit closer investigation.

DETECT ANY FUTURE DOWNTURN IN RESULTS AT THE EARLIEST POSSIBLE MOMENT

The next action should be to view a multigrade cusum graph of all the analysed results. If this does not show a recent downturn then it is not urgent to proceed further. If a downturn is seen, then it must be investigated. The first graph variable chosen will usually be the control age strength. Some clients prefer to use Actual or Predicted 28 day but the author prefers to see control age, 28 day and intermediate or predicted ex early age as separate variables. It is not useful to select both control age strength and 28 day prediction from control age strength as cusum graph variables as these two are identical and will be seen as one graph. On the other hand they are both very useful on a single grade direct plot.

Other variables should include at least density, slump and concrete temperature. If not too confusing on screen, average pair difference and control age to 28 day gain are also valuable. If a problem of that nature is suspected, sample delay may be worth plotting (but more often as a direct plot than a cusum since it is usually an individual truck problem).

Any downturn in strength will normally be accompanied by a downturn in density. If specimens are weighed and measured on receipt, the density graph may run 6 days ahead of a 7 day control age strength graph. This will assist in deciding whether the downturn is significant or only a statistical variation (or testing error).

Similarly a strength downturn is often accompanied by either a slump or temperature upturn (since both of these would increase water requirement). If either of these is the case then the cause of the downturn is known and cement contents can be adjusted if the higher temperature is expected to continue or the increased slump is to be allowed to continue. If these factors are not responsible, then batch quantities of cement, sand etc, may provide an explanation.

Alternatively an uncompensated increase in sand fineness (also increasing water requirement) could be the problem. Finally air content checks would be desirable. If a strength downturn is NOT accompanied by a density downturn, the cause is not additional water and is unlikely to be a cement quantity deficiency. The next suspect might be testing error and a cusum graph of average pair difference should be consulted. This graph often shows a change for either better or worse when a new testing officer is appointed (although this may only show if there are relatively few testing officers). Of course this graph runs 28 days behind concrete production, which is too long to await corrective action. Some indication of any such problem may be obtained from a cusum of Range of Specimen Density as this would indicate less care in compacting (or re-mixing concrete samples for) test specimens.

TAKE ADVANTAGE OF ANY READILY OBTAINABLE SAVINGS

If the initial multigrade analysis screen has been set (as recommended) to be sorted in order of departure from target strength, it will be easy to see any mixes which have an excessive strength margin. Cement reductions should be conservative, especially if based on small numbers of test results in the grade in question.

OPTIMIZE ALL MIXES

This should be the main objective of control. The above steps are sometimes referred to as ‘firefighting’ and should be infrequently required in a well-organized operation.

Perhaps the first item to check is the quality of testing, since everything else depends on it. The best measure of this is the average pair difference between pairs of specimens tested at the same age from the same sample of concrete. Decades of experience have established that it is very rarely possible to reduce this figure below 0.5 MPa (say 75 psi) and that it should be possible to achieve a figure below 1.0 MPa (145 psi). A figure of 1.0 MPa means that, on average, the mean value of a pair of tests will be at least 0.5 MPa below its true value ('at least' because even the higher of the pair may well register less than the true value). This is a cement cost of 2 to 5 kg/m^3 on every cubic meter of concrete produced, allowing an assessment of how much it is worth spending to improve the situation. If your average pair difference exceeds 1.5 MPa, it is time to make a substantial effort to reduce it. High strength concrete can also be tested just as accurately but the consequences of any shortfall are magnified. The above pair difference values could be increased by 0.5 or even 1.0 MPa for strengths above 50 MPa.

Next comes the variability of the concrete itself. The 'Basic SD' at the top of the multigrade report screen is the best evidence. In the UK, this is the way SD is officially determined. In other countries the method is often regarded with suspicion because the value so obtained is usually lower than that obtained for each individual grade by more conventional derivation. The reason for this is that any change points in average strength inflate the latter but not the former. Also individual exceptionally high or low values (often the result of error, testing or otherwise, rather than normal variability) have much more effect on the latter than the former.

The basic SD should be of the order of 2.0 to 2.5 MPa and such values have been obtained even on 100 MPa (say 15,000 psi) concrete. If your figure is substantially higher, then again this is costing you money. This time the value is going to be multiplied by 1.28 or 1.65 so the cost per additional MPa could exceed 10 kg of cement.

A reduction in basic SD is only to be achieved by examining the influence of variability in batching, in slump, in temperature, and in cement and sand quality.

On the other hand grades showing a substantially higher SD than the basic should be individually examined to see what the problem is. Such an examination might start with a multivariable but single grade cusum, to see whether there was a change point and if so what correlated with it. This may be followed by a direct plot, (also multivariable), looking for individual high or low strengths and the causes of these.

When variability has been reduced as far as possible, all grades can then be adjusted to give the mean strength required.

Cement margins

Cement margins, and the following program, Benchmark, are part of the proprietary ConAd program and can only be directly implemented as shown by purchasing that program from Command Alkon. However a brief summary of these programs is presented here to illustrate the concepts and perhaps inspire others to similar, and perhaps further, developments.

The concept of the Cement Margins Program is to examine past results to see whether or not they are giving the desired target strength. It is designed to help

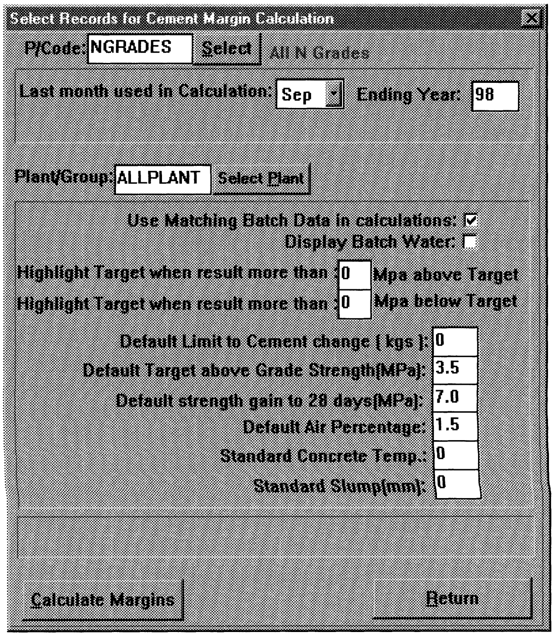


Figure 4.16 Cement margins record selection screen.

the operator quickly notice areas where either a saving of cement can be made, or an increase of cement is required to reduce the risk of rejection.

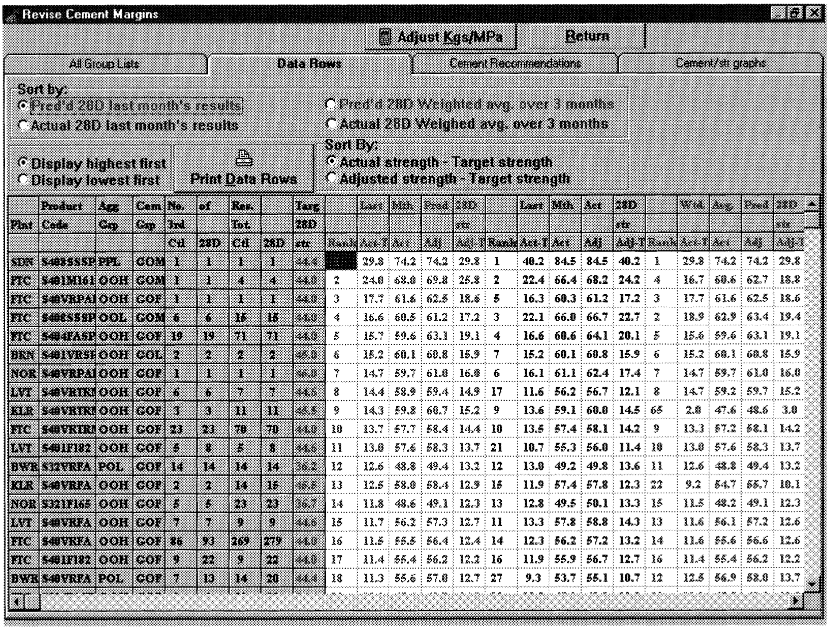
It serves two purposes:

- To fine tune cement contents for maximum economy.
- To serve as an initial alert on problems requiring investigation.

The program separates QC Test data into groups with the same;

- Month
- Product Code (Mix)
- Plant
- Cement Group (A different group is automatically set up for every combination of cementitious materials, when processing batch data)
- Aggregate source group
- Admixture may be included later.

The screen display (Fig. 4.17) and the basic printout are in order of the MPa deviation from target. This therefore highlights, at opposite ends of the list,



Plnt	Product Code	Agg Grp	Cem Grp	No. 3rd Cd	of 28D	Res. Tot. Cd	28D	Targ 28D str	Rank	Last Act-T	Mth Act	Pred Adj	28D str Adj-T	Rank	Last Act-T	Mth Act	Act Adj	28D str Adj-T	Rank	Wtd. Act-T	Avg. Act	Pred Adj	28D str Adj-T
SDN	S403SSSP	PPL	GOM	1	1	1	1	44.4		29.8	74.2	74.2	29.8	1	40.2	84.5	84.5	40.2	1	29.8	74.2	74.2	29.8
FTC	S401M161	OOH	GOM	1	1	4	4	44.0	2	24.0	68.0	69.8	25.8	2	22.4	66.4	68.2	24.2	4	16.7	60.6	62.7	18.8
FTC	S40VRPAI	OOH	GOF	1	1	1	1	44.0	3	17.7	61.6	62.5	18.6	5	16.3	60.3	61.2	17.2	3	17.7	61.6	62.5	18.6
FTC	S408SSSP	OOL	GOM	6	6	15	15	44.0	4	16.6	60.5	61.2	17.2	3	22.1	66.0	66.7	22.7	2	18.9	62.9	63.4	19.4
FTC	S404FASP	OOH	GOF	19	19	71	71	44.0	5	15.7	59.6	63.1	19.1	4	16.6	60.6	64.1	20.1	5	15.6	59.6	63.1	19.1
BRN	S401VRSI	OOH	GOL	2	2	2	2	45.0	6	15.2	60.1	60.8	15.9	7	15.2	60.1	60.8	15.9	6	15.2	60.1	60.8	15.9
NOR	S40VRPAI	OOH	GOF	1	1	1	1	46.0	7	14.7	59.7	61.0	16.0	6	16.1	61.1	62.4	17.4	7	14.7	59.7	61.0	16.0
LVT	S40VRTRI	OOH	GOF	6	6	7	7	44.6	8	14.4	58.9	59.4	14.9	17	11.6	56.2	56.7	12.1	8	14.7	59.2	59.7	15.2
KLR	S40VRTRI	OOH	GOF	3	3	11	11	45.5	9	14.3	59.8	60.7	15.2	9	13.6	59.1	60.0	14.5	65	2.0	47.6	48.6	3.0
FTC	S40VKTRI	OOH	GOF	23	23	70	70	44.0	10	13.7	57.7	58.4	14.4	10	13.5	57.4	58.1	14.2	9	13.3	57.2	58.1	14.2
LVT	S401F182	OOH	GOF	5	8	5	8	44.6	11	13.0	57.6	58.3	13.7	21	10.7	55.3	56.0	11.4	10	13.0	57.6	58.3	13.7
BWR	S32VRFA	POL	GOF	14	14	14	14	36.2	12	12.6	48.8	49.4	13.2	12	13.0	49.2	49.8	13.6	11	12.6	48.8	49.4	13.2
KLR	S40VRFA	OOH	GOF	2	2	14	15	45.5	13	12.5	58.0	58.4	12.9	15	11.9	57.4	57.8	12.3	22	9.2	54.7	55.7	10.1
NOR	S321F165	OOH	GOF	5	5	23	23	36.7	14	11.8	48.6	49.1	12.3	13	12.8	49.5	50.1	13.3	15	11.5	48.2	49.1	12.3
LVT	S40VRFA	OOH	GOF	7	7	9	9	44.6	15	11.7	56.2	57.3	12.7	11	13.3	57.8	58.8	14.3	13	11.6	56.1	57.2	12.6
FTC	S40VRFA	OOH	GOF	86	93	269	279	44.0	16	11.5	55.5	56.4	12.4	14	12.3	56.2	57.2	13.2	14	11.6	55.6	56.6	12.6
FTC	S401F182	OOH	GOF	9	22	9	22	44.0	17	11.4	55.4	56.2	12.2	16	11.9	55.9	56.7	12.7	16	11.4	55.4	56.2	12.2
BWR	S40VRFA	POL	GOF	7	13	14	20	44.4	18	11.3	55.6	57.0	12.7	27	9.3	53.7	55.1	10.7	12	12.5	56.9	58.0	13.7

Figure 4.17 Cement margins: 'full screen view of data rows'.

groups posing a risk of failure (or requiring further investigation) and groups where an opportunity exists for saving cement. The group summaries clearly show the relative economy of alternative materials.

The quality of information produced from the computer analyses is dependent on the quality of data entered. The program will reliably indicate excessive and inadequate margins but it may require operator expertise to determine their significance. Groups may vary from target due to factors other than a currently incorrect choice of margins and future mixes should not be adjusted for factors which may not apply in future. Such factors may include slump or temperature variation and testing error. In an effort to overcome these potential sources of an inaccurate analysis result, three kinds of checks have been built into the analysis:

- The analysis separately examines the most recent results and a weighted mean over the past three months.
- The analysis separately examines actual 28 day results and 28 day result predictions from 7 day results (to highlight any testing errors in addition to giving greater immediacy).
- In addition to 'actual minus target' strengths, the program also displays 'actual minus calculated' strengths. This alerts the operator to deviations

caused by abnormal slumps or temperatures (which should not be allowed to affect margins for material quality variation). The ConAd program is capable of generating mix revisions for individual mixes to take into account such circumstances, if foreseen, but few clients currently make use of this facility.

- The program also generates strength data adjusted for these deviations for use in determining desirable adjustments to future mixes. Such adjustments are obtained by graphing (actually fitting an equation to) the results to smooth out variability and reading revised figures from the graph (or generating them from the equation).

The Benchmark system

The Benchmark system is designed to compare the performance of a large number of mixes in production use over a wide area (perhaps internationally) by a major concrete producer. However it could also be used as an absolute comparison standard by small producers.

The input mixes may represent different:

- Aggregate sources (crushed, rounded, smooth, rough)
- Cements
- Grades of concrete (strength levels)
- Types of concrete (workability requirements)
- Climatic conditions
- Design philosophies (degree of sandiness, continuity of gradings).

The concept is to employ an absolute standard provided by the MSF (mix suitability factor or degree of sandiness) concept, together with the water content and strength calculations forming part of the ConAd system, to compare the performance of the input mixes.

The program generates eight sets of graphs:

- Cement content v MSF (with or without a 'shape correction factor') (Fig. 4.18)
- Cement content v Strength (actual or calculated)
- Cement content v Water content (actual or calculated)
- Cement content v Cement effectiveness (kg/MPa)
- Cement content v Strength ratio (actual/calculated)
- Cement content v Water ratio (actual/calculated)
- Calculated water v Actual water
- Calculated strength v Actual strength.

If a wide range of data is in fact available, users will not be at the mercy of the author's opinions of what is good, but will effectively be using their own data for the comparison.

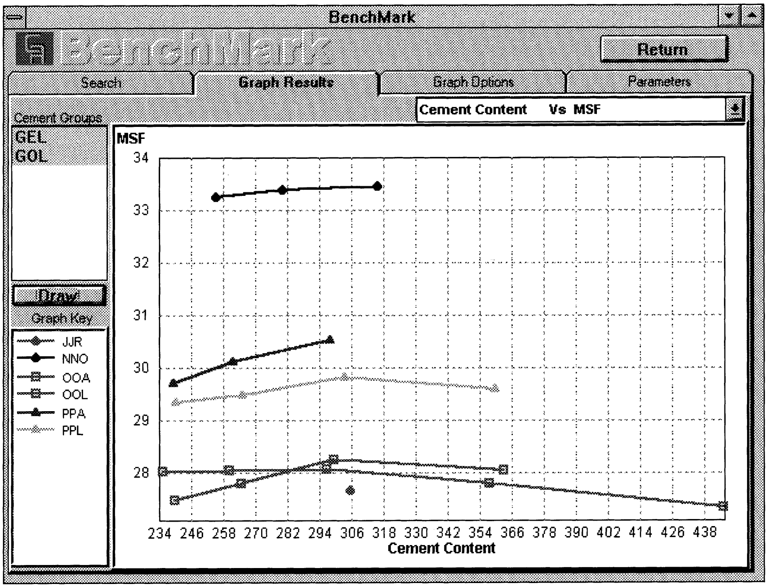


Figure 4.18 Benchmark sample graph (cement content v MSF).

- The graphs of MSF v cement content will reveal any differences in design concepts or aggregate properties.
- The graphs of actual strength v cement content will show relative cost efficiency.
- The graphs of actual water content and strength v calculated values will reveal whether cost efficiency variations are due to material characteristics, climatic conditions, or design philosophies.

The program should enable users to highlight uneconomic material sources and mix design practices, enabling differentiation between these two very different causes of excess cost and allowing for regional climatic variation.

4.13 EN206 – can we do better?

EN206 is the result of years of international committee work involving 19 countries. It therefore cannot be lightly discarded or altered, even if a distinctly better system were to emerge. However this is not to say that it is not worth examining a system clearly satisfying the intentions and outcomes of EN206 but having improved performance in some respects.

The basic concept discussed in this paper is an alternative approach to 'concrete families'. It is clearly important to include the maximum number of individual concretes in a family from the viewpoint of rapidity of corrective action and also for economy in testing costs. In order to enable this, EN206 requires several adjustments to be made to the basic test results. Such adjustments include for varying cement content, slump, admixture usage and pumpability.

The benefit of including more members in the family may be to some extent offset by an increase in the apparent variability due to the adjustments being less than perfect. It also involves a requirement for continually checking the validity of including each member in the family. Even with the adjustment formulas the range of mixes that can be included in a single family is quite limited.

Essentially the EN206 families concept is to adjust the actual test data on all the members of the family so that it is valid to analyse them as though from a single control mix. The alternative approach used in the ConAd system is to maintain separate running average values of all measured properties (slump, temperature, density, strength, strength gain to 28 days etc.) for every individual mix in use. This true current average value is then used as the cusum target for that property of the individual mix and the deviations from such individual targets are treated as though all targets for all mixes were the same. This approach is not a newly conceived idea but a feature that has been in very satisfactory use (in the ConAd system) by many organizations in many countries, in some cases for in excess of 10 years.

The advantages of the ConAd approach are listed as follows:

1 There is essentially no limit to the range of individual mixes that can be treated as a single 'family'.
2 There is no requirement for adjustment formulas.
3 There is no requirement for checking that constituent mixes remain as acceptable family members (except when a change point is detected).
4 As a consequence of the above, change point detection is much more rapid and multi-variable cusums become more effective in cause detection.

Points requiring further comment are:

1 Prediction of 28 day results from an early age (usually but not necessarily 7 days) in the ConAd system is by adding the average gain (which is automatically maintained for each individual mix) to the early age result. It is simple to compare the accuracy of this approach with any other approach and this has been done many times, invariably showing this approach to be more accurate than any other for early ages of 3 to 7 days. A technique employing temperature monitoring and Arrhenius equivalent age can be used to extend the prediction range down to a few hours but this is not relevant here. Divergence from the average gain figure is one of the 80 items for which a cusum can be selected on the control graphs so that any change (e.g. in cement properties) is rapidly and automatically detected.

2 All individual mixes can, with advantage, be initially treated as all from the same family. However it is also advantageous to split up mixes into several different groups for subsequent analysis following a detected change point. These groups should not be on quite the same concept as EN206 families. The basis of the groups should be solely a single common constituent not included in any other group for the same type of constituent for example, a cement, an admixture, or a fine or coarse aggregate. Individual mixes will appear in multiple groups (potentially one for each constituent material) and the system enables such groups to be rapidly examined in turn until the change point is seen to be isolated in one of the groups, thereby identifying the cause of the change. However it is often unnecessary to initiate such a search as the cause is frequently seen at first sight to be one affecting all groups (such as concrete temperature, or slump, or testing quality as shown by an average pair difference cusum).

3 After detection of a change point in any group of mixes (or the properties such as grading of any constituent of a group of mixes, or the standard deviation of a group of mixes) a program 'Mixtables' will be run to adjust the mix proportions of all members of the group. The program takes into account the need to balance the requirement for prompt corrective action against the risk of acting inappropriately on the basis of a limited number of early age results.

4 On pressing the single key to initiate a multigrade analysis of all mixes, the first screen automatically tabulates a large range of properties of every individual mix (one mix per line). The properties are pre-selected by the user from a list of over 40 that includes the mean strength and SD at several ages, and the number and percentage of any actual and predicted failures. The list can be (and normally is) presented in order of either predicted or actual divergence from target strength. Therefore it is easily seen if any individual mix is under or over performing and whether that performance assessment is or is not based on a significant number of results. The list currently does not include conformance to EN206 requirements but could easily be modified to do so.

5 Because the multigrade analysis requires virtually no effort and very little time (pressing a single key and waiting up to one minute) it can, and should, be run every day on conclusion of result entry for the day. Because the analysis is so completely up to date at all times, and so specific to each individual mix, a generous attitude can be taken to adjustments for a single mix. Either that mix will be of negligible economic importance to the producer (if not many results) or adjustment of any excessive correction will not be long delayed (if many results).

6 The system calculates a 'basic standard deviation' which is essentially that required by EN206. It also calculates an SD for each individual mix on the traditional basis. Where the latter is in excess of the former (and is based on a reasonable number of results) this alerts the user to the likelihood of a change point having occurred in that mix during the analysis period.

Summary and conclusions

It is important not to call into question the basic requirements of EN206 as this could delay the co-ordination of all European countries, which is so desirable and important. However it is considered that a system should be taken to satisfy the requirements of EN206 if it continuously and automatically applies all conformity criteria to each individual grade and to all combined results as a single family and those criteria are satisfied.

Because the author's system does not involve adjustment of results to a control grade, it requires much less skill and diligence to operate and much less previous data and expertise to initiate. It is also much very much quicker and easier for an inspector or observer to check whether a plant, or an entire organization, is providing satisfactory concrete.

Experience in operating the ConAd system over many years is that its capability for rapid detection and correction of the occurrence and cause of change, results in low overall variability. Furthermore it results in the proportion of results being lower than mean minus $1.645 \times$ SD being almost invariably of the order of 2 to 4% rather than the statistically anticipated 5%.

To the best of the author's knowledge, SDs in the range of 4 to 6 MPa are regarded as normal in the UK, whereas the author regards 2 to 3 MPa as being normal amongst ConAd clients. This requires a control margin of 7 to 10 MPa in the UK as opposed to 4 to 5 MPa common amongst ConAd clients.

EN206 is not really a control system but rather a means of checking whether mixes are under satisfactory control. In the UK an organization known as QSRMC (quality scheme for ready mixed concrete) is the real control system. This system does include cusum analysis of multigrade strength results but, as with EN206, uses results adjusted to a control grade and in limited families. A V-mask is used to establish whether or not a downturn (or upturn) on a cusum graph is a significant change or a statistical aberration. Such a mask is a simple and efficient way of applying a rigid mathematical test of significance. It completely ignores the fact that, in the ConAd system, multivariable cusums are used to reveal the cause of any strength change. If the explanation for a strength change is revealed, its actual significance is established regardless of its mathematical significance. Added to this that related variables such as density, slump and temperature can run 6 days ahead of even a 7 day strength result, whereas QSRMC cusums are normally based on 28 day results. It would seem that being able to make mix corrections in much less than one tenth of the time may be a major part of the reason for the ConAd system typically achieving half the variability common in the UK.

4.14 Use of ConAd test result entry and data analysis systems for early age

In Section 11.5 several systems of obtaining early age in situ strengths using maturity meters are given. However it appears that only the author's ConAd system uses an early age test in a QC system to predict 28 day strength, the strength at any nominated age, or the age at which any nominated strength will be reached. The system also provides the facility for the user to input maximum and minimum estimates of the temperature decay after switching off steam or other heat curing. The system can then advise the time at which heat curing can be switched off and still reach a nominated strength at a required actual time (which has enabled some clients to achieve a worthwhile saving on heating costs).

The early age strength is simply entered in the normal result entry system giving the age as an equivalent age in hours. The system immediately gives a predicted 7 and 28 day result just as it would if a normal 3 or 7 day result were entered.

When later standard cured 7 and 28 day results from the same sample of concrete are obtained and entered, the system automatically uses these to update its prediction constants.

It is not necessary to view graphical information to operate the process, but it is certainly desirable to do so from time to time to check that the system is operating accurately. The individual error of prediction will of course be seen as 7 and 28 day results of the same sample are entered. Fig. 11.4 displays the slope of every sample on the *strength* v *log EA graph* along with a red line showing the current average (which will be used for the next prediction). The system can also plot a direct or cusum graph over any selected period of the *K* value (the slope of the *strength* v *log EA graph*) along with any other desired graphs.

4.15 Batching control (by Don Bain)

Most producers take for granted that modern computer controlled concrete batch plants are capable of sustained, repeatable and accurate concrete production. In fact, they are, but not without continuous monitoring and adjustment. All computerized batch control systems have some type of error monitoring and alerting system and they all make errors from time to time. There are two types of errors which can be made, an over batch or an under batch of one or more materials. Generally the type of error reporting is the same for both and *both can usually be overridden by the press of a specific key*. Plant operators tend to get complacent and override both types of error, even though an under batch is generally easily corrected. In the real world what tends to happen is that few batching errors get corrected and occasionally problematic concrete is delivered to the jobsite.

It is important that quality control personnel are aware of, and in control of, the batching process. One of the easiest ways to do this is with an automated evaluation and alerting system. One such system is the ConAd BatchWatcher software. This software integrates with the PC based batching software and in effect looks over the shoulder of the plant operator. When a load of concrete is batched, what is actually loaded is evaluated based on criteria established by Quality Control and should that load be found to be outside the prescribed limits, an alert is generated. This alert is in the form of an e-mail and can be sent as such to an appropriate computer or in the form of a text message to a mobile phone.

Parameters for these alerts can be edited by recipient, region, plant, material and magnitude of error. These alerts arrive in the hands of the intended recipient in ample time to prevent sub-standard concrete from ever arriving on a jobsite.

Most concrete companies measure their batching accuracy by cumulative end of the day, week or month method. That is, if the cumulative total of what should have been put into the concrete, more or less equals the total that was put into the concrete then all is deemed to be well. For companies with multiple plants or regional operations, all plants are often lumped together. The quality implications of this practice are obvious. Inventory methods are not accurate enough to adequately establish the batching performance of a given concrete plant. A cumulative ending number only tells a small part of the story; it is necessary to determine the path to the ending number. Computer control systems will typically print out the ingredients of each batch and the difference between target and actual for each material. With a typical plant producing fifty to two hundred loads or batches per day it is not hard to see that vast amounts of paper are generated. It has been said that if you have the time to read what is contained in the boxes of paper, then you are probably not qualified to understand it. The answer to this dilemma is that the batching system software must analyse itself. BatchWatcher can eliminate substantial errors but the remaining errors may be sufficient to cause misleading conclusions as to concrete performance.

Small persistent errors, especially with cement, can lead to erroneous conclusions about concrete performance and compounded errors introduced at the mix design stage. This phenomenon is particularly important in multiple plant operations, especially if the two plants serve the same project. If one concrete plant tends to slightly over batch cement while another tends to slightly under batch, a false conclusion regarding required cement content will be drawn. Neither plant is producing the concrete as it was designed and the tendency is to increase the cement content at all plants to compensate. It is easy to see that concrete produced by these two plants not only will have more cement than required, but will also be far more variable, thus leading to perhaps even higher cement contents. However suitable software can separate both the batching errors and the concrete test data according to the originating plant. Uncorrected or unrecognized batching errors are one of the fundamental causes of concrete variability, and with modern monitoring systems used properly are manageable and often easily correctable. The major errors can now be controlled by the Batch Watcher system but the minor errors still need attention if low variability is to be attained.

In the ConAd system the computer automatically integrates the actual batch quantities with aggregate grading data. A combined grading (all materials, including cement, water, and even entrained air) of every truck of concrete produced is automatically put on file. On request, these are passed to an analysing computer. The latter is likely to be at one or more distant locations, such as the laboratory and the Technical Manager's desk. The system described here is designed to make it easy for supervisory personnel to check what action the operator did take, and also to see the accuracy with which the system is operating.

As noted, since at least the late 1970s systems have been available with the capacity to print out actual batch quantities. The difficulty has been the considerable volume of such data. This is such that no one with sufficient knowledge to make effective use of such data has had enough time available to analyse it as a routine. The effect has been that the data was referred to only after a problem has been discovered in some other way, e.g. a low test result. Such use discards the crucial advantage that, for the first time, a 100 % inspection facility is available.

There always has been, and probably always will be, a degree of error in the extent to which the test results truly represent the concrete batches tested (although the ConAd system assists in revealing and reducing such error). There has also been a degree of uncertainty in the extent to which the batches tested represent the whole of the concrete produced. It is this latter uncertainty that it has recently become possible to eliminate almost entirely.

The question arises, why do batching errors occur and what causes them, and perhaps more importantly what is their effect on the quality of the concrete. First of all the plant computer can be set too loose, accepting bigger errors. Changes in the physical characteristics of the batching materials can also have an effect on batching accuracy. Such things as moisture content, gradation, temperature, particle shape and resistance to flow can all have an effect on accuracy. Finally, perhaps the most important of all is the mechanical condition of the plant itself, or more correctly stated, the changes in mechanical condition over time.

With most batch computers, as the load size decreases, both the absolute and percentage errors increase. It can also be said that as the load size decreases the likelihood of an over batch increases. Many batch systems use a three stage batching sequence, sometimes called fast feed, timed feed and jogging. During any batching sequence there is also varying amounts of 'freefall' material, this is the amount of material which has come out of the storage bin but has not yet fallen into the scale. Obviously the amount of freefall is greater for small batched quantities; this is because there is farther for material to fall before it reaches the scale. The fast feed sequence is designed to get a large volume of material into the scale as quickly as possible. The quantity of material required less freefall and some predetermined safety margin is divided by the calculated or assigned flow rate for that material and the gate is held fully open until this predetermined value is met. A flow rate calculation is made and the gate is then held partially open for a specific time period. The batched amount should now be very close to the required amount. If it is not then the gate will 'jog' open for short intervals until the batched amount is within the tolerance set in the batching parameters. Batching errors for small loads can be reduced significantly if the fast feed function is disabled when batching loads smaller than a certain size. This size load will vary from plant to plant.

It is pointless to attempt to tune a Batch computer to batch accurately if the plant it is attached to is not in excellent mechanical condition. The plant must be well maintained and in good working order. Once the batch parameters for a given plant have been established, the most likely cause of increasing batch error frequency or magnitude is a changing or deteriorating mechanical condition of the plant. Over time, even a short time of a few loads, changes in batching errors are almost always a pre-cursor of impending plant problems or even breakdown. With the BatchWatcher system, more than two or three consecutive alerts from the same plant concerning the same material, almost certainly indicate that a breakdown is about to take place or in fact already has taken place. Over a longer time frame changes in batching errors are more indicative of deterioration in plant maintenance. Anything that causes gates to operate more slowly will have a significant effect on batching accuracy. Changing the parameters of the batching computer may be used to temporarily compensate for batching errors caused by mechanical issues of the plant, but should never be used as a permanent solution to the problem.

A further development has been the fitting of control and recording equipment to mixer delivery trucks (see Section 4.16). Such a system can detect and quantify the addition of water during delivery even if not from its own tank. However there remains the problem of addition of water to pump hoppers after test samples have been taken.

Perhaps someday it will be required that a continuous record of pumping pressures be automatically recorded and made available to those in charge of QC. It would be reasonably easy to detect, and even approximately quantify, addition of water from such records.

One aspect of the uncertainty is that the concrete sampled may have had a higher or lower than intended cement content (or other significant difference such as excessive sand content). It is now possible, using the ConAd system, to make a correction for such variations in the subsequent analysis, along with differences from intended slump, expected concrete temperature, air content and grading of input materials. In effect the actual test result can be converted into the test result which would have been obtained, had the sample been truly representative of the intended concrete, produced under the expected conditions, from the expected materials.

It is then possible to establish which actual batch of concrete had the most unfavourable combination of characteristics and therefore the lowest expected strength for the grade in question for any particular day or week. In effect, a test on any truck can tell you what test result would have been obtained had the truck with the lowest strength been selected. Theoretically, this may mean that only one or two samples per day need be taken for the characteristics of every truck of concrete produced to be known. This would be going too far, but certainly substantial reductions in frequency of sampling are justified.

The comprehensive analysis facility is semiautomatic but still a little too elaborate for frequent routine use during a normal day. As already noted, a substantial degree of immediate protection is already available with the BatchWatcher facility if a batching error outside a preset limit occurs. The ConAd system adds to this a facility to screen a graphical display showing, on the one screen, every error in every ingredient batch weight of every individual batch for the whole day to the time of calling the display. It takes only a few seconds to call up the display and check what variations have been occurring. Having done this, a ± limit value can be keyed in and the system will show a display truncated to the limits entered and expanded to fill the screen. It will also display and/or print out an ‘exceptions list’ of all non-conforming batches. Such a daily list (and it should be kept short) could be handed to a nominated person for further investigation. It is our experience that if this is done, the errors become fewer and smaller as time goes by.

A difficulty in analysing data is that, in spite of many technological advances, water content is often not fully reliable. To counter this, the system calculates a theoretical water content from slump, temperature and MSF value. The system can then display calculated or predicted strengths based on either or both of these water contents. For a single test result it may not be obvious where the truth lies. However multi-variable graphing over a period clearly shows the difference between defective testing, surreptitiously added water, and truly varied water demand (e.g. through grading variation, silt content and the like).

4.16 Truck-mounted mixing and workability control system

Section 11.7 deals with the measurement of workability but the author sees the future of workability control being in the fitting of automatic control to concrete trucks. In addition to workability adjustment, in-truck mixing and agitation also affect the standard deviation.

There are three main sources of variability in concrete: materials, batching of the materials, and finally mixing and agitation This section deals with the third source of variability: truck-induced variability, caused by various mixing speeds and durations, various agitation speeds and water additions and its remixing.

It has been shown that executing the same mixing cycle on every truck of the fleet and using a uniform agitation speed, combined with computer-assisted in-truck slump adjustment, result in a 30% reduction in production variability.

There is a truck-mounted system, called ‘Compu-Mix’, which was described in the last edition of this book, that takes charges of the mixing and agitation and assists the driver in workability adjustments. The first version was introduced in 1993. The system has been slow to gain market acceptance in the concrete field, even though the author has witnessed its third version in very satisfactory operation on a limited commercial scale in 1997.

The company who designed the Compu-Mix system was acquired by Systems D’Automotion DSS of Quebec city, Canada, for their expertise in truck-mounted control system using artificial intelligence and is using the technology on ‘more lucrative applications’ and has put a hold on marketing efforts for the system in the concrete field. DSS finds this market not quite ready for the Compu-Mix system, but believes that when cement reaches $200 a ton, the cement savings will be worth the investment for the concrete producer.

A probable reason for the slow market penetration, at least in the operation witnessed by the author, was dislike of the system by the truck drivers. This was partly because they felt that it indicated a lack of confidence in their ability and partly because it revealed such an exact log of their activities (including unauthorized stops etc.) This is a human relations problem outside the scope of a concrete technologist! Nevertheless the author sees such a system as being an inevitable feature of the best control systems at some future time.

The following account of the Compu-Mix system, kindly been provided by Dan Assh, P. Eng. and Christine Lemay, P. Eng. M. Sc.A. for the previous edition, has been condensed by the author for this edition:

The Compu-Mix truck-mounted mixing process and control system

Independent studies carried out by various ready mixed concrete-producers (in North America, South Africa and Australia) and by the SEM, a consulting firm founded by Michel Pigeon PhD, a leading researcher in concrete technology, have shown that Compu-Mix brings:

- Better slump control.
- Enhanced workability for a given slump.
- More consistent entrained air.
- Important reduction in production variability, whether in usual day-to-day production or intensive jobs like pouring a bridge deck in 20 hours. The reduction of variability is approximately 33%, whether the plant is dry-batch, a wet-batch, or a premix.
- Delivery time savings in certain applications.

Compu-Mix can also provide a complete management system to follow delivery operations with

- Load Histories.
- Driver-independent Truck Tracking statuses.
- Tachograph information to monitor driving habits.

Description of Compu-Mix process

The primary function of the control system is to control mixing and agitation to ensure that a specific sequence is performed, and that all trucks of a fleet can perform the same sequence. To do so, the system controls the speed and number of turns of the drum, independently of engine speed, during charging, mixing and agitation, which speeds and number of revolutions are all programmable to fit specific plant requirements. More specifically, the control system will:

- control drum speed during charging;
- perform a short high-speed mixing cycle at the plant that lasts approximately the wash time;

- perform a low-speed mixing that allows the truck to safely continue mixing while en route to the site;
- when all mixing cycles are finished, automatically slow down the drum to an optimised agitation speed, designed to keep the concrete homogeneous and fresh longer;
- measure slump and assist the operator in bringing the slump to the desired value;
- when water is added, automatically engage a specialized mixing cycle to ensure that this water is well distributed in the complete load and that water can react with the cement (this should minimize the detrimental effect on strength of the water addition);
- inhibit discharge until a certain percentage of mixing is completed (programmable 0–100%).

Compu-Mix slump control

With advances in computerization, Compu-Mix can now precisely measure the slump from 0–200 mm (0.8 in) even when part of the load has been discharged. The measurement is accurate to ±10 mm (0.4 in) and the reading is provided directly in mm or tenths of an inch, as opposed to pressure readings in psi provided by other 'slump meters'.

Compu-Mix also assists the driver in adjusting the slump. The purpose is to reach the desired value in a single attempt, saving time by avoiding the 'add a little, mix a little' guesswork and also avoiding the detrimental effect on strength brought by multiple water additions. The on-board 'Slump Change Expert System' displays to the operator, on the remote control screen, the predicted slump change (in mm) as water is being added. The driver simply opens the valve, keeps it open until he sees the desired change in slump (or target slump) indicated on screen, and then shuts it off. This operation could be automated using a solenoid valve but so far has never been required.

Finally, the mixing cycle following the water addition ensures that water will be distributed throughout the load and that the slump is uniform. This way, the

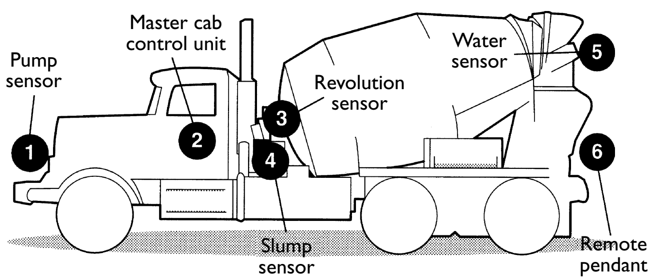


Figure 4.19 Elements of Compu-Mix workability control system.

slump will not have to be readjusted again because of insufficient mixing the first time (often mistaken for the load drying up while discharging).

Adjusting slump in truck v in plant

Even with the best moisture probes, batching at the right slump every time with a precision of ±10 mm is extremely difficult. Too many things can vary. Obviously, it is much easier to know what the slump is after the concrete is batched. The initial absorption is the highly unpredictable part. The slump is much easier to adjust after the initial hydration and absorption by the aggregates have occurred, and after having factored in any water left in the drum. Compu-Mix allows the concrete producer to take advantage of this. The procedure suggested by the Compu-Mix developers is to target the batch 30 mm below the desired slump. After 2 minutes of mixing, Compu-Mix will provide an accurate slump reading and then the driver may adjust the slump using the Slump Change Expert System, and should hit the desired slump with a precision of ±10 mm. The remaining mixing will be performed while travelling to the job, which saves time.

The science behind Compu-Mix slump control accuracy

The slump reading is more than a simple pressure reading. The developers of Compu-Mix determined that to make an accurate assessment of slump, one should take at least 20,000 readings, at controlled drum speed, and take into account the volume of concrete left in drum and also the shape of the drum and blades. Compu-Mix was then programmed with complex models using Artificial Intelligence to be able to perform a global analysis of all these factors. The method developed also has the important advantage of requiring very little recalibration due to wear of the drum.

Slump adjustment was also refined far beyond the commonly used linear function. The water required to change the slump of the load by, say 10 mm, depends of course on the volume of concrete remaining in the drum, but also on the initial slump, that is one of the reasons why these two variables are monitored constantly by the control system. The model structure in Compu-Mix can easily be calibrated to suit almost any concrete type.

Why does Compu-Mix reduce variability?

Aside from accurate slump control, executing the same mixing cycle on every truck of the fleet everyday, and controlling agitation speed has proven to reduce production variability, because mixing has an effect on strength and on entrained air. Also, the imposed mixing following any water addition insures that the water will be distributed throughout the load, again insuring a more homogeneous concrete and lower variability. Finally, an adequate and extended mixing cycle reduces bleeding and segregation, and brings enhanced workability for a given slump (study by SEM). Combined with controlled agitation at low speed, it also reduces the loss of strength occurring in longer deliveries, so that concrete life is extended (Riadh Azouzi PhD, University Laval, Quebec, PQ, Canada) and variability induced by different delivery times is reduced.

Consistent slump is a part but not all of the equation. Even if a plant produced perfect slumps with a precision of ±5 mm, different sequences of mixing-agitation performed by different drivers naturally induce variability. Wet-batch and premix plants benefit as much from the control system as dry-batch plants because none of them can control what happens after the concrete has left the plant mixer. A percentage reduction of variablity in the order of 33% has been shown in both cases, although in absolute value the reduction is normally less important in premix and wet-batch plants since the variablity when put into the mixer is usually lower.

Compu-Mix as a management system

History logs for quality management and liability protection

Although the primary goal of the control system is to avoid problems, Compu-Mix provides a complete history of the load that allows to retrace valuable information about the mixing cycle actually performed, the slump and water additions, volumes remaining, and times of the different steps of the delivery. This has helped to solve disputes between concrete producers and contractors (Fig. 4.20).

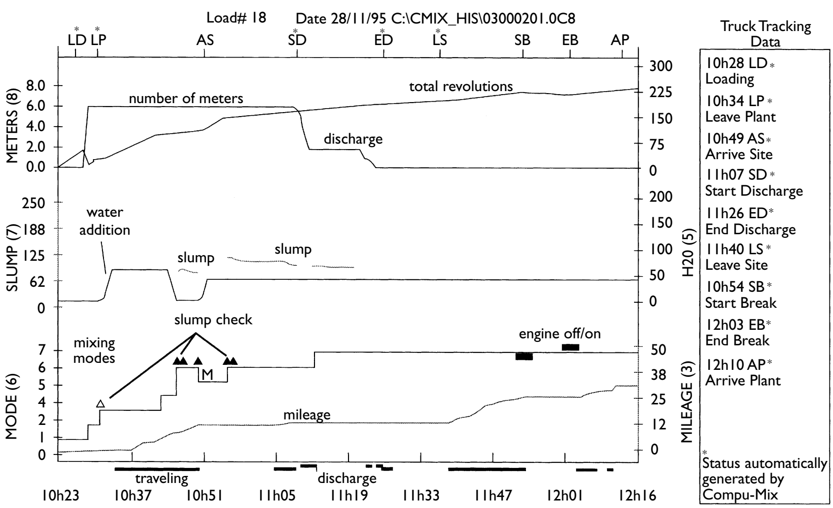


Figure 4.20 Compu-Mix history example including tachograph and truck tracking data.

The history indicates the slump as batched by the plant, the water added at the plant by the driver, the ensuing slump, the slump on arrival at site, the water added on site and the final slump. The records will also provide a time stamp of each action (water addition, discharge) and the volume of concrete left at that time.

To retrieve the histories, Compu-Mix is simply downloaded approximately once a month into a common IBM-compatible computer. These histories can then be used for ISO 9000 recording. A graphic program is supplied to rapidly visualize and analyze the information gathered.

Tachograph information and truck-tracking system

Compu-Mix is heading in the direction of a complete control system, designed to perform all tasks of monitoring and control that could be needed on a ready-mix truck. Tachograph and truck-tracking information have been added to the process control system. A major advantage of the Compu-Mix truck-tracking is that most of the statuses are automatic, and not driver entered, which reduces the chance of error and fraud. This can be done because of the intelligent monitoring performed by the control system.

The tachograph records truck mileage, speed, acceleration and engine r.p.m. This information combined with that of the Compu-Mix process control allows one to monitor driving practices, generate automatic statuses, detect some cases of time loss or fraud, like unauthorized stops, long washout times and stolen concrete.

Conclusion

The Compu-Mix system brings control to an important part of the process of making ready-mixed concrete. It includes slump measurements using Artificial Intelligence and a Slump Change Expert System to adjust slump. The studies performed have shown reductions of variability around 33%. Such reductions in compressive strength variability can bring significant cement savings when the amount of cement in a mix must be adjusted depending on the production variability. The higher the safety margin required, the higher the potential for savings.

Chapter 5

Concrete in the 22nd century

The author presented a paper of the above title to the CIA Biennial in October 2005 (now on his website, along with his PowerPoint presentation). The paper highlighted the possible limitation on the supply of Portland Cement due to CO_2 generation considerations. It also envisaged the continued rapid development and increasing complexity of materials, especially admixtures and supplementary fine materials. It suggested (as noted in Chapter 1 of this book) that the situation was likely to be beyond the knowledge of the kind of person (e.g. structural designers) who currently write specifications and that, in order neither to impose an unacceptable brake on progress, nor to risk failures, it would be necessary to require that specifications only be written by persons qualified in concrete technology and having evidence of continued professional development in the field.

The paper also implied that, while the nature of new developments in admixtures and other technology could not necessarily be foreseen, the kind of quality control and mix assessment techniques described in this book are likely to still be applicable. In particular they are likely to be more rather than less suited to a future in which a large proportion of concrete is likely to self-compacting and to include substantial replacement of Portland cement by supplementary or alternative materials.

Since that symposium, the author has enquired further into two technologies that could transform the future of concrete. These are Inorganic Polymers (better, but less correctly, known as Geopolymers) and Tec cements (being cements containing a proportion of magnesia). Having inadequate personal knowledge, the author has sought out the leaders in these two technologies and persuaded them to contribute their knowledge.

So the author sees concrete in the 22nd century as being more expertly designed, controlled and specified, more individually suited to the requirements of the purchaser and the exact situation; more likely to be self-compacting; and likely to be more durable, acid and fire resistant, and less polluting, through the use of supplementary cementitious materials or geopolymers/inorganic polymers.

5.1 Integrated mix design and QC

This section describes features of the ConAd program that have been in operation for more than a decade and some of which are even regarded as more or less superseded by Command Alkon, the current owners of the ConAd Program. Nevertheless these features remain far in advance of current practice in most of the world and therefore merit inclusion in 'Concrete for the Future'. The section goes on to describe the author's 'Just-in-Time' concept, which, although first presented at ACI Cancun in 2002, is still regarded as too futuristic for anyone to include in a commercially available program. Perhaps this book will still be in print by the time the concept becomes a reality.

All of these integrated techniques have things in common:

1. A database is required of all materials to be available. This includes aggregates and cementitious materials.
2. A database of created mixes is to be retained for future analysis.
3. A database of every batch of concrete produced is required.
4. A database of all tests on the resulting concrete is required and is to be integrated with the actual batch quantities in (3) above.
5. A formula is required to determine the w/c ratio necessary to provide an input strength. This should include a feedback from QC to improve its accuracy as test data becomes available.
6. A formula is desirable to predict the water requirement of a designed mix. The formula will certainly require a feedback or adjustment factor.

There are two ways in which mix design and QC integration can be valuable. One is to analyse test data and visual observation from the site and laboratory to gradually improve mix performance, integrating all mixes in a single analysis. The other is to use mix design data to combine selected test data from a range of mixes into a single analysis. The first of these has been one of the author's main interests for the past 50 years. The second is the basis of the 'Relational Mix Maintenance' reported in this volume and also of the EN206/QSRMC techniques used in the UK and Europe.

As described in the chapter on QC, and in the comparison of his methods with EN206/QSRMC, the author does not need to take any account of the mix designs in use in order to combine all results into a single multigrade analysis. On the other hand he has developed a number of systems to identify which mixes are under or over performing with respect to specified strength and to reveal the relative cost and cement content efficiency of the mixes in use. The techniques also identify any tendency of particular mixes to higher variability and can be used to identify any tendency for particular testing officers to find particular mixes more difficult to test (although the need for the latter may be less likely now that harsh, low slump mixes are no longer necessary for economical high strength concrete).

It is now easy to obtain accurate cement contents for every mix batched in a modern batching plant, and to link this with strength and workability test data. Accurate records of batched and subsequently added water can be obtained and both moisture probes and accurate physical tests for moisture content of aggregates are available. It seems that it should be possible to obtain an accurate water content from a sample of fresh concrete by either a volumetric analysis from water displacement or directly by microwave drying, and the author has done substantial work on the former. Nevertheless it remains difficult to obtain accurate and reliable water content data and this remains the biggest difficulty in assessing the accuracy of mix design programs.

All constituent materials test data is entered, preferably as it is produced. For example ConAd (and even the author's free program) allow actual sieve masses to be entered and automatically calculate percentages passing and retained, specific surface (and fineness modulus and logarithmic mean size, although ConAd does not currently use them). Flow value and bulk density from a flow cone test on sand (see Fig. 7.1 on p. 187) can also be entered since the author considers them to be of likely future significance (by the time you are reading this, they may have already been integrated into water requirement prediction and, if so, you will be able to read about it on the author's website). Past entries can be viewed graphically and the computer can produce the latest grading on any nominated date, or the average over any nominated period etc.

It is also possible to include a cusum graph of any entered property on the same screen as strength and other cusums – which is one place where flow values and/or bulk density could already be used if available.

Chapter 3 gives details of a suggested database for aggregates and cementitious material. The latter is repeated here because it is particularly important in understanding the Just-in-Time system.

Materials database: cementitious

All data appearing on cement test certificates should appear as dated records in the cement database. As with aggregates, any item in the database can be selected for cusum graphing along with strength test data in a search for change point correlation.

The only items actually used by the mix design system are likely to be a strength factor, a water factor and a cohesion factor. These are more likely to be opinions rather than test data although clients may choose to automatically relate these to actual test data by a formula of their own devising. The system will be more concerned with relativities than absolute values because the QC system will feed back correction factors. So if only a single cement and no cement replacement materials were used, the values could all be left at one. Where alternative cements, and especially materials such as fly-ash, slag and silica fume are in use, relative factors are required.

In the second edition the assumption was that such factors would remain constant over a range of proportionate additions, and this is built into the Automix and Mixtables programs. Analysis of production data has shown that this is not the case. It is a reasonable assumption for a simple mix design but can cause too much inaccuracy when trying to base feedback correction on a limited number of early age tests covering a wide variety of cementitious combinations – as for Just-in-Time mix design. The solution adopted by the author has been to regard each combination as a separate cementitious material – so that cement plus 20% fly-ash will be analysed as a separate cement to the same cement plus 30% of the same fly-ash. A 'wide variety' of cementitious combinations does not necessarily mean an unworkably large number of them. It is noted that, in the following section, Mark Mackenzie has adopted a different assumption, that is the relative strength values of a supplementary material will vary according to the strength level of the concrete – take your pick but do not assume fixed strength values!

The Just-in-Time mix design system is the author's latest effort. A paper of this title was presented in Cancun, Mexico (Day, 2002) in December, 2002. The formula (due to Feret) incorporated in the system is:

$$M \times 290 \times (C / (C + W + A))^2 + K$$

where:

C = cement vol
W = water vol
A = air vol

M and K are two constants to be found.

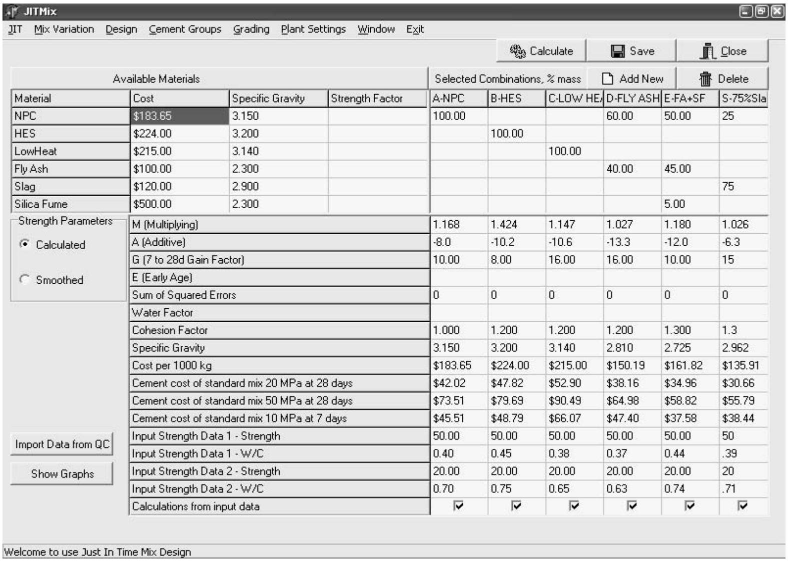
This formula makes an assumption that the strength/cement/water curves for different cementitious combinations will all be of the same shape but may have either, or both, a different basic inclination and/or a different average level.

With a single cement and a substantial number of 28 day results, together with the cement, water, and air content of each result, it is simple to originate a computer program to determine the optimum values of the constants M and K. (obviously the originator of the basic formula, Feret, intended that M should be 1 and K zero, but that was in 1894).

It gets a little more difficult to determine the separate M and K values for a number of different cements, or different cementitious combinations, each represented by a small number of early age results. The example given in the Cancun paper is shown in Fig. 5.1.

All available cementitious materials are listed in the left-hand column together with their cost and specific gravity. Optionally a nominal strength factor might also be listed. At the right-hand side are set out an unlimited number of combinations of these materials.

In the lower table the M, K (=A in formula), G and E factors are preferably obtained by analysis of test data, the 'Sum of Squared Errors' being an indication



Material	Cost	Specific Gravity	Strength Factor	A-NPC	B-HES	C-LOW HE	D-FLY ASH	E-FA+SF	S-75%Sla
NPC	$183.65	3.150		100.00			60.00	50.00	25
HES	$224.00	3.200			100.00				
LowHeat	$215.00	3.140				100.00			
Fly Ash	$100.00	2.300					40.00	45.00	
Slag	$120.00	2.900							75
Silica Fume	$500.00	2.300						5.00	
M (Multiplying)				1.168	1.424	1.147	1.027	1.180	1.026
A (Additive)				-8.0	-10.2	-10.6	-13.3	-12.0	-6.3
G (7 to 28d Gain Factor)				10.00	8.00	16.00	16.00	10.00	15
E (Early Age)									
Sum of Squared Errors				0	0	0	0	0	0
Water Factor									
Cohesion Factor				1.000	1.200	1.200	1.200	1.300	1.3
Specific Gravity				3.150	3.200	3.140	2.810	2.725	2.962
Cost per 1000 kg				$183.65	$224.00	$215.00	$150.19	$161.82	$135.91
Cement cost of standard mix 20 MPa at 28 days				$42.02	$47.82	$52.90	$38.16	$34.96	$30.66
Cement cost of standard mix 50 MPa at 28 days				$73.51	$79.69	$90.49	$64.98	$58.82	$55.79
Cement cost of standard mix 10 MPa at 7 days				$45.51	$48.79	$66.07	$47.40	$37.58	$38.44
Input Strength Data 1 - Strength				50.00	50.00	50.00	50.00	50.00	50
Input Strength Data 1 - W/C				0.40	0.45	0.38	0.37	0.44	.39
Input Strength Data 2 - Strength				20.00	20.00	20.00	20.00	20.00	20
Input Strength Data 2 - W/C				0.70	0.75	0.65	0.63	0.74	.71
Calculations from input data				☑	☑	☑	☑	☑	☑

Figure 5.1 Cement group screen.

of the accuracy of the derived factors, or the degree of scatter of the data. However to get the process started, or to cope with a new combination, provision is made to enter two test results from previous experience or trial mixes. A tick in the last row indicates that the program should determine M and K from this data rather than from an analysis of production data.

Limestone fines

For several years it has been permissible, in many countries, for a small percentage of limestone fines (say up to 5%) to be incorporated in cement. A newly emerging technology suggests that a much higher percentage of this material can be used with surprising benefits, including higher early strength. It is early days yet to determine whether the above approach is suitable for concrete using this technique, in particular whether the same shape of *W/C* v *Strength curve* will be obtained. This will determine whether the results from 'lime concrete' can be combined with those from normal concrete or must be considered alone.

Water requirement

The most difficult aspect of mix design is the prediction of water requirement. So many factors are involved that there is a temptation to nominate a likely value and simply adjust this from time to time as experience dictates. However water content is directly proportional to cement content for a given required strength, and so to the economy of the mix. Also water content variation is usually the largest factor involved in the variability of test results, again impacting on the required mean strength and therefore the economy of the mix. So it is necessary to be as aware as possible of all the factors affecting water demand from both the initial design and the quality control viewpoints.

The author's approach is to list as many of the influences as possible, to provide an empirical correction for each, and then to provide an overall adjustment factor. The user should be prepared to make a correction (as a percentage or otherwise) to any of the individual terms that appear to over or under estimate the effect but it is essential that the overall correction factor be adjusted by feedback from test data.

Any change in water content will certainly be reflected in both the strength and density of the resulting concrete. A change in slump or other workability measurement may or may not reflect a change in water content for example an increase in temperature may cause a reduction in workability at a given water content or an increased water requirement at constant workability. This is why it is so important to use a cusum of density along with strength cusums in quality control graphing.

For the author's estimation of water requirement see Table 3.4, also Fig. 3.4 and related discussion in Chapter 3.

Actual design of mixes – III

Following on the presentation of the simple program 'Automix' in Chapter 3, the next step is the author's 'Mixtable' system:

The Mixtable system

The author would like to see the practice of batching mixes from a table discontinued in favour of building a mix design facility into computer batching plants. However the customer is always right and Mixtable is an existing program within the ConAd system to generate a table with cement content ranging from 200 to 500 kg of cement (or cementitious material) in whatever cement content steps the user chooses, or in steps of 1 MPa. The program has value independently of actually being used to batch concrete, since it enables easy examination of a range of possible mixes and permits actual data to be compared, and used to modify the water and strength predictions of the system.

This system automatically produces a range of mixes from 200 to 500 kg content of cementitious materials. The user specifies (Fig. 3.9) the ratio of up to three coarse aggregates to each other and up to two sands to each other (or imports this from the Automix program). Requirements of MSF, slump, temperature, and air % are specified. The amount of fly-ash, silica fume or slag can be specified as either a fixed amount or a percentage. The generated table can be in steps of 1, 5, 10, 20, 25, 50 or 100 kg steps. (1 kg sounds ridiculous but enables a more accurate reading of cement content for a given required strength from the table.)

The table (Fig. 5.3) shows batch quantities and density and gives an estimate of compressive strength using the Feret formula.

The system permits actual concrete test data of cement content, water content, strength and density to be entered (or automatically obtained from the database on nominating the production mixes to use) and then has the facility to optimise the constants in the water prediction equation, and also the strength equations, to give the best match to the input data. Graphs are displayed of *strength* v *cement content*, *strength* v *w/c ratio*, *density* v *cement content* and *water content* v *cement content*. On each of these graphs the entered data points appear in addition to the graphs from the optimised equations. Three thumbnails of grading graphs are also provided. These expand on right clicking and are automatically printed out with each table of mixes. Fig. 5.2 shows the input screen, Fig. 5.3 a typical table of mixes, Fig. 5.4 the retrieved production data, and Fig. 5.5 the displayed screen of graphs.

Just-in-Time mix design

A paper of the above title was presented at ACI Cancun, Mexico, in December 2001. The program envisaged has not yet been integrated into a complete system and the author would be interested to hear from anyone interested in writing such a program.

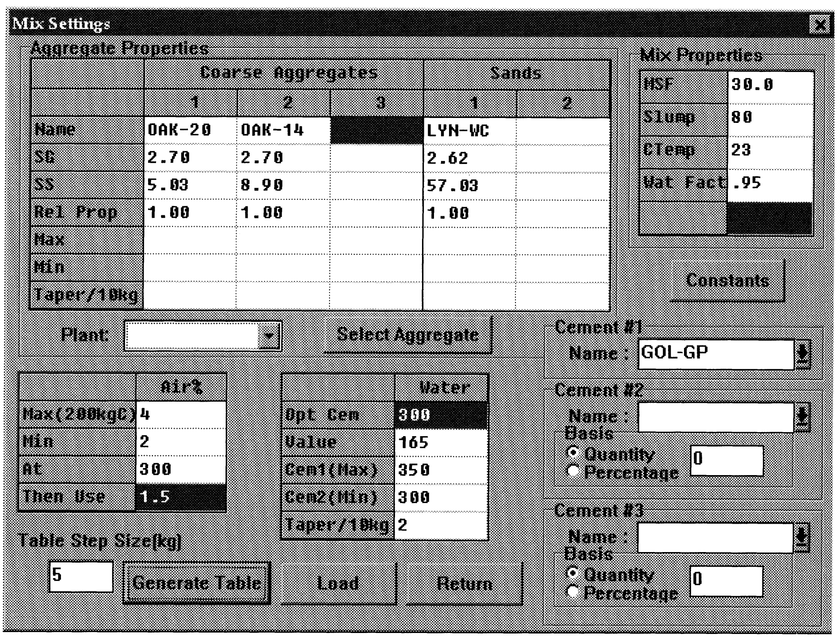


Figure 5.2 Mix Table: input design instructions/data.

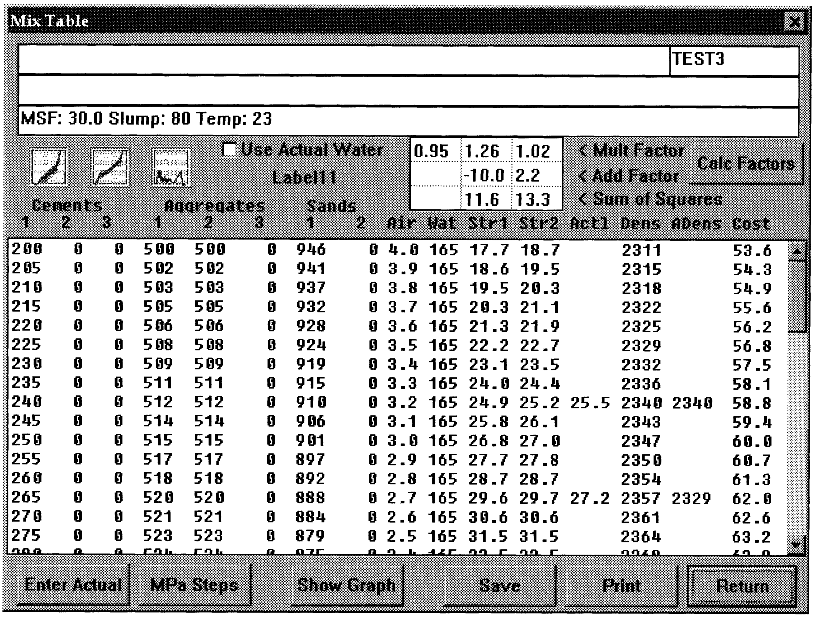


Figure 5.3 Mix Table: resulting table of mixes.

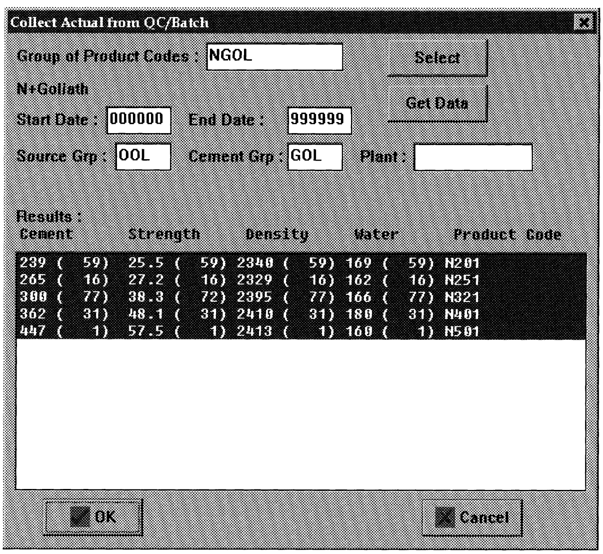


Figure 5.4 Mix Table: retrieved production test data.

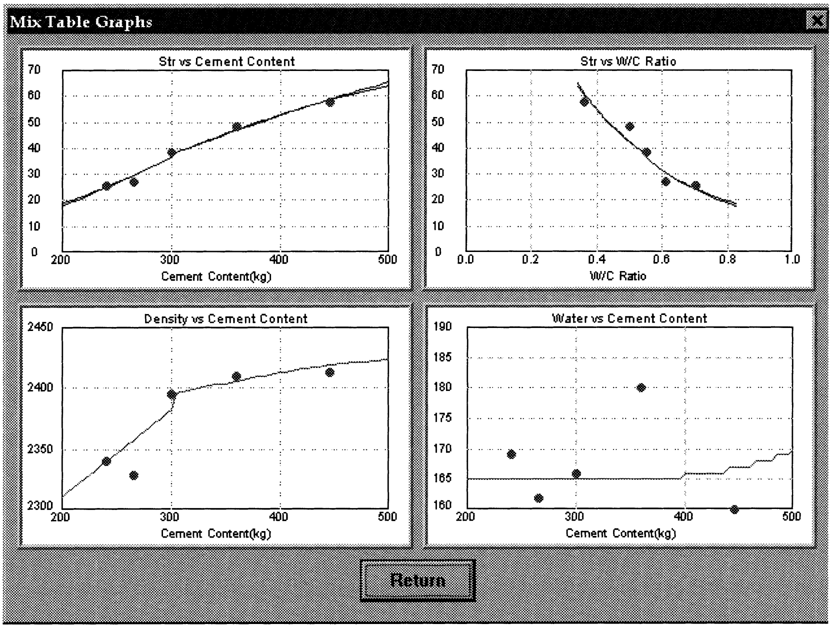


Figure 5.5 Mix Table: system equations optimized to retrieved production data.

The concept is similar to the above MixTables' program with feedback from production data and a materials database but is extended to a wide range of mixes and envisages the contents of each truck of concrete taking into account all available information at the time of batching. It has to be admitted that this is only of value if such information is itself kept up to date. If nothing else, the temperature and likely haulage time of each truckload will be known at the time of batching. However it is possible that this could be corrected for by adjusting the admixture dosage.

What is really envisaged is that, in the case of a large producer, several sections of the organisation will be operating independently. One section will be testing materials and another testing concrete. It is envisaged that test results will not only be entered promptly into the system but also analysed to detect change, probably by cusum analysis (this includes materials, it would not be satisfactory to assume that the latest result obtained, for example a sand grading, is necessarily accurate).

A particular producer may be satisfied to define workability by the MSF and a maximum Gap Index. The GI itself may be of any of the kinds discussed under 'Workability' in Chapter 2. The target grading alternative is shown here in combination with the grading screen shown in Fig. 5.6. The user need only enter the grading considered ideal in the central (% passing) row and the two figures

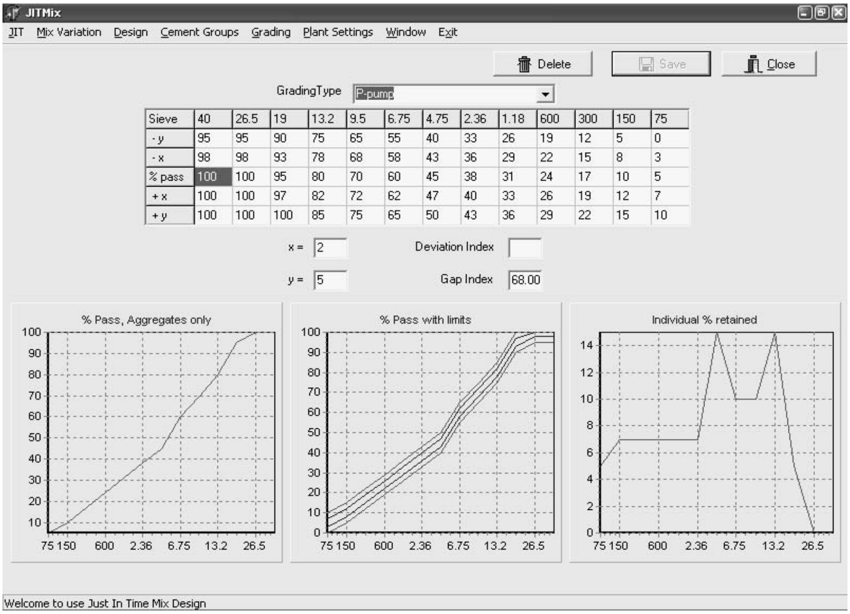


Figure 5.6 JIT gradation screen.

x and *y* below to define the inner and outer tolerances. The system then fills in the other four rows of the table and displays the ideal grading on the left-hand graph and the limits on the central graph. When a mix is actually designed, it appears superimposed on all three graphs.

The screen presented here is an attempt to show how every possible requirement could be specified rather than suggesting that all these restrictions will be necessary in every case. This system allows the entry of any number of different 'ideal' gradings, with different degrees of tolerance, for different purposes, to be entered in the system. However one of the two simpler alternatives should suffice in most cases.

Of more importance is the selection of the range of products to be marketed. A comprehensive system for this is shown in Fig. 5.7.

The mixes will be originated in ranges rather than individual mixes. A 'range' will be of mixes having the same fresh concrete properties and the same ingredients but any desired strength within a nominated range. Each range will be given a type code and description. Any minimum cement content will be nominated along with intended placement method, slump or flow, and strength range. A target grading may be specified or this may be left to an MSF and Gap Index entered on the right hand side of the screen. A water factor or a typical water content, and air % and a yield (some producers may want to discount air from the yield and others to slightly under-yield) are to be entered.

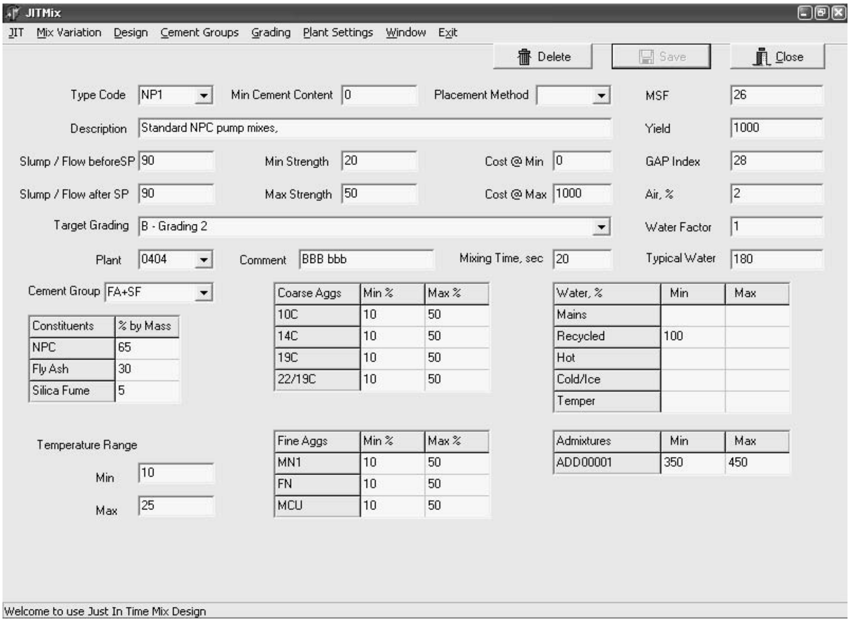


Figure 5.7 Concrete product screen.

The opportunity is provided to nominate a particular plant and a minimum mixing time and to provide a comment.

In the lower half of the screen, the cementitious combination is selected and the screen displays details of it retrieved from the database.

The coarse and fine aggregates to be used are nominated in the center of the screen and limits on the proportion of each can be specified.

The water content source is specified in detail, although the total amount will be calculated taking into account actual concrete temperature anticipated at the time of batching.

Admixtures are also specified.

An example is given of a standard pumped mix covering a strength range from 20 to 50 MPa. The main common features of all mixes in the range are the MSF, the workability and the cementitious combination.

The actual design of the mix, as indicated by the system title, takes place at the time of supply (although it may also be done at the time of ordering, in order to provide a quotation, and later adjusted if necessary).

The purchaser selects the type of mix required and, on entering the delivery address and delivery time required, the system consults internal records and nominates a plant to supply.

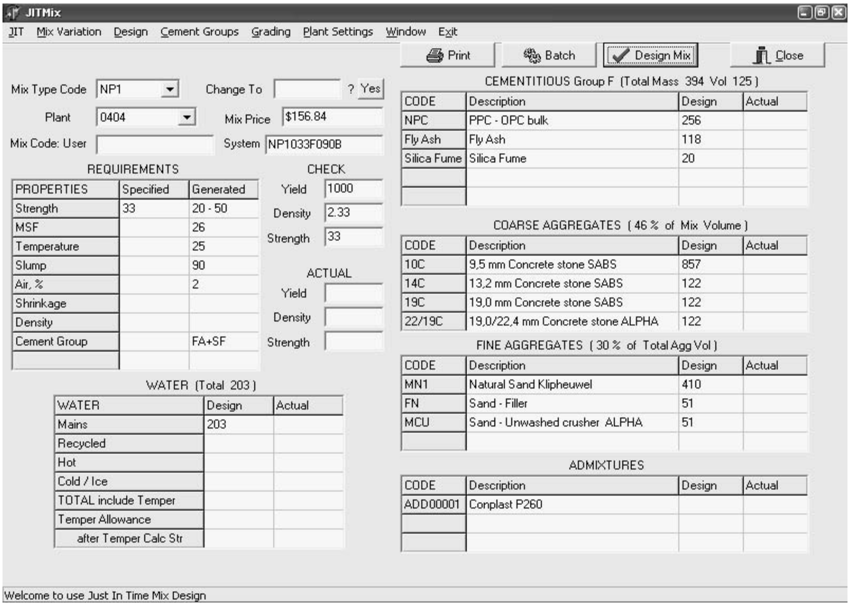


Figure 5.8 Just-in-Time proportioning screen.

The details of the mix range selected appear in the column headed 'Generated' and details of the proposed constituents appear on the right-hand side of the screen (initially without quantities) (Fig. 5.8).

The purchaser then enters the required strength under 'Specified' and keys 'Design Mix' the system can now calculate a quoted price and give quantities.

If the purchaser wishes to amend any of the details shown under 'Generated', he enters his requirements in the 'specified' column. The mix proportions and price will be seen to change to implement these requests. It is also possible that the purchaser's requests are outside the design limits of the range of mixes selected. In this case an alternative mix type will be nominated by the system in the 'Change To' box at the top of the screen and the purchaser will be asked to accept.

The 'Actual' column and boxes are for the system to automatically enter the actual quantities batched (i.e. including batching errors) and their significance for strength, density and yield. These items will be stored for later analysis and compared to actual test data on all tested mixes.

While mix details can be requested in advance, some purchasers or specifiers will, at least initially, be concerned that they do not actually know the exact mix they will receive. Fig. 5.9 shows that all properties except one can be input and the user can nominate a range for the final property (any). The system will then output a table showing how the mix will vary according to changes in this variable.

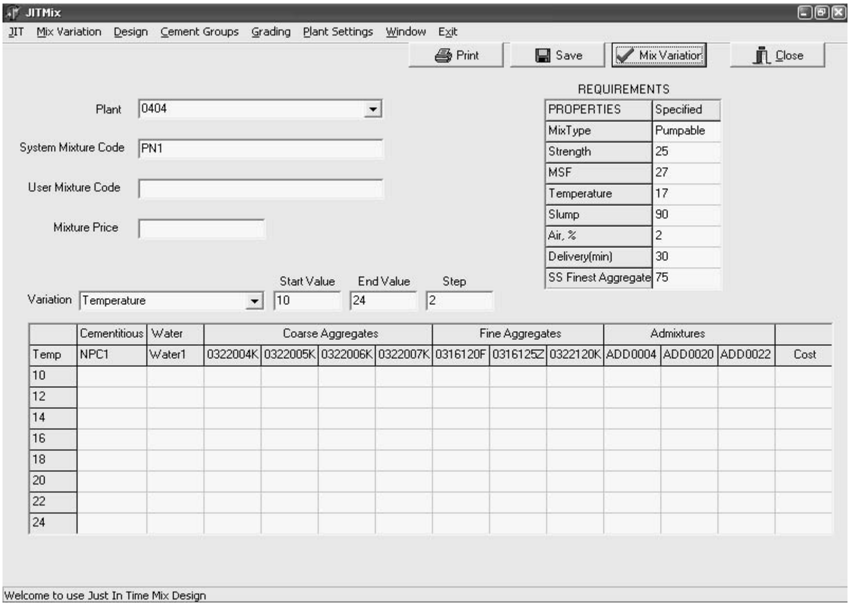


Figure 5.9 JIT mix variation print-out screen.

The example in Fig. 5.9 is set up to show how the mix will vary according to the likely concrete temperature at the time of supply but any other item listed under requirements could be selected. Of course a huge number of tables could be generated with varying input data. The purpose of this feature is to demonstrate that the mix to be supplied is not some chance generation by the computer. If the purchaser can specify the exact conditions of supply, the program can advise the exact mix that will be supplied.

A program having most of the features described is available for free download on the website www.kenday.id.au but readers should not be fooled that this is the real thing. In order to genuinely provide the advantages sought, the program must interact with an extensive and continually updated database of materials and concrete test data. In particular it must also incorporate a mechanism for determining the coefficients M and K in the strength equations of each cement group. While not intrinsically very difficult, this has not yet been done and the author no longer has a team available to work on it.

5.2 Relational mix maintenance (by Mark Mackenzie)

The evolution of the premixed concrete industry has left us with both benefits and challenges, as a result of this Technical Managers and their teams the world over have had their workload significantly increased along with business and industry demands on their services. An important area addressed in this chapter are the requirements to manage and optimise the large ranges of concrete products while accommodating raw material variability and changes, plus business and system processes and constraints, within the time constraints and demands placed on them by the dynamic environment in which they operate. And to do all of this with a high degree of confidence and with minimal risk.

These requirements are made even more difficult by:

- The fact that test results are generally focused on a narrow range of products and that there are generally insufficient or no test results relating to the majority of the concrete product range.
- The importance of adjusting all mixes to accommodate fresh and hardened concrete performance variances, which have been established from only a small proportion of the products. This is necessary to ensure that as an industry we can supply products meeting specified / expected requirements and optimization objectives, even if we have no actual test results for these products.
- Increased levels of automation and integration have enhanced our ability but have also created issues of their own, computer systems are not as tolerant as humans, nor do they have their flexibility. Systems have constraints for example numbers of ingredients within concrete products; maximum batch sizes based on scale capacity; business control processes, etc. To ensure we can both design and produce the required products, Technical Managers need to understand these constraints and comply with them at all times.
- The need is for a consistent philosophy and logic in the design, management and optimisation of mixes. This is important from numerous aspects; optimization; performance; predictability; production processes, etc.
- The need to make raw material changes quickly and with confidence.
- The need to adjust and optimise large numbers, groups, or individual products, at one or more plants.
- The ability to track and comply with important criteria for example ensuring that products specified by the client, which are based on submitted mixes or laboratory trials, are not changed and thereby become no longer compliant.
- The need to track mix design changes/uploads including – Total BOM's (Bills of Materials = full mix details) uploaded; Success & Failures; Creation & Completion Dates; Completed by; modified by; when last modified, and by whom.
- The need to be able to analyse changes/optimization of products for groups of products/plants, for given periods etc. and be capable of reporting and quantifying the variances.

Other tools available for managing this aspect of our business are often rudimentary and extremely manual, and generally only provide a product storage and maintenance system. While the author shares most of Ken Day's visions and views for the future management and optimization of concrete products, I am of the belief that, in order to achieve these long term objectives:

- A series of smaller steps are required.
- A framework or backbone must be created, to which these systems can be applied.
- We will have to develop reliable processes and establish:
  - Key performance relationship for example cement content versus concrete performance.
  - Methods of accurately and consistently quantifying the characteristics of raw materials in particular aggregates. The proliferation of crushed materials, in particular manufactured sands, raises a number of challenges.
  - Methods of linking raw material data, often tested at source, to the raw materials being used in the plant at any point in time.
  - Accurate and logical methods of defining concrete product performance and then using the relationships above logically and consistently in establishing concrete products meeting specified requirements.
- With these developments we are also going to need to overcome the conservatism of clients, often understandable considering the number of unknowns and potential risk involved in some decisions.
- It is critical that we establish industry confidence in the processes and systems, to develop this confidence we are going to need to ensure that we bring enough of the Technical Manager population along with us so that they can be involved in and develop the confidence they need to adopt the progressive logic.
- At the same time we need to demonstrate clear financial and other benefits across the full spectrum of the industry, as without this we have only limited support for investment in acceptance of the outcomes.

The philosophy behind the development of Relational Mix Maintenance was that there were numerous issues faced in the management and optimization of concrete products which could be addressed using current acceptable logic and processes, and could achieve a significant proportion of the benefits while creating a basis on which to build a system capable of achieving the ultimate objective of fully automated concrete management and optimization. Importantly the logic employed:

- Is well defined and can easily be followed.
- Offers a range of exceptional benefits.
- Provides a stepping stone which is tangible to the majority of the industry.

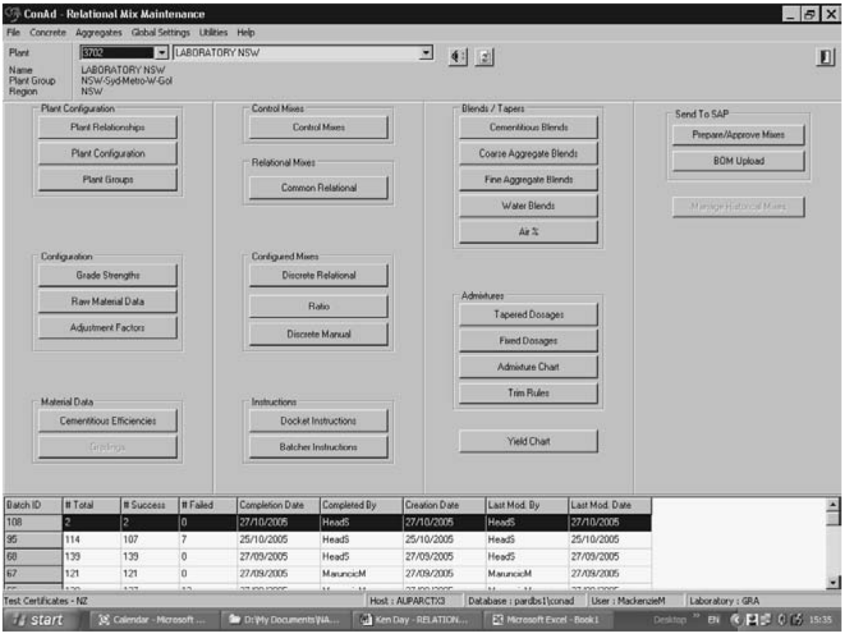


Figure 5.10 Relational mix maintenance – main menu.

The system addresses and makes use of familiar and recognisable key design criteria and factors, these are shown in the main menu (Fig. 5.10) and include:

- *Raw Material Data* – A comprehensive list of all materials allocated to a plant including 'Available Materials' – those currently in use in the plant, 'Unavailable Materials' – those allocated but not currently in use. Data includes all required codes and descriptions, costs, units of purchase and batch; specific gravity etc. (linked to ConAd – Raw Material Data); batching sequences – primary & secondary; scale allocation, details regarding method of batching – automated or manual.
- *Cementitious Efficiencies* – The primary cement (GP/OPC, etc.) is assumed to have an efficiency of 1.0 for all grades and other cementitious materials are allocated efficiencies relative to the efficiency of the primary cement. A cementitious efficiency is calculated for each characteristic strength in each blend i.e. we have found that efficiency varies over the strength range. The relative cementitious efficiencies of each blend are used in the algorithm which adjusts the *W/C* ratio when creating related mix designs.
- *Blends/Tapers (i.e. variation over ranges of mixes)* – This includes cementitious blends; coarse aggregate blends; fine aggregate blends; water blends and air %. With the exception of coarse aggregate blends, all blends/tapers can be varied

by grade strength. All blends/tapers are input as control and related blends. When setting up mix designs the user selects a set of each of these criteria and the factors setup against them are used in the calculation of the mix designs. For a given set of materials the potential practical permutations are limited and hence the blends are used in multiple product ranges. A change to a blend design criteria, results in the change affecting all product groups using the blend.

- *Admixtures* – These can be setup as either or both fixed (units/100 kg cementitious) or tapered (Units/m^3 of concrete by grade strength) dosages. In the Admixture Chart, combinations of admixtures are setup and linked to groups of products. Trim Rules – rules which can be setup linked to groups of products, and consistently applied to adjust admixture dosages for temperature changes.
- *Yield Chart* – The setup of yields. Every product has a target yield, with the exception of Discrete Manual (i.e. rigidly specified) mixes, all products are automatically calculated to achieve target yield.
- *Control Mixes* – These are a range of theoretical base mix designs from which all Common Relational and Discrete Relational mix designs are based.
- *Adjustment Factors* – These are the factors Water Requirements; Coarse Aggregate Volumes, Leanness Factors (Used to adjust *W*/*C* ratios to compensate for variances in workability, etc.) and are used to derive all Common Relational mixes from the Control Mixes.
- *Common Relational Mixes* – Are groups of mixes which can be established as entire ranges of predetermined strength and can be logically be linked to Control Mixes. These groups of products are setup as follows:

  – Determination of adjustment factors for Water and Coarse Aggregate Volume
  – Selection of a:

    · Batching sequence
    · Water Blend
    · Cementitious Blend
    · Admixture Group
    · Fine Aggregate Blend
    · Coarse Aggregate Blend.

  Based on this, mix design is derived from the Control Mixes using algorithms. In addition to the above, the user can select up to 5 manual additions, allocate intelligent and derivable batcher & delivery instructions to the group of products, set maximum and/or minimum *W*/*C* ratios and/or cementitious contents. Individual products or groups of products can be made active or inactive.

- *Discrete Relational Mixes* – Are individual mixes which can, and often need to be, individually highly configured but are still related to a nominated control mix and will be subject to changes within the restrictions nominated for each product, examples of these types of mixes include project mixes, early age mixes, etc. In Discrete Relational and Discrete Manual products can be linked to one or more approved trial/submitted mix designs, if the materials in the plant are changed the system will notify these products are no longer in compliance with the trial mixes.

- *Ratio Mixes* – This accommodates mixes which are designed based on ratios or percentages for example Sand/Cement mixes; No fines Mixes; etc.
- *Discrete Manual Mixes* – Even in the countries were the vast majority of concrete is sold on performance, there are still a few prescribed mix designs. This is the most rudimentary section of Relmix and the only area of the system in which mixes are not recalculated to achieve target yields. That said, the system will inform the user if a change e.g. a raw material with a different specific gravity causes one or more of the products in this area to have a calculated yield outside specified tolerances from the target yield.
- *Plant Configuration* – In every business there are restrictions on what we can do, and information that is useful in achieving objectives. This section addresses both of these areas.

  – Scale/dispenser configuration – in the majority of companies it is Technical Services responsibility to ensure that the maximum batch size possible is set for a mix design. Manually this is extremely difficult, time consuming and generally conservative. So this section requires the scales and their capacity to be entered, and when materials are used in a plant they are allocated to a batching system for example scale.
  – Based on this, every time products are prepared for upload to the business system, the maximum batch size is calculated by lowest denominator and set by product to the nearest 0.1 m^3.
  – Number of materials in a product (BOM) – while a number of newer systems do not have limitations on the number of each type of ingredient which can be contained within a product (BOM) for example 3 cements, 5 aggregates, etc. a significant majority of batching systems currently in use have these restrictions and while these may be upgraded in future these restrictions have been incorporated in the design of numerous business systems and interfaces. The net result is that these restrictions for one reason or another will be around for many years to come. The ramifications of uploading products which exceed these restrictions vary and have significant related risk e.g. product upload not being successful and the product not being available, the ingredient not being batched, etc.

Relational Mix Maintenance requires the maximum number of materials permitted in a single product to be nominated in the following categories – Cementitious, Aggregates, Admixtures & Water. Based on this, every time products are prepared for upload the system checks each product, informs the user of any non conformances and prevents the upload until the issues are rectified.

  – Storage configuration – Persons responsible for designing and managing concrete products, especially in larger companies are generally remote from the operations producing them, that said, they are constantly asked to meet customer expectations which at times may require new raw materials or need to change or include raw materials in a plant. It is important that they understand the plant restrictions in terms of storage capacity. The system assists the user by not allowing more materials than there are storage areas to be setup.

- *Grade Strengths* – The setups in this section are by total area of responsibility for example region
  - This area defines the various grade strengths required, if a grade strength is not setup no products can be designed or setup for this grade strength, it is therefore important that all required grade strengths are setup.
  - The control grade is setup and can be edited.
- *Plant Relationships* – In Relational Mix Maintenance all products are established according to plant. This section links the 3 primary Key Design Factors for the control grade – Water Content; Individual and Total Cementitious Contents and Coarse Aggregate Volume for the each of the plants within a predetermined group. This allows comparison of these factors across plants, changes within a plant are reflected in this area and a change to the plant group control design factors will result in the changes being applied to all of the mixes within the plants in the group.
- *Prepare/Approve Mixes* – A key responsibility of technical services is to manage risk. An area of potentially high risk is the creation/adjustment and upload of mix designs. The nature and magnitude of the risk varies depending on the type of system used:
  - Rudimentary systems which rely on manual adjustment of most ingredients are highly susceptible to finger errors, etc.
  - In a system like Relational Mix Maintenance, a single incorrect change can effect hundreds or even thousands of mixes.

  It is clearly extremely difficult, impractical and highly unlikely that comprehensive manual checks will be carried out or be effective in consistently identifying issues. To ensure that checks occur, errors are prevented and all personnel using the system and relying on the products are extremely confident in their accuracy, Relational Mix Maintenance contains a sophisticated section which checks the proposed mixes against the last mixes uploaded, identifies mixes which fall outside the user defined tolerances (established for each individual constituent), and then allows the user to view the proposed and historic mix design highlighting the non conforming variance. Based on this assessment the user can approve the mixes or make the necessary changes and repeat the check. In addition to this, the process also identifies mixes that have been made unavailable and are no longer supported by trial mixes.
- *BOM Upload* – Once approved mixes are uploaded to the business management or designated system. The system tracks mix design changes/uploads

including – Total BOM's uploaded; Success and Failures; Creation and Completion Dates; Completed by; modified by; when last modified and by whom.

- *Historic Mix Designs* – Every mix design uploaded, together with a comprehensive set of related data, are retained in the Historic Mix Design Data Base, the primary function of this is to make available:
  - The current mixes against which to compare the proposed mix designs in Prepare/Approve.
  - The ability to carry out detailed analysis of products, product groups, at individual or groups of plants over defined periods and filtered by important notes and characteristics.

Figure 5.11 attempts to detail how the system works:

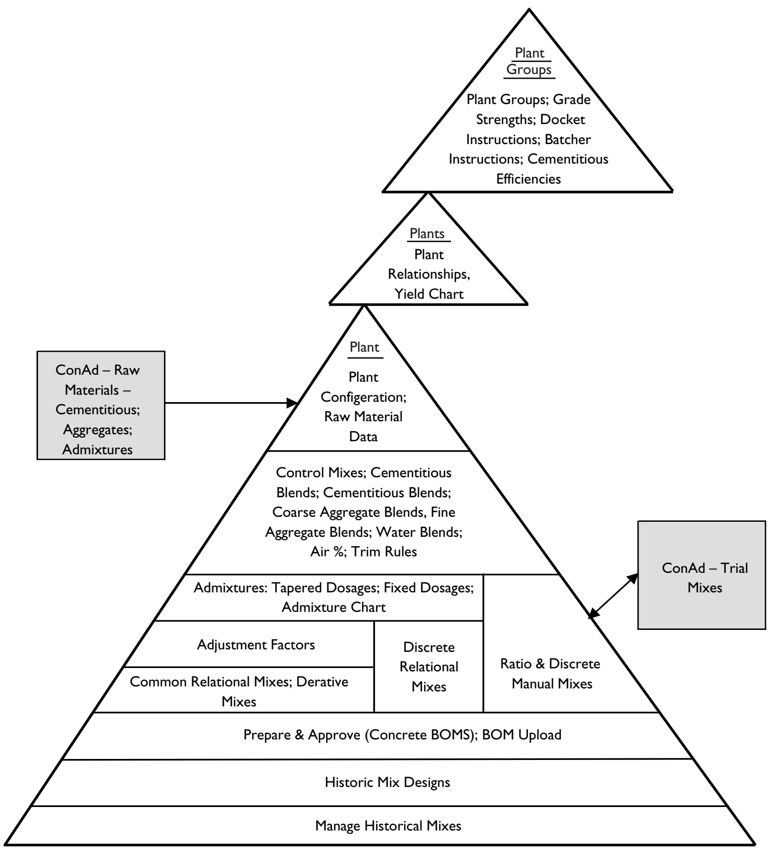


Figure 5.11 Mix maintenance flow diagram.

As stated previously Relational Mix Maintenance (Relmix for short):

- Is fully integrated with the rest of ConAd.
- Can stand alone or fully integrated with the business system.
- Changes within Relmix or related systems will result in changes to all to one or numerous mixes at one or more plants depending on how far down the line the changes are made.

This logic is applicable to all aspects of concrete product design and management within defined groups of materials. We as an industry have tended to be most confident in modifying and/or optimizing the products for which we have the most information, but we need to accept that other products may be needed at any time and we cannot afford to wait for initial delivery, or until enough results are available, before we make any necessary changes. The consequence of this vary in nature and severity for example,

- Concrete performance too high, mixes too fatty etc., cost of over performance, fresh and hardened concrete property issues.
- Concrete performance too low, risk of failure and rejection for all concrete produced within the period taken to address the issue, significant risk.
- Issues with concrete fresh or hardened properties for example difficult to pump, etc. can significantly affect a single order for example blocked pumps, delays, etc. but in most larger areas will cause a significant effect not only on that job but all other jobs due to the knock on effect of the delays.

These are all examples of unwanted, unnecessary and, if we are honest, unacceptable issues.

Relmix provides us with a tool that allows the user to:

- Make decisions on the available information.
- Apply these decisions with consistent logic to all related products.
- To do this in an extremely efficient manner and allow the user to achieve these objectives within a fraction of the time of current systems and well within the time available to a busy Technical Team.
- It gives the user and business a high degree of confidence that changes have been accurately and consistently applied and that in doing so all business requirements have been adhered to.
- Ultimately it has both the capability and potential to address all current issues pertaining to the management of large concrete product ranges.

5.3 High performance (SCC) concrete

Concrete may be regarded as high performance for several different reasons: high strength, high workability or high durability – and perhaps also improved visual appearance.

High strength concrete (HSC) might be regarded as concrete with strength in excess of 60 MPa and such concrete can be produced as relatively normal concrete with a higher cement content and a normal water-reducing admixture. However high performance concrete (HPC) will more usually contain cement replacement materials and a high-range water-reducer (HRWR) or superplasticiser (SP) (different names for the same thing). Already the term HSC does not cover the available range and the terms UHPC or UHSC (Ultra High Performance/Strength) have come into use with actual strengths in current structures in the range of 150–200 MPa (say 20–30,000 psi) being reported (if infrequently) and self-compacting concrete showing rapid growth.

With such concretes the emphasis shifts from the aggregate grading to the 'powder content' and the admixture. For conventional concrete of high slump, or even flowing consistency, the requirement to avoid segregation was to avoid gaps in the aggregate grading. For self-compacting concrete, the requirement is a suitable viscosity in the mortar fraction and a gap in the coarse aggregate grading may even be beneficial.

The assumption is that UHPC will be substantially more expensive per cubic metre than ordinary concrete, although this might not necessarily be true in an area where fly-ash and other fine fillers are very cheap. The justification for its use therefore needs to be sought through structural design using less of it, through a reduced labour cost, an improved durability or an improved appearance. A point often not considered with self-compacting concrete (SCC) is the reduced need for skill in placing. To some extent, if a high strength is required in any case, the extra cost in making the concrete self-compacting may be relatively small, but the extra cost of making low strength concrete self-compacting may be unacceptable. It should not be forgotten that the most expensive concrete is that which has to be replaced through inadequate durability, incomplete compaction or unacceptable appearance.

Previously, high workability meant a high water content and a high cement content and so gave rise to high drying shrinkage. In HPC, the workability is achieved by high admixture use, enabling very low water contents to be achieved at high workability.

Another aspect of high performance concrete is the inclusion of fibres for one of two reasons. One is the use of steel fibres to provide substantial tensile strength and avoid the use, or at least the extent, of secondary reinforcement. The other is the use of organic fibres that will melt readily in a fire. This permits the escape of steam that might otherwise cause explosive spalling in a fire.

Currently only a very small proportion of the world's concrete is UHPC, but the proportion could increase rapidly as the financial effects of past inadequate durability bite deeper, structural designers learn to use the higher strengths more effectively, labour costs increase, and on site skill levels reduce. Another factor is the increasing worldwide focus on CO_2 as a 'greenhouse gas'. Cement production is a major producer of CO_2 and could be taxed or limited by legislation at some future time.

Self-compacting concrete

Flowing concrete, using superplasticising or high range admixtures, has been around for many years now but has more recently been taken to a new level in self-compacting concrete (SCC). Originating in Japan, this material is rapidly becoming more popular in most countries and there are suggestions that, within a relatively few years, more than half of the world's concrete may be SCC. It is obvious then, that even if no one in their area is currently asking for it, it is imperative for all concrete producers to learn how to produce this material. Fortunately it is not being treated as a trade secret and knowledge and assistance are widely and readily available.

Apart from symposium papers by particular individuals, excellent publications have been produced by committees. One such 'Specifications and Guidelines for SCC' can be downloaded from the EFNARC website (www.EFNARC.org). Another, recently released by the Concrete Institute of Australia, is 'Super Workable Concrete'. (It appears that the latter name has been coined to reduce the possibility of legal action should the concrete not quite fully compact in some circumstances.)

SCC is more expensive to produce than 'ordinary' concrete but provides many compensating benefits including:

- Reduced level of skill in use
- Reduced labour content
- Faster placing
- Better surface finish
- Reduced noise level.

The properties that are of particular interest in flowing or self-compacting concrete are the ability to flow, to pass through reinforcement cages, to fill spaces without leaving internal voids and to avoid bleeding and segregation.

Segregation resistance

There is a clear philosophical distinction between flowing and self-compacting concrete, although in practice there is no dividing line between the two and a particular borderline concrete could be regarded as either or both. The difference is the mechanism by which segregation is resisted. In 'old' flowing concrete, and in its pre-cursor of readily pumpable concrete, the mechanism has been a continuity of aggregate grading, taking care to avoid gaps in that grading. In the 'new' self-compacting concrete, the mechanism is the cohesion of the mortar fraction, to the extent that a gap grading may be advantageous. In contrast to normal concrete, the most critical kind of segregation in an SCC is when it is finally at rest in place. There is then a tendency for the coarse aggregate to settle in the mortar fraction if the design of the mix is defective. This is also a point at which any tendency to bleeding will be revealed.

Bleeding resistance

Good SCC will necessarily have no tendency to bleed since it is reliant on the paste viscosity for segregation resistance. This is not necessarily the case with flowing concrete. The absence of bleeding is valuable in the ability to be cast against inward sloping faces and to avoid visual surface defects caused by moisture movement on vertical surfaces. It will also avoid problems sometimes caused by a delay in pumping. On the other hand the absence of bleeding makes SCC very susceptible to evaporation cracking and care must be taken to ensure that horizontal surfaces are not exposed to wind and sunshine in low humidity conditions.

Pumpability

SCC is a highly pumpable material, being completely resistant to bleeding and segregation. It is reported that SCC has been pumped 297 metres (92 floors) to the top of the Eureka building in Melbourne, Australia (Peruzzo, Kolasa and Titus, 2005) it is intended to pump SCC 600 metres to the top of the Burj Dubai. There is some possibility that the mechanism of movement in a pipeline may be different to that of other concrete. It is well established that normal concrete moves as 'plug flow' in a pipeline but it is possible that SCC moves as a fluid with coarse aggregate 'plums' suspended in it, as a result of having little or no yield strength as opposed to viscosity.

Mix design

It appears that a gap grading with little or nothing retained on a 4.75 mm sieve is desirable. Coarse aggregates can be single sized 10 or 20 mm but should not exceed 400 litres solid volume and perhaps as little as 300 litres (depending on sand grading). There is a divergence of opinion on to what extent particle shape is important, with some references considering rounded gravel to be highly desirable but several others happy to use crushed material.

Sand should be continuously graded, preferably with some material retained on a 2.36 mm sieve. A substantial silt content may not be a disadvantage (if suitable silt, i.e. no organic impurity). Well-shaped natural sand is preferred by most sources but crushed basalt fines would produce a heavier mortar, less likely to allow settlement of the coarse aggregate, and may also provide more suitable fine fines.

The most critical aspect of the mix design is the 'powder content', being material passing a 200 micron sieve. Cement content will be determined by the required strength performance. The cement volume must then be supplemented by other materials to reach a total volume of between 150 and 250 litres. At the low end of this range VMA (viscosity modifying) admixtures may be required. The supplementary materials used will depend on cost and availability. After cement and sand fines, the next most desirable material is fly-ash or slag. Air entrainment may occupy 20 to 40 litres. Other materials include limestone, silica fines and metakaolin. Silica fume is a highly effective material for these purposes but may be uneconomical except in the case where very high strength or impermeability is required in the set concrete. Some references suggest that minimum voids in the combined dry powders is an important criterion and may be best achieved by the use of three powders of different fineness (e.g. cement, fly-ash and silica fume).

Chemical admixtures

It is important that fluidity is not obtained by increasing water content. A figure of around 180 litres per cubic metre is regarded as the maximum desirable and an absolute limit of 200 litres should be observed. This of course will not produce high workability alone and must be supplemented with superplasticising admixture. The original superplasticiser used in Japan was sulphonated naphthalene, this is still used but most references now prefer polycarboxylates since a larger dose rate is possible without excessive retardation. However polycarboxylates tend to entrain excessive air and a de-foaming agent must be used to counteract this. The polycarboxylates normally incorporate a de-foaming agent but it should be noted that in many products this tends to settle out unless the admixture is continuously agitated. This is a field in which future developments are very likely, reducing the difficulty and cost of producing SCC and increasing its rate of acceptance.

On the author's MSF scale, it is suggested that a value between 35 and 40 is appropriate, although the author has little personal experience of this material. The suggested design process is to first consider the required strength and the materials available to attain it. This will enable a consideration of the necessary water/cementitious ratio. Where a high strength is not required, an initial assumption of around 180 litres of water may be suitable and for very high strength (say over 100 MPa, with 200 MPa being a likely maximum attainable) a figure as low as 130 litres. Strength is going to be more dependent on the attained density of the cement paste than just the *W*/*C* ratio, although the latter will of course be a major factor in the former. If there is no previous experience to guide, trial mixes will be essential from the strength viewpoint. It may be that initial trial mixes should be of mortar rather than concrete, since the coarse aggregate plays a more passive role than in normal concrete.

For maximum strength, a total cementitious content in excess of 600 kg, (just possibly 650 to 700), is unlikely to be worthwhile. With a cement only mix and water content 130, this may give around 100 MPa. For higher strengths a proportion of silica fume will be required and a proportion of fly-ash is also desirable. The *water/cementitious ratio* should probably not be less than 0.20 and 0.23 may be more realistic. A possible mix for 150 MPa might contain 350 cement, 150 fly-ash and 100 silica fume with 125 litres of water. It is emphasised that the strength of such a mix is highly dependent on the qualities of the cement, fly-ash and silica fume, and also on the aggregate fines (which, for high strengths, should probably not contain more than a maximum of 40–50 kg of material finer than 200#). Having determined an initial paste composition, the volume of total aggregates is obtained by subtraction and the sand percentage by subtracting the paste contribution from the MSF. A choice of MSF in the range of 35–40 should take into account that coarse aggregate content should be in the range of 28–35 litres (so 750–900 kg) and that a smaller coarse aggregate content (and a smaller maximum size) will be necessary for a more congested reinforcement situation. As noted, it may be satisfactory to conduct the initial trial mixes without the coarse aggregate fraction. Obviously there will be nowhere near sufficient water to attain self-compaction and a high range water reducer, probably a polycarboxylate, will be required. The volume of this will need to be increased when going from a mortar only trial to the full concrete mix but no other change should be made.

Where the strength requirement is low, it will still be necessary to have at least 160 litres, say 400 kg, of microfine material including cement, fly-ash and aggregate fines (with superfine limestone, or possibly magnesia [see Section 5.3], probably being desirable if economically available). The cost of silica fume would probably not be justified unless satisfactory fresh properties cannot be obtained without it. Even at this level of fines, a VMA will probably be needed to obtain satisfactory self-compacting properties. A water content of 180–200 litres might be appropriate, but not exceeding a *water/microfines* ratio of 1.1.

With both high and low strength requirements, satisfactory fresh properties have to be an over-riding concern. The slump flow test and other alternatives are described in 11.7. The occurrence of a paste halo in the flow test would be an indication of excess water or inadequate microfines. If the actual spread is higher than necessary, water content can be reduced, otherwise microfines content must be introduced or a VMA used.

5.4 TecEco concretes (by John Harrison)

The most common hydraulic cement is Portland cement (OPC), which hydrates to form mainly calcium silicate hydrates (CSH), Portlandite and minor components. Harrison theorizes that Portlandite and water are responsible for most of the problems of pre-mix concrete, In his view Portlandite is too soluble, mobile and reactive. It carbonates, reacts with Cl^- (chlorine) and SO_4^- (sulfate) and being partially soluble can act as an electrolyte. He has proposed removing Portlandite with the pozzolanic reaction and replacing it with a more stable alkali in the form of Brucite ($Mg(OH)_2$). It generally requires much more water to make concretes workable than can be chemically used in the hydration reaction. To form Brucite John adds reactive MgO in various proportions which, as it hydrates internally, consumes significant excess water in such a way that it is still available for the more complete hydration of PC over time. As a result of these changes in the chemistry of OPC he contends that concretes will have better rheology, shrink less, are stronger and much more durable. Although only relatively limited testing has been achieved to date, so far it appears as though he may be correct.

In the short term the pH of John's formulations is higher due to internal water removal and as a result more effective reactions with pozzolans occur. As the pH falls due to consumption of Portlandite an equilibria established between CSH, Brucite and water maintains the pH at a lower level than Portlandite would but sufficiently high to prevent the corrosion of steel. The internal pH over the long term is critical for durability and Brucite and Brucite hydrates are much more stable alkalis than Portlandite, providing lower pH immobilization of heavy metals that occur in waste streams.

A problem with high strength concretes is autogenous shrinkage which is caused by stoichiometric volume change during hydration reactions. Harrison claims to have solved this problem through the dehydration of brucite hydrates causing the more complete hydration of OPC although he admits it is still early days with testing. The built-in curing we try to provide by incorporating saturated lightweight aggregate and other un-natural techniques may no longer be necessary.

Magnesite is a naturally occurring magnesium carbonate ore and is calcined to produce MgO (magnesia or magnesium oxide) in the similar way limestone and clay are calcined to make OPC, but at a much lower temperature and therefore more efficiently. Magnesia also has to be ground, but is softer and easier to grind than OPC clinker. Because the process is so simple and efficient John hopes it to be the first in the world driven by non fossil fuel energy and that with volume the price of reactive MgO will fall below that of OPC. The magnesia used should be as reactive as is commercially feasible to prevent any risk of delayed hydration. Harrison has demonstrated that the hydration reactions of magnesia are not only independent of other reactions in Portland cement but that they occur sufficiently rapidly not to cause dimensional distress and that they have a wide and important role blended with them (contrary to the inclusion of crystalline magnesia {Periclase} in OPC which is regarded as an unsoundness risk in some specifications).

Harrison calls his formulations of magnesia with Portland cement Tec-cements, Eco-cements or Enviro-cements according to the degree of replacement of PC by magnesia and the type of concrete produced. Readers should consult Harrison's website www.tececo.com for voluminous details of his work as only a brief summary is possible here.

One ramification of the technology that has received considerable publicity around the world is that the Brucite in Eco-cements carbonates in *porous* materials resulting in the sequestering of CO_2. Combined with seawater extraction of Mg and a kiln technology he has also invented, because of the high volume of material used in the built environment, a partial solution to global warming is provided.

Portland cement concretes are already a relatively sustainable material. With low cost and high thermal capacity they supply essential thermal mass to buildings. With the advent of Harrison's technology, concretes will become even more sustainable with lower binder to strength ratios, greater durability, waste utilization and sequestration in the case of Eco-cements.

Two main formulation strategies have so far been defined:

Tec-cements (5–15% MgO substitution)

Tec-cements contain more Portland cement than reactive magnesia. As noted, reactive magnesia hydrates in the same rate order as Portland cement forming Brucite and Brucite hydrates which use up water reducing the voids:paste[1] ratio, increasing density and possibly raising the short term pH resulting in more effective reactions with pozzolans. Suitable pozzolans include fly-ash and ground granulated iron blast furnace slag as well as a large range of other material such as quarry wastes.

Eco-cement and Enviro-cement concretes (15–95% MgO substitution)

Higher proportions of magnesia are used in Eco-cements and Enviro-cements and neither is as strong as Tec-cements. The difference between Eco-cement and Enviro-cement concretes is that Eco-cement concretes carbonate in porous concretes such as masonry blocks, whereas Enviro-cement concretes are non-porous and do not contain other than surface carbonates.

Brucite in porous materials carbonates, forming strong fibrous mineral carbonates and therefore presents an opportunity for sequestration. Enviro-cements contain similar percentages of MgO to Eco-cements, but in non porous concretes, Brucite does not carbonate readily. Higher proportions of magnesia may be most suited to toxic and hazardous waste immobilization and when durability is required.

Tec-cement concretes

Tec-cements are suitable for a wide range of uses including any purpose for which Portland cement is currently used.

Claimed benefits include improvements in durability, density, strength, cohesion and workability, reduced bleeding, permeability and shrinkage, and the use of a wider range of aggregates, many of which are potentially wastes, without reaction problems. Greater strength, less shrinkage and cracking and greater durability, given adequate engineering back up, should result in widespread use.

1 We think of strength varying with W/C ratio but really only a small proportion of most added water ends up chemically combined with cement, the rest remains in pores and eventually evaporates, so it is the ratio of hardened cement paste to voids that determines strength. In initially using up free water, Brucite reduces the voids in this equation. Then, by giving up some of this water at a later stage, it enables more complete hydration of the OPC.

There are obvious advantages of including more stable alkalis or carbonates in cements so perhaps it is time to bury the dogma regarding magnesia and rewrite all cement standards so that they only contain a performance based test such as in ASTM C150 and ASTM C595M where autoclaving is required. No special comment should be necessary regarding reactive magnesia which would then be classed as a supplementary cementitious material. The water consumption stoichiometry of Tec Cement is variable but involves the formation of still to be characterised Brucite hydrates:

$$MgO\,(s) + H_2O\,(l) \leftrightarrow Mg(OH)_2 \cdot nH_2O\,(s)$$

Tec-cement formulations have a characteristic 3–4 day strength peak and this comparatively high and fast strength development is probably due the interaction of a number of factors. Most likely are:

- More and stronger silicification reactions including a more effective pozzolanic reaction during the early plastic stage whilst the pH is possibly elevated.
- A lower voids: paste ratio as a result of improved rheology due to better particle packing, some surface charge effects and high consumption of water by reactive magnesia as it hydrates.
- Slow release of water by hydrated Brucite gels ($Mg(OH)_2 \cdot nH_2O \rightarrow Mg(OH)_2 + H_2O$) resulting in more complete hydration reactions of PC.
- The possible formation of another compound such as magnesium aluminium hydrates analogous to the hydrogarnets sometimes formed in Portland cement concretes with insufficient gypsum.

Tensile strength is also improved up to about day 20 and this is probably the result of both more rapid early strength development and a change in the surface charge of the magnesia added from positive to negative at pH 12.

Noticeable from the moment water is added is the improved rheology. This is due to the lubricating affect of the smaller magnesia particles and their packing with other components as well as the introduction of a shear thinning effect due to the influence of the negative magnesium ion in solution on water which is a polar molecule with the result that weak hydration shells are formed.

As a consequence of the removal of Portlandite using the pozzolanic reaction and replacement by Brucite, Tec-cement concretes have a different pH curve to Portland cement concretes with or without added pozzolan. As the hydration of magnesia takes up a lot of water (Brucite is 44.65 mass % water; Brucite hydrate gels contain even more water) and because Tec-cement concretes do not bleed as much whereby alkalis remain in concrete, it is thought that during the early plastic stage the pH may be higher. In the longer term however the pH is controlled by Brucite which has an equilibrium pH of 10.52 and CSH which has an equilibrium pH of 11.2 and remains somewhere between.

The equilibrium pH is still however at a sufficiently high level for steel to remain passive[2] and for the stability of calcium silicate hydrates.[3] It is thought that dense concretes made using Tec-cement formulations should maintain reducing and ion free conditions at a pH over around 8.9 required for the long term survival of steel.

The removal of excess water by magnesia as it hydrates has a number of other consequences. Bleeding and the introduction of associated problems such as efflorescence, freezing of bleed water and weaknesses such as interconnected pore structures and high permeability do not appear to occur as much.

Tec-cement concretes tend to dry out from the inside due to the water demand of magnesia as it hydrates. As free water is required for delayed reactions they do not occur.

Brucite does not react with salts because it is a least 5 orders of magnitude less soluble, mobile or reactive than Portlandite. Sulfates, chlorides and other aggressive salts also have no effect. The Ksp (reactivity) of Brucite $= 1.8 \times 10^{-11}$ is much less that that of Portlandite is at 5.5×10^{-6}.

The advantages of using quick setting and convenient Portland cement such as ambient temperature setting, easy placement and strength are not diminished, however shrinkage is reduced, if not eliminated, due to low water loss and compensating stoichiometric expansion of magnesium minerals. In appropriate proportions the expansion of magnesium minerals balances the slight shrinkage of Portland cement concrete eliminating cracks and reducing porosity. Blended in the right proportions, concretes can be made that are dimensionally neutral over time.

Autogenous shrinkage does not occur in high strength Tec-cement concretes because equilbria are established between Brucite and its hydrates and CSH, Portland and water whereby the former can desiccate back to Brucite delivering water in situ for more complete hydration of Portland cement.

$$Mg(OH)_2 \cdot nH_2O\,(s) \leftrightarrow MgO\,(s) + H_2O\,(l)$$

$$CS + H_2O \leftrightarrow CSH + CH$$

As Brucite is a relatively weak mineral it can be compressed thereby also densifying the microstructure of concrete. Brucite is also well known as a fire retardant.

Eco-cements

Eco-cements have higher proportions of MgO than Tec-cements and require porous substrates to carbonate such as in bricks, blocks, pavers, mortars and

2 As Fe_3O_4 rather than oxides such as Fe_2O_3 or FeO_2 which tend to hydrate and are dimensionally unstable.

3 The neutralization of Lime by pozzolans results in a drop in the Ca/Si ratio in CSH and potential brittleness.

renders. In such substrates, as there are no kinetic barriers, the magnesia not only hydrates, but carbonates completing the thermodynamic cycle by reabsorbing the carbon dioxide produced during calcining.

Eco-cement concretes can include a large proportion of recycled industrial materials such as fly and bottom ash and are therefore likely to become a building material of choice in the future. Important uses will include providing a sustainable, low cost building material with high thermal capacity, low embodied energy and good insulating properties for construction in products such as bricks, blocks, stabilized earth blocks (mud bricks), pavers and mortars, porous pavements and in combination with wood waste and other waste for packaging and building components.

The large scale use of Eco-cements for such products would result in sequestration of very significant quantities of CO_2 if in conjunction with the kiln also invented by John Harrison (The Tec-Kiln).

When Brucite carbonates it forms an amorphous phase, lansfordite, and nesquehonite all of which are hydrated carbonates. Strength gain in Eco-cements is mainly micro structural because of more ideal particle packing (Brucite particles at 4–5 micron are under half the size of cement grains) and the natural fibrous and acicular shape of magnesium carbonate minerals which tend to lock together.

Magnesium is a small lightweight atom and the carbonates that form contain proportionally a lot of CO_2 and water. Total volumetric expansion from magnesium oxide to lansfordite, for example, is 811%, meaning that a little binder goes a long way.

$$Mg(OH)_2 + CO_2 \rightarrow MgCO_3 \cdot 5H_2O$$

Magnesium carbonates and hydrated magnesium carbonates are also fire retardants, releasing CO_2 or water vapour or both at relatively low temperatures.

Enviro-cements

Enviro-cements are essentially Eco-cements in that they have higher ratios of magnesia to hydraulic cement. The difference is only that they are used in non porous materials so little or no carbonation occurs. Chemically and physically they are potentially more suited to toxic and hazardous waste immobilization because they are more durable than either lime, Portland cement or Portland cement lime mixes. Enviro-cements do not bleed water, are not attacked by salts in ground or sea water and dimensionally more stable with less cracking. Ina Portland cement-Brucite matrix[4] OPC takes up lead, some zinc and germanium.

4 Portland cement minerals and Brucite are the main binder minerals. A host of minor species also form and are also present.

The Brucite in enviro cements is an excellent host for toxic and hazardous wastes as it has a layered structure and traps neutral compounds between the layers.

Heavy metals not taken up in the structure of Portland cement minerals or trapped within the Brucite layers end up as hydroxides. The pH, which is controlled in the long term by Brucite and CSH is between 10.4 and 11.2, an ideal long term value at which most heavy metal hydroxides are relatively insoluble.

Waste and on site excavation waste utilization by TecEco-cement concretes

As the price of fuel rises, the use of on-site natural/low impact low embodied energy materials, rather than carted aggregates, will have to be considered. The new hydraulic calcium-magnesium binders invented by Harrison provide benign environments allowing the use of many local materials and wastes without problems associated with delayed reactions.

Harrison maintains that using materials regardless of their chemical composition for the physical properties they impart to composites is fundamental to sustainability and Brucite and magnesium carbonates bond well to many different materials including wood and will hold a large proportion of waste. Many wastes such as fly-ash, sawdust, shredded plastics etc. can improve a property or properties of the cementitious composite. If wastes of any kind are to be incorporated in a cementitious matrix, such as Portland cement, it is essential that the long term pH is regulated in the region of the minium solubility of heavy metals, as is the case in TecEco cement concretes. In a Portland cement and Brucite matrix the calcium silicate hydrates take up lead, some zinc and germanium. Heavy metals not taken up in the structure of Portland cement minerals or trapped within the Brucite layers end up as hydroxides with minimal solubility.

5.5 Advances in inorganic polymer concrete technology

The author is indebted to Dr Grant Lukey and the team at the University of Melbourne, including Prof Priyan Mendis, Prof Jannie van Deventer and post graduate student Massoud Sofi, who provided appendix A on Inorganic Polymer Concrete (IPC). As will be apparent, Grant (formerly General Manager of Siloxo, a company established to exploit IPC – see www.siloxo.com) is a leading authority on the subject. The subject is of such importance, and their report so well presented, that it has been incorporated in an appendix (A) as submitted with its own separate index and numbering system.

It is apparent that a massive research into IPC is underway and is producing very promising results. The chemical reactions involved have been presented in detail since they will be novel to most readers. The material is extremely attractive since it not only uses a waste material but, in replacing Portland cement, reduces carbon dioxide emissions currently causing substantial concern world-wide.

It is clearly not to be viewed as an inferior substitute for OPC but as a material having some properties far in advance of that material. Examples are fire and chemical attack resistance and the expectation that it could provide long-term encapsulation of nuclear and other dangerous wastes.

While some caution may be justified in immediately launching into wide scale use of the material, it is to be hoped that it will not be subject to the unreasonable delays and prejudices so often experienced in the concrete field. Fifty years ago the author was involved in the laying of a short pipeline composed of several different experimental pipes in the most aggressive part of the Melbourne sewerage system. It is clear that such a trial using IPC is urgently justified (the trial referred to resulted in a decision to use plastic lining!). Considering the current extent of expenditure on anti-terrorism measures in general, it is surely obvious that no expense should be spared in the urgent large scale investigation of the use of the material for structures.

The very rapid strength development available, while a problem to be overcome in in situ structures, could make the material especially valuable for precast products.

In the (book) author's opinion, IPC (more commonly but less correctly known as GPC i.e. geopolymer concrete) will become an important material in the near future and he is more than pleased to be able to incorporate this report. It provides a brief insight into various aspects of IPCs, including their basic chemistry, synthesis, properties and application. The main differences in chemistry of Ordinary Portland Cement (OPC) based concrete and IPCs are discussed, with particular attention to the advantages and shortcomings of IPCs compared to ordinary concrete. The current technical, environmental, and commercial drivers for uptake of the technology are also discussed, as well as the challenges and obstacles faced during the successful commercialization of this promising technology. The report concludes with some of the typical and most recent applications of IPC materials. It is anticipated that this report will give the reader a general understanding of the current research and development work on IPCs and provide an introduction to a new and potentially very robust and versatile material in the field of civil engineering.

Chapter 6

Specification of concrete quality

By the time this edition of the book is published, it will surely be legitimate to assume that the ridiculous American practice of mindlessly specifying minimum cement contents and requiring mixes to be submitted and not subsequently varied will have finally died out. If not, reference can be made to the author's articles 'Perspective on Prescriptions' in Concrete International, July 2005 or 'An Australian perspective on P2P initiative: Lessons to learn' in The Indian Concrete Journal, early 2006 (which appear on the website) or to the excellent article 'The P2P Initiative' on the NRMCA website (not by this author).

However there are persons who have legitimate views that special concrete is required for their project and who have investigated their special needs quite thoroughly. So it is still relevant to consider such circumstances and how best to cope with them.

6.1 The philosophy behind specifying concrete

It is worthwhile to consider what effect we want our specifications to have before we consider what to write in them. It is even more worthwhile to consider the intended and unintended effects our specifications might have if written in various ways.

If we know exactly what concrete we want, and exactly how to make it (a very rare situation as pointed out in Chapter 1) the tendency is to write a specification requiring it to be produced in exactly that way. This is called a prescription specification. Although persons writing such specifications are usually reluctant to accept full responsibility for the performance of the concrete in such cases, that is where the responsibility should lie.

It is not really sufficient for the materials and mix proportions to be clearly set out. In such cases there is no incentive whatever for the concrete producer to know or care anything about concrete, to employ competent staff, to purchase good materials, to have good quality production facilities etc. It is therefore necessary for all such matters to be specified in detail and to employ supervision to ensure that all such requirements are complied with. Clearly this would be a substantial additional cost.

However, even so, there is an inevitable variability involved in the production of concrete and it will be a brave person indeed who specifies that the variability of concrete strength shall not exceed say 3 MPa (or 450 psi) (although such a requirement may not be so unreasonable in a non-prescription specification). So the prescribed concrete will have to incorporate a very substantial safety factor, again increasing cost. It would be possible to require all staff of the concrete producer to have appropriate qualifications, but it is not really possible for the specifier (as opposed to the employer) to require them to exert maximum effort to achieve low variability. In any case how could they do this if they are not allowed to change anything?

We see the effects of decades of this type of specification in USA today with much of the industry lagging behind much of the rest of the world in mix design and especially QC. There are certainly exceptions where particular producers on prestige projects are driven by the desire for a good reputation, regardless of the lack of direct financial benefit.

So what is the alternative? How can we ensure that suitable concrete is supplied to our project and how can the industry be encouraged to lift its standards?

It is clear that a basic requirement for a specification is that it should increase the likelihood that the contract will go to a producer who is highly motivated to provide the concrete of the required properties and who has equipped and staffed his organization with this end in view. For such a producer to remain in business, he must be economically competitive with other producers who may have cut costs to the bone. The only way the additional costs can be offset is by using a lower cement content. This may be achieved partly through greater skill in mix design, and partly through achieving reduced variability and thereby justifying a lower mean strength.

It is a legitimate question to ask whether it is a good swap to exchange a lower mean strength for a reduced variability. The answer is that the lower variability is to be preferred for the following reasons:

1. It is much more likely that a really bad individual truck of concrete will be produced in error, and escape detection, under a sub-standard control system.
2. For 5 or 10% defective at a given strength level, the level of 1% defective will be lower the higher the standard deviation that is the more variable concrete will have a greater spread below the specification limit.
3. It is easier to detect a downturn in a single grade of concrete of lower variability. If a concrete has double the variability it will take at least twice as many tests (so twice as much defective concrete) to detect a given downturn.
4. A good control system will include analysis techniques that will combine the results from many grades of concrete, effectively multiplying the frequency of testing.
5. More uniform concrete is likely to be better placed and of better appearance and more uniform durability.
6. A good control system will normally generate comprehensive periodical reports, substantially easing the task of supervision.

The question of the action to be taken in the event of sub-standard concrete being encountered is also worth careful consideration. A first point is that, in the author's experience, there is a substantial possibility that an individual test result may be the result of bad testing rather than bad concrete (Day, 1989). If there is any possibility that concrete sufficiently sub-standard to genuinely necessitate its removal, or strengthening of the structure, then a major investigation is needed, and is beyond the scope of this book. It is certainly not sufficient to confine such an investigation to finding and examining the particular concrete giving the highly sub-standard result. Rather, or in addition, the investigation should concentrate on parts of the structure where untested concrete would pose an especial risk if as sub-standard as the tested concrete.

Fortunately it is a relatively rare occurrence to encounter such 'structurally defective' concrete (at least, where an effective control system is in use). Much more usual is the occurrence of 'contractually defective' concrete, this is concrete that is quite capable of serving its intended purpose but is below the specified standard. Such concrete is sometimes removed 'in order to teach the supplier a lesson'. More often there are a number of meetings of all concerned, and perhaps an investigation involving coring or ultrasonic testing, and the concrete is then accepted at full price. It is wasteful to discard concrete that is usable, and there are often disadvantages in its removal. Such disadvantages may include unsatisfactory or unsightly joints and/or program delays. What is important is to ensure that a concrete producer never makes, or even thinks he can make, a profit from the deliberate supply of marginally defective concrete. This is partly because if he does, then he may be encouraged to go further next time. However the main reason is in fairness to competitors who failed to win the contract because their price allowed for acceptable quality.

Such a result can be achieved by requiring the producer to increase his mean strength by a specified amount for a specified period of time following the discovery of the marginally defective concrete. If the concrete is fair-faced, this is likely to cause a change of shade, which may be objectionable. Either the increase itself, or the period for which it is to apply, would need to be substantial for compliance to be clearly established. Such action would upset evaluation of the producer's long-term performance by artificially inflating the standard deviation.

A much better solution is to impose a cash penalty in such a situation. The characteristic strength (mean minus $k \times$ SD) can be accurately established by a run of 30 tests over a period, the cement increase to raise the mean strength to the acceptable level can also be determined with reasonable accuracy. A cash penalty of twice this amount would be sufficient to ensure that no producer ever made a profit by supplying marginally inferior concrete (Day, 1982b, 'Cash penalties can be fair and effective' also in Section 12.1). To save calculation and disputation, the penalty could be set at 1% of the ex-truck price of the concrete per 1% of strength deficiency.

For those who are philosophically opposed to cash penalties, a cash bonus system over a limited range of excess strength can be substituted to give the same effect. Why should a purchaser be prepared to pay extra for a strength in excess of his specified requirement? In another paper (Day, 1982a 'What is economical concrete?' – also in Section 12.2) the author argues that it is a foolish economy in many cases to specify a strength that is truly the minimum acceptable. So, for example, the alternatives might be to specify 20 MPa (2,900 psi) with a bonus clause or 25 MPa (3,600 psi) with a cash penalty. Whereas in the case of the penalty the figure might be aimed at twice the cost of remedying the deficiency, in the case of the bonus it might be reasonable to make the figure only half of the cost of achieving the excess. Another alternative would be to specify 22 or 23 MPa with both the penalty and bonus clauses. Of course it would be necessary to put limits on both the penalty and bonus provisions with actual rejection for a deficiency of more than 5 MPa and no additional bonus for an excess strength of more than 5 MPa.

As is clear from the dates of the references, the author has been proposing cash penalties as the best solution to marginal strength deficiencies for more than 20 years with very limited success. However it remains his strongly held opinion that the proposal will eventually be widely adopted. The 20-year period is short compared to the 30 to 50-year period over which he has been advocating such measures as strength specification, and multigrade, multivariable, cusum analysis, which are only recently coming into wider use around the world.

So the specification needs to include a strength requirement and should encourage a high standard of QC. A run of 30 results has been suggested as necessary to provide an accurate judgement of the strength/quality being supplied. However it would be quite unsatisfactory to allow defective concrete to be supplied for the period necessary to accumulate 30, 28 day results (although this is envisaged by some specifications). Control action needs to be based on results at not later than 7 days and it needs to be based on very few such results. To do this requires several measures:

1 An accurate means of predicting later age results from early age results.
2 A means of combining results from many or all grades of concrete to greatly increase the number of results available in a short period of time (i.e. multigrade analysis).
3 A system of analysing related variables to assist in determining whether a downturn is genuine or only a statistical aberration and in determining its cause (multivariable analysis).
4 A technique of extracting maximum certainty from an analysis (cusum graphical analysis is approximately three times as effective as Shewhart graphing).

Even with all these requirements satisfied it would still be difficult to legally require a producer to adjust his mix based on early age result in marginal cases. This is another advantage of the cash penalty option. If a producer suspects that current early age results indicate that a cash penalty will be imposed when sufficient later age results are available for analysis, he will be just as keen as the purchaser to take early corrective action.

This illustrates the point that there are two quite separate requirements for a satisfactory specification. One is to provide an accurate assessment of the quality of the concrete and the other to initiate prompt action in the case of a quality downturn. It is disastrous to attempt to combine these two requirements into a single requirement. The accurate assessment requires 30, 28 day results and has no requirement to make a rapid judgement or to identify which concrete is defective (assuming it is only marginally defective). The urgency requirement has no requirement to avoid occasional inaccuracy. If the following day's results indicate that the initial assessment was inaccurate, a further adjustment can be made and little is lost.

It is now possible to go further than this in early problem detection. At least one control system includes a 'batch watcher' facility that automatically emails or text messages a selected list of persons in the event of batching errors exceeding pre-set limits being exceeded (ConAd, see Section 4.15). There are also truck-based systems for controlling workability and water content (Compumix, see Section 11.7).

It may be necessary to specify many other requirements in particular cases. For example:

1 A particular type of cement or pozzolanic material on grounds of durability, heat generation or suppression of alkali-aggregate reaction.
2 A test for reactive aggregates where aggregates without a proven record are permitted.
3 An air content, for freeze-thaw resistance.
4 An early strength required for stripping, pre-stressing, de-propping etc.
5 A shrinkage limit.
6 A permeability test limit (for durability in aggressive groundwaters rather than watertightness, also the ISAT in situ test checks on curing in addition to basic impermeability).
7 A bleeding limit especially where good off-form finish is required.
8 Segregation – a really good test remains to be devised, but it could be specified that the concrete shall not display any tendency to segregation at the proposed workability.
9 A maximum Los-Angeles abrasion value for the coarse aggregate where extreme abrasion resistance is required (but note that the surface finishing technique may be substantially more important and a Chaplin abrasion test on the finished concrete may be more relevant).
10 There is a tendency to want to specify a w/c ratio, since this is the best overall criterion of concrete quality. There are two reasons not to do this. One is that strength is much easier to use as a control. The other is that if there is some factor causing a departure from the anticipated w/c v strength relationship (such as bond to coarse aggregate), then strength is the better guide.
11 Finally, the higher of the two strengths required for structural performance and for durability should obviously be specified.

An important question is whether mixes should be submitted for approval, and if so, approval by whom. It seems reasonable that a purchaser should be entitled to know what is in the concrete he is purchasing. The purpose of such a submission should be to ensure that the mix has no objectionable features. These might include admixtures containing calcium chloride, air-entraining agents known to give an excessive bubble size, potentially reactive aggregates and aggregates known to have high moisture movement or to cause popouts in exposed surfaces. The list is not extensive and a list of materials rather than mix proportions might meet the need, however, to be effective, assessment needs to be by a qualified and experienced concrete technologist.

Militating against detailed mix submissions is the desirability of using standard, well-proven mixes from the viewpoint of quality control and a proper degree of confidentiality from competitors. Also the producer needs to be entitled to vary his mixes from day to day to maintain control.

6.2 Development of standard mixes

Specifications have tended to assume that the concrete supplier will design a special mix to comply with the specification. This may be necessary in relatively rare cases, but it does have some disadvantages:

1 No history of previous satisfactory performance on actual projects.
2 No common pool of test results with same mix on other projects.
3 Truck drivers less familiar with mix – less able to judge workability and detect abnormality.
4 Variability may be increased if every now and then the standard mix is supplied in error.

It might be reasonable to provide a financial advantage to suppliers who have satisfactory standard mixes in use, under routine control and with a range of properties established. The form of encouragement could be to allow a reduced testing frequency for such mixes and to require pre testing, and a higher testing frequency for the first months, of new mixes.

The above points apply even for major projects, but their importance is far greater for the many ‘ordinary’ projects that probably account for most of concrete produced. Small projects cannot economically generate sufficient test data to maintain good control. This means that they are essentially dependent upon the producer’s quality assurance system. In such circumstances it is counter productive to specify non-standard mixes unless absolutely essential. It is possible that a very small project could nevertheless derive great advantage from the use of 100+ MPa concrete in a particular column, or involve a single wall of exposed aggregate concrete of super critical appearance. In such circumstances special mixes are obviously involved and control costs are of little importance. However a refusal to accept a standard mix for a 25 MPa internal floor slab would be justified only if the standard mix were distinctly unsatisfactory.

The specifier should generally concentrate on obtaining full information, both past and current, about standard mixes. The aim should be to check that the supplier's control system is working well rather than to supplant it. These remarks are relevant when only compressive strength is regarded as important. The following section deals with requirements other than strength and the importance of using standard mixes of established performance is much greater in respect of such requirements.

A time is coming when it may be less essential to use standard mixes. The control system being pioneered by the author enables results from many grades to be combined onto a single control graph. The performance of mixes may be seen in terms of factors in mix design equations rather than a stand-alone assessment. The same situation has been encountered in many different industries (Toffler, 1981). Initially, mass production requires acceptance of a reduced range of products. However as the technology of both production and quality control advance, the standardization necessary tends to be that of small parts of the whole. In this way products of very wide variety can be produced from components which are rigidly standardized. It is emphasized that this stage has not yet been reached in concrete technology and specifiers should currently concentrate on the second phase of reduced variety. However the author presented a paper 'Just-in-Time Mix Design' at ACI Cancun in 2002 (Day, 2002) that demonstrated the necessary technique for this development. This was further referred to in his paper 'Concrete in the 22nd Century' to the CIA Biennial in Melbourne, October 2005 (Day, 2005b).

6.3 Batch plant equipment

The availability of computer operated batching equipment, able to positively record the actual as-batched quantities for each batch of concrete, is an important factor in the control process. It provides the following advantages:

1. It gives a considerable degree of assurance that the batches sampled are in fact typical of the whole output. This greatly strengthens the argument in favour of a reduced rate of testing, allowing emphasis on quality of testing and a thorough analysis of the results rather than sheer volume of testing.
2. It provides a ready means of adjusting mixes and of keeping accurate records of what adjustments were made and when.

It is therefore fully justified to specify that such equipment should be used on any important work and that the resulting data bank should be made available to the supervising team. Should such equipment not be made mandatory, it would be reasonable to halve the otherwise envisaged sampling rate if it were provided.

6.4 Proposal – approval specifications

Without increasing cost excessively, it is virtually impossible to so specify a concrete mix that it will necessarily be satisfactory. Strength, slump and surface area (as measured by the author's MSF') can be specified but problems can still result from details of the combined grading. Mix design should be a matter of combining available materials so as to minimise any disadvantages they may have individually. It is possible to specify conformance of each individual material to ideal requirements so that they can be combined in standardized proportions, but this is usually only practicable on large contracts for which aggregates are being specially produced. Even so some variation is inevitable, and it is difficult both to require rigid compliance with specified proportions and to provide for variation. This path leads to full acceptance of total responsibility for concrete quality by the supervising authority, which is undesirable for many reasons (from needing to take over control of incoming materials quality to facing claims by the Contractor that any defects in the finished product are due to matters beyond his control). The Australian Government airfield construction branch used such techniques in the 1980s. Excellent concrete resulted, and it was considered by those in charge that the high cost sometimes caused was justified by the importance of the work.

The preferable course is to specify as closely as possible the properties required of the concrete and require the Contractor to set out in full detail exactly how he proposes to provide them, including his specification limits on incoming materials and within what limits and to what accuracy, he proposes to adjust the mix. This clearly gives the Contractor absolute freedom to propose the most economical and practicable way of providing concrete of the required properties. It is very much easier to detect any unsatisfactory features of such a proposal than it is to so specify a mix that it could not possibly have any unsatisfactory features.

Once the Contractor's proposals have been accepted by the Supervisor, they become the specification. Insistence on conformance to this specification is easier since the Contractor, having proposed it himself, cannot claim it to be unrealistic in any way and there can be no surprise 'loopholes' in the original specification.

Of course, in the author's opinion, even this type of individual attention to mix regulation by a purchaser would only be justified on very large projects, usually those with a dedicated supplying plant.

Chapter 7

Aggregates for concrete

7.1 Fine aggregate (sand)

The basic material of a natural fine aggregate is not usually a matter of concern. To some extent this has been 'tested' by the formation process and any weak material broken down. There are some sands (e.g. You Yang Sand, a granitic sand from Melbourne, Australia), that are absorptive and may show some moisture movement, but generally the concerns are only with impurities, grading and particle shape.

For too long the approach to sand quality regulation has been to consider what constitutes a 'good' sand, write a specification covering these features and accept or reject submitted sands on this basis. Sands satisfying typical specifications of this type are becoming unobtainable or uneconomic in many parts of the world and it is necessary to devise an alternative procedure. Moreover a 'good' sand is only good if used in the correct proportion – which is likely to differ within any reasonable specified range.

What matters to the eventual owner of the concrete structure is not the sand itself but the resulting concrete. Essentially this means that a technically satisfactory sand can be defined as one which enables the production of satisfactory concrete. The required concrete properties should be fully specified by the purchaser and the sand properties should be at the discretion of the concrete producer. Possibly the same situation could apply to coarse aggregates, but it is easier to justify with fine aggregates because the effects of a sub-standard fine aggregate tend to be more immediately experienced. Such effects may include retarded set, increased bleeding, excessive air entrainment, poor workability and increased water requirement, the latter in turn leading to increased shrinkage and extra cost.

The potentially deleterious features of fine aggregate

Seven features of a fine aggregate affect its suitability as a concrete aggregate:

1 Grading
2 Particle shape and surface texture
3 Clay/silt/dust content

4 Chemical impurities
5 Presence of mechanically weak particles
6 Water absorption
7 Mica content.

Any of these, with the possible exception of water absorption, can have such serious effects on concrete as to preclude the use of the aggregate (even under the relaxed and generous criteria proposed by the author). However this discussion will concentrate on grading, with only brief comments on features 4–7. This is partly because the author's views on the other six features are not significantly different to those of many others, whereas his treatment of grading is original and has permitted him to make use of sands considered not economically useable by others.

Much of the material in this chapter was presented in a paper entitled 'Marginal Sands' presented to an ACI Convention in San Antonio in March 1987 (and available on the website).

Grading

Grading can be regarded as the main feature of a sand, and the feature which most frequently stops a particular sand being exploited. However, to a considerable extent, a less than ideal grading can be fully countered by adjusting the mix proportions (i.e. the sand percentage) without additional cost in cement.

The basic concept is to use a smaller amount of a finer sand so as to leave unchanged both the water requirement and the cohesiveness of the mix. In any particular case, the ideal sand percentage is not solely a matter of its grading. Other factors influencing the ideal percentage include cement content, entrained air content, particle shape and grading of the coarse aggregate, and also the intended use of the concrete. As explained in Chapter 3, these factors lead to the selection of a suitable MSF and thence to a suitable combined specific surface of the coarse and fine aggregates. This allows the direct calculation of the required sand percentage from the modified specific surface (SS see Section 3.1) of the sand. *This process assumes that the actual grading of the sand will only influence the percentage of it to be used and have no other influence on concrete properties*. While this is the case over a wide range, there must be limits to its applicability. It is necessary to be very clear where the limits are and what happens if they are exceeded.

Chapter 3 includes a very thorough examination of the coarse and fine limits on the usability of a sand and on the selection of the most advantageous combination of two sands.

Grading indices

There has always been an attraction in representing a sand grading by a single number which will describe its performance in concrete. For example this would avoid the problem of sand gradings straying into two different zones and would permit adjustment of sand percentages on a continuous scale rather than three large steps.

The original and perhaps most widely known and used grading index is the Fineness Modulus. This is the sum of the cumulative percentages retained on each sieve from 150 micron upwards. This index is used in the ACI mix design system to adjust for sand fineness. However it is used to indicate adjustment steps rather than to give continuous adjustment in a formula.

The Specific Surface is the surface area per unit weight (per unit solid volume would be preferable but is not usually used). This is difficult to measure directly but may be estimated from measured or assumed values of Specific Surface for each individual sieve fraction in a manner similar to Fineness Modulus. If dealing with perfect spheres, halving the diameter exactly doubles the surface area per unit weight. This simple assumption gives a reasonable index for aggregate proportioning but what is really required is a prediction of water requirement. The greater the surface area and the higher the water requirement, but the effect of the finer sieve fractions on water requirement is not as great as surface area suggests (Day, 1959).

Table 7.1 (Popovics, 1982) sets out 10 lists of factors for the numerical characterization of individual sieve fractions. The author's modified specific surface has been added to form an 11th column (the origin of the author's values has been explained in Section 3.2). Some of these factors have been used as a basis for selecting the relative proportions of fine and coarse aggregates, some to calculate water requirement, and some (including the author's) for both of these purposes.

Popovics (Popovics, 1992) also sets out 26 formulas, 12 of which were originated by himself, for the calculation of water requirement. Some of the formulas are quite complex and tedious to evaluate, but this would be no disadvantage if the formula were included as part of a computer program. However only a dedicated research worker could consider the time and effort which would be involved in examining the relative merits of the 26, or even the 12, formulas over a range of actual mix data.

No doubt each proponent of a system (including the author) considers his own system quite simple to use.

It is not proposed to examine all the alternatives in the current volume but, in view of the widespread use of fineness modulus, some attention should be given to it.

Table 7.2 is given in two of Popovics' books (Popovics, 1982, 1992) and is derived from Walker and Bartel (Walker and Bartel, 1947). This table provides an optimum value for the fineness modulus of the combined coarse and fine aggregates.

Table 7.2 is valid for natural sand and rounded gravel having voids of 35%. 0.1 should be subtracted from the tabulated values for each 5% increase in voids. For air entrained concretes, add 0.1 to the tabulated values. The values are for 25–50 mm slump concrete, subtract 0.25 for 100 mm slump and for zero slump add 0.25.

Table 7.1 Various proposals for sand grading indices

Limits of size fraction (*Sieve*)	*d* (*mm*)	$d_{e'}$ (*mm*)	$d_{e'}$ (*in*)	$s_{e'}$ (m^2/m^2)	*m*	e	λ	ρ	*A*	*i*	f_s	*Modified SS*
$3–1\frac{1}{2}$ in	75–37.5	56.25 (100)	2.21 (100)	106.7 (100)	9.56 (100)	0.638 (100)	9.33 (100)	2.53 (100)	0.020 (100)	0.06 (100)	−2.5 (−100)	1
$1\frac{1}{2}–\frac{3}{4}$ in	37.5–19.0	28.25 (50.2)	1.11 (50.2)	212.4 (199)	8.56 (89.5)	1.01 (158)	11.34 (122)	3.57 (141)	0.035 (175)	0.12 (200)	−2.0 (−80)	2
$\frac{3}{4}–\frac{3}{8}$ in	19.0–9.5	14.25 (25.3)	0.561 (25.3)	421.4 (395)	7.58 (79.3)	1.59 (250)	13.95 (150)	5.03 (199)	0.055 (275)	0.19 (317)	−1.0 (−140)	4
$\frac{3}{8}$ in–No. 4	9.5–4.75	7.12 (12.7)	0.280 (12.7)	842.7 (790)	6.58 (68.2)	2.53 (396)	17.49 (187)	7.12 (281)	0.075 (375)	0.27 (450)	1.0 (40)	8
No. 4–No. 8	4.75–2.36	3.56 (6.32)	0.140 (6.32)	1685 (1,580)	5.58 (58.4)	4.02 (631)	22.3 (239)	10.07 (398)	0.096 (480)	0.39 (650)	4.0 (160)	15
No. 8–No. 16	2.36–1.18	1.77 (3.15)	0.0697 (3.15)	3,390 (3,178)	4.57 (47.8)	6.40 (1003)	29.2 (313)	14.28 (564)	0.116 (580)	0.55 (917)	7.0 (280)	27
No. 16–No. 30	1.18–0.60	0.89 (1.58)	0.0350 (1.58)	6,742 (6,321)	3.58 (37.4)	10.10 (1584)	39.0 (418)	20.14 (796)	0.160 (800)	0.70 (1167)	9.0 (360)	39
No. 30–No. 50	0.60–0.30	0.45 (0.80)	0.0177 (0.80)	13,333 (12,500)	2.60 (27.2)	15.94 (2500)	53.5 (573)	28.32 (1119)	0.24 (1200)	0.75 (1250)	9.0 (360)	58
No. 50–No. 100	0.30–0.15	0.225 (0.40)	0.0089 (0.40)	26,667 (25,000)	1.60 (16.7)	25.30 (3969)	76.8 (819)	40.06 (1583)	0.35 (1750)	0.79 (1317)	7.0 (280)	81
No. 100–pan	0.15–0	0.075 (0.13)	0.0030 (0.13)	?	0	—	?	?	?	1.0 (1667)	2.0 (80)	105

Notes
Values in parentheses are presented relative to the numerical characteristics of size fractions $3–1\frac{1}{2}$ in (75–37.5 mm). *d* = average particle size, mm; d_e = average particle size, in; *s* = specific surface (Edwards, 1918); *m* = fineness modulus; e = water requirement (Bolomey, 1947); λ = distribution number (Solvey, 1949); ρ = stiffening coefficient (Leviant, 1966); *A* = *A* value (Kluge, 1949); *i* = *i* index (Faury, 1958); f_s = surface index (Murdock, 1960).

Table 7.2 Optimum values of fineness modulus

Maximum size of aggregate		*Weight of cement*								
No.	*mm*	*280* *170*	*375* *225*	*470* *280*	*565* *335*	*660* *390*	*750* *445*	*850* *500*	*950* *560*	*(lb/yd³)* *(kg/m³)*
No. 30	0.60	1.4	1.5	1.6	1.7	1.8	1.9	1.9	2.0	
No. 16	1.18	1.9	2.0	2.2	2.3	2.4	2.5	2.6	2.7	
No. 8	2.36	2.5	2.6	2.8	2.9	3.0	3.2	3.3	3.4	
No. 4	4.75	3.1	3.3	3.4	3.6	3.8	3.9	4.1	4.2	
$\frac{3}{8}$ in	9.5	3.9	4.1	4.2	4.4	4.6	4.7	4.9	5.0	
$\frac{1}{2}$ in	12.5	4.1	4.4	4.6	4.7	4.9	5.0	5.2	5.3	
$\frac{3}{4}$ in	19.0	4.6	4.8	5.0	5.2	5.4	5.5	5.7	5.8	
1 in	25.0	4.9	5.2	5.4	5.5	5.7	5.8	6.0	6.1	
$1\frac{1}{2}$ in	37.5	5.4	5.6	5.8	6.0	6.1	6.3	6.5	6.6	
2 in	50.0	5.7	5.9	6.1	6.3	6.5	6.6	6.8	7.0	
3 in	75.0	6.2	6.4	6.6	6.8	7.0	7.1	7.3	7.4	

Equation 7.1, also from Popovics (Popovics, 1982) gives the water required to provide a 100 mm slump in units of lbs/cu yd (divide by 1.685 to convert to litres per cubic metre).

$$\text{water requirement} = c\{0.1+0.032[(2^m-60)^2 + 6570]/(c-100)\} \qquad (7.1)$$

where

m = fineness modulus of combined aggregates

c = cement content in lb/cu yd (=kg/m^3 × 1.685)

Murdock and Hughes also introduce a term for angularity of grains. This clearly influences water requirement but cannot conveniently be used to give an adjustment to these values (see next section).

The concept of specific surface mix design is that an appropriate specific surface for the overall grading be selected allowing for the intended use. A low workability high strength concrete (e.g. for heavy vibration into precast products) would require a low specific surface to reduce water requirement but a high slump mix would require a higher specific surface to avoid segregation (see Table 3.1 in Chapter 3).

The sand percentage is then calculated to provide the required specific surface. The method has produced usable concrete mixes with sand percentages varying from 15 to 55% of total aggregates in particular circumstances but 25–50% of sand is a fairly safe range.

The grading zones do not overlap because the 0.6 mm sieve is taken as the criterion. However looking at the SS values or even the FM values (Table 7.3), it

Table 7.3 Inter-relationship of old UK grading zones, specific surface and fineness modulus

Sieve size (mm)	*Grading requirements % passing*					
	Zone 1	*Zone 2*	*Zone 3*	*Zone 4*	*ASTM C33-71A*	*AS1465 1984*
10.000	100	100	100	100	100	100
4.750	90–100	90–100	90–100	95–100	95–100	90–100
2.360	60–95	75–100	85–100	95–100	80–100	60–100
1.180	30–70	55–90	75–100	90–100	50–85	30–100
0.600	15–34	35–59	60–79	80–100	25–60	15–100
0.300	5–20	8–30	12–40	15–30	10–30	5–50
0.150	0–10	0–10	0–10	0–10	2–10	0–15
0.075	–	–	–	–	–	0–5
SS	29.40–48.31	38.54–58.31	48.00–66.00	56.00–72.00	38.00–57.90	29.40–73.10
FM	4.00–2.91	3.37–2.11	3.00–2.00	2.00–1.00	3.00–2.15	4.00–1.35
Avg. SS	38.85	48.42	52.06	63.70	47.91	41.75
Avg. FM	3.35	2.74	2.44	1.82	2.76	3.17

is clear that the *properties of the sands* in different zones are likely to overlap. This can be avoided by defining a Zone 1 sand as a sand having an SS of 38.85 (or say 40, or 34–44), with Zone 2 being say 48 or 44–52, Zone 3 being 56 or 52–60 and Zone 4 being 64 or 60–70.

It has been contended that, to a very large extent, only the surface area and not the detailed grading of a sand is of importance. This is not completely true in all cases and the following exceptions are noted:

1 The existence of gaps in the grading (i.e. the absence of some sieve fractions) either between the sand and the coarse aggregate or within the sand grading itself can give rise to:

   a segregation at medium to high slumps
   b severe bleeding
   c concrete which will not pump
   d improved workability under vibration for low slump concrete.

2 Sands which are almost single-sized can give rise to poor workability through particle interference.
3 A proportion of large particles in an otherwise predominantly fine sand can cause problems through interfering with the packing of the coarse aggregate.

It is emphasised that these are rare exceptions, not glaring deficiencies in the general assumption.

Air entrainment

The use of admixtures can be of considerable assistance in solving grading problems. Air entrainment is well known to have the capacity to inhibit bleeding and to assist in overcoming problems of harshness with very coarse or very angular fine aggregates. An unusual use for air entrainment is worth recounting. The mix was specified not to contain any silicious aggregates (including natural sand) because it was to be used in the base of a furnace. This left, as the only available fine aggregate, a crusher dust with almost 20% passing a 150 μm sieve.

The author's system correctly predicted the proportion of this material that would make reasonable concrete and correctly predicted its water requirement. However, especially since a high minimum cement content was also specified, the mix was very sticky and difficult to handle from skips etc., even though it compacted quite well. These days a superplasticising admixture and a higher slump would probably be used, but this mix was encountered before such admixtures were readily available in Australia and in any case would have represented extra cost since the minimum allowable cement content already provided excess strength. Instead, an air entraining agent was used and did produce a substantial improvement. It is interesting that air entrainment can both increase the cohesion of a harsh mix and lubricate a sticky mix since these are virtually positive and negative effects on the same property of the concrete.

Particle shape

We have seen that a fine sand has a higher water requirement but, over a wide range, it can simply be used in smaller proportion to give a normal water requirement. An angular sand, or especially crusher fines, also has a higher water requirement for a given grading. However this does not justify a reduction in its proportion (it may even justify a small increase, thus further increasing water requirement, but this is too fine a piece of tuning to incorporate into a relatively simple system). There is therefore an inevitable increase in water requirement of the mix, and therefore an additional cost in cement when an angular fine aggregate is used. However the angular fines may be very cheap, or otherwise be the least costly alternative in overall concrete cost, or may be technically essential.

Examples of where crusher fines may be justified are:

- Where natural sand is very expensive (owing to long haulage distance or otherwise).
- Where the natural sand is so fine that it would have to be used in a mix of more than the otherwise desirable surface area.
- Where the natural sand has a high clay content and it is cheaper to accept the higher cement requirement than to wash the natural sand.
- Where the natural sand is so coarse that the crusher dust is necessary as a filler.

Apart from the above economic considerations, there may also be technical reasons for using or not using the crusher fines:

- A coarse grade of crusher fines may be needed to fill the gap between the top of a fine sand grading and the bottom of the coarse aggregate grading. This may be essential to provide pumpability or to avoid segregation where high slump is necessary.
- It should be remembered that a higher water requirement is not purely an economic disadvantage. It also gives increased shrinkage and so may be unacceptable for some purposes even if it is the most economical way of providing the required strength.
- There will normally be a distinct difference in colour between a crusher fines mix and a natural sand mix. One or other may therefore be architecturally either preferred or rejected for exposed architectural concrete.
- There may be a substantial difference one way or the other (depending on actual gradings) in bleeding characteristics which may have a substantial effect on surface appearance (coarse crusher fines being particularly susceptible to heavy bleeding but fine dust inhibiting it).

It is quite frequently a satisfactory arrangement to use a combination of crusher fines and natural sand. The author has formed an opinion (rather than definitely established) that there tends to be more benefit than expected from such a combination (see sand flow cone, following).

Apart from gradings often fitting well together (crusher fines tending to be deficient in middle sizes and natural sand to have an excess) a small proportion of a fine, rounded, natural sand appears to have a disproportionate effect on reducing the ill effects of angularity. Also the first 2% or so by weight of silt in a fine aggregate appears not to be deleterious so that halving the amount of a silty sand will more than halve the water increasing effect of its silt.

Air entrainment and crusher fines should be approached with a little more caution. Trial mixes will very clearly show a big advantage for air entrainment. However stone dust inhibits air entrainment and, if its proportion varies, can result in a high variability of air content which may be unacceptable in practice. Note that fly-ash (pfa) gives a similar effect on workability to that of air entrainment but is not susceptible to being inhibited or varied in its effect (other than its own inhibiting effect on air content, which is heavily dependent upon its carbon content, as measured by its loss on ignition). So crusher fines may be more acceptable in mixes containing fly-ash.

The extent of the effect of particle shape can be 10%, or even more, water increase with the fine aggregate being entirely of badly shaped (but still well graded) crusher fines. However 7% increase is more normal for crusher fines and a badly shaped natural sand may cause as much as 3 or 4% increase. Badly shaped natural sand usually comes from glacially formed pit deposits rather than rivers or beaches. (Note that sand flow cone experimenters claim to have found fine aggregates which increase water demand by as much as 15%.)

Clay, silt or dust content

The author's system does not provide for the incorporation of the effect of material finer than a 75 micron (200 mesh) on his 'Specific Surface' (it is counted the same as material passing the 150 micron (100 mesh) sieve and retained on the 75 micron sieve). This is for the same reason that the effect of angular grains is not incorporated, that is, it does affect water requirement but it does not justify an offsetting reduction in the proportion of the fine aggregate. A subsidiary reason is that the increase is not solely dependent on the weight of such material but also on its character.

It is arguable whether the 75 micron (200 mesh) sieve is worthwhile for checking fine aggregates for concrete. Certainly it is important how much of such material there is in the aggregate, but the percentage by weight gives only half the story and dry sieving rarely removes all such material. Some materials, such as the montmorillonite (smectite) clay in sand extracted in Singapore, can have three times as much effect per unit of weight as other fines such as fine crusher dust also passing the 75 micron sieve.

The definitive test for this property is undoubtedly the French '*Valeur de Bleu*' (Bertrandy, 1982). This test involves titrating wash water from the fines with methylene blue, which is essentially a dye composed of molecules that are single particles of absolutely standard size. The dye molecules are attracted to the surface of the fines and none remain in suspension so long as any surface area of fines remains exposed. It is possible to calculate the surface area of superfine material from the amount of the dye that has to be added before any remains in solution. This point is determined by placing one drop of the solution on a standard white blotting paper. As soon as any dye remains in solution, a faint blue halo surrounds the central muddy spot. This test is a French (tentative?) standard (also now ASTM C837-99(2003) and is fairly easy to do in a chemical laboratory (i.e. a laboratory mechanical stirrer and a burette are needed). However, there is no point in incorporating it into the author's system because the test result would rarely be available when needed.

The alternative is very simple indeed and is the standard Field Settling Test. Both the process of obtaining it and the use of this figure (a percentage of clay by volume when the fine aggregate is shaken up with salt solution or sodium hydroxide in a measuring cylinder and allowed to settle) are very crude indeed but it nevertheless greatly improves the accuracy of the water prediction. The assumption made is that every 100 kg of the fine aggregate will require an extra 0.225 litres of water for each 1% by which its silt content by volume exceeds 6%, for example, 600 kg/m^3 of fine aggregate with 8% silt content will require $6 \times 0.225 \times (8-6) = 2.7$ litres of extra water.

When the silt correction originated in Singapore, the sand was very coarse, requiring over 900 kg/m^3 and the silt per cent was over 25% by the settling test on occasions (9% by weight). This meant that over 20 litres of additional water was required, sometimes almost 30 litres. The figure was initially derived by taking a 44 gallon drum of the dirty sand, inserting a running hose to the bottom and overflow rinsing until the water ran clear. A repeat of the original trial mix before washing showed a water reduction of almost 30 litres. No excuse is offered for the blatant crudity of this 'clay correction' because for several years now it has given good results on many different sands in Australia and SE Asia.

The additional water figure can be translated into an additional cement figure when the required w/c ratio is known. This gives a fairly precise figure for the cash value of washing the sand and so a basis for deciding whether or not to set up a sand washing plant. However, it is often better to counteract the effect of the clay by using a superplasticising admixture than by accepting it and using additional cement. This view has been confirmed and quantified in the laboratory by Tam Chat Tim (Tam, 1982).

A final point on the subject of fines contents is that crusher fines dust can give a distinct (but not large) strength increase at a given water/cement ratio. In fact this is not surprising because Alexander (CSIRO Melbourne in 1950s) has shown that siliceous stone dust can have pozzolanic properties if it is ground sufficiently fine. Also calcareous stone dust (e.g. limestone) will react chemically. However the author's practice is to use the settling test to allow for the extra water requirement of the fine dust but to neglect the possible strength increase.

Other impurities

Chemical impurities

The question of more exotic chemical impurities is left to others but the two questions of salt and organic impurity must be addressed.

There is an extensive literature on chloride contents and their capacity to promote the corrosion of reinforcing steel. Beach sand is liable to have very high salt levels owing to the deposition of salt by evaporation. Sand dredged from the sea may be less of a problem but without washing with fresh water may still exceed a fully safe level. Salts can also cause efflorescence and higher shrinkage and affect setting and hardening rates.

Organic impurity is quite frequently encountered in pit sands. The author's practice is to combine the colour test (BS 812, 1960) for organic impurity with the settling test for clay content by using sodium hydroxide instead of the specified salt solution for the latter test. It is to be noted that the use of pure water will give a different result with the clay taking longer to settle and giving a higher reading. The important point to realize is that the test only establishes whether organic impurity is present and not whether it is deleterious. The colour test can be failed due to the presence of a few pieces of organic matter such as small twigs or other vegetation which are too few and too localized to have any significant effect on strength (but could produce a visual defect on a surface).

Sands failing the colour test should then be tested for setting time and initial strength development. If they are satisfactory in these respects, it is unlikely that there will be any long term problems (although another problem encountered has been of sands which automatically entrain air due to natural lignin).

The usual effect of impurity (if there is any effect) is of retarding or preventing chemical set. If there is no ill-effect on strength up to 28 days the sand is satisfactory. There may be a strength reduction at 1–7 days but no loss of strength at 28 days, which may or may not be satisfactory for particular applications. There may be implications, with early strength loss, of setting time extension and consequent surface finishing problems for slabs.

For organic impurity evaluation, comparative mortar cubes should have the same water/cement ratio, not the same workability.

Natural impurities are not the only kind and there have been instances of accidental contamination, especially with sugar. One example was of a barge used to transport sand after transporting a load of bulk raw sugar, one result of this was to cause a large floor slab in a multi-storey building not to set for several days. It takes very little sugar to cause a problem, for example, the author has experienced a concrete strength problem later traced to employees emptying the dregs of their morning tea onto the sand pile of a small manual batching plant.

Rivaling the frequency of occurrence of all the above combined in the author's experience has been the frequency of multiple dosing of retarding admixtures. This is outside the scope of the book but it has provided more examples of concrete that has eventually proved quite satisfactory after taking several days to set. The message here is not to panic too early. If a sample sets after being in boiling water overnight (inside a plastic bag of course) then the concrete in the structure will set eventually. The question is whether it will develop serious settlement cracks in the interim due to prolonged bleeding, or to water soaking into formwork or escaping at joints that are not watertight. It is certainly important to cover the concrete with plastic sheeting or wet hessian in order to stop it drying out.

Weak particles and high water absorption

These are not common in river sands but can be encountered in pit sands. Except in very high strength concrete, or concrete required to have wear resistance or frost resistance, the direct effect on concrete strength is not likely to be a problem. Degrading during mixing, increasing fines content and therefore increasing water requirement, is possible (but more likely in a coarse aggregate). A high water absorption may indicate an increased drying shrinkage and could also indicate a reduced freeze/thaw resistance.

Mica content

Except possibly in very high strength concrete, there does not appear to be a problem with moderate amounts (less than 5%) of mica directly weakening the mortar. Rather the problem appears to be an increased water requirement. Probably mica that can be seen does not do much harm but it may indicate the presence of finer mica particles that will have much more influence on water requirement and possibly significantly increase the moisture movement tendency of the mortar.

Mica is usually detected visually but can be extracted by the use of a liquid heavier than mica but lighter than sand. However its effect on the water requirement of mortar and therefore its strength, *this time at fixed workability*, is probably easier to determine and more relevant.

Update 2005 on crushed fine aggregate

The subject of natural sand was adequately covered in the 2nd edition and the section above has required little editing. However the availability of natural sand is reducing in many parts of the world. Also many are finding that, once understood and appropriately produced and used, manufactured sand or crusher fines (abbreviated to Msand) can be an advantage rather than a disadvantage. This is a rich subject for further experimentation, several points can be usefully presented but some remain as speculation. The views expressed have been substantially influenced by discussions with many people, especially including Norwood Harrison (see below), Aulis Kappi (Addtek, Finland), Stacy Goldsworthy and Chris Glass (Metso, NZ, makers of Barmac crushers) and Mark Mackenzie (Hanson, Australia, formerly Alpha, South Africa).

There is no dispute about the effect of replacing natural material between the 4.75 mm and 150 micron (0.15 mm) sieves with crushed material of the same grading. The effect is to increase water requirement to the extent of 5 to even 15% depending on particle shape and, in many cases, to increase strength at a given w/c ratio.

The interesting question is the effect of material finer than 150 micron. In a natural sand, such material is often deleterious and is usually removed or reduced by washing. Stacy Goldsworthy provides the following assessment:

High fines in concrete (by Stacy Goldsworthy)

Contrary to popular belief, the presence of quantities of microfines (minus200#) can be beneficial in most concrete mixes. The use of quality high microfines manufactured sand in concrete has been common practice in some countries for over twenty years. These markets have developed systems and procedures to make its use standard. Early trials suggested that satisfactory performance could be obtained, so experience and use grew.

Research and use of microfines indicates that there is a substitution effect with the use of cement. The optimum microfine content for 'lean' mixes is up to 15% and in some cases can be as high as 20%. However, for 'fat' mixes the optimum percentage of microfines will be as low as 5%. The microfine content acts as a filler, reducing void space that would otherwise require cement paste to fill. Field experience illustrates this point, where microfines are used in concrete, there is an associated increase in 28 day concrete density. The increase in density results in higher compressive strengths and even higher flexural strengths. Examples from the field have shown that the substitution of microfines for cement has improved the quality of the concrete and reduced the cost of production.

Most of the negative connotations concerning microfines relate to the increase in water demand and the increase in shrinkage. This will surely be true where mineral clays or similar materials are present. However, if the material passing 200# is purely from the fracture of rock then it will enhance performance. The presence of mineral clays is detrimental, but in the world of concrete this distinction is not very well known. So, what are mineral clays and why have specifications limited their use?

Clay minerals

The term 'clay mineral' refers to phyllosilicate (sheet-like) minerals and to minerals that impart plasticity to clay and which harden upon drying or firing. ASTM C33 is based on the use of natural sand and, for such a material, it is quite correct to strictly limit the percentage passing 200#. It is unfortunate that many people incorrectly apply this specification to crusher fines. The inclusion of mineral clays and other deleterious materials leads to poor performance in concrete. High water demand, high shrinkage and reduced compressive and flexural strengths being common.

There are many types of mineral clays. Their structure and, therefore, capacity to absorb water and shrink and swell varies. The sheet like structure of many of them allows water to be absorbed. Between the sheets there is a space called the interlayer. It is here that the clay particle absorbs water and cations.

Microfines, created by crushing rock, or rock flour as they are sometimes called, do not have the capacity to absorb water. By definition most of the particles contained are silt in grain size (2 to 63 micron) with a structure similar to that of sand. Absorption of water is over the surface of the particle not within it. Therefore they must be treated differently.

The slightest presence of clay minerals causes a reduction in the tensile strength. Montmorillonite, being a swelling clay, causes the greatest reduction in tensile strength. Kaolinite has a similar but lesser effect. Fines containing crusher dust improve the tensile strength up to a certain point and then there is a gradual tailing off in the strength at percentages somewhere in excess of 20%.

The reactivity of the microfines contained must be determined as part of the evaluation of a manufactured sand. If the source rock has weathered or contains certain minerals there may be the capacity for the microfines to be reactive. Reactive means having properties similar to that of mineral clays, the ability to significantly increase the water demand. Methylene Blue Titration provides a good measure of reactivity (ASTM C837-99(2003)). This test method determines the absorbency of the microfine fraction. Calibration of the test results to field performance provides good accept/reject criteria. However removing microfines by washing creates a manufactured sand that will increase the harshness of concrete.

Fine aggregate water requirement related to per cent voids and flow time (by Norwood Harrison)

The subject of water requirement cannot be left without discussing the sand flow cone test and percentage voids. The test consists of pouring a fixed amount of dry fine aggregate into a metal funnel and allowing it to discharge into a container below, (which overflows) (Fig. 7.1). The time taken for all the material to leave the funnel is recorded. Aggregate collected in the container is struck off to a level surface, and weighed in the container. This weight, together with the container volume and dry particle density of the test material are used to calculate the percentage of voids.

Flow time and percentage of voids depend on the shape and surface texture of the fine aggregate, and the grading. This is illustrated in Fig. 7.2, which shows plots of flow time and voids for sands having artificially adjusted gradings. The grading variations were applied to two basic sands to give two series, one having good particle shape and smooth surface texture (Series 1) and the other poorer particle shape and rough surface texture (Series 2). It can be seen that with deterioration of shape & surface texture, and the same specific surface (SS), the plot moves towards higher voids and longer flow time.

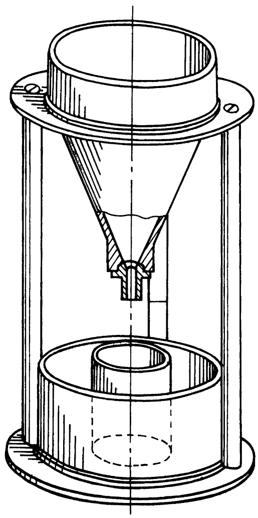


Figure 7.1 Sand flow cone apparatus

Malhotra (1964) used a form of the flow test to evaluate shape and surface texture of a range of sands and the effect on workability of mortars made with them. The sands were sieved to provide size fractions to comply with two grading criteria, and used in mortars of set composition for each of the two gradings. Workability of the mortars was assessed using a flow table. It was concluded that 'the orifice test appears to be a satisfactory means of determining the shape and surface texture, and hence the water requirement, of fine aggregate'.

The test has been further developed in New Zealand (Clelland, 1968; Hopkins, 1971) and independently in USA (Gaynor, 1968; Tobin, 1978). The voids result depends little, if at all, on the dimensions of the equipment or the sample size but different flow times will result from differences in the equipment and size of sample. It was found, for example (Kerrigan, 1972), that even the sharpness of the transition from conical to cylindrical profile at the orifice has a marked effect on flow time. Kerrigan (1972) and Elek (1973) describe a standardized test with defined sample size and dimensions of the test equipment, including the size and profile of the orifice. The specification also includes removing any particles of size greater than 4.75 mm from the test sample, as these interfere with the flow. Flow time results reported in this account of the test have all been obtained using the equipment & procedure developed & standardised by Kerrigan and Elek.

Correlation of voids in fine aggregate and corresponding water demand of concrete is acknowledged in the ACI publication 'Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly ash' (1998), which advises a factor of (percentvoids – 35) × 8lb/cu.yd. (app. 5kg/cu.metre) amounting to approximately 15% increase in water demand per 5% increase in voids, for fine aggregates having the same grading. As the voids property of commonly used fine aggregates ranges from below 40% to approaching 48% this represents a very significant change (more than 20%) in water demand, and corresponding cement content to obtain the same performance from the concrete.

Harrison (1988) analysed data from 37 examples of concrete mixes for which both the flow test parameters of the fine aggregate and water demand of the mixes were known. The latter was expressed as a dimensionless parameter 'relative water demand' (RWD), being the factor between water demand of a mix made with the fine aggregate in question and a corresponding mix having fine aggregate for which voids and time plot at a particular location on a chart with axes as shown in Fig. 7.3. Using linear functions, correlations were found between RWD and both per cent voids and the flow time.

The results shown in Fig. 7.2 and Harrison's data have subsequently been analysed further to find the positions and orientations of plane surfaces which best represent the dependence of specific surface and relative water demand separately on the flow test parameters. The outcome of this analysis is shown in Fig. 7.3. Following a line of constant specific surface we can assess the dependence of water demand on either voids or flow time. For example, for SS = 50 (a middle-of-the range value), RWDs of 1.05 and 1.20 (i.e. 15% increase) correspond to 40.2% and 45.0% voids respectively, a difference of just under 5% – very close to the estimate from the ACI parameter. The chart also shows that water demand is not linked uniquely to voids or flow time separately, but to combinations of the two properties.

The test offers a quicker and simpler means than sieve analysis of detecting changes in grading during production use of a sand. In addition it simultaneously checks for any deterioration in particle shape or surface texture. The latter may be considered fairly unlikely to change for a natural sand from a particular location but would be well worth monitoring for crusher fines and would be very difficult to check by any other means.

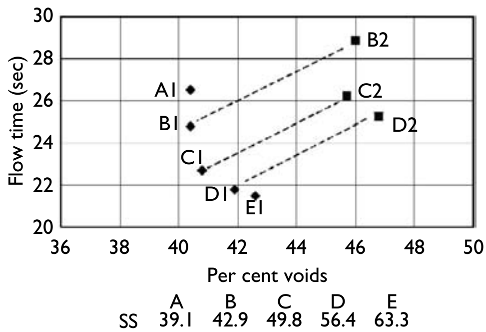


Figure 7.2 Flow test parameters of sands with controlled gradings.

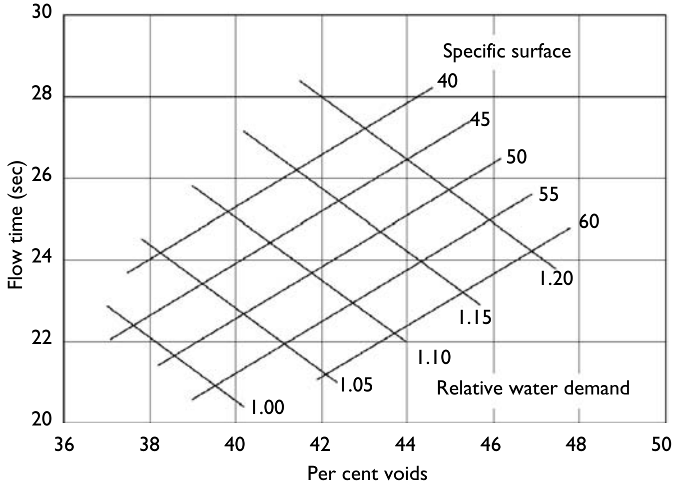


Figure 7.3 Correlation of water demand and specific surface with flow test properties.

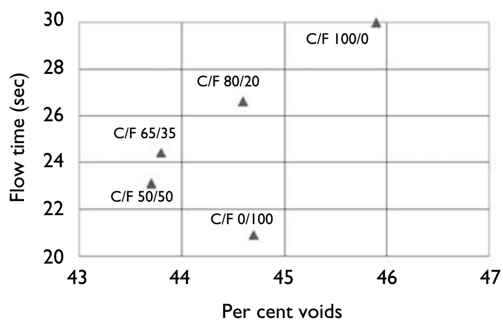


Figure 7.4 Blends of a coarse and a fine sand.

A further use for the sand flow cone is in blending two sands. It is a simple procedure to carry out a set of flow and voids tests with varying proportions of two sands, and a plot of the resulting properties from the flow test is very revealing as to the range of compatible proportions. An example is shown in Fig. 7.4, in which the coarse sand is a low cost material which is too coarse for use by itself in typical concrete mixes, but in blends with the more expensive fine sand gives a suitable and cost-effective fine aggregate for concrete.

In conclusion it must be emphasized that the flow test does not *measure* either the specific surface of a fine aggregate or its effect on water demand. Per cent voids and flow time are properties which respond to characteristics of the shape and surface texture of the particles, and the grading, to which both water demand

and specific surface are also related. Fig. 7.3 shows 'most likely' relationships based on limited data. Individual instances may not agree closely with the relationships shown, and the pattern itself can be expected to change, though perhaps not greatly, should more data become available.

Suggested further reading

1 B. M. Kerrigan, 'Sand Flow Test', Humes report RC.4243, 6/4/72.
2 V. M. Malhotra, 'Correlation Between Particle Shape and Surface Texture of Fine Aggregate and Their Water Requirement', Materials Research & Standards, December 1964, pp. 656–658.
3 J. Clelland, 'Sand for Concrete – a New Test Method', New Zealand Standards Bulletin, October 1968, pp. 22–26.
4 H. J. Hopkins, 'Sands for Concrete – a Study of Shapes and Sizes', New Zealand Engineering, 15 October 1971, pp. 287–292.
5 R. D. Gaynor, 'Exploratory Tests of Concrete Sands', JRL Series 190 Report, National Sand & Gravel Association / National Ready Mixed Concrete Association, Silver Spring (USA), 1967 and March 1968.
6 R. E. Tobin, 'Flow Cone Sand Tests', Title No. 75-1, ACI Journal, January 1978.
7 A. Elek, 'A Test for Assessment of Fine Aggregates', Hume News, August 1973, pp. 11, 12.
8 'Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash', ACI 211.4R-93, 1998.
9 N. L. Harrison, 'Description and Assessment of Sands', Humes report RC.1562, 6/9/88.

Author's comment: scope for further investigation

There is no doubt that good quality fine material can be beneficial. The questions arising are how fine? How much? In what circumstances? What is 'good quality'? Stacy has answered the latter question above and Norwood provides valuable information on evaluating the influences of grading and particle shape following this, but what of the 'circumstances'?

The circumstances to be considered are the content of cement, fly-ash, silica fume etc.; the properties required of the concrete, ranging from roller-compacted to self-compacting; and the presence or otherwise of air entrainment and (especially) water-reducing admixtures.

In straight cement mixes with no other material finer than 150 micron it is clear that water requirement will be higher with a very high cement content, will reduce with reducing cement content to some optimum range (perhaps 300–350 kg/m^3) and will then increase again with further cement reduction. What is happening is that, in the optimum range, cement paste fills the voids in the fine aggregate, excess cement requires additional water to form a paste with that cement, and if there is an inadequate amount of paste, additional water will be required to fill the fine aggregate voids.

Angular material in general has a higher void content than more rounded material, but the introduction of finer aggregate material, whatever its shape, may fill space that would otherwise be filled with cement paste or water. So it can easily be seen that this will be beneficial when cement content is below the optimum range and it should not be forgotten that cement particles are a crushed material of very poor particle shape.

The next question is whether superfine material can actually displace water from between cement particles in the same way that cement displaces water from between aggregate particles. The process seems possible considering relative sizes of particles but very fine particles tend to flocculate into clumps and behave as though they were substantially larger. Also such clumps would themselves contain water, making it unavailable to fill voids. This also occurs with cement particles, and dispersing such clumps is the mechanism by which admixtures reduce water requirement. So it is likely that, in a mix not containing admixtures, silica fume (for example) will increase water requirement, but in a mix containing a high range water reducer (HRWR) silica fume will give a further reduction in water requirement. A further consideration is that silica fume, when supplied dry, is usually deliberately 'condensed' into clumps for ease of handling. This is a good reason to purchase silica fume as a slurry, to convert it to a slurry prior to use, or to ensure that an HRWR is used and that adequate mixing time is allowed.

Taking all the above into account, it can be seen that there are no easy, universal answers to the question of whether fine crushed material might be beneficial. It may seem obvious that this would not be the case in mixes of high cement content, but if water is actually displaced from between cement particles, the benefit could be substantial.

An excellent tool for examining the properties of fine aggregates (natural or crushed) is the New Zealand sand flow cone, as described in the previous edition of this book. An update of this presentation has been provided by Dr Norwood Harrison. Norwood has extensive practical experience over many years in using this equipment on a wide range of fine aggregates in the Humes Ltd (now Rinker) laboratory in Melbourne and has also been responsible, along with the author, for spreading its use internationally, including to South Africa and Finland.

The sand flow cone is clearly suitable for examining the relative merits of different fine aggregates and different blends of two or more of such aggregates. However it seems unlikely that it could be adapted to examination of the effects of varying cement content or, especially, the effects of superfine materials such as silica fume or of chemical admixtures. More than 40 years ago, while a lecturer at the University of NSW, the author had an undergraduate research student (K. K. Mah) carry out an investigation into the effect of specific surface on water requirement of mortars (which confirmed the author's factors for modified specific surface, as still used in the author's system) using an ASTM mortar flow table. It seems that this technique could be used to study the full combination of materials in the mortar fraction of concrete. Using the author's MSF technique, the results on mortar could easily be transformed into concrete quantities.

However, particularly when thinking in terms of self-compacting concrete, it may not be necessary to use a jolting table but only to measure the flow diameter from a standard small cone, not necessarily that used in the ASTM flow table, but probably of similar volume.

The proposed technique would not be as rapid as the dry sand flow test, and would require the use of a Hobart mixer or similar, so it would probably not replace the latter. The objective should be to establish whether optimum gradings or grading combinations established by the sand flow were still optimum under a range of contents of cement, silica fume and other fine materials. A particularly important point would be to establish the optimum content and fineness of material passing a 150 micron sieve in manufactured sand for various types of concrete (since this is an item that could fairly readily be controlled). The test would also be useful to ensure that unfavorable reactions did not occur between cement, admixture and superfine material (as reported in Section 3.6 on mix design competitions). Perhaps small test cubes could be cast to yield a strength correction factor in mix design (i.e. to establish whether, and to what extent, the materials combination under test gave a strength increase at a given w/c ratio.

Using the author's MSF criterion, it is clear that the proportion of mortar in a cubic metre of concrete will be approximately inversely proportional to the SS of the fines used, or more strictly, to the MSF of the mortar (since the coarse aggregate makes a minor contribution to MSF). So, if the water content (*Wm*) of a mortar per cubic meter (of mortar) were determined, either experimentally or by a yet to be discovered calculation, that of the concrete (*Wc*) would be readily calculable as:

$$Wc = \frac{Wm \times (\text{Reqd MSF of concrete} - \text{SS of coarse aggregate})}{\text{MSF of mortar}}$$

If any reader is interested in a PhD thesis on this topic, the author would be pleased to engage in further discussion of it.

Talking of PhDs, reference is made in Section 11.7 on workability testing to work being done at ICAR (University of Texas at Austin) on the development and use of a highly portable rheometer that is yielding a number of interesting PhD theses. One of these is by Sinan Erdogan on investigating the effects of particle shape in both coarse and fine aggregates. The investigation is at too great a depth to present here and includes substantial work using X-ray Tomography and Microtomography to actually measure the shape of individual particles, in addition to using the rheometer. *Very* briefly he finds that the particle shape of coarse aggregate does not greatly affect yield stress (which is essentially what the slump test measures) but does greatly affect the plastic viscosity (which is the part of workability the slump test does not reveal). Equally clear conclusions are not reached in respect of fine aggregates and those interested should consult the thesis (Erdogan, 2005).

7.2 Coarse aggregate

The properties of a coarse aggregate depend on the properties of the basic rock, upon the crushing process (if crushed) and upon the subsequent treatment of the aggregate in terms of separation into fractions, segregation and contamination.

Most rock has an adequate basic strength for use in most grades of concrete. Even manufactured and naturally occurring lightweight aggregates, which can be readily crushed under a shoe heel, are used to make concrete with an average strength up to 40 MPa (although they do require a higher cement content than dense aggregates). Exceptions to this are some sandstones, shales and limestones (although other limestones are very strong and amongst the best aggregates for many purposes). A different type of exception is that use involving wear and impact resistance can require a more stringent selection of rock type.

Generally however the stability of a coarse aggregate is more important than its strength. Rock which exhibits moisture movement (swelling and shrinking) will add to concrete shrinkage. Again sandstone tends to be amongst the offenders, but some basalts will also display moisture movement and some breccias or conglomerates may be quite strong mechanically and yet literally fall part under a few cycles of wetting and drying.

Rock from an untried source must be tested for susceptibility to alkali-aggregate reaction. Whilst comparatively rare, this reaction produces such catastrophic results that its occurrence should not be risked without at least a petrographic report. There is a rapid chemical test for reactivity but it is not very reliable.

Another important feature of a coarse aggregate is its bond characteristics (especially in high strength concrete and where flexural or tensile strength is of special importance). This is a composite effect of its chemical nature, its surface roughness, its particle shape, its absorption, and its cleanliness. As an example of the importance of this feature we can use the author's experience with two different basalts in Melbourne. One of these is superior to the other on every tested feature, it is stronger, has a higher elastic modulus, is denser, has less moisture movement and a higher abrasion resistance. However the other aggregate was better able to produce concrete of average strength over 60 MPa. We assume that this was due to the first aggregate being so dense and impermeable that cement paste had difficulty in bonding to it. It is interesting to note that the subsequent introduction of silica fume appears to have reversed this situation, confirming the effect of silica fume on bond.

The particle shape of the aggregate is influenced by the crushing process. The stone type does have a distinct influence, some stones being more liable to splinter into sharp fragments and/or to produce a larger amount of dust than others. However the crushing process also has a large influence. Cone crushers are perhaps the most efficient and economical type of crusher but they do not produce as good a particle shape as a hammer mill. Other influencing factors are the reduction ratio (a large reduction in a single stage tending to produce a worse shape) and the continuity of feeding (choke feeding giving a better shape).

The effect of a poor particle shape (flaky and elongated) is to require a higher fine aggregate and water content (and therefore a higher cement content) for a given workability and strength. The best measure of this is the Angularity Number, being the percentage voids minus 33. Oddly enough Kaplan's work (Kaplan, 1958) on the subject suggests that the sharpness of the edges and corners tends to make more difference to this parameter than flakiness and elongation.

The question of particle shape must include considering the relative merits of crushed rock and rounded river gravel. Gravels are often reputed to give inferior results, particularly for high strength concrete. There is no denying that this is true for a given water/cement ratio and that it is true generally where tensile or flexural strength is concerned. However in terms of compressive strength, with equal cement content and equal ease of placing (reduced fine aggregate content and reduced slump [= higher yield stress] because the rounded aggregate will have a lower plastic viscosity and so can have a higher yield stress for equivalent workability) then rounded gravel may give as good or better results, depending on the particular use. Fifty years ago, the author made concrete of 85–90 MPa from London area gravel (which is one of the gravels which have been claimed to give inferior results for high strength concrete). Gravels tend to have been adequately tested by the formation process as regards weaker particles and moisture movement susceptibility. However this provides no security against alkali-aggregate reactivity and any coatings on pit gravels in particular should be regarded with suspicion.

The subject of coatings on coarse aggregate is worth consideration. Generally if the coating is removed during the mixing process (and assuming it to be chemically inactive) it is not likely to cause a severe problem. Very fine material will merely add to the water requirement in the same way as fine aggregate silt. This will increase water requirement but, unless excessive, should cause only a small strength depression. However if a coating remains intact after the concrete is in place, a substantial effect on strength and durability can occur through loss of bond. The amount of fine material adhering to coarse aggregate is often substantially affected by the weather, with more material adhering during wet periods. This effect should be considered when looking for causes of strength variations in concrete.

The ideal maximum size for a coarse aggregate has usually been assumed to be 40 mm or 20 mm ($1\frac{1}{2}$ inch or $\frac{3}{4}$ inch) according to the size of section and the reinforcement spacing. Of recent years there has been a worldwide trend to higher concrete strengths and work done many years ago in U.S.A (Blick, 1974) is gradually being rediscovered the hard way in many other places. This work showed that the optimum size of aggregates depended on the required strength level, being smaller for higher strengths. This is provided optimum is defined as that which gives the minimum cement requirement for a given strength. (See Fig. 7.5.)

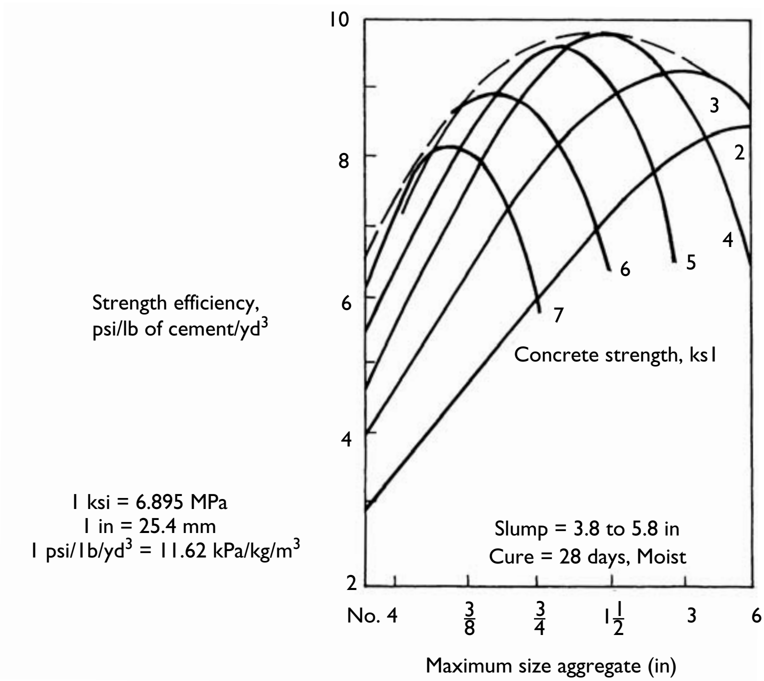


Figure 7.5 Effect of maximum size of aggregate on mix efficiency (Blick, 1974).

If optimum is defined in terms of water/cement ratio or shrinkage or (less certainly) wear resistance, larger sizes may be best. Whilst the optimum size may vary from 40 mm at 20 MPa to 14 or even 10 mm at strengths over 50 MPa, the margin is not usually large and little harm is done by standardizing on 20 mm. The exception to this is where difficulty is experienced in obtaining a high strength, in which case a smaller aggregate should certainly be tried. It is interesting to note that this effect has now been seen to extend further than most would have believed possible. In reactive powder 'concretes' with strengths of several hundred MPa, the coarsest aggregate used is a fine sand.

Another hotly debated question is the relative merit of gap and continuous gradings. A basic difference is in segregation resistance and pumpability. High slump and pump mixes require continuous gradings but low slump, non-pump mixes compact faster with gap-gradings. Two further points worth noting are that single sized aggregates do not segregate in stockpiles and that it is more critical that the exact optimum sand percentage be used in the case of a gap grading than in the case of a continuous grading.

Lightweight aggregates

Many types of lightweight aggregates are in use and a full coverage is beyond the scope of the current volume. However some indication of the possibilities may be of assistance.

Non structural lightweight concrete is not only outside the scope of the book, but also outside the scope of the mix design and QC systems with which the book is mainly concerned. Such concretes are produced either by the use of foaming agents or the introduction of extremely lightweight aggregates such as polystyrene foam or expanded vermiculite. The range of lightweight concretes is a continuous one. It is difficult to say where non structural stops and structural starts. There may indeed be some overlap, with some concretes strong enough to be regarded as structural being lighter than others not having enough strength for structural purposes.

Structural lightweight concrete may be regarded as concrete having a strength at least in excess of 10 MPa and, perhaps more importantly, having a good degree of durability. It should also be capable of bonding to and protecting reinforcement. Such concrete is likely to have a density in the range of 1,200–2,000 kg/m^3. Coarse aggregates used include naturally occurring pumice and scoria (of volcanic origin), cinders from coal burning, and manufactured aggregates produced by bloating clay or shale in rotary kilns similar to (and often formerly used as) cement kilns.

The main difficulty with lightweight aggregates is usually that they have a very high water absorption. Some aggregates, especially those manufactured in kilns, may have a relatively impermeable, sealed surface. Those which are supplied as crushed material, especially the natural materials, may absorb 20% or more of their own weight. Such materials must be used in a fully saturated state if difficulty is to be avoided. If this is not done, water will be absorbed during mixing, transporting, and placing, with consequent rapid loss of workability. A particular difficulty is that of pumping such concrete. Upon coming under pressure in a pump pipeline, water will be forced into any unsaturated aggregate particles. This tends to cause pump blockages through severe slump loss. The problem tends to be most experienced on two or three storey work where an attempt may be made to pump concrete which is not fully saturated. This may be successful for a time but, as soon as any difficulty is experienced, the concrete comes under greater pressure and the problem is greatly intensified. Once the aggregate is fully saturated, such concrete can be pumped just as well as dense aggregate concrete. Indeed, being lighter, it may well be easier to pump to heights of 50 storeys or more.

It is interesting to note that at least one of the Scandinavian floating oil platforms uses lightweight aggregate concrete. What is particularly interesting is that the aggregate is deliberately used dry. The Norwegians admit that this causes the problems outlined above but state that it is necessary in order to achieve the desired low density. On a dry land project, this would be ridiculous because the concrete would eventually have the same moisture content and the same density whether the aggregate was initially wet or dry. The Norwegians say that this is not the case when the concrete is to be permanently immersed in water from a relatively early age.

The use of saturated aggregate has other benefits than improved slump stability. The weight differential between the mortar and the aggregate is reduced and therefore less trouble is experienced with floating aggregates. This differential is also reduced by the use of air entrainment and the air also impedes the movement of water through the mix, so reducing slump loss. The entrapped water makes lightweight concrete a little easier to wet cure, having a built in reservoir of water, but this should not be totally relied upon. The density of the concrete is substantially affected by the moisture content and the weight loss on drying can be as much as 200 kg/m^3 with some concretes. It is also important to note that the crushing strength of the concrete may be substantially reduced by its being fully saturated at the time of test. Unlike dense aggregate concrete, lightweight concrete should not be tested fully saturated unless it will be fully saturated in use.

It is interesting to note that it has been proposed to use a proportion of saturated lightweight aggregate in high strength concrete. The objective is to provide water for hydration in concrete which would otherwise self-desiccate (even if sealed to prevent the loss of any moisture) and so be subject to autogenous shrinkage and incomplete hydration.

Lightweight concrete should not be thought of as necessarily permeable, nondurable, or less capable of protecting steel. Such material has been used to produce concrete ships and found to protect the steel very well over many years. It has been shown to give improved resistance to rain penetration in precast housing.

Strength capacity of different aggregates and different mixes varies considerably, some aggregates can be used to produce concretes of 50 MPa and more, but 40 MPa is a more likely figure.

Shrinkage tends to be somewhat higher, and a higher cement content is usually needed for a given strength. These are probably both for the same reason. This is that lightweight aggregates will usually have a substantially lower elastic modulus, and will therefore tend to shed more stress into the surrounding mortar.

The lighter kinds of lightweight concrete also use lightweight fines, but this depends substantially on the type of lightweight fines available. It is generally quite satisfactory to use any fines produced by a rotary kiln type of process, although a proportion of sand will probably be needed to give a suitable grading. However fines produced by crushing lightweight material are often unsatisfactory. Low density is often a matter of air voids in the aggregate rather than a basic low density material. As the material is crushed finer, more voids are exposed to penetration by the cement paste. There is a tendency to achieve little benefit in lighter concrete and a substantial disadvantage by increasing water requirement. Much structural lightweight concrete uses natural sand as the whole or part of its fine aggregate.

Although a slightly higher fines content may be necessary, structural lightweight concrete is generally amenable to a mix design process similar to that for normal weight concrete. Sometimes it is better to use volume batching for the lightweight material. This would apply where moisture content will vary substantially. However it is generally a matter of using the different SG of the material in a similar design process. The ConAd Mixtune process described in Chapter 3 can be used for structural lightweight concrete. If so used, it is likely to require a 'strength factor' of less than one. The value may be of the order of 0.7–0.9 but there are too many different kinds of such concrete to offer any useful guide. A trial mix will provide a factor that may prove applicable to a range of mixes using the same aggregate.

Blast-furnace slag

The blast-furnace slag used as a concrete aggregate is quite different to the slag ground as cement. It is the same material in the molten state but has substantially different properties as a result of the cooling process. For use as an aggregate, slag must be cooled slowly to allow attainment of a crystalline state. The material is massive, requiring crushing in the same manner as a natural rock. It is also vesicular, usually to a sufficient extent to make it lighter, but not very much lighter, than a natural coarse aggregate (although it can be deliberately foamed, specifically to make a lightweight aggregate). The vesicularity means that care is needed to use the aggregate in a saturated condition if rapid slump loss and lack of pumpability are to be avoided. It also tends to cause a distinct difference in SG (particle density) between different size fractions. Excellent bond tends to be developed owing to both the vesicularity and the chemical composition of the aggregate and particle shape tends to be better than natural aggregates.

Some sources of slag may have a tendency to cause popouts as a result of remnants of crushed limestone deliberately added to provide the desired conditions in the blast-furnace. However this can be avoided if the limestone is added in smaller particle sizes and combustion is very thorough and even. Slag processing companies undertake measures to oxidize any sulphides present to prevent blue spotting. With these possible exceptions, the material tends to be a stable and satisfactory aggregate, even under fire conditions. Drying shrinkage is usually relatively low, perhaps because some chemical reaction takes place at the aggregate surface, causing a slight expansion which partially offsets drying shrinkage.

The author has found that crusher fines produced from a particular slag source, when combined with a local dune sand, make a very satisfactory fine aggregate in terms of strength at a given cement content and workability, even compared to a good, long graded, natural sand. However it should be noted that the granulated slag which can be ground to produce the 'ggbfs' (ground, granulated, blast-furnace slag), although it may look like sand, does not perform well when so used (in the author's experience). This is because it is in a puffed up state like rice bubble cereals and so the grains are weak.

Concrete aggregate from steel slag (by Alex Leshchinsky)

Steel furnace slag is a non-metallic product consisting of calcium silicates and ferrites combined with fused oxides of iron (15–25%), aluminium, calcium, magnesium and manganese. The material, a by-product of steel manufacturing, is produced in a molten condition simultaneously with steel in a basic oxygen furnace. After the air-cooling, the material has a predominantly crystalline structure. Air-cooled steel slag is crushed and screened for the aggregate.

Steel slag aggregate is being used in asphalt and roadbase. In asphalt, replacing natural aggregate with steel slag aggregate brings some advantages, like, improvement in skid resistance, enhancement in durability, etc. However, the demand for steel aggregate is much lower than its output from steel operations. Therefore, usually steel slag aggregate is very cheap. The average world market price for steel slag aggregate is of the order of US$0.5/t.

The surplus of this cheap material has led to attempts to accommodate it concrete. In a paper (Maslehuddin M. *et al.*, 1999), the results of the detailed research of steel slag as concrete aggregate have been presented. The authors of this paper have investigated compressive and flexural strength, water absorption, drying shrinkage and other properties of concrete. Steel slag aggregate used in the experiments contained clay lumps and friable particles in the range of 0.07–0.31%. Concrete with coarse aggregate from steel slag has been assessed against concrete with limestone aggregate. On the basis of the results of the study, its authors have made a conclusion that steel slag aggregate can be beneficially utilized in portland cement concrete.

With regard to these conclusions, it should be pointed out that steel slag aggregate should not be used as aggregate in concrete, due to

- The possible durability problems caused by the lime expansion
- The aesthetic problems associated with the rust on the surfaces.

Steel slag aggregate is a very abrasive material and will result in substantial wearing of plant equipment (conveyer belts and bins) as well as agitators. Due to the high density of steel slag (an apparent particle density of the order of 3.3 t/m^3), concrete density will go up reducing the maximum size of a concrete load. For instance, concrete with 1 t/m^3 of crushed river gravel (an apparent particle density of 2.65 t/m^3) has a density of 2.44 t/m^3 and is delivered in maximum size loads of 6 m^3. If crushed river gravel is replaced with steel slag aggregate, the maximum load size will be only 5.45 m^3, which will increase concrete transportation cost.

Suggested further reading

Leshchinsky A. (2004), Slag sand in ready-mixed concrete, *CONCRETE*, Vol. 38, No. 3, March, pp. 38–39.

Maslehuddin M., Shameem M., Ibrahim M. and Khan N. U. (1999), Performance of steel aggregate concretes, Exploiting Waste in Concrete, *Proceedings of International Conference 'Creating with Concrete*', Dundee, Scotland, UK, Thomas Telford Ltd, pp. 109–119.

Chapter 8

Cementitious and pozzolanic materials

8.1 Portland cement

Introduction

No attempt is made in this book to provide a general background and description of Portland cement. Such information is available in almost any textbook on concrete, as well as many specialized books on cement. A particularly recommended reference is the ACI 'Guide to the Selection and Use of Hydraulic Cements' (ACI 225, 1985). This is a very comprehensive 29 page dissertation with an equally comprehensive list of further references. Another useful reference is High Performance Concrete (Aitcin, 1998) which provides substantial detail on cement, and also on cementitious materials and admixtures.

Some guidance has been provided in Chapters 1 and 6 to the selection of different types of cement for different purposes. What is attempted in the current section is a guide to the extent to which changes in concrete properties may be due to changes in the cement in use.

What can go wrong with cement?

(A) As the user experiences it:

1. Setting – it can set too quickly or too slowly.
2. Strength development – it can develop less strength than usual.
3. Water requirement and workability – it can have a higher water requirement or act as a less suitable lubricant than usual.
4. Bleeding – it can inhibit bleeding less successfully or at the other extreme produce a 'stickier' mix than usual.
5. Disruptive expansion.
6. Reduced chemical resistance.
7. Too rapid evolution of heat.
8. Deterioration in storage (either before of after grinding).
9. It can arrive hot that is, hotter than usual.

10 It can be delivered from the same depot, and even ground at the same plant, but be produced from a different clinker. that is, imported clinker using different materials and produced in a different kiln may have been used.

(B) As it is produced:

1 Variation in raw materials.
2 Segregation at any of several stages.
3 Incorrect proportion or uneven distribution of gypsum ($CaSO_4 \cdot 2H_2O$).
4 Variable firing and grinding temperatures.
5 Unsatisfactory grinding – including overall fineness, particle size distribution and particle shape.
6 Deterioration (including segregation) of clinker in storage.
7 Seasonal variations.

Significant test results

Cement users in some parts of the world can obtain test certificates from their cement suppliers. The following may be of assistance in interpreting the kind of information usually provided on such certificates. Where no test data are obtained in this way, it may be considered too expensive to undertake routine testing on behalf of a single project or small readymix plant. A solution to this problem is to take a sample either daily, or from each truck of cement (whichever is least). The sample should be kept in a (well labelled!) sealed container until the 28 day concrete test results are obtained and then discarded. A sample is then available, and should be tested if unsatisfactory concrete test results are encountered for which no other explanation can be found.

Where regular test data are obtained, it is useful to maintain graphs of the information provided. As with concrete test data, cusum graphs are far more effective at detecting change points (see Section 4.3).

The main results likely to be provided are:

1 *Setting time.* Initial and final set are both arbitrary stages in smooth curve of strength development.

Abnormal results can indicate incorrect proportion of gypsum, excessive temperature in final grinding (which dehydrates gypsum and alters its effectiveness) or deterioration with age.

2 *Fineness*. Finer cement will:

i React more quickly (faster heat generation)
ii React more completely
iii Improve mix cohesion (or make 'sticky')
iv Reduce bleeding
v Deteriorate more quickly

vi Be more susceptible to cracking

vii Generally require more water (note that this may be less due to any direct effect of fineness than to the reduced range of particle sizes normally resulting from finer grinding).

3 *Soundness* (Pat, Le Chatelier and autoclave tests). Intended to detect excessive free lime (perhaps due to incomplete blending rather than wrong chemical proportions). Some experts disagree that the intention is achieved, but this is beyond the present scope. Magnesia can also cause unsoundness (if as periclase) but perhaps too slowly for pat or Le Chatelier – needs autoclave or chemical limit (and see Section 5.5 for intentional use of a proportion of magnesia).

4 *Normal consistency*. Generally just a starting point for other tests but can show up undesirable grinding characteristics. Where very high strength concrete is involved, large amounts of cement will be required and a very low w/c ratio will be sought. A cement with a high water requirement is at a disadvantage in such circumstances. Interesting uses for this test are as a compatibility check between admixtures and cement or to determine the effect on water requirement of a percentage of fly-ash or silica fume etc.

5 *Loss on ignition*. Mainly a check on deterioration in storage. The test drives off any moisture or carbon dioxide which may have been absorbed. A 3% loss on ignition could mean a 20% strength loss. However up to 5% of limestone ($CaCO_3$) is permitted to be added to cement and this test would drive off CO_2 from limestone.

6 *Sulphuric anhydride (SO_3)*. Check on proportion of gypsum, has considerable significance for setting time, strength development and shrinkage. The test determines the content of SO_3 from all sources (e.g. added gypsum, oxidized sulphur in fuels etc.) and in all states. It therefore may not be an accurate guide to the amount of *active* (soluble) SO_3 present. It is the amount of *active* SO_3 which affects setting time, rate of strength development, tendency to shrinkage and cracking etc.

7 *Insoluble residue*. Check on impurities or non reactive content only, the effect is the same as reducing the cement content by the percentage of the insoluble material. However this test may characterize fly-ash as insoluble residue.

8 *Compressive strength*. This should be directly related to concrete performance but there can be differences with admixture interactions, different water cement ratio etc. In some countries cement is sold as being a particular strength grade. Generally higher strength grades are more expensive but less can be used to meet a strength specification. The selection of a high strength cement becomes important when very high strength concrete is required, since an increase in cement content will not give a strength increase beyond a certain point.

It is very desirable for readymix producers in particular to develop a good working relationship with their cement supplier. A variation free product cannot be expected, but honesty in reporting current test results, and help in interpreting and compensating for their likely effects on concrete, and cooperation in tracking down any problems is valuable. This kind of cooperation is unlikely if all concrete problems are automatically blamed on the cement, and the concrete producer fails to carry out, and keep proper records of, control tests on concrete.

An important, if relatively rare, occurrence is an unfavourable interaction between the cement and admixtures in use. Examples have been encountered where a particular cement and admixture, both satisfactory with other admixtures and cements, have given trouble in combination. In a recent example the trouble was a false set. A false set is one that occurs for a limited time and can be overcome by continued mixing. This may give no trouble when held in a truck mixer until directly discharged into place but cause a severe loss of pumpability if discharged into a pump hopper during, or prior to, its occurrence. If suspected, such an occurrence can be investigated using a Proctor Needle penetrometer on mortar sieved from the concrete to construct time v penetration resistance curves.

A particularly delicate question is that of the lower value of cement that provides a lower strength. It is of very substantial assistance to a concrete producer if he can rely upon the cement producer advising him of a strength downturn. This enables the concrete producer to increase his cement content and avoid low test results. However, since the cement producer is responsible for the need for the additional cement, there is a natural tendency for the concrete producer to feel that the cement producer should bear the additional cost. It will obviously not encourage the cement producer to provide the early warning if the result is a deduction from his invoice.

The reverse kind of assistance is also valuable. Cement suppliers tend to receive unjustified complaints from customers who have inadequate control systems. It is of value to them to find a regular user who has a good control system so that they can rely on feedback data.

In summary, the development of a good relationship and an effective early warning system with your supplier can be of considerable benefit, and your own good control system is a necessary starting point for such a relationship.

Types of cement

Cement chemistry is extremely involved and not within the scope of the current work, however limited comment on the different types of cement commonly available may be useful. All Portland cement is conveniently regarded as composed of four compounds:

C_2S	Di-calcium silicate	Slow acting, low heat generation, best long term strength and durability.
C_3S	Tri-calcium silicate	Quicker acting, more heat generated, still good strength and durability but not as good as C_2S.

C_3A	Tri-calcium aluminate	Very rapid reaction, high heat generation, responsible for early (but not high) strength and setting, easily attacked by chemicals.
C_4AF	Tetra-calcium alumino-ferrite	Relatively little influence on properties of concrete (except colour), present because needed during manufacture.

The relative amounts of these compounds are varied to produce different types of cement to suit different uses:

Type I – also known as Type A, OPC (ordinary Portland cement), GP (general purpose)
Type II – modified low heat cement
Type III – high early strength or rapid hardening
Type IV – sulphate resisting cement
Type V – low heat cement.

A fifth compound, $CaSO_4$ (gypsum) is interground with the cement clinker to control setting.

It is also thought to have a substantial beneficial influence on shrinkage and to produce improved strength. However an excess can cause slow setting and also unsoundness (destructive expansion). Gypsum can be rendered less effective by excessive heat during grinding.

The reader will be able to work out from the aforementioned, or consult other sources, which compounds will predominate in which cements. However there are a few matters that are often misunderstood and so should be brought to the readers attention:

1 Sulphate resisting cement is made so principally by limiting the amount of C_3A. Unfortunately C_3A, whilst of general low durability, happens to be the compound most of use in combating the penetration of chlorides. Too often this cement is assumed to be a general high durability cement and used where chloride resistance is as important, or even more important, than sulphate resistance (e.g. in marine structures). What should be used in these circumstances is blast-furnace cement, fly-ash substitution, or silica fume incorporation. Where none of these are available, a higher strength grade of OPC concrete should be used.
2 Low heat cement is generally as sulphate resisting as sulphate resisting cement (since C_3A is also limited to reduce heat generation), however sulphate resisting cement is not necessarily low heat generating. This is because most of the heat generation comes from the C_3S component (of which there is always much more than the C_3A) and the proportion of this is not necessarily limited in sulphate resisting cement.

It is now coming to be recognized that suitability for different purposes is often better attained by the use of variable proportions of fly-ash, blast-furnace slag or silica fume than by the use of different types of cement. These alternative materials, being essentially waste products, used to be thought of as inferior substitutes for cement, used only to reduce cost. It is typical of the reaction of concrete specifiers to new developments that they were often prohibited or strictly limited in proportion.

An interesting justification of fly-ash is used on occasions. Faced with a statement that it is a new-fangled, unproven material, it is reasonable to point out that the use of volcanic ash by the Romans has shown such material to be good for 2,000 years if correctly handled, whereas Portland cement has yet to show it can last 200 years (and much already has not done so).

8.2 Fly-ash (pfa)

General characteristics

Fly-ash, otherwise known as pulverized fuel ash (pfa), is a pozzolanic material. This means essentially that it is capable of combining with lime (in a suitably reactive form) in the presence of water, to form cementitious compounds. As lime is liberated in substantial quantities when normal cement reacts with water, and is present as reactive calcium hydroxide, there is a distinct attraction in adding pfa to concrete.

Fly-ash looks like cement to the naked eye, but will not set at all (unless a Class C ash, which is a type of ash that contains substantial calcareous material) when mixed with water. It is usually even finer than cement, has a very rounded particle shape, including some partly broken hollow spheres known as a cenospheres (as opposed to the extremely jagged particle shape of cement) and is of lower density (SG usually 1.9 to 2.4 compared to 3.15 for cement).

Fly-ash has a varying 'pozzolanicity' that is, some fly ashes give much better strength than others. No fly-ash is as good as cement on a volume for volume substitution basis and but some fly ashes are as good as cement in terms of 28 day strength and better at later ages when substituted on a mass for mass basis and when account is taken of their water-reducing action as well as their strength production at a given w/c ratio.

There are few materials which do not have some drawbacks and with fly-ash substitution these include:

1 Reduced early strength.
2 Increased setting time.
3 Reduced heat generation (which is an advantage in hot weather, or for mass concrete, but a disadvantage in cold).
4 Inhibition of air entrainment, if of high carbon content (easily corrected by higher dosage or specially formulated products for use with fly-ash, but may give rise to higher variability if carbon content varies).
5 Added complication – one more factor requiring knowledge and skill to give best results.

Fly-ash concrete does not automatically display all the advantages (or disadvantages) of which it is capable. Crude substitution of fly-ash for cement can yield better or worse concrete depending on the circumstances and requirements. It could be said that fly-ash puts another useful tool in the hands of competent technologists and presents another trip-wire for the uninitiated to fall over. Also there are considerable differences between different fly ashes and there is not an automatic ‘best buy’ for all circumstances. There are examples of troubles exacerbated if not caused by fly-ash and, on the other hand, of the use of fly-ash not being permitted through ignorance or blind prejudice in circumstances where it would have been highly desirable.

The composition of fly-ash

There are two types of fly-ash, according to the classification in ASTM 618, Class F and Class C. Class F ash is the true pozzolanic material, silica (as SiO_2) being the most important constituent, and alumina and iron oxide are also active (see Table 8.1). Class C ash also contains appreciable amounts of calcium compounds and may have some minor cementitious value in the absence of cement (a very few sources may produce usable concrete without any cement at all). Certainly it is possible to use it in larger proportion than Class F ash in a similar manner to, but not to the same extent as, a blast-furnace slag. Class C ash may be less effective than Class F ash in providing sulphate resistance.

The author’s experience is with Class F ash. Class C ash may in general produce similar effects but (as noted in the section on mix design competitions) substantial differences are possible.

Carbon is the most important impurity as it can inhibit the action of admixtures, particularly air entraining admixtures. It is measured by loss on ignition which should not exceed 8% and should preferably be very much less. However the really important requirement is that it should be as consistent as possible since otherwise it may be very difficult to control air content. However, there has been a report (see Section 8.5) of rice hull ash containing up to 23% of carbon being successfully used in particular circumstances, so possibly higher percentages in fly-ash would not necessarily render it useless in all circumstances.

Table 8.1 Typical chemical composition of cementitious and pozzolanic materials

	Portland cement	*Fly-ash*	*Slag*	*Silica fume*
SiO_2	20	50	35	93
Al_2O_3	5	30	15	2
Fe_2O_3	4	10	1.5	<1
CaO	65	2.5	40	<1
MgO	<2	<2	7	<1
Na_2O	<2	<2	<1	<1
K_2O	<2	<2	<1	<1
SO_3	<4	<2	<1	<1
LOI	<2	<2	—	<2

Other impurities are alkalies and magnesium which need to be limited as in cement but are not often a problem.

The effects of fly-ash

There are three kinds of effect from the incorporation of fly-ash in concrete. These are:

(a) Physical effects on both fresh and hardened concrete
(b) Chemical effects on setting process and hardened concrete
(c) Physical chemistry (or surface chemistry) effects on setting process.

(a) Physical effects

The fly-ash particles are very similar in size and shape to entrained air bubbles and have many very similar effects namely

1 Water reduction. Perhaps of the order of 5% but varies with different ashes. A very few ashes (e.g.. some Hong Kong ash) slightly increase water requirement.
2 Reduction of bleeding.
3 Improved cohesion and plasticity.
4 Improved pumpability.
5 Reduced slump loss with time.

Fly-ash is not compressible, and probably does not help frost resistance at all (and tends to inhibit air entrainment so that a larger dose of AEA is needed). However, this property (incompressibility) makes fly-ash even more valuable than entrained air for pumpability. Also fly-ash has the benefit that it is present as a clearly defined quantity.

Being so fine, the pfa particles are very valuable as pore-blockers, substantially reducing permeability in the hardened concrete.

(b) Chemical effects

When cement hydrates, it releases free lime. This lime is the softest, weakest and most chemical attack and leaching susceptible of all the constituents of concrete.

The fly-ash combines chemically with the free lime to form compounds similar to those produced by the rest of the cement. This reaction is quite slow (7 days before it produces much effect), and generates little heat during the setting process. This is generally a valuable property in hot climates and for mass concrete, but may be a distinct disadvantage in colder climates.

Fly-ash is effectively reactive silica – the very material causing problems in coarse aggregates through alkali-aggregate reaction. Actually this is a valuable feature since there is so much reactive silica that all alkali is used up during an initial reaction, leaving none to cause problems later, however reactive the coarse aggregate.

(c) Surface chemistry effects

It appears that fly-ash can act as a catalyst or a starting point for crystal growth in the cement paste. Such effects are beyond the scope of this book but it should be realised that there is more to the story than has been told earlier. This may provide some explanation for a smaller early age strength reduction than chemical effects alone would predict when equal mass substitutions are made.

Dr Malcolm Dunstan (in the UK) and Mohan Malhotra in Canada (Malhotra and Ramezanianpour, 1994) have done interesting work on roller compacted and other concrete with 50–60% of fly-ash substitution. A very revealing point is that good results are obtained with high fly-ash in either earth dry concrete (roller compacted) or concrete with a normal slump attained through using a superplasticiser. However poor results are obtained with high fly-ash at normal water contents. It could be said that the w/c v strength relationship is even more marked in the case of fly-ash than in the case of cement.

Dangers to avoid with fly-ash

1 Since fly-ash is lighter (and cheaper) than cement it might be thought that it would be especially useful in low strength concrete. In fact it does produce much better looking, more segregation resistance and less bleeding prone concrete for a given (relatively high) water to cementitious ratio. However this is sometimes its undoing. Uninformed or thoughtless people tend to over water it to a greater extent than plain concrete, yet in fact its strength is *more* affected by a given amount of excess water. Thus fly-ash should be used with care and conservatism for low strength requirements. Properly used it is valuable for such uses but is less resistant to over-watering abuse.

2 Because strengths take longer to develop, more efficient and prolonged curing is necessary for fly-ash concrete. It is true that fly-ash concrete is substantially less permeable than plain concrete of similar strength, and therefore may be to some extent ‘self-curing’ in larger masses (and especially for below ground or on ground foundations). However, this does not help the outside 20 mm of exposed concrete, which has to protect reinforcement. Fly-ash concrete reacts extremely well to steam curing.

3 The same calcium hydroxide that has the disadvantages of being soft, weak and easily dissolved by water or chemicals, is the source of the alkalinity which protects steel from corrosion. Therefore, by combining with it, fly-ash reduces the chemical protection available for the reinforcing steel. The question is whether or not this is compensated for by the reduced impermeability of the fly-ash concrete. The answer lies in the curing, yes if well cured, no if not well cured.

4 Because fly-ash concrete gains strength more slowly, it is susceptible to creep if depropped (beams and slabs) too early. The need to prop longer may be an additional cost.
5 Due to reduced bleeding tendency, evaporation cracking will occur slightly more readily (but the tendency to thermal cracking is reduced).
6 Readiness for trowelling will be delayed – perhaps very significantly delayed in cold weather.

Advantages of fly-ash

1 Reduced heat of hydration in the critical period.[1] In the author's opinion the temperature rise in mass concrete is almost the same as if only the cement and no pfa were present. However not everyone shares this opinion so you should conduct trials before implementing it.
2 More readily workable fresh concrete – easier to pump, compact, trowel, less bleeding and segregation, better off-form surface usually.
3 Substantially more impermeable concrete (if adequately cured).
4 More durable concrete, for example, more resistant to sulphate attack than most sulphate resisting cements (Kalousek, 1972).
5 Higher strengths possible – adding fly-ash is distinctly better than using cement contents in excess of 400/450 kg/m^3 in most cases. (However higher cement contents can be used if the cement is low heat).
6 More economical than straight cement in most parts of the world.
7 Fly-ash is particularly useful in marine structures (where curing time is available before inundation) as otherwise there is the conflict of requiring high C_3A to resist chlorides and low C_3A to resist sulphates whereas fly-ash concrete resists both.

Summary

The use of a proportion of fly-ash is generally desirable except where high early strength is required, heat generation is advantageous or, especially with strength grades below 30 MPa, adequate curing is uncertain and corrosion protection of reinforcement is required. Where fly-ash is used, care must be taken to ensure that reported strengths are realistic and not the result of assuming that water cured cylinders necessarily correctly represent poorly cured in situ concrete.

The circumstances in which it may be worthwhile specifying that fly-ash be used would include hot weather concreting, large sections where low heat cement or ice might otherwise be needed, projects in which exceptionally high strength or good pumpability is needed and projects where high sulphate resistance is needed.

1 This is the period during which heat is being generated faster than it is being dissipated and the temperature of the mass is therefore rising.

8.3 Blast-furnace slag

Properties of granulated, ground, blast-furnace slag (ggbfs)

The properties of cementitious and pozzolanic materials depend on their *chemical composition*, their *physical state* and their *fineness*. This is particularly the case with blast-furnace slag. Since it is a by-product of the production of iron, its composition may differ from different sources but is likely to be reasonably consistent from a given source. Table 8.1 shows its composition to be more similar to that of cement than to typical pozzolanic materials. However to develop satisfactory properties it is essential that the molten slag be rapidly chilled (by quenching with water) as it leaves the furnace. This causes the slag to *granulate*, that is, break up into sand sized particles. More importantly it causes the slag to be in a glassy or amorphous state in which it is much more reactive than if allowed to develop a crystalline state by slow cooling. In the latter state it is highly suitable as a concrete aggregate but not as a cementitious material. It is important to note that the unground granulated material does not make a good fine aggregate because often the grains are weak, fluffy conglomerates rather than solid particles.

To use as a cementitious material, the granulated slag must be ground as fine or finer than cement. The fineness of grind will (along with the chemical composition and extent of glassiness) determine how rapidly the slag will react in concrete.

Slag cannot be used alone to make concrete but can be used in much larger proportion than pozzolanic materials. Portland cement clinker or some other activator is required to initiate the hydration of the slag. The latter may form 80% or more of the total cementitious material but 60% or less is more usual. An alternative activator is calcium sulphate, producing a product known as ‘supersulphated cement’. This cement is beyond the scope of the present volume but those encountering it should note that, whilst it offers valuable properties of chemical resistance and very low heat generation, it requires special care and understanding in use to offset its slow setting and strength development and needs very thorough extended curing.

In Portland blast-furnace cement, the slag may be interground with the cement clinker or added as a separate material. The cement clinker is softer than the slag and therefore will be ground to extreme fineness when the materials are interground. Even when sold as a composite ‘blended cement’ (which term is also applied to fly-ash blends) the ggbfs cement may have been either interground or post-blended.

Properties of ggbfs concrete

Concrete using ggbfs cement will develop early strength more slowly than Portland cement concrete. However, if thoroughly cured, it may have as good or better eventual strength. It normally has a greater resistance to chemical attack, and is particularly suitable for marine works. Its normally greater fineness confers resistance to bleeding in the fresh state and lower permeability when hardened.

The glassy surface of the slag may give a slightly reduced water requirement even though it does not have the favourable particle shape of fly-ash. The water requirement may however be substantially dependent on the fineness of grind.

It can be added as a separate ingredient at the mixer but is more normally sold interground with cement. There is a long history of extensive use in this form as Portland blast-furnace cement, particularly in Europe and the former Soviet Union. The proportion of slag can exceed 80% of such cement.

To some extent this product is sometimes seen as a low grade cement, since it develops strength more slowly and sometimes has a lower strength at 28 days. However, it usually exhibits better resistance to chemical attack and is noted as particularly suitable for marine works. Obviously the properties of such a material will be very dependent upon the composition of the particular slag. Since ggbfs is a by-product material, there may be a wide variation in quality between cements from different sources. The author has had personal experience of only two sources of slag and the works of local authors should be consulted.

When used in lower proportion, the resulting material is described as a 'blended cement' and this term is applied equally to blends of Portland cement with fly-ash. Whilst such cement may be marginally cheaper, and will almost certainly gain strength more slowly, it is by no means necessarily inferior.

Heat generation

It is important to fully appreciate the situation with heat generation. There are three aspects to consider. These are cold weather concreting, hot weather concreting and mass concrete.

Because it can be used in large proportion, ggbfs can give rise to problems with slow setting, slow strength gain and lack of early resistance to frost in cold weather. These same properties can be very advantageous in hot weather. The assumption may be made that the slag cement will provide reduced peak temperatures in mass concrete as does fly-ash concrete. In fact unless a very high proportion of ggbfs (over 75%) or a very coarse grind is used, the cement can give rise to even higher temperatures than with normal Portland cement. This is because, marginally and with some slags, even more total heat can be generated and the slower generation may or may not give a better result depending on whether the heat can be dissipated. It should be clearly understood that there is no question that slag cement generates heat more slowly and so produces distinctly lower peak temperatures in most applications. It is only in situations that are effectively adiabatic (such as foundation rafts more than a metre thick), that slag concrete may not provide the anticipated benefit. It is certainly particularly useful for general use in hot climates.

Blue spotting

Ggbfs concrete is notorious for the development of discoloured patches, known as ‘blue spotting’. This is caused by the initial formation of ferrous salts. These oxidize to colourless ferric salts on drying but can be a problem in continuously damp conditions or where a transparent sealer has been applied.

Ternary blends

Ternary (i.e. triple) blends of ggbfs, fly-ash and cement are sometimes used and have a good reputation. The addition of different proportions of fly-ash during batching can give a flexibility of properties to a fixed blend of ggbfs and cement.

8.4 Silica fume

Silica fume is a relatively new and very powerful tool at the disposal of the concrete technologist. As with other such tools, the material has to be understood and correctly used if full benefit is to be obtained and deleterious side effects avoided. Being relatively expensive, it is should, in the author’s opinion, be used in proportions of no more than 5–10% of the cement content of a mix. However some specifications call for as much as 15% to be used and this is not deleterious – just expensive.

The material (also known as micro-silica) is a by-product of the manufacture of silicon, ferrosilicon, or the like, from quartz and carbon in electric arc furnaces. It is usually more than 90% pure silicon dioxide and is a superfine material with a particle size of the order of 0.1 micron and a surface area of over 15,000 m^2/kg (i.e. a hundred times greater than cement or fly-ash). Its relative density is similar to that of fly-ash at about 2.3 but, owing to its extreme fineness, it has a very low bulk density of only 200–250 kg/m^3 in its loose form. For this reason it is usually handled either in a densified form or as a 50/50 slurry with either water or a superplasticising admixture. In the densified form, particles are deliberately induced to flocculate into clumps which are still as fine or finer than cement particles.

There is disagreement as to whether use of silica fume increases water content or not. This may depend on the particular material but certainly also depends on how it is used. To be fully effective it must be dispersed so that it occupies spaces between cement grains and must not remain in clumps of fume particles. It seems doubtful that this is achievable without the use of a superplasticiser and, in the author’s opinion, it should not be used without a superplasticiser. A possible exception may be for shotcrete but even for this purpose the author insists on using a superplasticiser. It may be that, used with a superplasticiser, silica fume does not increase and may even reduce water content at a given superplasticiser dosage. It may also be that if any substantial increase in water requirement results, much of the potential value of the fume will be lost (especially for high strength concrete).

There is a tendency for silica fume to be regarded as only justified for very high strength concrete but this is far from the truth. Its uses are many and varied. It can provide unprecedented reductions in permeability and increased durability and its effects on the properties of fresh concrete are more important for many uses than its effect on hardened properties. These effects include a very substantial increase in cohesion and an almost complete suppression of bleeding or any other form of water movement through concrete (in either the fresh or hardened state). Whilst the suppression of bleeding is desirable in many ways, it does cause exposed flat surfaces of fresh concrete to be very susceptible to evaporation cracking.

Some of the main applications of silica fume in concrete are:

1 *High strengths*. The actual strength level attainable is dependent upon other factors (notably coarse aggregate characteristics) but in many instances silica fume permits the easy attainment of strengths in excess of 100 MPa when, for highly workable concrete, 80 MPa might be difficult to attain without it.

   The action of the fume appears to be partly chemical and partly physical. It is both superfine and in a highly reactive form. Its pozzolanic reaction with the free calcium hydroxide released by hydrating cement is therefore very effective. The author has described it as being 'like fly-ash squared', that is, fly-ash with a second order of effectiveness, for this and other properties.

   The physical effect of densification, and of improving the structure of the cement paste at its interface with the coarse aggregate, has been considered to be of similar magnitude to the chemical effect.

2 *Durability*. Silica fume concrete provides a previously unattainable level of low permeability in addition to the chemical conversion of the most vulnerable calcium hydroxide into durable calcium silicates. It gives a physical uniformity of cement paste structure through avoiding bleeding effects and creating a smaller scale gel structure. Thermal stresses are reduced compared to attempting to improve durability by increased cement content.

   Any tendency of the coarse aggregate to alkali-silicate reaction will be forestalled since the alkalis will be consumed in a non-deleterious diffused reaction with the silica fume.

   The combined effect of these factors is to provide a new degree of resistance to sulphates, chlorides and general aggressive chemicals. Two aspects which are not necessarily greatly improved by silica fume addition are carbonation and resistance to freezing and thawing deterioration. In the case of carbonation, the consumption of the free calcium hydroxide in the pozzolanic reaction counteracts the beneficial effect of the reduced permeability. However silica fume concrete has lower electrical conductivity (Vennesland), which will assist in providing greater resistance to steel corrosion.

   Resistance to deterioration by freezing and thawing poses an interesting question for high strength concrete in general. There is no question either that entrained air still provides greater resistance to freezing and thawing of saturated concrete or that it makes high strength much more difficult and expensive to attain. The question, especially with silica fume concrete, is whether laboratory tests using saturated concrete are realistic. If the concrete is not saturated, there may be no water to freeze and cause damage. A different answer to this question may be appropriate in an exposed high strength column and in a bridge deck.

3 *Cohesion and resistance to bleeding*. These properties certainly make silica fume a most desirable ingredient of pumped concrete (and also of self-compacting concrete). A particularly severe test of pumpability occurs in stop-start situations. Many mixes pump satisfactorily on a continuous basis but fail to restart after a delay. The usual cause of this effect is internal bleeding. There is no better cure for this problem than silica fume. Using silica fume and a high solids superplasticiser enabled single-stage pumping of concrete to the top of Petronas Towers, formerly the world's tallest building.

Resistance to bleeding also means resistance to bleeding settlement. An important future technique for very high strength columns is to fill steel pipes from the base with fluid, self-compacting concrete. The author has experienced this technique in four storey lifts but there may be almost no limit to the height attainable from the viewpoint of the concrete. Such columns often involve penetrations by other steelwork at each floor level. In these circumstances any bleeding settlement would be disastrous in causing cracking at vital locations.

Tremie concrete, and particularly any concrete which has to resist free falling through water, also benefits from the incorporation of silica fume, although other thickening agents such as methyl cellulose are also used.

4 *Shotcrete*. Silica fume concrete can transform the economics of shotcreting and greatly improve repair performance by its ability to reduce rebound and improve adherence to the substrate in both the fresh and hardened state.

5 *Surface finish*. The inhibition of water movement through the mix is very beneficial for surface appearance. Effects such as hydration staining, sand streaks, bleeding voids on re-entrant surfaces and settlement cracking are avoided.

A possible problem is that the properties of the particular silica fume can cause a substantial effect on colour. This is due to any carbon content and is apparently more influenced by the *size* of the carbon particles than by their percentage by weight.

8.5 Rice hull ash (RHA)

This material is produced by burning rice hulls (i.e. husks or shells) which invariably contain a large proportion of silica. It has similarities with silica fume and with blast-furnace cement. Chemically it is like silica fume in being almost pure silica. Its similarity to slag is that the conditions of production are very important. As slag must be cooled very rapidly to achieve a glassy or amorphous state (glassy *is* amorphous as opposed to crystalline, they are not alternatives) so RHA must be burnt at a relatively low temperature to achieve that state. Burning at too high a temperature gives essentially a very fine, but not reactive, silica sand. However, it is essential that the burning should be complete or the ash will have a high carbon content, which is anathema to the uniform and effective performance of admixtures. However, there has been a report (Dalhuisen *et al.*, 1996) of ash with up to 23% of carbon being used successfully. This was in tropical conditions where air entrainment was not required.

Like slag, the particles are initially 'fluffy'. They are much larger than silica fume particles and yet have a higher surface area. It is necessary, and relatively easy, to grind such particles to avoid excessive water demand and resistance to compaction. With such a material, it is clearly important to evaluate product from a particular source for performance and uniformity since it can range from being as valuable as (and similar to) silica fume to being as deleterious as silt when incorporated in concrete.

There are substantial quantities (tens of thousands of tons) of rice hulls available annually in many parts of the world. They constitute a potentially valuable resource if suitably prepared, rather than being a large scale nuisance even after burning indiscriminately to reduce volume.

8.6 Superfine fly-ash

In some parts of the world a superfine grade of fly-ash is available which can be regarded as midway between normal fly-ash and silica fume in cost, effectiveness, and desirable dose rate. The material can be highly competitive depending on relative costs and availability. It neither requires such large volume batching facilities as normal fly-ash nor is as difficult a material to handle and disperse effectively as silica fume.

8.7 Colloidal silica

A French development is of silica chemically produced in a colloidal form rather than resulting as a by-product from ferrosilicon production. The material is even finer than silica fume but, being in a liquid suspension, does not present the same handling difficulties. It is more expensive, but used at a lower dose rate than silica fume. It is claimed to be particularly effective and economical for shotcreting (Prat, 1996).

8.8 Metakaolin

Metakaolin is a relatively new entrant to the pozzolan for concrete field. It is produced by calcining Kaolin, otherwise known as the China Clay used for ceramics. As with rice hull ash, it is important that it be fully calcined but that the temperature does not much exceed 800°C as this would cause the formation of 'dead burnt', non-reactive mullite. The material is an aluminosilicate that reacts with free lime in a similar manner to silica fume and producing similar benefits when used in similar proportions of 5–15%.

Proponents point to the fact that it is a purpose-made controlled product whereas most pozzolans are by-products or waste materials. Being essentially a white pigment, it produces concrete of a lighter shade. Since it also reduces efflorescence, it is particularly suitable for coloured concrete.

8.9 Superfine calcium carbonate (pure limestone)

This is another recent introduction. The author does not have personal experience of it but hears reports from several countries of its successful use. It is usually available in varying degrees of fineness, with the superfine material being distinctly more expensive. It has been used as up to 5% of OPC for many years, being seen as essentially a diluent and cost-saver. A coarser grade is used as to some extent a substitute for fly-ash and the superfine grade (5 micron) as a partial replacement for silica fume. Since the material is simply calcium carbonate, it is difficult to see any chemical basis for its beneficial effects, reported to include improved workability and, more surprisingly, higher very early strength. The assumption is that better particle packing is at least part of the explanation.

Chapter 9

Chemical admixtures

The days when it was defensible to take the attitude that admixtures are an unnecessary complication passed in the 1950s. It is now quite clear that admixtures can both solve otherwise intractable technical problems and save substantial cost. They also have the potential to create technical problems if improperly selected or used.

High strength (HS) or high performance concrete (HPC), especially self-compacting concrete (SCC) is a current hot topic (although its ranking in terms of production volume is nothing like its ranking in volume of technical literature). In presenting the theme report on production of HSC/HPC at BHP96, the Paris symposium (Day, 1996) the author remarked that, of the more than 20 submitted papers included in his report, only one specifically dealt with a superplasticiser but all the concrete covered by the reports contained superplasticiser. There may be a temptation to think that the use of silica fume, or high strength, is the outstanding characteristic of high performance concrete but probably its most basic and essential feature is the use of a superplasticiser (now more usually described as an HRWR, High Range Water Reducer).

The technology of admixtures is both extensive and virtually a foreign language to many in the concrete industry and related professions. It is easy to provide more detail than can reasonably be absorbed and retained by such persons. This chapter is therefore aimed at providing guidance rather than at providing detailed knowledge. What is new is that the situation has now become so complex that even the technical representatives of major admixture suppliers do not have all the answers. As set out on his website (www.kenday.id.au) the author has recently experienced a situation in which his mix submitted as a competition entry actually completely failed to set at all. The cause was a complex interaction of the admixture, the particular cement, and a large proportion of Type C Fly-Ash. The effect was predictable by the most senior researcher of the admixture supplier but unknown to quite senior company technical representatives in both Australia and USA – and the product is described on the web as 'especially suitable for use with fly-ash mixes' without any warning as to type and proportion of the latter. The admixture in question was Grace WRDA but it is emphasized that this admixture is a very normal lignosulphonate that has been in wide use in many countries for many years. It seems that the same effect might have occurred with other similar competing products. The point in relating this incident is that, until its occurrence, the author has for many years been happy to design concrete mixes ‘over the telephone’ in many countries, and recommend that the first trial mix be a full size delivery to the actual structure, without encountering any problem. He has also recommended readers to find and rely on the technical representative of a reputable admixture supplier. Now clearly this advice must change and concrete producers, while still listening to advice, must satisfy themselves through trial mixes before believing it.

It is important to realize both the complexity of the situation and the inaccuracies inherent in any attempt to compare the relative value of different admixtures. Different admixtures can have significantly different relative values when used with different cements or other different conditions. A particular brand name of admixture may be differently formulated in different parts of the world. A difference in the time of addition (relative to that of the cement first coming into contact with the water) can substantially affect the performance of an admixture. Different results may be obtained from the same mix and admixtures when mixed in a truck or in a laboratory mixer.

The basic cost of most admixture raw materials is relatively low compared to the selling price of the admixture. This is at least partly due to the very considerable costs of R & D, quality control, technical service and marketing. However, with the possible exception of very large concrete producers with good facilities and very knowledgeable staff, the availability of technical assistance from an admixture supplier may be good value for money.

If one admixture enables the saving of 5 kg of cement per cubic metre of concrete more than another, this may save several hundred tonnes of cement per annum. However the strength difference at the same cement content would only be of the order of 1 MPa and this may be within the margin of error of the trial mixes used.

If it is accepted that trial mixes may be inaccurate and that other user’s production results may not be applicable, the only remaining practical selection basis is an extended parallel trial. This may be simply a matter of using the admixture on trial in one or two trucks per day and always testing these trucks. Over a period it will be accurately seen whether there is any significant advantage from using the new admixture. It may be considered necessary, for a short initial period, to supply the special trucks to a non-critical location or for a use for which a lower grade has been specified.

On the whole it is probably of greater importance to select the correct type of admixture and to use it in the most advantageous way than to obtain the most cost effective admixture. It is therefore again emphasized that most concrete producers should be seeking the ideal admixture supplier rather than the ideal admixture. That is, the correct advice may be more important than the best admixture.

9.1 Specifying admixture usage

It is very important that concrete users do not specify the use of particular admixtures unless absolutely essential for a particular purpose. If they do so, the responsibility of the concrete supplier for the performance of the concrete will be substantially reduced and any and every problem encountered will in some way be blamed on the specified admixture. As far as possible the concrete supplier must be left to formulate his concrete and this should include the use of his choice of admixtures. Where a particular admixture is considered essential, this should be discussed with the concrete supplier and an attempt made to have him use it ‘of his own volition’. If it became normal to impose the concrete user’s choice of admixture on the concrete producer, this would sabotage his entire control system. This would occur because results could not be grouped together for analysis.

As with other aspects of mix design, the purchaser should be entitled to know what is being used in his concrete and to have the right of objecting to unsatisfactory proposals. In general, this right should not be used lightly. The purchaser should certainly refuse permission to use admixtures containing any significant amount of calcium chloride in concrete to contain reinforcement. This is because it is well-established that calcium chloride strongly promotes the corrosion of reinforcement.

Where resistance to freezing and thawing is required, the purchaser should certainly specify that air-entrainment be provided. It may also be reasonable to object to an air-entrainer that produces too large a bubble size. This is because it is the spacing of the air bubbles that matters for frost resistance, whereas the total volume is what is measured by all typical tests and what affects the strength of the concrete. The spacing can only be determined by microscopic examination of a cut and polished face of hardened concrete. It would only be undertaken if, for example, your local reputable admixture supplier advises you that a particular air entrainer your concrete supplier is using is in fact only appropriate as a car washing detergent.

9.2 Possible reasons for using an admixture

1 To save money – by reducing cement content for a given strength and workability.

2 To improve concrete properties

   For example, reduction of bleeding or segregation,
   Compensation for aggregate grading deficiencies,
   Reduced permeability,
   Improved pumpability,
   Reduced shrinkage.

3 To compensate for weather conditions or haulage distance, for example, retarders and accelerators.

4 Reduction of labour costs – superplasticisers/HRWR (high range water reducers).
5 Production of self-compacting concrete.

9.3 Types of admixtures available

Water reducers

These are basically lignosulphonates which are natural retarders but may be modified by the addition of accelerators such as triethanolamine (hopefully no longer calcium chloride as in the past).

A water reduction of the order of 5–10% is obtained and the admixture is used basically to enable cement reduction. Some of the water reduction is due to the unavoidable entrainment of 1.5–2% of air by this type of admixture. The accelerating part of the admixture causes an increases in shrinkage at a given water cement ratio, but this is offset by the water reduction. There is some evidence that early shrinkage is less compensated than later shrinkage and this may lead to slightly increased susceptibility to early cracking.

The time of addition of these admixtures may be important, a delayed addition giving substantially more effect.

In some cases readiness for trowelling of slabs may be delayed even when 24 hour strength is not reduced.

Water reducing strength increasers – ‘polymers’ – hydroxy-carboxylic acids and polysaccharides.

These are sometimes regarded as very similar to lignosulphonates. The cement saving is of a similar order but the action is a little different since water reduction is slightly less and there is a small direct strength increase at a given water cement ratio.

These admixtures are in some cases a little more effective in cement saving than lignosulphonates (especially at higher cement contents) but are more sensitive to variations in cement characteristics.

Newer types of admixture (described as ‘synergized’ by some manufacturers) often combine polymers and lignosulphonates in an attempt to get the best of both characteristics.

Retarders

Set retardation to any desired extent is readily available with no deleterious effects – with or without water reduction.

Sugar is a violent retarder and very small quantities can produce a dramatic effect.

It should be noted that set retardation is not the same thing as workability retention. Mixes containing lignosulphonates may lose slump more rapidly than plain concrete in some circumstances.

Delayed addition may be very important because a greater water reduction is obtained by a delay of the order of 5 minutes after the water has been in contact with the cement. When retarding admixtures are added already dispersed in the mixing water, the retarder can retard the going into solution of the gypsum which is added to cement during manufacture to control rapid setting. In this way a more rapid set may be caused by a retarder. It is not usually practicable to actually delay addition in readymix operations, but the same effect may be obtained if the admixture is added in concentrated form and takes some time to disperse through the mix. Suppliers now deny that this problem still exists, it certainly used to, but now some producers add their admixtures to the mixing water with apparent impunity.

Accelerators

Set acceleration, unlike retardation, is only obtainable within limits and with some risk (or certainty) of deleterious side effect. The field of accelerators in particular is one in which development work is occurring and details are not readily available. The information given below is likely to prove outdated. Purchasers will need to carry out their own trials.

Triethanolamine and salicylic acid are only mild accelerators and are not used alone.

Calcium chloride is by far the most economical and effective accelerator. However, it has the severe disadvantage that it strongly promotes the corrosion of reinforcement (and any other embedded steel). Many, but not quite all, authorities claim that it also increases shrinkage quite substantially.

Calcium formate and calcium nitrite produce almost similar strength gains but less effect on setting times. Both are substantially more expensive than calcium chloride.

Sodium silicate and aluminate and sodium or potassium carbonates are powerful set accelerators but reduce strength at later ages.

Hot mixing water or steam curing can also be used to accelerate set and strength gain. Hot water is in fact often a quite suitable choice as an accelerator, especially in cold climates. A recent major project involving thousands of very large precast segments for an elevated roadway again demonstrated this. Faced with a requirement to attain 18 MPa in 7 hours, only 2 weeks were available to solve the problem. It took only a theoretical analysis and two sets of four trial mixes each to convince the client that hot mixing water was a more economical solution than steam curing, chemical accelerators, or extra cement. The point is, given the very short curing period, that hot mixing water takes immediate effect whereas steam curing has to be applied gradually. Of course a superplasticiser was also used and the author's early age system (see Section 11.4) was an integral part of the solution.

Superplasticisers are very useful for high early strengths, because they enable low water/cement ratios which not only increase eventual strength, but also increase the proportion of that strength developed at earlier ages. Also they give a strong dispersing effect which makes more effective use of high cement contents. Some producers, particularly in tropical climates, find that using a superplasticiser is an economical substitute for steam curing precast units. Of course such a substitution provides a very large strength margin at later ages.

Air-entrainers

It is of interest that most concrete of up to 30 MPa (4,500 psi) in Australia contains entrained air but the practice appears unusual in SE. Asia. Worldwide, one of the principal benefits of air-entrainment is greatly enhanced resistance to damage by freezing and thawing, but in Australia, as in SE. Asia, this is not a problem.

The other reasons for using air-entrainment are:

1 Reduced bleeding
2 Improved cohesion
3 Grading rectification
4 Reduced permeability
5 Improved pumpability (but high air content decreases pumpability)
6 Better surface finish.

The amount of entrained air required for these purposes is somewhat less than may be required for high frost resistance, 3–4% being normal in Australia.

The disadvantage of air-entrainment is that it is an additional factor to control and test, since excessive air can severely reduce strength and pumpability.

Entrained air is generally considered undesirable in mixes of high cement (or other fines) content where frost resistance is not required. However the author has used entrained air to provide lubrication in mixes where fines were excessive and strength relatively unimportant.

Many investigations show that entrained air is still necessary for resistance to freezing and thawing, even in very high strength concrete. The author is dubious about this, considering that it may only apply to fully saturated specimens used in laboratory investigations rather than to real structures. However this is unproven and the omission of entrained air in concrete subject to freezing and thawing represents a risk.

'Waterproofers' (or, more realistically, permeability reducers)

These comprise calcium, ammonium or butyl stearates or oleates, asphaltic emulsions, also silicones and methacrylates.

Their action is generally intended to be either to block pores or to produce a hydrophobic (water repelling) action either at the concrete surface or on the surface of the pores in the concrete. This action may have a limited life in terms of years. On the whole, and for most purposes, chemical waterproofers are not worthwhile. An adequate cement content, a suitable pozzolan, good curing and a low w/c ratio are the most important factors in achieving low permeability. A failure to provide any of these will not be adequately compensated by the use of a waterproofer and if they are all provided, the concrete will be satisfactory for most purposes without a waterproofer. However this simplistic view is not the full story and it is becoming apparent that the inclusion of an appropriate amount of an appropriate HRWR is very beneficial.

A distinction should be made between concrete which will repel surface water (such as rain), concrete which will retain water under pressure without apparent leakage (e.g. water tanks) and concrete which will not even permit the passage of water vapour under pressure (e.g. to avoid damp spots inside buildings via floor slabs or retaining walls). It is the latter which is very difficult to achieve (impermeable membranes such as polythene sheet or coal-tar epoxy paint, applied outside, i.e. on the side from which the water is coming, are normally used). However, at least one proprietary admixture (Caltite) has an established long term record. Also there is an expanding use of silica fume to accomplish the same objective at lower cost. It should be pointed out that workmanship is extremely critical in achieving watertightness without a membrane. Any admixture supplier who provides an effective guarantee to achieve watertight concrete, will certainly insist on being engaged to supervise the production and placing of the concrete. Clearly the use of self-compacting concrete is a substantial benefit.

Repelling surface water is relatively easy although the action may not be permanent. It may be achieved by the integral (i.e. incorporated in the mix) or surface (i.e. painted on) use of silicones or methacrylates or (surface only) chlorinated rubber. These treatments are useful and desirable for example when applied to coloured split block concrete masonry. When untreated, the colour of such concrete appears to fade, but this is due to the deposition of efflorescence on the surface. The colour is restored to some extent when the concrete is wet or the efflorescence is removed by acid washing. It is maintained by a clear surface coating (while this lasts).

Retaining water in a water tank requires only good, well compacted concrete of say 40 MPa (6,000 psi) grade which is both cheaper and better than say 25 MPa concrete with an integral waterproofer of the typical stearate type. The concrete does permit the passage of some water, but not sufficient to cause any noticeable loss of contents and not more than will immediately evaporate from the surface, which appears completely dry.

The author has undertaken a limited trial of the admixture known as 'Caltite' which is understood to contain an asphaltic emulsion in addition to an integral chemical waterproofer. It is claimed that the action is to block the concrete pores with the asphalt particles in such a way that the greater the pressure the more effective the plugging action. The trial compared the Caltite mix with a control mix having 100 kg additional cement, and a superplasticising admixture to substantially reduce water content. Under a pressure of 35 psi the author was surprised to find that the Caltite mix performed better than the control, permitting virtually no passage of water at any stage. However the control mix was cheaper than the Caltite mix and it also gradually ceased to permit any passage of water after the first few days. Thus the Caltite was very satisfactory, at least in the short term, but its expense may not be essential.

Xypex and Krystol (very similar materials) were originally clear solutions claimed to have the ability to penetrate concrete against the flow of water (i.e. against seepage) and to grow crystals in the pores so as to block the flow of water. They were painted on the surface towards which the water is moving. This sounds like science fiction (or advertising exaggeration) to the author but he has seen a number of situations in which it has been apparently effective. Nowadays these materials are more likely to be used as a component of a grout injected in repair situations, or as a component of original concrete. One interesting example of the effect of Xypex was its use as a basic ingredient in a retaining wall for a Singapore basement (since much of Singapore is on reclaimed land, basements tend to be below water level). The reason for the author's involvement was that the contractor was unable to render the wall since, being non-absorptive, mortar would not stick to it.

The use of fly-ash reduces permeability, but silica fume is clearly even more effective. Ggbfs is also reputed to reduce permeability. These materials have been dealt with in Chapter 8.

Pumping aids (now called VMAs or Viscosity Modifying Agents)

(a) Wax emulsions
(b) Thickening agents (methyl cellulose, polyethylene oxide)
(c) Fly-ash
(d) Silica fume

Wax emulsions and thickening agents do improve pumpability, but the improvement is not dramatic. Expense and difficulty may be appreciable. Fly-ash is a big help if available. Silica fume is very effective but also quite expensive.

It has been said that the only satisfactory test for pumpability is to pump the concrete but that the most effective cheap and simple test is to test bleeding. It is probably true that concrete which bleeds will not pump but the reverse is not necessarily the case. It can be seen that the above admixtures are all in effect bleeding suppressants. This old dictum has recently been taken to a new level by Kaplan, de Larrard and Sedran (2005) in a research project involving a specially assembled 148 m closed circuit of piping and over 60 special truckloads of concrete. A technique using a standard pressure air meter with tetrachloroethylene instead of water and so measure water squeezed from the concrete was employed to measure bleeding under pressure. It was found that the rate rather than the quantity of bleeding was significant. This was fortunate because it allowed a rapid result to be obtained. Pumping procedure was also found to be very important. Avoiding delays between trucks and defective joints in the line, and pumping slowly during priming and when difficulties are experienced, is well-known advice. An interesting new wrinkle is the importance of *not* allowing the first concrete to intermingle with the priming grout or to otherwise be more fluid than normal. It confirms the author's own experience that too high a slump can be as harmful as too low a slump if the mix is inadequately cohesive at the high slump.

VMAs are particularly important when SCC of relatively low strength (and therefore low cement content) are involved. SCC has been reported to demonstrate excellent pumpability on the Eureka building in Melbourne, Australia – currently, the tallest in the Southern hemisphere.

Superplasticisers (more correctly known as high range water reducers HRWR)

HRWRs have become distinctly more important in the years since the first edition. HPC (high performance concrete) can almost be defined as concrete containing an HRWR. Their wider use and greater importance have been accompanied by a better understanding of their strengths and weaknesses. It is becoming apparent that denser packing of the paste fraction of concrete is the key to higher strength, greater impermeability etc. This requires the use of finer materials such as silica fume, finer cement, superfine fly-ash etc. Such finer materials have a higher water requirement which offsets their benefit. The answer to this is to use the fine material together with an HRWR to counter the higher water requirement. It has also become apparent that not all HRWRs are compatible with all cements. The best way to check on this is to use the admixture at the intended dose in an otherwise normal Vicat setting test. Better still the test can be repeated at different dosage rates to establish the 'saturation dosage' (i.e. that dosage above which no further water reduction is obtained) as well as checking on the possible rapid workability loss which is the nature of the incompatibility of some admixtures and cements. Alternatively it may be found that excessive retardation of set is experienced in some cases. It is also desirable to include in this test any pozzolanic materials intended for use in the concrete.

The original superplasticisers were melamine formaldehyde and sulphonated naphthalene. The former originated in Germany and the latter in Japan. These are highly effective water reducers with a short period of effectiveness and apparently no permanent effects (no retardation or air-entrainment). They are relatively expensive (although less so than formerly, now that they are in higher volume production and usage) and cannot be justified on cement reduction grounds for ordinary concrete, as can normal water reducers.

They can be used in three ways:

(a) *To produce 'flowing' concrete*. Such concrete is virtually self compacting and may be justified on labour saving grounds. It may also be worthwhile where excellent surface finish (on vertical formed surfaces) is required or for very congested sections.

(b) *To produce very high strength or durability.* At normal workability the water reduction can give high strength increases. This may only be financially worthwhile when the strength required cannot be obtained by increased cement content. On the other hand a superplasticiser is very desirable with high cement content as the cement may not otherwise be adequately dispersed.

(c) *To limit shrinkage.* In thin walls with congested reinforcement a small aggregate, high slump mix may be necessary to achieve full compaction. Such concrete would have excessive shrinkage if the high workability were attained by increased water and cement content, but not if obtained by using a superplasticiser at normal water and cement contents.

The above remarks apply to what are now described as 'first generation' superplasticisers, being the pure materials listed. The situation has now become much more complicated in that there are '2nd and 3rd generation' HRWRs which retain their action over a considerable period of time (in some cases more than 2 hours).

The original materials derived their effectiveness not so much from a new property as from an absence of two old properties. They are able to be used at much higher dose rates than normal water reducers because they do not either retard set or entrain air. As an example of this, it was required to produce a highly fluid mortar with a very low w/c ratio to surround and protect a steel tension pile (or ground anchor). High strength was really only essential at the rock anchorage over 30 m below ground level. A superplasticiser was considered, but it was realized that a normal water reducer at the same dosage would produce a similar water reduction at lower cost. It was an advantage that a very long retardation resulted (because the mortar was placed first and the pile was lowered into it). The high air percentage was reduced to a very modest amount by the fluid pressure at the full depth.

There is now an enormous variety of HRWRs available, from a dozen or more different countries. The original materials have been supplemented and/or replaced by others, including lignosulphonates formulated to entrain reduced amounts or air and produce less retardation. Their cost, relative to the cost of labour, is reducing. The value of very high strength concrete is becoming more widely realized. Perhaps more important still, it is being realized that these materials are not only labour content reducers, but also skill requirement reducers. For all these reasons, the use of superplasticisers is on the increase.

The 'new kid on the block' is a polycarboxylate. These admixtures are particularly favoured for use in SCC (self-compacting concrete), having a longer workability retention with less set retardation and apparently giving some bleeding resistance. A problem with this type of admixture is that it tends to entrain excessive amounts of air. This is countered by the inclusion of a 'de-foaming agent' (i.e. air-entrainment suppressor). However, some such combinations require continuous agitation to avoid settling out.

In America especially, these materials are now called ‘high range water reducers’. This recognizes that they are often not used to superplasticise concrete but only to produce a substantial water reduction. Similarly the emphasis is no longer on high strength concrete but on ‘high performance concrete’, it having been realized that much concrete uses high range water reducers, silica fume etc for reasons other than high strength.

Shrinkage compensators

1 Finely divided iron
2 Calcium sulpho-aluminate

These materials work but require careful use to avoid the expansion tendency being disruptive. Also it must be remembered that they do not actually work by reducing shrinkage. In both cases an expansion is produced whilst the concrete is kept damp (i.e. before any shrinkage occurs) and the concrete then shrinks normally. The initial expansive tendency is restrained by reinforcement or by abutting concrete and develops a compression which dies away under the later influence of shrinkage. In addition to the risk of excessive expansion causing disruption, there can also be a ‘threshold’ effect in which the expansive tendency is inadequate and the pre-compression is all lost in creep of the concrete, leaving no effect on subsequent shrinkage.

In USA shrinkage compensating cements are available, and even expanding cements designed to automatically apply prestress to cast-in steel tendons. This is done by the incorporation of calcium sulpho-aluminate in the cement during manufacture.

‘Eclipse’ is a new shrinkage-reducing admixture of which the author is yet to have personal experience. However, it is reported to be quite effective in almost halving shrinkage, but rather expensive. The mechanism is understood to be based on reducing the surface tension of water in the pore space of the cement paste. It would seem to the author that not all concrete would retain enough water for this to be applicable but time will tell.

Chapter 10

Statistical analysis

Statistical analysis is not an exact science. However rigorous and elaborate the statistical techniques used, the conclusions can be no more reliable than the assumptions on which they are based. Where a limited amount of data has been obtained from a one off experiment or series of observations, it can pay handsome dividends to apply very elaborate analysis techniques to squeeze out the last drop of knowledge. However, QC is not a one off experiment but a continuing flow of data. Furthermore it is a field that is, or should be, rigidly governed by economic considerations.

The requirement is to ensure a given minimum quality of concrete in the structure. This can be accomplished by using a higher average quality, at a higher cost in materials, or by achieving a lower variability through higher expenditure on control. The higher control expenditure itself can be in the form of a large amount of rough testing with little analysis or in a smaller amount of more carefully monitored testing and a more thorough analysis of the results. A balance should be sought which yields the minimum overall cost for a given required quality. The balance must take into account the standard of personnel and equipment economically available. There is no merit in devising a system that requires that every testing officer be a qualified engineer and every team include a professional statistician, if the result is a higher cost for a given minimum quality.

The concern should not be to apply elegant or rigorous statistics but only to achieve accurate control of concrete quality. Relatively crude statistical techniques can be used if their limitations are very clearly understood and the controller must always be prepared to overrule or revise unrealistic conclusions produced mathematically. It is quite difficult to do this without permitting bias to cloud judgement but there are several factors that save it from being almost impossible. One of these is that in QC work a conclusion is usually provisional and subject to revision as further results are received, thus a downturn in results may be dismissed as a chance variation or testing error when first spotted, but if it is confirmed by subsequent results, it must then be accepted. Another is that related variables such as slump, density and concrete temperature can confirm or deny an unusual result by demonstrating what caused it. Thus if a single low test result is from the lighter of a pair of specimens, it can be neglected, but if a low pair of strengths are accompanied by a high slump reading they must be accepted as fact, but still may not indicate a need for a mix revision – only for better slump control.

Some crude statistical techniques have been used by the author. This has been done quite deliberately since in his opinion more mathematical sophistication would not help. Rather, what is needed by way of sophistication is a very thorough realization of what factors may cause conclusions to be unrealistic, how unrealistic they might be and what can be done to ensure that such conclusions are weeded out and do not lead to inappropriate control action. The total amount of sophistication in a scheme must be limited to keep it within the capability of ordinary practitioners. It must always be borne in mind that the objective is to achieve more economical operation rather than to display virtuosity.

10.1 The normal distribution

If a mathematical description or *pattern* of a set of results can be found, it may be possible to establish what the pattern is from a limited number of results already obtained and use it to predict what future results will be obtained *if the current pattern continues to apply*. It may for example be possible, without ever having obtained a result below some particular value, to predict that a result below that value will inevitably occur unless action is taken to change the pattern. We shall be in a much stronger position to control concrete quality if it can be established that control action is necessary without experiencing even one ‘failure’ than if we have to wait for failures before reacting to them. The position will be even stronger if it can be established from early age tests, or even from tests on the freshly supplied concrete, rather than from 28 day results.

If each result is considered as a ball and a number of slots corresponding to strength ranges are set up (e.g. 22.5–25 MPa, 25–27.5 MPa, 27.5–30 MPa etc.) each result can be placed in its slot giving a picture like Fig. 10.1.

Such a figure is known as a ‘histogram’. If we have a very large number of balls and divide them into narrower slots, the result may approximate to a smooth curve as shown in Fig. 10.2.

One purpose of introducing Fig. 10.1 was to make it clear that area under the normal distribution curve represents number of results. Just as each ball occupies the same area in the two-dimensional representation, so each unit of area in the normal distribution represents a fixed proportion of test results.

This type of graphical representation is called a ‘frequency distribution’ or just a ‘distribution’. There are many different shapes of distribution curves known to statisticians but the particular bell shaped curve shown is called a ‘normal distribution’. It can be constructed from a standard table of figures (‘ordinates’) appearing in any statistics textbook. This table will be accompanied by a second table (Table 10.1) listing the areas under the graph more than a given distance away from the mean (the high point).

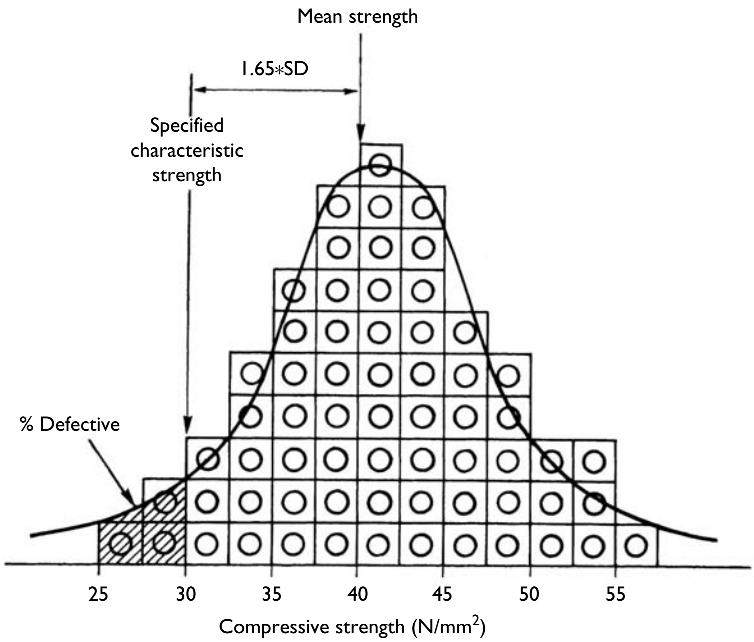


Figure 10.1 Simulated distribution of test results.

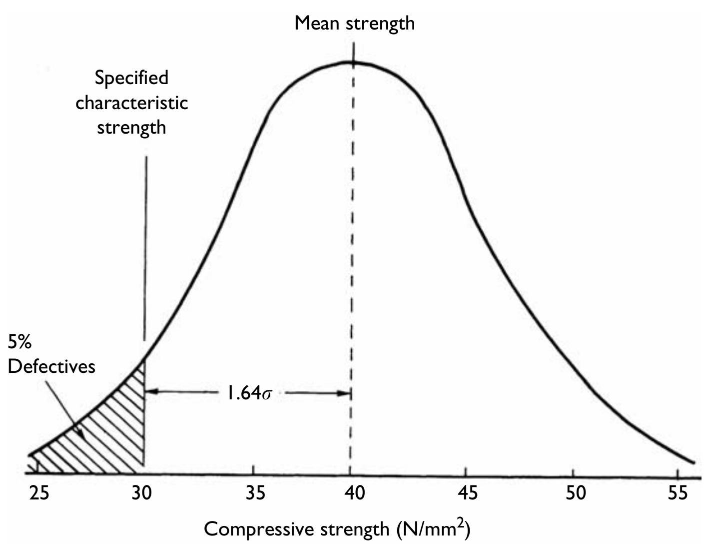


Figure 10.2 The normal distribution.

Table 10.1 Percentage of results outside statistical limits

A (%)	k
0.1	3.09
1.0	2.33
2.5	1.96
5.0	1.65
10	1.28

The information needed to construct the graph (apart from the table of figures) is only the mean (average) of all the results which we shall call X and a quantity called σ or SD which is the 'standard deviation' and is a measure of how widely the results are spread. The numbers X and σ can be read from many simple calculators when a series of results are entered, they can also be automatically produced by a computer. The standard deviation is the square root of the average of the squares of all the differences between each individual result and the average of all results, that is,

$$\sigma = \sqrt{\left[\sum(x_i - x_m)^2/n\right]}$$

where:

x_i = individual result
x_m = mean of all results
n = number of results

Another method of determining the standard deviation is from the difference between successive results.

$$\sigma = \frac{\text{Average difference}}{1.13}$$

This method gives the same answer as the above 'standard method' if the data analysed is a true normal distribution. However there is a very useful significant difference if the data analysed is a time sequence of results having a change of mean somewhere in the sequence. In such a situation the standard method gives an inflated value for the standard deviation because it effectively involves a change of the true mean of the results both before and after the change to a new intermediate mean. We do not need to go into the mathematics of this (although they are quite simple), it is sufficient to realize that it occurs and to take it into account. The difference method is almost totally unaffected by such a change. It is particularly useful in assessing the variability of multigrade results since it is quite easy for the computer to be programmed to average differences from the last result *in the same grade*. In this way a much more meaningful SD can be obtained from a relatively small number of results scattered over a large number of grades.

The UK QSRMC quality control system uses the difference method since it assumes that change points will be relatively rare and effectively re-starts the analysis after one has been experienced.

The author's QC system prints out the SD from the difference method at the top of its result table display but then gives the SD by the standard method for each separate grade of concrete in the table itself. Of course grades with few results are likely to show large fluctuations in SD, but looking at grades with say 20 or more results, a standard method SD much in excess of the difference method SD at the top of the table usually indicates that there has been a change point in that grade, which should be investigated. However, it could also indicate that there are particular problems causing high variability in that grade (also requiring investigation).

The difference method SD can also be applied to the within sample (or testing error) SD. Where pairs of specimens are tested, the within sample SD is given by:

$$\frac{\text{Average difference}}{1.13}$$

Where three specimens are tested at the same age, the SD is given by:

$$\frac{\text{Average range difference between highest and lowest}}{1.69}$$

Returning now to illustrating the principles of statistics, Fig. 10.4 shows three distributions with the same mean but different values of standard deviation.

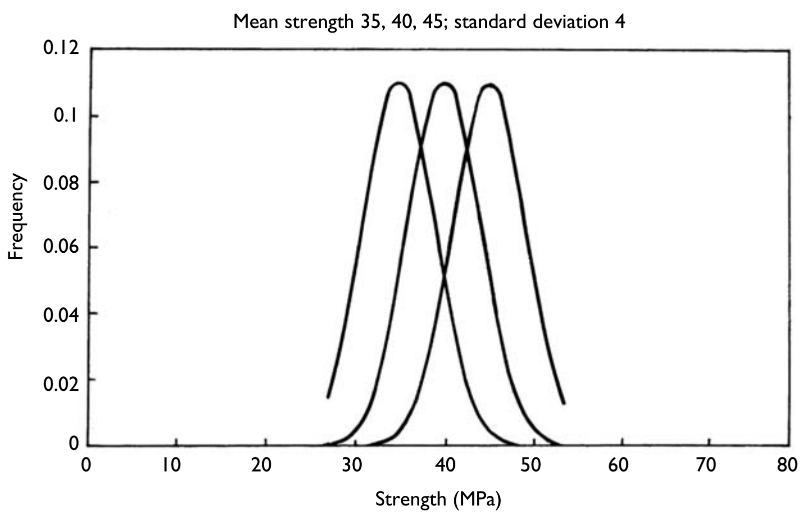


Figure 10.3 Three distributions with the same σ but different values of X.

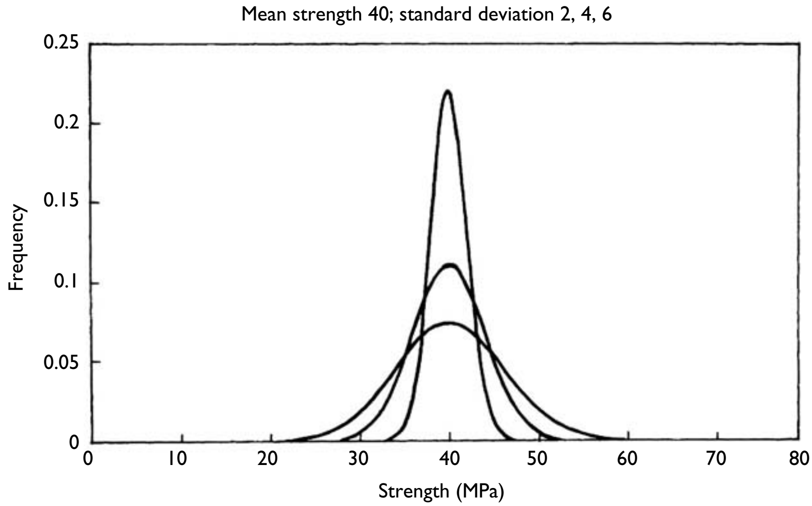


Figure 10.4 Three distributions with the same X but different values of σ.

Fig. 10.3 shows three distributions with the same standard deviation but different mean values.

We are interested in the percentage of results less than a certain strength (i.e. the percentage defective). Looking again at Fig. 10.2, the distance below the mean (or above, the curve is symmetrical) can be expressed as a parameter k (i.e. a variable number) times σ and the area as a percentage of all results. The published tables relate the area to the value of k, Table 10.1 is an extract from such a table.

Permissible percentage defective

There is logic in using a 5% defective level (or even a 10% defective level) in that adherence to the assumed statistical distribution is not exact. The assumption predicts reasonably well the level below which 5% of results fall (in the author's experience there are likely to be actually 2–3% below the level below which 5% are predicted to fall, but more about this later) but at the 0.1% level, the assumption has become highly theoretical and any result actually below this level is almost certainly the result of some ascertainable special cause rather than normal variability. So if the intention is to actually predict what results will be obtained, the 5% level is as far as it is reasonable to go and the USA use of 10% may be even more realistic. However if the results are to be judged by analysis of an adequate number of them rather than by whether any results are actually below a particular level, the Fig. 10.5 situation can be considered because it then becomes a matter not of whether the distribution is accurately followed, but simply of how much incentive it is desired to provide to achieve low variability.

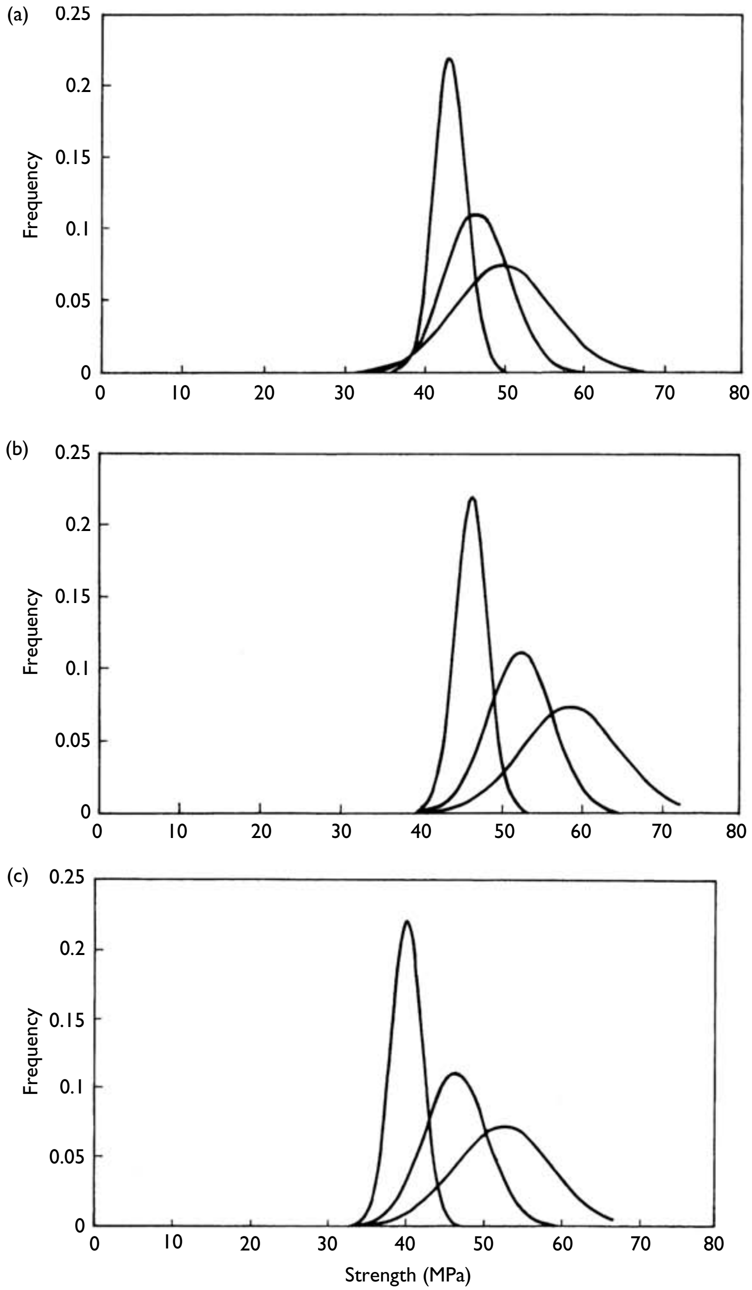


Figure 10.5 Specification options to encourage better control.

Figure 10.5 illustrates the available options: Fig. 10.5(a) shows 5% below specified strength, as used in most parts of the world.

Fig.10.5(b) shows the effect of decreasing the permitted percentage defective to 0.1%. This option would provide a greater financial incentive to achieve low variability (i.e. good control) but would substantially increase the average cost of concrete.

Fig. 10.5(c) shows that, by adjusting the specified strength level, the average cost of concrete can be kept unchanged while still providing an increased incentive to good control.

Any suggestion to specify an 0.1% defective level is certain to encounter the criticism that this is highly theoretical and unrealistic. It is very important to clearly make the point that this is true but immaterial. What matters is to realize that it is possible to make use of any desired relative value of mean strength and standard deviation without affecting the cost of concrete from an average producer. If s is the standard deviation considered to be average, then the required mean strength x for a specified characteristic strength $F'c$ could be required to be:

$$x = F'c + k\sigma - (k - 1.65)s$$

or, in USA,

$$x = F'c + k\sigma - (k - 1.28)s$$

k can be given any desired value without affecting the mean strength required of an average producer. The larger the value of k the greater the cost advantage given to a lower variability producer and the greater the disadvantage suffered by a higher variability producer. There is no requirement to select a value of k which represents a particular percentage of results (e.g. from Table 10.1). Users should not forget the table and its significance but it may be reasonable to select a value of 1, 1.5, 2, 2.5 or 3 (or even 4, which would have no statistical significance) according to the relative importance attached to mean strength and variability.

Looked at in this way, the American choice of 1.28 is seen to provide a lesser incentive to achieve low variability than the more usual 1.64 or 1.65 and the author would prefer to use a value of 2 or even 3. The reduced incentive may explain a reduced interest and attainment in the USA in matters of QC.

Having discounted the realism or otherwise of the theoretical percentage defective as a basis for choosing the value of k, there is another consideration. This is the accuracy with which σ can be assessed. Section 10.3 below provides details.

Taking the data from Tables 10.2 and 10.3 together, it is seen that the error of estimation of the mean of three results is about five times the error in estimating the standard deviation from the last 30 results and almost four times that from 20 results. A proposal to multiply the standard deviation by 2 or 3 would therefore be reasonable if the σ were based on at least the last 30 results. However it should be realized that a standard deviation change of less than $\pm 25\%$ from its previous value would not be significant.

There is a further consideration in increasing the number of results on which the standard deviation is based. If the results analysed extend across a change point in mean strength, the standard deviation will be artificially inflated. Care is necessary in determining the desired result. As discussed in Section 4.2 the variability between change points is the basic variability of the production process. The frequency, extent, and time to react to, change points depend largely on the control system, including control of incoming materials. The purchaser of the concrete will be interested in the overall combined effect of all causes of variability. However, a consideration of the worst concrete supplied would more accurately concentrate on the mean strength and basic variability between the two change points enclosing the concrete in question.

10.2 Variability of means of groups

So far we have considered only how well the assumption of normal distribution portrays the actual distribution of strength in the whole of the concrete. It is now time to consider how well an analysis of a limited number of samples portrays the distribution which would be obtained if the whole of the concrete supplied were made into test specimens and tested. It is conventional to consider that about 30 results are needed to give a reasonably accurate picture but it is instructive to look into the actual situation. One way of doing this is by the use of another distribution called the 'Student's *t*' distribution. This is a very useful method for evaluating comparative laboratory trials of such things as alternative admixtures or alternative cements but it will not be considered here.

If the whole of the concrete were made into test specimens and divided into groups each of 'n' samples, the mean of each such group would in general differ at least a little from the mean of the other groups and from the 'grand mean' of all samples. In fact the means of the groups would be found to themselves be normally distributed but of course not so widely as the individual results. Statistical theory tells us that the standard deviation of the means of groups of n results is related to that of the individual results by the formula

$$\sigma\,(\text{groups}) = \frac{\sigma\,(\text{individuals})}{\sqrt{n}}$$

So the means of groups of 4 results will have half the σ of individual results and the mean of groups of 25 will have one-fifth the individual σ.

If we take limits within which 90% of results fall (i.e. 5% outside each limit) the mean of the group of n results will be within $\pm\ 1.65/\sqrt{n}$ of the true value. Table 10.2 summarizes this.

Table 10.2 Error in mean for various values of standard deviation

Standard deviation (SD) values>	*2*	*3*	*4*
No of results			
1	3.30	4.95	6.60
2	2.33	3.49	4.65
3	1.91	2.86	3.81
5	1.47	2.21	2.95
10	1.04	1.56	2.08
20	0.73	1.10	1.46
30	0.60	0.90	1.20

At this point it is perhaps necessary to point out that the conformance of practice to theory is nowhere near good enough to justify the use of a second decimal place in Table 10.2. The object of the exercise is to get a feel for the order of magnitude of the errors involved.

It is worth noting that the variability of the results being examined has a strong influence on the accuracy with which they can be assessed. This is a generally applicable statement and is another reason for preferring low variability concrete.

It will be seen that if a single test result is obtained to represent a truck of concrete, or even the mean of a pair, the assessment will not necessarily be very precise, particularly if we are dealing with variable concrete. However variation within a batch, that is, within a single truckload, is a different matter to variability between batches, and is largely a matter of testing error rather than variability of concrete, see Section 11.5.

Likewise if a day's supply of concrete is assessed on the basis of three samples of concrete, a considerable error may be involved.

10.3 Variability of standard deviation assessment

In a similar manner, the value of the standard deviation (σ) obtained from analysing a limited number of results will differ from the true value for all the concrete. In this case the standard deviation of the distribution of standard deviations (no, it isn't a misprint!) is given by SD where:

$$\mathrm{SD} = \frac{\sigma}{\sqrt{2n}}$$

A table (Table 10.3) similar to Table 10.2 can be constructed. Although these errors are a little smaller than those in the case of the mean, they are a very much larger percentage error. Note that a group of 5 will only yield a σ value to $\pm$ 50% accuracy approximately. What this means is that the variability of a group of less than 10 results simply cannot be determined with reasonable accuracy.

Table 10.3 Error in standard deviation for various values of true standard deviation

Standard deviation values>	*2*	*3*	*4*
No of results			
2	1.65	2.48	3.30
5	1.05	1.58	2.09
10	0.74	1.11	1.48
30	0.42	0.63	0.85

This has had a profound influence on the basis of specifications because, if we persist in trying to judge the quality of concrete on the basis of a small number of samples, it is not possible to give any credit for low variability (unless this is assessed on a basis external to the group of results in question). Even the inaccuracy in the mean value noted previously is large enough to require a large tolerance if good concrete is not to be rejected and this tolerance results in excessive leniency for poor concrete (see Fig. 10.5(a)). However there is no objection to framing a criterion involving the mean of the last 3, 4 or 5 results and the standard deviation of the last 10, 20 or 30 results.

10.4 Components of variability

One further piece of statistical theory is needed. This is how variabilities due to separate causes combine to give an overall variability. There is a famous example of a wrong assumption about this marring an otherwise excellent paper on concrete quality control (Graham and Martin, 1946). The square of the standard deviation is called the 'variance'. Standard deviations are not additive but variances are. This can be illustrated using the famous example in question (the standard deviations are in psi).

Source of error	*Standard deviation (psi)*
Cement (C)	240
Batching (B)	462
Testing (T)	188

The overall error is *not* given by C + B + T = 890 but by

$$\sqrt{(C^2 + B^2 + T^2)} = 553$$

The effect of this situation is that the contribution of all but the largest component of overall variability is reduced. Thus totally eliminating cement variability would give an overall variability of $\sqrt{(B^2 + T^2)} = 499$, a reduction of only approximately 10%. But in the famous paper, the variability of the cement was further exaggerated by including the error in testing the cement and it was reported that cement variability accounted for 48.2% of total variability. This was a very significant error because it suggested that much of the variability was outside the concrete producer's control. Thus one would be led to putting much of the control effort into cement testing, instead of where it was most needed (slump control).

This is a lesson that must be learned if economical control is to be achieved. The primary (largest) cause of variability must be found and control action concentrated on it (see also Pareto's Principle, Section 4.7).

Of course it is necessary to monitor subsidiary causes as well, in order to establish which is the major cause (and to check that what was initially the major cause has not been overtaken by some other cause) however the real control effort must be correctly directed.

10.5 Testing error

It has been argued elsewhere (Section 11.3) that testing itself is a significant source of error on a typical project and that it must be monitored.

The author has experienced two different testing organisations testing the same truck of concrete and getting results differing by as much as 10 MPa (1,450 psi) on occasions and as much as 3 MPa (435 psi) on average over a substantial number of samples (Day, 1979).

The error in question covers all aspects of taking a representative sample and casting, curing, capping and testing specimens. It is only possible to fully establish the magnitude of this error by taking two samples from the same truck and this is rarely economically practicable unless serious malpractice is suspected and is to be investigated for a short period. However, the 'within sample' error can be established providing that two (or more) specimens from the same sample of concrete are tested at the same age. The author introduced a system by which the concrete supplier's own control testing was accepted as the project control providing that he produced double sets of specimens at specified intervals and delivered them to an independent laboratory for test. This is much more economical than having an independent sampler on site and avoids the concrete supplier claiming that the independent samples have been incompetently sampled, cast or field cured. The only remaining problem is that someone has to ensure that the selection of trucks for test is unbiased. This system is highly recommended wherever there is any concern about the veracity of the supplier's own testing. However the nett result is often that the supplier's testing is seen to be acceptable and comparative testing discontinued.

It has been pointed out that even five specimens would not permit a meaningful direct determination of standard deviation for a single sample. However, another piece of statistical theory provides the information that the average difference between many pairs of specimens from different samples is related to the within sample standard deviation by the simple equation:

$$\text{within sample standard deviation} = \frac{\text{average pair difference}}{1.13}$$

(in the case of sets of three specimens the difference between highest and lowest, i.e. the range, may be used in the same way and in this case the 1.13 becomes 1.69).

Generally there is no point in converting to standard deviation for our purposes and the average pair difference is directly monitored. The best achievable average pair difference on normal concrete is 0.5 MPa (say 75 psi) and between 0.5 and 1.0 MPa can be considered acceptable. However the author has encountered laboratories of high repute with a pair difference consistently in excess of 1.5 MPa. The seriousness of this situation can be appreciated when it is realized that even this figure does not include sampling error and that a really top class producer can work to an overall standard deviation of concrete quality below 2.0 MPa. As discussed above we must not fall into the error of saying that testing is three quarters of the total variability (and remember the 1.13 factor) but nevertheless such testing is grossly unfair to the producer.

10.6 Coefficient of variation

Another measure of variability is the 'Coefficient of Variation'. This is the standard deviation divided by the mean strength and expressed as a percentage. The question is which of the two parameters best measures relative performance on different grades of concrete. The argument resurfaces from time to time even though in the author's opinion general agreement that standard deviation should be used was reached in the 1950s. The author has personally monitored thousands of test results covering 20, 25, 30, 40 and 50 MPa grades of concrete from the same plant over long periods of time. There has never been any question in his mind that standard deviation remains reasonably constant over the 20–40 MPa grades. (i.e. mean strengths from 25 to 45 MPa or 3,600 to 6,500 psi). This opinion was formed in the early 1950s when he consistently achieved a standard deviation of less than 250 psi on very tightly controlled factory production with a mean strength in excess of 9,800 psi. This was certainly abnormal concrete produced in tiny quantities and, being of earth dry consistency, visual water control was very easy. However, if this figure is expressed as a coefficient of variation of less than 3%, it would represent a standard of uniformity impossible to achieve on concrete of normal strength, even under laboratory conditions.

The above firm opinion, even allowing for the quoted high strength experience, must be tempered by an acknowledgment that a slightly higher standard deviation is normally experienced on 50 MPa and higher grades. This appears to be largely due to the greater difficulty in achieving accurate testing, perhaps in turn due to the different mode of failure of higher strength concrete (where bond failure, or even aggregate failure, rather than matrix failure tends to be experienced). The increase in both average pair difference of specimens and overall concrete standard deviation is of the order of 0.5–1.0 MPa.

Since publication of the first edition interesting further evidence is to hand. The Petronas Towers project (at that time the world's tallest building, in Kuala Lumpur, Malaysia) involved more than 40,000 cubic metres of 80 MPa grade concrete. Being under a UK type specification, this required a mean strength of approximately 100 MPa (cube, at 60 days). It can be imagined that, in view of the importance of the project, the initial concrete supply was at a conservatively high mean strength of just over 110 MPa. This caused the overall standard deviation for the whole of the 632 samples tested at 56 days to be inflated to 4.7 MPa. However when things had settled down later in the project, a run of 237 consecutive results gave a standard deviation of 2.8 MPa with a mean strength of 99.3 MPa.

An even lower SD value of 2.6 MPa on 80 MPa concrete for the Chateaubriand bridge is reported (de Champs and Monachon, 1992).

Set against these figures are the decisions of ACI Committees 211 (Mixture Proportioning), 214 (Evaluation of Test Results), and 363 (High Strength Concrete) to adopt coefficient of variation as the meaningful index of variability. The leading advocate of this view was Jim Cook but of course, the decision was that of the committees as a whole. A recent paper by the author (Day, 1998) suggests that high strength concrete offers more scope for increased variability if either the testing process or the regulating analysis system is of less than the highest standard, but does not necessarily have higher variability. Cook's view is that lower coefficients of variation on high strength concrete are obtained simply because the producer is trying harder than with his normal concrete. This contrasts with the often expressed view that a producer makes his reputation on his high strength concrete but his profit on his low strength concrete. For this reason, Australian concrete producers are certainly trying every bit as hard to achieve low variability on their low strength concrete. However, it may well be the case in USA, where specifications often do not allow the producer to derive any financial benefit (i.e. any cement reduction) from the attainment of lower variability.

The author's strong advocacy of standard deviation as the measure of compressive strength variability does not mean that the coefficient of variation is a useless parameter. Obviously the same standard deviation cannot apply to such variables as tensile or flexural strength, much less to slump or density. A 5–10% coefficient of variation in anything generally represents a variable under reasonable control although, for example, a modern batch plant can achieve much better than 1% in cement batch weight (if properly maintained).

10.7 Practical significance of the foregoing

The most obvious point emerging from the foregoing is that it is not feasible to take a quantity of concrete small enough to be regarded as a unit for purposes of acceptance or rejection and to represent it by a sufficient number of test results to assess its quality with reasonable accuracy. (The fact that it is also economically ridiculous to consider physically rejecting concrete which is only slightly under-strength is only another nail in the coffin.) Since the future progress of the concrete industry depends on encouraging reduced variability, it is absolutely essential that quality be assessed on the basis of a large enough pool of results to enable not only mean strength but also variability to be accurately assessed. Since not even a madman would consider rejecting a month's concreting because it is slightly understrength, there is simply no other way to go than cash penalties or cash incentives (although it is feasible for the real diehards to impose this penalty in the form of increased cement content or increased testing as noted in Chapter 6).

The next point is that we do not wish to sit back and watch the contractor dig his financial grave for a month or so without taking any action. An eventual cash penalty may bring justice to the situation and may avoid him repeating his error, but it will not provide the quality of concrete required in the current structure. Therefore a method of closely monitoring the situation and taking early action to revert to the desired quality is very desirable. This used to mean keeping a graph known as a Shewhart QC chart, however these have been superseded by cusum control charts in the author's system.

As we have seen, a substantial error is possible in assessing the standard deviation, mean and 5% minimum of a small group of results, so that they cannot be used with any degree of fairness to reject or penalize. Nevertheless more than 50%, perhaps as much as 70 or 80%, of such assessments are quite realistic. They are therefore very useful as a guide to the state of affairs provided they are used only as a warning that the situation should be carefully considered and not as a basis for precipitate action. Having isolated the rigid legal requirement as based on an unquestionably accurate assessment of a large quantity of results, it is then possible to informally consider a large number of factors in deciding when a small mix adjustment may be desirable. There will be scope for a small difference of opinion between concrete producer and supervisor from time to time but the latter can afford to concede graciously and wait for the fullness of time to bring retribution if it was merited, secure in the knowledge that the quality shortfall will be minor and the retribution precise, inevitable and indisputable.

A very interesting matter is a comparison of the standard deviations considered normal in Australia and the UK. The author has for many years considered 3 MPa (say 450 psi) to be a normal figure for an average ready mix plant in Melbourne. Of recent years the better practitioners are attaining 2 MPa, or even fractionally less. In the UK, a figure of 4–6 MPa is considered normal. It is not likely that physical control of production is genuinely twice as good in Australia and an explanation is likely in the statistical concepts applied. In the UK, results are corrected or normalized according to cement content so as to provide a basis for combining results from different grades. It would appear that this does not work very well. Having created an artificially higher variability in this or some other manner, the task of detecting change becomes more difficult. When a rigid mathematical requirement (in the form of a V mask) is applied to determine whether an adjustment should be made, the difficulty is compounded. When adjustment is delayed in this manner, a genuinely higher variability is created or allowed to continue. This question is further examined in Section 12.3 (‘how soon is soon enough?’).

Chapter 11

Testing

11.1 Philosophy of testing

It is very important to understand the philosophy of testing. Only persons ignorant of the true situation regard a test result as an accurate portrayal of the property tested. Unfortunately this tends to include many persons in authority such as specifiers, controllers and legal people.

In the first place no test can be perfectly accurate and it is as well to consider how inaccurate it might be. In the second place the sample tested may not be truly representative of the mass being assessed. So for example, standards may require great care in checking the equipment and following a rigid procedure to get an accurate sand grading. It may also lay down clear rules for obtaining a representative sample. But if you are doing QC on concrete, there is nothing to beat doing frequent rough checks (twice the number of tests in half the time), looking at the results on a cusum graph of specific surface, and taking a second sample to confirm if the first one says there has been a change.

A very important distinction between QC and research is in continuity. A research project, however large and long, must eventually come to an end, and some very elaborate statistical techniques and great care to achieve testing accuracy may be of substantial value in reaching an accurate conclusion. QC is a continuing flow of data that may necessitate revised conclusions from time to time. Many factors may affect the desirable level of sophistication of both testing and analysis techniques. The cost benefit must be assessed of the relativities of expense and accuracy against volume and simplicity, especially taking into account the standard of personnel who will be operating the system.

As with the rest of this book, the author strives for truth and reality over regulation and convention but warns readers that there may be times when his often unconventional views are unacceptable to someone who has to be humoured.

11.2 Range of tests

A very large number of tests on concrete have been devised. A partial list is given below.

Tests on hardened concrete

Compressive strength (cylinder, cube, core)
Tensile strength: Direct tension
Modulus of rupture
Indirect (splitting)
Density
Shrinkage
Creep
Modulus of elasticity
Absorption
Permeability
Freeze/thaw resistance
Resistance to aggressive chemicals
Resistance to abrasion
Bond to reinforcement
Analysis for cement content and proportions
In situ tests: Schmidt Hammer, pull-out, break-off, cones etc.
Ultrasonic, nuclear.

Tests on fresh concrete

Workability (slump and over 20 other)
Bleeding
Air content
Setting time
Segregation resistance
Unit weight
Wet analysis
Temperature
Heat generation

Of these many possible tests, in practice well over 90% of all routine tests on concrete are concentrated on compression tests and slump tests which should be, but are not always, accompanied by fresh concrete temperature and hardened density determinations.

Before considering whether this is a desirable state of affairs, it is first necessary to consider the purpose and significance of the testing.

There are at least three possible purposes:

1 To establish whether the concrete has attained a sufficient maturity (for stripping, stressing, de-propping, opening to traffic etc.).
2 To establish whether the concrete is basically satisfactory for the purpose intended.

3 To detect quality variations in the concrete being supplied to a given specification.

It is very important to be clear about the purpose of the testing because attempts to fulfill all these purposes simultaneously usually lead to inefficiency in fulfilling any of them. The true essential purpose of the majority of tests is the detection of quality variations.

The selection of compressive strength for the great majority of control testing relies upon three basic assumptions:

1 That all or most other properties of concrete are related to compressive strength.
2 That compressive strength is the easiest, most economical or most accurately determinable variable amenable to test.
3 That compressive strength testing is the best means available to determine the variability of concrete.

The second of these assumptions will be examined in detail later.

The first assumption is probably correct in so far as the purpose of the test is to detect quality variations but is not necessarily correct if the purpose is to establish whether the concrete is basically satisfactory (e.g. shrinkage may increase as compressive strength increases if the strength increase is obtained by increasing cement content but would reduce with increasing strength if this was obtained solely by reducing water content).

It may well be impracticable on most projects to use other forms of test for quality control purposes (although rapid wet analysis has been so used). However, especially where we are dealing with standard mixes from a premix plant, or a special mix designed for a specific purpose, it is certainly practicable to carry out a much wider range of tests to initially verify a new mix design and to repeat a wide range of tests at say annual, or six monthly, intervals for standard mixes. An excellent example of this is the shrinkage of concrete in the Melbourne (Australia) area. For many years structural designers had been concerned about excessive shrinkage but the only action resulting from this concern was to prohibit the use of pumped concrete on some projects and limit sand percentages on others. However, in 1977/8 CSIRO (the Australian Govt. Commonwealth Scientific and Industrial Research Organisation) carried out shrinkage tests on a range of standard Melbourne area pump mixes and showed a wide range of variation with clearly definable causes. It then became practicable to specify a limiting shrinkage and in most cases to permit the use of pumped concrete since the tests showed that some pumped mixes had a lower shrinkage than some non-pump mixes (the factor involved being the influence of the coarse aggregate).

Similar action is now needed in respect of splitting strength, permeability, durability, abrasion resistance and also workability (other than slump), segregation resistance, bleeding and surface finish characteristics. These were all matters on which we were flying as blind as we used to be on shrinkage at the time of writing the first edition. In the intervening years there has certainly been substantial action in respect of durability and permeability (with the latter seen as the best available criterion of the former). With a 100 year durability requirement specified by the client for a major project in Melbourne, the author translated this into a maximum VPV of 9%. VPV is volume of permeable voids and is determined by the loss of weight on drying an initially saturated sample of concrete. However, this basis was chosen because there was no local experience of other techniques such as the James Instruments adaptation of the two Figg tests or the UK Wexham Developments variant of this type of equipment.

11.3 Compression testing

Considering now the accuracy and convenience of compressive strength as a routine control, the situation is not so simple as was thought 20 or 30 years ago. In Australia we are fortunate to have the world's first and most highly developed National Association of Testing Authorities (NATA). We have a better system than most other countries for ensuring that test specimens are cast by competent persons, taken to laboratories with satisfactory curing facilities, capped with a sound cap and tested in a standard manner in a properly calibrated and maintained testing machine. Without being able to quote chapter and verse, but having used both extensively, the author is also coming to the view that the cylinder specimen is at least a little more reliable than the cube specimen. Nevertheless it is now apparent that NATA certification is not sufficient to ensure that different laboratories obtain essentially the same test strength on concrete from the same truck of concrete. Isolated differences of over 10 MPa and consistent differences of the order of 2–4 MPa have been documented in Melbourne (Day, 1979, 1989).

There are two aspects to the problem:

1 The technology of compression testing machines.
2 Day to day performance variation.

Testing machines

A compression testing machine is usually by far the most expensive item in a routine concrete QC laboratory. As such machines are also very durable items, there is a tendency for quite antique versions to be still in service (and indeed they may give better results than a cheap new machine).

It is apparently an extremely simple thing to apply a compressive load to a test specimen using a hydraulic ram. However, in practice it is far from simple because the results obtained must be very consistent and must bear comparison with other testing machines.

The author has had a wide experience of operating different classes of compression testing machine over many years, but such general experience is of little value. What matters is access to comparative results on samples from the same truck of concrete and preferably cast by the same person. A requirement that this be done as a regular routine has been part of the author's standard specification for some years and such data is therefore available covering a number of different pairs of laboratories. The Australian National Association of Testing Authorities (NATA) also organizes occasional comparative tests in which a large number of specimen are cast from a single truck of concrete and distributed to many laboratories. There is a distinct difference in the extent of variation found when each laboratory is 'on its mettle' in a major isolated comparative exercise and that found when the comparison is under every day routine conditions. In the latter case individual samples can differ by more than 10 MPa (1,500 psi) and a consistent average difference of up to 2 MPa (300 psi) can be experienced over a long period. These matters have been reported by the author in two papers to ACI Conventions (Day, 1979, 1989).

A 2 MPa strength difference is equivalent to a cement content difference of between 10 and 20 kg/m^3 (17–34 lb/cu yd). A single testing laboratory may well be controlling a production of 10,000–100,000 cubic metres of concrete per month (from several plants). So that 'high' cost of a testing machine may be little more than the difference in the cost of cement requirement according to two different machines *per month*.

Testing machine technology

Obviously a correct result will not be obtained unless the stress is uniformly distributed over the test specimen (and any incorrectness in this respect will lead to a lower result).

An assumption is made that the faces of both the test specimen and the testing machine platen are absolutely plane and that the load will be applied concentrically. Quite small differences in planarity can make very large differences in contact area and therefore in stress distribution. With cube specimens this problem will worsen with older and higher strength specimens because the older concrete (i.e. 28 day rather than 7 day) will be more rigid, that is, less subject to plastic distortion. With cylinders the problem is different. Here the capping compound (e.g. where sulphur caps are used) will flow equally at any age. The platen planarity may be slightly less critical but any plastic flow allows stress concentrations to develop unless the original cylinder ends are very close to flat.

Spherical seatings are provided to allow one platen to rotate to compensate for any tendency for the two opposite faces of the test specimens not to be exactly parallel. This introduces its own problem in that, if the spherical seating were effective during the whole test, any eccentricity at all would lead to a bending moment in addition to an axial force, so reducing the failure load. *Therefore spherical seatings must be lubricated with a very light machine oil specifically so* *that the oil will break down under pressure* and allow the seating to lock solid after an initial adjustment. Extreme pressure lubricants, such as graphite grease, must be avoided as they will produce lower and more variable results. For cubes this is even more important because, since the specimen is tested perpendicular to the direction of casting (and therefore water gain or bleeding), its physical centre may not be its 'centre of resistance', that is, if the cube is stronger at the bottom than at the top, its centre of resistance would be displaced towards the previously bottom face when turned on its side for testing.

A further influence of the platen/specimen interface, again especially with cubes, is that friction provides a lateral restraint to the Poisson's Ratio spreading effect and so increases the test strength. The author (inadvertently) demonstrated this many years ago when he tested cubes coated with a wax curing compound. The compound may have increased the actual concrete strength but it certainly caused a drastically reduced load at failure. The reason for test cylinders to have a height/diameter ratio of 2 is to avoid this effect in the central area where failure actually takes place. This is probably the main reason for the difference between the test strength of cubes and cylinders from the same concrete. It may also be the reason why this effect is reduced at higher strengths (?). However, a further reason is that bleeding voids, which are more likely at lower strengths, may have a greater effect on cubes than cylinders owing to the different orientation during testing.

Bad concrete or bad testing?

The author was invited to give a paper on the above topic to the 1989 ACI San Diego Convention (Day, 1989). The paper has not been published (it is however now on the author's website), but the conclusions presented, and the fact that an ACI session organizer *requested* a paper on this topic, indicate that the question merits close attention.

The first half of the paper presented factual data showing that it is far from a reasonable expectation that a properly presented result from a reputable testing laboratory will be a necessarily accurate representation of the quality of the concrete. Examples were provided of individual differences exceeding 10 MPa, and consistent average differences of up to 2 MPa, in the results obtained by different registered laboratories testing the same trucks of concrete. It was emphasized that the laboratories concerned were NATA approved.

Pair differences exceeding 5 MPa were noted for apparently identical test specimens from the same truck of concrete tested by the same laboratory. Seven to twenty-eight day strength gains were also shown to be capable of $\pm$ 50% variation from sample to sample of concrete of the same mix design using the same materials.

The clear conclusion was that a strength test result is a totally unreliable piece of information. The audience awaited the author's proposal of some more satisfactory means of assessing concrete quality than a compression test.

The second half of the presentation showed that the very same data used in the first half could be analysed to show quite accurately when a genuine change in concrete quality occurred. Cusum graphs of 7 and 28 day strength showed downturns and upturns on exactly the same dates in spite of individual differences. The two laboratories showing the large differences on individual samples nevertheless agreed exactly as to when these change points occurred.

The overall conclusion presented was that an appropriate analysis of a series of test results can yield very reliable conclusions but that any individual test result should be regarded with great suspicion.

Some of the conclusions presented were:

1. Concrete producers are not so good that it is unnecessary to test concrete nor testing labs so bad that it is ineffective to do so.
2. There is no better complete replacement for traditional cylinder testing because it is the only way in which the combined effects of batch quantity variation, material quality variation, silt and dust content variation, air content and temperature variations, delivery delays and added water effects can be integrated.
3. We must cease to think of a single test result as an invariably accurate judgment as to whether a particular truck of concrete is or is not acceptable. In the first place it may well not be accurate, *and in the second we should show as much concern for those trucks we did not test as for those we did test.*

Rather we should regard the analysed pattern of test results as an important part (but only part) of the evidence we require in order to establish whether the totality of concrete being delivered to the project (or leaving the plant) is or is not of the required quality.

4. Before concrete of a particular grade is even ordered, it should be established that it is almost certain to be satisfactory. This may be done on the basis of trial deliveries, laboratory trials, analysis of past data or even just the reputation of the supplier. This assessment needs to take into account variability as well as mean strength. For an important project it may be inadvisable to obtain concrete from a supplier who cannot show either or both of substantial analysis of past data showing low variability and/or a computer batching plant which records the actual batched weights of every truck load delivered.
5. A particular individual (perhaps with assistants on a major or widely spread project) should have the responsibility of visually inspecting every truck of concrete and rejecting or further testing any suspect loads.
6. When a truck is sampled and test specimens cast, there should normally be at least three specimens. This is to permit an early age test and a pair of 28 day tests. The early age (not later than 7 days) is because any necessary mix adjustments must be carried out long before 28 day tests are carried out. The 28 day test is necessary to establish the current significance of the early age results. Two 28 day specimens are needed partly because the average pair difference is the best measure of testing quality and partly so that one can be brought forward to confirm or amend a low early age test result.

7 The sampling procedure should also include measuring and recording slump and concrete temperature, and also cylinder density on receipt at the laboratory. This is because such information is less expensive to obtain than the compressive strength yet at least doubles the value we can extract from it. Entrained air tests are also useful but this test is little more expensive so it is not invariably justified. J. M. Shilstone (Shilstone, 1987) has suggested that the fresh density of concrete may be a better quality indicator than slump. If taken it should certainly be combined with an air content determination, but it involves on site weighing equipment and it is not so simple to attain the required precision. Also it is not such a direct check on the relative water content of successive loads. It may be that hardened specimen density is sufficient providing that it is measured on receipt of the specimens at the laboratory (i.e. within 24 hours) and that it is immediately followed up by air testing when a significant density change is experienced. It may be that fresh density measurement is mainly of use if rejection of trucks is contemplated, but this should be abnormal.

8 The test results should be analysed to detect, at the earliest possible time, any departure from the previously acceptable concrete properties. This can best be done by drawing Cusum graphs of early age and 28 day results, slump, temperature, cylinder density, 28 day pair difference and early age to 28 day strength gain.

Such graphs are of substantial value not only in showing a strength downturn quickly and obviously but also in making it much easier to see whether the downturn is due to basic concrete quality, weather conditions, site abuse (excessive waiting time, water addition etc) or only the testing process.

9 It is very desirable to separate the functions of mix amendment and contractual acceptance. Mix amendment should take place based on early age results and can be reversed without excessive cost having being incurred if found unnecessary a few days later. It can therefore be done on relatively slender evidence. Contractual acceptance is best regulated by a cash penalty or cash bonus based on a statistical analysis of at least thirty 28 day results.

Physical rejection of hardened concrete, or even its further investigation by coring, etc., should be totally unnecessary if these recommendations are followed. One very desirable result of a cash penalty/bonus specification is that it avoids any need to argue about a possible mix amendment based on slender evidence at an early age. The decision can happily be left to the supplier as it is his penalty/bonus which is at risk rather than the structural integrity of the concrete.

The implementation of the above principles enables excellent control of concrete quality at very low sampling frequencies. The reduced volume of testing easily pays for the analysis but much larger savings are made by the elimination of disputes, investigations, delays to program, rejections etc. The paper certainly did not advocate a greater expenditure on control by adding the cost of elaborate analysis to the cost of the present level of testing. The proposal was rather to minimize the total cost of a given degree of assurance of concrete of a given minimum quality. This cost includes the necessary minimum cost of the concrete, any extra costs imposed by restrictive specification requirements, the cost of testing, the cost of test result analysis and any costs imposed by failures, including further investigation, partial demolition, legal costs, program delays and wasted time in meetings.

Rounding results

It is extremely bad practice in any technical field to fail to recognize and take account of the inaccuracies inherent in test results. One aspect of this is to avoid expressing results to more significant figures than their accuracy justifies.

In accordance with this various authorities *require* that certain test results be rounded. An example is the Australian NATA which requires that compression test results be rounded to the nearest 0.5 MPa (= 75 psi) and densities to the nearest 20 kg/m^3 (= app 1 lb/cuft). The author believes that this practice requires reconsideration.

Take compressive strength. Why should 0.5 MPa be selected? The answer is not that this is the order of accuracy, because different (competent) laboratories can easily differ by 2 MPa and *average* pair differences can exceed 1 MPa. Rather the answer is that in the days before computers were used, results were 'worked out' from tables and 0.5 MPa steps gave about as large a table as was convenient. The tables would have been five times as large had 0.1 MPa been selected.

The important question is what use is to be made of the test result. Originally the answer was to accept it as totally accurate and reliable and compare it to the specified strength. From this viewpoint it should certainly be taken as ± 2 MPa and so labeled.

It is bad practice to round calculations before the very last step. The strength of the individual specimen *used* to be the last step but now we have hopefully realised that this should no longer be the case. Action on compressive strength results should always be based on the analysis of groups of test results, effectively ignoring individual results. So it is the mean and standard deviation of a number of results that has significance. It would be better to use less rounded results, but it may not make a great deal of difference. However when analysing (as we should) such items as within sample ranges (based on average pair differences) and 7 to 28 day strength growth, rounding to 0.5 MPa is fairly obviously unsatisfactory.

It is proposed for compressive strength that it be expressed to 0.1 MPa and given the written qualification '± 2 MPa' where appropriate. This (apart from the ± 2 MPa) will not consume any more paper and will marginally reduce the computer program.

For density a similar situation exists. It is not so much the absolute density of a single specimen which should be of interest, but the range of densities of all specimens from a single sample of concrete (since this will reveal the competence of the specimen casting and enable its variation to be monitored). Detecting any change in the average density of concrete being produced, that is, of a group of samples, is the major reason for the test.

The proposal for density is that it be expressed as a four-digit integer, since again this takes marginally less computer effort and no more paper. The accuracy limits in the case of density may be much different for different organizations. For those to whom it matters, their control system will be providing a within sample standard deviation. Density may be unlike strength in that small variations in assessment of the same concrete by different laboratories is probably unimportant. Detection of change in average density or change in within sample variation are probably what matters.

Cubes v cylinders

The world is divided as to whether it is better to assess concrete strength by cube or cylinder specimens. The UK, much of Europe, the former USSR and many ex-British colonies use cubes, USA, France and Australia use cylinders.

The advantage of cubes is that they are smaller and do not require treatment (capping) prior to testing.

The advantage of cylinders is that they are less dependent upon the quality and condition of the moulds and that their density can be more readily and accurately established by weighing and measuring.

Both proponents naturally feel that the specimen with which they are familiar is preferable. The debate should be settled on the basis of which gives the most accurate (i.e. repeatable) result. This is best judged by the average pair difference achievable, or the average range of three. Either of these can be converted into the *within sample (sometimes called within test) standard deviation*. In the case of pairs the average pair difference is divided by 1.13 to obtain the within sample σ. For the average range of sets of three, the divisor is 1.69.

The author received his initial concrete QC experience in the UK on cubes and has owned and operated testing laboratories in Australia using mainly cylinders and in Singapore using mainly cubes. Both specimens are perfectly satisfactory and capable of very low pair differences if used carefully and cast in well-maintained moulds. The problem is that the test specimens must be prepared in the field by relatively low-level technicians. The quality of training provided is crucial and is often inadequate. The really basic fault is often that the persons training the technicians have inadequate knowledge, practical experience, or dedication to the task.

Capping used to be something of a problem with cylinders, although more of an initial than a continuing problem. Once the proper equipment is obtained and the operator has gained experience, capping was never much of a problem. The capping referred to is the use of a molten sulphur mixture to achieve a smooth test surface on the end of the cylinder.

The essential items are:

1 A heavy, accurately machined steel mould into which to pour the sulphur mixture.
2 A guide along which to slide the cylinder to ensure the cap will be perpendicular.
3 A thermostatically controlled melting pot in which to heat the sulphur mixture.
4 A scoop holding an exactly suitable amount of the mixture to produce a cap.

There are a number of difficulties to be overcome by the uninitiated:

1 Neat (undiluted) sulphur is not suitable because it shrinks too much and sets too quickly. A mixture with finely ground silica, fly-ash or other inert material should be used. Proportions are trial and error, depending on the particular sulphur and the particular filler. Some like to include a proportion of carbon black. Commercial blends are available.
2 The temperature of the mixture must be 'just right', too cool and it will not flow and sets too quickly giving a thick cap, too hot and it goes rubbery and shrinks too much. Again it is trial and error.
3 The first cap is difficult because the mould is cold, later the mould gets too hot and causes delay waiting for setting.
4 The mould must be very lightly oiled between each use.
5 The cap must be thin, preferably only 2–3 mm.
6 Especially for high strength concrete, a sulphur cap will not overcome a rough cylinder end. The cap will exhibit slight plastic flow under load and allow load concentration on high spots.
7 The hot sulphur emits fumes and requires at least an exhaust fan and preferably a fume hood.

All the above makes it quite clear why users of cubes are not tempted to turn to cylinders but has no bearing on the question of which is the more reliable test.

A significant improvement is that of rubber caps. Instead of sulphur capping, the cylinder is simply fitted with a rubber pad restrained in a metal mould. A suitable side clearance is essential since, under the high pressure, the rubber behaves almost like a fluid. If the clearance is too great the neoprene will be extruded and will provide excessive side restraint. The mould is illustrated in Fig. 11.1.

A relatively recent development which could be very important is a new capping technique called the 'Sand Box', although the author has heard no more of this development since including it in the previous edition. The test was developed by Claude Bouley and Francois de Larrard and was reported in Concrete International (Bouley and de Larrard, 1982). The 'box' in question is a circular cup, very similar in appearance and function to the restraining ring used in the rubber cap test but deeper (30 mm). The rest of the apparatus is a positioning

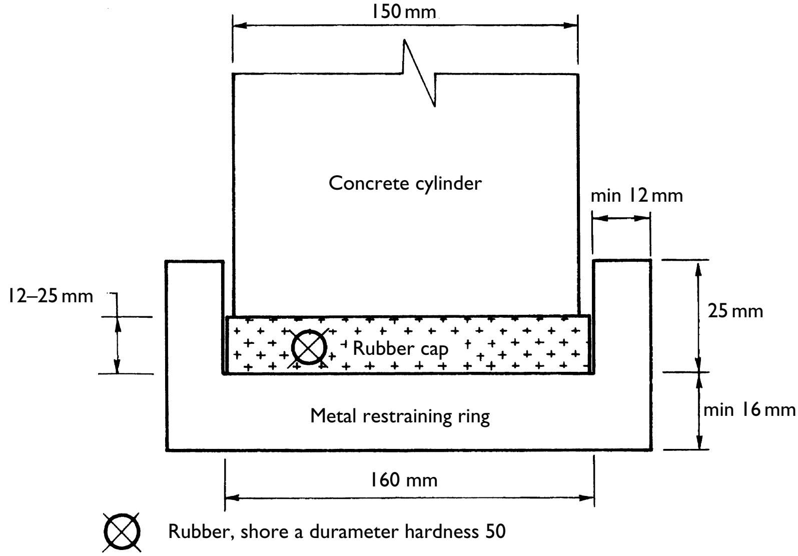


Figure 11.1 Rubber cap and restraining ring.

frame and guide similar to that used in sulphur capping, except that a small, air driven vibrator is incorporated. The technique is to place a 10 mm layer of dry sand in the cup, position the cylinder in the frame and vibrate so that the cylinder compacts the sand (20 seconds). The cylinder is then sealed into the cup by filling around the periphery with molten paraffin wax.

The test may initially look unattractive compared to sulphur or rubber caps since it involves a capping process with molten material, a vibrator and does not permit re-use of the mould before testing. However, it does not appear to involve as much manual dexterity as sulphur capping, avoids sulphur fumes and permits immediate testing of a prepared specimen. It uses only sand and recyclable wax and so should be inexpensive in use. More importantly, it appears to give test results on very high strength concrete only slightly less reliable than the best achievable by end grinding and much better than even slightly sub-standard grinding. The trials have included successful use on extremely rough cylinder ends that would have had to be sawn off before any other technique could have been used.

The use of large aggregate concrete, except for special uses such as dams, is becoming rare. For high strength concrete, aggregate with a maximum size of more than 20 mm (3/4 inch) is a disadvantage and for very high strengths a smaller size still, 10–14 mm (3/8 to 1/2 inch) gives better results. Therefore previously used specimen sizes of 150 mm (6 inch) cubes and 150 diameter × 300 mm long cylinders can be replaced by 100 mm cubes and 100 × 200 mm cylinders.

Table 11.1 Cube/cylinder strength conversion

Concrete grade	*Compressive strength at 28 days MPa (N/mm)*	
	Cylinders 150 mm dia. × 300 mm	*Cubes 150 mm × 150 mm*
C 2/2.5	2	2.5
C 4/5	4	5
C 6/7.5	6	7.5
C 8/10	8	10
C 10/12.5	10	12.5
C 12/15	12	15
C 16/20	16	20
C 20/25	20	25
C 25/30	25	30
C 30/35	30	35
C 35/40	35	40
C 40/45	40	45
C 45/50	45	50
C 50/55	50	55

Some researchers consider that the smaller specimens will give higher strength (up to about 5% higher) and greater variability. Others find that smaller cylinders give *lower* variability, but the differences are not sufficient to concern us unless they affect a comparison between different laboratories.

Whilst considering such matters, reference must be made to the cube/cylinder ratio. A previous British Standard nominated this ratio as 1.25 for all circumstances but this is not the author's experience, which is that the ratio varies from over 1.35 to less than 1.05 as strength increases. A formula giving results in accordance with the author's experience, but not claimed to be thoroughly established, is:

$$\text{cube strength} = \text{cylinder strength} + 19/\sqrt{(\text{cylinder strength})}$$

or

$$\text{cylinder strength} = \text{cube strength} - 20/\sqrt{\text{cube strength})}$$

where cube and cylinder strengths are both in MPa or N/mm^2.

Table 11.1 gives an alternative version that has greater official standing.

The smaller cylinders, which weigh around 4 kg rather than 13 kg for the larger ones, are much easier to handle and cap.

11.4 The maturity/equivalent age concept

Concrete gains strength with age. It also gains strength more rapidly the higher the temperature. It is desirable to establish a relationship between strength, time and temperature so that the strength of a particular concrete after any particular time and temperature cycle can be established from a knowledge of its strength after any other time and temperature cycle.

There have been two attempts to achieve this and both are detailed in ASTM C1074. Although the two terms 'maturity' and 'equivalent age' are sometimes used in a qualitative way as interchangeable, they each have a precise meaning in numerical terms.

Maturity is the age of a particular concrete expressed as degree-hours, that is, as the area under a temperature-time graph.

Equivalent age is the age at which a particular concrete would have developed its current strength if maintained at a nominated standard temperature.

Both of these definitions are incomplete in that the base temperature in the case of maturity, and the standard temperature and an 'activation energy' in the case of equivalent age, remain to be nominated.

The maturity (or '*TTF*' = *time temperature function*) concept was developed in the UK in the 1950s and is generally attributed to Saul (Saul, 1951) or Nurse (Nurse, 1949). The base temperature should theoretically be that temperature at which concrete does not gain strength. This is often taken to be either +10°F or −10°C or as +11°F or −11°C (which are almost the same temperature). It is also often taken as 0°C for convenience, although concrete does gain strength at 0°C (but see Figs 11.2A and B and associated explanation for selecting a different value).

The equivalent age ('EA') concept is older and more accurate, but also more complicated. The concept was not originated specifically for concrete but as a general concept for all chemical reactions. The general formula is attributed to Arrhenius (Arrhenius). The concept was applied to concrete in the 1930s in USSR in the form of coefficients by which the length of time at each temperature should be multiplied to give equivalence.

The relationship is exponential and is given by the formula:

$$\mathrm{EA} = \sum\left(te^{-Q(1/T_a - 1/T_s)}\right)$$

where:

EA = equivalent age (hours)
Q = activation energy divided by the gas constant
T_a = temperature (°K) for time interval t.
t = time (hours) spent at temperature
T_s = reference temperature (°K = C+273).

The reference temperature (T_s) is the standard curing temperature at which test specimens are kept. In many parts of the world it is 20°C (293K) in Australia it is 23°C in temperate zones and 27°C in tropical zones, it may be that 30°C would be appropriate in some tropical countries (if this is the average temperature of unheated curing tanks). The Q value can range from below 4,000 to over 5,000 depending on the characteristics of the particular cement. It is often taken as 4,200.

A discussion of the relative merits of these two approaches follows, but it is important for the general reader not to get lost in the detail and worried about minor pitfalls, but to realise that the basic concept is very simple and enables powerful solutions to two problems:

1 Prediction of 28 day strength from an early age test.
2 Establishment of the strength of in situ concrete.

Previously problem (1) was approached by setting down a fixed accelerating (heated curing) regime and experimentally determining a correlation curve.

Problem (2) used to be handled by setting a time, such as 7 or 14 days before some activity such as stripping, de-propping, stressing, lifting was permitted. Alternatively 'field cured' specimens were used, assuming that cylinders cured alongside in situ concrete would have a similar maturity. This of course is very far from the truth.

Almost any sort of rough application of any maturity approach is vastly superior to these 'old fashioned' solutions.

An initial approach to implementing the 'new' concept was to construct a strength – maturity or strength – equivalent age curve experimentally in the laboratory. Having logged in situ temperatures, either the maturity or equivalent age could be determined at any time and the corresponding strength read from the graph. The weak aspect of this technique is the basic assumption that the in situ concrete is of identical strength to that previously used to construct the curve. There are two problems with this, one is that concrete is a variable material so that identical mixes are subject to a spread of results. The other is that there could (hopefully rarely) be a substantial problem with batching or quality of materials that would not be picked up by this approach.

Fig. 11.2A and B provided, along with valuable discussion, by Dr Steve Trost, Director, R&D for Strategic Solutions International, LLC (see http://ssi.us).

The question of the accuracy of the two rival approaches (TTF and EA) arises. It seems to be generally conceded that the EA function is correct in that the effect of temperature is exponential rather than linear. However, the opponents of using this approach point out that very large errors can result from an incorrect value for activation energy, whereas TTF is conservative. This is illustrated in Fig. 11.2A and B which compare the two concepts and also the effects of varying the activation energy in EA and the datum temperature in TTF. The comparison is also affected by the standard curing temperature in use, in the illustrated cases this are 23°C and 40°C. This is the temperature of the water bath or fog room in which the specimens used to establish the standard maturity curve are kept. The assumption is that the true curve is between the two estimates of the Arrhenius curve and it can be seen that substantial error could occur if the wrong one were chosen. It can also be seen that using the 'correct' datum of −10°C for

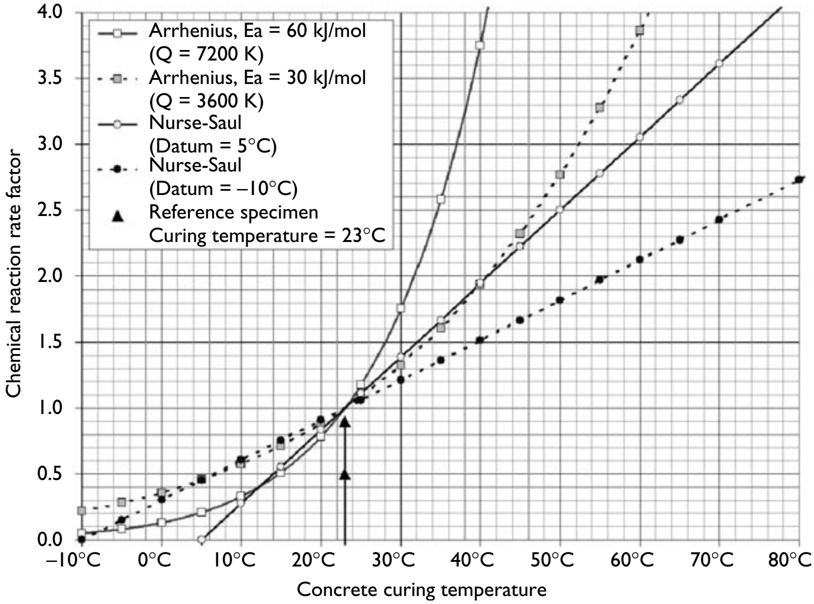


Figure 11.2A Graphical comparison of maturity and equivalent age functions (23°C).

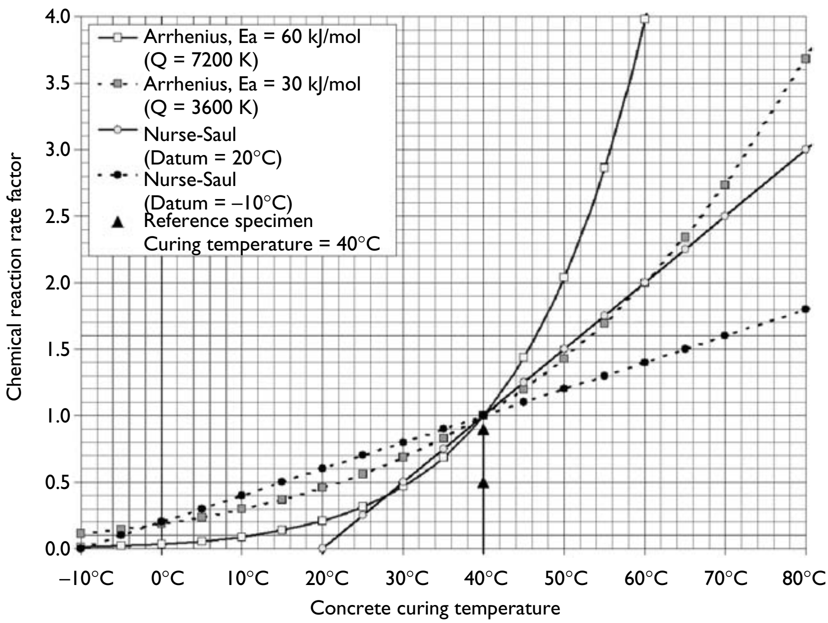


Figure 11.2B Graphical comparison of maturity and equivalent age functions (40°C).

TTF is unconservative for temperatures below the standard curing temperature and over-conservative for higher temperatures. Changing the datum to +5°C, while not theoretically correct, both avoids unconservative readings at low temperatures and reduces the over-conservatism at higher temperatures. If the reference specimens are to be kept at any other temperature than 23°C then Trost recommends that the TTF datum should be set at between 18 and 20°C below the reference specimen temperature, specifically 5°C for 23°C and 20°C for a 40°C reference temperature. The reason for, and effects of, this can be seen by comparing graphs 11.2A and B. It is unlikely that anyone would use a reference temperature of 40°C (however 27°C or even 30°C is normal in tropical countries) except possibly in the case of steam curing. However, it can be seen from Fig. 11.2B that in the steam curing case, the TTF assumption, even with the 40°C reference temperature, would be likely to be underestimating true maturity by a factor of around two (with an average activation energy of say 4,200 and a steaming temperature around 80°C). This would be a substantial disadvantage in heating cost or cycle time and the only competitive alternative to using the author's EA system would be temperature matched curing (TMC). This may be a reasonable solution in a precasting factory but not otherwise. However TMC would not provide an early prediction as would the author's system and the TMC equipment would be more of a hindrance to production.

The author's approach to the concept was from the viewpoint of the other problem, the prediction of 28 day strength from early age tests. A paper by (Guo Ghengju, 1989) suggested that using EA directly for this purpose did not work very well, but that a good prediction of 7 day strength was obtained. This is not surprising, given that the author has clearly established that predicting 28 day strength as a percentage of 7 day strength does not work very well either. The author's concept therefore was to use EA to predict 7 day strength and then, as in his normal control system, to predict 28 day strength by adding the average gain for 7 to 28 days (a figure already in, and continuously and automatically updated, in his control system).

If the relationship between strength and EA is truly exponential, then a graph of strength against log(EA) will be a straight line, regardless of the activation energy. The slope of this line, the Q value in the EA formula, could be determined by entering any two results. In general, the average 7 day result is likely to be already in the system and being continuously updated. So testing a specimen at any early age, the slope of the graph between this point and the 7 day result can easily be determined. When the actual 7 day result from the same batch of concrete becomes available, the slope between these two results from the same batch can be substituted for the initial value and used to project the result from the next early age test to give a 7 day prediction and thereby a 28 day prediction. It was simple to write a program to automatically average all the Q values obtained in this way. The program also graphically displays all the slopes (Fig. 11.4) so that it can be seen whether a consistent value is being obtained. Also it is very obvious if one or two results are clearly in error and these can be deleted. The system can also display a graph (cusum or direct) of the variation of the *Q* values so obtained, in the normal ConAd QC system, permitting a consideration of what factors might be influencing any observed changes in *Q* value. So the system provided an apparently foolproof means of applying the EA concept.

As often happens in concrete technology, things were not quite as simple and foolproof as appeared theoretically likely. Some inconsistencies were experienced and, in investigating them, many graphs of strength against log(EA) were drawn with multiple specimens from a single mix. It was found that, in all cases, the results formed not a single straight line but a broken straight line that is a straight line with a single change of direction somewhere along it. The change has subsequently been detected at anywhere between 2 and 7 days, and can occur in either direction (although almost always from an initially steeper slope to a later shallower slope). It is well-realized that the hardening of concrete is not a single chemical reaction[1] and what this means is that two combinations of reactions with different activation energies are involved at different stages of the hardening process.

It is immaterial to our purposes what happens after 7 days, since this is covered by the addition of the average 7 to 28 day strength gain for the particular mix. However a change of slope prior to 7 days would mean that the slope of the line joining an early result and that at 7 days on the strength v log(EA) graph would be affected by the particular early age. It was seen that the requirement for satisfactory operation is that the graph should be a straight line up to a predicted 'control age' and that the amount to be subsequently added should be the average strength gain from the control age to 28 days. Therefore it is recommended that, for any new mix, a number of specimens are taken from a single batch of concrete, tested at a range of ages, and plotted on the strength v log(EA) graph (Fig. 11.3) to determine the age at which the slope changes for that particular mix. This of course is quite different to the practice of using a pre-determined strength v maturity graph. There is no suggestion that subsequent mixes will have the same strength at the same EA, only that the rate of strength development will change at the same EA.

If the change in slope occurs at later than 7 days, then it will be convenient to continue to use 7 days as the control age, it is only when the change occurs earlier that the control age must be changed to get accurate results.

Having selected/established the control age, one specimen is always tested at the control age and at least one at some earlier age. The ConAd program automatically calculates the slope of the line joining these two points (we call this the *K* value) as the results are entered in the normal Test Result Entry system (but having selected early age in the set-up program). The program continuously averages all previous *K* values for the grade in question as each new result is entered (Fig. 11.4).

Using the previous average *K* value, the control age strength is predicted as soon as the early result is entered. When the true control age result is later obtained, the true *K* value for that sample is evaluated and included in subsequent averages.

1 Farro Radjy has used his 'heat signature' technique to quantify the proportions of different chemical compounds present in a cement by their different rates of heat generation.

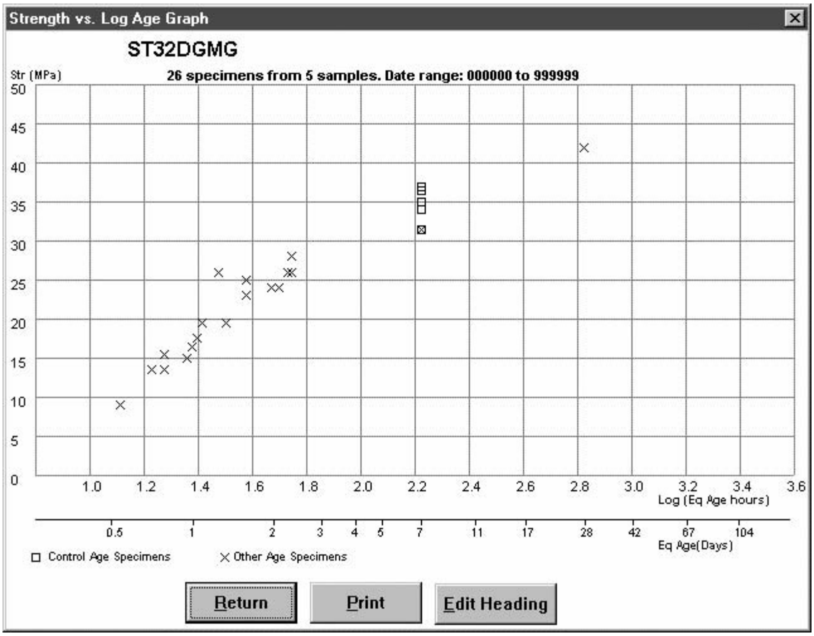


Figure 11.3 Strength v log equivalent age graph.

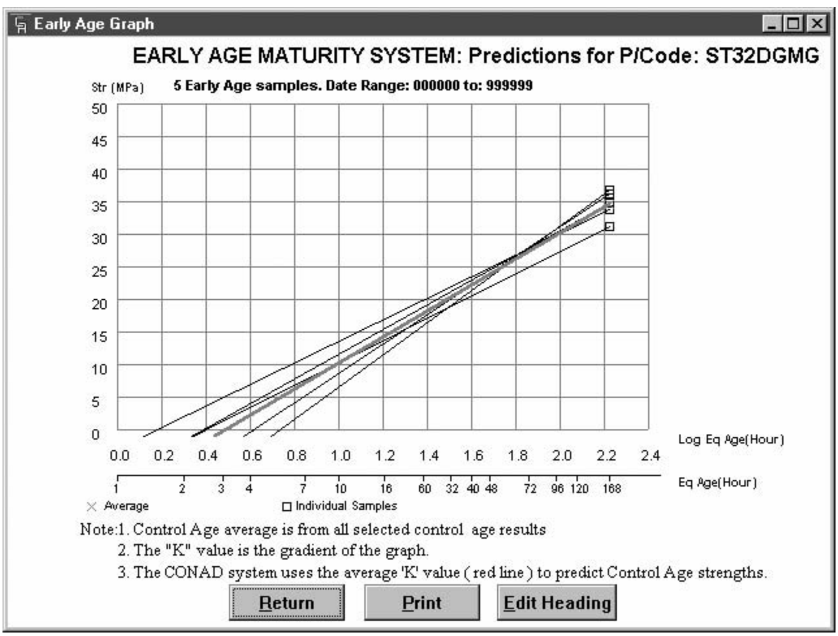


Figure 11.4 Automatic updating of *K* value (slope of strength (MPa) v log equivalent age (hr) graph).

The system can display and print out the graphs and these can be used to establish the age at which any particular strength will be attained or the actual strength at any particular age. However it is simpler and more accurate to use dummy results in the Test Result Entry system. The nominated strength is entered and the age varied until the previous average control age (and 28 day) strength is predicted. Alternatively the nominated age is entered and the strength entry varied until the anticipated eventual strength is predicted.

An important use for the graphs included in the early age section is to check that the results do conform to a reasonable pattern. If Fig. 11.4 shows some lines of distinctly different slope, then a problem exists. If no bias or particular period causing this can be found, then it may be due to testing error (the error can be either in the strength or the equivalent age of the early result). *K* values can be plotted in the normal QC system and it can be seen which results are abnormal. Such results can easily be excluded from the analysis by finding them in the table of all results (in 'Full View' in the ConAd program) and so labelling them.

Fig. 11.5 also shows clearly how much scatter of results there are and whether they are scattered about the correct line. For example the illustration chosen here does show some tendency to change slope at earlier than the seven days selected as a control age.

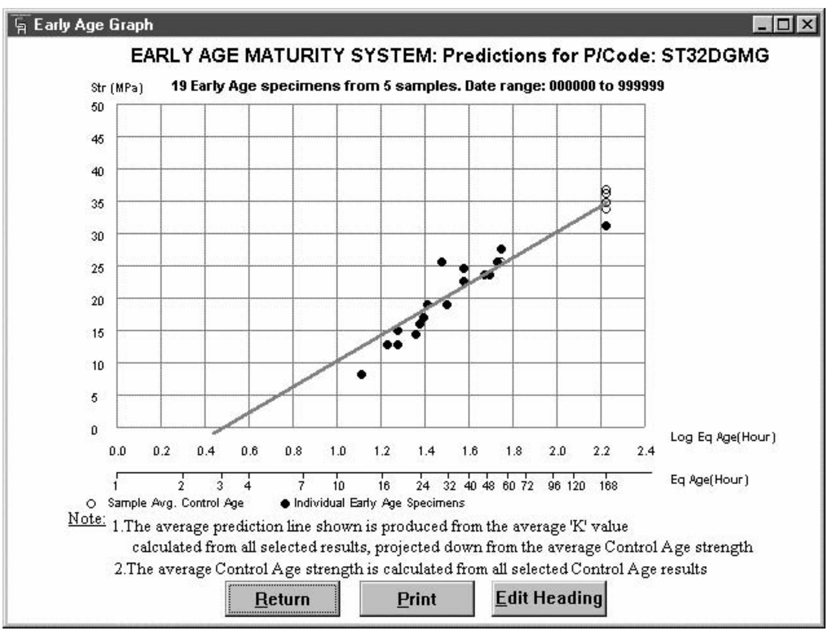


Figure 11.5 Early age specimen results.

Fig. 11.3 is the kind obtained with multiple test ages and again shows a change at two days rather than the 7 day used. The error due to this assumption is quite small in this example but can be larger in other cases.

Limitations of the equivalent age concept

Concrete which has been heated:

(a) too early
(b) too rapidly
(c) to too high a temperature.

Will attain a lower 28 day strength than the same concrete cured at normal temperatures. The limiting values to avoid such problems differ for different cements and especially for different combinations of pozzolan and cement. It does not follow that routines which involve a loss of 28 day strength should not be used, only that the loss should be understood and allowance made for it if necessary.

It can be anticipated that concrete containing a pozzolan or ggbfs will withstand higher curing temperatures without loss of potential 28 day strength. Such concretes may show an increased 28 day strength through higher temperature curing.

Any particular curing regime for any particular concrete can be readily checked by comparing the strength v logarithm of equivalent age curves for heated and normally cured test specimens. As a rough guide, a delay of 2 equivalent hours at 20°C, a rate of rise of 0.5°C per minute and a maximum temperature of about 70°C will usually avoid any significant loss of 28 day strength when using normal Portland cement.

Carino (Carino, 1984) concluded that a parabolic relationship may be simpler to use and equally, or even more, accurate than the Arrhenius relationship. We have not experimented with such a relationship since it is easier to continue using the Arrhenius relationship now that it has been incorporated in a user-friendly computer program.

Low temperature application

It is necessary to protect concrete from freezing and thawing damage, and also from dehydration, until it has attained a critical strength beyond which further protection is not essential. This has been recognized for many years and various national codes have laid down specified periods of protection. In some cases the protection period is varied according to ambient temperature but much greater precision and flexibility is now feasible by defining the protection period in terms of measured equivalent age or of in situ strength determined from equivalent age.

Temperature cycles and stresses

The author is unaware of any definitive work on the subject (other than relating to freezing and thawing) but the subject of temperature variation should be considered. It is well known that the number of cycles of freezing and thawing rather than the lowest temperature reached is the significant parameter in frost damage. This means that more such damage may be experienced in a marginal climate where concrete may freeze and thaw 50 or 100 times per annum than in a much colder climate where concrete may remain solidly frozen for several months.

Hearsay evidence suggests that a similar situation may occur at high temperatures, although to a different extent. Thus a concrete specimen that is cast hot and stays hot until it attains substantial strength, or is heated and stays heated, may be less damaged than one that is cast hot and taken into an airconditioned laboratory.

The possibility is that changing temperature may cause bond stresses at the paste/aggregate interface and/or microcracking in the paste or mortar fractions. It seems likely that such events would have greater significance for tensile and flexural strength, and for durability, than for compressive strength. A thermally caused reduction in compressive strength may be the tip of an iceberg in terms of total resulting damage.

Update on maturity/early age

As predicted in the first and second editions, maturity monitoring has become more popular as the principle and economic benefits are more widely understood and accepted, and as instrumentation becomes more sophisticated and affordable.

What is surprising is that the less sophisticated, degree-hour maturity concept (abbreviated to TTF or temperature-time function) is more frequently used than the more scientifically valid Arrhenius Early Age (EA) concept. This is apparently due to three factors:

1. TTF is easier to explain and much easier to calculate.
2. The determination of the activation energy, as set out in ASTM C1074 is an onerous process and substantial error can result if undetected changes in this occur.
3. The author's concept, described above, of a continually automatically updated constant in the log EA v strength relationship (avoiding the need for prior calibration), while used enthusiastically on diverse projects in several countries by ConAd licensees for over a decade, has yet to be adopted (or possibly comprehended?) by anyone else.

Proper concern has been displayed as to whether recorded temperatures are accurate (thermocouples v thermistors), and whether the EA constant is accurate, but most commercial systems (all other than the author's ConAd?) have been prepared to work on a pre-determined strength-maturity curve. However in evaluating early age strength it seems not to be generally understood/realized/allowed for that concrete is a variable material. Although concrete can be produced with a compressive strength standard deviation of 2 MPa (300 psi), unsophisticated producers may easily experience a figure of triple this. So the 28 day strength of a batch of concrete can vary by 1.28 (or 1.65 depending on country) $\times$ 6 = say 7 to 10 MPa or 1,100–1,400 psi. Generally the higher the strength of a concrete and the larger the percentage of that strength developed at a given early age, so early strength, at least as a percentage of average early strength, could be expected to vary at least as much as later strength. So an early age strength might be expected to vary by say +/− 2 MPa or 300 psi at a given maturity/equivalent age (even if no batching or other errors occur). This puts concern over the accuracy of equivalent age determination in a proper perspective.

There are now a number of instruments on offer that will log temperatures and even calculate maturity or EA within the concrete. This obviously avoids the problem of theft or damage of the recording device at the cost of it being sacrificial. Some such devices even incorporate radio transmission so that not even wire access is necessary. Again, this is an advance in convenience at an increased cost. In general the cost of physical equipment tends to reduce with time while their efficiency and the value of convenience is perceived to increase. On this basis it is a reasonable assumption that such devices will become more popular.

To date the author's ConAd system has used two DataTaker instruments. One is the single channel DT5. This instrument is very small and very robustly constructed but does not have any display panel. It is ideal for fixing to a cylinder mould with its single thermocouple wire inserted in the cylinder. On arrival at the lab the cylinder can be placed in the curing tank and the DT5 hooked up to the computer for reading. If desired the cylinder and DT5 can be placed together in the tank to check how long it takes for the cylinder to reach the curing tank temperature (usually less than 30 minutes with a water tank but perhaps much longer in a fog room). It is also possible to place the cylinder in warm water to expedite strength gain (still measuring the temperature) but accurate predictions have been obtained from cylinders as low as 2 MPa (300 psi) at the early test age so heating is rarely needed. An exception may be for precast prestressed units where detensioning at say 16 hours after a steaming cycle may be involved. In such a case it is not at all necessary that the test cylinders go through the same steaming cycle but it may be desirable to heat them enough to obtain a prediction prior to discontinuing steaming the units. Substantial expense can be saved by knowing precisely when this can be done.

The second instrument is a DT50. This is a five channel instrument encased in a strong container which can have a display panel. It is also programmable and there is a program to display equivalent age (and even actual strength at any time) and fitted the instrument with switches to enable each channel to begin recording independently of the others. This instrument also uses very economical thermocouple wire probes. Some users in the past have had these instruments locked away in a shed with thermocouple wires trailing out up to 50 metres to be embedded in precast units or in situ concrete.

Now Hanson, one of the early ConAd clients, have modified the latest DT50 to include a mobile phone, so that they are able to telephone it and it can provide updated temperature/time logging without human intervention. The contractor is able to contact Hanson for an update at any time if required, but generally is simply advised by Hanson (who never visit the site) when the concrete has reached the strength they specified. Alternatively some clients prefer to receive an update on all current strengths each morning in order to plan the day's work. Hanson report that, in spite of our initial advice to the contrary, they have found it very satisfactory to solder the thermocouple junction after twisting together and have not had problems with damage or theft. However, their thermocouples are now pre-prepared by one specialist sub contractor.

There are many factors to be considered in choosing equipment. Principal amongst these is confidence in the knowledge, ability and good faith of the marketer and, as consequence, the acceptability of the results to supervising authorities. Unconservative assessment is one aspect of the risk and unreliability of the equipment is another. Either of the EA and TTF methods can be made to give satisfactory results by a knowledgeable operator using any of the available equipment. However some equipment requires greater skill, care and understanding than others and this can be involved/provided in different ways and at different stages. Decades ago, the author achieved satisfactory results by personally making and installing thermocouples and assessing results. Any faulty readings were recognized as such and discarded. Judgments on readiness for stressing etc were made in a full knowledge of current test results and circumstances and past performance and with appropriate safety margins. This does not mean that the methods used would be satisfactory applied by the average site worker or an inexperienced young engineer.

It is difficult to generalize on the economics of alternatives.

1 In most cases the savings made from the information gained far outweigh the cost of the testing. To this extent whatever it takes to satisfy authorities is worthwhile.
2 Also, in many cases the results reveal a substantial margin between the strength developed and that necessary for the purposes envisaged. In such cases a large margin can be allowed for inaccuracy.
3 The cost of personnel is often a major factor. They may be involved in pre-assembly, calibration, installation, reading, evaluating results and equipment recovery. The level of skill and ability required varies significantly between different equipment and different work scenarios.
4 The number of probes installed may be influenced both by their perceived reliability and the consequences of an occasional failure.
5 The risk of damage or theft of external equipment will vary extremely between different working scenarios (e.g. site or precasting factory) and even different countries and locations.

6 The curing situation, varying from in situ slabs in winter to steam cured precast units, may be an overriding factor.

There is clearly a need for one or more kinds of certification but this also may not be easy to arrange. One kind is the training and certification of operators by equipment providers. Another is the certification of equipment providers (as opposed to particular equipment). However, it is not clear who would be sufficiently competent and independent to provide such certification. It would be important not to introduce regulation that could rule out satisfactory solutions.

Section 4.14 gives details of the use of early age data in the ConAd QC program. While this program can display the graphs described above, it is not necessary to use them in the normal course of events, except for checking purposes. Entry of a strength and its associated EA in the normal QC program provides predictions of 7 and 28 day strengths and a method of predicting the strength at any desired EA or the EA at which any nominated strength will be attained. For steam curing situations, the user is able to nominate maximum and minimum estimates of the decline of temperature enabling the system to advise when steaming can be switched off to provide a specified strength at a nominated actual time.

11.5 Permeability testing

The original Figg tests originated in the UK but have subsequently been neatly combined into a single instrument by James Instruments in USA. A hole is drilled into the concrete (which may be in situ concrete or a test specimen) and a plastic plug inserted to create a cell below the surface of the concrete. A hypodermic needle is inserted through the plug to provide access. The first test involves applying a suction to the cell so as to draw in air through the surrounding concrete. The (very small) volume of air is measured by the movement of mercury in a tube through which the suction is applied. The second involves filling the cell with water and using movement in the same tube (but in the opposite direction) to measure the rate at which water is absorbed into the surrounding concrete.

The Wexham variant identifies two problems sometimes encountered with the above test. One is that air permeability is substantially affected by moisture content. The other that air may be entering via defects in the concrete or a leaking plug rather than via permeable concrete. These two potential problems are solved first by using a slightly larger diameter hole and including an instrument to measure humidity in the hole. Second pressure rather than suction is employed so that any leaks can be detected by bubbles in a soapy water film on the surface.

An additional advantage of these kinds of in situ test are that they can be used to measure the adequacy of curing (which has a large effect on permeability). Potentially a contractor could be required to continue or resume water curing until an acceptable permeability is achieved.

11.6 Non-destructive testing

With non-destructive testing (NDT) it is necessary to be particularly careful to clarify the objectives of the testing and the assessment of the results. Clearly the strength of the concrete in the structure is not necessarily the same thing as the potential strength (according to a standard compression test) of the concrete as it leaves the mixer or delivery truck. If it is not clear which of these is being sought, it is unlikely that the relative merits of different testing procedures will be correctly assessed.

From one viewpoint, the strength of the concrete in the structure is what really matters. However, even if this is accepted, we still have to consider whether what matters is the *current* strength of the concrete in the structure or its *eventual* strength. If the requirement is to assess readiness for early stripping or prestressing, or termination of curing protection, then the current strength is the more important. If it is the load carrying capacity of the structure, or its durability, then the eventual strength will probably be more significant.

If the intention is to regulate the proportions of the concrete mix currently being produced, it is equally not obvious whether the potential standard specimen strength or the current actual strength in the structure is what matters. If considerations of eventual strength and durability in a particular structure require a 30 MPa (4,350 psi) strength but construction efficiency requires 22 MPa (3,190 psi) at 22 hours for prestressing, then the latter requirement will clearly rule. If day to day temperatures vary very widely (as they do in parts of Australia) then it could be necessary to supply concrete of 40 MPa (5,800 psi) 28 day strength one day and 60 MPa (8,700 psi) 28 day strength the next. Of course it is always possible that it is economically preferable to supply 60 MPa throughout, rather than complicate the situation, but this option can be ignored for the purposes of this example.

In the more usual case, a particular concrete mix will have already been assessed as suitable for its intended purposes and testing will be being undertaken only to determine when any change takes place in that mix. In this case any extraneous factor that affects the test result, such as variable compaction of the test specimen, or variable temperature, either of the supplied concrete or of the specimen during curing, will add to apparent variability and so reduce the efficiency of the control process.

Assessing the above range of possibilities, it appears that the only case in which NDT testing could be considered as a total replacement for typical compression testing of standard specimens is where an early age requirement ensures such a large excess of 28 day strength that control of that strength is unnecessary. Even in this circumstance, standard testing may still be desirable if any problems are encountered, as otherwise it may be difficult to establish whether the problems are mix problems or usage problems. To some extent the decision would depend on the quantities of concrete involved since the cost of control measures may be to a large extent 'per pour' whereas the cost of providing excess strength to avoid or reduce control is definitely per unit volume of concrete. Thus if a few small units totalling say 1 cubic metre of concrete per day were involved, it would be economical to use an excessively high strength and do little testing of any kind. However, if 200 cubic metres per day were used in floor slabs to be prestressed at an early age, both specimen testing and some form of insitu testing would be obviously justified.

An important consideration is that it is not only the accuracy of a test that matters but also its relevance and the accuracy of the assumptions made in evaluating it. For example a test cylinder left on an in situ slab may give a very accurate strength but may have a very different maturity and therefore a very different strength to the slab itself. A pullout test on the same slab may be much more variable but at least it is measuring the actual strength. A standard test cylinder combined with a maturity (e.g. equivalent age) measurement of both the cylinder and the slab *may* be more accurate than the in situ-cured cylinder, and as relevant as the pullout test, but it does depend on the accuracy of the *maturity/strength* correlation and, for example, the compaction of the slab. An ultrasonic test would also be very relevant and may be quite repeatable and accurate but would be totally dependent on the strength/velocity relationship assumed, which would be affected by such factors as moisture content.

The reader is referred elsewhere (Bungey) for further details of various NDT tests but the author certainly sees a place for such tests in the overall control operation. Particular examples are pullout tests on suspended floor slabs prior to early stripping or stressing, and Schmidt Hammer tests on freshly stripped columns. The latter is not a very accurate test (especially if used informally rather than according to the manufacturer's routine) but it is an extremely quick and cheap test which could be used on every column as it is stripped and would give early warning of any severe problems. It has even been suggested that the test could be worth performing even if the strength scale is not read. The implication is that the depth of indentation, or even the sound of the impact, would alert a daily user to any drastic problem. The author found this to be the case with spun concrete pipes, where sound was a good indication and the process could be compared to tapping the wheels of railway carriages to detect cracks. However, a thorough examination by a US university team (Telisak *et al.*, 1991) concluded that in situ maturity determination was the most accurate criterion of early age strength.

When regular NDT tests are carried out it is very desirable to enter the results in the control system for graphing and analysis alongside the other test data. Such action will soon establish the extent to which the variation of strength in the structure is a consequence of basic concrete variation.

A development pioneered by Dr A. M. Leshchinsky, is that of using multiple techniques of NDT concurrently. The idea is that whilst the correlation of any one such set of test results with compressive strength may be upset by some influence (e.g. ultrasonic pulse velocity is greatly affected by moisture content), it is less likely that two or more different tests will be similarly affected. Therefore the use of two or more techniques will give more certainty of a correct assessment than any number of repetitions of the same type of test. This is a further illustration of a point previously raised, that is, the relevance of a test result may be even more important than its accuracy in many circumstances.

Another point of interest is the author's experience in the 1970s of the use of two standard ultrasonic testers, the UK Pundit and the Dutch CSI. The author conceived the idea of casting pairs of test cylinders instead of the conventional threes and using an ultrasonic test on these in place of a third early age cylinder. The two instruments agreed on the ultrasonic reading, establishing that they were both accurately reading a fundamental property of the concrete, however the readings did not correlate well with compressive strength. So UPV may possibly be as relevant as compressive strength in determining the quality of concrete, but it cannot be used to establish compliance with a compressive strength specification.

11.7 Fresh concrete tests/workability

Fresh concrete can be tested for workability, air content, temperature, density, moisture content and analysed to give its composition. As in most matters connected with concrete, it is again very important to have a clear idea of exactly what it is desired to achieve before deciding which tests are worthwhile and which are not.

Workability

A large number of tests for workability have been devised. The previous edition discussed the subject in great depth, and relied heavily on a book by G. H. Tattersall (Tattersall, 1991). Tattersall, recently deceased as this is written, was a very important figure in the understanding of workability. Briefly, his major contribution was the realization that no 'single point' test could adequately quantify the workability of concrete. He established that concrete is not a 'Newtonian Fluid' in which displacement is proportional to the applied force but rather a 'Bingham Body' in which there is an initial resistance to displacement followed by displacement proportional to further applied force.

This principle is now universally accepted, the initial resistance being known as the *yield stress* and the proportionality constant for subsequent displacement being known as the *plastic viscosity*. Since some concretes may have a lower yield stress but a higher plastic viscosity than others, they will be assessed to have a different relative workability depending on the force applied during the test. This is particularly important since the slump test essentially only measures the yield stress and compaction by vibrator is mainly dependent on the plastic viscosity. Tattersall proposed that it was necessary to conduct a 'Two Point' Test at two different degrees of applied force in order to measure both the yield stress and the plastic viscosity. This concept has given birth to a number of 'rheometers' being generally devices that measure the resistance to rotating paddles, cylinders or discs in a reservoir of concrete at different speeds.

Figure 11.6 The prototype ICAR rheometer.

Since the first edition substantial work on new types of rheometer has been published. This includes the BTRheom of de Larrard (Hu *et al*., 1996, BHP96) and the rheometer of Wallevik (Wallevik and Gjorv, 1990). Unlike most other rheometers, the BTRheom is relatively portable and can be used on site. However, a new and highly portable device has been developed by ICAR at Texas University. Currently, it is only a prototype but should soon be available. Information can be sought on the University website: www.icar.utexas.edu. It remains to be seen whether this device will finally bring rheometry to the work-site or whether it will remain largely a laboratory tool and the slump test will continue its dominance on site. This question is complicated by the likely increased usage of SCC. A rheometer is certainly suitable for control of this but then again the slump flow test (described later) is a much more satisfactory test than the standard slump test. The author continues to consider that the real eventual answer to workability control must be a device fitted to every concrete delivery truck on any significant project. Such a device has been developed and was described in the previous edition. The author has observed it in technically quite satisfactory operation several years ago but it does not appear to have been a commercial success. Nevertheless the description is repeated here because this book tries to point the way to the future and the author is convinced that such a device will be the eventual answer. There is reason for hope in this since ICAR have reported a brief (one day) exercise in attempting to use a concrete truck as a rheometer. They found that it was effective in providing a yield stress but less so in providing a plastic viscosity.

The problem with the slump test is that it is a very widely and firmly established test but is a poor measure of the relative workability of *different* mixes. It survives because of its simplicity and robustness and also because it is (when properly conducted) quite a good measure of the relative consistency (i.e. wetness) of successive deliveries of the *same* mix. With today's much more accurate batching and using the author's 'MSF' (Mix Suitability Factor) we can have defined and controlled the other aspects of workability so that it may now be adequate to accept the slump test as defining *consistency* for the particular mix (especially if an 'equivalent slump', adjusted for time delay and temperature, is used). What we must *not* do is to use slump in specifications on the assumption that it defines workability on an absolute scale. It may be acceptable for special purposes to specify slump limits *in addition* to precisely specifying the type of concrete required (the author has done this for special wear resisting floors) but generally workability (slump or otherwise) is the business of the concreter, not the specifier. The concreter should be permitted to strike his own balance between the higher cost of more workable concrete and the reduced cost of placing, always providing that such aspects as shrinkage, segregation, bleeding settlement etc., are given adequate consideration.

Even the above half-hearted endorsement of the slump test does have its limits. Obviously it cannot be used for no-slump (or almost no slump) concrete. Such concrete is likely to be used only in precasting factories and in such locations a V-B consistometer (AS1012.3, 1983) (in which essentially a slump test is performed in a cylindrical container and the time taken to re-form the slump cone into the cylindrical shape under standard vibration is measured) is likely to be convenient.

At the opposite end of the scale, flowing superplasticized/self-compacting/super workable concrete is becoming more popular A flow table (DIN 1048) used to be the choice for accurate measurement of its workability. In this test it is the diameter of spread under a slight jolting motion that is measured. However, with the higher fluidity now available, a simpler variant, the Slump Flow test has taken over.

The upper limit for which the slump test can be used is very dependent on the type of concrete. Harsh, gap-graded concrete (MSF of 20 or less, see Chapter 3, Section 3.2) will fall apart on a slump test at slumps not much higher than 50 mm. On the other hand continuously graded mixes of high sand content (MSF of 27 or more) will give a measurable and reasonably repeatable slump up to 200 mm or more.

The technique of carrying out a slump test is also important in obtaining a true reading and it should be realized that the slump itself is measured in different ways in USA, UK and Australia.

What is important is not to stop using the slump test but to realize and allow for its limitations. For example a limiting slump value is often included in a job specification. With few exceptions, this is, not the best way to achieve the specifier's objective. First of all there should be an objective for the specification of anything, rather than it having been included in a previous specification and so mindlessly continued in the current document. The objectives may be to avoid high shrinkage, segregation and bleeding or to avoid an excessive w/c ratio leading to inadequate strength or durability. However, any of these faults can be encountered at almost any slump, however low, and avoided at any slump, however high. It is also easy to detect from a theoretical mix submission which mixes will be subject to one or other of these problems. The contractor should therefore be permitted to submit his mix for approval at whatever slump he chooses providing it is designed to accommodate his own slump limit without detriment. It is quite possible to produce fully flowing (250 mm slump or more) concrete having none of the potential faults noted and to produce almost all these faults in a 50 mm slump mix.

The rejection of a truckload of concrete on the basis of slump should also be approached in a reasonable manner. The slump test is both quite sensitive to small changes of water content and very easy to perform inaccurately. Certainly the truck driver should always be allowed to insist on the test being repeated. An extra 10 mm of slump probably involves about an extra 3 litres of water per cubic metre of concrete and may depress strength by about 1 MPa. The person charged with concrete acceptance should be kept continuously aware of the current strength margin of the mix in question and therefore of whether or not it is essential to reject slightly overslump concrete on strength grounds (and similarly for any shrinkage limit which may have been specified). It is more usual to find that a need to reject first arises on the grounds of wet properties or surface appearance. Slump variation will cause colour variation on a fairfaced wall and slump in excess of that designed for can involve segregation, bleeding, delayed finishing and/or floors of poor wear resistance.

While continuous perfection is impractical, a slump test will only be asymmetrical if it has been produced by an asymmetrical process. It is often possible to know where the slump operator has stood, how he has used his scoop and how he has held his rod, all by looking at the resulting slumped concrete after the test. A failure to rotate the scoop will usually cause a higher coarse aggregate content opposite the point of discharge from the scoop. This will often cause the cone to lean towards the point of discharge on stripping. It is not easy to rod the foot of the cone opposite the operator if the rod is held in a 'dagger grip'. To accomplish this the operator must project his elbow over the slump cone in order to rod parallel to the side of the cone around the entire circumference. An alternative is to use a 'rope grip', that is, to hold the rod as though pulling a rope.

The slump test is based on a standardized degree of semi-compaction, unlike compression test specimens which should be fully compacted whatever it takes. Therefore it is important that the correct number of strokes be used in the slump test whilst being only a required minimum in compacting compression specimens. It is also important that the rod have the correct end shape. A flat ended rod (e.g. a piece of reinforcing bar) pushes coarse aggregate to the bottom and tends to leave a hole rather than compact. The British rod has a hemispherical end, which is a distinct improvement over a flat end. However, the Australian and American rods, which taper to half the original diameter before having a hemispherical end give greater compaction. It should also be realized that slump measurement is different in the UK, US and Australia. In the UK, measurement is to the highest point, in the US to the point on the centreline of the original cone and in Australia to the average of the original top surface. One may have personal preferences but the important thing is to be consistent on a particular project and to be on the lookout for new operators who may have been trained by site engineers of different nationality.

A concept proposed by the author is that of an ‘equivalent slump’ (Day, 1996b). As Bryant Mather has so firmly pointed out (Mather, 1987) slump loss is proportional to temperature and leads to the (strictly incorrect but workable) view that water requirement increases with temperature. Everyone realizes that slump reduces with time. Putting the two effects together, it is clear that slump only has a real meaning if accompanied by a time and temperature reading. The author’s current proposal is to combine the time and temperature into an equivalent age according to Arrhenius (Section 11.4 on early age strength for more detail). Thus an ‘equivalent slump’ could be evaluated, being the slump which would be obtained had the concrete been kept for 30 mins at a temperature of 20°C. It can be imagined that if compression specimens were stored at anywhere from 10 to 30°C and tested at anywhere between 10 and 40 days, poor correlation would be obtained with w/c ratio. This is what we are currently doing with slump tests (i.e. ignoring time and temperature effects).

It would be quite easy to arrange for a slump value to be converted into its equivalent value as it is entered into a computer, although less easy to arrange for this to be available during a field acceptance test. What becomes quite clear when these matters are considered is the absurdity of some rejection decisions currently taken in the field. A slump of say 150 mm taken 15 mins after batching on a cold morning may indicate a lower water content, and therefore a stronger concrete, than a slump of 50 mm taken an hour after batching on a hot afternoon. Rules of thumb could be developed to allow approximately for this effect with at least more equity and realism than assuming that a slump is a slump and that’s it.

With the above points considered, adequate attention given to correct sampling and remixing of the sample; correct bedding, cleaning and moistening of a rigid metal baseplate; and use of a square mouth scoop (because a round mouth scoop leaves mortar behind in the sampling tray) the slump test can give more reliable guidance than is often the case. Nevertheless one does encounter the occasional cheeky operator who asks what you would like the slump to be before carrying out the test. Suitably instructed, such persons are at least usually competent, since they obviously know what causes incorrect results.

Assessing the workability of Self-Compacting Concrete

Several special tests have been devised to measure the workability of SCC. These include the U Box, L box, Fill Box, Orimet and J-Ring in addition to rheometers, and are adequately described on the website www.efnarc.org (EFNARC being a European federation dedicated to specialist construction chemicals and concrete systems). These are essentially laboratory tools to be used in devising SCC mixes and are too cumbersome to be likely to find site use.

The test likely to become the standard for site use (with the possible exception of the ICAR rheometer) is the Slump Flow test. This test uses the current standard slump cone but, instead of measuring the height of the cone, the diameter of spread is measured. The time for the outward flow to reach a diameter of 50 cm, known as the 'T50 time' is desirably also recorded. A further variant is to surround the slump cone by a steel ring of 300 mm diameter with evenly spaced 'feet' of vertical 100 mm steel bars known as a J-Ring. The diameter and spacing of the feet can be varied according to the congestion of the reinforcement in the section to be cast. Some J-Rings are invertible with different spacing of feet according to orientation. Apart from a visual observation of the flow through the J-Ring, the depth of concrete inside and outside the ring can be measured.

For self compaction, a flow diameter of at least 650 mm is required, with a T50 time of 2 to 7 seconds. Visual observation of the edge of the spreading concrete is important. The concrete should appear to roll out with a blunt edge and no toe of fluid paste (which would indicate bleeding) advancing in front of it. Coarse aggregate must be present right up to the edge and evenly spread over the area of concrete. There should be no concentration of coarse aggregate in the centre of the spread (which would indicate segregation).

Interestingly, the same diameter of spread is obtained whether the slump cone is used in its normal orientation or inverted. Although both alternatives currently have their advocates, it is clearly the inverted option that will survive long term for the following reasons:

1 The fluid concrete exerts a pressure on the sides of the slump cone mould. In the normal orientation this pressure has an upward component and, especially since the fluid contents leak very easily, the operator has to concentrate on maintaining foot pressure on the mould feet while filling. In contrast, in the inverted position the fluid pressure has a substantial downward component and can even be filled without being held in position (once partly filled).
2 In the inverted position the large open end is obviously easier to fill without spilling.

3 When using a J Ring, the feet of the slump cone are a problem in the normal orientation.
4 Two operators are often used in order to obtain a T50 time, but it is just possible to juggle a stopwatch when using the inverted position.
5 In the inverted position the T50 time is a little longer and so a little more tolerant of inaccuracy in timing.
6 Any tendency to segregation in the form of a concentration of stone in the centre of the spread will be exaggerated by use of the inverted position.

So in summary, the inverted position is easier to use and is a slightly more severe (and therefore better) test.

Compacting factor

The compacting factor test achieved a degree of success in the UK at replacing the slump test but is virtually unused commercially elsewhere and must now be regarded as historical. It is a device using two hoppers mounted above each other in a frame, with the lower hopper discharging into a standard cylinder mould. The concept is that the first hopper fills the second in a standard manner and the drop from the second hopper into the cylinder mould subjects the concrete to a standardized compactive effort. The result is expressed as a proportion of full compaction achieved by dividing the weight of concrete in the mould by the weight of a fully compacted cylinder.

The test is a little more accurately repeatable and is a more absolute basis of comparison between the relative workabilities of different concrete mixes than the slump test. However the test is not greatly superior to the slump test in quantifying variations in water content of successive deliveries of the same mix and, since it is less widely used, and involves more cumbersome and expensive equipment, it does not seem likely to survive. It may be reasonable to assume that if anything more elaborate than a slump test is desired, a portable rheometer is the way to go.

It is again emphasised that slump plus an MSF (Mix Suitability Factor that is relative sandiness) and adjusted for time after batching and concrete temperature, is a more meaningful measure of workability than slump alone.

Air content

Entrained air is used for two different purposes, to improve resistance to freezing and thawing and to improve workability and inhibit bleeding.

For the freeze-thaw application a higher percentage (6–8%) is required than is normally used for workability improvement and bleeding inhibition (3–5%). At the higher percentage, entrained air costs money in the form of needing a higher cement content for a given strength and workability. At the lower percentage, and at concrete strengths of 30 MPa (4,350 psi) and below, the water reduction enabled by the air entrainment may fully offset the weakening effect at a given w/c ratio.

The water reduction may be of the order of 10% and the strength loss at a given w/c ratio about 5% per 1% of air entrained. It should not be forgotten that non-air-entrained concrete is likely to contain 1–2% of voids so that the extent of the *extra* weakening may be only 5–10%.

It should not be forgotten that frost resistance depends upon bubble spacing, while strength reduction is proportional to total air volume, so that it is highly desirable that bubble size is as small as possible.

It is obviously necessary to specify the required air content where this is 5% or more, since otherwise it would be omitted on economic grounds by the concrete producer. It would also be reasonable to regularly test the air content in this case.

Where the air is not required for freeze-thaw durability, it may be unnecessary to specify it. Partly because it may be provided in any case and partly because fly-ash, with particles similar in size and shape to entrained air, has a similar effect (although a smaller water reduction). The amount of entrained air can be deduced reasonably accurately from the hardened density of the test specimens (cube or cylinder). When this density indicates that the air content may have changed, it may be desirable to immediately institute air content testing until the reason for the changed density is established.

Density

Some concrete controllers like to carry out regular fresh density testing. It is certainly true that there is often a good correlation between strength and density for a particular mix. However, as noted earlier, the density of hardened test specimens on receipt at the laboratory may be an adequate substitute for routine control purposes. Where the purpose of the density test is to settle a dispute on the yield of the mix (i.e. whether a nominal cubic metre is in fact a full cubic metre) it is certainly necessary to carry out a very formal fresh density check. In any case it is desirable to carry out such a check initially or very occasionally to verify or modify the assumption that it is adequately represented by the hardened specimen density. *In such a test it is very important not to omit the use of a glass top plate* since, however carefully it is done, striking off level is *never* accurate enough (usually the measured density is too high without a plate, but it can be too low).

When such arguments get to very fine tolerances, the question arises as to whether the concrete supplier must provide a full cubic metre of hardened concrete. Obviously the purchaser is entitled to fully compact the concrete as regards entrapped air, but is he entitled to vibrate out some of the entrained air? Also, if the concrete displays bleeding settlement, is it the volume before or after this which counts? These differences are quite small but in a situation where a great deal of concrete is placed with low labour and formwork costs (e.g. thick, unreinforced aerodrome paving) they can constitute a substantial proportion of the profit margin. There is no ‘correct’ answer to the foregoing questions, they are subject to negotiation, but it is as well to realise the situation if negotiating.

The correlation between strength and density arises because air and water are the two lightest ingredients of concrete and cement is (almost always) the heaviest ingredient. The only other factor likely to influence is the specific gravity of the coarse aggregate. In lightweight concrete the moisture content of the coarse aggregate may also be a significant factor.

Temperature

The cost of measuring the temperature of concrete at the time of casting test specimens is negligible, so it should always be done. There is often a good correlation between temperature and strength (higher temperature, lower strength) arising mainly from the increase in water requirement at higher temperatures. However it is possible that early age strength will *increase* with increasing supply temperature, the additional maturity being sufficient to more than offset the increased water requirement. This is more likely to occur with say a 3 day test than a 7 day test and in cold climate countries rather than hot ones.

Moisture content

It would seem that, with the low cost and ready availability of microwave ovens, there should be an increasing use of measuring moisture content by drying a sample of wet concrete taken back to the laboratory. The author's experience is that the largest source of error would be in a non-representative ratio of mortar to coarse aggregate in the sample. This could be counteracted by sieving the concrete through a normal garden sieve and drying only the mortar fraction.

Wet analysis

The UK RAM (Rapid Analysis Machine) is an apparatus designed by CACA(UK) to split a sample of fresh concrete into its constituent parts. It is well known but apparently little used outside the UK. The author's comments are made without the benefit of personal experience of using a RAM. Again we return to the twin questions of the accuracy of the result and a clear understanding of the purpose of the test. The principle of being able to analyse a sample of delivered fresh concrete is superficially extremely attractive. However Neville (Neville, 1981) reports an investigation by BRMCA which found that the measured cement content may be inaccurate to the extent of $\pm$ more than 40 kg/m^3 and tended to underestimate the true value by more than 20 kg/m^3 on average.'

As regards the relative proportions of the dry ingredients, the test may be more useful in some areas than others, and at some time in the past rather than today in other areas, that is, it depends whether the supplier is likely to be trying to cheat. However in projects served by a computer batching plant as described elsewhere in this section, the results would probably have more to do with mixing efficiency, sampling technique and test accuracy than with actual batch proportions. This is because hard copy computer records can be used to settle any question of deliberate deception.

As regards variability of the grading of input aggregates, a direct test of this together with a computer simulation of combined grading may be more accurate and economical.

To a considerable extent the answer to the usefulness of the test in routine control (there is no question of its usefulness for research and such uses as mixer efficiency tests) should be settled by graphing the results alongside compression tests and other data to examine the degree of correlation. The author has not had the opportunity to do this. It would seem that the best correlation would be anticipated from strength and w/c ratio. The author did have limited success 20 years ago in establishing the w/c ratio of fresh concrete by a method involving measurement, using a hydrometer, of the SG of water into which a standard volume of mortar extracted by wet sieving from a concrete had been thoroughly shaken. Such a test, and even the RAM, appear unsuitable now that concrete is likely to contain fly-ash, silica fume or other fine materials such tests may not be able to distinguish from cement.

Conclusion

It can be seen that the question of which tests are worth doing, and how frequently and thoroughly it is worth doing them, is greatly influenced by the circumstances. The circumstances include the extent of the remaining variability and its sources and also the assumptions made about the co-operativeness and trustworthiness of the concrete producer by the organization imposing the control (which may or may not be part of the producing organization).

Chapter 12

Unchanging concepts!

12.1 Cash penalty specification

It remains the opinion of the author that only a cash penalty basis can provide fully fair and effective regulation of concrete strength (and thereby, quality). However, it is fairly clear that this is unlikely ever to be accepted by either the industry or its clients. The most pressing reason why concrete might desirably incur a penalty is in fairness to other suppliers who allowed in their quotation to supply the specified strength in full and thereby failed to obtain the contract to supply. If well-intentioned suppliers do not see this as an advantage, then so be it. However, the section remains in the book to satisfy the author's conscience that he has done everything reasonably possible to bring about this desirable but rejected reform.

This section was first published by the author (Day, 1982b) as an article in *Concrete International: Design and Construction*, September 1982 under the title: Cash penalty specifications can be fair and effective. Permission granted by the American Concrete Institute to reproduce it here is gratefully acknowledged.

A cash penalty of twice the cost of the extra cement which would have been required to avoid defectiveness is proposed. It is shown in detail that if this is based on the statistical analysis of any 30 consecutive 28-day test results, very little inequity would result to either party (in contrast to the substantial risk of inequity under current specifications based on inaccurate, small sample criteria). The aspect of legal enforceability is considered and examples are provided of a suitable cash penalty provision used in a major Australian structure, and of several situations where cash penalty provisions would have been desirable.

A good specification system accomplishes the following (Day, 1961):

1 Ensures the detection and penalization of unsatisfactory concrete.
2 Avoids the penalization of good concrete.
3 Encourages good quality control.
4 Avoids any doubt of fairness and eliminates disputes.
5 Is based on sound theoretical principles.

Typical concrete specifications around the world continue to levy one penalty of rejection and continue to base judgement on criteria which are known to be inefficient at distinguishing the actual quality of the concrete assessed (Chung, 1978). The result of this ostrich-like attitude is to leave supervising engineers in untenable positions, to subject concrete suppliers to gross unfairness on occasions, frequently to allow unsatisfactory concrete to be supplied with impunity, and worst of all, to fail to encourage responsible producers of low-variability concrete.

The proposed system

The quality of concrete is assumed to be represented by the mean and standard deviation of strength. Quality should be specified by the requirement:

$$\bar{x} \geqslant f_c' + k\sigma$$

where

$\bar{x}$ = mean concrete strength
f_c' = specified strength
σ = standard deviation of strength
k = constant.

Any deficiency in strength can be readily assessed in terms of inadequate mean strength. The cost of remedying that deficiency can be readily assessed in terms of cement content.

For a limited extent of deficiency, a penalty of twice the cost of remedying the deficiency could be imposed. This penalty is negligible for small deficiencies, but if the criterion is sufficiently accurate, the penalty will be sufficient to ensure that no concrete supplier can make additional profit by supplying understrength concrete. This penalty system benefits producers of low-variability concrete and encourages improved quality control.

The key to this system is the determination of the values of mean strength and standard deviation with sufficient accuracy, and the selection of a suitable value for k. It is immaterial whether the cement-content change required to provide a given strength change is truly a constant for all concrete, providing the change is never more than twice the assumed value.

Accuracy of assessment

The gross inaccuracy of assessment encountered under most specifications arises from an inadequate number of test results (Chung, 1978), and from attempting to assess the quality of an amount of concrete sufficiently small to accept or reject as a whole. There is no such requirement in a cash-penalty specification.

A secondary reason for basing a criterion on a small number of results is to enable a judgement to be made quickly, thus limiting the amount of defective concrete supplied before a halt is called. This pious intention becomes a joke when the results are obtained at 28 days.

The solution to this dilemma is to separate the functions of (1) acceptance/ penalization and (2) detection and arrest of adverse quality.

An interesting and valuable result of operating under a cash-penalty scheme is that the interests of the supervisor and the concrete supplier coincide in their joint desire to detect and eliminate adverse trends at the earliest possible moment. This cooperative type of relationship is in contrast to the traditional requirement to establish with legal precision that concrete strength is inadequate and then require the unwilling supplier to rectify the matter.

The suppliers generally recognize that rapid reaction to warnings of low strength from the quality control engineer can save the supplier money. A graphing system can provide such information based on a few early age test results and will enable the supplier not only to avoid extensive periods of low strength but also to reduce the overall variability (a double saving in potential penalties) (Day, 1981).

The standard error of assessment of the mean strength of a group of n test results is $\sigma/\sqrt{n}$, while that of its standard deviation is $\sigma/\sqrt{(2n)}$.

The standard error of assessment of the criterion $\bar{x} - k\sigma$ is therefore:

$$\sqrt{\left[\frac{\sigma^2}{n} + \frac{(k\sigma)^2}{2n}\right]}$$

where

k = 1.28 (a 10% defectives criterion)
σ = 3 Mpa (435 psi)
n = 30 results

The expression gives a standard error of approximately 0.74 MPa (107 psi). This means that 90% of assessments will be within $\pm$ 1.65 $\times$ 0.74 = 1.22 MPa (177 psi) of the correct value.

If it is further assumed that a 1 MPa (145 psi) strength change requires 7–8 kg/m^3 (12–14 lb/yd^3) of cement change (the actual value could range from 5 to 10 kg/m^3 (8–17 lb/yd^3) for different concretes), then the inaccuracy amounts to a maximum of $\pm$10 kg/m^3 ($\pm$17 lb/yd^3) in cement content, or a cost of around \$0.70 (Australian)/m^3 (approximately \$0.56 (US)/yd^3).

Operation of the system

The specification might then read as follows.

'The specified strength of the concrete shall be X MPa and for every 1 MPa (145 psi) that the mean strength of any 30 consecutive samples minus 1.28 times the standard deviation of strength of those samples falls below X MPa, the contractor shall pay a penalty of \$1 (Australian)/m^3 (\$0.80 (US)/yd^3) of the whole of the concrete represented by the 30 results in question.'

(\$1 equals twice the cost of the 7.5 kg (16.5 lb) of cement assumed to be required to increase the concrete strength by 1 MPa (145 psi).)

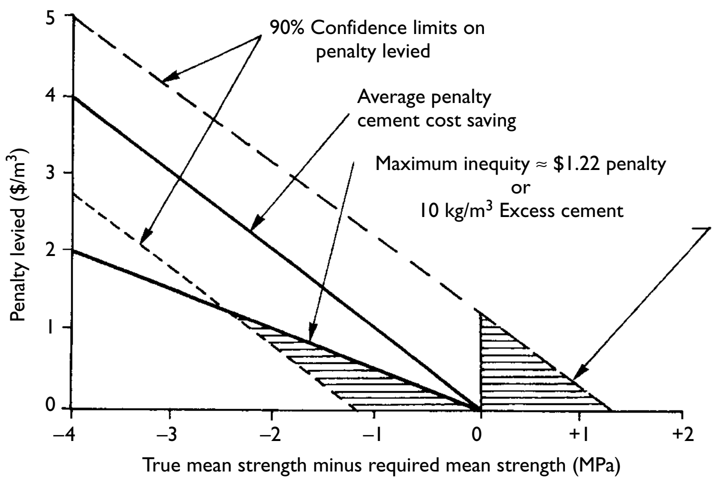


Figure 12.1 Graph of average penalty applied.

To avoid occasional unmerited penalties under such a specification, the concrete supplier would have to work to 10 kg/m^3 (17 lb/yd^3) excess cement content, increasing the cost of concrete by $0.70 (Australian)/m^3 ($0.56 (US)/yd^3) above the cost strictly required, with the idea that this increase in cost is justified by the quality control benefits of the entire system.

On the other hand, a concrete supplier would occasionally escape penalization when actually supplying concrete as much as 1.22 MPa (177 psi) under strength. On average, though, the supplier would be paying a penalty of $1.22 (Australian)/m^3 ($0.98 (US)/yd^3) to set against the cement cost saving of around $0.70 (Australian)/m^3 ($0.56 (US)/yd^3).

Fig. 12.1 shows the average penalty which would be applied and the 90% confidence limits on that penalty for strength shortfalls up to 4 MPa (580 psi). The graph shows there is very little risk of any significant unmerited penalty and even less chance of the cement saving outweighing the penalty.

Effect of k value changes

The effect of an increasing *k* value would be to increase the required mean strength. This could be offset by a reduction in the specified strength below that used in the structural design. The effect of such a compensated increase in *k* value would be to provide a greater incentive to attain a low variability in the concrete strength by imposing a larger safety margin on suppliers of higher variability concrete. The actual minimum strength (say, the three standard deviation limit below which only one in a thousand results would fall) would be raised by such a specification.

In the author's view, an increased incentive to reduce variability and increase security against the occurrence of very low strengths would be highly desirable. It is suggested to use a k value of 3 and to reduce the specified strength by 5 MPa (725 psi) in compensation.

For a k value of 1.28 (existing US practice) and a specified strength of 30 MPa (4,348 psi), the effect of this would be:

1 $\sigma = 2.5$ MPa (362 psi) (good control):

(a) required mean strength $= 30 + (1.28 \times 2.5) = 33.2$ MPa (4,812 psi)
(b) effective minimum strength $= 33.2 - (3 \times 2.5) = 25.7$ MPa (3,725 psi).

2 $\sigma = 5$ MPa (725 psi) (poor control):

(a) required mean strength $= 30 + (1.28 \times 5.0) = 36.4$MPa (5,275 psi)
(b) effective minimum strength $= 36.4 - (3 \times 5.0) = 21.4$ MPa (3,101 psi).

For a k value of 3.0 (preferred), and a specified strength of 25 MPa (3,623 psi), the effect would be:

1 $\sigma = 2.5$ MPa (362 psi):

(a) required mean strength $= 25 + (3 \times 2.5) = 32.5$ MPa (4,710 psi)
(b) effective minimum strength $= 32.5 - (3 \times 2.5) = 25$ MPa (3,623 psi).

2 $\sigma = 5$ MPa (725 psi):

(a) required mean strength $= 25 + (3 \times 5.0) = 40$ MPa (5,797 psi)
(b) effective minimum strength $= 40 - (3 \times 5.0) = 25$ MPa (3,623 psi).

The effect of the change would be to worsen the competitive position of the high-variability supplier and limit the occurrence of occasional low strengths in the concrete supplied. The low-variability supplier would be virtually unaffected, except for the supplier's improved competitive position.

Fig. 12.2 shows the relative situation under exact compliance with a 10% defective criterion for both high and low-variability suppliers. The upper graph shows that under the present (US) 10% defective basis, the low-variability supplier has a reduced incentive and the high variability concrete includes some deliveries of very low strength. The lower graph shows an enhanced competitive position for the low-variability supplier under the proposed 0.1% defective basis. Both suppliers in this case provide effectively the same minimum strength.

The benefits of low-variability concrete are substantial:

1 Helpful to the concrete placing crew.
2 More uniform compaction.
3 More uniform appearance.
4 More accurately assessed on a given number of test results (possibly less frequent testing required).

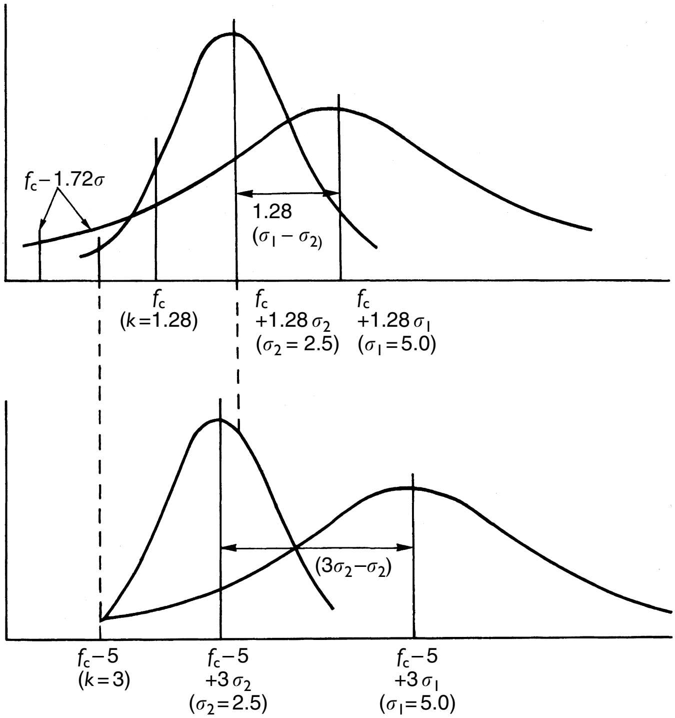


Figure 12.2 Effect of compensated increase in k is to improve competitive position of low-variability supplier and rule out low results from high-variability supplier.

The influence of change points

The proposed technique assumes that there will be a gradual drift of either mean strength or variability and that it will be legitimate to select 30 results incorporating the worst period. Analysis has shown, however, that changes are usually 'step' changes rather than gradual drifts. Thus, a specific number of results constitute the low period and all of them (and no more) should be analysed to represent the low period rather than taking an arbitrary 30 results. This is too complicated and indefinite for use in a specification but could be applied with mutual agreement in practice. The effect of analysing 30 results overlapping a change point is to give an artificially inflated standard deviation which is only slightly compensated for by the increased mean strength obtained from the inclusion of a few higher results and, therefore, causes a higher penalty. An alternative, slightly lower penalty based on the actual defective period can be offered, but the specification can be strictly enforced without substantial unfairness.

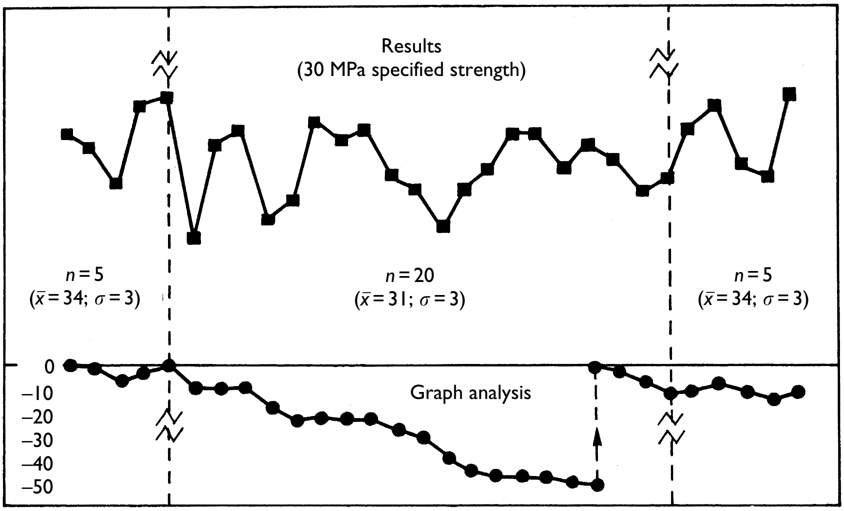


Figure 12.3 Graphical analysis of run of understrength results which merits a penalty.

Fig. 12.3 shows a run of understrength results which merits a penalty. Under the proposed specification, the lowest 30-result section (representing 600 m^3 (785 yd^3) of concrete) must form the basis. A penalty of \$2.28 m^3 would be applied, totalling \$1,368.

Close analysis, however, reveals that the low strength concrete is confined to a 20–result section (representing 400 m^3 (523 yd^3) of concrete). The penalty/m^3 based on the 20 results would be greater but the overall penalty would be less at \$1,136. The latter penalty is the more equitable and is the one which should actually be imposed. However, the difference is only \$232 and the 30-result basis is reasonably satisfactory and much simpler to incorporate into a specification.

The assumption is that the concrete supplier would have had to spend approximately \$1.50 m^3 in extra cement on the 400 m^3 (523 yd^3) of concrete to avoid penalization (total saving: approximately \$600 in cement cost), so the net cost to the supplier is approximately \$600. Obviously, the supplier would prefer to pay this penalty rather than delay the work and pay the costs of coring and investigating 400 m^3 (523 yd^3) of concrete, with the risk that some or all of it might be rejected.

Importance of quality of testing

It is of obvious importance that the test results forming the basis for a cash penalty should provide an accurate assessment of the quality of concrete as supplied by the producer. This is by no means something which is easy or can be taken for granted.

A minimum requirement is that samples should be taken, cured and tested by a competent, accredited and preferably independent organization.

The best criterion of testing accuracy is the average difference of pairs of test results from the same sample of concrete. This average difference should not exceed 1 MPa (145 psi) for normal concrete (specified strength less than 50 MPa (7,246 psi) and possibly excluding very low slump mixes). It is suggested that the highest of a pair of specimens is likely to be a better estimate of the true concrete strength than the mean of the pair.

The person responsible for result analysis should be alert for clearly established cases of incomplete compaction and improper curing and testing, and should be prepared to exclude such results from a penalty assessment. The previously recommended graphical analysis system, including analysis of related variables such as slump, strength and testing, has been found valuable in distinguishing causes of variability and early detection of problems.

Parallel tests by two laboratories on the same truck of concrete reveal useful information and should be arranged from time to time.

The whole question of the reliability of concrete testing results is a matter which has received far too little attention. However, it is not a valid reason for failing to institute the type of cash penalty specification advocated here, as it causes even more trouble under existing types of specification.

No one can afford cheap testing. The best prospect of reducing testing costs is to reduce the frequency of testing, made possible by better testing, better specifications, better analysis of results, and a reduction in the variability of concrete.

Legal enforceability

Extremely crude forms of penalty are sometimes encountered, particularly on government work. Such penalties are enforced on the basis that future contracts will be withheld if they are disputed.

In British and Australian law, the key to legal acceptability is to relate the penalty to the harm suffered. It is assumed that a building owner would prefer to pay for the grade of concrete specified rather than accept a lower grade of concrete at lower cost. If the owner is supplied a lower strength concrete than specified, then he must have suffered harm in excess of the cost difference (in terms of margin of safety, durability, etc.) between the two strength levels.

Actually, the penalties considered here are too small to be worth a contractor's expense to legally challenge. However, the penalties are sufficient to ensure his co-operation in avoiding them.

What the law does object to are penalties specified to scare the contractor into compliance.

Experience in Australia

Although this proposal is now 20 years old, it has been applied to only one major contract to the author's knowledge. This was the Victorian Arts Centre (the Melbourne equivalent of the Sydney Opera House). On only one occasion did the results actually merit a cash penalty, which was paid.

However, thousands of cubic metres of concrete have been supplied to dozens of structures using the previously discussed control system, but without the cash penalty provision. On no occasion has it proved necessary to actually remove concrete from any of these structures.

Generally, concrete suppliers have been responsive to requests to adjust cement contents based on early age analysis. However, there have been frequent occasions when the strength provided, assessed as above, has fallen below that strictly required, for extended periods, by 1 MPa (145 psi) or less.

Such minor deficiencies have no structural significance but do waste time in repeated requests and reports and arguments with concrete suppliers (who are ever optimistic that the 7- to 28-day strength gain will improve on current production). Suppliers complain that precise enforcement is unrealistic, yet without strict controls, deficiencies would no doubt tend to gradually increase. A cash penalty as proposed would avoid all need for such argument. The deficiencies would be acceptable with the penalty paid, but it is suspected that deficiencies would rapidly disappear in such circumstances.

There have been suggestions that, in fairness, penalty clauses should be balanced by bonus clauses. This is not recommended because excess strength beyond that specified is of little benefit to the owner. The type of cash penalty clause advocated here is a real benefit to the good concrete supplier. He can aim at the mean strength truly needed without restriction. If he slightly miscalculates, the penalty is very moderate and involves no cost of delays or further investigation. He is defended from unfair competition by less competent or less scrupulous competitors. Finally, he can include his own bonus in his pricing if he wishes.

Conclusions

It is concluded that a cash penalty of twice the cost of the cement deficiency can be accurately established by the analysis of a group of 30 consecutive test results. Such a penalty would be capable of regulating concrete strength with fairness. The system would result not only in an improved degree of contractual compliance but also in a cooperative attitude in day-to-day control between the contractor and the supervising engineer. It would provide an effective incentive to improve control which would, over a period, produce significant improvements in concrete production techniques.

12.2 What is economical concrete?

This section appeared in *Concrete International* (Day, 1982a). It is quoted verbatim as the author's views have not changed. Permission granted by the American Concrete Institute to reproduce it here is gratefully acknowledged.

The question 'What is economical concrete?' may seem a ridiculous question, but consider the example of the Rialto project in Melbourne. This project is very unusual in that the concrete supplier, the builder and the eventual owner were one and the same. It involved (amongst nearly 1,00, 000 m^3 of total concrete) 6,000 m^3 of a 60 MPa (8,700 psi) grade, which was the highest grade of concrete yet specified for such a project in Australia. Accordingly construction started with a very conservative mix which actually provided a mean strength of over 80 MPa (11,600 psi) and a characteristic strength of approximately 75 MPa (10,875 psi). Considerable cement content reductions (say, 100 kg/m^3 (170 lb/yd^3)) were clearly possible but no reduction was in fact made on the following grounds:

1 The possible saving of say $60,000 was trivial compared to the total project cost of several hundred million dollars.
2 The huge strength margin virtually ensured that there would be no delays due to strength problems.
3 The very high early age strength permitted early stripping, etc. with no concern for damage, weather conditions, need for intensive in situ or early age testing, etc.
4 The additional safety margin against any unexpected factors was also of some value.

As another example, Australia's billion dollar Parliament House is a major concrete structure, containing about one quarter million cubic metres of concrete. At around 25 million dollars, the cost of the concrete supply represents about 2.5% of the total cost. It really would not matter very much if this cost increased 5% to 2.63% of total cost.

Of course, the extra cost in the case of the Rialto would be a little less trivial if the same argument were applied to the whole of the concrete in the project but the real point is that this attitude could never be taken by an independent concrete supplier because the cost would probably exceed the entire profit margin. The strength margin (but more likely 5 MPa (700 psi) than 15 MPa (2,000 psi)) could therefore only come about by either the owner specifying a higher grade or the builder ordering a higher grade than specified. Either party might take this action on the basis of expediting construction, or at least of avoiding any risk of delay. In fact the best way of organizing this is for the owner to specify a higher strength but to impose a cash penalty rather than rejection or further investigation for strength shortfalls of up to 5 MPa (700 psi) (or whatever margin has been allowed). The same effect could be obtained by offering a bonus for excess strength (of course within a strict limit) and *not* raising the specified strength.

The benefits accruing from the proposed technique (of specifying a higher strength than strictly necessary and providing a cash penalty for strength deficiencies within the margin) would be:

1 A relaxed attitude to minor strength deficiencies by the owner.
2 A keener attitude to minor strength deficiencies by the concrete supplier.

3 A smoother running project.
4 The provision of better concrete, probably at only a very marginal overall cost increase.

There is yet one remaining possible turn of the screw of increased strength margin. This is to obtain the extra margin not by specifying a higher strength but by specifying a lower percentage defective at the original strength. This would have the effect of putting a higher premium on low variability and could be a substantial factor in discriminating in favour of better producers and so providing a beneficial pressure towards improved performance by the industry. If a strength increase of the order of 5 MPa (700 psi) is desired, it would amount to around 1.5 times the standard deviation. In most of the world, a 5% defective level is used, so that a mean strength of specified strength plus 1.645 times standard deviation is required. Raising the margin to 3 times standard deviation would go close to the 1 in 1,000 defective level (mean $-$ 3.09 $\times$ standard deviation) and would mean, for a typical 3 MPa (435 psi) standard deviation, providing a margin of 9 MPa (1,300 psi) between mean and specified strengths. The margin would vary between 6 MPa (870 psi) and 12 MPa (1,740 psi) from the best concrete producers (SD of 2 MPa (290 psi)) to the worst we should tolerate (SD of 4 MPa (580 psi)). With such a pressure to improve, it is likely that in 5 or 10 years time, we would find the good operators down to below 1.5 MPa (220 psi) SD (margin of around 4.5 MPa (650 psi) as currently typical) and the rough operators out of business.

Perhaps an intermediate solution would suffice. A margin of much less than 5 MPa (say, 2 MPa (300 psi)) is probably quite adequate for the operation of a cash penalty system and this would be provided with an SD multiplier of 2 (giving around the 2.5% defective level of 1.96 $\times$ SD). Incidentally it is time we stopped thinking of SD multipliers primarily in terms of permissible percentage defective. The real grounds on which they should be selected is the relative value we place on mean strength and standard deviation in assessing concrete quality (on this ground, a multiplier of 3 is highly desirable). The relationship between the desirable mean strength (or the 10%, 5% or 0.1% defective level) and the strength used in structural design calculations should be a subsequent rather than an initial decision, but is clearly an *independent* decision.

Interestingly, the cost of the additional strength margin now being proposed (or more) has often been incurred in the past by the specification of 20 MPa (3,000 psi) characteristic strength together with a minimum cement content requirement of the order of 300 kg/m^3 (500 lb/yd^3). There is however a very substantial difference in the results of the two specification bases. Whilst the former offers distinctly better concrete, a smoother running project (due to the cash penalty basis) and a pressure towards a better performing concrete industry, the latter offers scope for cheating on cement content, for the use of sub-standard aggregates and oversanded, high shrinkage mixes and, most important of all, a removal of any incentive for the technical competence of producers.

There are two important provisos which should be made in advocating cash penalties and greater emphasis on standard deviation:

1 The standard deviation (and the mean strength – but that is much easier) must be accurately determined.
2 The cash penalties (which may be described as ‘liquidated damages’ or ‘provision for reduced durability’ or formatted as a bonus clause rather than a penalty) should be very moderate, only about twice the cost of the additional cement which would have avoided any penalty (i.e. about 10 kg/m^3 per MPa (12 lb/yd^3 per 100 psi) of deficiency so, in Australia, about a \$2 penalty per MPa of deficiency).

The requirement for an accurate SD is easily satisfied under a cash penalty system because it is not necessary to identify *which* concrete is slightly understrength – only how much and how defective. Therefore the penalty can be levied on the concrete represented by 30 consecutive results with great accuracy (Section 12.2).

Does anyone have a convincing counter argument? If not, how long do you think it would take to implement this proposal? 5? 10? 20 years? It may be of interest that the outline of this argument was advanced in papers published by the author in 1959 and 1961 (Day, 1959, 1961).

12.3 How soon is soon enough?

The first edition contained a 21 page account of an investigation using a massive computer analysis of synthetically generated data to clearly establish the superiority of cusum analysis over any other system known to the author for the early detection. That section of the first edition is not repeated here but is available on the website.

The two most significant points arising were:

- No computer analysis is as efficient and reliable as the eye examination of a cusum graph in detecting a small change in mean strength.
- The mathematical significance of a downturn in a single variable (i.e. strength) is in any case immaterial when the significance of the downturn is confirmed by simultaneous changes in other variables such as slump, temperature and density.

The economic value of a more efficient analysis system is briefly compared to that of other factors affecting the attainment of the desired concrete quality, such as better equipment, more skilful personnel and higher testing frequency. It is pointed out that a more efficient detection system is equivalent to a higher testing frequency in achieving early detection. It is shown that the average number of results required to achieve detection of a change is directly proportional to the standard deviation of those results. Since early detection in turn enables a reduction in variability, a self-intensifying cycle of variability reduction is commenced.

The questions of early age and/or accelerated testing, of monitoring batching performance, of analysing related variables such as slump, density, temperature etc. have been addressed elsewhere in this volume. For the purpose of this investigation it was assumed that a continuous string of test results is being received and converted into predicted 28 day results. The relative efficiency of the different techniques in detecting a downturn in such a string of results was examined.

The assumption made by the author, after 40 years of plotting quality control charts for concrete, is that the downturn is usually a sudden event or 'step change' rather than a gradually worsening trend.

A Lotus spreadsheet computer program was set up to automatically produce a string of 100 random, but normally distributed, results of any selected mean and standard deviation. It then appends a further 30 results of the same standard deviation but a lower mean. This enabled examination of the performance of a control system in respect of whether it raised false alarms during the initial stable period of 100 results and how long it took to detect the imposed change point at the 100 result mark. The results were automatically analysed by up to six different detection systems at a time and the results reported as:

1 The number of results prior to a false alarm in the first 100 results, if the number is 100, there were no false alarms.
2 The number prior to the first detection of change after the imposed change point, if the number is 30, there was no detection.

The best detection system is not necessarily the one that shows the lowest average number of results to give a detection. Any type of system can be made more sensitive by narrowing its limits, at the cost of experiencing more false alarms. It was not considered sufficient to find that one system was extremely good at detecting changes but gave many false alarms, while another gave few false alarms but was a poor detector. It is certainly of interest to compare the relative severity of different national codes but the author's primary interest is in finding the most efficient way of detecting a change. The exercise was therefore repeated after adjusting the nominal specified strength so that each system gave similar false detection frequencies when assessing the same concrete.

It was found to be important whether the adjustment was in the form of a constant or that of a multiplier of the SD. The various national systems often incorporate a fixed adjustment, for example, ACI 214 requires not more than 1 in 100 results to be more than 500 psi (3.45 MPa) below the specified strength and BS 5328 requires the running mean of four results to exceed the specified strength by at least 3 N/mm^2 (3 MPa). This investigation has shown that unless such adjustments are expressed in terms of a multiple of the standard deviation, the systems would give a substantially different relative performance according to whether the production was at high or low variability. Another aspect of system efficiency is the use of multiple criteria. A system can be made to give a better ratio of correct detections to false alarms by composing it of several sub-systems running in parallel. In this case the better performance is obtained at the cost of a more complicated criterion, a larger program and slower operation. With computer assessment, these costs would be negligible compared to increasing physical testing frequency *and it should be realized that a more efficient analysis system has as much value as additional testing*. For example, it would be possible to analyse results using a combination of all the systems and to accept that a downturn had occurred when one was detected by any two, or any three, of the nine systems shown. This would no doubt give both a better detection rate and less frequent false alarms. However, the improvement would probably be relatively small since false alarms are frequently due to aberrations in the results, affecting several systems, rather than to aberrations in one of the detection systems. (In this respect it would be of value to persons involved in concrete QC to examine a selection of the data generated for this investigation in order for them to realize the extent to which apparently convincing downturns in a set of results occur as a result of normal statistical variation.)

The real reason militating against the multiple criteria approach is that they must still be suitable for the average user. Complication must be avoided as far as possible, both to ensure comprehension by all concerned in their enforcement, and to avoid the much greater effort of examining compliance by manual calculation by persons not having computer knowledge or facilities.

Relative performance of the systems

All the systems, except ACI 214, are nominally directed towards assuring a characteristic strength which 95% of results will exceed. Therefore that characteristic strength is given by Mean minus 1.645 times Standard Deviation, that is, for this exercise, 35–1.645 SD.

In the case of ACI 214 the requirement is for only 90% of results to exceed the specified strength. Therefore that strength in this exercise becomes 35–1.28 SD. However in the adjusted limit section, the ACI system is still comparable as what is reported is the amount of adjustment required.

It can be seen that both the ACI and the UK systems, in their original forms, give rapid detection of a downturn but also give a high rate of false alarms. The Australian system on the other hand appears unduly lenient. The numerical cusum was adjusted to comply with the 70/80 false alarm frequency during the process of selecting the deduction margin and detection limit. This was done as a separate exercise using the techniques of this investigation in which a large range of margins and limits were tried (in sets of six) to find the most efficient combination.

The basic techniques embodied in the national codes (individual result limit and limits for running means of 3, 4, 5 and 30) were also separately examined. This was necessary because some of the combinations were optional and also to avoid concluding that the code incorporating the largest number of individual criteria (ACI 214) was necessarily the best.

Visual Cusum

In the (very lengthy) initial stages of the investigation, hundreds of graphs of the run of 130 results were examined. It was noted that the basic cusum graph almost invariably showed a quite distinct downturn at the exact point of the artificial downturn, even when the 'drop ratio' was so small that the numerical system detection efficiency was poor.

It should be noted however that this is far from the same thing as concluding that the detection efficiency of the basic cusum is almost perfect. The technique looks better in retrospect than it does in genuine use. Examination of the overall 130 result trial tells nothing of how many of the small false downturns in the cusum graph might have been mistaken for the real downturn, or for how long an operator might have regarded the real downturn as such a false one. So, while the keen and experienced operator using cusum graphing will already have acted before the detection system provides a signal, the less experienced operator will be glad of the confirmation provided by the system and the less keen operator will be prodded into action.

What is clear is that, on looking back after concluding that a downturn has occurred, the basic cusum graph will show exactly when that downturn occurred. This is very valuable information because the same logic applies to any other variable for which a cusum graph is drawn, and therefore it is usually easy to match up cause and effect.

Numerical Cusum

The previous mean value is subtracted from each result and if the difference numerically exceeds a selected margin, the difference (less the margin) is accumulated in a register. If the accumulated total exceeds a selected limit, a detection has occurred. In practice positive and negative registers are maintained (because detection of an upturn means that cement can be saved, which is a further reason to prefer numerical cusum) but for the current exercise, only a negative register was maintained.

For any selected margin, a limit can be chosen to give whatever frequency of false alarms is considered acceptable. It is conventional to choose a margin of about half the minimum change it is desired to detect. If this is considered to be $0.5 \times$ SD, then SD/4 might be the chosen margin. The investigation reported started with a margin of SD/3 and a limit of $4 \times$ SD but after comparative trials, the best results were obtained with a margin of SD/6 and a limit of $5.5 \times$ SD.

The use of a numerical cusum in this way is exactly equivalent to using a graphical V-mask technique (Devore) as is used in the UK.

Assessment of alternatives

Table 12.1 show that, on average, and after adjustment to a comparable false alarm frequency, the ACI running mean of five gives the quickest detection.

Table 12.1 National criteria as in national codes

	ACI 214	*AS 3600*	*BS 5328*	*N Cusum*
False alarm frequency	52.36	93.6	46.81	70.74
Average detection delay	1.75	12.90	2.64	4.11
Maximum average detection delay	8.06	20.15	7.26	10.54
Adjusted (by constant margin) to comparable false alarm frequency:				
Adjustment in char strength (MPa)	1.75	−0.60	6.50	NA
False alarm frequency	63.8	64.5	77.8	71.2
Average detection delay	6.4	10.4	12.0	6.3
Maximum average detection delay	17.6	20.3	22.5	16.0

However the numerical cusum follows close behind and is better at detecting very small drops. Numerical cusum is also more directly aimed at detecting change from a previous situation rather than infringement of a specified limit. Since a producer would be ill-advised to work right down to the limit, the latter is likely to be the more useful feature. Numerical cusum is also equally at home in detecting upturns and this is important to the producer. Of course a running mean of five can be adapted to all these purposes but this is not often done.

The national systems are not strictly comparable as they have different intended methods of application. The American ACI 214 publication sets out a range of possibilities together with several pages of excellent advice and information with the objective of allowing specifiers to make their own informed decisions. It also includes a recommendation to maintain control charts and detailed advice on how to do so.

The British BS 5328 condenses its unequivocal requirements into a small table and four carefully chosen sentences. To be fully comparable with the ACI system it would also be necessary to make reference to the requirements of the British ‘Quality Scheme for Ready Mixed Concrete’ which is an industry based self-regulatory body and recommends cusum control charts or an alternative ‘counting rule’ system involving not more than eight consecutive results below the previous mean.

The Australian system provides a rule by which concrete producers are required (regardless of individual project specifications) to regulate the whole of their production. It then also provides a rule by which individual projects can check the quality of concrete received by that project.

In comparing the requirements it should be remembered that the British code is anticipating approximately double (4–6 MPa) the standard deviation normal in Australian capital cities (2–3 MPa), with USA covering a larger and intermediate range. It could also be said that the Australian code is designed to avoid unfair condemnation of the producer and allow full benefit for the attainment of low variability, while the British code is aimed solely at providing near certainty that the supply of sub-standard concrete will be eliminated in all circumstances. It appears that the carrot may be currently showing greater benefit than the stick.

The use of a minimum required strength for any individual specimen has good and bad points. It is reasonable to put a limit to the downward spread of results which could be obtained with very high variability concrete whilst still providing a mathematically acceptable mean. However test results are subject to error and an individual specimen criterion can require action on the basis of a badly made test if not intelligently administered, and the author's experience is that such matters are often not intelligently administered.

The use of a fixed lower limit for individuals may have its merits but the use of a fixed numerical limit for the running mean of a set of four, as in the UK Code BS 5328, has the unfortunate effect of severely limiting the financial benefit obtainable from good control. As previously noted, any kind of requirement involving a constant produces distortions in performance over a range of SD values.

One final answer to the 'how soon?' question must be 'before anyone risks their neck'. It is quite possible to assess concrete quality within 24 hours and it is probably legally, and certainly morally, indefensible not to do so prior to prestressing, early demoulding, jump form movement etc.

Other significant considerations

Where cost competition is negligible, it is easy to provide a large safety margin totally avoiding failures. In these circumstances a highly-tuned control system may not be essential but is obviously affordable.

Where cost competition is severe, a control system which can detect a shift in mean strength of as little as 1 MPa (150 psi) within 2 or 3 days of its occurrence may be an excellent investment. Where operating conditions and materials are very stable, the additional cost of early age testing may not be justified. Seven day testing has the advantage that, on detection of a suspected downturn, a reservoir of test specimens from 1–6 days age is available and can be immediately brought forward for test to confirm or negate the change. This is providing one is sufficiently knowledgeable (and has done the necessary prior investigations) to correctly interpret results at a range of early ages.

The control process should be considered as a whole, ensuring value for money in several different types of expenditures for example:

1 Batching equipment
2 Quality of testing
3 Frequency of testing
4 Computer equipment
5 Computer software.

The ability to work to a 1 MPa (150 psi) lower mean strength for a given specified strength is worth about 5 kg of cement per cubic metre (8.4 lb/cu yd). This is a sufficient saving (on high volume production) to pay for a very elaborate control system. The ability to detect a downturn in strength a day earlier may avoid a major penalty. It may also justify a lower safety margin.

It should be noted that all criteria relate to the standard deviation of results. Lower variability concrete is easier to control more precisely. As already noted, this is not tautology but a recognition of a multiplier effect of control improvement. A reduction of 1 MPa in standard deviation makes a direct difference of 1.28 or 1.65 MPa to the required target strength (depending on whether the specification is based on 90% or 95% above). It will make at least a further 1 MPa reduction in the strength margin required for the detection of a change. Improved quality control may also be a major sales point. The standard deviation of the concrete strength is obviously affected by the quality and effectiveness of both the batching system and the testing process, as well as by the variability of input materials.

The frequency of testing is an important cost factor to be weighed against the quality of testing, the securing of additional data such as slump, concrete temperature and density, and the cost of result analysis. The cost of elaborate analysis is rapidly reducing compared to that of physical testing *and an increase in one can justify a reduction in the other*.

The ability of a control system to combine results from many different grades of concrete into a single analysis can be equivalent to a several fold increase in testing frequency.

The time between a downturn and its detection and rectification is also affected by the age at test. The days in which mix revisions were based on 28 day test results are hopefully gone, but the choice of test age in the interval of 1 to 7 days is open to consideration. In temperature stable tropical conditions, 3 days is a good choice. Depending on the protection provided to the specimens, and on the time of collection, a three day strength may be too variable in other climates. Further options are to use accelerated specimens or to measure thermal maturity in order to obtain a result at 1–2 days.

A consideration of the above factors makes it clear that:

1. Except in very low volume situations, there is ample saving in cement cost to offset a high standard of control.
2. The cost of computer analysis with a good class of computer and software is modest compared to other factors in achieving timely control of concrete quality.

Chapter 13

Troubleshooting

There are several aspects to troubleshooting in concrete technology. One of them, separation of its costing from that of QC, was raised in the first edition and is repeated here.

Another is the examination of existing structures with a view to repair. This is a field in which the author has considerable experience but has for several years done his best to avoid. Some reasons for this attitude are:

- The field is a very extensive and rapidly developing one and, to provide good professional service, requires that the practitioner keep fully up to date with a myriad of constantly changing techniques and proprietary materials. The author is unwilling to divert enough time and effort to this aspect of concrete technology from his chosen fields of mix design and QC to satisfy his conscience in being such a practitioner.
- Repairs to concrete structures are very often temporary (unintentionally that is) and may provide only a short term cosmetic effect at considerable expense. The author does not wish to be involved in such situations.
- Clients are often unwilling to face up to the very expensive solutions that may be necessary to achieve a degree of permanence.
- Even the experts have difficulty in establishing which of several competing repair proposals represents best value for money (or whether any proposal offers good value).

However, it should be pointed out to younger readers that this field is likely to absorb something like half the total expenditure on concrete structures in the next few decades. It is also likely to generate distinctly more than half the profits to be made out of concrete technology in this period. This is because typical clients are far more willing to pay for cure than for prevention (even if not enough for reasonably permanent cure). Therefore the author does not encourage others to adopt his own attitude.

The author is from time to time paid significant amounts of money to sort out problems with concrete still in the production stage. Advice on the procedure to follow seems desirable since the kind of action necessary in many (but not all) such situations is reasonably easy to learn (compared to repair), and since even relatively amateur attempts to follow the advice given are likely to be beneficial, even if not necessarily optimum.

The first action must be to establish exactly what the problem is. Some possible problems are:

- Inadequate strength
- Lack of pumpability
- Inability to compact
- Unsatisfactory appearance
- Excessive segregation or bleeding
- Inadequate retention of workability
- Failure to set or stiffen sufficiently rapidly
- Presetting cracks or later age cracks
- Excessive cost of imported materials
- Excessive variability.

Possible problem sources are:

- Unsatisfactory aggregates
- Unsuitable mix design
- Poor testing (including sampling, casting and curing of specimens)
- Cement or pozzolan quality
- Unsuitable admixtures or admixture usage.

Data to request (having relevant past data available on arrival can often shorten the investigation by a day or more):

- Mix details
- Aggregate gradings
- Concrete test records (including times, temperatures and specimen collection details)
- Cement test certificates if available
- Cores and failed test specimens to inspect.

Of course it is desirable that records go back to a period before occurrence of the problem if possible. Where aggregate testing records seem inadequate, a rapid visit to the stockpiles is desirable before (further) change occurs. Segregation of coarse aggregates, silt content of the sand and contamination with subgrade material by front-end loader are items to look for.

13.1 Strength, pumpability, appearance

Inadequate strength

The typical steps taken by the author when called in to investigate problems may be of interest.

The steps are:

1 Restore strength to a safe level so work can continue while investigating. Cement content adjustments should always overshoot when increasing and undershoot when reducing. Use 8–10 kg per MPa to adjust upwards, 4 kg per MPa to adjust downwards. If adjustment gives cement content over 500 kg use 500 kg plus 2 kg of fly-ash for each 1 kg of cement not added, or 0.5 kg silica fume ditto, or 100 ml superplasticiser ditto.
2 Start casting at least 4, perhaps 6, test specimens per sample. Test at 2, 3, 7, 28 and perhaps 56 days. Assume gain in MPa will remain the same with the revised mix. In default of prior data, conservatively assume that strength will increase 33% from 2 to 3 days, another 33% from 3 to 7 days and 10 MPa from 7 to 28 days. Substitute actual figures as soon as available.
3 Draw cusum graphs of strength (at all available ages), density, concrete temperature, slump, 7 to 28 day gain (for example). If data is available, cusum graphs of sand silt content and/or specific surface should also be drawn on the same presentation. A cusum of average pair difference between pairs of specimens from the same sample will show whether there has been a deterioration in quality of testing (an average pair difference in excess of 1.5 MPa is an indication of poor testing quality). Such graphs will usually show when the problem started and what caused it.
4 Examine batching records (assuming a computer operated plant which records actual batch quantities) before and after the downturn for signs of cement shortfall or aggregate, especially sand, over-batching.
5 Calculate MSF (Mix Suitability Factor) using formulas in Chapter 3. MSF is a measure of the sandiness of the mix taking into account sand grading, sand per cent, cementitious material content and entrained air. Calculate water content using formula in Chapter 3. Is actual water content really known? An MSF in excess of 30 represents oversanding and high water requirement unless for flowing, superplasticized concrete.
6 Calculate strength according to one or more of formulas in Chapter 3. If this agrees with strength obtained/being investigated, then high water content is the explanation and the reason and cure are obvious (may be any combination of high MSF, silt in sand, concrete temperature, high slump).
7 If calculated water or strength does not agree with actual, re-check sand silt per cent and grading. Check concrete density as this will confirm water and/or air content and/or compaction of test specimens. *The water content is the major separating factor between alternative directions of investigation*. If water is the end cause, then the basic cause is likely to be in the area of dirty or finer sand, high sand content, high slump or high concrete temperature. If water is not the cause, then the basic cause is likely to be in the area of poor testing (including sampling, compaction, curing, capping {if cylinders}, defective or badly cleaned/assembled moulds {if cubes}, centering, load rate etc.), or of cement quality or quantity.

Poor workability/pumpability

Generally the causes are an excess or deficiency of fine material, a gap in the grading, or an excess or deficiency of fluidity.

1 Does the concrete bleed? If so there is either a gap in the grading, a deficiency of fine material, or excessive fluidity. If the concrete pumps reasonably at the start, but will not re-start after a delay, this is often due to bleeding.
2 Using the author's MSF, the value of this must be at least 24 to 25 for pumping to be possible. The higher the desired fluidity, the higher the MSF value will have to be, however values in excess of 32 will exhibit excessive friction unless superplasticized to high slump.
3 Draw a graph or produce a table of individual percentage retained on each standard sieve. Ideally all sieves below the largest will have similar percentages of around 7 to 10%. One size missing may not be fatal if those either side are normal. Any two consecutive sieves with a combined total retained of less than 7% would be a potential problem. More than 20% on a single sieve finer than 4.76 mm might also create a problem in pumping.
4 Is there at least 300 kg/m^3 of material passing the 0.15 mm sieve (including cement)? If not additional fines may be needed as either fine sand, crusher fines, fly-ash or cement.
5 If the (single) sand is so coarse that more than 55% (perhaps 50%) of it is necessary to provide an MSF of 25 there is likely to be a problem with bleeding, segregation and pumpability. Additional fines as in (4) above are necessary.
6 Air-entrainment, fly-ash and silica fume (in increasing order of effectiveness) are effective suppressors of bleeding and so assist pumpability. The author has witnessed a huge foundation 4.5 metres deep filled with concrete of more than 200 mm slump and containing 40 kg/m^3 of silica fume, which exhibited no bleeding whatever.
7 Although nothing to do with mix design, it should be borne in mind that it is pressure that causes a problem in pumping and faster pumping requires higher pressure. Also a delay caused by a gap in deliveries is an aggravating factor. Therefore, if pumping problems are being experienced, pumping more slowly and ensuring that one truck is not emptied before a replacement arrives may assist.

Unsatisfactory appearance

This may be due to inept placing, poor formwork or many other things which are beyond the scope of this book. However it is also often due to bleeding, the remedies for which have been covered above. If bleeding happens at all, it often results in a flow of water up the face of formwork, leaving clearly visible signs. A slight formwork leak (just of water) can cause an internal surface flow of water over an area of more than a square metre and result in a large black stain, known as a hydration stain.

Presetting cracks – There are two kinds of presetting cracks with diametrically opposite causes: settlement cracks and evaporation cracks.

Settlement cracks – These result from settlement of the concrete due to loss of bleedwater. In settling, the concrete 'breaks its back' over anything resisting settlement in one location and not another, for example, reinforcing bars, cast-in plumbing, sharp changes in depth of section. Measures to avoid bleeding have been dealt with above.

Evaporation cracks – These result from evaporation of water from the surface layer of concrete. If a concrete has very low bleeding, for example, silica fume concrete, it is susceptible to such cracks and measures must be taken to avoid evaporation, for example use of an aliphatic alcohol evaporation retardant such as 'Confilm', a sheet material such as polythene, or a mist spray of water drifting across the surface.

Thermal stresses – Another frequent cause of early age cracking is thermal stress. This can be reduced by substituting pozzolanic material for cement in the mix design. However, action other than mix change may be needed, such as avoiding restraint to thermal shortening (in the case of long slabs); maintaining more uniform temperatures by insulating the exterior surfaces of large masses of concrete.

Adiabatic shrinkage should not be forgotten as a cause of early cracking in cement-rich mixes. This removes free water from the concrete by chemical combination. It can produce similar results to drying shrinkage but much more rapidly, and in spite of any measures taken to reduce or prevent evaporation.

Excessive variability

The first thing is to establish whether the variability is in the concrete or in the testing. Two places to look are the average pair difference in the 28 day results and the range of densities of test specimens from the same sample of concrete. The average pair difference should desirably be below 1.0 MPa and densities should not have an average range exceeding 50 kg/m^3. However calculated densities may vary through inaccurate measurement of specimens rather than variable compaction or segregation and this would have no effect on strength variability.

A second place to look is at multivariable cusum graphs of strength and other variables. If slope change points in strength correlate with those of other variables, the cause will be clear. Direct plots of multiple variables will show whether individual high or low results have an explanation. If there is no explanation, and especially if 7 and 28 day results do not correlate, testing would be suspect.

Having established that the variability is actually in the concrete and not just the testing, batch quantity records should be available if batching is by computer-operated plant. It should not be overlooked that the correct quantities may be weighed out but may be insufficiently mixed to give uniformity. There have also been examples of short central mixing times (prior to further mixing by agitator trucks) which have not permitted time for all the metered admixture to enter the mixer. Similarly part of a particularly critical ingredient such as silica fume may ‘hang up’ in the batching skip from time to time and finish up in the next load.

13.2 Causes of cracking in concrete slabs

The causes of cracking in concrete are sufficiently well known to permit their automatic diagnosis in most cases. The author has in fact written an expert computer system for this purpose, which unfortunately used a now superceded shell and is therefore not currently operative. An expert system is a computer program that asks questions of a user in order to be able to diagnose the cause of the user’s problem; the better ones are also able to explain why the particular question is being asked, on request by the user.

The first question to be asked is the age of the concrete at cracking. If the age was less than 10 hours, the crack would be classified as a pre-setting crack caused by either excessive evaporation from the surface or by restrained or differential bleeding settlement. If the age was more than 10 hours but less than 48 hours (and especially if the crack occurred in the early morning following pouring) the crack would probably be a thermal contraction crack. If the age exceeded two days (and was after termination of moist curing if any) it may be due to drying shrinkage.

To determine whether pre-setting cracks are caused by evaporation or settlement, questions are asked about the shape, size and location of the crack and about whether the concrete bled substantially or was subjected to drying winds and low humidity. Evaporation cracks may be quite wide on occasions but they are usually short and randomly orientated. However, they can sometimes be concentrated in an area of the slab which is more exposed to wind and can form parallel lines. In the latter case they may be more difficult to distinguish from settlement cracks occurring over a steel mesh, except that it would not be likely that evaporation cracks would be parallel to the direction of the mesh, or at the same spacing. As already noted the settlement cracks can occur over reinforcing bars, installed plumbing or the like. They can also occur at lines where the section deepens, such as dropped capitals for columns, haunched beams or the edge of thickened areas of a slab.

A classic situation for thermal cracking exists when a thin concrete wall is poured between restraints. The restraints may be a heavy foundation beam with starter bars or substantial columns with projecting reinforcement. When a wall in such a situation is poured on a warm afternoon using a mix rich in a high heat generating cement (e.g. a white cement) the width of the crack to be anticipated on stripping next morning can be calculated if a maximum reading thermometer is inserted. Such cracks are often widest at the base, next to the restraining foundation beam, and taper away to nothing 2 or 3 metres up the wall.

A commonly encountered situation is where a crack runs parallel to, and often close to, a sawn control joint. It is easy to see that either the joint was not deep enough to be effective or, more likely, it was actually cut after the slab had already cracked, although perhaps before it had opened sufficiently to be noticeable.

Another useful distinguishing test is to place a straightedge at right angles across a crack. If the straightedge will rock, this indicates that the slab has deflected and therefore that the crack was probably caused by subgrade or formwork movement, or structural inadequacy in the case of suspended slabs.

Where cracks are three pointed, they are usually caused by a swelling or settlement resisting rock immediately below the junction of the cracks, for example, a 'floater' in a soft subgrade subject to moisture movement.

In the case of suspected thermal cracks, it is useful also to check whether the concrete had a high cement content, making it likely to generate more heat, whether it was poured on a hot afternoon followed by a cold morning, and whether there was a delay in pouring, which could have allowed the concrete to heat up whilst kept waiting in the truck.

Surface crazing occurs when the surface layer shrinks relative to the body of concrete below it. This can be caused by allowing the surface to dry or cool quickly and is more likely when a high shrinkage surface layer, rich in cement paste and fine sand and of high w/c ratio, is present.

There is an almost universal tendency to use quality control personnel for troubleshooting of the above nature. This may be a reasonable use of any spare time, but it is important to ensure firstly that it does not disrupt the QC routine and secondly that such work is separately costed from QC. This is because the economic justification of QC should be clearly established as it otherwise tends to be regarded as a luxury item, first in line for cutting in hard times. Troubleshooting in general is not QC, indeed it may be the result of inadequate QC, and it is rarely cost saving or revenue generating. Many QC departments (not only in the concrete industry) have been axed or decimated through a failure to recognize this.

Summary and conclusion

The book has attempted to increase the understanding of all readers from research scientists to undergraduates and interested but unqualified workers in the concrete industry. Non-technical executives in the concrete industry may also be enlightened, and avoid future regrets, by a skim through the book. So far as possible, jargon and mathematical formulas have been avoided, to keep the book readable and widen the audience, but an attempt has been made to explain in fine detail what actually happens, and why, in the production and control of real concrete, and to look as far as possible into the future.

I have often said that it is not possible for one person to know more than 10 per cent of what there is to know about concrete. I now add to this it is not possible to put anywhere near 10 per cent of the knowledge and experience gained over 50 years into a single book. I can only hope that readers will have found that good choices were made in what to present in the tip of the iceberg this book represents.

It is hoped that those who specify concrete will forgive the diatribe against the profession. If you have been sufficiently interested to read this book, you are unlikely to be one of the guilty ones.

Appendix

Advances in inorganic polymer concrete technology

A.1 Introduction (KWD)

With the exception of this introduction, Dr Grant Lukey and the team at the University of Melbourne, including Prof Priyan Mendis, Prof Jannie van Deventer and post graduate student Massoud Sofi, substantially wrote this chapter. As will be apparent, Grant (former General Manager of Siloxo Pty Ltd (Australia), a company established to exploit IPC) is a leading authority on the subject.

In the (book) author's opinion, IPC (more commonly but less correctly known as GPC, i.e. inorganic polymer concrete) will become an important material in the near future and he is more than pleased to be able to incorporate this chapter. It provides a brief insight into various aspects of inorganic polymer concretes (IPCs), including their basic chemistry, synthesis, properties and application. The main differences in chemistry of ordinary Portland cement (OPC) based concrete and IPCs are discussed, with particular attention on the advantages and shortcomings of IPCs compared to ordinary concrete. The current technical, environmental and commercial drivers for uptake of the technology are also discussed, as well as the challenges and obstacles faced during the successful commercialization of this promising technology. The chapter concludes with some of the typical and most recent applications of IPC materials. It is anticipated that this chapter will give the reader a general understanding of the current research and development work on IPCs and provide an introduction to a new and potentially very robust and versatile material in the field of civil engineering.

Background

IPCs are aluminosilicate polymers synthesized by reacting aluminium and silicon containing materials with an alkaline silicate solution. The resultant mixture is then cured at ambient temperature and pressure (Lee and van Deventer, 2001) to form a hardened 'concrete-like' material in appearance. Inorganic polymers are a relatively new set of materials that were reportedly discovered in the 1970s by a French scientist, Joseph Davidovits. A series of catastrophic fires in 1970 and 1973 had led Davidovits to carry out extensive research to find a new heat-resistant material in the form of non-flammable, non-combustible 'plastic materials' (Davidovits and Davidovics, 1988).

In 1978, the term 'Inorganic polymer' was created by Davidovits in analogy to organic polymers undergoing polycondensation and forming rapidly in the space of a few minutes at low temperatures. The term expresses the idea of a material that is inorganic, non-flammable, hard and reportedly stable at temperatures up to 1,250°C (Davidovits and Morris, 1988). It has been reported in the scientific literature that inorganic polymers can have outstanding physical and chemical properties including durability, high compressive strength, high acid resistance and high fire resistance (Davidovits and Davidovics, 1988). Provided that these reported material properties can be confirmed, then inorganic polymer technology provides that basis for the manufacture of potentially desirable and extremely valuable construction materials (Smith and Comrie, 1988; van Deventer, 2002). Although the potential usage of IPCs as building and construction materials is not new, it has not been well defined until more recently. It is interesting to note that the cement formulations used by the Romans (BC 200–100 AD), the Egyptians (BC 2500) or in Tell-Ramad (BC 7000) involved inorganic polymer setting, yielding a zeolitic material of the phillipsite and analcime type (Davidovits, 1987).

Davidovits has proposed that the pyramids and temples of the Old Kingdom of Egypt were not constructed from massive blocks of limestone, but rather by mixing a muddy limestone [including the fossil-shells] with lime and zeolite-forming materials, such as kaolin clay, silt and the Egyptian salt natron [sodium carbonate] (Davidovits and Morris, 1988). According to the same reference, no stone cutting or heavy hauling or hoisting was ever required for pyramid construction but rather that wooden moulds were used and the concrete [an IPC mix] poured just as is done presently. Although the majority of Egyptologists and geologists still strongly disagree with this theory, Davidovits published it successfully in a book in 1989, entitled: *The Pyramids: An Enigma Solved*. In this book, Davidovits further asserts that stone vessels found in the Step Pyramid of Zoser were made of very hard stone materials, basalt metamorphic schist and diorite. Looking at the structural features of those vessels, it was proposed further that inorganic polymer (or so called 'inorganic polymer') technology is the only viable explanation of these features. Ceramic type materials, or low temperature Inorganic Polymer (IP) setting of ceramics, continue to exist today. While expressing his appreciation of the new material (Grimal, 1999) has proposed that IPs are the perfect materials for use in decorative indoor applications. At the present time, however, there are numerous other applications of IPC materials, which will be elaborated further in the following sections.

More recently, interest in IPC gained a new momentum due to the materials inherent fire resistant and potentially blast-resistant properties. Responding to the threat of terrorist attacks around the world, engineers are seeking new methods to prevent damage to high-risk facilities. In Australia and other countries, after the September 11 event in 2001, a great deal of concern has been raised by building owners and tenants on the vulnerability of structures under extreme conditions such as intensive hydrocarbon fire. The threats to structures may come from accidental sources such as gas explosions, chemical fires or terrorist types of loading, such as the blast of a car bomb or the impact of a missile or an aircraft. For these extreme loadings, the source can originate either external or internal to the structure. Impact and blast loadings are usually accompanied by fire. In such instances, inorganic polymer materials could be potentially used as fire protective coatings on the inner steel structural framework, or indeed, the construction material itself.

The ultimate goal of fire protection is to minimize injuries and loss of life and facilitate the evacuation and rescue of survivors. The main lesson learnt from the collapse of the World Trade Centre in the United States, due to the subsequent fire more than the initial impact, was that special attention must be given to the behaviour of the structural elements to improve their fire resistance. The inherent fire-resistance of inorganic polymeric materials may mean that structural concretes formed using this technology would be an improved construction material for such applications. This may have considerable significance for the rapid adoption of the material. Environmental drivers and real potential for cost-competitiveness and chemical resistance is one thing, but terrorist resistance could prove irresistible for further uptake of the technology.

In summary, inorganic polymers (referred to as 'inorganic polymers') have emerged as novel engineering materials with the potential to form the basis of a new and environmentally sustainable construction materials and building products industry. These materials are formed by the alkali activation of industrial by-products such as coal ash and blast furnace slag. These materials can exhibit superior chemical and mechanical properties to ordinary Portland cement (OPC) counterparts. In addition to the formation of conventional pre-cast or architectural products, these high performance mineral binders are well suited for the encapsulation of mine tailings, the immobilization of heavy metals, and the paste back-filling of mines. A key attribute to the technology is the robustness and versatility of the manufacturing process; it enables products to be tailor-made from a range of coal ash sources and other alumino-silicate raw materials so that they have specific properties for a given application at a competitive cost (e.g. high strength concrete; fire and acid resistant coatings etc.). Despite these attributes, the commercial uptake of the technology by the cement and concrete industry is still relatively low. This may be due to the technology being considered as disruptive to the industry, not cost-effective, or perhaps as high-risk given that quantitative data relating to the long-term durability of IPCs is not currently available in the public domain. However, the recent increased pressure on industry by governments world-wide to identify new sustainable technologies that offer reductions in CO_2 emissions, energy consumption, as well as being able to add value to existing industrial by-products, may facilitate the wider uptake of the technology in the near future.

A.2 IPC reaction and chemistry

The term 'inorganic polymer' describes a family of mineral binders that are reported to have a polymeric silicon–oxygen–aluminum framework structure similar to that found in zeolites. These materials may be synthesized at ambient temperature or higher by alkaline activation of aluminosilicates obtained from industrial by-products such as coal ash and blast furnace slag (Krivenko, 1994; Cheng and Chiu, 2003), as well as calcined clays (Rahier *et al.*, 1996; Hos *et al.*, 2002), melt-quenched aluminosilicates (Xu and van Deventer, 2003), natural minerals (Xu and van Deventer, 2002), or mixtures of two or more of these categories. Similar to OPC binders, filler materials or aggregates may also be used to optimize desired properties including strength and density.

It is important to note that the reaction chemistry between inorganic polymers and Portland cement are entirely different in many respects. While pozzolanic cements generally depend on the presence of calcium, inorganic polymers do not strictly utilize the formation of calcium–silica–hydrates (CSH) for matrix formation and strength. Instead, IPCs utilize the polycondensation of silica and alumina precursors at a high alkali concentration to attain structural strength. These structural differences give IPCs certain advantages compared with conventional cement-like binders.

In recent years, there has been academic debate on the exact reaction mechanism for inorganic polymerisation, as well as the final molecular structure of the material. The traditional structural model proposed by Davidovits describes an inorganic polymer binder as a fully amorphous three-dimensional polymeric chain and ring structure consisting of Si–O–Al–O bonds (Daviovits, 1987) (Fig. A.1) whereby

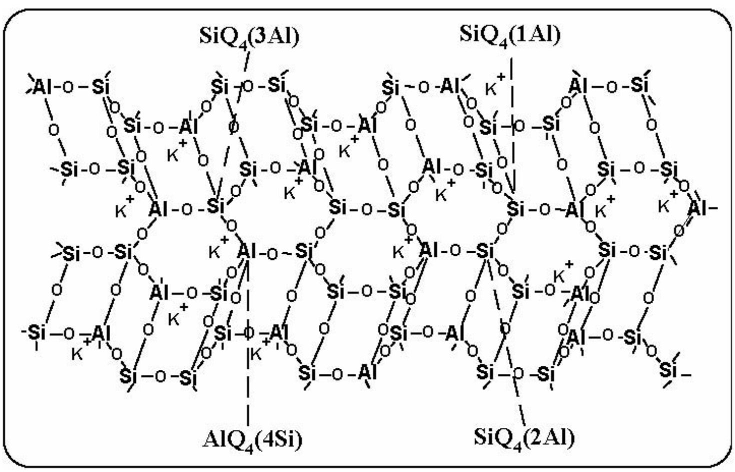


Figure A.1 Proposed three-dimensional structure of a inorganic polymer.

Source: Davidovits, 1987.

aluminium and silicon are both in a four-fold co-ordination state (Davidovits, 1988). Other studies have confirmed Davidovits' statement by showing that the structure of inorganic polymer binders was indeed amorphous to semi-amorphous (van Jaarsveld and van Deventer, 1999).

For chemical designation of these binders based on silico-aluminates, the term poly-sialate was suggested (Davidovits, 1999a). Sialate is an abbreviation for silicon-oxo-aluminate. Chemical reaction between various aluminosilicate oxides with silicate under highly alkaline conditions yields a sialate network consisting of SiO_4 and AlO_4 tetrahedra linked alternately by sharing all the oxygen. In order to balance the negative charge of Al^{3+} in IV-fold coordination, positive ions (Na^+, K^+) must be present, resulting in Si–O–Al–O polymeric bonds (van Deventer, 2002). The chemical reactions, depicted by Equations (1) to (3), best describe the polysialation process:

$$(Si_2O_5, Al_2O_2)_n + 3nH_2O \xrightarrow{NaOH} n(OH)_3 - Si\text{–}O\text{–}Al^{-}\text{–}(OH)_3 \qquad (1)$$

(Orthosialate)

$$n(OH)_3 - Si\text{–}O\text{–}Al^{-}\text{–}(OH)_3 \xrightarrow{NaOH} (Na, K)\text{–}\left(\text{–}\begin{array}{c} | \\ Si \\ | \\ O \\ | \end{array}\text{–}O\text{–}\begin{array}{c} | \\ Al^{-} \\ | \\ O \\ | \end{array}\text{–}O\text{–}\right)_n + 3nH_2O \qquad (2)$$

(Na, K)–poly(sialate)

$$(Si_2O_5, Al_2O_2)_n + n2SiO_2 + 4nH_2O \xrightarrow{NaOH} n(OH)_3\text{–}Si\text{–}O\text{–}Al^{-}\text{–}O\text{–}\begin{array}{c} Si \\ | \\ (OH)_2 \end{array}\text{–}(OH)_3 \qquad (3)$$

(Ortho(sialate-siloxo))

To assist in the understanding of the basic building-blocks of IP materials, (Davidovits, 1991) introduced a new scientific notation and terminology (Table A.1).

This conceptual basis for the formation of the molecular structure of an inorganic polymer binder has served as a basis for the majority of research work relating to the technology until 2004. However, it is important to note that no study has confirmed the existence or otherwise of the simplified monomeric building blocks as depicted in Table A.1. Furthermore, the polysialate nomenclature inherently fails to satisfactorily describe the full connectivity of each silicon and aluminium centre or how they relate to one another in a continuous network. For example, this model does not account for non-integer values of the Si/Al ratio (Provis *et al.*, 2004), or the possible formation of Al–O–Al linkages in the structure

Table A.1 Classification of inorganic polymer binders

Category	*Structure*	*Alkali cation*	*Utilization*
Polysialate (PS)	$M_n-(\text{Si–O–Al–O})_n$ O O	K–PS Na–PS	Thermal insulation Fire-resistant
Polysialate-siloxo (PSS)	$M_n-(\text{Si–O–Al–O–Si})_n$ O O O	K–PSS Na–PSS K, Ca–PSS	Refractory Fire-resistant Performance cement Toxic waste
Polysialate-disiloxo (PSDS)	$M_n-(\text{Si–O–Al–O–Si–O–Si–})_n$ O O O O	K–PSDS Na–PSDS K, Na–PSDS	Tooling composites Refractory Fire-resistant

Source: Davidovits, 1991.

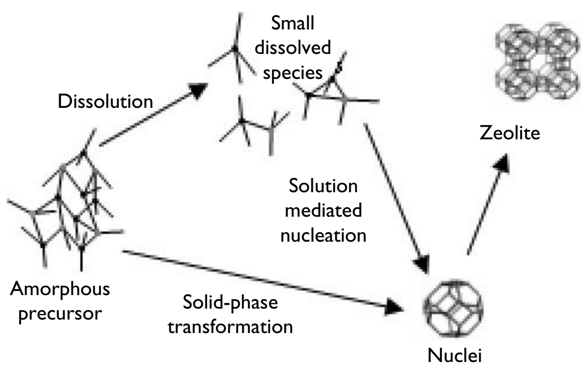


Figure A.2 Proposed formation of nanocrystallites, resembling zeolites in inorganic polymers.

(Duxson, 2004). This structural model also describes an inorganic polymer as being an amorphous aluminosilicate gel (Lee, 2002); however, recent work has observed the formation of semicrystalline or polycrystalline phases, particularly in binders products synthesized at a higher temperature (van Jaarsveld, 2000).

A recently modified structural perspective of an inorganic polymer describes the system as potentially an agglomeration of nanocrystallites, which resemble zeolites, surrounded by an amorphous aluminosilicate gel (Provis *et al.*, 2005). Given correct synthesis conditions, then such phase separation is predicted from both zeolite and glass chemistry. Although zeolite nuclei in inorganic polymers have not been observed by XRD previously, it is thought that due to their size (<10 nm) and quantity (<5%) then they would remain undetected by XRD

(Provis *et al.*, 2005); although crystalline regions have been observed using HREM on well prepared specimens (van Jaarsvel, 2000). Recent work directly observed and theoretically predicted the presence of Al–O–Al bonds in inorganic polymers at specific synthesis conditions (Duxson, 2004).

The potential of zeolite formation and the presence of Al–O–Al bonds in a inorganic polymer is a significant breakthrough in the fundamental understanding of the technology. This means that by a variation of synthesis conditions and changes in reactivity of raw materials, then potentially a inorganic polymer having a different microstructure (and therefore properties) would form. Combined with an understanding of reaction kinetics, this makes possible the tailor-design of inorganic polymer products to possess specific properties resulting from changes in the inherent fundamental structure. It is an understanding of this chemistry, and methods to manipulate and control it, that enable the successful IPC product development as will be described later in this chapter.

Recent studies have investigated the role of calcium addition in the formation of inorganic polymer gels. Such work is important because blast furnace slags and coal ashes (the main feedstocks used to form IPCs) can contain significant quantities of calcium. In particular it has been shown that the inorganic polymeric gel and calcium silicate hydrate gel (CSH) could form simultaneously within a single inorganic polymeric system, depending on the level of alkalinity (Fig. A.3). CSH gel formed in such a system has a significantly lower Ca/Si ratio than the CSH commonly formed from the hydration of OPC. Also, there is some calcium precipitate along the interface between the CSH and inorganic polymeric gels. It

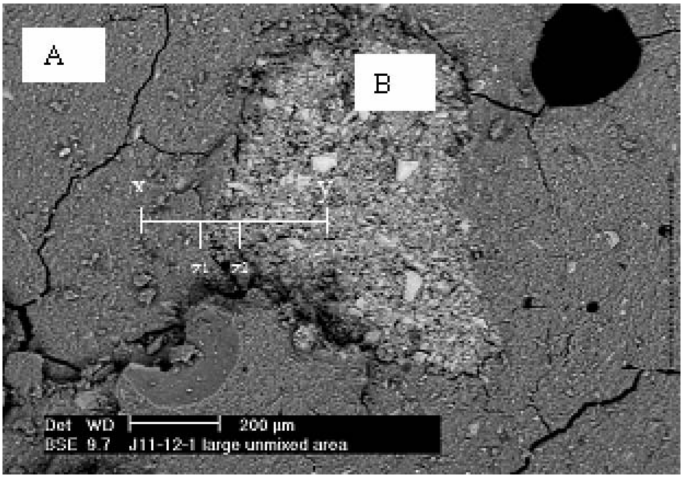


Figure A.3 SEM micrograph of a inorganic polymeric matrix containing slag and metakaolinite at low alkalinity. (A) Inorganic polymeric binder with low calcium and (B) CSH with a small proportion of aluminium.

is suggested that the properties (e.g. size, elemental composition) of the inorganic polymeric and CSH gels forming simultaneously, and the reactivity of the calcium precipitates along the interfacial region, will hold the key in reformulating a new generation of concrete that matches the durability of ancient concrete (Yip, 2004). The relationship between IPCs, Portland cement, zeolites and alkali-activated cements will be discussed further in the following section of the chapter.

These studies (Granizo *et al.*, 2002; Yip *et al.*, 2003) provide further evidence that the microstructure (and therefore resulting properties) of an inorganic polymer material changes significantly depending upon raw material composition and synthesis conditions. Therefore, an understanding of how to control the formation and presence of different phases in the final material will enable a inorganic polymer to be tailor-designed to a particular application.

A.3 Fundamental differences between the chemistry of Portland-concrete and IPC

There are some obvious differences between conventional Portland cement chemistry and inorganic polymerisation. While it is known that the binding property of cement is due to the presence of calcium through formation of calcium silicate hydrate in a semi-crystalline phase (Gani, 1997), inorganic polymer polycondensation reaction (i.e. the formation of inorganic polymer gel or binder phase) can effectively take place without it. There are a number of advantages associated with the non-requirement of calcium in the IP condensation reaction that are worthwhile mentioning here. The absence of calcium and the superior microstructure of IPCs provides good resistance to acidic environments. Laboratory testing indicates the system is more resistant to aggressive microbial induced corrosion environments than calcium aluminate with calcium aluminate clinker aggregates (Silverstrim, Martin *et al.*, 1999). The reason behind this lies in the chemistry and micro-structural differences of two materials.

The hydration reactions of Portland cement have been established by a number of investigations that have focused on the hydration of pure cement compounds (Taylor and Broms, 1964). The principal hydration product formed is calcium silicate hydrate which is formed by the reactions of dicalcium or tricalcium silicate with water (Mindess and Young, 1981):

$$2C_2S + 4H \rightarrow C_3S_2H_3 \text{ (calcium silicate hydrate)} + CH \quad (4)$$

$$2C_2S + 6H \rightarrow C_3S_2H_3 \text{ (calcium silicate hydrate)} + 3CH \quad (5)$$

Hardened cement paste consists of poorly crystallized hydrates, notably calcium silicate hydrate (CSH) and calcium hydroxide (CH), minor components, unreacted cement particles as well as the residues of water-filled spaces (capillary pores) (Navi and Pignat, 1996). The nanostructure of CSH is poorly defined with only an approximate stoichiometric formula ($C_3S_2H_3$), and is highly disordered and irregular (Young, 1981). Major variables in calcium silicate hydrate are the calcium oxide to silicate ratio ($CaO:SiO_2$) which ranges from as low as 0.6 up to 2.4, and the water content, which are controlled by the age of mix, the water cement ratio and the temperature of the hydration.

In short, all of the main hydrates in cement contain calcium. The concentration of calcium in the interstitial solution participates in all chemical reactions and is therefore of particular interest. It is impossible to determine the profile for the concentration of Ca^{2+} in situ when the paste is in contact with an aqueous solution. However, the nature of this profile can be deduced from the solid Ca/Si profile in the degraded zone of a model paste containing only hydrated tricalcic silicate ($3CaO$, $SiO_2 = C_3S$). At equilibrium and at a given pressure and temperature, the $CaO–SiO_2–H_2O$ system is univariant (Gibbs law) and completely determined by one parameter. By measuring the solid Ca/Si of a C_3S paste totally hydrated, the Ca^{2+} concentration in the interstitial solution can thus be deduced (Faucon, Adenot *et al.*, 1998).

It has been mentioned that inorganic polymer reactions result in Si–O–Al–O polymeric bonds yielding a sialate network consisting of SiO_4 and AlO_4 tetrahedra linked alternately by sharing all the oxygen. These reactions take place in a highly alkaline ($pH > 14$) environment and do not require the presence of calcium in the system. Investigation of the properties of IPC type materials has led researchers to refer to IPCs as low-temperature aluminosilicate glasses (Rahier, Mele *et al.*, 1996a,b). Others have determined a relationship between the micro-structures of natural zeolites and IPCs (Yip and van Deventer, 2001). On the other hand, the structure of different types of C–S–H present in Portland cement paste has been presented as a network of layers (Faucon, Adenot *et al.*, 1998). This is clearly shown in Fig. A.4.

As discussed in the preceding section, it has been reported that an increasing concentration of Ca/Si in the system will effectively turn an IPC mix into a

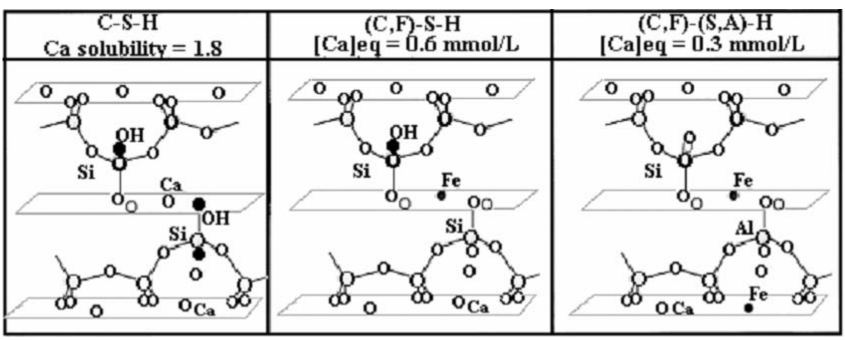


Figure A.4 Structure of different types of C–S–H present in the superficial layer of cement paste (results of ^{57}Fe Mossbauer spectroscopy and ^{29}Si and ^{27}Al solid NMR).

Source: Faucon, Adenot *et al.*, 1998.

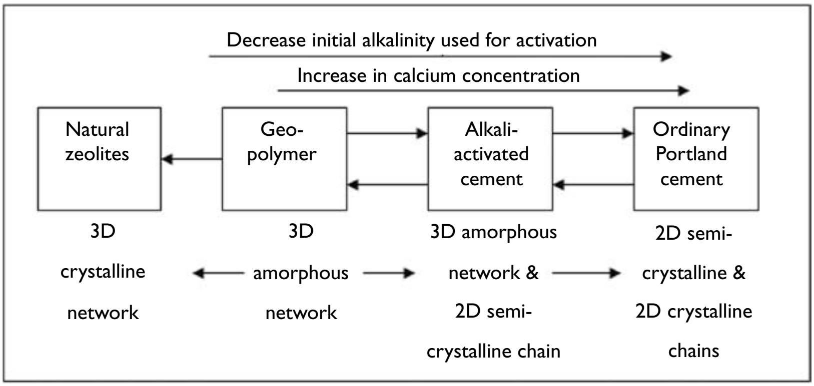


Figure A.5 Conceptual mapping of the relationship between natural zeolites, IPCs (Inorganic polymers), alkali-activated cement and ordinary Portland cement.

Source: Yip and van Deventer, 2001.

blended cement-based system (Yip and van Deventer, 2001). As reported earlier, the tetrahedral structure of IPC without the presence of reactive Ca^{2+} ions could, in fact, be one of the main reasons behind the superior mechanical and physical properties of IPCs relative to OPC based concrete. Calcium silicate hydrate, which is the major binding phase in cement based cementitious materials, could well be formed with the IPC binder given that soluble calcium is available in the mixture (Yip and van Deventer, 2001).

A conceptual map of the relationship between natural zeolites, IPCs, alkali activated cement and ordinary Portland cement has been presented by Yip and van Deventer (2001) and is shown in Fig. A.5.

Given that calcium rich ground granulated blast furnace slag (GGBFS) is added to the system, a cement-based product may result depending upon the systems alkalinity. Yip and van Deventer (2001) used electron microscopy and X-ray diffraction (XRD) to study for the first time the effects of GGBFS on the IPC binder. They found that soluble calcium in GGBFS took part in the formation of a semi-crystalline calcium silicate hydrate, which is the major binding phase in cement based cementitious materials. They also showed that depending upon pH, the IPC binder could remain the same in the presence of GGBFS and that a higher compressive strength was obtained when GGBFS was completely dissolved in the system.

A.4 Physical properties of inorganic polymer concretes

Numerous studies have investigated the chemical and mechanical properties of various inorganic polymer materials. Given correct formulation development and mix-design, inorganic polymeric materials can exhibit combinations of the following properties (Lukey and van Deventer, 2003):

- High compressive strength gain and good abrasion resistance.
- Rapid controllable setting and hardening.
- Fire resistance ($<$1,000°C) and no emission of toxic fumes when heated.
- High level of resistance to a range of different acids.
- Low shrinkage, and low thermal conductivity.
- Adhesion to fresh and old concrete substrates, steel, glass and ceramics.
- High surface definition that replicates mould patterns.
- Inherent protection of steel reinforcing due to high residual pH and low permeability of binder.

It is important to note that not all inorganic polymer products will possess all of these properties. As discussed previously, the ability of inorganic polymers to attain such properties is highly dependent upon raw material composition and synthesis conditions. For this reason, standard chemical dissolution tests, XRF analysis and infrared absorption spectra will provide almost all the necessary information to predict how specific feedstocks, when used as starting materials during synthesis (e.g. coal ash, slag etc.), will affect the final structure and physical properties of IPCs.

Strength gain over time

It has been established that IPC mixes can harden in a matter of a few hours, depending on the mix-design, but the increase in strength of the material continues for months as the reaction proceeds. With the right combination of reactants, it is not difficult to obtain compressive strengths exceeding 90 MPa at 7 days (van Deventer, 2002). Both the early and long term strength of IPCs can be optimized by regulating the composition and particles size of reactants. The chemical composition of the matrix is usually seen as a function of starting materials and synthesis conditions. As mentioned earlier in this chapter, aluminosilicate minerals of different origins react quite differently depending on the phases present, when added to alkaline solutions. An investigation of the effect of mineral properties on the compressive strength of the synthesized IPC has shown that all of the naturally occurring aluminosilicate minerals were at least partly soluble in concentrated alkaline solutions (Xu and van Deventer, 2000).

IPC matrix–aggregate interface

The adhesive property of the cementitious paste (matrix/binder material) is a prime factor in the integrity of any composite material. Enhancement of the adhesive properties of the matrix will result in a stiffer and more stable composite material.

Studying the interface transition zone in concrete, Olivier *et al*. (1995) reported that the microstructure of Portland-cement paste is modified at the interface of the aggregate zone in such a way that the water to cement ratio (w/c) is higher than it is within the bulk matrix. In other words, a zone where the paste has a microstructure different from the surrounding bulk exists around the aggregate particles; hence the overall properties of Portland-concrete depend significantly on the local properties of the interfacial transition zone (ITZ). For example, (Hoshino, 1988) has reported that a considerable increase of the W/C ratio occurs under the aggregate before hardening. Olivier *et al*. (1995), translated this higher w/c ratio into a diffusion process during hydration and described this zone as a heterogeneous area with a porosity gradient. In short, occurrence of micro-cracks in OPC concrete reported as early as the 1960s (Taylor and Broms, 1964) can be attributed to a higher porosity at the interfacial transition zone.

In contrast, the interface between inorganic polymer gels and aggregate (in IPCs) has been found to resemble that of the bulk binder, that is, there is no apparent interfacial transition zone where the porosity of the gel changes (Lee and van Deventer, 2004). Furthermore, it was shown by (Lee and van Deventer, 2004) that the presence of soluble silicates in the initial activating solution was effective in improving the interfacial bonding strengths between coarse basalt aggregates and the inorganic polymer mortar. Despite limited studies on the aggregate/gel interface in IPC systems, this initial work suggests that binder properties (e.g. porosity) remain effectively constant from bulk to aggregate surface and that a greater degree of chemical bonding can occur at the interface, which gives rise to failure under compression through the aggregate particle rather than the gel/aggregate interface (Lee and van Deventer, 2004).

Shrinkage and durability

The deterioration of concrete structures is a major problem in many countries throughout the world prompting the search for methods of predicting the service life of both existing and new structures (Long, Henderson *et al*., 2001). Naturally, the cost associated with replacing the ageing infrastructure is very high. In the US alone, repair and rehabilitation of deteriorating concrete structures costs $100 billion each year, with the total cost of replacing the ageing infrastructure estimated to be over 6 trillion dollars (Penttala, 1997). Scientific research has, therefore, shifted its focus in helping to prolong the durability of concrete by adding admixtures such as fly-ash, condensed silica fume and other fillers (Alexander and Magee, 1999; Aïtcin, 2000; Hassan, Cabrera *et al*., 2000; Bouzoubaâ, Zhang *et al*., 2001). Durability of normal cement based concretes relies primarily on the hydration products associated with calcium silicate clinkers. Workability requires the addition of excess water, generating shrinkage when concrete dries and causing the formation of cracks. These cracks, associated with high porosity, seriously reduce the durability of concrete (Davidovits, 1983).

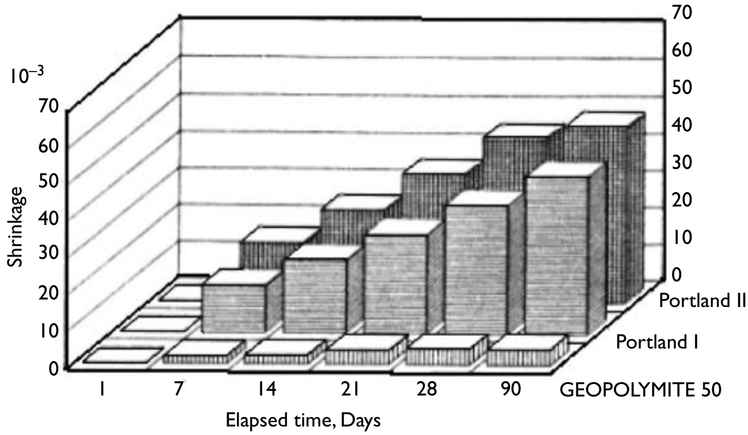


Figure A.6 Shrinkage of inorganic polymer cement (GEOPOLYMITE 50) compared with Portland cement.

Source: Davidovits, 1983.

IPC concretes have been shown to behave quite differently. As mentioned previously, the main reason behind this difference is the fact that the molecular structure and therefore the properties of IPC are different from that of OPC-based concretes. Commenting on the durability of IPCs, it was reported that alkaline cements have higher durability due to their stability and dense microstructure, weather resistance and the absence of long term alkali-aggregate reactions. Davidovits reported that the shrinkage of IPCs in air is very low, preventing the formation of cracks when the IPC dries. This has also been observed on a qualitative basis in recent commercial development trials which are discussed later on in the chapter. A comparison of the shrinkage of IPC with that of OPC-based concrete is reported in Fig. A.6.

Further to the problems associated with aging concrete structures in developed countries, a more serious case where the durability and the stability of concrete are important is the containment of nuclear wastes and heavy metals. With a growing number of nuclear reactors being developed in Third World countries, the issue of long-term encapsulation of the radioactive waste is a problem. Davidovits, Comrie *et al.* (1990), have proposed that IPCs could fulfill this role.

Acid resistance

The reported non-reactive character of inorganic polymers in an aggressive medium provides the basis for the chemical durability of the material. There has been significant research conducted over the years, towards improving the chemical durability of OPC-based concrete. Sewer piping for example has proved such an intractable problem that plastic lining of concrete pipes has been resorted to. However, progress in this regard has usually revolved around the addition of supplementary reactive materials such as silica fume or fly-ash which reduce the porosity of the concrete. Research shows that the mechanical erosion resistance increases moderately in silica-fume concrete (Khedr and Abou-Zeid, 1994). It is clear therefore that there is a need to develop an alternate concrete material, with improved performance in an acidic environment at a cost-competitive position to OPC-based concrete.

Sewer pipes are generally exposed to acid and sulphate attack (the latter due to bacterial conversion of sulphate species to sulphuric acid). In OPC-based concrete, the mechanisms of sulphate attack are reported to be the precipitation of gypsum (i.e. reaction between free lime and sulphate ions) or the formation of ettringite. The formation of both gypsum and ettringite results in expansion and therefore formation of cracking. As IPC contains no free-lime and significantly less calcium than OPC, inorganic polymer concretes are theoretically significantly more sulphate resistant. Although studies are limited at this stage, published work has confirmed such acid resistance (van Jaarsveld, 2000; Allahverdi, A. and Skvara, 2001; Wallah *et al.*, 2003). It is evident however that significantly more work is required to demonstrate the superior acid-resistance of IPC compared with OPC-based concretes in different environments.

Fire resistance

While cement based concrete disintegrates at temperatures above 300°C, inorganic polymers have been successfully used for durable fire-proof protective coatings of concrete structures (Garon, Balaguru *et al.*, 1999). Once again, the advantage is provided by the microstructure and the chemistry of the material. It has been reported that (Na,Ca)-Poly(sialate) and (K,Ca)-Poly(sialate-siloxo) cements provide excellent fire resistant properties up to 1,200°C. The following two reasons are provided (Davidovits, 1999b; Davidovits, Buzzi *et al.*, 1999):

1 The tecto-alumino-silicate type 3D network possesses a nano-porosity that allows physically and chemically bonded water to migrate and evaporate without damaging the cement. The compressive strength of (K, Ca) Poly (sialate-siloxo) cement is in the range of 20 MPa after 3 hours at 1,100°C (90 MPa at 20°C). By comparison, a high-performance Portland blended cement (100 MPa at 20°C) exploded between 300°C and 400°C.
2 The same bonded water (−OH groups) provides high endothermic properties to the substrate. Endothermic regulation is a function of Si:Al ratio.

For a 10 mm thick panel exposed to a 1,000°C flame, the measured reverse-side temperatures reached after 30 minutes of exposure are: Na-Poly(sialate) 180°C; K-Poly(sialate-siloxo) 270°C; K-Poly(sialate-disiloxo) 300°C.

Fig. A.7 shows the strength retentions at elevated temperatures for concretes made of Portland Cements (Portland I, Portland III), high-performance blended Portland (PYRAMENT®), inorganic polymeric cement (K,Ca)-PSS (GEOPOLYMITE 50®) (adapted from 1).

It can be clearly seen that the resistance of (Na, K, Ca) Poly(Sialate-siloxo) is approximately twice that of OPC based concretes at failure. It goes without saying that the performance of reinforced IPC concretes should follow the same pattern. However, the heat transfer and fire resistant properties of fly-ash based IPC concrete are yet to be established. Although significant research work has recently been conducted on the high temperature thermal and structural evolution properties of metakaolinite-based geopolymers (Duxson, 2005); this work has shown the superior thermal conductivity and structural stability of such materials up to 1,000°C.

In comparison to IPCs, Portland cement based concrete appears to be very vulnerable under these actions due to its brittle and inhomogeneous nature and resulting spalling. Different to normal fires that occur due to office combustibles and furniture, the hydrocarbon fires caused by severe blast or impact ignite extremely quickly and may reach almost 1,000°C within a few minutes (Fig. A.8). In Fig. A.8, the standard fire curve is taken from the Australian Standard AS1530.4 (1997) and the hydrocarbon fire curve is plotted from the recommendations of Eurocode EC1 (1995). Concrete deteriorates and looses strength and dimensional stability at high temperatures. The effect is more pronounced with the high rate of temperature rise.

An important part of the fire-resistant design of structures is to ensure that the structure does not collapse due to strength deterioration and instability of

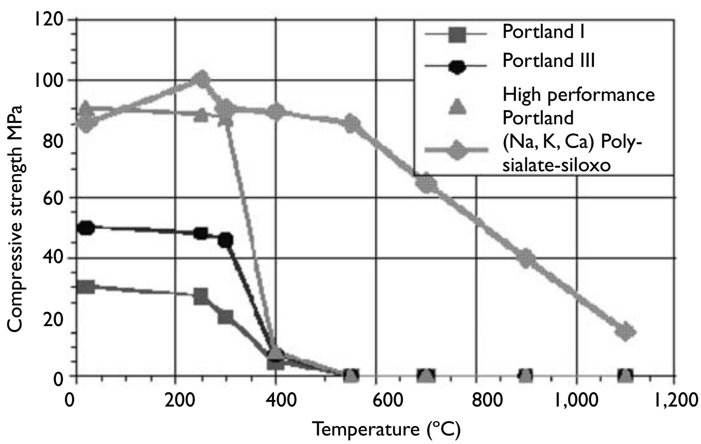


Figure A.7 Strength retentions at elevated temperatures for concretes made of Portland cements and geopolymeric (inorganic polymer) cement.

Source: Davidovits, 1999b.

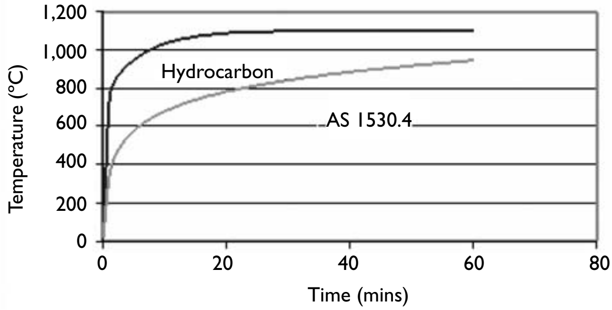


Figure A.8 Temperature development of different heating regimes.

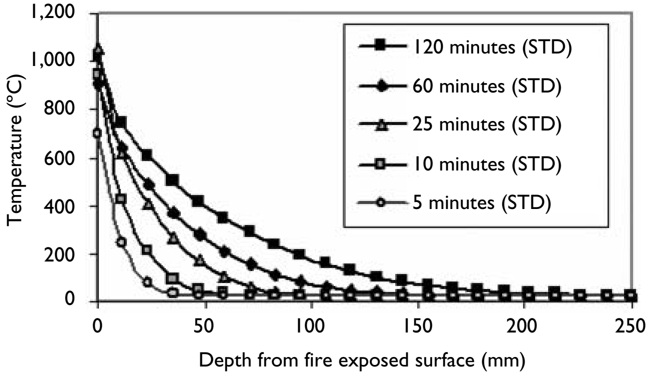


Figure A.9 Temperature development across the thickness.

structural elements during an accidental fire causing human loss. Recent analytical work conducted at the University of Melbourne in collaboration with Monash University has shown that the behaviour of a concrete panel may change significantly if a hydrocarbon heating regime is used instead of the standard heating regime suggested in AS1530.4. The temperature distribution across the thickness of a 250 mm thick concrete panel calculated from a program developed as part of this collaborative study is shown in Fig. A.9. As seen, the hydrocarbon fire (HC) heats the specimen rapidly, with a steep temperature gradient across the section. The probability of a structure subjected to a serious fire is significant enough that it is a major design consideration in many important structures. Most materials deteriorate and lose strength and dimensional stability at high temperatures.

Given this background on the known deterioration of large concrete structures (i.e. walls, ceilings, buildings, etc.) at elevated temperatures, with subsequent loss of human life, there is an immediate need to develop and test new structural load-bearing materials with superior fire resistant properties. In simple terms, there is an immediate need to gain a better understanding of how and on what basis could IPCs be developed and used as a replacement for conventional concrete where loss of life due to extreme fire is a concern.

A.5 Engineering properties of IPCs

Due to their difference with OPC-based concretes, both in terms of chemical reaction and matrix formation, IPC may exhibit different engineering properties from that of ordinary concrete. Simply relying on compressive strength of the material and extrapolating models and equations meant for OPCs may lead to unsafe designs. Therefore, it is imperative to be aware of the structural behaviour and the properties of IPC before it is considered as a suitable substitute for OPC-based concrete in reinforced structural applications.

At the present time, there are two main groups developing a knowledge base around the engineering properties of IPCs; namely Professor Vijay Rangan's Group at Curtin University in Australia, and that of Professor Priyan Mendis in the Department of Civil and Environmental Engineering at the University of Melbourne, Australia. It is expected that more testing and validation of the engineering properties of IPCs will be conducted once non-structural IPC components are well accepted in the market and the cement and building products industry.

A series of work published by Prof Rangan has established drying shrinkage, creep and sulphate resistance properties for a selection of IPC mixes. In particular it has been shown that drying shrinkage strains are extremely small indeed, and the ratio of creep strain-to-elastic strain (the creep factor) reached a value of 0.30 in approximately 6 weeks (Hardjito *et al.*, 2004a). Moreover, large inorganic polymer concrete members have been formed and subjected to extensive testing, including the determination of compressive, flexural and tensile strength properties pursuant to relevant Australian Standards and Codes of Practice (Hardjito *et al.*, 2004a,b,c; Wallah *et al.*, 2004a,b). Overall the work demonstrates that IPCs exhibit at least the same engineering performance criteria, and many instance improved performance, compared to OPC-based concretes.

Sofi (2003), has also investigated the engineering properties of a series of inorganic polymer concretes; each mix-design was fully-costed and shown to be cost-competitive to a comparable OPC-based concrete. The mix-designs developed by (Sofi, 2003) were different in composition and the type of fly-ash used. The engineering properties that were tested included the compressive, tensile, flexural strengths, Young's modulus of Elasticity and Poisson's ratio and bond performance of reinforcing bars in IPC concrete. The results were compared with similar tests carried out on OPC concrete. The key findings of this impressive selection of work are detailed later. Further details on exact mix-designs, curing methods and testing regimes can be found in (Sofi, 2003) and will not be repeated here.

Compressive, splitting tensile and flexural strength of IPCs

The compressive strength and splitting tensile and flexural strength values for the IPC mixes used by (Sofi, 2003) are listed in Table A.2. These were measured following the procedure prescribed by AS 1012.9, 10 and 11, respectively. It is clear from Table A.2 that the splitting tensile strengths of the IPC mixes are, on average, approximately half that of the square root of their compressive strengths. However, it is reasonable to assume that the splitting tensile strength of IPC also depends on other parameters such as the mix compositions and curing methods.

Similarities between the splitting tensile and flexural strengths of the IPC mixes can be observed from Table A.2. On average the difference between flexural and splitting tensile strength of IPC mixes is about 2.0 MPa, both for 7 and 28 days. It is noteworthy that this is the case even though, for example, mixes 1, 2 and 3, are essentially the same mix-design yet a different source of fly-ash has been used. This shows that robust formulations that exhibit similar engineering properties can be achieved.

Bond to steel reinforcement

Reinforced concrete functions effectively as a composite material because the steel reinforcement is bonded to the surrounding concrete (Warner, Rangan *et al.*, 1998). Bond between the rebar and the surrounding concrete matrix ensures the rebar does not slip relative to the concrete and therefore allows local forces to be transferred across the steel-concrete interface. Without any bond, or other mechanical connection, the steel is completely ineffective and does not contribute to a greater stiffness and flexural resistance of the structural member.

In the work of (Sofi, 2003), the bond behaviour of inorganic polymer concrete to rebar and the influence of different variables has been investigated by two types of bond tests, pullout and beam-ends. Beam-end specimens (also referred to as inverted half-beam specimens) are used as a more realistic bond test. They are used extensively in experiments to evaluate bond strength of rebars with concrete

Table A.2 Compressive (f_c) splitting tensile (f_{sts}) and flexural strengths (f_{cf}) of IPC mixes. Tested in accordance to relevant Australian standards

Mix	*ρ (kg/m^3)*	*7 day f_c (MPa)*	*28 day f_c (MPa)*	*7 day f_{sts} (MPa)*	*28 day f_{sts} (MPa)*	*7 day f_{cf} (MPa)*	*28 day f_{cf} (MPa)*
1	2,231.3	35.2	55.4	3.2	3.4	4.9	6.1
2	2,232.1	44.4	54.0	2.9	2.8	4.8	4.9
3	2,147.7	37.6	48.6	2.4	2.8	4.5	5.4
4	2,408.0	41.8	56.5	3.6	4.1	5.3	6.2
5	2,212.1	42.0	47.0	3.5	3.9	5.3	5.9
6	2,246.4	38.3	52.8	2.7	3.3	4.2	5.3

Source*:* Sofi, 2003.

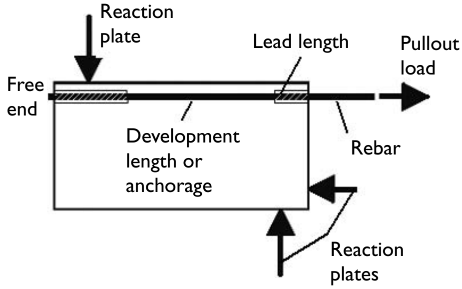


Figure A.10 Beam-end specimen and terminology.

and its derivatives (Abrishami and Mitchell, 1996; Mendis and French, 2000). The specimen allows the test rebar to be in an area of flexural tension as shown in Fig. A.10. (ASTM A 944–99, 2000) provides a description of the methods of construction and testing of beam-end specimens, which was adopted by (Sofi, 2003).

A splitting type failure was observed for all beam-end IPC specimens. In almost all of the specimens, the bond-splitting cracks happened perpendicular to the smallest concrete cover. The splitting type failures were explosive and sudden for all samples studied denoting the brittle nature of the material irrespective of the composition. The failures were mainly over the development length irrespective of the rebar size.

The similarity between bond stress (U_{av}) and splitting tensile strengths (f_{sts})

Bond failure occurs when the hoop tension exceeds the tensile capacity of the concrete. When this occurs, longitudinal cracking develops and since the force in the 'struts' can no longer be equilibrated, failure occurs, the cover breaks off and the rebar pulls out. In beam-end specimens, tensile strength of the material relates closely with bond strengths (Fig. A.11) for the variety of IPC mixes studied. Once again, these results show that given robust mix-design of IPC concrete, then the engineering properties of the final materials are quite similar.

Comparison with the code provisions

A comparison of the standard design equations with the experimental bond stress results have been carried out recently (Sofi, 2003). It gave valuable insights into the level of bond stress attained by the IPC mixes in comparison with the models provided by the code provisions. The code provisions give the l_d values required at yield strength of rebars in tension. The experimental bond stress values were normalized by the standard models, for example, $U_{test}/U_{AS\ 3600}$ for the Australian

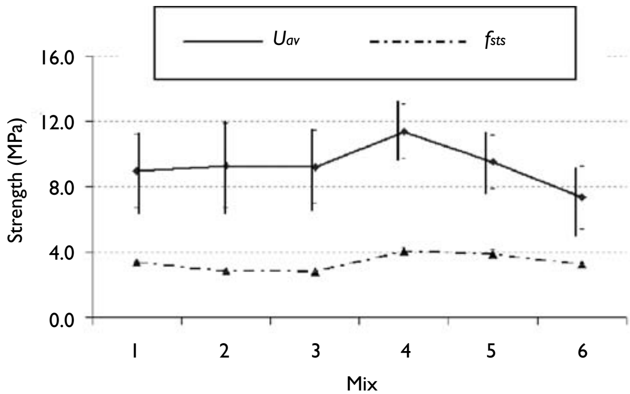


Figure A.11 Average beam-end bond strength *(U_{av})* and corresponding tensile strength *(f_{sts})*.

standards. Comparison of bond stress values from the current investigation with AS 3600, ACI-02 and EC2 recommendations showed that the provisions are conservative for predicting l_d values for IPC mixes.

The physical and engineering properties attainable from inorganic polymer materials are only one aspect that needs to be considered for successful commercialization of inorganic polymer technology. The ability to identify the drivers for the uptake of the technology as well as develop a well-defined value proposition for each of these specific opportunities is also essential.

A.6 Drivers for IPC technology uptake

Technology drivers

The key technology drivers for the uptake of inorganic polymer technology are very powerful and will therefore contribute to the longer term commercial success of the technology. These drivers include (Lukey, 2005):

- The ability to use a variety of industrial by-products as feedstocks, including coal ash and granulated blast furnace slag.
- The production of building materials with superior chemical and mechanical properties such as acid and fire resistance, thermal stability and durability.
- Versatility of formulations that enable the tailor-design of mixes to achieve desired workability specifications, for example stiff mixes to quite fluid and self-compacting concretes.
- Reduction in greenhouse gas emissions relative to OPC.
- Competitive total costs relative to OPC-based binders.

The important point to make is that these are properties and product attributes that are generally obtained for inorganic polymers. However, not all inorganic polymers exhibit all these properties, that is there is no single all-encompassing formulation. However, with correct knowledge of raw material behaviour and chemistry, it is possible to tailor the material to attain combinations of the above properties for both cost and technical performance.

Environmental drivers

Due to the use of industrial by-products such as coal ash and blast furnace slag as reactive raw materials, the manufacture of inorganic polymeric materials does not in itself generate or liberate carbon dioxide. In comparison, the manufacture of Portland cement (essentially the calcining of limestone) generates approximately one tonne of carbon dioxide per tonne of cement. The world-wide cement industry currently accounts for approximately 8–12% of total global carbon dioxide emissions, and potentially this value could rise in future years as large-scale plants in China to come on-line to satisfy the increasing demand for concrete in new infrastructure projects.

With the introduction of carbon emission taxes in the UK and Europe, there is an immediate need for the cement industry to move towards a sustainable base to ensure continuing viability in the future. Currently, the industry has addressed this issue mainly through the use of alternative fuels to fire the kiln and also the addition of supplementary materials such as blast furnace slag and fly-ash to clinker (up to 30% and 70% substitution of clinker with fly-ash and blast-furnace slag respectively). However, it is commonly acknowledged that these initiatives will not go far enough in terms of satisfying emission reduction targets.

Although calcium aluminate and sulfo-aluminate cements are potential alternatives, it is important to note that even when the emissions involved in making the activators (alkali and silicate) used in a inorganic polymer process are included, the production of inorganic polymer binders emits as little as one-tenth the amount of greenhouse gases of Portland cement.

Commercial drivers

The key commercial drivers for uptake of inorganic polymer technology by the cement and concrete industry are:

- Unique and superior performance properties to OPC-based products, thereby enabling use in diverse applications, that is refractory adhesives, cement replacement, advanced low-temperature ceramics and composites.
- Robust and versatile manufacturing process.
- Potential reduction in cost of production (the most important driver, irrespective of superior product performance).

In determining the value proposition that inorganic polymer technology may offer compared to ordinary Portland cement in a given application, it is important not to limit the assessment of cost or value to that of tonne of cement or a cubic metre of concrete. This is because in many cases the cost of coal ash, blast furnace or metakaolinite is artificial and can therefore vary considerably from different sources or countries. Moreover, the cost of the activating materials to form inorganic polymers (i.e. alkali and silicate) can vary by an order of magnitude depending on country and level of purity needed. Other factors should also be included in the value proposition, including for example the decrease in variable costs due to reduced mould turn around times, thinner cross-sections in manufactured product (without loss of material property), decreased curing times and temperatures, lower cranage costs and also lower foundation requirements.

A.7 Potential IPC applications

Given that the chemistry of inorganic polymers can be tailored, this gives rise to a diverse range of possible applications for inorganic polymer products in the construction and building products industry (Fig. A.12).

As noted from Fig. A.12, inorganic polymer materials have the potential to be used as either substitutes or total replacements for OPC-based concretes, organic polymer based coatings, as well as some ceramic materials in low-temperature refractory applications. The commercial uptake of inorganic polymer technology therefore provides an opportunity for bulk use of coal ash and/or blast furnace slag to form a variety of value-add products.

In the first instance, it is to be expected that initial IPC products developed to a commercial-ready status will be non-structural and therefore suited to low-risk applications (e.g. un-reinforced small diameter pipe, pavers, interior partitions etc.). This is necessary because IPC is a ‘new’ technology, considered to be unproven by the industry, and insufficient data currently exists on the durability of IPCs when exposed to various components. The building products industry is risk averse; this is by necessity given that failure of product could lead to loss of human live. For this reason, structural IPC members are not expected to be taken to market for quite some time at this stage of development. This is despite recent development work demonstrating clear improvement in performance for such structural components (to be discussed later in this chapter).

In addition to manufacture of building products, inorganic polymer technology could also find application in the mining industry, namely stabilization and solidification of mine tailings, paste-back filling of mines, and immobilization of heavy metals and radioactive wastes (Davidovits, 1988; Davidovits, Comrie *et al.*, 1990; van Jaarsveld, van Deventer *et al.*, 1997; Herman, Kunze *et al.*, 1999).

With respect to waste treatment and immobilization, a literature review by van Jaarsveld (2000) summarized the possible applications of inorganic binders as follows:

(a) Surface capping of waste dumps and landfill sites where a rigid high strength structure is needed to prevent contact by rainwater and to provide a solid and

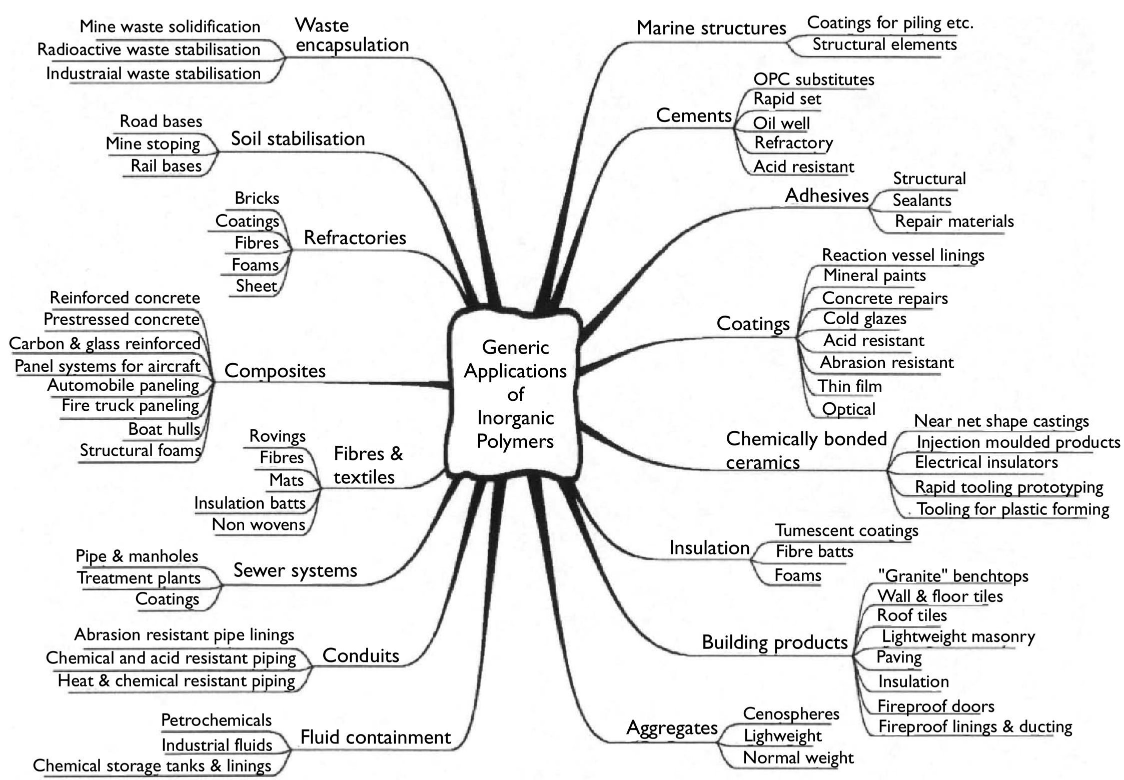


Figure A.12 Potential applications of inorganic polymer technology.

Source: Harper *et al.*, 2002.

safe cover which can also assist in utilizing the area for building purposes (CANMET, 1988).

(b) Low permeability base liners of landfill sites where minimum leakage of contaminants into the groundwater is desired or where fresh water reservoirs need a lining to prevent water from seeping away as in regions where not enough clay is present in the soil.

(c) Vertical barriers and water control structures where water deflection is needed, both above and below the surface.

(d) Dam construction as well as the stabilising of tailings dams, the latter being a large problem in countries with high humidity. The in situ treatment of tailings in order to increase their solidification potential will also enable mining in environmentally sensitive areas where it might not be possible to mine at the moment due to the threat of not only physically unstable tailings dams but also of leaching of toxic metals into fresh water drainage systems.

(e) Heap leach pads are another possible application where a large, cheap, non-porous, non-permeable and non-reactive surface is needed for the leaching of ores and collection of leachate.

(f) Structural surfaces like floor and storage areas as well as runways have also been proposed and (Malone, Kirkpatrick *et al.*, 1986) investigated the feasibility of the latter.

(g) Intermittent horizontal barriers in waste masses, used to keep waste masses stable and prevent contact between various layers stacked on top of one another. In this case the properties required include low-permeability and intermediate strength.

(h) Back fill for cut-and-fill and under-cut-and-fill type mining methods. Fast setting and high early strength are required for this application, both of which can be met by IP reaction. The abundance of mine tailings as well as the relatively high temperatures found in most mines should favour the application of IP reactions and definitely merits a thorough investigation.

(i) Immobilization of toxic waste such as arsenic, mercury and lead.

(j) Inexpensive but durable encapsulation of hazardous waste such as asbestos and radioactive wastes. Manufacturing inorganic polymer materials from waste should provide cheap encapsulation media for a variety of applications where Portland cement might be too expensive or not sufficiently durable.

A.8 Current challenges and obstacles

The current challenge for inorganic polymer technology is to establish itself as a recognized, viable and proven technology that can be utilized in a range of applications. Many published studies have independently verified the favourable results for material properties. However, to date the technology has not been adopted on a routine commercial scale for such applications. Despite the afore-mentioned drivers for the further development and commercialization of inorganic polymer technology, there are a number of commercial barriers that remain in place, namely:

Scientific credibility of inorganic polymer science. Due to the lack of uniform nomenclature, there is a perceived lack of fundamental knowledge regarding inorganic polymer chemistry. Some workers have termed the material an aluminosilicate gel, while others a glass, alkali-activated cement, or even a low-temperature ceramic. It is clear therefore that successful commercialization of the technology will require the various structures proposed for these materials to be substantiated and quantified.

Use of a high-level of alkali. Inorganic polymer technology essentially involves the alkali-activation of coal ash to form a hardened solidified product. However, it is universally known in the cement industry that high alkali is bad and leads to degradation of cement products, namely due to alkali-aggregate reaction. Although inorganic polymer materials can resemble OPC products and have similar applications, the chemistry is entirely different. There is therefore a need to overcome such entrenched attitudes across an industry (i.e. that alkali is bad for all systems), prior to inorganic polymer technology being adopted on a larger scale.

Standards and product liability. Inorganic polymeric materials do not contain OPC and therefore they still need to be tested, accredited and approved into building codes and standards. In most countries these standards are shifting from a compositional based approach to that of being performance based. This facilitates the use and acceptance of alternative materials such as inorganic polymers to conventional cement because both the need for regulation (which provides confidence or safety as to performance) and innovation are addressed. The often raised issue of durability and how these new materials will perform over time is also a commercial barrier at present.

Conservative industry. The conservative nature of the construction and building products industry is rightly justified; if a product fails, there is a likelihood of human loss. However, due to this conservatism, innovation in the industry is low and the uptake of new low-risk technologies is not a priority. In order to develop a inorganic polymer industry, it is necessary to gain the greater acceptance of the technology by potential manufacturers and end-users, that is the technical and commercial virtues need to be 'de-risked'. This can be achieved through a more open dialogue approach between academia and industry, and also the wider dissemination of basic knowledge and information in the public domain which is suitable for specifiers and engineers.

A.9 Advances in commercial development of IPC products

As shown in Fig. A.12, there are numerous applications for inorganic polymer technology. It is this diverse range of applications which has most probably held up the commercialization of the technology so far. This is because no single entity has remained focused on a single product development initiative; rather an almost 'shot-gun' approach has been taken to product development, that is ceramic components, precast concrete, protective mortars, paste back-fill of mines. Each of these products would require an inorganic polymer that exhibits completely different performance specifications and cost criteria; thereby making the development of a fully tested single product almost impossible.

The approach taken by Siloxo Pty Ltd (Australia) in conjunction with its foundation partner was to develop the fundamental understanding and practical expertise working with the technology to manufacture ready-mix IPC. The key objective was to deliver a lower cost product with improved performance properties. This was considered a 'blue-sky' approach to the technology. However, if it were technically established that ready-mix IPC could be developed, mixture behaved the same as ordinary concrete, cost-competitive and so forth, then the transfer of such knowledge to the manufacture of pre-cast elements in more controlled environments would be rather easy. This is because all the technical challenges (i.e. retention of workability, increased setting time, controlled strength profile development, durability etc.) would have been overcome. Detailed later are two examples of the key product development initiatives undertaken and outcomes successfully achieved.

Ready-mix

Fig. A.13 shows two examples of successful trials undertaken with IPC ready-mix concrete. In particular, Fig. A.13(A) is of a low-strength (25 MPa) IPC mix using a Class C fly-ash (i.e. a high calcium content in the ash). As depicted, the IPC slab is to be used as a road for trucks and other traffic to pass over on a regular basis. Fig. A.13(B) is of a high-strength (+80 MPa) IPC mix developed using a Class F fly-ash (i.e. low calcium content in the ash).

It is important to note that the mix-designs have been tailor-designed so that conventional concrete-batching facilities are used for charging the concrete trucks, followed by normal delivery of concrete to the work site, pouring and placement of the material, followed by standard concrete finishing techniques. These trials conducted over many years have established that IPC technology is, provided correct mix-design and formulation development, a replacement for Ordinary Portland Cement. This has significant commercial implications for companies with a current cement position, and also those with no cement operations but looking at establishing themselves as a leader in IPC technology.

Interior beam-column joints

Work has also been undertaken more recently on the development and testing of structural inorganic polymer concrete members (Brookes *et al*., 2005). In particular, interior beam-column joints specifically designed using the capacity design methods given in a draft of the new New Zealand concrete design standard

(A)

(B)

Figure A.13 Application of ready-mix IPC low-strength (25 MPa) and high-strength (+80 MPa) materials developed by Siloxo Pty Ltd (Australia) in conjunction with industry partners.

(DZ 3101.1 rel.2, 2004). For all units the nominal concrete strength was 30 MPa, and external dimensions were identical (Fig. A.14).

Before it is possible to adopt inorganic polymer based concrete for structural purposes it is essential that existing design procedures are verified for the new material. The work by Brookes *et al.*, 2005 describes the cyclic testing of three beam-column joint sub-assemblies made of inorganic polymer based concrete, in

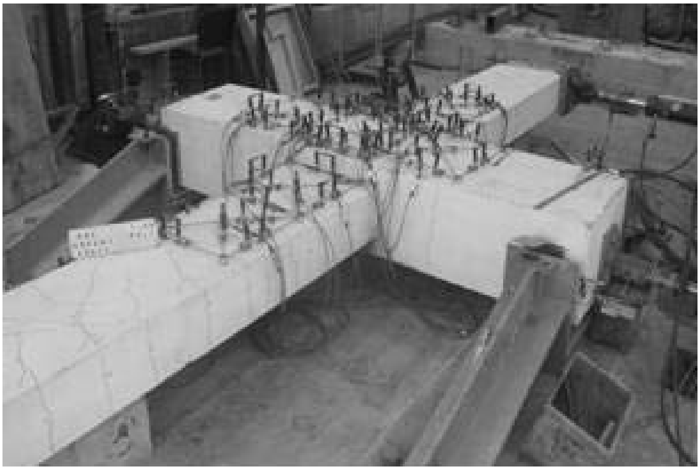


Figure A.14 IPC member being tested for structural properties.
Source: Brookes *et al.*, 2005.

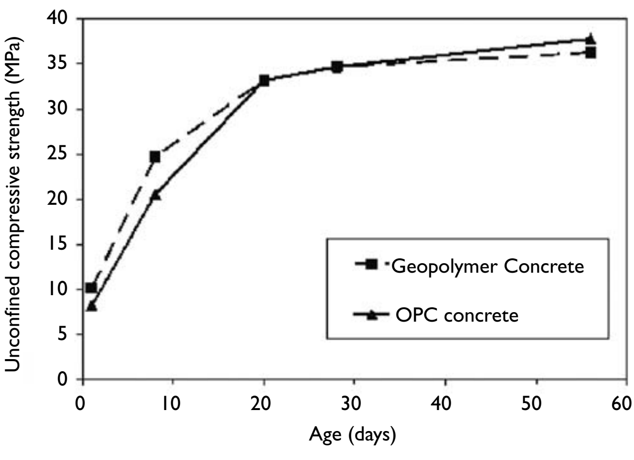


Figure A.15 Variation of strength with time for the two concrete types used.
Source: Brookes *et al.*, 2005.

parallel to the testing of a control unit made of ordinary Portland cement (OPC) concrete.

Fig. A.15 shows how the strengths of the inorganic polymer and OPC based concrete mixes used in the work of (Brookes *et al.*, 2005) increased over time. Both materials had a design compressive strength at 28 days of 30 MPa. It can be seen that both materials achieved this strength, and continued to increase in strength at a similar rate, showing that the strength of inorganic polymer concrete can be predicted reliably.

Paulay and Priestley, 1992 have stated that to ensure good ductile performance of reinforced concrete it is important that multiple cracks form at a fairly regular intervals to allow plastic deformation to spread over a significant length of reinforcement, thus preventing reinforcement strains from reaching excessive levels. It is therefore important to note that the work by Brookes *et al.*, 2005 has shown that no significant differences were detected between the crack patterns of units constructed of inorganic polymer concrete and Portland cement concrete. This is a significant observation because it indicates the IPC performs essentially the same as OPC-based concrete under seismic loading.

The hysteretic performance of IPC and OPC-based members were also studied in the work of Brookes *et al.*, 2005. It was shown that the performance of the two units was essentially identical. Given that these units had identical reinforcement, this similarity of hysteretic shape is a strong indicator that inorganic polymer based concrete performs very similarly to Portland cement concrete.

It was concluded in the work by Brookes *et al.*, 2005 that inorganic polymer concrete can be used in beam-column joints designed to meet seismic performance criteria. Such joints can be designed using existing design standards, and performance will be indistinguishable from similar joints constructed of OPC concrete. This is a significant result and paves the way for further and more substantial development of structural IPC.

A.10 Conclusions (KWD)

It is apparent that a massive research into IPC is underway and is producing very promising results. The chemical reactions involved have been presented in detail since they will be novel to most readers. The material is extremely attractive since it not only uses a waste material but, in replacing Portland cement, reduces carbon dioxide emissions currently causing substantial concern world wide.

It is clearly not to be viewed as an inferior substitute for OPC but as a material having some properties far in advance of that material. Examples are fire and chemical attack resistance and the expectation that it could provide long-term encapsulation of nuclear and other dangerous wastes.

While some caution may be justified in immediately launching into wide scale use of the material, it is to be hoped that it will not be subject to the unreasonable delays and prejudices so often experienced in the concrete field. Fifty years ago the author was involved in the laying of a short pipeline composed of several different experimental pipes in the most aggressive part of the Melbourne sewerage system. It is clear that such a trial using IPC is urgently justified. (the trial referred to resulted in a decision to use plastic lining!). Considering the current extent of expenditure on anti-terrorism measures in general, it is surely obvious that no expense should be spared in the urgent large scale investigation of the use of the material for structures.

The very rapid strength development available, while a problem to be overcome in in situ structures, could make the material especially valuable for precast products.

Glossary

ACI	American Concrete Institute
CI	Concrete International – (ACI magazine carrying many of author's papers)
CIA	Concrete Institute of Australia
CSIRO	Commonwealth Scientific and Industrial Research Organisation (of Australia)
EA	Equivalent Age (Arrhenius function for concrete maturity)
EFNARC	European Federation for Specialist Construction Chemicals and Concrete Systems (website www.efnarc.org)
EN206	A concrete code established by a European Committee of 40+ nations
FM	Fineness Modulus
GI	The author's Gap Index, a measure of grading continuity.
GPC	Geopolymer Concrete, more properly called IPC
ICT	UK Institute of Concrete Technology
IPC	Inorganic Polymer Concrete
ISO 9001	An administrative QC procedure originated by the International Standards Assn – it is not specific to concrete
LHS	Left Hand Side
MSF	Mix Suitability Factor – the author's index of cohesion/sandiness
NATA	Australian National Assn of Testing Authorities
NDT	Non-Destructive Testing
NRMCA	US National Ready Mixed Concrete Assn
P2P	An acronym for a Prescription to Performance change in specifications
QA	Quality Assurance
QC	Quality Control
RHS	Right Hand Side
Road Note 4	A mix design system developed by the Road Research Lab (UK) in the 1940s, now only of value for its type grading curves
RWD	Relative Water Demand (of two sands)

SCC	Self-Compacting Concrete
SG	Specific Gravity, otherwise known asAPD or Average Particle Density
SS	Specific surface, usually the author's modified version of this
SWC	Super Workable Concrete – essentially another name for SCC
TTF	Time Temperature Function (for concrete maturity)
VMA	Viscosity modifying agent (e.g. methyl cellulose)
Website	The author's website www.kenday.id.au

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NB: Many of the author's papers and publications appear on his website www.kenday.id.au

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