# Development of an expert system for reinforced concrete bridge repair

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# Abstract

Current techniques for selecting reinforced concrete repair materials are often based on ad-hoc methods for specifying repair material properties. The inherent lack of understanding of material behaviour in this approach can lead to premature failure of repairs.

This research has examined state of the art methods for repair material property specification and has developed a technique, specifically for application in a computer program, which recommends optimum repair material properties tailored to given repair situations. The technique developed achieves compatibility between the repair material and the substrate concrete through a sophisticated balancing of those repair material properties identified as important, specifically; elastic modulus, shrinkage, creep and tensile strength. Adopting the developed technique minimises the possibility of failure of the repair material.

The developed repair material property selection technique is seamlessly integrated into an *expert system for reinforced concrete bridge repair* also developed as part of this research. A technique has been produced to quickly elicit the complex decision making process of reinforced concrete experts and represent their information in a computer program.

The developed expert system diagnoses the causes of reinforced concrete defects. Importantly, the program utilises its in-built intelligence to determine if the severity and extent of the defects identified warrant genuine concern.

In order to facilitate efficient inputting of data into the expert system by prospective users, an elemental graphical interface was developed, allowing users to quickly assemble on-screen three dimensional representations of the affected concrete elements. Thereafter, program users locate areas of defects onto the on-screen concrete elements and the inputted data can be interrogated by the expert system.

Adopting the mainly graphical approach of data input, the expert system diagnoses reinforced concrete defects, proffers prognoses for concrete elements themselves (such as piers, columns, abutments), recommends testing regimes to confirm the expert system output, and recommends repair techniques.

Should the recommendation of the expert system be to break out and replace defective concrete, the technique to recommend optimum repair material properties, developed in this research, will offer its recommendations.

The developed expert system for reinforced concrete repair acts as an expert guide through all aspects of bridge inspection and repair. For the assessment of defects it draws together best practice recommendations from literature and experts. For the recommendation of repair material properties it implements the technique developed in the research.

The completed research has been incorporated into a commercially available bridge management system (www.bridgemanagementexpert.com).

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## Candidate's declaration

I hereby declare that no portion of the work referred to in this thesis has been submitted in support of an application for another degree or qualification of this or any other university or other institution of learning. All sources of information have been duly acknowledged.

Candidate

Laurence Green – October 2005

Director of studies

Pritpal Mangat – October 2005

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# 1 Introduction

*Chapter objectives*

- To introduce the research
- To discuss the aims, objectives and methodology of the research
- To describe the layout of the thesis

## *1.1 General*

An expert system for concrete repair is an intelligent software adviser that can assist an engineer across the range of activities involved in the concrete repair process, specifically:

- Inspection
- Diagnosis
- Testing
- Repair methods
- Repair materials
- Prioritisation

Expert systems, also known as knowledge based systems, are software programs which employ logical reasoning instead of quantitative calculations in their processes.

## 1.2 Objectives

This research develops a software expert to aid reinforced concrete evaluation and repair. Existing expert system and knowledge based system development tools will be used to develop a program to diagnose concrete defects. A method for assessing the severity and extent of reinforced concrete defects will be developed to work seamlessly with the defect diagnosis component.

Crucially, a routine will be developed, based on state of the art research, to recommend the properties of materials for reinforced concrete repair.

To the best of the author's knowledge, this is the first attempt to create an expert system for concrete repair with the intelligence to judge (and consider in its recommendations) the severity and extent of defects.

The aim of this project is to develop a software system to provide decision support for civil engineers involved in reinforced concrete maintenance. The system will upgrade the performance of engineers and enhance and verify their decision making.

## 1.3 Methodology

Rules and guidelines for reinforced concrete repair are poorly structured. The collation of these rules into a knowledge base requires the input of well qualified experts. This has been achieved in the thesis and as such, knowledge in this domain is well suited for exploitation in an expert system. Conversely, the calculation of suitable properties required for satisfactory performance of repair materials for reinforced concrete repair is purely mathematical and can be handled with standard algorithmic programming. Problems which can be solved heuristically only, are suitable for use in expert systems[1], therefore, the approach of this project will be to address the selected problems with the appropriate software technology.

The system will be formed by three key modules each working together seamlessly behind a front-end bridge management system developed by a collaborator. A heuristic expert system module will diagnose concrete defects and recommend tests and solutions. An algorithmic system will asses the severity and extent of defects. A second algorithm based system will recommend repair material properties.

In order to satisfy the objectives of this research the following key tasks were performed:

- Knowledge acquisition sessions with professional experts
- Heuristic system development
- Severity and extent algorithm software development
- Repair material property recommendation algorithm software development
- Interaction with bridge management system development.

## 1.4 Layout

| | |
|---|---|
| Chapter 1 | General introduction to thesis and discussion of research objectives and methodology. |
| Chapter 2 | A review of the key domains relevant to this thesis, specifically: bridge management, concrete defects, defect identification, and concrete repair. This chapter also includes a brief review of existing prototype expert systems in this domain. |
| Chapter 3 | This chapter contains an extensive literature review on current thinking regarding patch repair of reinforced concrete. It establishes known misconceptions amongst practitioners and highlights new research thereby establishing a sound theoretical basis on which to develop a system to recommend optimum repair material properties. |
| Chapter 4 | Presents the development of an analytical procedure to recommend optimum properties of reinforced concrete repair materials. The technique developed is exploited in a computer program integrated seamlessly into the expert system for concrete repair. |
| Chapter 5 | Develops the diagnostic expert system for reinforced concrete repair and the tools to assess the severity and extent of concrete defects. The chapter |

establishes the knowledge acquisition processes and how the knowledge obtained is interpreted and coded into the software.

Chapter 6 Reviews the completed expert system, which is embedded into a bridge management system in order to fully computerize concrete structure maintenance, from inspection to repair. The chapter discusses how the software modules developed in the thesis are practically applied in the expert system.

Chapter 7 Reviews and assesses the research, discussing the conclusions drawn.

Chapter 8 Discusses future research efforts and ways the system can be updated and expanded.

# 2 Reinforced Concrete Bridge repair: an overview.

## 2.1 Chapter Objectives

- Discuss reinforced concrete repair
- Discuss application of expert system technology in reinforced concrete repair domain
- Identify existing expert systems in field of concrete repair

## 2.2 Introduction

In the UK, there are over 50,000 bridges constructed from reinforced or pre-stressed concrete[2]. Those on motorways and trunk roads in England fall under the jurisdiction of the Highways Agency[3] and similar agencies are responsible for structures on such roads in the rest of the UK. Bridges on local roads are the responsibility of local authorities.

Concrete was once considered to be a durable material requiring little or no maintenance[4,5], however, it has been recognised for some years that concrete is susceptible to degradation caused by aggressive chemical attack and adverse reactions to the natural environment[6]. As a result of deterioration, bridges can become aesthetically unacceptable, deterioration can lead to faster rates of further degradation, the service life of structures can be reduced and in severe cases the structural capacity of structures can be affected.

In his study of 200 concrete bridges in England, *Wallbank*[4] classed 114 as being in fair condition and 61 as having serious defects. More recently, almost one third of United States bridges were reported as substandard[7].

Although modern structures can be constructed with inbuilt protection against commonly known causes of deterioration, the vast majority of reinforced concrete bridges are over 20 years old. The replacement value of all UK concrete bridges is many billions of pounds and as such, the only option for the preservation of bridge infrastructure is suitable maintenance and repair schemes.

## 2.3 Bridge inspection

The purpose of a bridge inspection is to allow an inspector to observe and record defects present on a structure[8]. Diagnosis therefore, is not strictly a part of the inspection process, although the two are closely related. Current practise for the inspection of bridges in the UK is set out in volume three of the Design Manual for Roads and Bridges[9]. Inspections currently fall into four categories.

- Superficial Inspection : Cursory checks of structures whenever those with responsibilities towards a bridge happen to be passing it. Any major visible problems are reported to the bridge engineer.
- General Inspection : A visual examination of all parts of the bridge in order to ascertain the condition of all elements. These inspections are undertaken every two years and observations are made from the ground using binoculars where necessary.
- Principal Inspection: This inspection involves a closer examination of all parts of a bridge. It is usually a surface inspection which can involve the use of access equipment and traffic management. A principal inspection is completed every 6 years. A full report is produced from the inspection.

- Special Inspection: A close inspection of a particular area or defect. Usually as a follow up on a defect identified in a previous inspection[10].

Recently, the Highways Agency has decided to review the inspection procedures, and implement new inspection routines to take account of the condition of bridges and bridge elements in order to decide the type and frequency of inspections. This was to involve a 'Benchmark Inspection' replacing the current 'Principal Inspection'. The Benchmark Inspection would take place at intervals between 6 and 24 years depending of the reported condition of the bridge from the last benchmark inspection[11]. To date, the Highways Agency have not replaced the current inspection procedures with these proposed ones and there is no information indicating if the change will now occur.

Concrete bridges are required to maintain their serviceability over long periods of time[12]. Typical concrete, cast as part of a highway bridge, is unlikely to resist deterioration over its design life (usually 120 years), and is likely to require repairing in order to maintain the serviceable state of the bridge.

Figure 2.1 shows how a typical bridge becomes less reliable with age, and how regular maintenance combats the fall in reliability. To counteract the loss of reliability with age, concrete in bridges needs to be maintained and treated for disease whenever it exhibits typical signs of distress (and occasionally when there are no visible signs of distress). If deterioration is not regularly arrested, the cost of restoring reliability becomes much greater than a regularly maintained structure. Assessing the effect of short term expenditure through regular maintenance on the long term financial costs of a bridge (or any structure) is known as whole life costing.

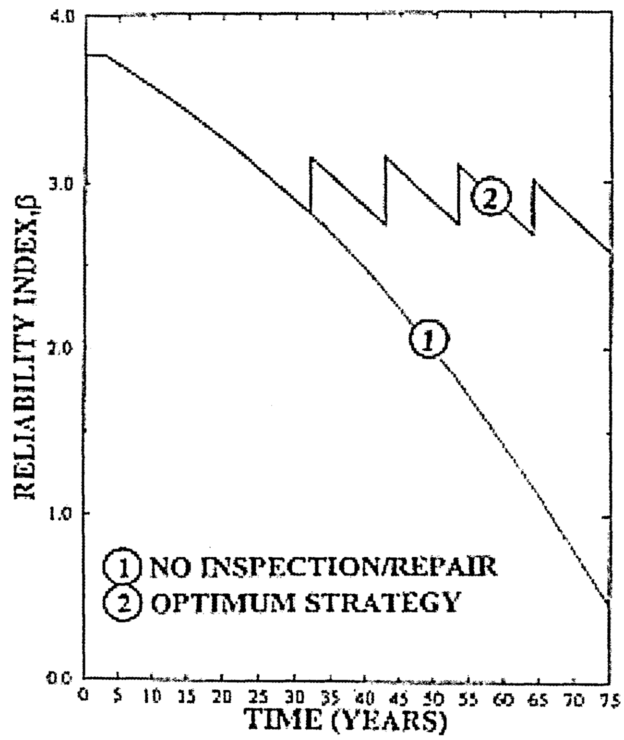



**Figure 2.1 Typical optimum repair strategy**[13]

## 2.4 Deterioration of reinforced concrete in bridges

The most common form of deterioration affecting concrete in bridges is reinforcement corrosion[14,15]. Although concrete can contain moisture long after curing, and the micro voids within the concrete matrix can also contain oxygen, reinforcement in concrete does not usually rust. This is due to the inert barrier formed around it by free alkalis (usually calcium hydroxide)[5,16]. However, if this barrier is broken, and if sufficient oxygen and water are present, the steel reinforcement will provide anodic and cathodic sites, and the moist concrete provides the electrolyte necessary to initiate the electrochemical reaction whose end product is ferric oxide (rust). Corrosion causes further problems for the symbiotic concrete/steel element[17]:

- Reduction in steel area
- Corrosion products occupy a larger volume than original steel. This exerts expansive pressure on the concrete causing cracking, spalling and delamination.
- The bond between steel and concrete deteriorates and the composite action of steel and concrete is lost.

Any aggressive agent diffusing to the steel reinforcement is aided in its journey by insufficient cover or areas of poor compaction.

Other forms of deterioration can also blight the performance of concrete in bridges.

There are a number of ways in which concrete deterioration can be categorised. One effective way is to categorise three types of defects: early age, medium to long term, sudden defects[18]. Early age defects are attributed almost solely to moisture movement during curing, but can also be the fault of poor mix design or faulty workmanship. Medium to long term defects are often caused by environmental aggression and long term concrete 'disease'. Sudden defects occur through fire, physical impact, seismic event, overload of the structure or settlement of foundations etc.

### 2.4.1 Reinforcement Corrosion

Invariably, the cause of corrosion is an aggressor which breaks down the passive layer formed around reinforcing steel, allowing the electrochemical process to take place. The known aggressors that cause corrosion in reinforced concrete are chlorides and carbon dioxide which diffuses from the air to neutralise the alkalinity of concrete.

The common cause of chlorides in the non-marine environment is from de-icing salts applied to roads and bridge decks during periods where there is a risk of surface-water freezing[19]. The transit of salt laden water from pavement to concrete structure is shown in Figure 2.2.

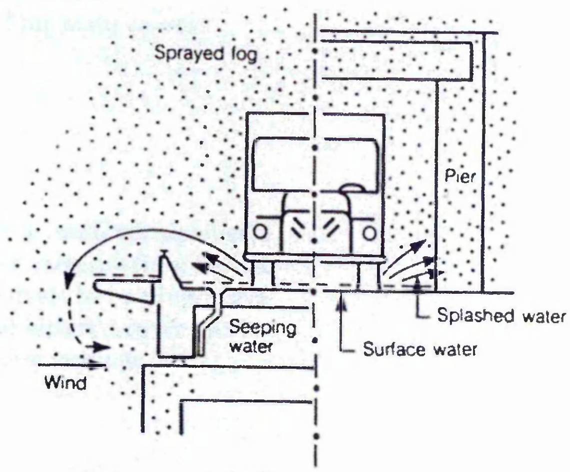



**Figure 2.2 The dispersion of vehicle salt spray[19]**

These salts make their way onto bridge elements either via leaky drains and joints on the bridge or in splash water sprayed at the bridge from the wheels of passing vehicles. With the help of surface moisture, chloride salts in solution can permeate into concrete[20]. This action is accelerated in concrete that is already damaged through some other mechanism, such as the effects of freezing and thawing cycles (see section 2.4.4). Chloride ions cause depassivation of the reinforcing steel even in alkaline environments.

The speed at which chloride penetration approaches the reinforcing steel is dependent upon[21]:

- The amount of chlorides coming into contact with the concrete
- The permeability of the concrete
- The amount of moisture present

When the concentration of chloride ions exceeds 1% of the mass of cement in concrete, the corrosion of reinforcement is inevitable[5]. Once corrosion begins, the expansive corrosion products cause tensile stresses in concrete which lead to cracking and delaminations.

Carbonation in concrete is a reaction between natural or industrially produced carbon dioxide in the air and calcium hydroxide dissolved in the pore water contained in the concrete microstructure[22]. From the time concrete is cast, its surface zone is subjected to attack from carbon dioxide continuously[5]. This gradually degrades the alkalinity of the concrete which passivates reinforcement against corrosion. Therefore, carbonation is only likely to be a problem in older bridges, areas of concrete with low cover to the reinforcement, or poor quality porous concrete[8]. When the carbonation front reaches the steel reinforcement, its passivation is dissipated and, in the presence of moisture and oxygen, corrosion can begin.

*Wallbank's* survey of 200 bridges in the UK found that 90% of bridges had a carbonation depth of 5mm or less[4]. As such, in practice, carbonation is not as common a problem in the UK environment as chloride ingress for the corrosion of reinforced concrete. However, carbonation is sensitive to temperature and relative humidity of the environment and is accelerated in warm, dry climates.

The rate of penetration of carbonation through concrete can be approximately represented by[23]:

$$x = (2Dt)^{0.5} \qquad \text{Eq. 2-1}$$

Where x = depth of penetration

D = diffusion coefficient of $CO_2$ in concrete

t = time in years

The depth of a carbonation front into concrete can be measured by breaking out a small section of concrete and spraying the exposed sub-surface concrete with a phenolphthalein spray which reacts with carbonated concrete. If the age of the structure is known, and the depth of carbonation is determined, then the diffusion coefficient can be calculated. This coefficient can then be used to determine the age of the structure when the carbonation front reaches the reinforcing steel.

The rate of chloride penetration into concrete as a function of depth can be represented by Fick's Law of diffusion[23a] shown in equation 2-2:

$$C_{(x,t)} = C_0\left[1 - erf\left(\frac{x}{2\sqrt{D_c t}}\right)\right] \qquad \text{Eq. 2-2}$$

Where $C_{(x,t)}$ is the chloride ion concentration at a distance x (cm) from the concrete surface after time t (s)

$C_0$ is the equilibrium chloride concentration on the concrete surface

$D_c$ is the chloride diffusion coefficient in $cm^2/s$

erf is the error function

Determining the chloride ion concentration at the steel reinforcement can give an indication of the likelihood that corrosion of the reinforcement is occurring.

### 2.4.2 Sulphate attack

Atmospheric sulphur dioxide can affect concrete in a similar way as carbon dioxide[19]. It can also act in conjunction with carbon dioxide to increase the rate of loss of alkalinity in concrete. Alternatively, sulphates contained in ground water can attack concrete in a similar way to chlorides.[24]

In addition to the depassivation which airborne or soluble sulphates can cause to reinforcing steel, sulphates can react chemically with hydrated lime in the cement paste producing solid products (gypsum and ettringite) with greater volume than the products entering the reaction. As a result surface scaling can occur followed by mass disruption of the concrete. Hence sulphates can cause surface defects in concrete that not only render the concrete aesthetically unacceptable, but also aggravate the effects of carbon dioxide, chlorides, further sulphate attack and freeze-thaw cycles[22]. Sulphate attack is uncommon in the UK.

Sulphates can cause deterioration of the concrete matrix itself, but concrete deterioration is more commonly caused by other aggressors which lead to corrosion of the reinforcing steel.

Sulphate attack can occur in a severe form known as Thaumasite. Thaumasite has the effect of very seriously degrading the concrete matrix. To combat the risk of this aggressor, careful selection of concrete mix constituents is necessary, particularly for substructures in ground contaminated by sulphates.

### 2.4.3 Alkali aggregate reaction

Alkali-silica reaction, alkali-carbonate reaction and more generally alkali-aggregate reaction[19] are rare combinations of reactive aggregate, high alkali cement and moisture which can cause adverse chemical reactions in the concrete matrix which produce an expansive gel in structures[5]. The gel, when exposed to moisture, expands generating tensile forces which cause cracking in a distinctive '*manx*' pattern[21] as shown in Figure 2.3.

**Figure 2.3 Typical 'manx' cracking, in the early stages of AAR**

AAR is often referred to as 'concrete cancer' due to its incurability, although corrective measures can be taken to arrest its development[25]. Despite the alarming manifestation of the defect, there is evidence that the effects of the "disease" are less serious than appearances suggest[26,25]. This is due to the fact that cracks permeate only to a limited depth.

### 2.4.4 Freeze thaw attack

The effects of cyclical freezing and thawing of concrete are alternatively described as 'frost attack' or more generally 'weathering' (although this term also includes damage from wetting and drying, and heating and cooling cycles). Frost attack is a common cause of surface scaling and spalling in concrete[19]. Water is absorbed into concrete through capillary action, as this water freezes its volume increases by approximately 9%[27,19]. The expanding water causes hydraulic pressures in the pores of concrete and a number of cycles can be sufficient to cause surface concrete to scale away from its parent mass.

The effect of freeze-thaw cycles on a concrete surface can exacerbate chloride ingress and carbonation by allowing easier access for chloride ions in solution and airborne carbon dioxide to the reinforcement.

### 2.4.5 Other forms of deterioration

Often vehicles will collide with reinforced concrete on highways. Typically the wing mirror of a large vehicle may clip concrete at high speed, causing a piece of the concrete to break away, or an accident may cause more serious damage. If reinforcement becomes exposed, the result can be depassivation of the steel leading to corrosion and worsening defects.

Aggressive industrial substances such as Aluminium Chloride or Calcium Bisulphate can sometimes come into contact with concrete, often through leakages but also via accidents. These can attack concrete surfaces causing rapid disintegration.

### 2.4.6 Non-structural cracking in concrete

Non-structural cracking in concrete is often a precursor to delamination and spalling caused by corrosion. Many of the deterioration processes already described in section 2.4.1 initially manifest themselves as cracking over the areas that they affect. Alkali-Aggregate reaction has a distinctive crack pattern, and corrosion from any source will initially cause cracking as the tensile forces created by the corrosion products exceed local tensile strength of concrete. Some crack types affect newly built structures (e.g. shrinkage cracking), others are the results of defects which emerge in the longer term.

#### 2.4.6.1 Crazing

Crazing is the cracking of the surface layer of concrete into small irregularly shaped contiguous areas[28]. Crazing is not structurally significant, and apart from accelerating other concrete defects such as carbonation, it is only a cosmetic defect. Crazing is caused when the surface concrete upon curing is different to the underlying concrete (e.g. it is subjected to excessive moisture). This can occur through over-trowelling or a number of other effects. Crazing usually occurs shortly after casting but may occur at later ages if the climatic conditions are severe enough.

#### 2.4.6.2 Plastic cracking

This type of cracking mainly occurs on exposed horizontal surfaces of concrete. It usually occurs through differential shrinkage of surface and underlying concrete[29]. Plastic settlement cracking occurs when the usual continued consolidation of concrete after vibration is restrained by reinforcing bars.

#### 2.4.6.3 Drying shrinkage

Drying shrinkage is the reduction in the volume of concrete caused by the chemical and physical loss of water during the hardening process[28]. In newly cast concrete, this shrinkage is restrained by the sub-base, by reinforcement, or by the concrete element which the fresh concrete has been cast against. This restraint to shrinkage causes tensile stresses to develop, which, if they exceed the tensile strength of the concrete can cause cracking. Drying shrinkage occurs during the hardening phase of a concrete and, therefore, can be expected to occur several weeks or months after casting[19].

### 2.4.7 Other defects

Concrete can exhibit a number of defects after casting. These are invariably only of cosmetic importance[30].

Honeycombing

Honeycomb surfaces are caused by the use of dry mix concrete that is not properly consolidated. The lack of consolidation means that mortar does not effectively fill the voids in between the aggregate particles.

Sandstreaking

Sandstreaking is a cosmetic defect caused by the use of wet concrete mixes which bleed excessively.

Blowholes

Blowholes are small air pockets formed during placement and consolidation. They are thought to be caused by excessive amounts of oil placed on the formwork. Counter-intuitively, the more air-entrainment in the concrete, the less likely blow holes are to occur.

## 2.5 Establishing the causes of concrete deterioration

Once the suspected causes of concrete defects have been established, a course of testing procedures is decided to confirm the original diagnosis and to provide details of the extent of the problem. The majority of defects that signify concrete has become less reliable are exhibited as either spalling or cracking. Other defects, such as patches of honeycombing, require no testing to establish their causes because the cause is self evident. An engineer can usually make an intelligent estimation of the cause of any spalling or cracking. However, if such defects are of sufficient magnitude to warrant concern, a testing regime will be required to confirm the engineer's original diagnosis and establish the extent of deterioration.

### 2.5.1 Ingress of Chloride Ions

Testing to confirm the presence of chlorides is undertaken if a defect shows signs that its cause could be the ingress of chloride salts, and if the defect is of sufficient magnitude. Concrete dust is extracted at regular intervals on the affected element by drilling into the substrate and collecting the resulting dust in a small plastic receptacle. These samples, taken at various spacings and depths, can be analysed chemically in the laboratory by analytical means[5]. Often, an amount of chloride has been cast intentionally into the concrete and this must be allowed for when determining the amount of chlorides which have entered from the surface. This is done by taking dust samples from an area of the suspect concrete where chloride ingress is not thought to have taken place[21]. The presence of chlorides in concrete, even at depths equal to or greater than the reinforcing steel cover, does not prove that electrochemical corrosion is taking place. Corrosion will only occur with both moisture and oxygen present. Therefore, whenever there is suspicion that corrosion is taking place in reinforced concrete, an electrochemical survey is conducted (e.g. by half-cell potential survey) for the presence of corrosion activity. When the passivity of steel is destroyed by carbonation, chloride ingress or any other agent, electrochemical cells develop. When this happens, an electro-potential difference exists between the anodic and cathodic areas of the steel[21]. This difference is measured using an electrode probe passing over the concrete surface, the probe is attached to the reinforcement (a small removal of concrete on an affected member is necessary) and the readings are taken from a high impedance voltmeter. During the test, the concrete must be of uniform moisture content[5]. The results are plotted as a grid over a drawing of the affected element and contours are mapped. The probability of corrosion taking place (when measured using a standard copper/copper sulphate half-cell) is high if the potential ranges between –0.2 and –0.4 volts. The concrete acts as the electrolyte for the electrochemical reaction, therefore, its resistivity can indicate how effectively it will perform as an electrolyte and support the formation of corrosion cells. Some repair practitioners also recommend the use of a resistance meter to detect the passage of current between metal electrodes cast in the concrete in order to determine its resistivity.

### 2.5.2 Ingress of Carbon dioxide

If the nature of a defect suggests its cause could be the approach of a carbonation front caused by carbon dioxide diffusion, then a test to confirm the presence of a neutral-alkalinity front would be conducted. This simple test requires a fresh area of concrete to be broken out on site; this area is sprayed with alcoholic vinyl phenolphthalein. If the concrete has retained its alkalinity, the spray turns pink. If the alkalinity has been neutralised by $CO_2$ action, the spray remains colourless[31]. The interface of the pink and colourless film of spray represents the depth of carbonation at the test location. The seriousness of carbonation is generally determined by the depth of carbonated concrete relative to the depth of cover to the reinforcement. If the reinforcing steel is sufficiently far from the approaching carbonation front then corrosion will not occur in the short term. Remedial measures can be taken if the approach of carbonation to the steel is deemed as a long term threat, see section 2.6.4. Therefore, the depth of cover to the reinforcement in an element affected by carbonation needs to be determined. This is established using a Covermeter test. Covermeter tests detect the distance between the surface of the concrete and the reinforcing steel by generating a magnetic field and measuring the effect of reinforcing steel below the surface on the field. The device used is known as a Covermeter or a

Pachometer[22,23]. The device is affected by reinforcement congestion, but generally produces accurate results. The sensitivity of the test improves if the meter is calibrated using a known diameter of the reinforcing bar.

### 2.5.3 Alkali Aggregate reaction

Surveys have shown that bridges with the constituent materials necessary for an alkali aggregate reaction (AAR) take at least 10 years to exhibit the symptoms of the disease[32] and as many as 20 to 30 years for the reaction to fully develop. The technique used to detect AAR is petrographic analysis[33] (or Petrography), which involves the examination of polished plates of the material. The polished concrete samples taken from suspected AAR sites are examined for networks of micro-cracking through the concrete matrix, and the presence of gel; the tell-tale signs of AAR[34].

The effect of alkali-aggregate reaction on the likelihood of corrosion to occur is difficult to determine[35]. A high pH is needed for AAR to occur but a high pH environment protects reinforcing steel from corrosion. Concrete cracking caused by AAR should accelerate the carbonation process by allowing faster access of carbon dioxide towards the reinforcing steel but the high moisture levels associated with AAR also slow the carbonation process. Additionally, the cracks caused by AAR often become filled with a gel, preventing the access of carbon dioxide and chloride solution into the concrete.

### 2.5.4 Other causes of concrete deterioration

Some defects may be of such a nature as to leave the engineer or expert unsure of their cause. In these cases multiple tests are conducted (if the severity and extent of the defect warrant such action) in order to establish the cause. For example, an engineer who is unsure if a concrete spall containing exposed reinforcement was caused by chloride or carbonation induced corrosion might recommend testing to ascertain the presence of both aggressors. Some defects, such as freeze-thaw cracking, are the effects of undetectable aggression from the external environment. They can easily be confused with chloride ingress defects or vice-versa. It may be necessary to test for other defect causes in order to be able to diagnose the cause of a defect by a process of elimination.

## *2.6 Repairing defective concrete*

### 2.6.1 Dealing with corrosion

Generally, for any defect in need of repair, all cracked, spalled and delaminated concrete is cut away to a depth just exceeding the steel reinforcement. If there is reason to suspect that corrosion is taking place in an area that displays no visible sign of such (for example, the results of the half-cell potential survey) then it may be necessary to break out concrete in those additional areas. Certainly, the removal of concrete should continue along the reinforcement until the signs of corrosion are no longer evident. Carbonated concrete in contact with reinforcement must be removed, as this will not provide the steel reinforcement with the layer of passive alkalinity necessary to impede corrosion. Reinforcement is cleaned using grit-blasting or high pressure water jetting and these techniques are also used to prepare the surface of the substrate (parent) concrete, ready for application of a repair material. Chloride contaminated concrete must also be removed wherever possible[20].

For seriously affected concrete elements where removal of concrete becomes economically prohibitive there are a number of repair techniques that can be employed that minimise concrete repair. These are discussed in section 2.6.4. However, in cases where spalling or cracking have occurred, or where the extent of corrosion is such that the structural capacity of an element may have been affected, concrete will always require removal or replacement (or both the structural capacity will have to be reinstated and corrosion arrested by other means).

### 2.6.2 Repairing spalls

Spalling is repaired by the application of a suitable repair material using one of a variety of methods.

- Patch repair – This type of repair involves the application of hand applied mortar. It is suitable for small repairs.
- Sprayed repair – This technique is the most widely used concrete repair particularly since it does not require shuttering. It also provides a good bond between substrate and repair material.
- Flow repairs – These repairs involve the use of shuttering to form a cast into which the repair material can be poured.

The selection of a suitable repair material is both important and complex. For each method of application the factors involved in the selection of a repair material are discussed in Chapter 3.

Typically, these methods (remove and replace) are used to repair the majority of defects associated with reinforced concrete highway bridges. These include spalling and cracking caused by carbonation, chloride ingress, attack from sulphates, impact and freeze-thaw damage.

### 2.6.3 Repairing AAR affected concrete.

In AAR affected concrete, it is important to establish (from the petrographic analysis) the amount of reactive material in the concrete matrix. This information will determine whether the total effects from the reaction have been exhibited or whether the current condition of the concrete will worsen[32]. Generally, if the alkali-aggregate reaction has run its course and the structural capacity of the element has not been impaired (a structural survey may be required to determine this), then the surface cracking caused by the AAR can be sealed to restore the aesthetic appearance of the surface and prevent the access of aggressive agents below the concrete surface. Even if the AAR has caused a degree of structural instability, the cracking can be injection grouted by the technique where resin is forced under pressure to permeate the crack pathways in the substrate concrete. If laboratory testing shows that the alkali-aggregate reaction will continue in the concrete, then this reaction must be arrested by reducing the internal humidity of the concrete. This can be achieved by sealing surface cracks (or impregnation) and the application of a water repellent surface coating[25].

### 2.6.4 Unobtrusive alternatives to repair

Occasionally the extent of concrete deterioration is such that the removal and replacement of the affected concrete becomes tantamount to replacement of the element. In these cases, or in cases where corrosion is expected to continue after repairs have been completed (e.g. in chloride infested concrete) a number of alternative approaches are available[36].

#### 2.6.4.1 Cathodic protection

Reinforcing steel in concrete seriously affected by chlorides (and occasionally carbonation) can be protected by a cathodic protection system. When a corrosion circuit has been formed (after the passivity of the steel has been compromised) the steel acts as both cathode and anode in an electro-chemical circuit. A cathodic protection system maintains the steel as a cathode in an electrical circuit driven by an impressed current[37]. Anodes are installed on the concrete surface and are electrically connected to the steel reinforcement, this process reverses the electrical current flow which causes corrosion[21]. Cathodic protection systems require constant monitoring and adjustment. This process is often automated using expensive equipment. As such, careful economic comparisons need to be conducted before embarking on such a scheme.

#### 2.6.4.2 Re-alkalisation

An electrochemical technique is available which restores the protective alkalinity around reinforcing steel without the need to remove the carbonated concrete. It introduces an alkaline solution through a process of electro-osmosis. Another process stimulated by equipment at the concrete surface is electrolysis which results in the generation of ions which re-passivate the steel surface. The re-alkalisation process must be accompanied by the application of a surface coating to the concrete to prevent a reoccurrence of the carbonation process[37].

#### 2.6.4.3 De-salination

De-salination is a technique which extracts chlorides from concrete. It utilises similar techniques to those of electro-chemical re-alkalisation to remove negatively charged chloride ions from the concrete. The electro-chemical reaction results in the migration of the chloride ions to the surface mounted anode. In tests salt concentrations of 6 to 12 $kg/m^3$ were removed over a period of 100 hours[24]. The process of desalination is known to take between 3 and 8 weeks to complete and may result in repassivation of the reinforcing steel by the generation of hydroxyl ions.

### 2.6.5 Concrete Protection

#### 2.6.5.1 Corrosion countermeasures

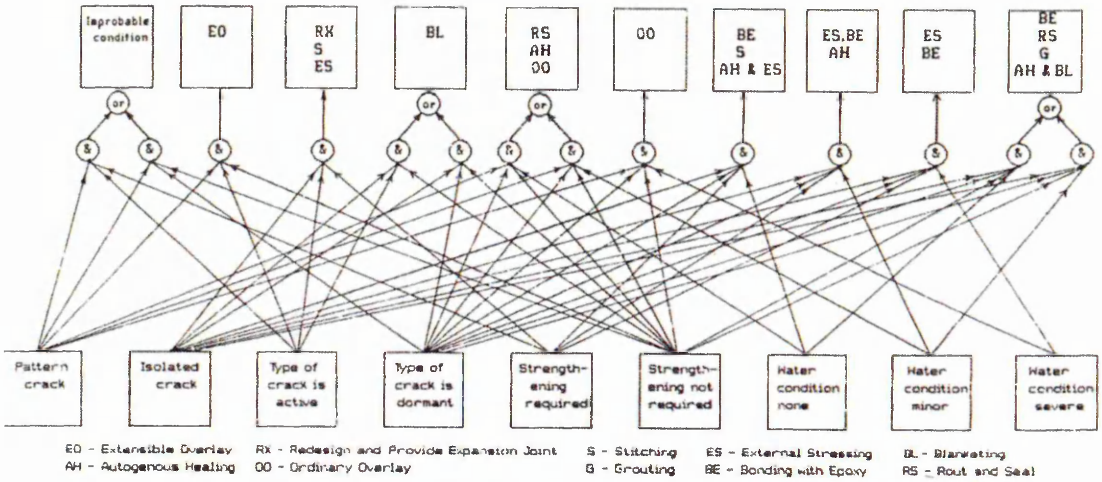
Concrete surfaces can be coated or sealed to prevent the access of aggressors into the substrate. These measures are often taken after repairing to ensure defects do not re-occur through similar mechanisms, or to prevent other likely deterioration processes. Coatings and sealers generally fall into the following groups:[19]

- Water-repellent surface impregnants
- Surface hardeners and pore blockers
- Cement-modified polymer coatings
- Elastometric polymer membranes

The purposes of these coatings are generally to reduce or stop the ingress of oxygen, carbon dioxide, chlorides, and water. Oxygen and water are necessary to support reinforcement corrosion and alkali-aggregate reaction. Carbon dioxide causes carbonation which destroys the passive alkaline environment of reinforcing steel[5].

### 2.6.6 Dealing with cracking

Pattern cracking is a term used to define areas of concrete affected by a large number of small cracks. It is generally expected to be repairable in a similar manner to spalled or delaminated concrete. Larger cracks, structural or otherwise can be repaired using established methods. The root cause of non-structural cracking is often the same as the cause of eventual spalling. Additionally, moisture and thermal effects during curing can cause cracking, as can AAR. Repair strategies for these were described in section 2.6.3. Larger cracks will often be caused by structural effects such as overload or differential settlement. Discovering the cause of a crack is essential before repair methods can be determined. In addition to the cause, the status of the crack is also important. Status is defined by three categories: category 1, the crack is actively widening, category 2, the crack is active but not widening (i.e. opening and closing), category 3, the crack is dormant. If a crack is of sufficient size and nature as to alarm an engineer a crack movement indicator is employed to establish its status[29,38]. The results of such a survey, and other determining conditions can be used to match the crack defect to a suitable repair method, as recommended by Kalyanasundaram et al[39] and represented in Figure 2.4




Sample Knowledge Net for Suggestion of Repair Method for "Cracking In Concrete"

**Figure 2.4 Repair methods for cracks**[39]

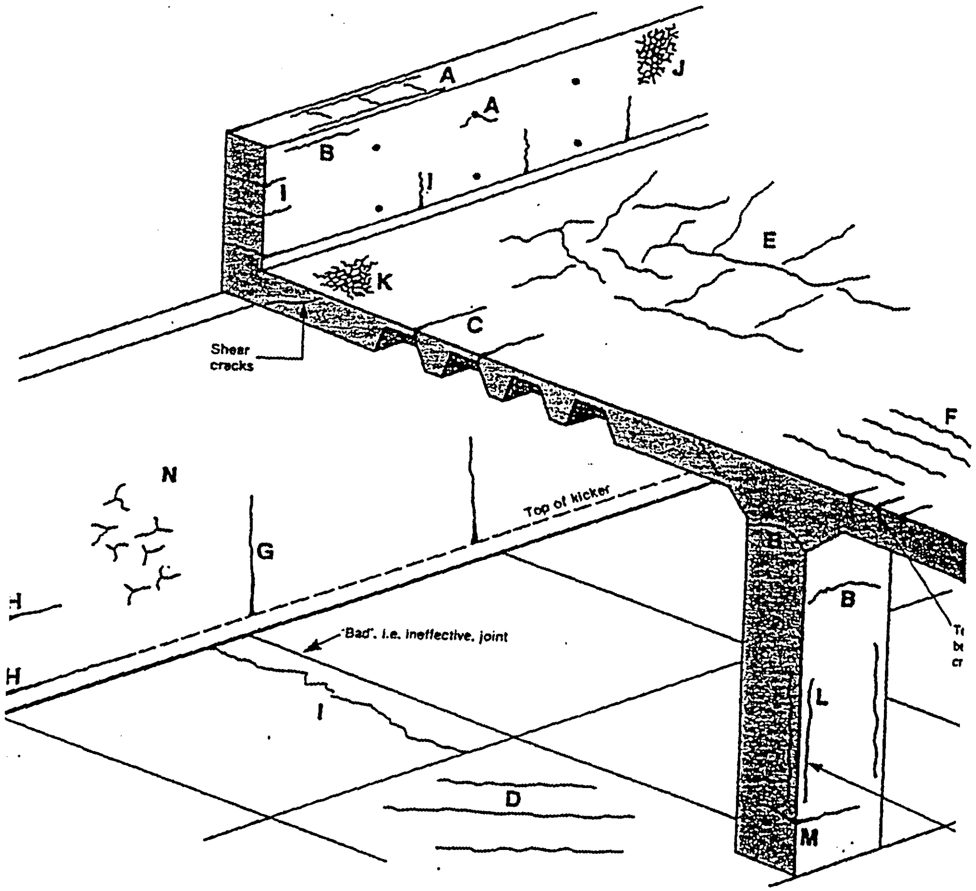
A diagram of common crack types and an accompanying table giving their details are given in Figure 2.5. There are various techniques of crack repair which can be employed depending on what needs to be achieved by the repair. Crack repairs provide some of the following functions[29]:

- Restore or increase strength or stiffness
- Improve functional performance
- Provide water-tightness
- Improve appearance
- Improve durability
- Prevent access of corrosion inducing material to reinforcement

Depending on the performance requirement of the repair, one of the following repair methods is usually considered:

- Epoxy injection – The technique involves drilling holes at regular intervals along a crack and filling the void created with epoxy.
- Routing and Sealing – this is the most common method of crack repair. The procedure involves cutting out a groove along the length of a crack (routing) and then sealing the groove, this prevents ingress of moisture. A bond breaker is usually added to the unsealed routed groove.
- Stitching – Stitching involves the drilling of holes either side of a crack and placing '*stitching dogs*' or ties holding the two sides of the crack together.
- Flexible Sealing – this involves active cracks being routed out, cleaned, and filled with flexible sealant.
- Grouting – Wide cracks in thick walls can be repaired by filling with cement grout.
- Polymer Impregnation – Cracked concrete surfaces can be dried and flooded with a monomer which is polymerised by heating.
- Overlays – These can restore structural integrity and prevent the access of aggressors into the concrete.
- Autogenous healing – this is a natural process of crack repair (self heal) in concrete. It has a practical application for closing narrow dormant cracks in a moist environment[29].
- Movement joints – Live cracks are often candidates for conversion into movement joints. In this procedure, a recess is cut along the line of the crack and filled with a flexible material[40].

Higgins[40] details the performance characteristics of various crack repair methods. An expert would take into account all the necessary factors before selecting the most suitable repair method.




| Type of Cracking | Letter (See Figure 8.2) | Subdivision | Most Common Location | Primary Cause (Excluding Restraint) | Secondary Causes/ Factors | See Section |
|---|---|---|---|---|---|---|
| Plastic settlement | A | Over reinforcement | Deep sections | Excess bleeding | Rapid early drying conditions | 2.2.1 |
| | B | Arching | Top of columns | | | |
| | C | Change of depth | Trough and waffle slabs | | | |
| Plastic shrinkage | D | Diagonal | Roads and slabs | Rapid early drying | Low rate of bleeding | 2.2.2 |
| | E | Random | Reinforced concrete slabs | | | |
| | F | Over reinforcement | Reinforced concrete slabs | Ditto plus steel near surface | | |
| Early thermal contraction | G | External restraint | Thick walls | Excess heat generation | Rapid cooling | 2.2.3 |
| | H | Internal restraint | Thick slabs | Excess temperature gradients | | |
| Long-term drying shrinkage | I | | Thin slabs (and walls) | Inefficient joints | Excess shrinkage Inefficient curing | 2.2.4 |
| Crazing | J | Against formwork | 'Fair faced' concrete | Impermeable formwork | Rich mixes | 2.2.5 |
| | K | Floated concrete | Slabs | Over-trowelling | Poor curing | |
| Corrosion of reinforcement | L | | Areas subject to leakage or spray from road run-off | Chloride ingress | | 2.2.8 2.5.2 |
| | M | | (Sheltered areas) | Carbonation | Porous concrete. Low cover | 2.2.8 2.5.1 |
| Alkali-silica reaction | N | | (Damp locations) | Reactive aggregate plus high alkali cement | | 2.2.9 |

**Figure 2.5 Diagram of typical crack types and explanatory table[40].**

## 2.7 *Expert Systems for concrete bridge repair*

### 2.7.1 Review of existing developments

Concrete bridges are required to maintain their serviceability over long periods of time. The reinforced concrete which constitutes these bridges is susceptible to attack from aggressive environmental agents, attack which if left unchecked can severely reduce the service life of a bridge.

Managing the condition of bridge networks involves teams of dedicated engineers inspecting and monitoring the performance of all elements of the bridges under their jurisdiction. This also involves decision making on when repair and remediation is necessary. The knowledge used in this decision making process is not well documented. There are no comprehensive formalised standards for concrete repair material selection[41], and no standard documents to aid engineers in the diagnosis of defect causes.

The aim of this thesis is to construct an expert system which can dispense the best practice instruction tailored to fit any concrete repair situation. This has been attempted to varying degrees by others in the past, with limited degrees of ambition and success. Anumba and Bowron[18] suggest in their proposed system that accurate diagnosis is a '*sine qua non*' in the repair of concrete structures. They suggest an expert system could provide a more objective approach to the choice of concrete repair materials.

The American system, HYWCON[42], developed in the SHRP programme demonstrates knowledge engineering in the bridge repair domain at a very basic level, its structure is shown in Figure 2.6. A sub-system of the HYWCON program diagnoses and recommends repair strategies for three key concrete defects defined by it: cracking, spalling, and disintegration.

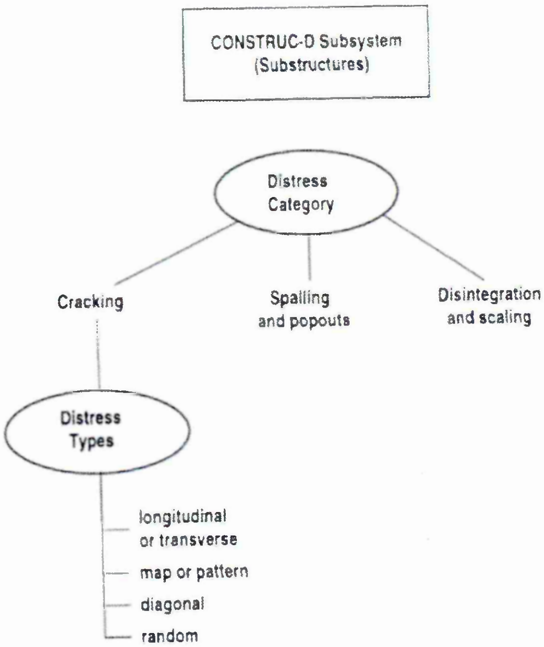



**Figure 2.6 Structure of HYWCON expert system[42]**

The system is structured into three sub-systems, each one diagnosing causes for different types of surface defects.

Figure 2.7 to Figure 2.11 give an example of the process a user would follow from beginning to end. This program functions only in the Microsoft Windows 3.1 graphical user interface environment; an indication of its age!

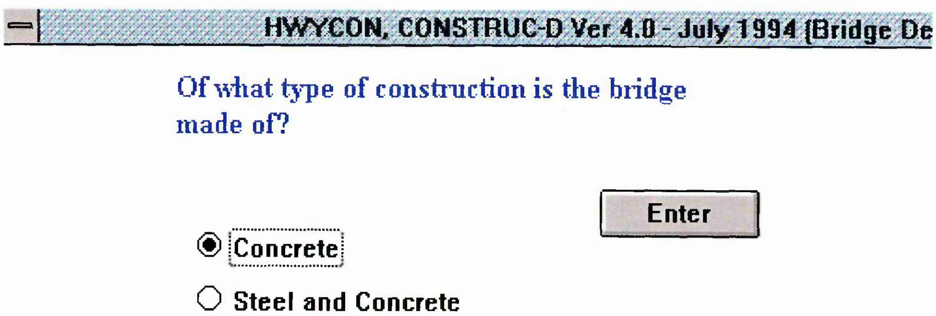



**Figure 2.7 American HWYCON system step 1.**

The program is used to diagnose the cause of defects in concrete bridge decks.

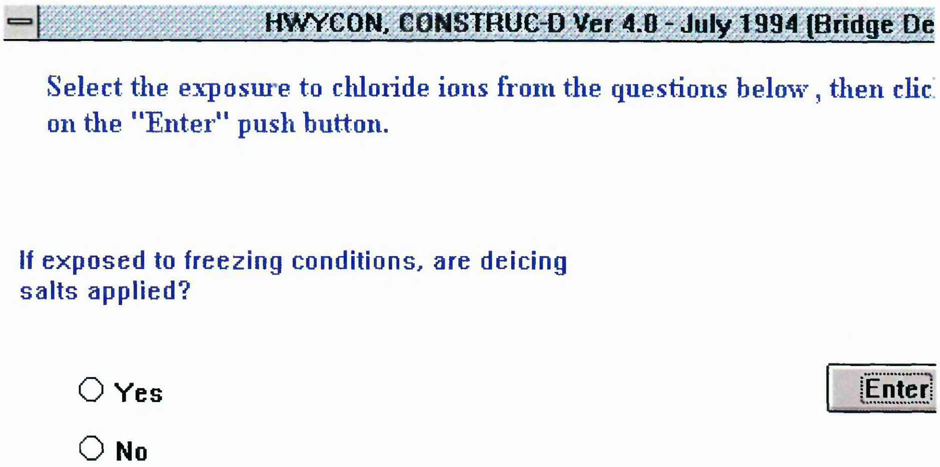



**Figure 2.8 American HWYCON system step 2.**

This system next requires the user to enter the bridge's likelihood of exposure to chloride ions.

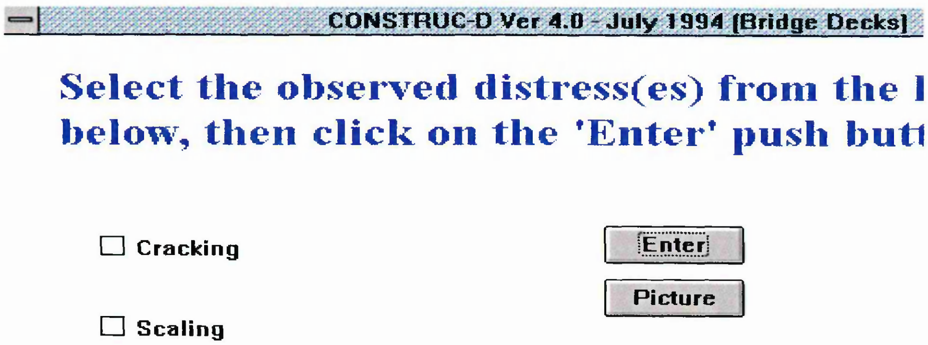



**Figure 2.9 American HWYCON system step 3.**

At this stage, the user of this system is requested to inform the knowledge base of the effects of the defect. The user is provided with graphical examples of the likely effects (Figure 2.10).

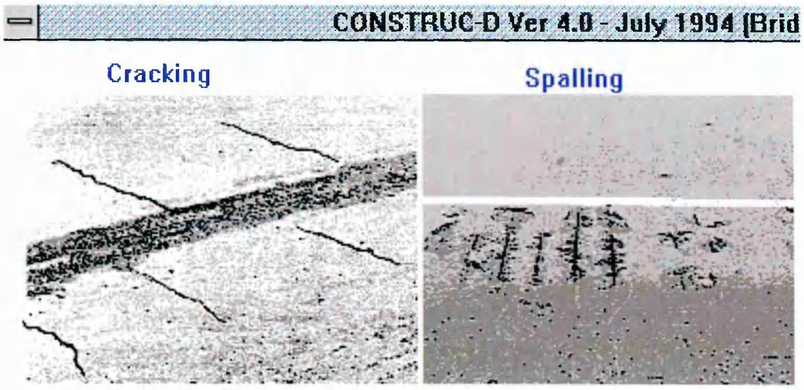



**Figure 2.10 American HWYCON system step 4.**

Finally, the system uses the acquired information to generate a diagnosis for the encountered defect (Figure 2.11).

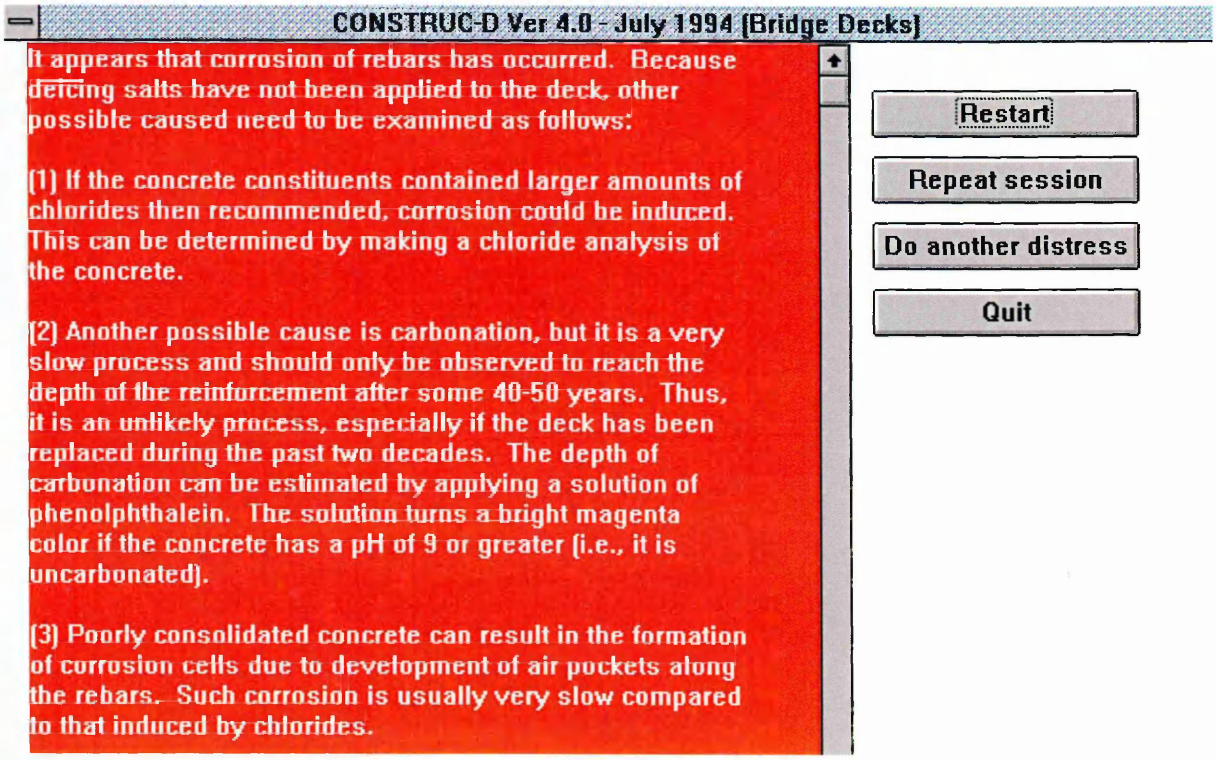



**Figure 2.11 American HWYCON system step 5.**

Immediately clear from this example is the narrow scale of the knowledge base. Complex decision making is avoided as the user is limited in the options available at each stage. The finishing inference is, in all cases, a very general piece of repair advice (such as 'break out the affected concrete and repair'). The program is the only purchasable expert system for reinforced concrete repair discovered in this review. Such a basic system is of limited use to bridge engineers as it delivers only simple advice and lacks intelligence. Here a distinction is made between an 'expert system' and an 'intelligent expert system'. For example, in such a system as HWYCON, if a user encountered a very small defect in some reinforced concrete, the advice generated by the program would be identical to the advice generated for a very large defect with the same symptoms (e.g. such common generated advice as; cracking through exposure to chlorides). Hence, such a system lacks the intelligence to account for severity and extent of defect when making a decision. Severity and extent are identified as crucial factors in the production of an intelligent expert system for concrete repair[18]. Additionally, HYWCON evaluates distresses individually and there is no provision for advice in situations that involve multiple causes of distresses that occur simultaneously at one location[42]. It is recognised that an intelligent expert system should be able to examine defects collectively[43]. A system such as HWYCON which generates repair advice on a defect by defect basis and fails to advise the user when an element is severely affected by multiple defects lacks basic intelligence.

The use of graphics in the system, although limited, is a great advantage over textual descriptions. If expanded to include for example, different crack types and their causes, the extent of the system could be improved.

The MENTE-KUN prototype expert system[44] once again uses a knowledge base to question a user about the nature of concrete defects. It concentrates on the nature of a defect, (i.e. spalling, cracking, abrasion) and does not account for severity and extent. In the program, the user is expected to judge if the severity and extent of the problem are of such magnitudes as to warrant action.

REPCON is a text-based prototype expert system for concrete repair[39]. It is essential the user communicates concrete defects unambiguously to the program, otherwise the communication gap increases, leading to wrong diagnosis[39]. Similarly, this program generates generic advice for repair of reinforced concrete defects. It does not attempt to judge the severity and extent of deterioration before giving its advice.

CODBA[43] (concrete bridge deterioration assessment) is a prototype system developed to diagnose deterioration in concrete bridges. The program attempts to facilitate the visual assessment of concrete bridges in order to recommend in-depth testing procedures.

Generally, the majority of expert systems for concrete repair are text based prototypes[45,46]. Additionally, these existing systems have concentrated on diagnosing the cause of singular defects and not the effect of multiple defects on a single element. In addition, existing systems have generally failed to take account of the extent and severity of defects when diagnosing their causes and effects.

An intelligent expert system should be able to assess individual defects and the effect of multiple defects, including their causes. It should also be able to judge the effect of the size and severity of defects on an element and, thereby, assess the condition of the element. After the assessment of an element, an intelligent system should be able to recommend a suitable regime of test procedures to confirm the initial diagnosis. Furthermore, once testing has been completed, optimal repair recommendation should be made. An intelligent system will be flexible and have the ability to cope with the fact that for any given situation, often more than one cause may have led to the defect, or it may be difficult to identify the cause and several causes may be suspected. A number of repair options may be possible as a result.

### 2.7.2 Architecture of expert system for reinforced concrete repair

Anumba & Bowron[18] suggest the architecture of an expert system for concrete repair should include two key components, an intelligent diagnostic component, and a repair specification component.(Figure 2.12)

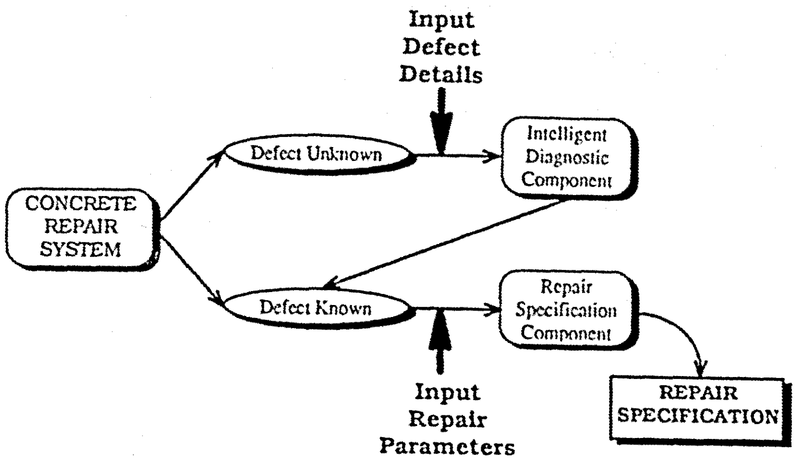



**Figure 2.12 Concrete repair expert system: basic architecture[18]**

#### 2.7.2.1 Diagnostic component

The purpose of the diagnostic component of an expert system for reinforced concrete repair is to firmly establish the cause of a defect. Defect effects are exhibited in three key ways, cracking, spalling and disintegration[39]. A diagnostic system should assess the severity and extent of such effects, and use simulated expert knowledge to derive a suspected cause. An intelligent system should also be able to determine if the extent of the defects is of sufficient magnitude to be a cause for concern. Similarly, if a system did suspect defects were of a sufficient magnitude to cause concern and as a result recommended some tests be carried out, then based on the results of the testing the software should be able to recommend if the defect is significant enough to require repair.

In a bridge assessment procedure it is crucial to integrate all defects discovered in an element in order to form a combined subjective rating of the element with accuracy[43].

The first stage in the development is to collect the relevant knowledge in the field. This knowledge is expressed in a format which maps easily into an expert system (knowledge net). One such system[47], makes no account for severity and extent, although it generates a confidence factor which describes the confidence the system has in its diagnosis being correct. These confidence factors appear to be static, i.e. for a certain set of inputs the confidence factor in a decision can only be generated as a pre-specified figure of 40%, 60% or 100% (for example). Therefore in this example (Figure 2.13) the issue of confidence factors is similar to the use of natural language qualifiers (i.e. low, medium, high). The approach of adding an indication of the certainty of a decision makes a system more intelligent.

Some attempts or outlines for the creation of expert systems for concrete repair have suggested that the difficulty in diagnosing a unique cause to a defect limits the development of expert systems in this field[47]. This assumes that each defect has a unique cause, which is not always the case. In addition, the decision making process will consider different causes as diagnostic data are incrementally provided to the expert system.

It can be inferred, from the attempts to create expert systems for concrete repair, that expert system technology is ideally suited to application in this field[47].

Bridge maintenance is particularly suitable for exploitation through an expert system, owing to the fact that many problems exist in the domain that can be only be solved heuristically[1].

Any program must find the cause of a defect by analysing the symptoms[1]. Figure 2.13 demonstrates an interactive session with the DIAGCON expert system. It demonstrates a simple text based interface expert system.

> WELCOME TO DIAGCON
Please answer the following questions with either "true" or "false" or with relevant data, as the case may be. The proforma should have been made available to you before this session.
> basic symptom is cracking? T
> direction of cracking? VERTICAL
> rust stains or spots present? F
-----------------------------------------------------------
> structural element? BEAM
> cracks originate in maximum moment region? T
> crack width maximum at top or bottom of beam? T

> PROBABLE CAUSE :
Cracking due to flexural capacity of beam being exceeded.
> crack determination method? TWO
> glass strip is cracked or disjointed? T

**Figure 2.13 Typical session with DIAGCON expert system[47]**

DIAGCON, and the other prototype expert systems identified in this chapter attempt to diagnose effectively the two key causes of concrete defects: cracking and spalling[39].

Diagnosis of concrete defects can be standardised into common procedures as demonstrated in Figure 2.14.

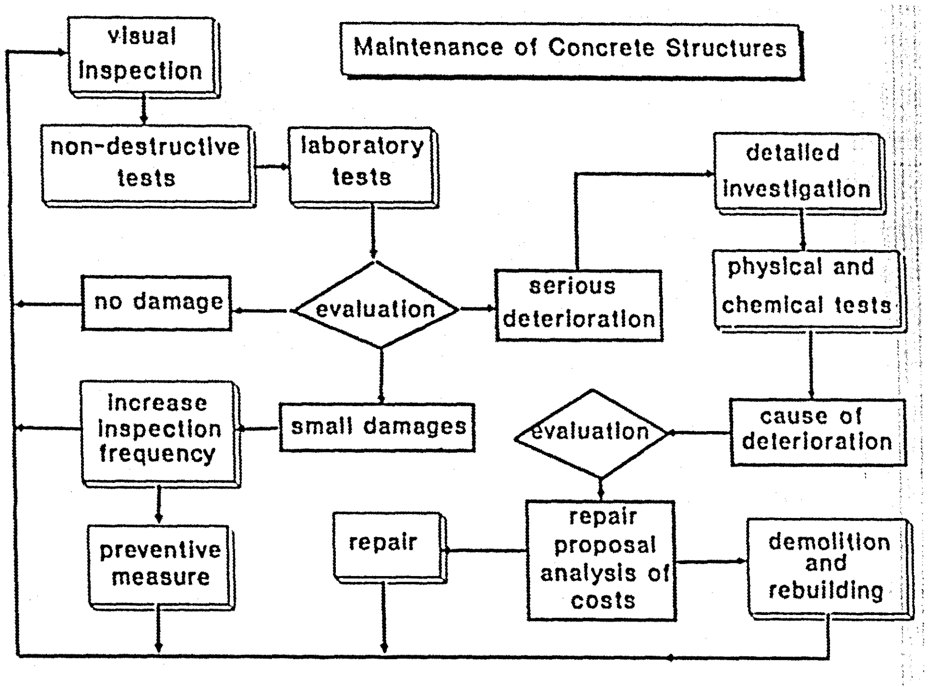



**Figure 2.14 Approach to concrete structure maintenance[1]**

Another expert system discovered through this review is REPCON which is designed to aid experienced engineers in finding out the causes of damage to concrete structures and give tentative repair recommendations[1]. The prototype of REPCON showed that the use of expert system technology in this domain is a possible way to provide the knowledge, which is dispersed in numerous publications and in a few human experts. The structure of REPCON is shown in Figure 2.15.

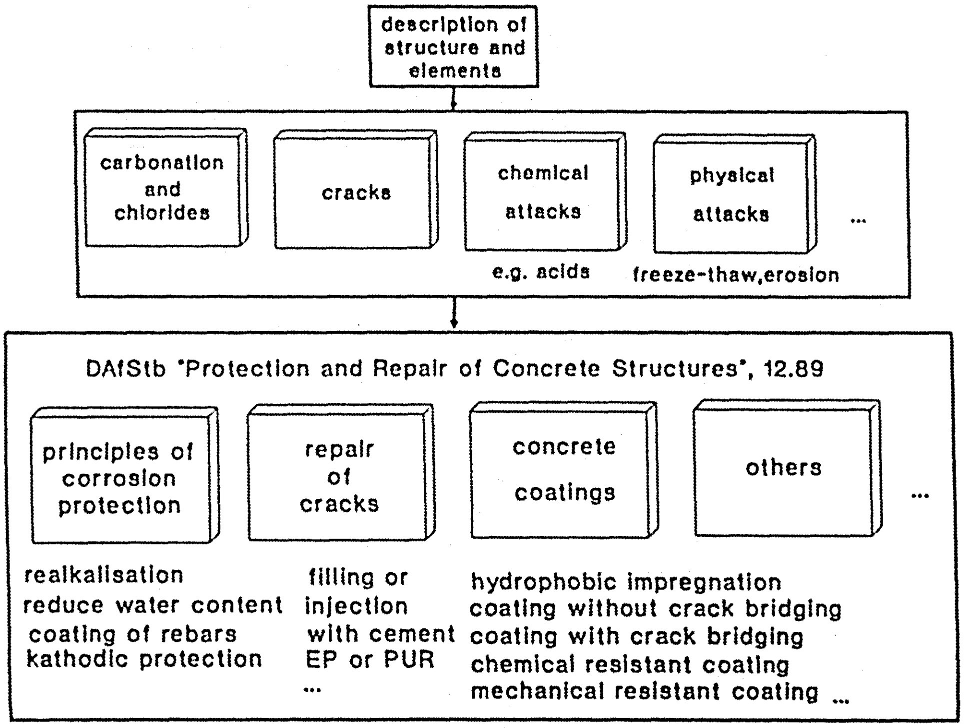



**Figure 2.15 Structure of REPCON**[1]

Rajeev and Rajesh[47] (Diagcon) state that it is unfortunate that the results of their system do not always lead to a unique conclusion. However, the opinion of experts from a visual survey will often not lead to a unique conclusion as to the cause of the visual defects. According to Rajeev and Rajesh[47] once the cause of deterioration has been identified, the next step is to decide a suitable repair method. Although they recognise that repair should only be undertaken when the defect has been diagnosed with some certainty, their system does not recommend testing procedures to confirm the diagnosis of the visual inspection.

## 2.7.2.2 Severity and extent ratings

Using fuzzy logic, the extent and severity of each defect or each cause can be expressed in terms of linguistic variables, and both extent and severity can be combined[43,48]. Fuzzy logic is a subset of conventional logic which has been extended to handle the concept of partial truths. Figure 2.16 shows a typical fuzzy set. It demonstrates a relationship between natural language and numerical judgement. If someone was to assess a statement as being 'very true' and that statement was converted from language into a numerical value (where unity represents absolute truth) then in effect fuzzy logic ascribes a zone of values instead of a singular value. The vertical axis in the figure represents certainty. The technique can be used where natural language qualifiers such as 'small, medium, large' need to be handled numerically, but the vagueness of the language also needs to be modelled (Figure 2.16)

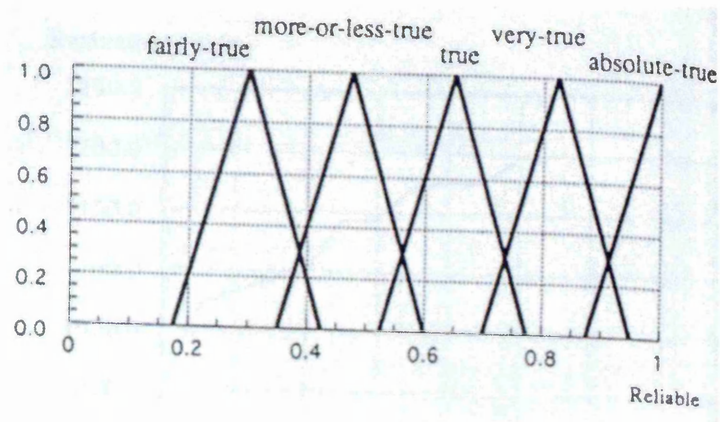



**Figure 2.16 An example of fuzzy sets[49,50]**

### 2.7.3 Handling uncertainty

The Mente_Kun prototype utilises certainty factors. An expert who constructs an expert system ought to know the importance of 'fuzziness'[44]. An approach should be identified which can handle uncertainty when developing an expert system.

A certainty factor (CF) is a numerical value that indicates a measure of confidence in the value of that parameter[1]. For example IF DAMAGE = CRACKS

CRACK TYPE = RANDOM PATTERN

CRACKINFO = WHITE RESIDUE

CRACK SIZE = LARGE DEFECT

THEN CAUSE = AAR CF 60%

A similar approach is adopted by Rajeev & Rajesh[47] where a 'confidence factor' is appended to the result of a rule.

If data available is reliable and extensive, the expert can pin-point the most appropriate repair method with full confidence. With uncertain information, repair procedures may still be specified, but with less confidence[39].

## *2.8 General application of Information Technology in Bridges*

### 2.8.1 Bridge Management Systems

Bridge management systems were first developed in the 1980s in the USA[51]. They consist of databases that store the key information regarding bridges; bridge inspectors and engineers are required to refer to these databases in their day to day practises. Early bridge management systems were developed for data storage and retrieval purposes, such as:

- Entering bridge data
- Viewing inspection results
- Viewing and editing bridge data
- Viewing forthcoming inspection dates for bridges

However, modern systems contain advanced modules that can be used to predict the most cost effective long term repair strategies by using 'whole life costing' methods[52]. In addition, modern systems attempt to model the deterioration of bridges using complex algorithms[53]. Bridge controlling authorities in some countries have constructed extensive bridge management systems to catalogue a large number of structures. The American (US) PONTIS[54] system is used in many states to create inventories of their large bridge stock. The Highways Agency in England use the SMIS system (structure management and information system). The results of all principal inspections on Highways Agency structures are fed into SMIS by competent trained personnel. As a result an accurate electronic record of the conditions of the Highways Agency's bridge stock is kept. The Highways Agency hope to procure new software which will interrogate SMIS in order to identify which bridge repairs will provide the best value from the Agency's yearly budget. Information technology is rapidly expanding in the bridge repair and maintenance field as the benefits of more esoteric software (such as expert systems) are promoted.

# 3 Selection of materials for optimal performance of concrete repair

## 3.1 Chapter Objectives

In order to function satisfactorily, a repair material applied to reinforced concrete must perform several functions:

- Bond strongly to the substrate concrete
- Bond strongly to the reinforcing bars
- Have an adequate tensile strength to accommodate restrained volume changes
- Prevent penetration of water, chloride solution and carbon dioxide to reinforcing steel
- Share load with the substrate concrete if necessary

The ability to determine how well repair materials will perform, under the varying conditions in which they may be employed, would enable an engineer to make intelligent choices when repairing concrete defects. Currently however, there is disparity amongst the opinions of researchers regarding which properties of repair materials are the most important to specify. As a result of the lack of a clearly defined method for the selection of repair materials, they are currently selected by many practitioners on an *ad hoc* basis. Some engineers recommend using similar values for the respective properties of both the substrate concrete and repair materials; others recommend high compressive strength and low shrinkage repair materials for the majority of situations. The number of permutations of different recommendations is high, and the lack of a coherent opinion can often lead to the simple practice of the highest strength material being selected.

Recent research[55,56,57,58] suggests that the key mechanical properties of a repair material to be considered when selecting a material for reinforced concrete repair are:

- Elastic Modulus
- Shrinkage
- Creep

Knowledge of the growth with time of these properties in a repair material, and of the interaction between the substrate concrete and repair material at their interface has allowed the development of a technique to predict the short and long term performance of concrete repairs. In this chapter, the development of the method to predict the in-situ performance of repair materials for reinforced concrete is outlined. The method requires knowledge of certain mechanical properties of both the substrate concrete and the repair material. Curing effects caused by local seasonal temperature and relative humidity variations, along with dimensional shrinkage differences are also taken into account.

The chapter begins with a literature review of the relevant domain.

## 3.2 Literature review

### 3.2.1 Introduction

Reinforced concrete is a strong and durable material; approximately eighty percent[59] of bridges in the UK are constructed using it. Reinforced concrete structures give excellent durability when designed, constructed and maintained correctly, justifying their design lives of 60 to 120 years[60]. Approximately 500 million pounds is spent in the UK each year on the repair of concrete[61]. Reinforced concrete deteriorates due to environmental effects such as the ingress of carbon dioxide which neutralises the natural alkalinity of the concrete, or the diffusion of chloride solution through small cracks and the pores which depassivates the reinforcing bar's environment. Both these effects can lead to cracking of the concrete surrounding the reinforcing steel. This cracking can delaminate the concrete, and eventually delaminated concrete can break away from the parent concrete (substrate). This process is known as spalling, and the remaining patch of exposed sub-surface concrete is known as a spall. These effects, and a number of other aggressors can either directly cause a spall, or persuade the engineer (or expert system) to recommend the removal and replacement of the affected concrete. The result of these defects can be a loss of strength in the affected members, demanding immediate attention. Alternatively, defects can merely prove aesthetically unacceptable; these defects require repair to halt further deterioration and to reproduce the original appearance of the substrate.

Once a defect is discovered and diagnosed, loose concrete and other defective areas are removed and the exposed substrate concrete is prepared for the application of a repair material. The interaction of a repair material with the substrate concrete is the crucial factor in determining the performance of the repair patch[55,56,57,58,61,62,63,64,65]. Volume change in the repair material (usually shrinkage) is restrained by the substrate concrete, and occasionally this restraint to shrinkage can cause tensile stresses which exceed the tensile strength of the repair material. Understanding the interaction between the repair material and the substrate concrete will allow the engineer to carefully select the properties of the repair material to ensure adequate performance.

The current standard for the specification of materials for concrete repair on Highways Agency structures is BD 27/86[66]. This standard recommends storage methods, densities, aggregate size and constituent proportions and some mechanical property values for:

- Decks and vertical surface to piers, columns and abutments
- Sides and soffits of beams and crossheads

Although BD 27/86 recommends material types and cement contents, the only mechanical property of repair materials recommended is compressive strength.

A more thorough standard for concrete repair will be the eurocode ENV 1504-1:1997[22,23] currently available in draft. This code takes a more sophisticated approach to the specification of concrete repair properties which encompasses modern thinking on which properties are crucial to specify.

### 3.2.2 Selecting repair materials for reinforced concrete

Any successful repair material should have the ability to[67]:

- Arrest the deterioration of the structure by preventing access of oxygen, water and aggressive ions
- Provide an environment that chemically passivates the reinforcement
- Restore the structural integrity of the element

- Restore the aesthetic appearance of the element

Selecting repair materials that can deliver these performance requirements involves a thorough understanding of material behaviour in anticipated service and exposure conditions[68]. In truth, the performance of concrete repair materials has been an under-researched field. The resulting lack of understanding of the behaviour of in-service repair materials necessitates a systematic approach to concrete repair design and construction. The selection of design values and decisions needs to be more rational[69] and, ideally, a broad range of research - particularly new, state of the art research in the field - needs to be collated and consolidated to form rigorous new guidelines. The financial benefits to ensuring the success of repair materials are considerable.

A repair to reinforced concrete can be affected by five key factors[70]:

- The effect of the constituent materials on the properties of the repair material
- The effect of the properties of the repair material on its durability
- The effects of environmental conditions on curing and durability
- Effects due to the interaction between the repair material and the substrate concrete
- Loading effects (transfer of load into repair, depropping etc)

The importance of accurately predicting the performance of a repair at the design stage is crucial. Studies[69] have shown that the level of influence on the durability of a repair material is at its highest during the design phase of the repair (Figure 3.1).

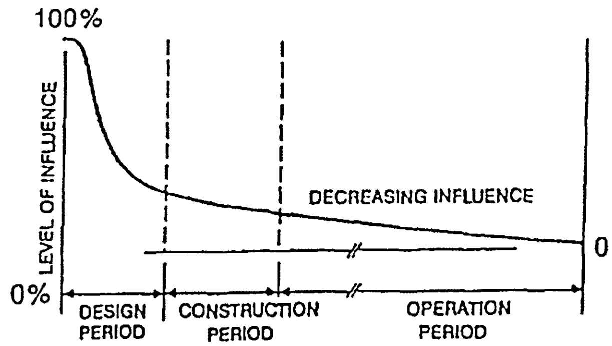



**Figure 3.1 The effects of different periods on project quality (durability)[69]**

Figure 3.1 demonstrates the high level of influence over the quality of a repair project which is wielded at the design stage. This reinforces the need for a critical evaluation of current methods for the specification of reinforced concrete repairs.

Concrete repair may be broadly categorised as structural or protective[68,69]. Failure of repair materials is undesirable in either of these circumstances. In order to alleviate, at the design stage, all possibility of failure, the key properties that influence the performance of the repair need to be identified. As stated previously there is some disparity amongst engineers about the key properties to be considered when attempting to combat the failure of repair materials. Furthermore, there is also some disagreement, once key properties have been established, as to the relative values these properties should hold in order to reduce the risk of failure.

If failure does occur, it is invariably through two key mechanisms; restrained volume changes and loss of bond.

### 3.2.2.1 Restrained volume changes

Cement based repair materials are volumetrically unstable[69]. During the curing process the fluctuation in moisture levels within the repair causes volume changes.

Reinforced concrete repair materials undergoing volume changes are restrained by the substrate concrete and also partly by the reinforcing steel. As this occurs, tensile strains are induced which, if greater than the tensile strain capacity of the repair material, will cause cracking[69]. Volume changes must, therefore, be controlled in concrete repairs to prevent or minimize cracking. However, predictions of the magnitude of developed strains in repair materials need to take account of the complex interaction between properties such as[55,56,57,58,62,63]:

- Elastic Modulus
- Shrinkage
- Creep

Figure 3.2 shows that restrained volume change has one of two outcomes[64,71] which can contribute to the failure of concrete repairs. The restrained volume changes can lead to cracking, which provides a passage for moisture though to the steel. If the moisture contains aggressive agents, the steel can become depassivated and corrode, larger cracks can occur, and the cycle can perpetuate, leading to spalling, and potentially a dangerous loss of strength. The other cause, bond strength, is a factor very much affected by on-site preparations. If a surface is thoroughly prepared for the application of a repair, following best practice guidelines, and if the bond strength of the repair material is adequate, then a repair should not fail through this mechanism.

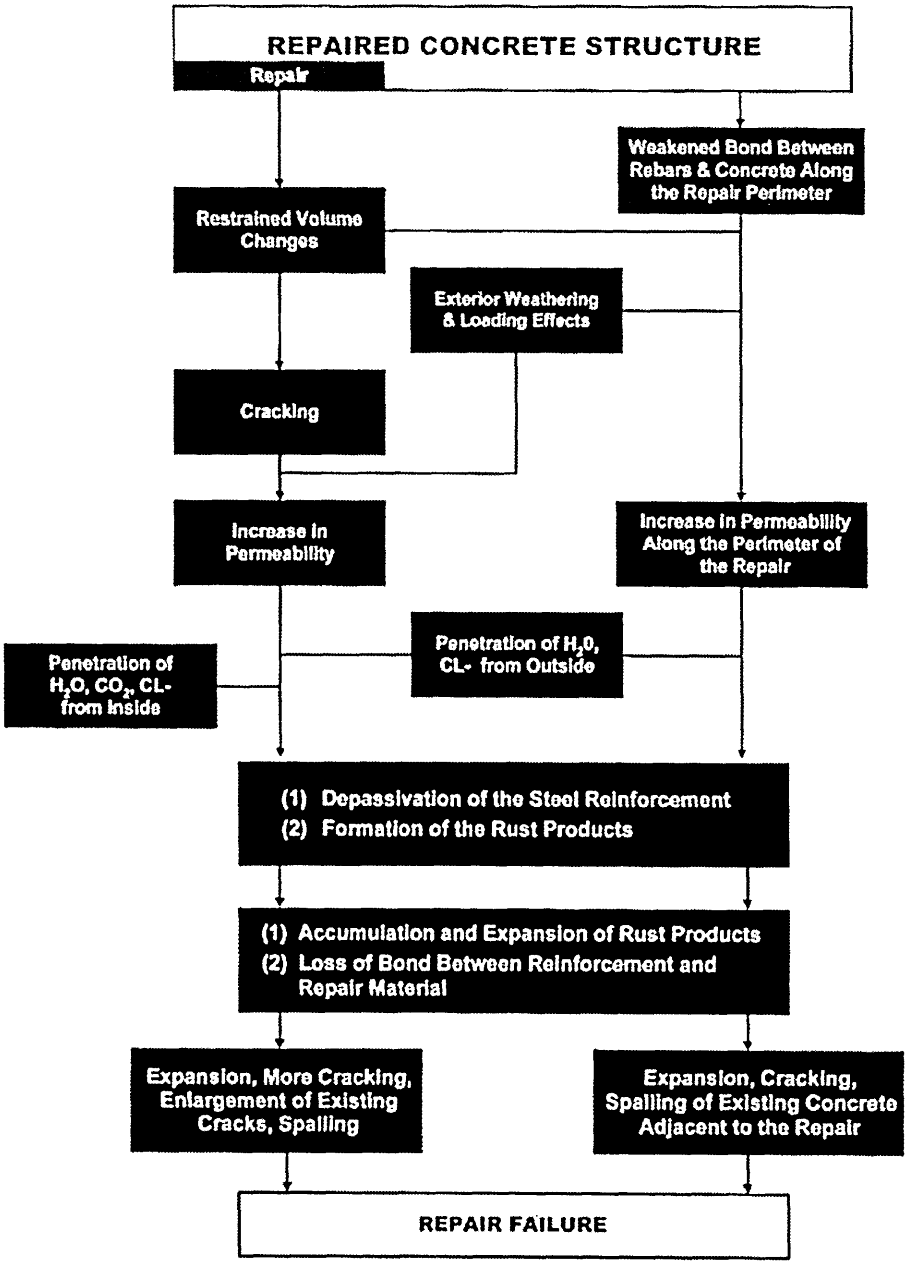



**Figure 3.2 A model of repair failure[70]**

Measures to combat the effects of restrained volume change (usually shrinkage) are more complex[72,73]. Emmons and Vaysburd suggested that repair materials with low strength, low shrinkage, high creep and low modulus of elasticity were most desirable for non structural, protective repairs[69]. This combination of properties, it is suggested, will produce a high strain capacity repair system. Their hypothesis is based on their intuitive understanding of the cumulative effects of these properties.

Morgan[74] suggests that the potential for success or failure of repairs depends upon:

- The magnitude and the state of the stress field
- Whether the load is left on the structure during the repair operations
- The creep capacity of the repair materials
- The quality of tensile and shear bond strength of the repair material to the substrate concrete
- The temperature at which the repairs were carried out and subsequent range of temperatures during service life

These deductions specify tensile and bond strength as well as creep being the key mechanical properties. Once again, these deductions are based on an intuitive understanding of the effect of these properties on repair material behaviour.

Table 3.1 represents the conclusions into a study of the significance of property mismatch between repair and substrate. It attempts to stipulate ideal relationships between substrate and repair material properties for a successful repair, although it does not recommend values of these properties, just their values relative to each other.

Emberson and Mays[61] have stated that repair materials can be deemed suitable on the basis of their compressive, tensile and flexural strengths alone. They also recommend high strain capacity in the repair material, and a modular ratio (the ratio of the elastic modulus of the repair material to the elastic modulus of the substrate concrete) of unity.

**Table 3.1 Requirements of patch repair material for structural compatibility[61]**

| Property | Relationship of repair mortar (R) to concrete substrate (C) |
|---|---|
| Strength in compression, tension and flexure | $R \geqslant C$ |
| Modulus in compression, tension and flexure | $R \approx C$ |
| Poisson's ratio | Dependent on modulus and type of repair |
| Coefficient of thermal expansion | $R \approx C$ |
| Adhesion in tension and shear | $R \geqslant C$ |
| Curing and long term shrinkage | $R \leqslant C$ |
| Strain capacity | $R \geqslant C$ |
| Creep | Dependent on whether creep causes desirable or undesirable effects |
| Fatigue performance | $R \geqslant C$ |

Table 3.2 summarises the recommendations for values of repair material properties given by eleven independent researchers. It recommends whether the values of compressive strength, tensile strength or Young's Modulus for a repair material should be greater than, lesser than, or equal to those of the substrate concrete. A lack of agreement is shown on the relative importance of the strength and elastic modulus. Where available, the opinion on creep confirms its importance. Additionally, shrinkage is considered important, and should be low.

**Table 3.2 Recommended repair material property limits and values for effective application**

| Reference | Properties of repair materials | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 28 day compressive strength $F_{c28}$ | | | 28 day tensile strength $F_{t28}$ | | | 28 day elastic modulus $E_{rep28}$ | | | Creep | Shrinkage | Noteworthy comments from reference |
| | >sub | =sub | <sub | >sub | =sub | <sub | >sub | =sub | <sub | | | |
| [69]Emmons | low | | | low | | | low | | | high | | For non structural applications |
| [74]Morgan | | | | important | | | | | | important | | Bond strength also important |
| [61]Emberson | ✓ | | | ✓ | | | | ✓ | | Effect undetermined | Low (high strain cap.) | Able to decide suitability based on strengths alone |
| [67]Rizzo | | ✓ | | | ✓ | | | | | | No shrinkage (or 'very slight') | 'this match of properties is generally accepted in the profession' |

Table 3.2 Continued on next page

| | Properties of repair materials | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Reference | 28 day compressive strength $F_{c28}$ | | | 28 day tensile strength $F_{t28}$ | | | 28 day elastic modulus $E_{rep28}$ | | | Creep | Shrinkage | Noteworthy comments from reference |
| | >sub | =sub | <sub | >sub | =sub | <sub | >sub | =sub | <sub | | | |
| [75]Mays | | | | important | | | ✓ | | | important | important | Bond strength important. $E_{rep} = E_{sub} \pm 10 kN/mm^2$ |
| [76]McDonald | Low | | | Low | | | low | | | high | low | 'the recommendations are generally accepted in the profession' |
| [76]McDonald | | | | 2.8MPa | | | 24GPa | | | | low | 'proposed performance criteria' |
| [68]Poston | 27.6MPa | | | Min $0.08f_{cu28}$ | | | Recommendation 'cannot be made' | | | | low | |

Table 3.2 Continued on next page

| Reference | Properties of repair materials | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 28 day compressive strength $F_{c28}$ | | | 28 day tensile strength $F_{t28}$ | | | 28 day elastic modulus $E_{rep28}$ | | | Creep | Shrinkage | Noteworthy comments from reference |
| | >sub | =sub | <sub | >sub | =sub | <sub | >sub | =sub | <sub | | | |
| [77]Dector | Specified* | | | Specified* | | | Specified* | | | | Specified* (restrained) | Hong Kong Housing Authority specification |
| [78]Plum | | | | important | | | important | | | important | important | Bond strength Thermal expansion Saturation expansion |
| [56,57,58,62,63] Mangat, O'Flaherty, et al | | | | high | | | ✓$^{\perp}$ | | | important | low | |

* Table 3.3 Hong Kong Housing Authority repair material specification[77] $^{\perp}$ Recommended $E_{rep} \geq E_{sub} \times 1.32$

$F_{c28}$ = 28 day compressive cube strength

$F_{t28}$ = 28 day tensile strength

$E_{rep28}$ = 28 day Elastic Modulus (in compression)

$E_{sub}$ = Elastic Modulus of substrate concrete

The Hong Kong Housing Authority have developed detailed specifications for classes of concrete repair mortars to be used in the repair of structures[77], which are given in Table 3.3.

**Table 3.3 Hong Kong Housing Authority repair material specification[77]**

| Characteristics for repair mortar | | | | |
|---|---|---|---|---|
| *Characteristics* | | *Required values* | | |
| | | *Class 40* | *Class 25* | *Class 15* |
| TM1 | Range of compressive strength at 28 days in $N/mm^2$ | 30-60 | 20-40 | 10-30 |
| TM3 | Minimum tensile strength at 7 days in $N/mm^2$ | 2.0 | 1.5 | 1.0 |
| TM4 | Range of modulus of elasticity at 28 days in $kN/mm^2$ | 15-25 | 9-15 | 5-9 |
| TM5 | Minimum bond strength at 7 days in $N/mm^2$ | 2.0 | 1.5 | 1.0 |
| TM6 | Cracking in Coutinho ring test at 7 days | 0 | 0 | 0 |
| TM7 | Minimum Figg air permeability in seconds | 200 | 150 | 100 |

Recommendations in this field often tend to be based on researchers' intuitive understanding of the interaction of repair materials with the substrate concrete and opinions tend to be divergent. A general consensus identifiable from the research is that when selecting a suitable repair material for reinforced concrete repair with the aim of combating excessive strains due to restrained volume changes, the following material properties are important:

- Strength (particularly tensile)
- Elastic Modulus
- Shrinkage
- Creep

Specification of materials for reinforced concrete repair is hampered by a paucity of information on the optimum mechanical properties of repair materials required for a particular substrate[78]. Faced with this difficulty, designers react by adopting materials that appear to have properties close to those of the original concrete. In doing so they risk selecting materials based on incorrect assumptions; materials which may fail through this poor specification method.

### 3.2.2.2 Repair Materials

The materials for all types of reinforced concrete repair fall into four general categories[60]:

- Cement based
- Resin based
- Polymer-modified cement based
- Cement—pozzolanic materials

Since the 1960s a plethora of new, enhanced concrete repair materials and systems have been introduced and have found increasing utilization[61,74].

General aims of these enhancements are to improve tensile strength or reduce shrinkage in the materials. The aim of reducing shrinkage is to limit the tensile strains caused by restrained shrinkage.

Table 3.4 and Table 3.5 show the different categories of repair materials and their typical properties respectively. Intuitively, combining the low shrinkage of a repair material with properties that produce a high tensile strain capacity would produce a material less likely to fail. However, the low shrinkage materials are often prohibitively expensive. This, combined with a lack of understanding of the benefits of reducing the tensile strains that develop in repair materials, can lead to the selection of materials with the simplistic requirement of high compressive strengths. The high strength seems intuitively acceptable to engineers and such materials are often more affordable. Moreover, manufacturers' data on shrinkage properties of their materials often provides lower values than the material can realistically achieve in practice[58].

**Table 3.4 Categories of systems for concrete patch repair[74]**

| Resinous materials | Polymer modified cementitious materials | Cementitious materials |
|---|---|---|
| A: Epoxy mortar | D: SBR modified | G: OPC/sand mortar |
| B: Polyester mortar | E: Vinyl acetate modified | H: HAC mortar |
| C: Acrylic mortar | F: Magnesium phosphate modified | I: Flowing concrete |

**Table 3.5 Typical mechanical properties of repair materials[74]**

| Property | Resin mortar | Polymer modified cementitious mortar | Plain cementitious mortar |
|---|---|---|---|
| Compressive strength, MPa | 50–100 | 30–60 | 20–50 |
| Tensile strength, MPa | 10–15 | 5–10 | 2–5 |
| Modulus of elasticity in compression, GPa | 10–20 | 15–25 | 20–30 |
| Coefficient of thermal expansion (per °C) | $25–30 \times 10^{-6}$ | $10–20 \times 10^{-6}$ | $10 \times 10^{-6}$ |
| Water absorption (% by mass) | 1–2 | 0.1–0.5 | 5–15 |
| Maximum service temperature (°C) | 40–80 | 100–300 | >300 |

Recent trends have led to the modification of cement based materials with polymers. Polymer dispersions allow the formulation of repair materials that can provide a wide range of property requirements: brittle to ductile, impermeable to porous, water shedding etc.[79]. This is achieved through utilising the polymer's ability to alter the mechanical properties: elastic modulus, creep and shrinkage, bond strength, temperature and humidity effects[79]. The objective of adding polymer fibres into the repair mix is to improve tensile strength and to distribute and limit cracking[80].

There appears to be some incongruity between the supposed desire to avoid property mismatch, and the vast disparity between repair material and substrate properties caused by the use of polymer modified materials[74]. Regardless of the general recommendation to avoid property mis-match, manufacturers have continued to develop polymer modified materials producing higher strength materials with lower shrinkage.

### 3.2.2.3 Compatibility, durability and property mismatch.

The current Highways Agency standard for reinforced concrete repair (BD 27/86) does not take into account the mismatch in basic material properties such as elastic modulus, shrinkage and creep[55]. To overcome this lack of standardisation, the greatest challenge faced in the advancement of concrete repair material selection techniques is controlling the relative dimensional behaviour of the repair material when compared to the substrate[68]. This relative behaviour requirement is known as dimensional compatibility. Compatibility in a repair system can be more fully defined as the balance of physical, chemical and electrochemical properties and relative dimensions of the repair patch and the surrounding substrate[69]. Researchers agree that 'compatibility' between substrate and repair material is a key factor in deciding the performance of the repair. Hence the term 'compatibility' has become a popular buzz word in the repair industry[74].

Some researchers have attempted to show that the physical characteristics of the repair material and substrate concrete should be as close as possible (Young's Modulus, coefficient of expansion, strength)[60,61,74,75,80]. This definition of compatibility is misleading and suggests that in order for materials to be 'compatible', they must have similar properties. This is not the case. Compatibility should mean that the relative values of properties of the repair and substrate materials are 'complimentary' and only compatible in as much as that they are at the ideal ratios to ensure optimum performance of the repair. Other studies[61] have recommended that although certain physical characteristics should be similar, others should vary to aid durability.

Two such studies (Figure 3.3 and Figure 3.4), each regard dimensional compatibility as the most complex state to achieve, it being reducible into four or five properties. Interestingly, both regard shrinkage, creep and elastic modulus as crucial.

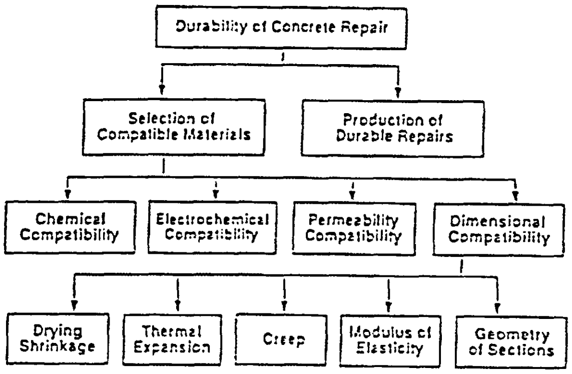



**Figure 3.3 Compatibility and durability[74]**

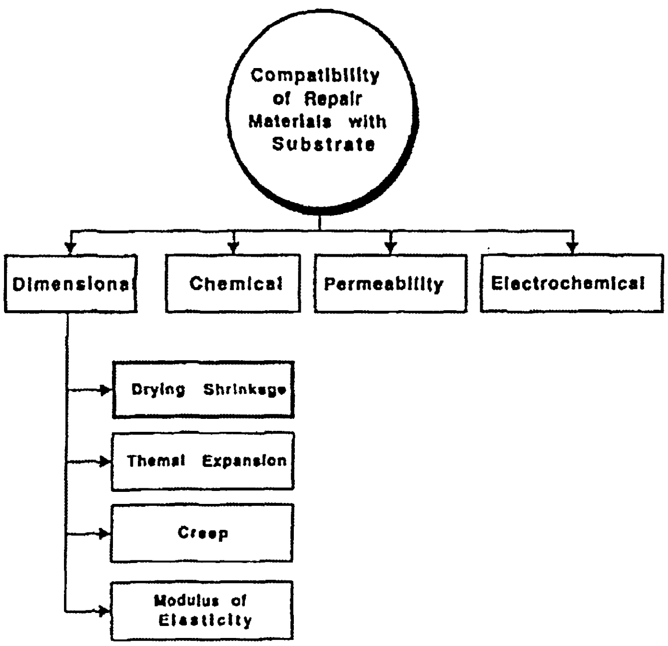



**Figure 3.4 Factors affecting dimensional compatibility[69]**

Other studies[61] have also recognised that incompatibilities in the form of differing elastic moduli and different thermal movements between the repair and substrate concrete can create difficulties and that creep of the repair material may render a repair less effective over time.

Dector[64], in another of her studies, recommended a range of properties to specify for a repair material which would lead to adequate 'compatibility'. His recommendations are shown in Table 3.6.

**Table 3.6 Recommended values for compatibility[64]**

| Characteristic | Suggested requirement |
|---|---|
| Strength and movement properties | Compatibility required. Properties should be similar to the substrate concrete, especially with respect to movement. |
| Bond strength | Greater than 0.8 $N/mm^2$ (Ref. 3) |
| Shrinkage | As low as possible. Proposed limits: <500 microstrain at 28 days (USA Ref. 10)* **or** <300 microstrain at 7 days (HKHA Ref. 8)* * see conditions relevant to test |
| Permeability | Less than Concrete Society low permeability limits. In areas of high risk, lower limits may be imposed. |

Table 3.6 recommends values for certain properties which it is claimed will ensure compatibility. However, the majority of research[56,55,61,62,63] suggests that such a specification is unwise, as knowledge of the properties of the substrate concrete is essential before a suitably compatible repair material can be selected. Emmons and Vaysburd[69] represent the factors necessary to achieve compatibility diagrammatically (Figure 3.5).

Crucially, they show that knowledge of the substrate properties and the exposure conditions are required to achieve compatibility.

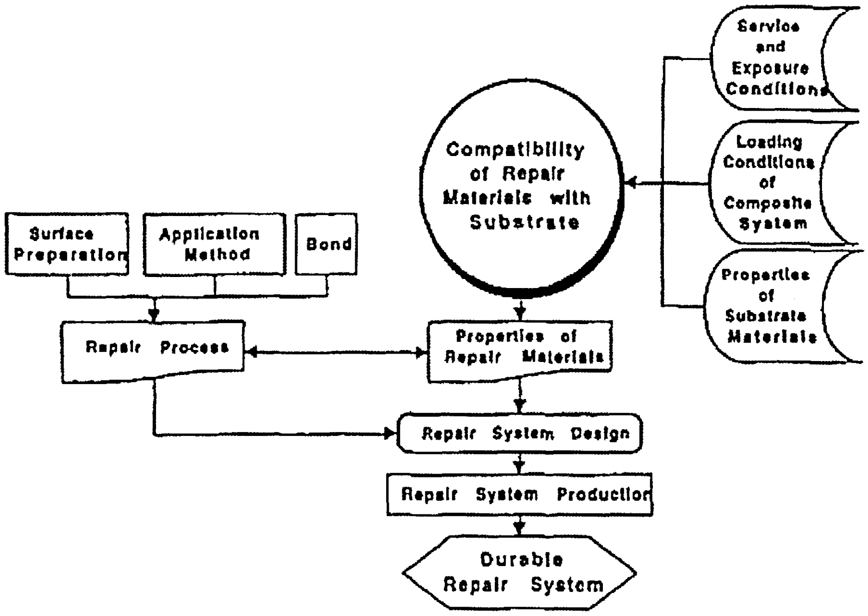



**Figure 3.5 Factors affecting the durability of concrete repair systems[69]**

The research thesis by O'Flaherty[58], reports by Mangat & O'Flaherty[55,56,57,62,81] and a series of reports by Emberson & Mays[61] suggest that successful specification of repair materials for reinforced concrete is more complex than is implied by the Highways Agency standard BD 27/86[66]. These works suggest that the key properties of repair materials, which affect long term performance, are:

- Elastic Modulus
- Shrinkage
- Creep
- Tensile Strength

The durability of concrete repair depends, to a large degree, on the appropriate choice and application of repair materials[68], a lack of durability may manifest itself in spalling, cracking, scaling and subsequent loss of strength in the repaired concrete member[73].

### 3.2.3 Properties of repair materials

#### 3.2.3.1 Key properties

Mangat and O'Flaherty[55] suggest that a key property which will determine the long term performance of concrete repair is the Elastic Modulus.

The restraint to the free shrinkage of a repair material caused by the bond between substrate concrete and repair material is the main factor which contributes towards the likelihood of a repair material failing through cracking.

A survey of three bridges was performed by O'Flaherty[58] by attaching vibrating wire strain gauges to exposed reinforcing steel in a repair patch, and internally at the substrate concrete interface and inside the repair material. These gauges measured the developing strains throughout the drying process of an applied repair material.

O'Flaherty postulates that if (or when) a patch repair material becomes stiffer than the repaired substrate concrete, some of the tension inducing shrinkage strains of the repair material being restrained at the interface with the substrate concrete will be transferred into the substrate, (Figure 3.8). This transfer of strain from the repair material into the substrate was measured using the vibrating wire strain gauges. These measured values were related to the ratios of elastic moduli of the substrate concrete and the repair materials. It was shown that, the higher the ratio of the elastic modulus of the repair material to the elastic modulus of the substrate concrete, the larger is the percentage of shrinkage strain that is transferred from the repair material into the substrate concrete. This shrinkage transfer alleviates the tension due to the restrained shrinkage and increases the possibility of a successful repair.

These results (explained more fully in section 3.2.5) form the basis of a sound understanding of the properties of a reinforced concrete repair material that are most crucial in affecting the performance of the repair.

#### 3.2.3.2 Elastic Modulus

Research has shown the pre-eminent influence of the relative elastic moduli of the repair material and substrate concrete on the performance of repairs[62]. It has been identified through field studies[55,56,57,58,63] that as a repair material becomes stiffer than the substrate concrete it is repairing, it can transfer its shrinkage strains into the substrate. It is also known that the degree of strain transfer increases with increasing $E_{rm}/E_{sub}$ ratio up to 1.32 (where $E_{rm}$ = Elastic modulus of repair material, $E_{sub}$ = Elastic modulus of substrate concrete). As a consequence restrained shrinkage tension in the repair material is reduced[62]. This phenomenon had also been identified implicitly in other studies, for example, Emberson[61] states, 'the repair material with a low modulus caused an increase in concrete stress, whereas a repair material with a high modulus resulted in a decrease.'

In addition to the tensile strain transfer benefits of a high elastic modulus repair material, it has also been shown that materials with a high modulus tend to attract load away from the substrate concrete[61] in the long term. Such an interaction is essential if the intention of a concrete repair is to restore structural capacity to a member.

Conversely, one study concluded that increased cracking is usually attributed to higher modulus of elasticity (amongst other factors) and stated that it is generally agreed that the potential for cracking for cement-based repair materials decreases with decreases in modulus of elasticity[76]. However, it found no significant correlation between modulus of elasticity and field performance[76].

Other studies recommend moduli similar to that of the substrate[74,75], one such study stated that an ideal repair material would undergo neither shrinkage nor expansion and would display a similar modulus of elasticity to the substrate concrete[74]. Such a specification would probably perform satisfactorily in most cases as the lack of shrinkage would not induce any tension in the repair material. In fact, in such a case the material should theoretically perform satisfactorily even with a much lower or higher modulus of elasticity. However, in reality even polymer modified materials with the best shrinkage compensation will exhibit shrinkages of 200 to 300 microstrain at 28 days, and the majority of materials will shrink much more than this. Other studies have found that they cannot make a definite recommendation for limits of modulus of elasticity based on field results[68].

Older research has generally not considered Elastic Modulus to be an important property of a successful concrete repair material. Authors have recommended that the modulus of repair materials should be lower than the substrate modulus or the same as the substrate modulus. However, new research, based on verifiable field testing has proven that the optimum modular ratio between the repair material and the substrate is higher than unity. Therefore, it is shown that the modulus of elasticity of a patch repair material in relation to the substrate concrete may have a significant influence on the distribution of stress within a repaired reinforced concrete member.

When the elastic modulus of the repair material is greater than that of the substrate concrete, the repair material carries more load than materials having modulus values equal to or less than that of the concrete[58,61]. This can be considered detrimental, because high modulus repairs can cause localized areas of maximum principal stress adjacent to the transverse interface that are greater than those in the concrete remote from the repair site[61]. However, if an aim of the repair procedure is to restore structural capacity to an ailing concrete member then the transfer of external load into the repair patch will be beneficial when a high modulus repair material is used.

### 3.2.3.2.1 *Compressive and tensile elastic moduli*

Tests to establish Elastic Modulus invariably produce compressive elastic modulus values. However, the tensile stress strain relationship of a repair material, which is mobilised when the free shrinkage of a repair material is restrained by the substrate concrete, is described (the linear portion) by the tensile elastic modulus of the material. Although there is little data available for the modulus of elasticity of concrete in tension, an assumption can be made that the elastic modulus of concrete in tension is approximately the same as the elastic modulus of concrete in compression[82].

More precisely, when the compressive and tensile elastic moduli of concrete are measured on identical specimens at 0.3 $f_{cu28}$ ($f_{cu28}$ = the compressive cube strength of a sample at 28 days after air curing) the elastic modulus in compression is 7.5% higher than the tensile elastic modulus[83]. When the compressive elastic modulus is measured at a stress equal to 0.3 $f_{t28}$ ($f_{t28}$ = The tensile strength of a sample at 28 days after air curing), then the tensile elastic modulus has been shown to be 2.5% greater than the compressive elastic modulus[83]. Therefore it is shown that for concrete at a very young age, the tensile elastic modulus may be marginally higher than the compressive elastic modulus if measured on two identical specimens. At approximately ten days after curing, the compressive elastic modulus and tensile elastic modulus will be similar, thereafter the compressive elastic modulus will be marginally larger. For the purposes of this research, the tensile elastic modulus of a specimen will be assumed equal to its compressive elastic modulus.

For the purposes of the procedures developed in the thesis to determine suitable repair material properties, it is assumed that because any repair material will be in tension, the highest stresses it can accommodate will equal its tensile strength. This in turn will be considerably lower than the compressive strength of the material and hence it is acceptable to consider that tensile and compressive elastic moduli are the same.

### 3.2.3.3 Shrinkage

Shrinkage is caused by the withdrawal of water from the repair material through drying. All cementituous repair materials shrink. Shrinkage is increasingly recognised as a major factor in the long term durability of a repair[77]. The restraint provided to the repair material by its bond to the existing concrete substrate is a major factor in increasing the complexity of repair patches as compared to new construction[68]. As the substrate concrete restrains the free shrinkage of a repair material, tensile strains are developed.

In addition to the hydration shrinkage of cement based repair materials, increasingly popular resin based additives are known to shrink during polymerisation (the hardening process of resin materials). Pure resins can typically shrink between 4% (epoxies) to 10% (polyesters) during this process. However, resin materials are viscoelastic and these (shrinkage) stresses will partly relax.

In addition to elastic modulus, shrinkage of a repair material has been identified as the property which controls long term cracking at the repair/substrate interface[57]. It could equally be stated that Elastic Moduli, shrinkage and creep combined are the primary material properties which can be utilised to specify materials for concrete repair with success; the actual singular cause of cracking and debonding of concrete repairs is excessive shrinkage strains[74]. The ability of a material to cope with these strains depends heavily upon its elastic modulus in relation to that of the substrate and its creep characteristics.

Awareness of the importance of controlling drying shrinkage in repair patches has been increasing[77] as recent studies have identified its crucial importance [55,56,57,58,62,63]. Results from many studies have shown that unacceptable performance of repair materials is based on high shrinkage[61]. Clearly, if the main cause of failure of concrete repairs is high shrinkage, then a logical action to combat failure is to specify low (or nil) shrinkage. However, all repair materials shrink, and even relatively low shrinkage repair materials, if accompanied by low tensile strengths, will still fail. Research has attempted to specify values for shrinkage, from the optimistic 'no shrinkage'[67] to recommended 28 day shrinkage values of 400 microstrain for specimens exposed at 50% RH[68]. Some national standards also attempt to limit shrinkage, for example, the Australian standard AS 1012 has a limit of 450 microstrain at 28 days[77]. Attempts to limit shrinkage in such ways fail to take into account the interrelationship of properties which determine the overall performance of a patch repair. For example, a repair material with an elastic modulus higher than that of the substrate has the ability to transfer a proportion of its free shrinkage to the restraining substrate, hence this combination of elastic modulus and shrinkage could allow for higher shrinkage values. In addition (as discussed in the next section) whenever tensile strains occur these will be relaxed to a certain extent by tensile creep, hence taking creep into account could also allow for a higher amount of shrinkage to be specified.

There are two good reasons to develop a method that allows practitioners to select repair materials that have relatively high shrinkage properties. Firstly, often repair materials that have been specified as 'low shrinkage' by manufacturers actually shrink much more in the field than suggested by the manufacturers' literature. Secondly, the vast majority of available materials cannot achieve shrinkages as low as 330 or 400 microstrain and, therefore, limiting shrinkage to such low values will put uneconomical restraints to repair solutions available in practice.

### 3.2.3.4 Creep

Any strains which develop in a repair material as a result of restrained shrinkage will be relaxed, to a certain degree, by the action of tensile creep. It has been correctly stated that cracking at the repair/substrate interface is primarily controlled by the shrinkage and creep characteristics of the repair materials[57]. The amount of tensile strain in a repair material is dependent on the sum of the restrained shrinkage and the negative effect of the relaxation through creep. Current research has shown that a more accurate statement would be that Elastic Modulus, Shrinkage and Creep fully control the possibility for cracking to occur (assuming satisfactory bond), since the effective restrained tensile strain in the repair material is dependent upon the amount of free shrinkage transferred to the substrate concrete through optimum modular ratio usage and the relaxation of the tensile strain through creep. Excessive creep in the repair material may, however, render a repair less effective over time, as it has been shown that creep reduces the effective Elastic Modulus in the long term[61].

Creep exhibits itself in two primary forms; as instantaneous elastic strain, and creep strain. Instantaneous elastic strain is the creep that occurs immediately as the result of the applied load onto a material. The creep strain is the relatively slow flow of the material with time thereafter and is caused by movement of the water adsorbed onto the surface of hydrating cement gel. Tensile loads are applied in gradual increments in a repair patch with steadily increasing shrinkage. The elastic strain capacity of a repair material in tension is very small, typically 200 microstrain[84], and cracking is prevented if instantaneous elastic strains remain below this value.

*3.2.3.4.1 Tensile and compressive creep*

Research has shown that creep in tension is a significant phenomenon, and can play an important role in reducing stress due to restrained shrinkage[84]. If the tensile strains developed in the repair material due to restrained shrinkage are relaxed by tensile creep, then theoretically a higher initial shrinkage could be accommodated. This would be a benefit, as lower shrinkage materials are generally more costly and less common. Generally, materials are tested to assess their compressive creep properties, as testing for tensile creep is more difficult[85]. It should be added that, currently, repair material manufacturers in the UK do not provide even compressive creep data for their materials. Although, from research literature, a great deal of information is available on the creep of concrete in compression, experimental data on the tensile creep properties of concrete is scarce[84,85]. Brookes and Neville[83] state that tensile and compressive creep can be considered as similar in most conditions. However, during drying, tensile creep can be higher than compressive creep. In the absence of clear information on the comparison of tensile and compressive creep, it will be assumed that compressive creep is similar to the tensile creep of a repair material in the work presented in this thesis.

### 3.2.3.5 Strength

A concrete repair that is intended to restore structural capacity to a member should be designed to withstand the compressive stresses to which it may be subjected. However, it is a fallacy that specifying a high compressive strength will ensure adequate performance of a repair material. The research literature reported in the thesis has shown that key properties which govern repair material performance are elastic modulus, shrinkage and creep. Other research has shown directly that there is no significant correlation between compressive strength and dimensional stability[76]. It is generally agreed that the potential for cracking of cement based repair materials increases with high compressive strengths, despite inherently higher tensile strengths[76]. Conversely, some practitioners have attempted to recommend minimum values of compressive strength for structural application, without regard for the effect of this on the durability of the repair[68].

It should be noted however, that the tensile strength of a repair material and its elastic modulus determine the tensile strain capacity of the material; in this respect the tensile strength (which is related to compressive strength) is important.

### 3.2.4 Influence of material constituents on mechanical properties

Repair materials for reinforced concrete are generally cement based. It is common for manufacturers to use additives to have desired effects on the mechanical properties of the hardened repair material. For example, some additives increase strength and bond whilst some reduce shrinkage. Different constituents will have varying effects on the important mechanical properties of the repair material.

The scope of the current research does not encompass the specification of material constituents. It is an explicit aim of this research to recommend the suitability of repair materials for reinforced concrete repair based on the key mechanical properties which determine their effective performances. Hence it is the mechanical properties of 'off the shelf' repair materials that will be used to determine their performance in patch repairs by developing a routine for a computer. A knowledge of the constituents of those repair materials can provide an understanding of the material properties but will not aid in determining their performance in patch repairs. For this reason an in depth study of the effect of constituent materials of repair materials is not attempted.

### 3.2.5 Testing to establish repair material properties

A variety of test methods are employed to determine the properties of repair materials. The two most widely used standards are the British Standards and the ASTM standards (USA), with many manufacturers using tests from both sets of standards to provide most favourable data for their materials. Table 3.7 shows a variety of British Standard test methods and the procedures employed therein, which was used in a research programme on repair materials[61].

**Table 3.7 Strength and Modulus test methods[61]**

| Property | Test method | | Property | Test method | |
|---|---|---|---|---|---|
| | Resinous materials | Cementitious materials | | Resinous materials | Cementitious materials |
| Compressive strength | 40 mm cubes<br>BS 6319 : Part 2 : 1983 | 70 mm cubes based on BS 4550 : Part 3 : 1978, section 3.4<br>Tested in accordance with BS 1881 : Part 116 : 1983 | Compression modulus | Prisms 40 × 40 × 160<br>BS 6319 : Part 6 : 1984 | Prisms 40 × 40 × 160<br>BS 1881 : Part 121 : 1983 |
| Tensile strength | Briquettes<br>BS : 6319 : Part 7 : 1985 | Briquettes based on BS 12 : 1971. Part 2 (appendix H). Omitted from BS 12 : 1978 and BS 4550 : 1978 | Tensile modulus | Prisms with trapezoidal ends: 40 × 40 × 160. Derived from BS 6319 : Parts 6 and 7. Tested in accordance with BS 1881 : Part 121, but in tension* | Prisms with trapezoidal ends: 40 × 40 × 160. Derived from BS 6319 : Parts 6 and 7. Tested in accordance with BS 1881 : Part 121, but in tension* |
| Flexural strength: three-point loading | Prisms<br>BS 6319 : Part 3 : 1983<br>For direct comparison with cementitous materials, prisms 40 × 40 × 160 were used instead of prisms 25 × 25 × 100† | Prisms 40 × 40 × 160<br>EN 196 : Part 1 | Flexural modulus: three-point loading | Prisms 40 × 40 × 160.<br>Similar to BS 6319 : Part 3 | Prisms 40 × 40 × 160.<br>Similar to EN 196 : Part 1 |
| Flexural strength: four-point loading | Beams 25 wide × 12·5 deep × 200 long | Beams 25 wide × 12·5 deep × 200 long | Flexural modulus: four-point loading* | Beams 25 wide × 12·5 deep × 200 long | Beams 25 wide × 12·5 deep × 200 long |
| Flexural strength: four-point loading | Beams 25 × 25 × 320 long<br>BS 6319 : Part 3 : 1989 (draft for comment) | Beams 25 × 25 × 320 long<br>BS 6319 : Part 3 : 1989 (draft for comment) | Flexural modulus: four-point loading* | Beams 25 × 25 × 320 long<br>BS 6319 : Part 3 : 1989 (draft for comment) | Beams 25 × 25 × 320 long<br>BS 6319 : Part 3 : 1989 (draft for comment) |

*Secant modulus values were determined at 80% of failure strength.
†Use of larger prisms with the same aspect ratios is permitted under the terms of this standard.

A variety of international test methods also exist to establish most properties. Table 3.8 shows seven test methods for determining drying shrinkage. However, using different sized specimens and different curing conditions will yield different final results. Hence a manufacturer has the opportunity to legitimately select the test method that will produce the lowest shrinkage value for any repair material. Any method attempting to predict the development of stresses in a repair patch will require an accurate (absolute) prediction of the amount of shrinkage that will occur in the repair material. Therefore, it is necessary to specify a recommended test method, so that a standard datum for specimen size and curing conditions can be set.

**Table 3.8 Test methods to establish drying shrinkage[64]**

| Specification or Standard | Conditions | Prism Dimensions (mm) | Limits |
|---|---|---|---|
| Proposed Eurostandard | 20°C, 65% RH | 40 x 40 x 160 | Not yet established |
| Hong Kong Housing Authority (HKHA) | 27°C, 55%RH | 25 x 25 x 285 | 300 microstrain -7 days |
| Australia AS1012 Pt.13 - 1970 | 23°C, 50%RH | 75 x 75 x 285 | |
| USA ASTM C157 - 1989 | 23°C, 50%RH | 25 x 25 x 285 | 500 microstrain -28 days* |
| Germany DIN 52450 - 1985 | Various - 20°C, 65%RH<br>- 23°C, 50%RH<br>- 20°C, 45%RH<br>- 20°C >95%RH<br>- 20°C, Wet | 40 x 40 x 160 | |
| UK BS1881 Pt. 5 - 1970 | This standard does not relate to drying shrinkage therefore test conditions are not included. | 75 x 75 x 150 - 300 | |
| Netherlands CUR 21 | 7 days 20°C >95% RH<br>21 days 20°C, 65%RH | 40 x 40 x 160 | 12 x10$^{-4}$ |

## *3.3 Determining the key properties of repair materials*

It has been established that the current standards for the specification of reinforced concrete repair materials do not take into account the mismatch in basic material properties such as elastic modulus, shrinkage and creep[55]. It is generally recognized that the restraint provided by the substrate concrete (and the steel reinforcement) to the free shrinkage of the repair patch can cause tensile cracking. There are no recommendations in current standards pertaining to the optimal relationship between repair material and substrate concrete properties.

Figure 3.6 demonstrates the strains which develop in a repair material that is restrained. The restraint to the free shrinkage causes tensile stresses to develop.

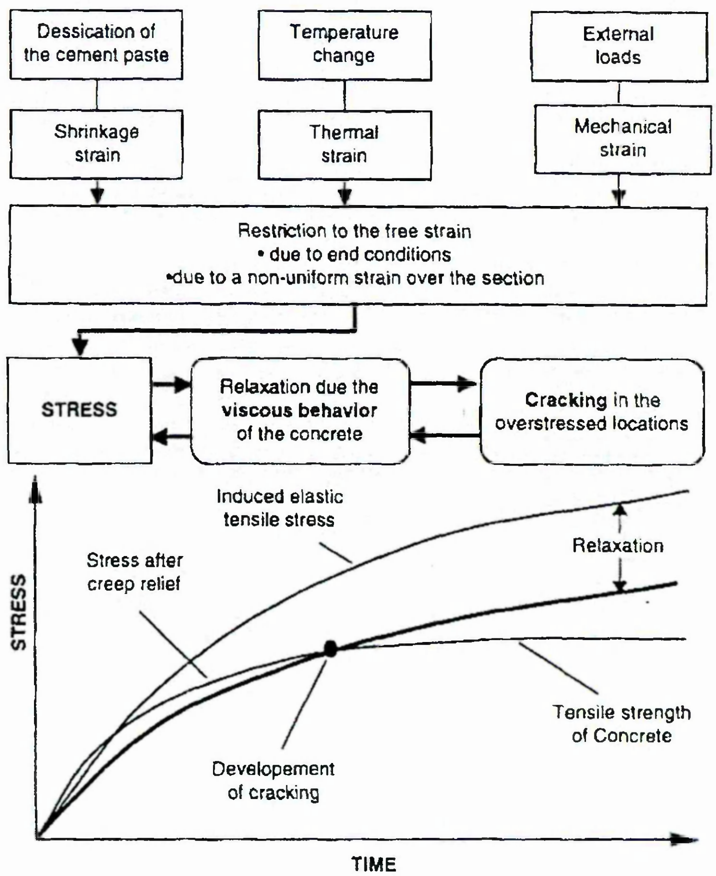



**Figure 3.6 Schematic illustration of stress build up in repairs**[72]

In Figure 3.6, the thick black line represents the stress in the repair material after relaxation of the tensile stresses has occurred. The relaxation occurs through tensile creep. If the tensile stress in the repair material exceeds its tensile strength, then cracking occurs. A patch repair provided for aesthetic improvement is deemed to have failed due to this cracking since assessment codes preclude the inclusion of any steel it encases being used in assessment calculations. If the failed material was applied to reinstate the structural capacity of a member, it will be unable to share any load and consequently has failed in this purpose. It can be seen that avoiding the excessive development of tensile strain can enable a repair material to perform adequately. Therefore, a technique will be developed which utilises the proven phenomenon of shrinkage strain transfer through optimum modular ratio specification. The shrinkage strain of the repair material can be partially transferred to the substrate concrete with appropriate selection of relative $E_{rm}$ and $E_{sub}$ thereby reducing the risk of shrinkage cracking[62] ($E_{rm}$ is the elastic modulus of the repair material at time t days and $E_{sub}$ is the elastic modulus of the repair material at time *t* days). Data was obtained from a field study on the performance of reinforced concrete repair materials; subsequent examination of this data demonstrated that an optimum modular ratio of $E_{rm} \geq 1.32E_{sub}$ will ensure a high level of free-shrinkage transfer from the repair material to the substrate concrete[55]. The specification of suitable creep and shrinkage characteristics will also ensure satisfactory long-term redistribution of service load from the substrate to the repair patch[62].

Field tests were carried out[58] to determine, at daily intervals, the strains developed in both the substrate concrete and the repair material, directly following the application of a repair patch. A summary of these measurements is shown in Table 3.9, for four different repair materials.

**Table 3.9 Strains developed in repair material and substrate[58]**

| Material | Location | Strain at end of: microstrain* | | | |
|---|---|---|---|---|---|
| | | Zone 1 (week 11) | Zone 2 (week 25) | Zone 3 (week 47) | Zone 4 (week 60) |
| L2 | 'subs' | +120 | −120 | +300 | −300 |
| | 'steel' 'emb' | +7 | −7 | +54 | −54 |
| L3 | 'subs' | +107 | −107 | +137 | +137 |
| | 'steel' 'emb' | +45 | −45 | −108 | −108 |
| L4 | 'subs' | −154 | +154 | −297 | +297 |
| | 'steel' 'emb' | −42 | +42 | −142 | +142 |
| G1 | 'subs' | −92 | +92 | −183 | +183 |
| | 'steel' 'emb' | −9 | 9 | −4 | 4 |

* Negative values indicate tensile strains.

In Table 3.9 'subs' represents strain gauges located at the interface of the substrate concrete and the repair patch; 'steel' and 'emb' represent strain gauges attached to the steel reinforcement and embedded in the repair material respectively.

Strains that developed in the substrate concrete after application of the repair material were compared to the free shrinkage properties of the repair material (Table 3.10). In this way it was possible to establish the percentage of the free shrinkage strain of the repair material which was transferred into the substrate concrete.

**Table 3.10 Percentage of free shrinkage transferred into substrate concrete[58]**

| Repair material | $m$ ($E_{rm}/E_{sub}$) | $\varepsilon_{sub,shr}$: microstrain | $\varepsilon_{rm,free}$: microstrain | $\lambda$: % |
|---|---|---|---|---|
| L2 | 1·27 | 120 | 136 | 88 |
| L3 | 1·15 | 107 | 210 | 51 |
| L4 | 1·22 | 154 | 238 | 65 |
| G1 | 1·10 | 92 | 329 | 28 |

Figure 3.7 shows that there is a clear relationship between the modular ratio and the amount of shrinkage strain transferred from the repair material to the substrate concrete. It

can be seen that at a modular ratio of $E_{rm} = 1.32E_{sub}$, all the shrinkage strain of the repair material is transferred to the substrate at the interface of the repair patch.

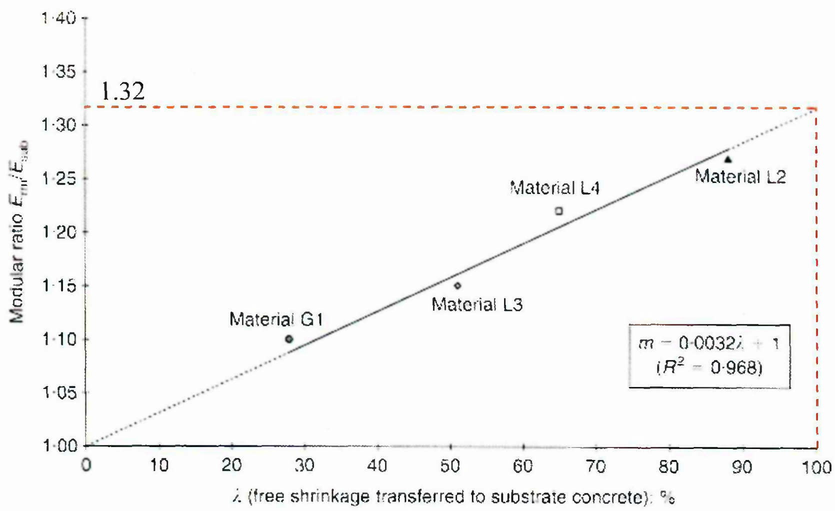



**Figure 3.7 Relationship between modular ratio and free shrinkage transfer**[58]

The graph establishes the relationship:

$$m = 0.0032\lambda + 1$$

where m = the modular ratio

λ = percentage of free shrinkage strain transferred from repair to substrate concrete at the interface.

This relationship can be used to determine the amount of free shrinkage which will be transferred from any repair material into any substrate. It will be used to develop a model for the prediction of in-situ performance of concrete repairs.

The process of transfer of tensile strain from a repair material to the substrate is explained clearly with the aid of Figure 3.8, Figure 3.9 and Figure 3.10. Accompanying these figures is a key that indicates the magnitude of stresses through colour changes. Figure 3.8 represents a repair material freshly applied to substrate concrete. The green equidistant lines represent the concrete over which they lay, similarly the red equidistant lines represent the repair material. In Figure 3.9 the elastic modulus of the repair material is less than that of the substrate concrete. As the free shrinkage of the repair material occurs, it is restrained by the stiffer substrate, this is seen in the figure after the repair has been in place for 28 days. The effect of the restrained shrinkage is shown exaggerated. The repair material away from the restraint is allowed to contract freely but the repair material adjacent to the substrate is severely restrained. This restraint causes tensile strains (virtual), if these tensile strains exceed the tensile strain capacity of the repair material, it will fail (crack). The tensile strain (virtual) in the repair patch reduces as distance from the restraint (substrate) increases. At a certain distance away from the substrate, the effect of the restraint has no influence, and the material exhibits its natural tensionless free shrinkage strain. The distance over which the tensile strains caused by the restraint to shrinkage exert an influence on the repair material is known as the 'zone of influence'. Figure 3.10 demonstrates a repair situation where the elastic modulus of the repair material is higher than that of the substrate concrete. As the repair material shrinks, some of the shrinkage strain at the repair / substrate interface is transferred into the substrate by the stiffer repair material. The effect of this strain transfer is shown exaggerated. Instead of (as in Figure 3.9) the repair material having to withstand the whole tensile (shrinkage) strains, some strain is transferred to the substrate. The sharing of shrinkage strain leads to lower tension in the repair material at the substrate and compression in the substrate. The region of the substrate concrete in Figure 3.10 that is affected by the transfer of shrinkage strain from the stiffer ($E_{rm} > E_{sub}$) repair material is also known as the 'zone of influence'. Strain transferred from the repair material will cause compressive stresses in the substrate which will be at their highest at the interface, and will gradually reduce as distance from the interface increases until finally strain transfer from the repair material has no effect on the substrate. The depth of the zone of influence is of little importance to this study, as it is the critical tensile strain (virtual), occurring at the restraint interface, that the repair material will be designed to withstand.

This phenomenon of strain transfer can be utilised to design successful repair patches and provide optimum selection of repair materials.

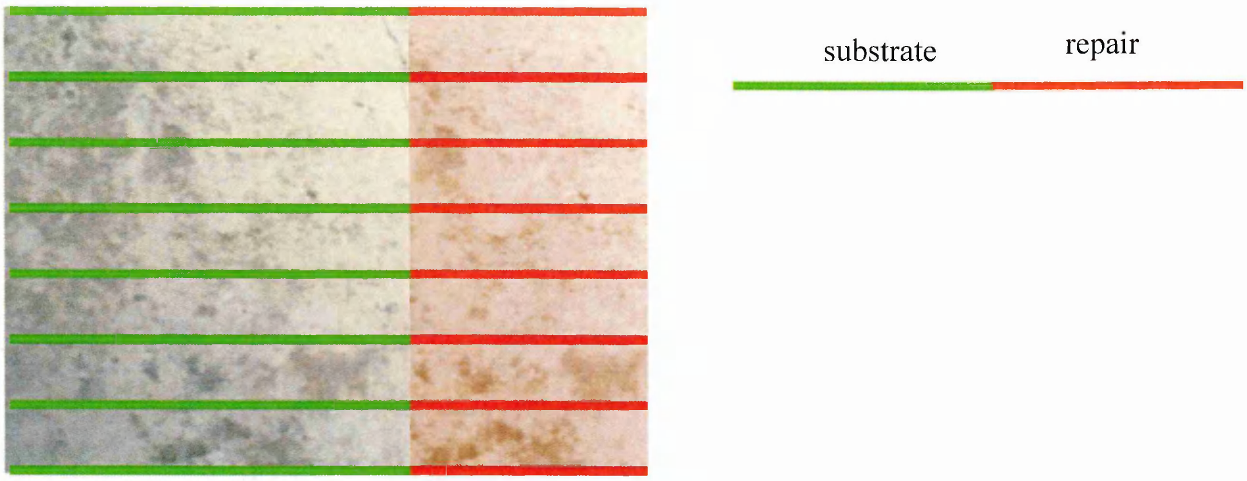
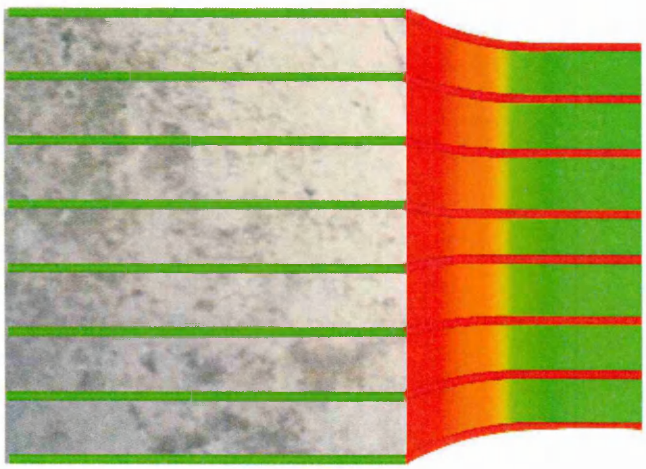



**Figure 3.8 Shrinkage: Substrate and repair material interaction. t = 0 (on application)**



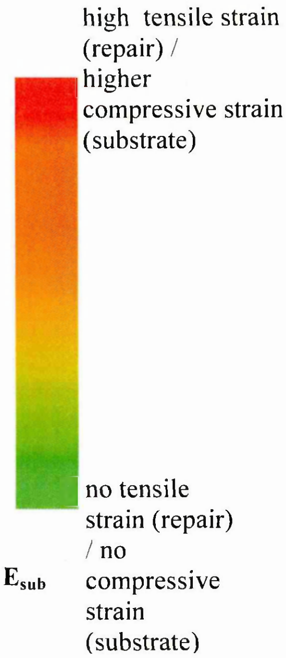



**Figure 3.9 Shrinkage: Substrate and repair material interaction, t = 28 days. $E_{rep} < E_{sub}$**

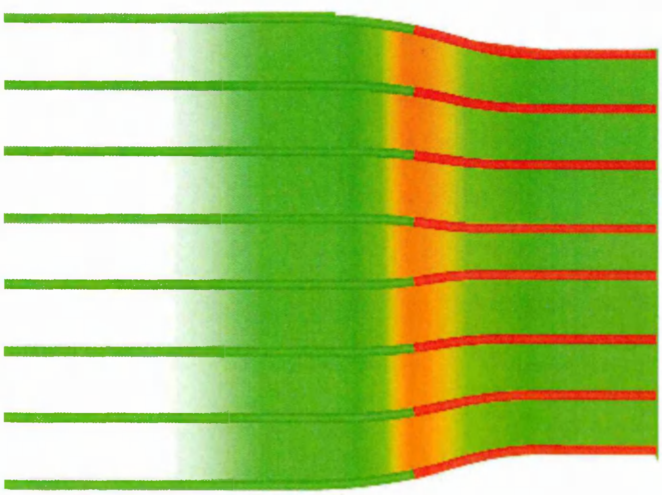


**Figure 3.10 Shrinkage: Substrate and repair material interaction, t = 28 days. $E_{rep} = 1.1\ E_{sub}$**

### 3.4 Development of a method to predict the performance of repair materials in-situ

The current standard for repair material specifications, BD 27/86[66] does not give adequate importance to the necessary marriage of properties between substrate concrete and repair material. Recent research recommends that the key properties for consideration when selecting a repair material are the respective elastic moduli, creep and shrinkage strains of the repair material and substrate concrete. The shrinkage inherent in all repair materials, restrained by the substrate, will attempt to transfer itself to the substrate concrete at the interface. If the stiffness of the substrate is greater than that of the repair material, this transfer cannot take place and the shrinkage may exhibit itself as tensile cracking of the repair material. If the stiffness of the repair material is greater than that of the existing substrate concrete some of the shrinkage may be transferred. An additional factor for consideration is creep. Generally, when a patch repair is applied, the substrate concrete in service has already undergone most of the total creep it will endure in its lifetime. Clearly this is not the case for the repair material and any creep occurring would reduce the effect of restrained shrinkage. An added complication, however, is the fact that creep affects stiffness. High creep can effectively reduce the stiffness of the repair material.

Mangat and O'Flaherty[58] suggest an optimum modular ratio (the ratio of elastic modulus of repair material to substrate concrete) ranging between 1.2 and 1.4 depending on the values of the other key characteristics. These values are based on field data which included the cumulative effect of creep and shrinkage of a range of commercial repair materials.

Knowledge of the properties of both the substrate and the repair material in a concrete repair situation enables the design of a method by which the in-situ performance of a repair can be determined. A software tool is developed in the thesis, in which a database of repair materials is queried by a software routine in order to find all the repair materials which would be successful in a certain repair situation. The properties required to optimise the selection of repair materials are given in Table 3.11. Throughout, a satisfactory bond between the repair and substrate is assumed.

**Table 3.11 Key properties for the optimisation of repair material selection.**

| At 28 days | | Substrate | Repair |
|---|---|---|---|
| Compressive strength | $N/mm^2$ | ✓ | ✓ |
| Tensile Strength | $N/mm^2$ | - | ✓ |
| Shrinkage | microstrain | - | ✓ |
| Creep strain | microstrain | - | ✓ |
| Stress/strength ratio | - | - | ✓ |
| Elastic Modulus | GPa | ✓ | ✓ |

### 3.4.1 Determination of properties

The procedures developed in this thesis will standardise the material properties used in design to a common datum representing different test methods. The two test standards that this procedure adopts and accommodates are the British Standard and the ASTM tests for materials which are widely accepted in the UK. European Standards may be accommodated in the future. Different test specifications recommended by these standards (BS and ASTM) can yield varying values for some properties.

### 3.4.1.1 Compressive strength and Elastic Modulus

In order to determine the compressive strength of insitu concrete, a core is taken in accordance with ASTM C42 – 90[86]. The diameter and height of cores are measured and after conducting the specified tests, correction factors are applied to relate the compressive strength to a datum diameter/height ratio.

BS 1881-116:1983[87] is the recommended British standard for determining the compressive strength of a repair material. 100mm or 150mm cubes are subjected to an increasing load at a rate of between 0.2 N/(mm$^2$.s) and 0.4 N/(mm$^2$.s). The maximum load is divided by the cross sectional area of the cube and the resulting compressive strength, $f_{cu}$, is expressed to the nearest 0.5 N/mm$^2$. ASTM C 39-94[88] is the equivalent standard from the USA. This test is conducted on concrete cylinders. The cylindrical samples are loaded to failure at a rate of between 0.14 N/(mm$^2$.s) and 0.34 N/(mm$^2$.s). A length/diameter correction factor for cylinders is applied as part of the test method to relate the strength to a datum height/diameter ratio.

The conversion relationship for ASTM compressive strength to the BS compressive strength is given by [82,87,88] :-

$$f_{cy} = f_{cu} * 0.8 \qquad \textbf{Eq. 3-1}$$

Where $f_{cy}$ = cylinder strength of concrete specimen

$f_{cu}$ = cube strength of concrete specimen

BS 1881-121 1983[89] is the recommended standard for determining the Elastic Modulus of a repair material or substrate concrete. ASTM C 496-94[90] and ASTM C 580-93[91] are acceptable equivalent tests which require no modification to relate their output values with the British Standard.

Occasionally, a supplier will not provide a value for the elastic modulus of the repair material and it is not practical to demand this information from suppliers. Conversions, therefore, are needed to be performed to estimate the elastic modulus based on other basic inputs (e.g. strength). The following expression can be used for this purpose[82]:

$$E_{c28} = 4.73 * f'_{c28}{}^{0.5} \qquad \text{Eq. 3-2}$$

Where $f'_{c28}$ = 28 day compressive strength of standard test cylinders in MPa

$E_{c28}$ = 28 day Elastic Modulus in GPa

It should be noted that Eq. 3-2 utilises the cylinder strength of a core to determine the Elastic Modulus, the equation is strictly valid for concrete but has been assumed for repair materials. This cylinder strength should be corrected to allow for length/diameter ratio before being used in the equation. For the purposes of the rest of the procedure described below this value for cylinder strength requires a conversion to cube strength (Eq. 3-1)

#### 3.4.1.2 Tensile Strength

Any of the following standards are acceptable for the determination of flexural strength (or modulus of rupture) of repair materials:

C 560-93 Standard test method for flexural strength and modulus of elasticity of chemical-resistant mortars, grouts, monolithic surfacings and polymer concrete[91]; C 78-94 Standard test method for flexural strength of concrete using simple beam with third point loading[92]; C 293-94 Standard test method for flexural strength of concrete using simple beam with centre point loading[93]; BS 1881-118: 1983 Method for determination of flexural strength. (third point loading)[94]. All these methods use conversion factors to return corrected values of tensile strength thus negating any differences in the test results which may be caused by the different test methods themselves.

### 3.4.1.3 Shrinkage and creep

The surface to volume ratio of the insitu repair to be undertaken is required, as is the surface to volume ratio of the specimen of repair material which will be used to establish the *free shrinkage* of the material at *28 days.*

Although Table 3.8 shows many international standards for the determination of shrinkage, few of these are accepted in general practice in the UK. The standards readily accepted are:

- ASTM C 531 - 95[95]
- ASTM C 157 - 93[96]

The standard method for conducting creep tests on concrete in the United States is

- ASTM C 512

This method is primarily for conventional concretes, though is can be adapted for use with cementituous based repair materials by reducing the size of the cylindrical specimen from 150 x 300mm to 100 x 200mm[97]. Creep testing should be conducted under similar environmental conditions to Shrinkage testing. In order to fully define the creep properties, the stress/strength ratio under which the testing was performed should be given.

### 3.4.2 Development of repair material properties

The performance of a repair with time is governed by the development of its material properties with time. A procedure is outlined for establishing the relationship of a range of properties with time. This procedure involves properties selected in Table 3.11, which were identified as crucial to the overall performance of a concrete repair in section 3.2. A manufacturer typically provides limited information about a repair material, often giving the 28 day values for a number of properties. Using these values solely, it would not be possible to design a repair patch for the worst case scenario which could apply to the critical combination of properties at an unknown age. Hence it is necessary to be able to establish the properties of the repair material at any age (i.e. define property-time relationships).

The repair material manufacturers may provide limited data, typically giving the elastic modulus after 28 days of curing and similar data for compressive and tensile strength and also shrinkage. Creep data is rarely provided. In order to use this limited information to predict the early-age and long-term performance of the repair material, it is necessary to extrapolate this basic data to provide the value of key properties at any age. To achieve this, generic property versus age relationships are established.

The approach used is to examine the development with time of these key properties in generic repair materials and to relate the value of a property at any time *t* with the 28 day value. The resulting ratio of a property at time *t* to the value at *28* days provides a relatively accurate relationship that is true for a wide variety of repair materials. The aim of this chapter is to verify that such unique relationships can be achieved for the key properties (e.g. Elastic Modulus, shrinkage, creep) for a variety of generic repair materials. The relationships can hence be utilised in an algorithm to predict the magnitude of tensile strain in a repair material at any time and compare this with the tensile strain capacity of the material to ascertain the likelihood of cracking of a repair patch.

O'Flaherty[58] determined experimentally various properties of a number of repair materials with age. The repair materials were tested in the laboratory and the key properties (Elastic Modulus, shrinkage, creep and strength) were measured at set intervals. All information of thirteen materials was collated for the derivations reported in this thesis. The original numbering system of the materials has been maintained, where G (Gunthorpe), L (Lawns Lane) and S (Sutherland Street) are the initials of the bridges on which the materials were used in patch repairs. The manufacturers' data sheets provided the 28 day values of some key properties of these materials which are given in Table 3.12.

**Table 3.12 The 28 day strength, elastic modulus and tensile strength of the thirteen generic repair materials.**

| Material | Constituents | $f_{cu}$ (N/mm²) | $E_{rep}$ (kN/mm²) | $f_t$ (N/mm²) |
|---|---|---|---|---|
| G1 | Polymer modified; limestone aggregate; dust suppressant; RH Portland cement; 5mm aggregate Microsilica and copolymer | 60 | 31.1 | 4.2 |
| G2 | RH Portland cement Microsilica; Fibres; Chloride free admixtures; Spray dried styrene acrylic copolymer | 56.6 | 17.6 | 2.5 |
| G4 | Styrene acrylic copolymer; Admixtures; Portland cement; Fibres; 6mm aggregate | 50 | 24 | 6..2 |
| G5 | Spray dried styrene acrylic copolymer; Portland cement; Sulphoaluminate | 50 | 19.6 | 4.2 |

| Material | Constituents | $f_{cu}$ (N/mm²) | $E_{rep}$ (kN/mm²) | $f_t$ (N/mm²) |
|---|---|---|---|---|
| | cement; Microsilica; Fibres and other pozzolanic material | | | |
| G6 | Microsilica; Styrene acrylic copolymer | 30 | 11.5 | 2.9 |
| L1 | Microsilica; Limestone aggregates Admixtures; 3mm aggregates | 60 | 22.7 | |
| L3 | Shrinkage compensated Portland cement; Graded aggregates; Special fillers; Chemical additives | 28 | 27.4 | 3.9 |
| L4 | Portland cement; Silica sand Admixtures including plastic fibres | 40 | 29.1 | 2.8 |
| L5 | Fibres | 80 | 29.1 | 5.3 |
| S1 | Cementituous material; 5mm graded aggregate; 500 kg/m³ cement content | 79 | 24.2 | Not provided |
| S2 | Shrinkage compensated | 60 | 32.2 | " |
| S3 | Microsilica; Shrinkage compensated styrene- acrylic copolymer | 70 | 31.9 | " |
| S4 | 10mm rounded aggregate; PFA Superplasticiser; Polypropylene fibres | 39 | 27.4 | " |

Where $f_{cu}$ = compressive strength, 28 days age

$E_{rep}$ = elastic modulus, 28 days age

$f_t$ = tensile strength, 28 days age

#### 3.4.2.1 Creep-age relationship

The compressive creep data (creep versus time) at a stress/strength ratio of 30% are given in Table 3.13, for the thirteen repair materials. The creep data excluded the instantaneous elastic strain that occurs upon load application.

**Table 3.13 Development of Creep (microstrain) during period under load, at 30% stress/strength.**

| | Day (under load) | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mat. | 0* | 2 | 4 | 6 | 7 | 8 | 12 | 15 | 20 | 21 | 28 | 30 | 40 | 50 | 60 | 70 |
| G1 | 0 | 100 | 150 | 170 | 180 | 190 | 200 | 250 | 270 | 273 | 294 | 300 | 305 | 310 | 310 | 310 |
| G2 | 0 | 90 | 130 | 160 | 175 | 190 | 220 | 240 | 310 | 314 | 342 | 350 | 440 | 590 | 690 | 740 |
| G4 | 0 | 220 | 320 | 350 | 365 | 380 | 460 | 480 | 490 | 497 | 546 | 560 | 620 | 710 | 760 | 810 |
| G5 | 0 | 140 | 620 | 740 | 785 | 830 | 930 | 1020 | 1040 | 1046 | 1088 | 1100 | 1230 | 1280 | 1340 | 1360 |
| G6 | 0 | 490 | 680 | 750 | 795 | 840 | 900 | 990 | 1030 | 1035 | 1070 | 1080 | 1090 | 1130 | 1160 | 1190 |
| L1 | 0 | 220 | 400 | 450 | 470 | 490 | 600 | 620 | 640 | 645 | 680 | 690 | 710 | 760 | 770 | 770 |
| L3 | 0 | 130 | 190 | 210 | 225 | 240 | 320 | 410 | 500 | 510 | 580 | 600 | 700 | 720 | 720 | 720 |
| L4 | 0 | 130 | 230 | 250 | 265 | 280 | 330 | 390 | 420 | 421 | 428 | 430 | 460 | 520 | 520 | 520 |
| L5 | 0 | 10 | 150 | 180 | 210 | 240 | 280 | 340 | 370 | 378 | 434 | 450 | 520 | 520 | 520 | 520 |
| S1 | 0 | 80 | 200 | 230 | 255 | 280 | 300 | 370 | 400 | 401 | 408 | 410 | 420 | 430 | 430 | 430 |
| S2 | 0 | 60 | 80 | 100 | 115 | 130 | 170 | 210 | 240 | 242 | 256 | 260 | 330 | 330 | 340 | 340 |
| S3 | 0 | 130 | 320 | 390 | 445 | 500 | 510 | 520 | 530 | 537 | 586 | 600 | 600 | 610 | 610 | 610 |
| S4 | 0 | 100 | 190 | 220 | 245 | 270 | 280 | 290 | 320 | 326 | 368 | 380 | 400 | 420 | 440 | 440 |

*Corresponds to the time of initial load application, which was 28 days after curing began.

The value of creep at each age can be expressed as a proportion of the 28 day creep value of the material ($C/C_{28}$). For example, considering the data for material *G1* (Table 3.13) and dividing throughout by the 28 day creep value, gives the proportions listed in Table 3.14.

**Table 3.14 C*reep* ($C/C_{28}$) in material *G1* as a ratio of the 28 day creep ($C_{28}$).**

| Days under load | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Ratio of *creep* ($C/C_{28}$) | 0.00 | 0.34 | 0.51 | 0.58 | 0.61 | 0.65 | 0.68 | 0.79 | 0.85 | 0.92 | 0.93 |

| Days under load | 28 | 30 | 40 | 50 | 60 | 70 |
|---|---|---|---|---|---|---|
| Ratio of *creep* ($C/C_{28}$) | 1.00 | 1.02 | 1.04 | 1.05 | 1.05 | 1.05 |

This procedure was completed for each of the thirteen repair materials listed in Table 3.13. The creep ratios ($C/C_{28}$) against age under load are plotted for all the thirteen materials in Figure 3.11. An average relationship (best-fit line) of creep ratio with age under load of all thirteen repair materials is also plotted.

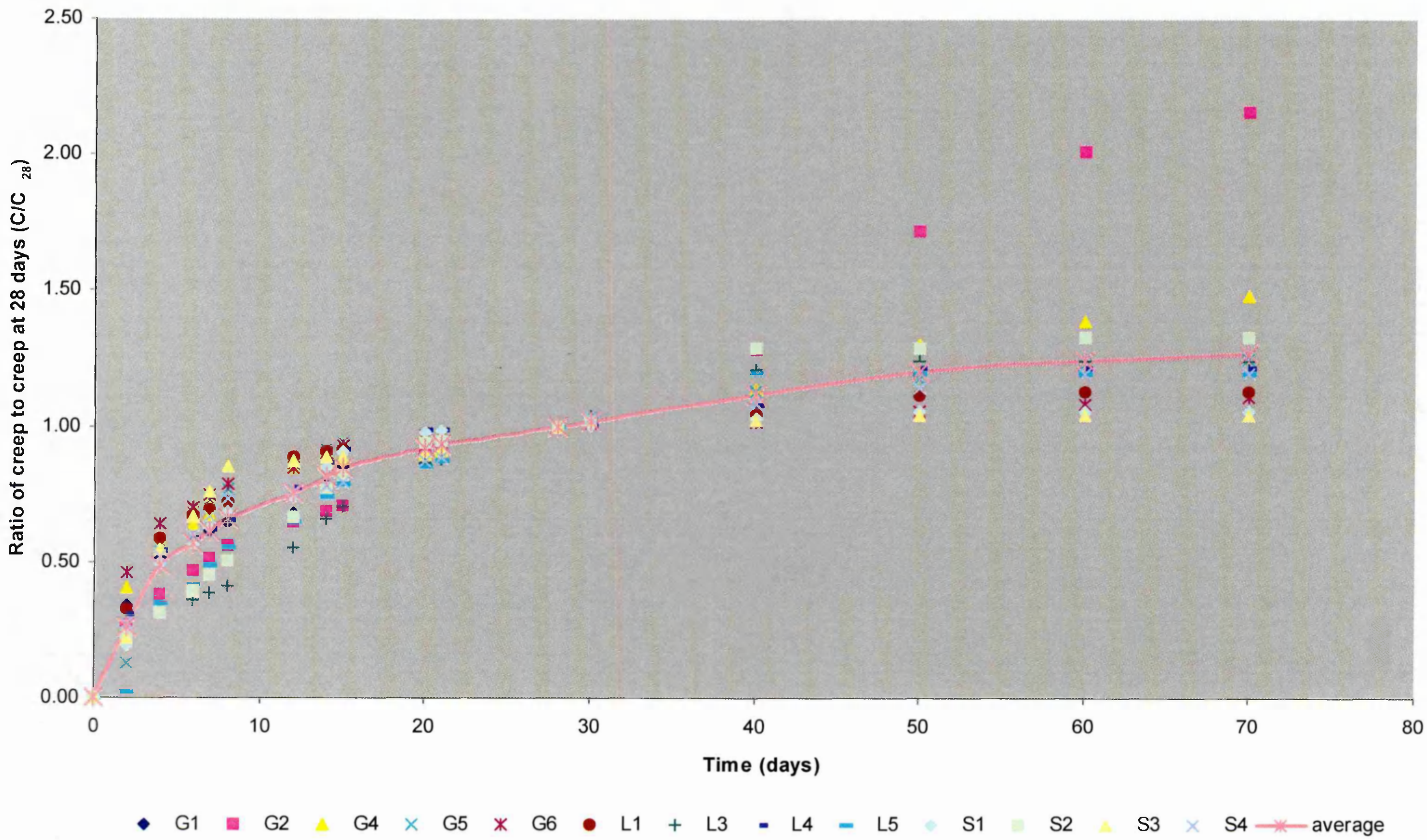



**Figure 3.11 Creep ratio ($C/C_{28}$) versus time under load relationship for thirteen repair materials**

The ability of the best fit line to represent any repair material can be indicated by its correlation coefficient. A correlation coefficient indicates the directness of the relationship between two sets of data x and y. The correlation coefficients of each repair material with the average (best-fit) curve of all thirteen materials are shown in Table 3.15. The table shows high coefficients of correlation exceeding 0.9 for all materials, thereby justifying the assumption that the average creep curve represents each material with a reasonable degree of accuracy.

All materials, except material G2 at later age, show a similar relationship of creep ratio with age throughout the 70 day period plotted in Figure 3.11.

**Table 3.15 Correlation coefficient of average creep ratio curve with creep of each material**

| Material | Coefficient of Correlation (R) | Material | Coefficient of Correlation (R) |
|---|---|---|---|
| G1 | 0.983 | L3 | 0.983 |
| G2 | 0.907 | L4 | 0.907 |
| G4 | 0.984 | L5 | 0.984 |
| G5 | 0.981 | S1 | 0.981 |
| G6 | 0.960 | S2 | 0.960 |
| L1 | 0.979 | S3 | 0.979 |
| | | S4 | 0.972 |

Material G2 shows significantly greater creep ratios than represented by the average curve at ages beyond 30 days of loading. The impact of this long-term underestimation of creep in material G2 by the average relationship for the materials will be explained later in this chapter. However, intuitively, it can be recognised that the best fit line would predict a conservative amount of creep for material G2. In practice, the extra creep which would occur for this material over that predicted by the average relationship, would provide greater relaxation of restrained shrinkage stress, and hence is less worrying than a material developing less creep than that predicted by the best fit line.

The creep versus time under load data represented by the average (best-fit) relationship plotted in Figure 3.11 is listed in Table 3.16.

**Table 3.16 Best fit relationship data of $C/C_{28}$ with time under load.**

| Days under load | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Ratio of creep ($C/C_{28}$) | 0.00 | 0.26 | 0.49 | 0.56 | 0.61 | 0.66 | 0.75 | 0.81 | 0.85 | 0.92 | 0.93 |

| Days under load | 28 | 30 | 40 | 50 | 60 | 70 |
|---|---|---|---|---|---|---|
| Ratio of creep ($C/C_{28}$) | 1.00 | 1.02 | 1.12 | 1.20 | 1.25 | 1.27 |

### 3.4.2.2 Hyperbolic expression for creep/time relationship

Ross[98] and Lorman[99] recommend the use of a hyperbolic expression to describe the relationship between creep and time under load; which is expressed as follows:

$$c = \frac{t}{a + bt}$$ **Eq. 3-3**

Where $t = time$

$c = creep$

$a$ and $b$ are empirical constants which will be determined from the experimental results of the 13 materials used in this study.

Rearranging Eq. 3-3 and multiplying each side by $C_{28}$ gives:

$$\frac{t}{C}C_{28} = (a+bt)C_{28}$$

$$\therefore \frac{t}{C_r} = C_{28}a + C_{28}bt$$

$$\text{where } C_r = \frac{C}{C_{28}}$$ **Eq. 3-4**

$$\frac{t}{C_r} = a' + b't$$

which is the equation of a straight line with a\` and b\` as constants. Therefore, plotting $t/C_r$ against $t$ produces a line whose slope represents $b$\`, and the intercept represents $a$\`. The data listed in Table 3.16 for the average $C/C_{28}$ versus time relationship are plotted according to equation 3.4 in Figure 3.12.

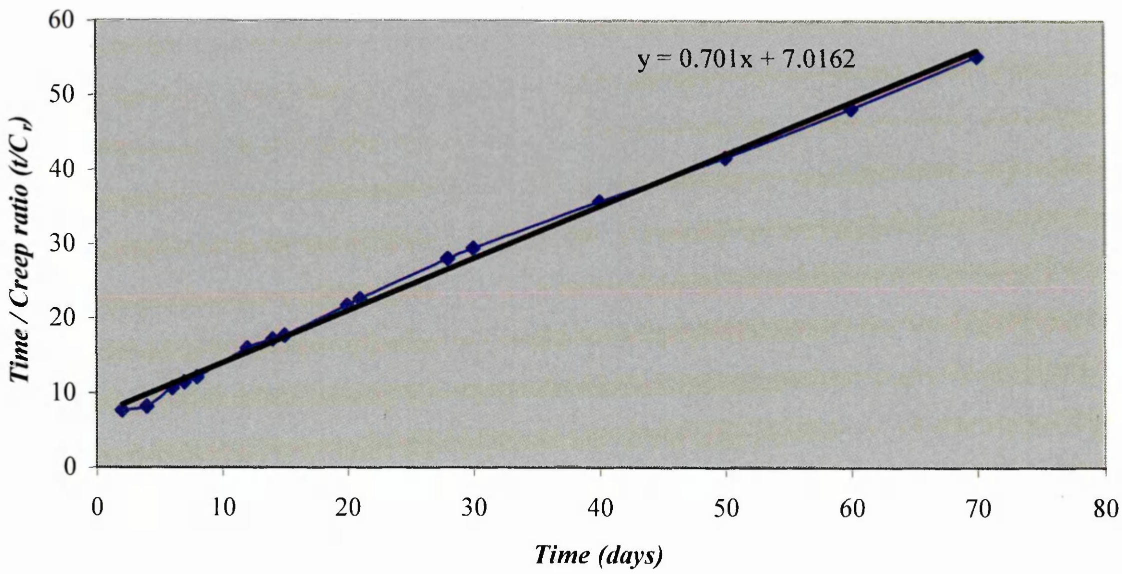



**Figure 3.12 $t/C_r$ versus t relationship for thirteen repair materials ($R^2$ = 0.9964).**

Hence the equation can be written to describe the development of creep ratio with time, as:

$$C_r = \frac{t}{7.0162 + 0.701t} \qquad \textbf{Eq. 3-5}$$

where,

$$C_r = \frac{\text{creep at any age under load}}{\text{creep at 28 days under loading}} = \frac{C}{C_{28}}$$

The hyperbolic relationship determined in Figure 3.12 and represented by equation 3.5 is plotted in Figure 3.13 for an extrapolated long-term period of 400 days. The corresponding experimental data of the thirteen repair materials is also plotted up to 70 days under load.

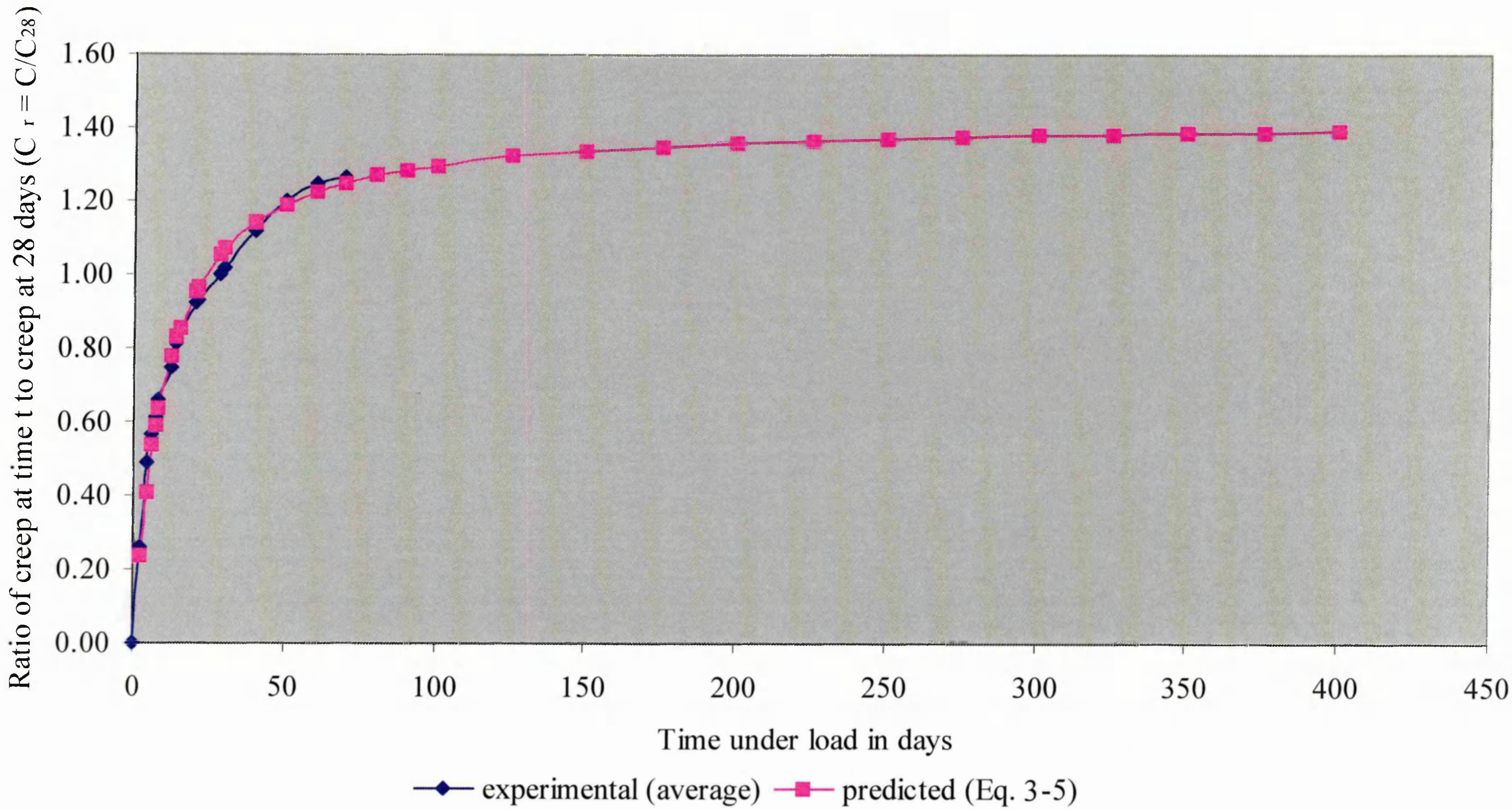



**Figure 3.13 Comparison of average experimental creep ratio ($C_r$) with calculated creep ratio ($C_r$) (Stress/Strength ratio 30%)**

The correlation coefficient between the experimental curve (Figure 3.13), representing the average behaviour of the thirteen materials and the curve based on the predicted values (Eq. 3-5) is R = 0.9944. This close correlation between experimental and predicted value means the correlation coefficients for individual materials between the experimental and predicated creep ratio would be similar to those in Table 3.15.

The expression adequately describes the performance of the average repair material for the purposes of predictive models developed for the design of patch repairs. It allows extrapolation to ages which enables long term performance to be predicted.

#### 3.4.2.3 Shrinkage-time relationship

The shrinkage data of the thirteen repair materials (shrinkage versus time relationship) are given in Table 3.17.

**Table 3.17Development of shrinkage (microstrain) with time for thirteen repair materials**

| | Days after casting | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mat. | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 15 | 20 | 21 | 28 | 30 | 40 | 50 | 60 | 70 |
| G1 | 0 | 20 | 30 | 50 | 55 | 60 | 80 | 93 | 100 | 110 | 114 | 142 | 150 | 200 | 240 | 270 |
| G2 | 0 | 10 | 60 | 80 | 100 | 120 | 180 | 193 | 200 | 260 | 266 | 308 | 320 | 440 | 500 | 560 |
| G4 | 0 | 10 | 40 | 60 | 65 | 70 | 80 | 87 | 90 | 100 | 104 | 132 | 140 | 150 | 160 | 160 |
| G5 | 0 | 100 | 160 | 210 | 225 | 240 | 290 | 297 | 300 | 360 | 368 | 424 | 440 | 470 | 510 | 510 |
| G6 | 0 | 90 | 150 | 200 | 215 | 230 | 280 | 287 | 290 | 350 | 357 | 406 | 420 | 460 | 510 | 540 |
| L1 | 0 | 20 | 60 | 70 | 85 | 100 | 140 | 153 | 160 | 200 | 205 | 240 | 250 | 300 | 320 | 340 |
| L3 | 0 | 60 | 120 | 150 | 165 | 180 | 220 | 253 | 270 | 320 | 323 | 344 | 350 | 400 | 420 | 460 |
| L4 | 0 | 40 | 50 | 70 | 75 | 80 | 100 | 107 | 110 | 140 | 142 | 156 | 160 | 220 | 240 | 260 |
| L5 | 0 | 20 | 40 | 60 | 65 | 70 | 80 | 93 | 100 | 140 | 142 | 156 | 160 | 220 | 240 | 260 |
| S1 | 0 | 30 | 60 | 70 | 85 | 100 | 120 | 133 | 140 | 160 | 164 | 192 | 200 | 230 | 260 | 280 |
| S2 | 0 | 100 | 340 | 350 | 375 | 400 | 430 | 450 | 460 | 500 | 505 | 540 | 550 | 600 | 620 | 620 |
| S3 | 0 | 50 | 60 | 80 | 95 | 110 | 140 | 153 | 160 | 180 | 182 | 196 | 200 | 210 | 240 | 260 |
| S4 | 0 | 20 | 50 | 70 | 85 | 100 | 130 | 143 | 150 | 170 | 172 | 186 | 190 | 200 | 240 | 260 |

The *shrinkage* at each age can be expressed as a proportion of the 28 day shrinkage value of the material ($S/S_{28}$). For example, considering the data for material *G1* (Table 3.17) and dividing throughout by the 28 day shrinkage value, gives the proportions listed in Table 3.18.

**Table 3.18 Shrinkage in material G1 as a ratio of the 28 day shrinkage**

| Days after casting | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Ratio of *shrinkage* ($S/S_{28}$) | 0 | 0.14 | 0.21 | 0.35 | 0.39 | 0.42 | 0.56 | 0.66 | 0.70 | 0.77 | 0.80 |

| Days after casting | 28 | 30 | 40 | 50 | 60 | 70 |
|---|---|---|---|---|---|---|
| Ratio of *shrinkage* ($S/S_{28}$) | 1.00 | 1.06 | 1.41 | 1.69 | 1.90 | 1.97 |

This procedure was completed for each of the thirteen repair materials using the data for each repair material in Table 3.17. The shrinkage ratios ($S/S_{28}$) against age after casting are plotted for all the thirteen materials in Figure 3.14. An average relationship (best-fit line) of shrinkage ratio with age after casting of all thirteen repair materials is also plotted.

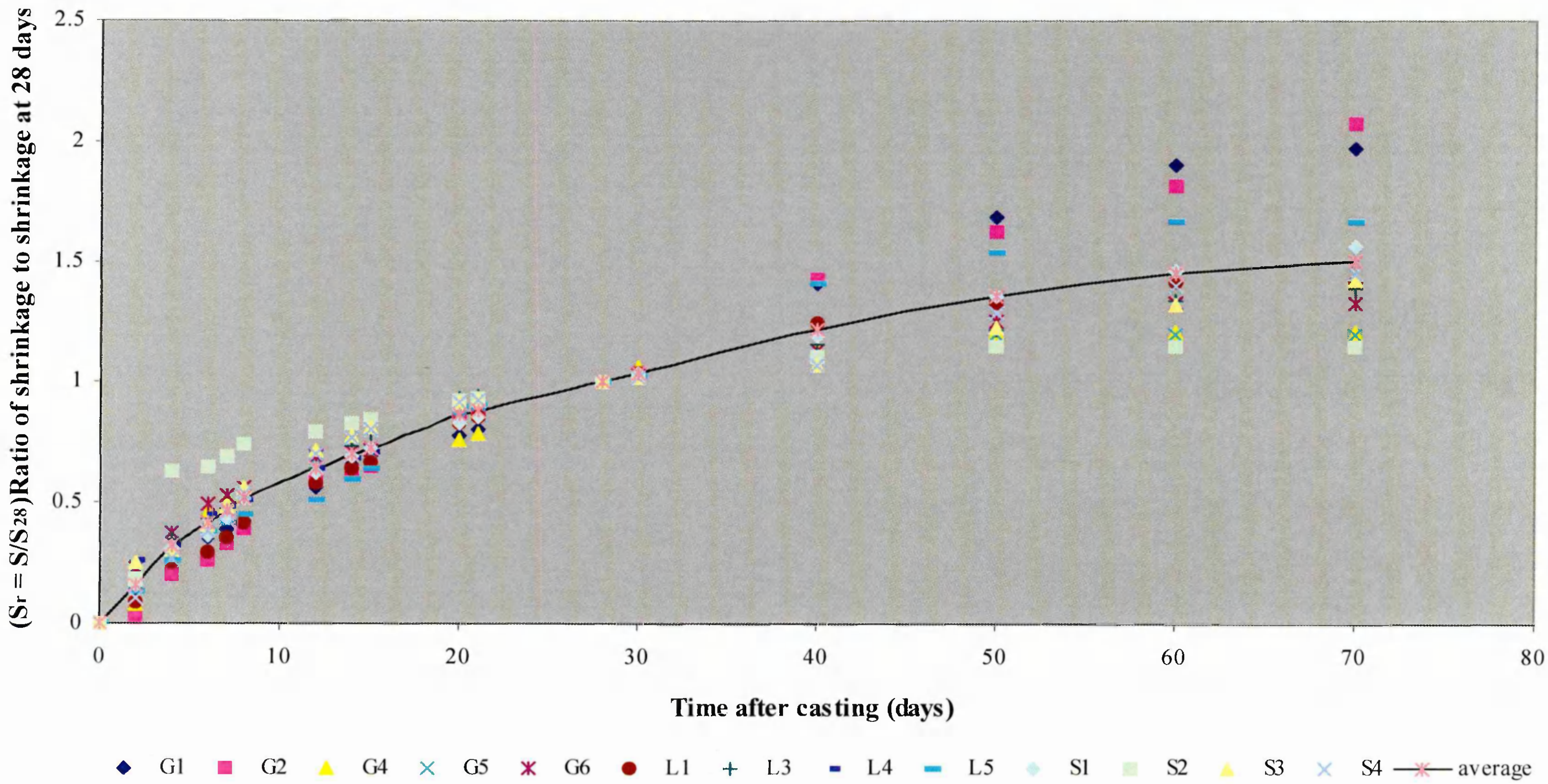



**Figure 3.14 Ratio of shrinkage at each age to 28 day shrinkage ($S/S_{28}$) versus time relationship.**

Table 3.19 lists the coefficient of correlation of the shrinkage data for each material in Figure 3.14 with the average curve of $S/S_{28}$ versus time relationship of the thirteen materials. The very high coefficients of correlation (>0.927) confirm the validity of the data to the average curve.

**Table 3.19 Correlation coefficients for the shrinkage ratio versus time curves of each material with the average relationship.**

| Material | Coefficient of Correlation (R) | Material | Coefficient of Correlation (R) |
|---|---|---|---|
| G1 | 0.980 | L3 | 0.994 |
| G2 | 0.982 | L4 | 0.992 |
| G4 | 0.985 | L5 | 0.991 |
| G5 | 0.985 | S1 | 0.999 |
| G6 | 0.995 | S2 | 0.927 |
| L1 | 0.996 | S3 | 0.989 |
| | | S4 | 0.991 |

It can be seen in Figure 3.14 that the standard deviations of individual materials from the average curve are generally higher at later times than was witnessed for creep (Figure 3.11). Therefore, some materials may shrink more than the average value derived from the shrinkage ratio versus time relationship. It is likely that these materials would also creep more, thus offsetting the discrepancy which would result between predicted and field behaviour - this is discussed further in Chapter 4. One possible design approach is to assume a higher growth of shrinkage ratio with time than the average determined for the thirteen repair materials in Figure 3.14. However, this would be unduly conservative; in the interests of accuracy, the average relationship is accepted as adequately representing the shrinkage behaviour of each of the thirteen materials.

Taking the data for the development of shrinkage for the thirteen repair materials from Figure 3.14, the average value of shrinkage at any time as a ratio of the material's 28 day shrinkage can be derived from the best fit curve (Table 3.20).

**Table 3.20 Best fit relationship data of S/S28 with time after casting.**

| Days after casting | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Ratio of *shrinkage* ($S/S_{28}$) | 0.00 | 0.16 | 0.32 | 0.42 | 0.47 | 0.52 | 0.64 | 0.70 | 0.73 | 0.86 | 0.88 |

| Days after casting | 28 | 30 | 40 | 50 | 60 | 70 |
|---|---|---|---|---|---|---|
| Ratio of *shrinkage* ($S/S_{28}$) | 1.00 | 1.03 | 1.22 | 1.36 | 1.45 | 1.50 |

### 3.4.2.4 Hyperbolic expression for shrinkage/time relationship

Ross[98] and Lorman[99] also recommend the hyperbolic expression to describe the relationship between shrinkage and time; which is expressed as follows:

$$S = \frac{t}{a + bt}$$ **Eq. 3-6**

Where $t = time$

$S = shrinkage$

$a$ and $b$ are empirical constants which will be determined from the experimental results of the 13 materials used in this study.

Rearranging Eq. 3-6 and multiplying throughout by $S_{28}$ gives:

$$\frac{t}{S}S_{28} = (a+bt)S_{28}$$

$$\therefore \frac{t}{S_r} = S_{28}a + S_{28}bt \qquad \textbf{Eq. 3-7}$$

$$\frac{t}{S_r} = a' + b't$$

Plotting time over shrinkage against time produces a line whose slope is *b'*, and the intercept is *a'*. This is done for the average value for the thirteen repair materials (Figure 3.15).

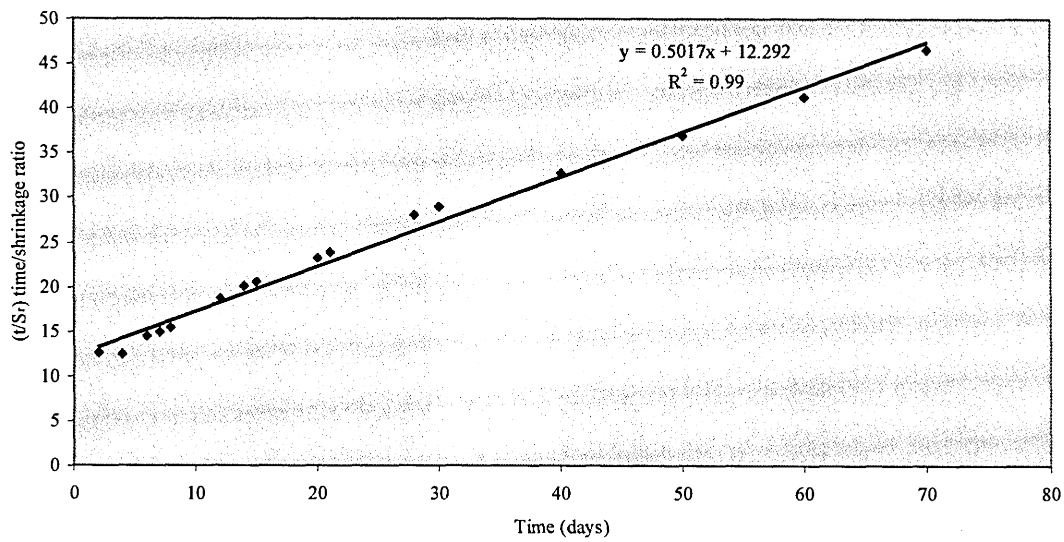



**Figure 3.15 t/$S_r$ versus t relationship for the thirteen repair materials ($R^2 = 0.99$)**

Hence the equation can be written to describe the development of shrinkage ratio with time:

$$S_r = \frac{t}{12.292 + 0.5017t} \qquad \textbf{Eq. 3-8}$$

This expression is plotted for a period of 450 days and also compared with the experimental data in Figure 3.16:

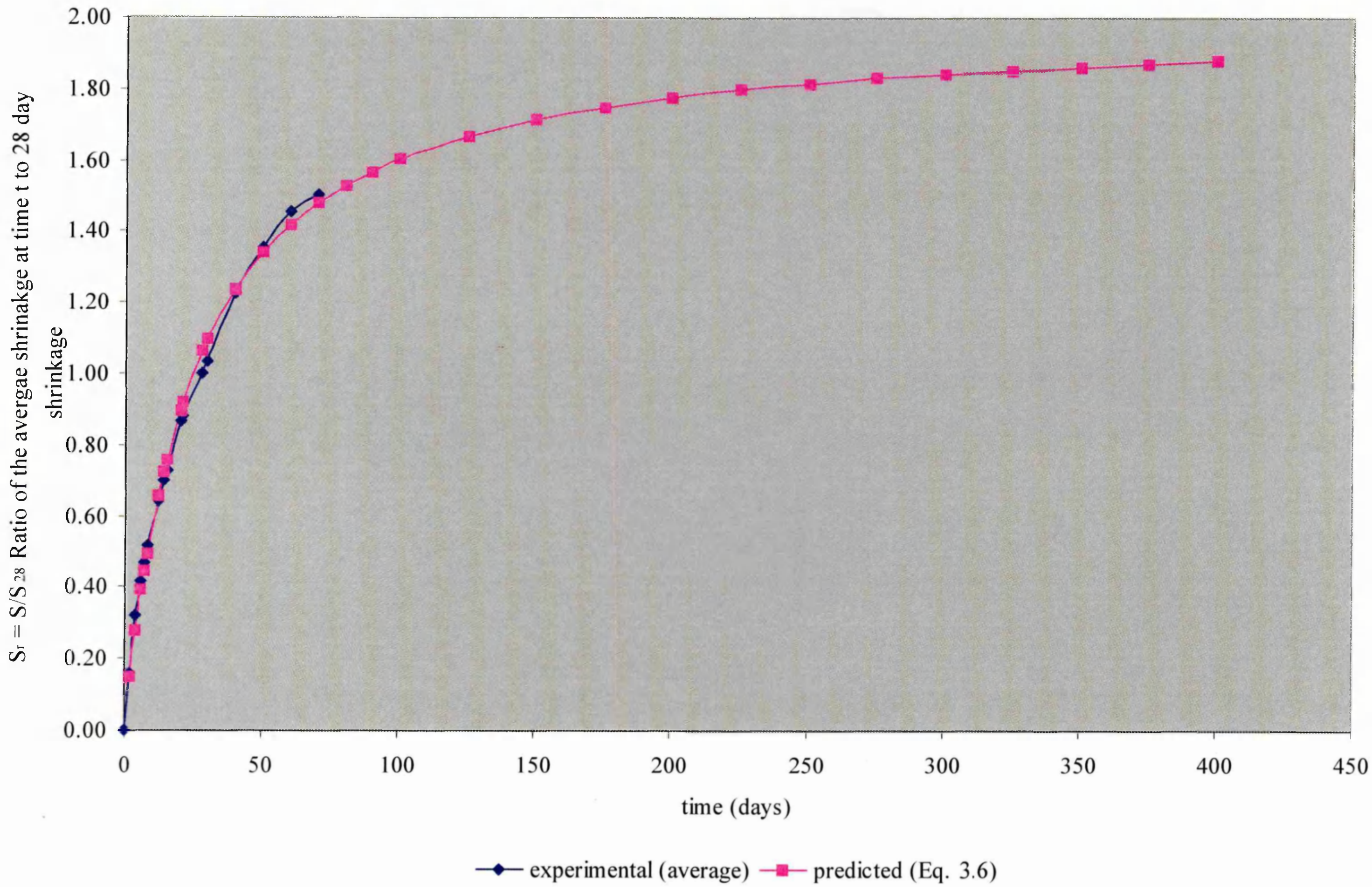



**Figure 3.16 Sr versus time (after casting) relationship**

The correlation coefficient between the experimental curve (representing the average behaviour of the thirteen materials) in Figure 3.16 and the curve based on the predicted values (**Eq. 3-8**) is $R^2 = 0.9965$.

The expression adequately represents the shrinkage-time relationship of the average repair material. It allows extrapolation to later ages to represent long term performance.

### 3.4.2.5 Development of Compressive Strength

Although there is a lack of test data on the compressive strength versus time relationship of repair materials, there is comprehensive data of this relationship for concrete. In the absence of strength-time relationship information for repair materials, it will be assumed that their behaviour will be similar to that of concrete. The extensive data for concrete available in literature[82] are used to derive a general strength time relationship. The compressive strength versus age relationships[82] are plotted in Figure 3.17, in this figure, part (a) shows the long term development of compressive strength for a number of laboratory specimens made up of three different water/cement ratios. Part (b) shows the development of the same ratio for specimens cured under differing atmospheric conditions.

**Table 3.21 Development of Compressive strength with age**

| Age (days) | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 | 28 | 90 | 400 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Compressive strength ratio ($f_c/f_{28}$) | 0 | 0.26 | 0.45 | 0.56 | 0.62 | 0.66 | 0.78 | 0.82 | 0.84 | 0.91 | 0.93 | 1 | 1.11 | 1.11 |

$f_c$ = compressive strength at age t days $f_{28}$ = compressive strength at 28 days age.

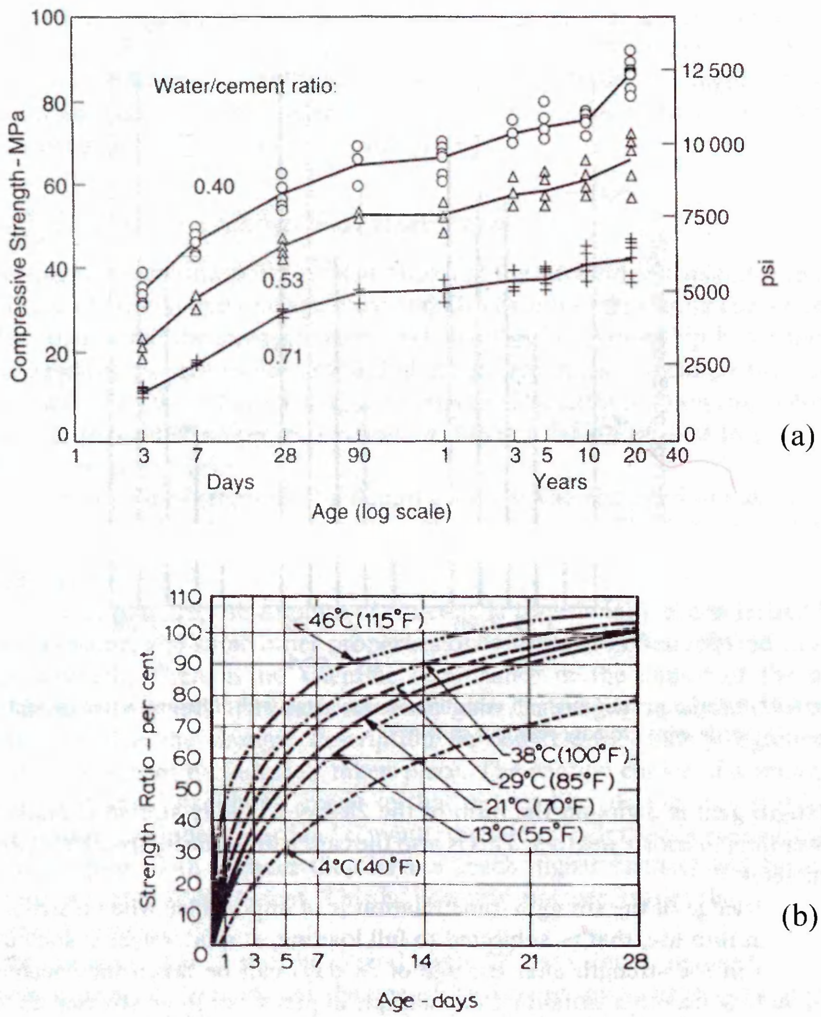



**Figure 3.17 Development of strength of concrete with age[82]**

The data for concrete mixes of water/cement 0.53, cured at 21°C are extracted from Figure 3.17 and the strength ratios ($f_c/f_{28}$) are listed in Table 3.21. In the absence of a known mathematical profile for the strength-time relationship, the hyperbolic expression used for the prediction of creep or shrinkage[98,99] is also applied to the development of compressive strength with time. If the derived hyperbolic expression correlates well with the test data, then its application will be justified.

$$f_c = \frac{t}{a+bt}$$ **Eq. 3-9**

Where $t$ = *time (days)*

$f_c$ = *compressive strength*

*a* and *b* are empirical constants which will be determined from the experimental results of the concrete specimens used in the reference study.

Rearranging Eq. 3-9 and multiplying throughout by $f_{28}$ gives:

$$\frac{t}{f_c} f_{28} = (a + bt) f_{28}$$

$$\therefore \frac{t}{f_{cr}} = f_{28}a + f_{28}bt \qquad \text{Eq. 3-10}$$

$$\frac{t}{f_{cr}} = a' + b't$$

The operation in the previous sections (as applied to shrinkage and creep data) is repeated for the compressive strength data in Table 3.12 and a graph is produced by plotting time over compressive strength against time. This produces a line whose slope represents b', and the intercept represents *a'* (Figure 3.18)

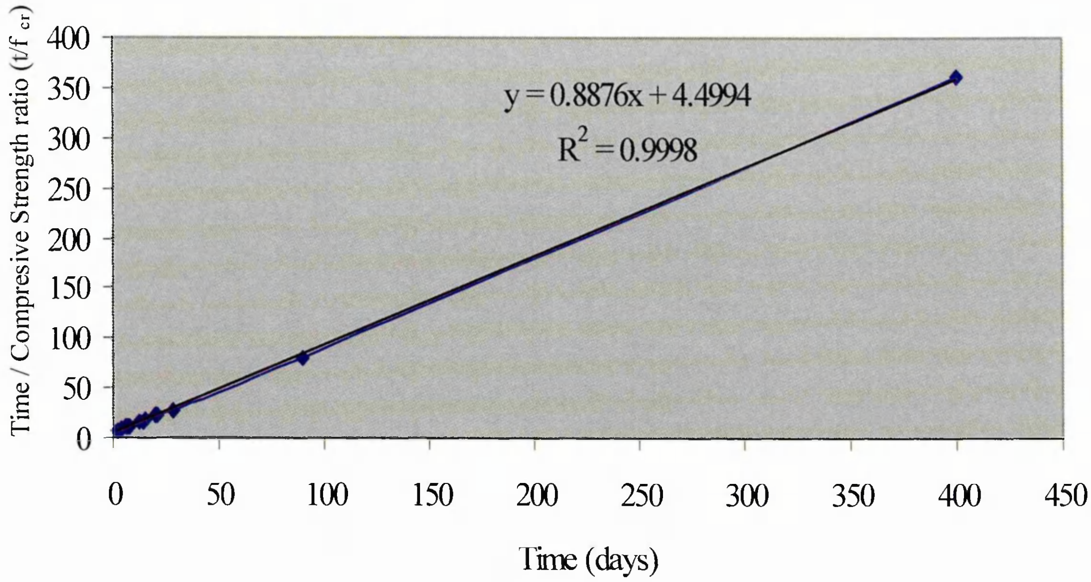



**Figure 3.18 Development of compressive strength ratio with time**

Hence the equation can be written to describe the development of compressive strength ratio with time:

$$f_{cr} = \frac{f_c}{f_{28}} = \frac{t}{4.4994 + 0.8876t} \qquad \textbf{Eq. 3-11}$$

This expression can be compared with the experimental data which are plotted in Figure 3.19 :

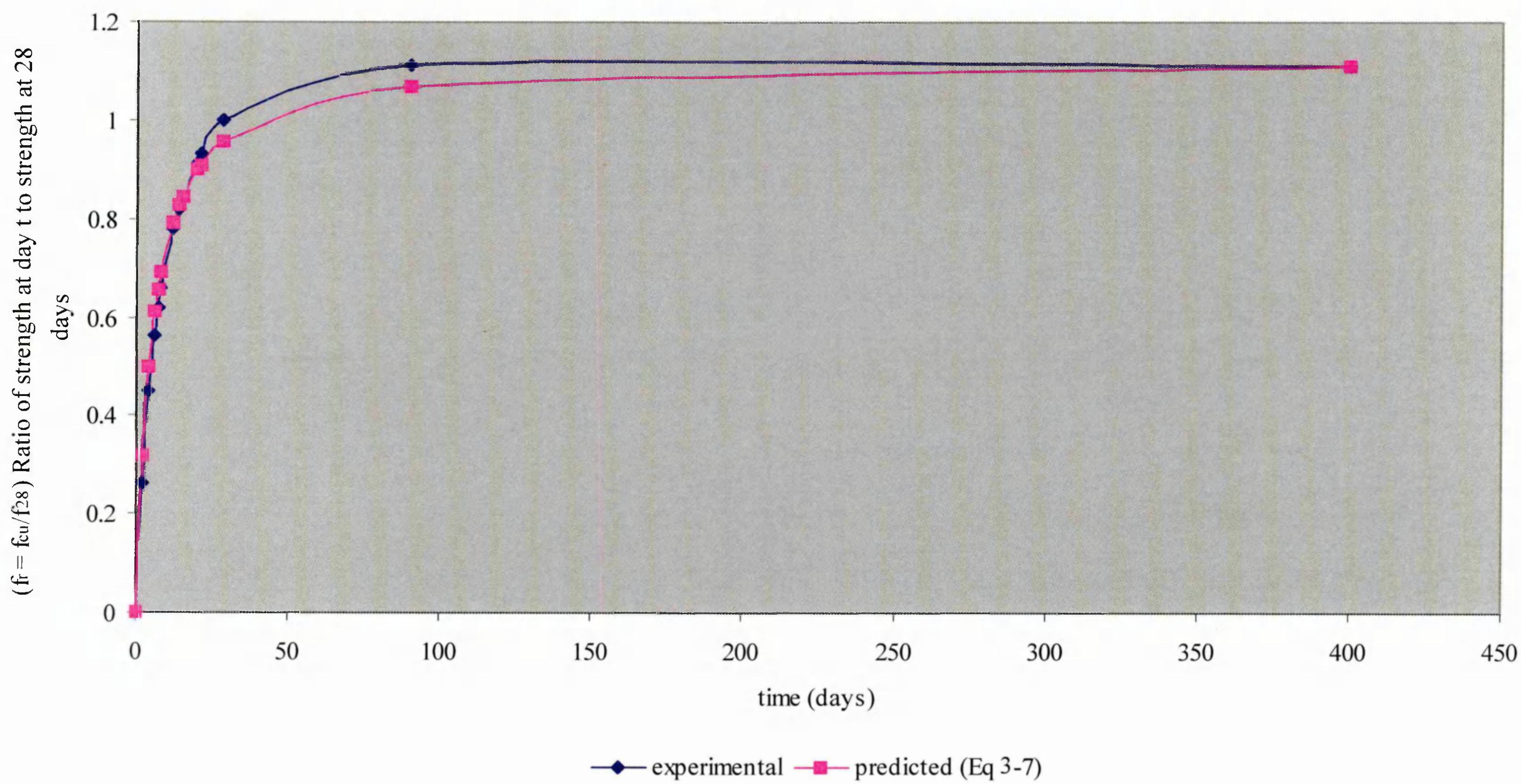



**Figure 3.19 Comparison of measured Compressive strength with predicted Compressive strength**

The coefficient of correlation, $R^2$, between the two curves in Figure 3.19 is 0.9964. This indicates that the predictive equation and the experimental curve are convincingly related. Henceforth, the technique which generates the hyperbolic equation for strength-age relationship is accepted. The technique will be further utilised to predict the development of tensile strength.

#### 3.4.2.6 Development of tensile strength

In order to obtain an expression for the development of tensile strength of concrete with age, the relationship between tensile strength, $f_t$, and compressive strength, $f_c$, is considered[82]:-

$$f_t = 0.12 f_c^{\,0.7} \qquad \textbf{Eq. 3-12}$$

Rearranging Eq. 3-12 gives:

$$f_c = \left[\frac{f_t}{0.12}\right]^{1/0.7} \qquad \textbf{Eq. 3-13}$$

Therefore, considering the 28 day values,

$$f_{28} = \left[\frac{f_{t28}}{0.12}\right]^{1/0.7} \qquad \textbf{Eq. 3-14}$$

Substituting for $f_t$ and $f_{28}$ from Eq. 3-13 and Eq. 3-14 respectively into Eq. 3-11 gives:

$$\frac{\left[\dfrac{f_t}{0.12}\right]^{1/0.7}}{\left[\dfrac{f_{t28}}{0.12}\right]^{1/0.7}} = \frac{t}{4.4994 + 0.8876t} \qquad \textbf{Eq. 3-15}$$

Simplifying,

$$\left(\frac{f_t}{f_{t28}}\right)^{1/0.7} = \frac{t}{4.4994 + 0.8876t} \qquad \textbf{Eq. 3-16}$$

Therefore,

$$\frac{f_t}{f_{t28}} = \left[\frac{t}{4.4994 + 0.8876t}\right]^{0.7} \qquad \textbf{Eq. 3-17}$$

Equation 3-17 is used to generate tensile strength ratio $f_{tr} = \frac{f_t}{f_{t28}}$ values for t = 0 to 400

days.

These are listed in Table 3.22.

**Table 3.22 Development of Tensile strength ratio ($f_t$ / $f_{t28}$) with time**

| t | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 | 28 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $f_{tr} = \frac{f_t}{f_{t28}}$ | 0.00 | 0.44 | 0.63 | 0.73 | 0.76 | 0.79 | 0.87 | 0.90 | 0.91 | 0.95 | 0.95 | 0.98 | 1.02 | 1.05 | 1.06 | 1.07 | 1.07 | 1.07 | 1.08 | 1.08 |

An equation for the development of tensile strength ratio is required in the form:

$$\frac{f_t}{f_{t28}} = \frac{t}{f_{t28}(a + bt)}$$

$$\therefore f_{tr} = \frac{f_t}{f_{t28}} = \frac{t}{a' + b't}$$

A graph of time (t) against time / tensile strength ratio ($t/f_r$), is plotted in Figure 3.20 to determine the constants *a'* and *b'*.

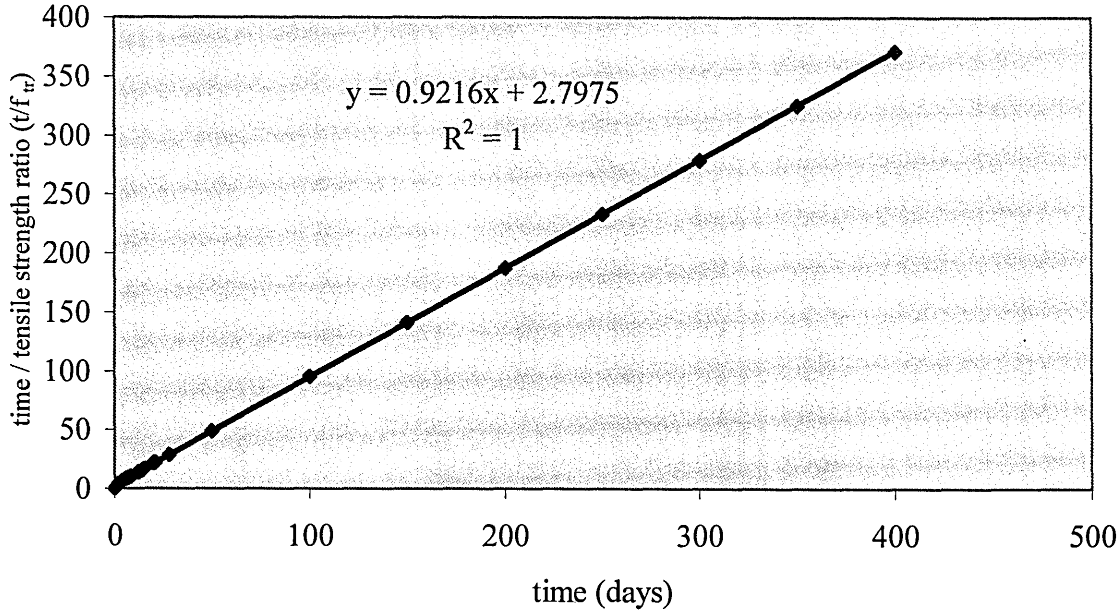



**Figure 3.20 Development of tensile strength ratio with time**

Therefore:

$$\frac{f_t}{f_{t28}} = \frac{t}{2.7975 + 0.9216t} \qquad \textbf{Eq. 3-18}$$

The expression can be compared to the limited experimental data available on the development of tensile strength in repair materials with time[58]. Three materials were tested over a period of 28 days; a styrene acrylic concrete, an SBR concrete and an acrylic concrete. A comparison between the growth of tensile strength as predicted by equation 3.18 and the actual growth of tensile strength in the three repair materials is shown in Figure 3.21.

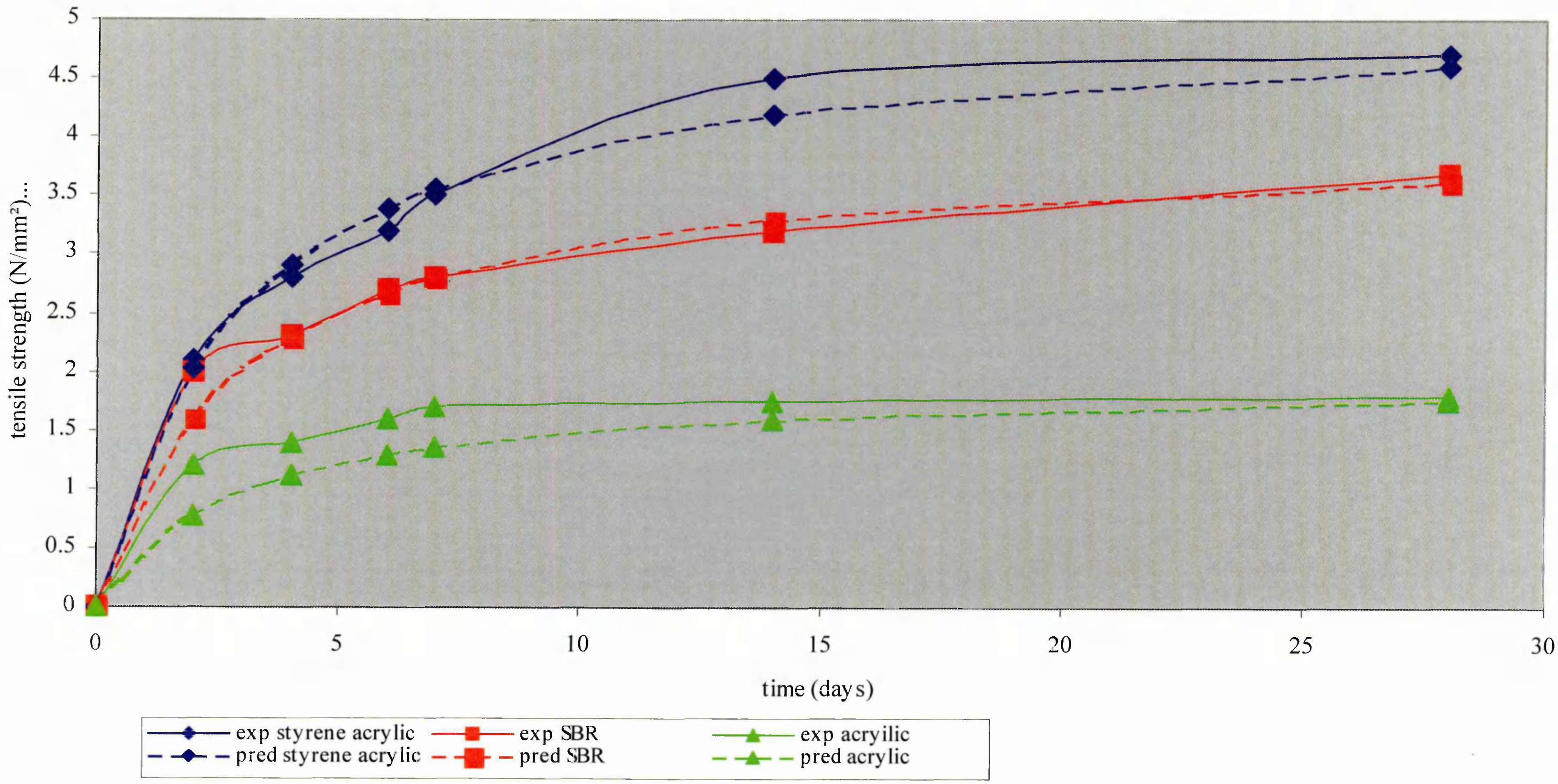



**Figure 3.21 Comparison of predicted and experimental tensile strengths**

The predictive quality of Eq. 3-18 in Figure 3.21 shows good correlation with the experimental results. This is particularly significant, as the predictive approach is based on the results of different experiments to those on which is was tested and is derived from compressive strength relationships of concrete while the experimental tensile strength data plotted in Figure 3.21 is for repair material formulations. The correlation coefficients for the styrene acrylic and SBR concrete are both over 0.99. The coefficient of correlation for the acrylic material is slightly lower at 0.967, as is evidenced from the graph.

#### 3.4.2.7 Development of Elastic Modulus

*Pinelle*[100] provides data on the development of Elastic Modulus from early age in commercial repair materials with varying constituents. Data from three different materials is provided (shown in Table 3.23). The materials consist of an unmodified cementituous repair material, acrylic based material and a vinyl acetate based material.

**Table 3.23 Development of elastic modulus with time in repair materials**

| Time after casting (days) | Cementituous ($kN/mm^2$) | Acrylic ($kN/mm^2$) | Vinyl acetate ($kN/mm^2$) |
|---|---|---|---|
| 0 | 0 | 0 | 0 |
| 7 | 480 | 310 | 205 |
| 14 | 680 | 420 | 250 |
| 21 | 770 | 500 | 270 |
| 28 | 790 | 530 | 273 |
| 35 | 800 | 560 | 276 |
| 42 | 810 | 585 | 279 |
| 56 | 820 | 588 | 282 |
| 63 | 830 | 591 | 285 |
| 70 | 840 | 594 | 288 |
| 77 | 850 | 597 | 291 |
| 84 | 860 | 600 | 294 |
| 91 | 870 | 603 | 297 |
| 98 | 880 | 606 | 300 |

The value of elastic modulus at each age can be expressed as a proportion of the 28 day elastic modulus of the material ($E/E_{28}$). For example, considering the data for the Cementituous repair material (Table 3.23) and dividing throughout by the 28 day elastic modulus, gives the proportions listed in Table 3.24.

**Table 3.24 Elastic modulus of cementituous repair material as a ratio of the 28 day elastic modulus**

| Days after casting | 0 | 7 | 14 | 21 | 28 | 35 | 42 | 56 | 63 | 70 | 77 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Ratio of elastic modulus ($E/E_{28}$) | 0 | 0.65 | 0.86 | 0.97 | 1.0 | 1.03 | 1.05 | 1.06 | 1.07 | 1.08 | 1.09 |

| Days after casting | 84 | 91 | 98 |
|---|---|---|---|
| Ratio of elastic modulus ($E/E_{28}$) | 1.10 | 1.11 | 1.12 |

This procedure was completed for each of the three repair materials. The elastic modulus ratios ($E/E_{28}$) against age after casting are plotted for all three materials in Figure 3.22. An average relationship (best-fit line) of all three materials is also plotted.

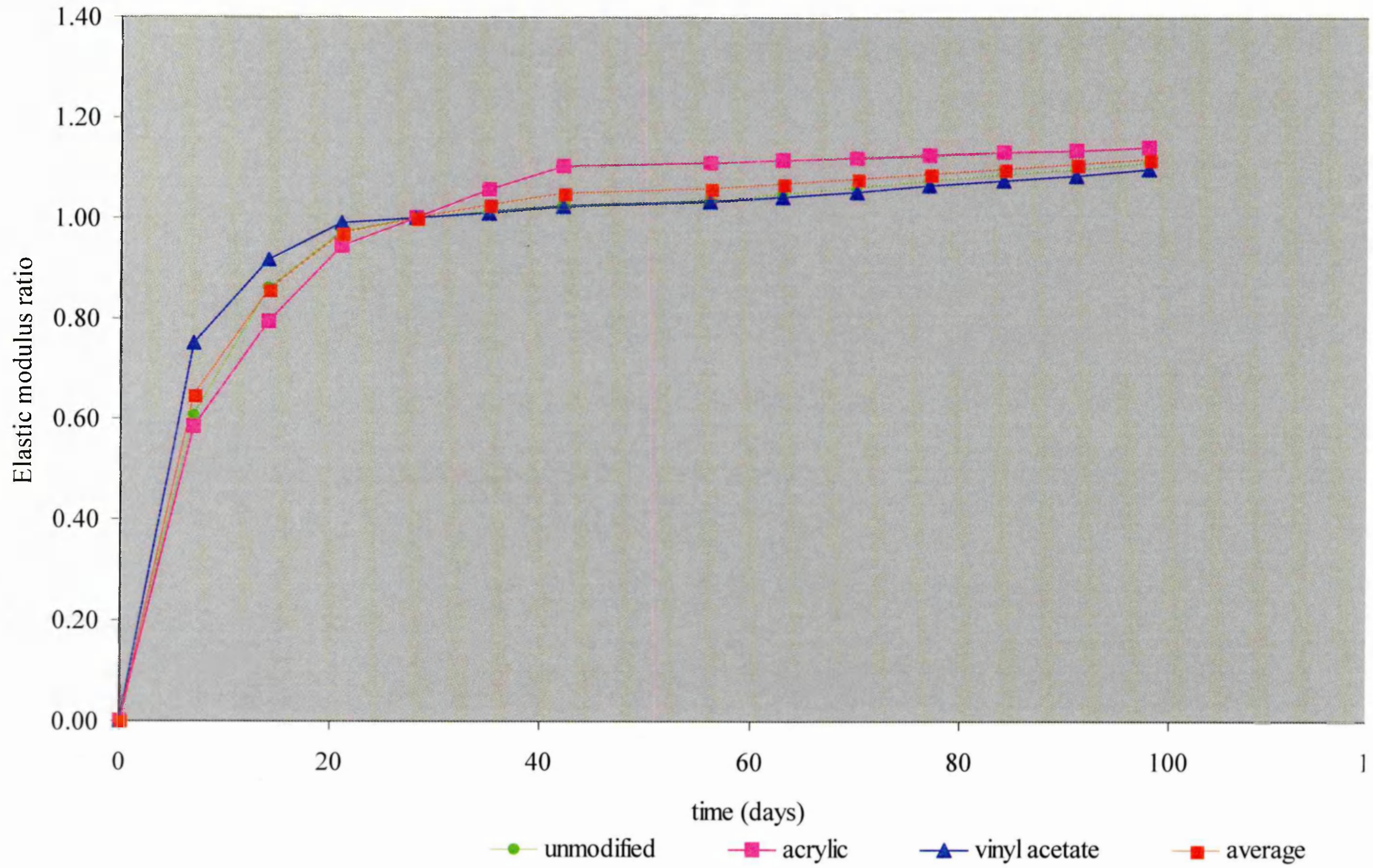



**Figure 3.22 Relationship between elastic modulus ratio ($E/E_{28}$) and time for the three repair materials**

The line representing the average relationship between elastic modulus ratio and age (also shown in Table 3.25), correlates well with the test data for the three materials (coefficient of correlation $R^2$ = 0.999, 0.993 and 0.992 respectively). This means that the average relationship can be applied to any repair material to a reasonable degree of accuracy.

**Table 3.25 Average ($E/E_{28}$) ratio versus time relationship for the repair materials**

| Age (days) | 0 | 7 | 14 | 21 | 28 | 35 | 42 | 56 | 63 | 70 | 84 | 98 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Average Ratio of Elastic moduli $E/E_{28}$ | 0.00 | 0.65 | 0.86 | 0.97 | 1.00 | 1.03 | 1.05 | 1.06 | 1.07 | 1.08 | 1.10 | 1.12 |

This is used to determine a general expression for the development of Elastic Modulus ratio using the following hyperbolic relationship of the type used previously for the other properties .

$$\frac{E}{E_{28}} = \frac{t}{E_{28}(a+bt)}$$
$$\therefore E_r = \frac{t}{a'+b't} \qquad \textbf{Eq. 3-19}$$

By plotting time/elastic modulus ratio against time the constants *a'* and *b'* can be found in equation 3-19.

Figure 3.23 shows good correlation and gives the following relationship between the average elastic modulus and age.

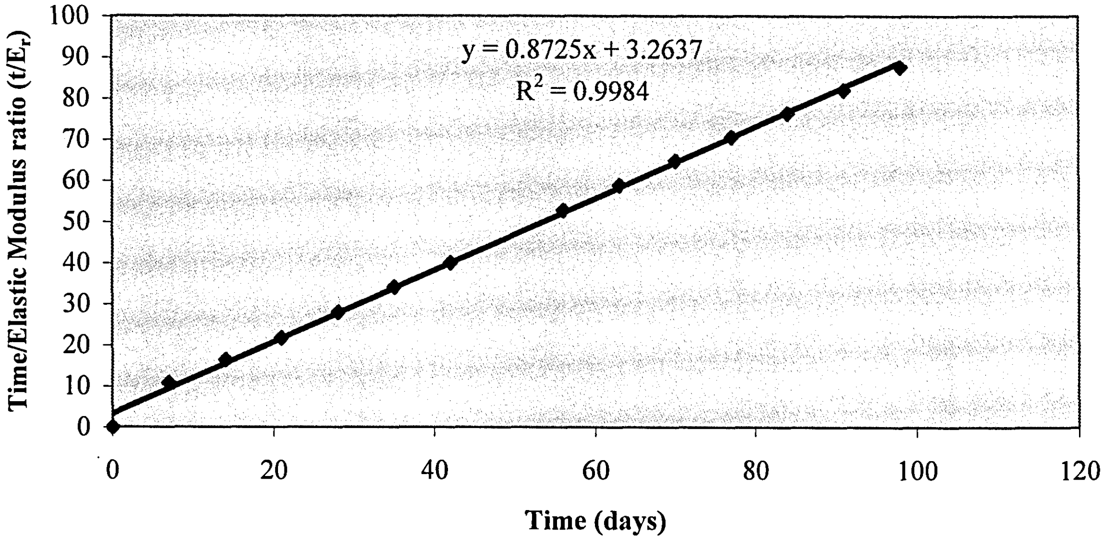



**Figure 3.23 Development of elastic modulus ratio ($E_r$) with time based on the average of 3 repair materials**

$$E_r = \frac{E}{E_{28}} = \frac{t}{3.2637 + 0.8725t} \qquad \textbf{Eq. 3-20}$$

The elastic modulus ratio as predicted by equation 3-20 is compared to the actual experimental data corresponding to the average curve for the three repair materials in Figure 3.24.

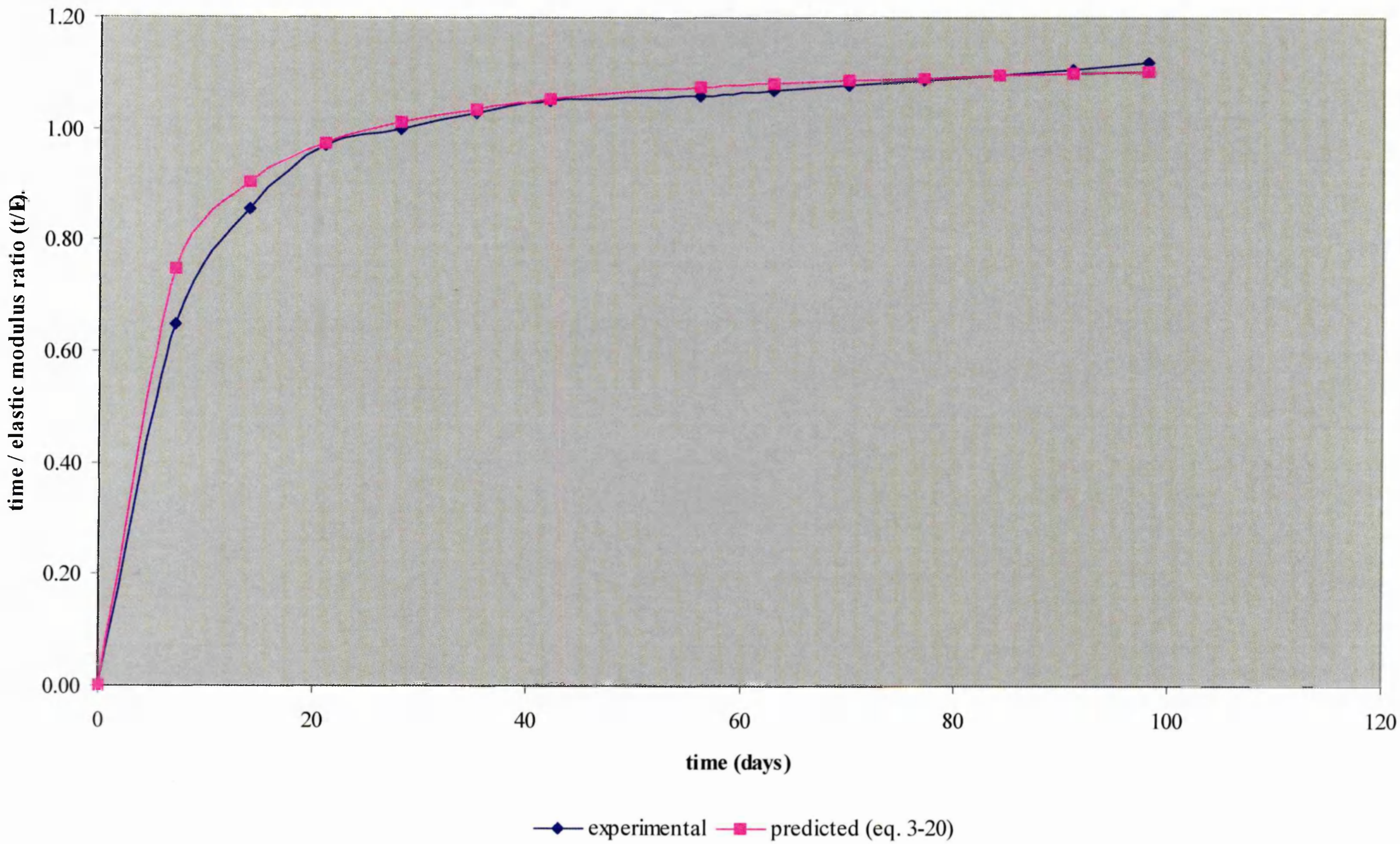



**Figure 3.24 Graph comparing the predicted and experimental curves of elastic modulus ratio versus time relationship**

The coefficient of correlation $R^2 = 0.996$. The accuracy of the predictive equation can be shown by plotting the experimental data and the predicted growth of elastic modulus based on the 28 day value (Figure 3.25) for one of the repair materials.

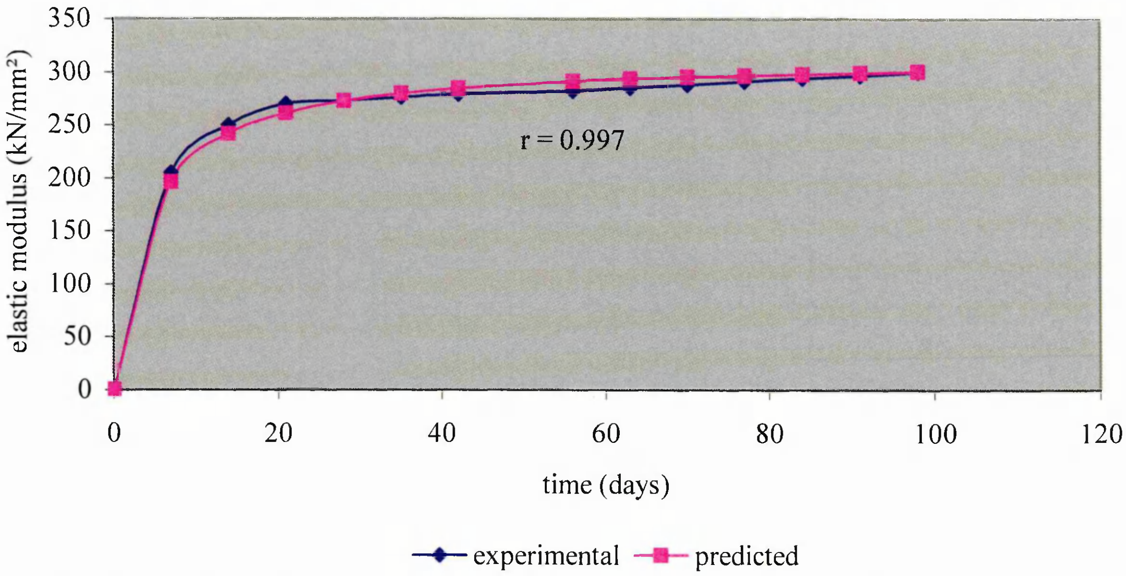



**Figure 3.25 Comparison of experimental and predicted values of elastic modulus for vinyl acetate material**

Figure 3.25 shows that if only the 28 day elastic modulus of the vinyl acetate material had been provided, then using the predictive hyperbolic equation, the elastic modulus of the material at any age can be predicted with a good degree of accuracy.

### *3.4.2.7.1 Tensile and compressive elastic moduli*

The elastic modulus of concrete is usually determined under compression. However, repair materials are subjected to tensile stress as their inherent free shrinkage is restrained, mainly at their interface, with the substrate concrete. It is assumed, for the purposes of the procedure developed in this thesis, that the elastic modulus of a repair material in tension and compression is equal. The reasons for this assumption are discussed in 3.2.3.2.1.

# 4 The procedure for determining the in-situ performance of repair materials.

## 4.1 Chapter objective

The objective of this chapter is to develop a procedure for determining the in-situ performance of concrete repair. The equations which predict the development of material properties with time, derived in the previous chapter, will be utilised. The procedure will be developed for implementation in a computer program.

## 4.2 Introduction

In Chapter 3, the properties of repair materials which are crucial to the successful performance of the repair are identified and the development of those properties with age is described by generic relationships derived empirically using a hyperbolic function. These equations will be used in this chapter to determine critical tensile strains developed in a repair patch. This critical strain will then be compared with the tensile strain capacity of the repair material in order to establish the likelihood of failure (cracking) of the repair, and, should a repair have been deemed to fail, to also suggest the likely time of failure (in days) after the application. The time of failure can range between the short term, typically within the first 50 days of application, to the longer term of 400 to 500 days. The developed algorithms are incorporated into the computer expert system to provide an expedient method for determining the performance of a patch repair.

## 4.3 Procedure for determining the performance of a repair material

Consider the repair material *Shucrete 1* which has the following properties at 28 days age (Table 4.1). *Shucrete 1* is an imaginary material whose properties are generally typical of a repair material :

**Table 4.1 The properties of *Shucrete 1* at 28 days age.**

| | | |
|---|---|---|
| Compressive strength | N/mm$^2$ | 30 |
| Modulus of Rupture | N/mm$^2$ | 8.5 |
| Tensile Strength | N/mm$^2$ | 5 |
| Shrinkage | microstrain | 630 |
| Creep (at 30% stress/strength) | microstrain | 794 |
| Elastic Modulus | kN/mm$^2$ | 34 |

These properties represent the information typically required from repair material manufacturers. These properties are usually provided by manufacturers, with the exception of *Creep* which, due to a lack of coherent understanding amongst specialists concerning which properties of a repair material are crucial to its overall performance, is currently not considered important and hence rarely specified by manufacturers.

### 4.3.1 Determining Tensile Strength from modulus of rupture

It is usual for the Modulus of rupture to be provided in manufacturers' literature. This property can be converted to an approximate tensile strength using a conversion factor[82].

Tensile strength = Modulus of rupture / 1.7

For Shucrete 1:

Modulus of rupture = 8.5 MPa

Therefore, Tensile strength = 8.5 / 1.7 = 5 MPa

### 4.3.2 Modifications for climate

Shrinkage and creep are affected by climatic conditions, specifically temperature and relative humidity. Considering 70% relative humidity as a datum, shrinkage increases by approximately 2% for each per cent decrease in relative humidity and decreases by approximately 3% for each per cent increase in relative humidity[101].

Shrinkage can be assumed to increase or decrease by 1% of the 15 °C shrinkage value with each relative increase or decrease in temperature[101].

It can be assumed that creep changes linearly with temperature at a rate of 1.25% of the 15°C creep for every degree change in temperature[101].

Nawy[102] gives a chart to determine the effects of relative humidity on creep of concrete, which is shown in Figure 4.1.

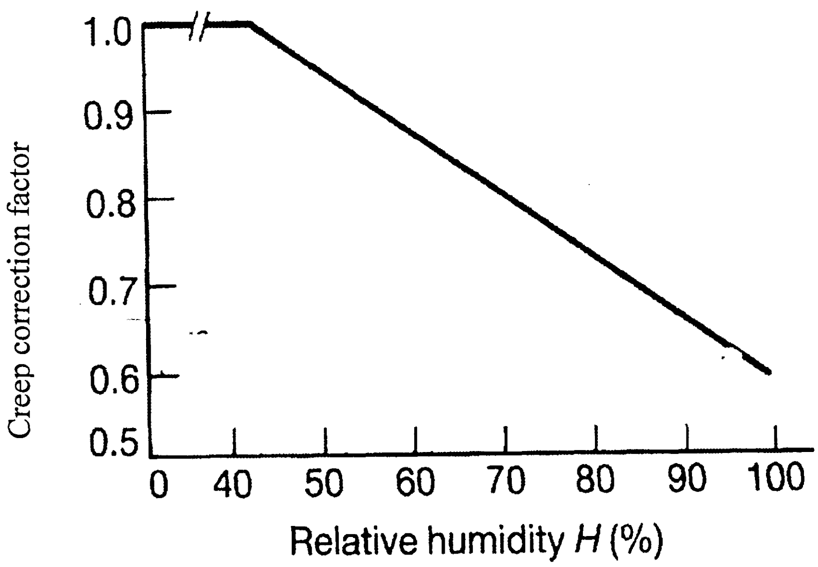



**Figure 4.1 Effect of relative humidity on creep[102]**

The equation specifying the relationship in Figure 4.1, within the limits of relative humidity 42.9% and 100%, has been determined as:

$$k_{rh} = -0.007(RH\%) + 1.3 \quad \textbf{Eq. 4-1}$$

Where $k_{rh}$ = creep correction factor for relative humidity

*4.3.2.1.1 Strength and elastic modulus modified by climate*

Temperature during curing is known to have an influence on the development of compressive and tensile strength[82].

Figure 4.2 gives the 28 day compressive strength of the same concrete cast at different times of the year in the UK. Generally, the summer months can expect a reduction of between 5% and 10% of the strength compared with cooler months. The temperature during the crucial first days of curing is deemed to be responsible for this effect.

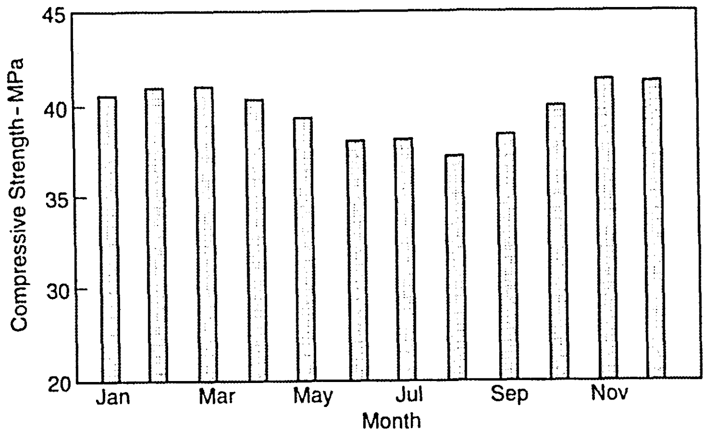



**Figure 4.2 Influence of initial temperature on average monthly compressive strength in the UK**[82]

Little data is available on the development of strength of concrete or repair materials at different temperatures out of doors. Most research is based on specimens cured at constant temperature in the laboratory. It is, therefore, difficult to recommend a correction factor for strength based on seasonal temperature, or indeed relative humidity. If it is assumed that any seasonal reduction in strength is also accompanied by a corresponding reduction in elastic modulus then any seasonal effects on tensile strain capacity of a repair may be negated. It is, therefore, assumed that seasonal effects on strength can be neglected.

### 4.3.3 Establishing seasonal temperature and RH variations

The procedure developed in the thesis for predicting tensile strain in a repair material (repair patch) will take account of the effect of seasonal and geographical temperature and relative humidity variations. As previously stated, these variations affect the development of creep and shrinkage with time.

Mean temperatures and relative humidities can be determined to a satisfactory degree of accuracy seasonally[103]. In order to do this, it is necessary to be able to evaluate temperature and relative humidity in different geographical regions at different times of the year. Table 4.2, Table 4.3 and Figure 4.3 represent an expedient method for determining local temperatures and relative humidities in the UK[103]. Great Britain is divided into three zones: north, mid, and south. For each zone average temperatures and relative humidities are given. These figures can be used to determine the conditions in which repair materials will cure in the field. Manufacturers' data, which is based on standard specified curing conditions, can then be modified to account for the effects of temperature and relative humidity at the location and the period that materials are used in patch repairs.

**Table 4.2 Seasonal average temperature variation in the UK[103]**

| | Dec - Feb | Mar - May | Jun - Aug | Sep - Nov |
|---|---|---|---|---|
| North | 4° | 9° | 15° | 11° |
| Mid | 5° | 10° | 16° | 12° |
| South | 8° | 11° | 17° | 14° |

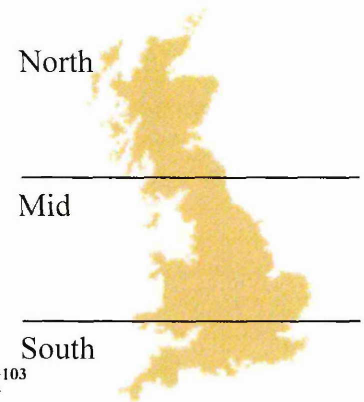



**Table 4.3 Approximate seasonal average relative humidity in the UK[103]**

| | Dec - Feb | Mar - May | Jun - Aug | Sep - Nov |
|---|---|---|---|---|
| North | 85% | 70% | 70% | 85% |
| Mid | 85% | 70% | 70% | 85% |
| South | 85% | 75% | 75% | 85% |

**Figure 4.3 Map for climate tables 4.2 and 4.3**

For example, assume a repair will be carried out in December 2005 in Edinburgh (Northern Zone).

Temperature = 4°C, RH = 85%

## 4.4 Shrinkage of patch repair: correction factors for temperature and humidity

For the material Shucrete 1, from Table 4.1:

Free shrinkage at 28 days = 630 microstrain obtained at 23°C, 50%RH

(where 23°C, 50%RH, are the laboratory ambient conditions specified by both British and American standards)

The datum for conversion, as specified in 4.3.2 is 15°C. The shrinkage test was conducted at a temperature of 23°C. This is 8°C higher than the datum temperature. Therefore, assuming a 1% reduction in shrinkage per degree centigrade, predicted shrinkage at the datum temperature of 15°C and 50% RH:

$$Shrinkage_{15°C} = \frac{Shrinkage_{23°C}}{1.08} = 583 microstrain$$

The temperature in Edinburgh in December, from Table 4.2, is 4°C.

Therefore, applying the temperature correction for shrinkage at 4°C relative to the datum temperature gives the Shrinkage at 4°C, 50% RH:

$$Shrinkage_{4°C} = 583 * \left(1 + \left(\frac{4-15}{100}\right)\right) = 519 microstrain$$

Therefore, the expected free shrinkage in a patch repair made with Shucrete 1 in Edinburgh, assuming the RH to remain at 50%, is 519 microstrain.

For the purposes of applying a correction for relative humidity to the shrinkage of the field patch repair, the datum value is taken as 70% RH.

In accordance with section 4.3.2, for each percentage point decrease in relative humidity from the datum RH (70%), shrinkage increases by 2%. In order to modify the current shrinkage (519 microstrain) at 50%RH, to a shrinkage obtained at 70%RH, a 2% decrease per percent increase in RH is applied. Therefore, the shrinkage of a repair patch made with Shucrete 1under a field temperature of 4°C and datum RH of 70% is given as:

$$shrinkage_{RH\,70\%} = \frac{Shrinkage_{RH\,50\%}}{1.4} = 370\,microstrain$$

From Table 4.3, the relative humidity in the northern zone of the UK (into which Edinburgh falls) in December is 85%.

In accordance with 4.3.2 , for each percentage point increase in RH above the 70% RH datum, a 3% shrinkage reduction is applied to determine the shrinkage of the field patch repair on a site in Edinburgh exposed to 4°C, 85% RH.

Shrinkage of the patch repair at 4°C, 85% RH:

$$370\left[1-(85-70)\frac{3}{100}\right] = 204 \text{ microstrain}$$

## 4.5 Creep of patch repairs: correction factors for atmospheric conditions, specimen size, and age at loading

In addition to modifying values of creep given in manufacturers' literature to account for difference in temperature and relative humidity between laboratory cured specimens and conditions in the field, there is a need to consider other variations between the laboratory and the field, such as dimensional differences and age of specimen at loading. This section examines the effect of the parameters that have been identified as important in determining the field creep properties of a repair material based on its laboratory values (manufacturers' data).

### 4.5.1 Modifying creep for early age loading

Generally, creep tests in the laboratory are conducted on specimens that have been cured unloaded for 28 days. A repair material in a patch repair, however, will begin to creep as soon as it is subjected to restrained shrinkage tensile stresses, which occur immediately after the application of the patch repair. It has been shown[84] that creep of concrete and of repair materials is influenced by the age of the material when load is applied to it. The effect on creep of the age of loading is mainly due to the increase in strength of the concrete with age[101].

Data has been obtained to compare the creep performance of materials when loaded at different ages (1 and 7 days) in a tensile creep rig[84]. The materials tested were concretes with admixtures such as macrofibres, microfibres and superplasticizers. The data for four materials are listed in Table 4.4. The four materials are labelled C35, C55, S35, S55 where suffixes represent the water to cement ratios. The 'C' prefix indicates normal Portland cement and ‘S’ indicates normal Portland cement with silica fume. The 28 day tensile strengths of the materials are given in Table 4.4.

**Table 4.4 Specific Creep of concrete loaded at different days ($\mu m/mm/N/mm^2$)**

| | | Specific Creep Strains (Microstrain/N/mm²) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Material | C55 loaded at | | C35 loaded at | | S55 loaded at | | S35 loaded at | |
| | Age at load application | 1 day | 7 days | 1 day | 7 days | 1 day | 7 days | 1 day | 7 days |
| | 28 day tensile strength | 4.4 N/mm² | | 4.9 N/mm² | | 4.8 N/mm² | | 5.6 N/mm² | |
| Time after load application (days) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | 2 | 60 | 25 | 30 | 18 | 62 | 24 | 70 | 26 |
| | 4 | 70 | 33 | 47 | 21 | 87 | 37 | 89 | 34 |
| | 6 | 80 | 42 | 55 | 24 | 100 | 50 | 100 | 40 |
| | 10 | 100 | 54 | 70 | 32 | 120 | 68 | 117 | 50 |
| | 15 | 110 | 67 | 80 | 41 | 134 | 88 | 130 | 56 |
| | 20 | 120 | 73 | 85 | 49 | 145 | 100 | 140 | 60 |
| | 25 | 128 | 80 | 91 | 53 | 152 | 111 | 149 | 64 |
| | 30 | 135 | 88 | 96 | 58 | 158 | 120 | 155 | 69 |
| | 35 | 141 | 97 | 99 | 61 | 162 | 128 | 160 | 72 |
| | 40 | 147 | 100 | 100 | 63 | 165 | 132 | 163 | 75 |

Materials loaded at 1 day were subjected to a constant stress of 0.77 $N/mm^2$. Materials loaded at 7 days were subjected to a constant stress of 1 $N/mm^2$.

The most important factor that controls the creep strain of the repair patch is the stress/strength ratio induced by the applied load. The constant applied stresses to the test samples whose creep data are given in Table 4.4 will result in different stress strength ratios as the aging of the test samples during the creep test period results in a gradual increase in strength. In order to determine the stress strength ratio at each time of creep strain monitoring, the tensile strength of the repair material at each age is determined from the following expression, equation 3.18, derived in Chapter 3:

$$\frac{f_t}{f_{t28}} = \frac{t}{2.7975 + 0.9216t}$$ **Equation 4.1**

This equation relates the tensile strength at any age (t) to the 28 day tensile strength ($f_{t28}$) of a material. The resulting tensile strength at each age is given in Table 4.5.

For the samples loaded at seven days, those materials have already achieved a seven day tensile strength, the materials loaded at one day have achieved a one day strength. This accounts for the different tensile strengths apparent in Table 4.5 between repair materials loaded at 1 and 7 days.

The procedure for calculation of tensile strength at a particular time is illustrated by the following example:

Consider material C35 loaded at an age of 7 days to commence the creep test. At 10 days after the start of the creep test, the age of the specimen is 17 days. The 28 day tensile strength of the material, $f_{t28}$ = 4.9 N/mm²

Therefore, substituting into Equation 4.1 for $f_{t28}$ = 4.9 N/mm², t = 17 days gives:

$$\frac{f_t}{4.9} = \frac{17}{2.7975 + 0.9216 * 17}$$

Therefore, $f_t$ at 17 days (10 days under creep load) = 4.66 N/mm². This value is listed in Table 4.5 for material C35 loaded at 7 days age to commence the creep test and represents the tensile strength of the material at 10 days after the application of creep load (age 17 days).

Because the materials were subject to a constant stress in the creep tests, the stress strength ratio changes with time.

**Table 4.5 Development of Tensile Strength (N/mm²) during creep testing**

| | | Tensile strength (N/mm²) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Material | C55 | C35 | S55 | S35 | C55 | C35 | S55 | S35 |
| | Age at load application (days) | 1 | 1 | 1 | 1 | 7 | 7 | 7 | 7 |
| Time after application of load (days) | 0 | 1.22 | 1.36 | 1.33 | 1.56 | 3.44 | 3.83 | 3.75 | 4.38 |
| | 2 | 2.45 | 2.73 | 2.68 | 3.12 | 3.69 | 4.11 | 4.03 | 4.70 |
| | 4 | 3.07 | 3.42 | 3.35 | 3.91 | 3.87 | 4.31 | 4.22 | 4.92 |
| | 6 | 3.44 | 3.83 | 3.75 | 4.38 | 4.00 | 4.45 | 4.36 | 5.09 |
| | 10 | 3.87 | 4.31 | 4.22 | 4.92 | 4.19 | 4.66 | 4.57 | 5.33 |
| | 15 | 4.15 | 4.62 | 4.52 | 5.28 | 4.34 | 4.83 | 4.73 | 5.52 |
| | 20 | 4.31 | 4.80 | 4.70 | 5.49 | 4.44 | 4.94 | 4.84 | 5.65 |
| | 25 | 4.42 | 4.92 | 4.82 | 5.62 | 4.51 | 5.02 | 4.92 | 5.74 |
| | 30 | 4.49 | 5.01 | 4.90 | 5.72 | 4.56 | 5.08 | 4.97 | 5.80 |
| | 35 | 4.55 | 5.07 | 4.96 | 5.79 | 4.60 | 5.12 | 5.02 | 5.86 |
| | 40 | 4.59 | 5.12 | 5.01 | 5.85 | 4.64 | 5.16 | 5.06 | 5.90 |

(where shaded boxes represents values given in the illustrated examples within the text)

The stress/strength ratios are obtained by dividing the applied constant stress (0.77 N/mm$^2$ for specimens loaded at 1 day age, 1 N/mm$^2$ for the specimens loaded at 7 day age) by the tensile strengths of each test material at the specific time under creep loading. For example, consider material S35 loaded at 1 day age to a constant stress of 0.77 N/mm². After 10 days under creep load (age of specimen = 11 days) its tensile strength is 4.92 N/mm$^2$ (Table 4.5). Therefore the applied stress/strength = 0.77 / 4.92

= 15.6%

The stress/strength ratios at each time (days) under creep loading, corresponding to the specific creep and strength data of specimens given in Table 4.4 and Table 4.5, have been calculated by the above procedure and are listed in Table 4.6. Table 4.6 shows that the stress/stength ratios are generally less than 30% throughout except for specimens loaded at the age of 1 day which showed high stress/strength ratios at the first day of loading.

**Table 4.6 Stress/Strength ratios**

| | | Stress / strength ratio (%) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Material | C55 | C35 | S55 | S35 | C55 | C35 | S55 | S35 |
| | Age at load application (days) | 1 | 1 | 1 | 1 | 7 | 7 | 7 | 7 |
| Time after application of load (days) | 0 | 63.0 | 56.5 | 57.7 | 49.5 | 29.1 | 26.1 | 26.6 | 22.8 |
| | 2 | 31.4 | 28.2 | 28.8 | 24.7 | 27.1 | 24.3 | 24.8 | 21.3 |
| | 4 | 25.1 | 22.5 | 23.0 | 19.7 | 25.9 | 23.2 | 23.7 | 20.3 |
| | 6 | 22.4 | 20.1 | 20.5 | 17.6 | 25.0 | 22.4 | 22.9 | 19.6 |
| | 10 | 19.9 | 17.9 | 18.3 | 15.6 | 23.9 | 21.4 | 21.9 | 18.8 |
| | 15 | 18.6 | 16.7 | 17.0 | 14.6 | 23.1 | 20.7 | 21.1 | 18.1 |
| | 20 | 17.9 | 16.0 | 16.4 | 14.0 | 22.5 | 20.2 | 20.7 | 17.7 |
| | 25 | 17.4 | 15.6 | 16.0 | 13.7 | 22.2 | 19.9 | 20.3 | 17.4 |
| | 30 | 17.1 | 15.4 | 15.7 | 13.5 | 21.9 | 19.7 | 20.1 | 17.2 |
| | 35 | 16.9 | 15.2 | 15.5 | 13.3 | 21.7 | 19.5 | 19.9 | 17.1 |
| | 40 | 16.8 | 15.0 | 15.4 | 13.2 | 21.6 | 19.4 | 19.8 | 17.0 |

Table 4.7 transforms the data in Table 4.6 to show the specific creep strains that would have occurred in the materials if they were loaded at constant stress/strength ratio of 30%. Assuming a linear relationship between stress/strength ratio and creep, the data listed in Tables 4.4 and 4.6 are used to determine the specific creep values at a stress/strength ratio of 30%. These values are given in Table 4.7.

It is well established in existing literature that there is a linear relationship between stress/strength ratio and specific creep strain of a given cementituous material[101]. The linear relationship is less valid at very high stress/strength ratios where micro-cracking within the concrete matrix can lead to non-linear behaviour. For the purposes of analysis in this thesis, linear behaviour is assumed since if high stress/strength ratios do occur in a repair patch, their duration is extremely short relative to the creep period. Table 4.6 shows high stress/strength ratios immediately after loading on the first day of specimens loaded at 1 day age but the stress/strength ratio decreases rapidly under sustained loading. For example, consider C55 loaded at 1 day age in Table 4.6. The stress/strength ratio at loading (curing age: 1 day, age at load application: 0 day) is 63% which rapidly reduces to 31.4% after age at load application of 2 days. The reduction is rapid within the first few hours of creep loading.

**Table 4.7 Specific creep of specimens extrapolated at 30% stress/strength ratio**

| | | Specific creep strains (microstrain/N/mm²) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Loaded at 1 day | | | | Loaded at 7 days | | | |
| | Material | C55 | C35 | S55 | S35 | C55 | C35 | S55 | S35 |
| | Age at loading (days) | 1 | 1 | 1 | 1 | 7 | 7 | 7 | 7 |
| Time after application of load (days) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | 2 | 57 | 32 | 65 | 85 | 28 | 22 | 29 | 37 |
| | 4 | 84 | 63 | 114 | 136 | 38 | 27 | 47 | 50 |
| | 6 | 107 | 82 | 146 | 171 | 50 | 32 | 66 | 61 |
| | 10 | 151 | 118 | 197 | 224 | 68 | 45 | 93 | 80 |
| | 15 | 178 | 144 | 236 | 267 | 87 | 59 | 125 | 93 |
| | 20 | 202 | 159 | 266 | 299 | 97 | 73 | 145 | 102 |
| | 25 | 220 | 175 | 286 | 327 | 108 | 80 | 164 | 110 |
| | 30 | 236 | 187 | 302 | 345 | 120 | 88 | 179 | 120 |
| | 35 | 250 | 196 | 313 | 361 | 134 | 94 | 193 | 127 |
| | 40 | 263 | 199 | 322 | 371 | 139 | 98 | 200 | 133 |

The following example shows the procedure adopted for calculating the creep strains at 30% stress / strength ratio. Consider material C55 loaded at 7 days age to commence creep testing and determine the specific creep strain corresponding to the applied stress/strength of 30% at 15 days after commencing the creep test (age of specimen, 15+7 = 22 days).

Creep data in Table 4.4 gives:

Specific creep at 15 days after load application = 67 microstrain/N/mm$^2$.

The corresponding stress / strength ratio at age of load application 15 days, from Table 4.6, is 23.1%. Hence, specific creep at stress/strength ratio of 30% = 67 * (30/23.1)

$$= 87.2\ \mu m/mm/N/mm^2$$

The specific creep data corresponding to the applied stress/strength of 30% are calculated for the materials from Table 4.4 to Table 4.6 and are listed (rounded to the nearest integer) in Table 4.7. The average specific creep data in Table 4.7 at a stress/strength ratio of 30% for materials loaded at 1 day age and 7 day age are plotted in Figure 4.4. It is quite clear that at a constant stress/strength ratio, the specimens loaded at 7 days age show much lower specific creep than corresponding specimens of the same material loaded at 1 day.

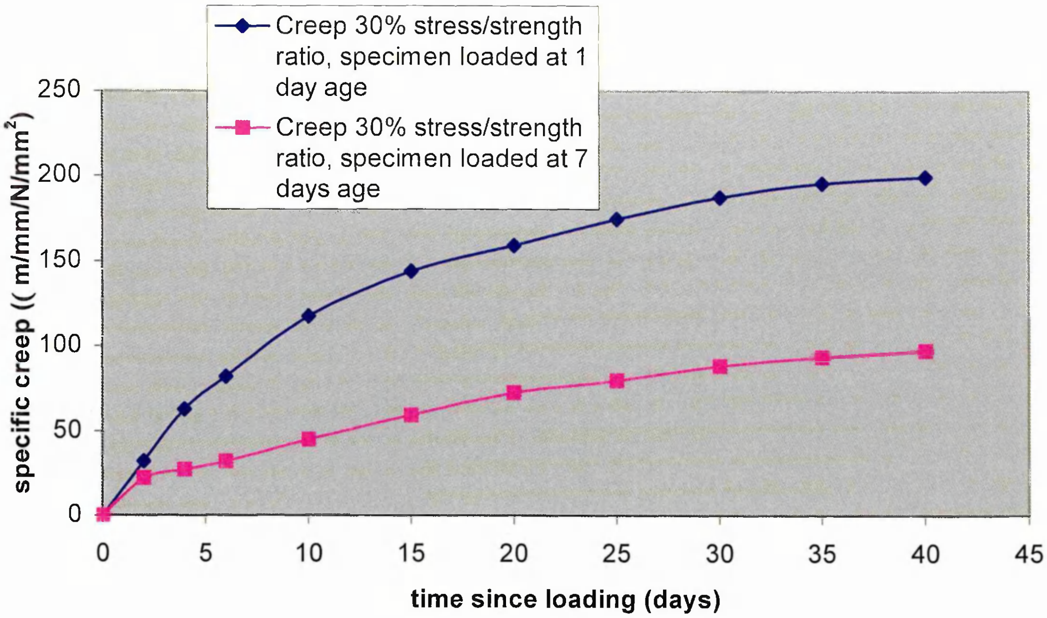



**Figure 4.4 Specific creep-time relationship for concrete loaded at 1 and 7 day ages at a stress/strength ratio of 30% (average of Table 4.7 materials).**

The ratio of specific creep of the material loaded at 1 day to specific creep of the material loaded at 7 days both at 30% stress/strength ratio can now be determined. These values are listed in Table 4.8 at various incremental times under creep loading.

**Table 4.8 Ratio of specific creep due to loading at 1 day to loading at 7 days, at a stress strength ratio of 30%.**

| Time after load application (days) | Material | | | |
|---|---|---|---|---|
| | C55 | C35 | S55 | S35 |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 |
| 2 | 2.07 | 1.44 | 2.23 | 0.65 |
| 4 | 2.19 | 2.31 | 2.42 | 0.95 |
| 6 | 2.13 | 2.56 | 2.23 | 1.15 |
| 10 | 2.22 | 2.62 | 2.12 | 1.24 |
| 15 | 2.04 | 2.42 | 1.89 | 1.28 |
| 20 | 2.07 | 2.19 | 1.83 | 1.20 |
| 25 | 2.04 | 2.19 | 1.74 | 1.25 |
| 30 | 1.96 | 2.12 | 1.69 | 1.26 |
| 35 | 1.87 | 2.08 | 1.63 | 1.28 |
| 40 | 1.89 | 2.04 | 1.61 | 1.27 |

The specific creep ratios listed in Table 4.8 are plotted in Figure 4.5 against the time under creep loading. The graphs in Figure 4.5 show that after the early period under load (about 5 days), the specific creep ratios attain a relatively constant value.

For example, considering material C55 at day 10 under creep loading at a constant stress strength ratio of 30%, the specific creep of the material loaded at 1 day age is 2.22 times higher than the specific creep of the material loaded at 7 days after casting.

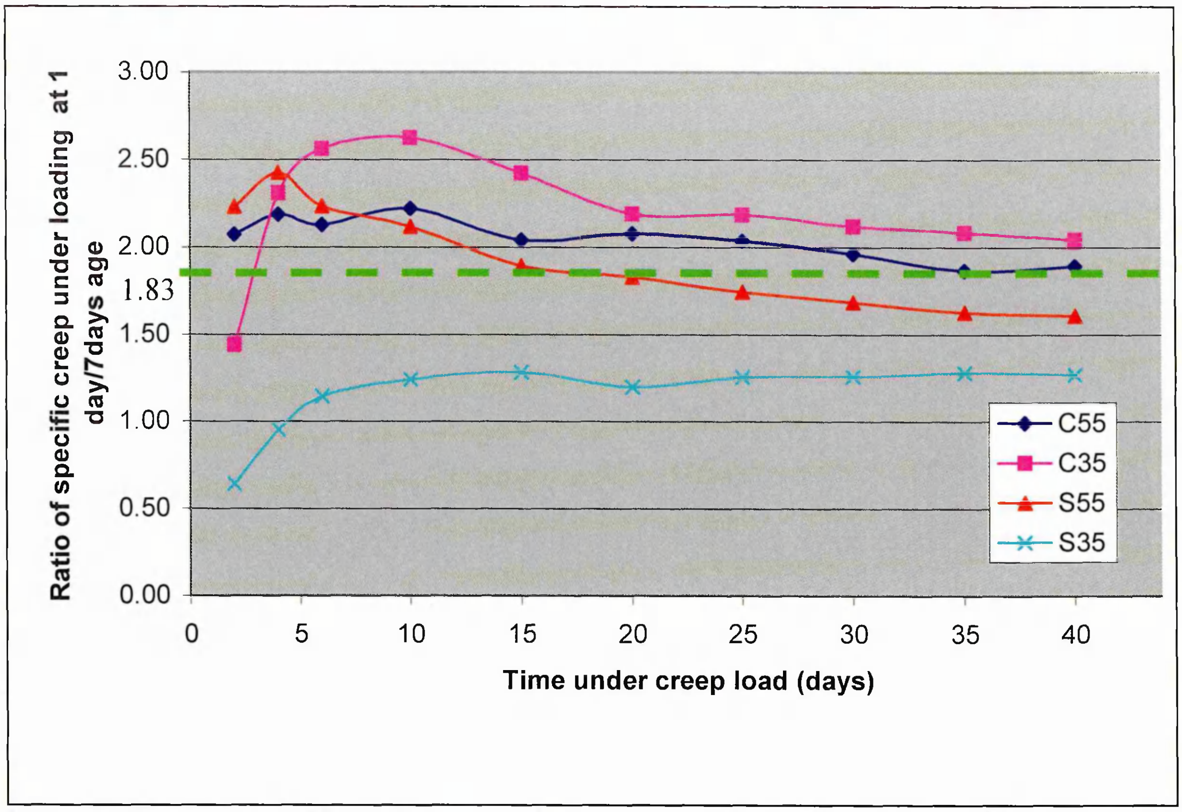



**Figure 4.5 Increase in specific creep due to loading at early age (stress/strength ratio 30%)**

Considering all data in Figure 4.5 (and Table 4.8) and excluding data points at up to 5 days under creep loading, the average increase in specific creep due to loading at 1 day as opposed to 7 days is 1.83 or 183%. This factor is incorporated into the procedure for determining the performance of a patch repair by calculating the tensile creep strains produced by the restrained shrinkage tension in the repair material at regular incremental ages from the time the repair patch is applied (Chapter 3).

In situations where the difference between the age of creep loading in the laboratory test (representing manufacturer's data) and actual insitu creep due to early loading is represented by a factor greater than 1.83, the material will creep more in practice than the predicted amount and, therefore, result in greater relaxation of tensile stresses. This provides an additional factor of safety for the repair patch performance against cracking, which is acceptable. In situations where the increase is less than provided by the factor

1.83 (such as would occur with material S35 in Figure 4.5) the material will theoretically creep less than the analytical procedure has assumed. This is not desirable. In practice, the creep data provided in the analysis is likely to be based on test specimens loaded for creep testing at 28 days age. This would represent the typical basis for manufacturers' test data on their repair materials. The actual transfer of tensile stress in a repair patch, on the other hand, is immediate after the application of a repair patch, which follows the onset of shrinkage. The above data from which the factor of 1.83 was established is based on the difference in specific creep between specimens loaded at 7 days and 1 day age. Therefore, an additional factor of safety is inherent for patch repairs whose creep data is obtained by load application at 28 days age. This is representative of the actual information the software system will be supplied with. This clearly suggests that the actual creep which occurs in the repair patch will be higher than that which the software system will estimate. Thereby providing a factor of safety.

Figure 4.6 shows an estimate of the actual difference in creep which occurs through loading creep specimens at 28 days and 1 day age. The curve representing the development of specific creep with time for a material loaded at 28 days was developed using the linear relationship between specific creep and stress strength ratio[101].

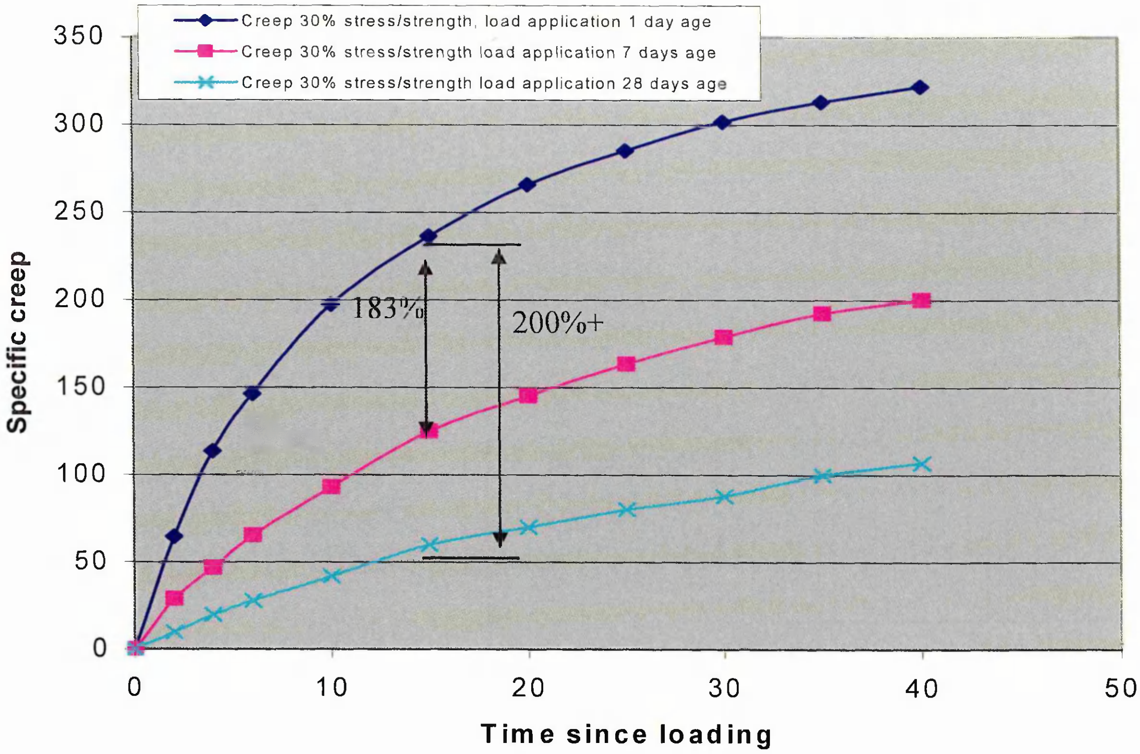



**Figure 4.6 Estimated specific creep of material loaded at 1 day, 7 days and 28 days age**

This hypothesis correlates well with the results of experiments by Ulitskii[104] who showed that for concrete loaded at 3 days, a correction factor for creep of 2.0 is required to compare the creep value with the concrete loaded at 28 days. (see Table 4.9). All specimens were loaded at the same stress/strength ratios.

**Table 4.9 Creep modification factors for early age loading (concrete)[104]**

| Age of concrete at loading (days) | 3 | 5 | 7 | 10 | 14 | 20 | 28 | 40 | 60 | 90 | 180 | 360 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Correction factor for normal curing | 2.0 | 1.8 | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 | 0.8 | 0.7 | 0.6 | 0.5 | 0.45 |
| Correction factor for autoclaving and steam-curing | 1.5 | 1.4 | 1.3 | 1.25 | 1.2 | 1.1 | 1.0 | 0.8 | 0.7 | 0.6 | 0.5 | 0.45 |

## 4.5.2 Creep of patch repair: correction factors for temperature and relative humidity

Currently, there are no standards for the determination of creep in repair materials, therefore, an adapted version of ASTM C 512 is the recommended standard. Standard creep tests should be carried out at 20°C and 50% RH.

### 4.5.2.1 Temperature correction

For the material Shucrete 1, from Table 4.1:

Creep at 20°C = 794 microstrain

(where 20°C is the laboratory ambient condition to ASTM C 512)

In accordance with section 4.3, there is linear change in creep of 1.25%, for every percentage point increase or decrease in temperature under which creep takes place.

The datum for conversion, as specified in section 4.3.2 is 15°C. The creep test was conducted at a temperature of 20°C. This is 5°C higher than the datum temperature. Therefore, assuming a reduction of 1.25% in creep per degree centigrade, predicted creep at the datum temperature of 15 °C is :

$$Creep_{20°C} = Creep_{15°C} * \left(1 + \frac{(1.25 * 5)}{100}\right)$$

$$Creep_{15°C} = \frac{Creep_{20°C}}{1.0625} = 747 microstrain$$

The temperature in Edinburgh in December, from Table 4.2, is 4°C.

Therefore, applying the temperature correction for creep at 4°C relative to the datum temperature gives:

$$Creep_{4°C} = Creep_{15°C} * \left(1 + \frac{1.25*(4-15)}{100}\right)$$

= 656.7 microstrain

### 4.5.3 Modifying creep by relative humidity

In accordance with section 4.3.2,

$$k_{rh} = -0.007(RH\%) + 1.3 \qquad \text{Eq. 4-1}$$

Where $k_{rh}$ = creep correction factor for relative humidity of exposure

In order to calculate its effect on the creep value, it is necessary to know the relative humidity under which the standard creep test is conducted on the repair material specimen, and the relative humidity which will be expected on site when the repair is applied.

The calculation procedure is described with reference to the example introduced in section 4.3.2, as follows:

In accordance with Table 4.3 and Figure 4.3, RH for Edinburgh in December = 85%. This represents the site environment where the repair application will be made.

The creep data for the repair material has been obtained in accordance with ASTM C512, testing under a RH of 50%.

From equation 4-1,

$$k_{rh50} = -0.007(50) + 1.3 = 0.95$$
$$k_{rh85} = -0.007(85) + 1.3 = 0.705$$

Referring to section 4.3.2 and equation 4-1 the following expressions for creep at the standard RH (50%) and the site RH (85%) can be written in terms of a creep value at the datum relative humidity:

$$Creep_{RH50} = k_{RH50} Creep_{RHdatum}$$
$$Creep_{RH85} = k_{RH85} Creep_{RHdatum}$$
$$\therefore \frac{Creep_{RH50}}{k_{RH50}} = \frac{Creep_{RH85}}{k_{RH85}}$$

The creep strain at RH 50% and temperature 4°C (representing the Edinburgh site) has been determined in the previous section (4.5.2.1) as $Creep_{4°C,RH50} = 656.7$ microstrain.

$$\therefore \frac{656.7}{0.95} = \frac{Creep_{RH85}}{0.705}$$
$$Creep_{RH85} = \frac{656.7}{0.95} * 0.705 = 487.3 microstrain$$

### 4.5.4 Modifying creep for specimen size

The size of a concrete member (or repair patch) will determine the degree to which changes in ambient temperature and relative humidity affect its creep[105]. Creep strain decreases with an increase in the size of a concrete member[101] for any given stress/strength ratio. A correction factor to account for this must also be applied to repair patches.

Neville[104] provides correction factors for concrete member thickness (which equates to volume / surface ratios for a cube or cylinder specimen, because as a concrete member becomes larger, the volume/surface ratio tends to half the concrete depth). Neville gives these factors in a tabular form, and a corrective factor is provided for situations where one side of the concrete member is sealed (as for a repair material adjacent to the substrate concrete surface) where the volume surface ratio tends to the concrete depth. Therefore, Figure 4.7 fits two curves to Neville's data, one for concrete repairs (where one side of the repair is sealed through contact with the substrate surface) and another for elements of concrete where all the surface area is exposed to the air.

Nawy[102] also provides information on the relationship between specimen size and creep. A linear relationship is suggested but no correction factors are provided for changes in the volume/surface ratio between concrete repair and newly cast members. However, Figure 4.7 shows clearly, through its similarity with Neville's data, that Nawy's linear relationship represents concrete where all surfaces are exposed to air. The data provided by Neville (Figure 4.7) fits a logarithmic relationship and these equations will be used to establish the factors that correct creep for specimen size.

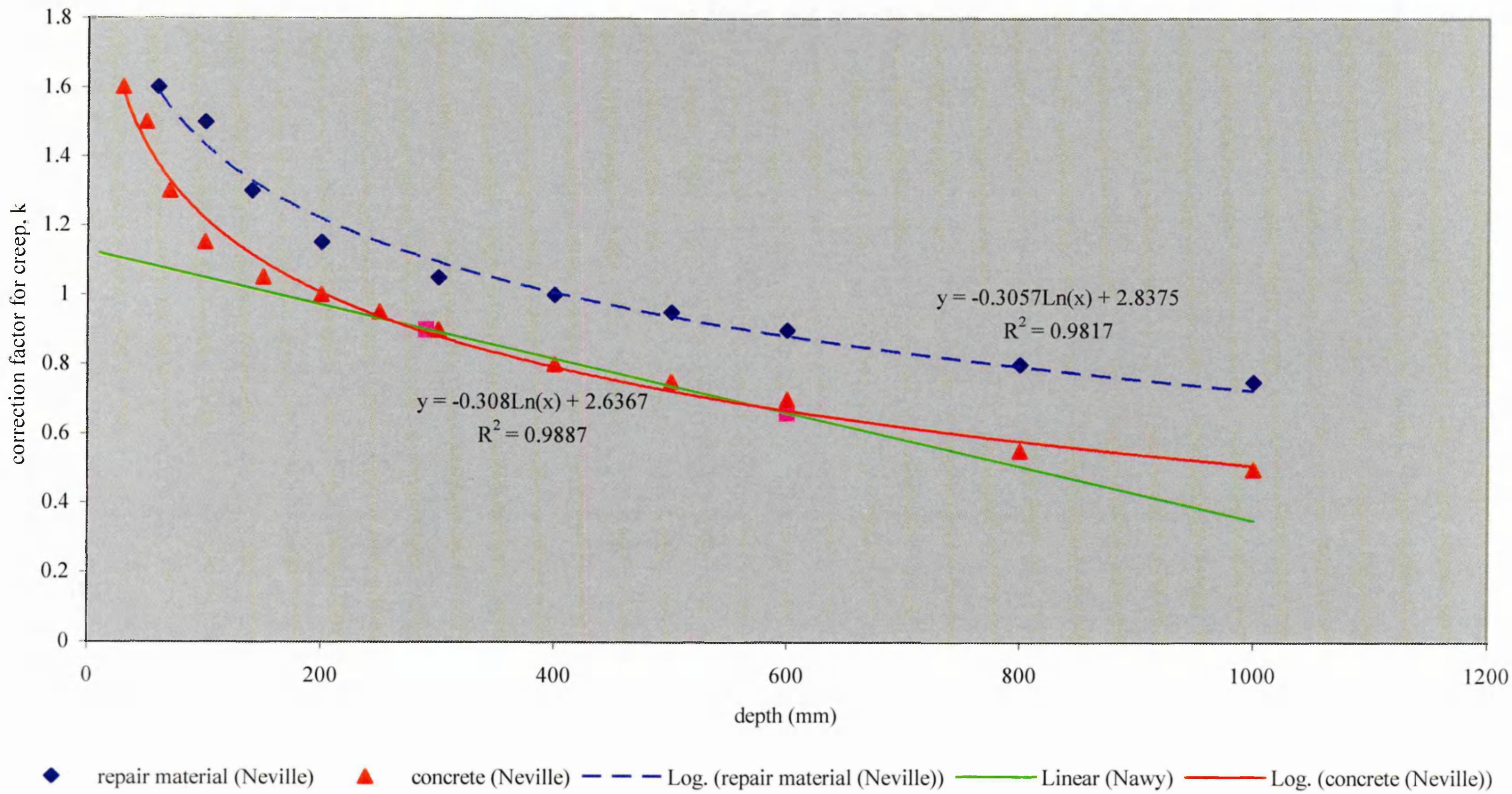



**Figure 4.7 Modifications for effect of specimen size on creep in concrete and repair materials**

A patch repair is surrounded by the substrate on all faces except the top surface exposed to the atmosphere. The volume of the patch repair is equal to the top surface area multiplied by depth and clearly volume divided by the exposed surface area equates to depth. Therefore volume/surface ratio of a patch repair which has only its top surface exposed can be said to be equivalent to depth. Hence, as shown in Figure 4.7,

$$k_{sizerep} = -0.3057\ln(depth) + 2.8375 \qquad \text{Eq. 4-2}$$

Where $k_{sizerep}$ is the creep correction factor for the size of concrete repairs

Also shown in Figure 4.7 is the relationship for the creep correction factor for test specimens in the laboratory which are exposed to the atmosphere on all surfaces, which is:

$$k_{sizelab} = -0.308\ln(volume / surface) + 2.6367 \qquad \text{Eq. 4-3}$$

Where $k_{sizelab}$ is the creep correction factor for the size of laboratory specimens

Equation 4-2 gives the creep correction factor for concrete repairs which are sealed by the substrate except on one face and Equation 4-3 gives the creep correction factor for laboratory creep test specimens which are exposed to air on all faces. Both these equations are taken from Figure 4.7.

The volume surface ratio of the specimen from ASTM C 512 = 25mm;

Assume for simplicity the depth of the proposed repair is 50mm.

Therefore, using equation 4-2, it can be determined for the proposed repair:

$$k_{sizerep} = -0.3057\ln(50) + 2.8375 = 1.642$$

and, similarly, using equation 4-3 for the laboratory specimen:

$$k_{sizelab} = -0.308\ln(25) + 2.6367 = 1.645$$

These modification factors need to be applied to the datum creep data (modification factor of 1.0) so that the creep strain modified for repair size can be calculated.

The following expressions for creep in laboratory specimens, and creep in a patch repair can be written in terms of creep values of a datum specimen size:

$$Creep_{rep} = k_{sizerep} Creep_{datum}$$
$$Creep_{lab} = k_{sizelab} Creep_{datum}$$
$$\therefore \frac{Creep_{rep}}{k_{sizerep}} = \frac{Creep_{lab}}{k_{sizelab}}$$

The creep strain at RH 85%, temperature 4°C (representing the Edinburgh site in December) for a standard (ASTM) specimen of volume to surface ratio of 25mm has been determined in section 4.5.3 as Creep = 487.3 microstrain.

$$\therefore \frac{487.3}{1.642} = \frac{Creep_{rep}}{1.645}$$
$$Creep_{rep} = \frac{487.3}{1.642} * 1.645 = 488.2 microstrain$$

where $k_{sizerep}$ = creep modification factor due to size of repair

$k_{sizelab}$ = creep modification factor due to size of test specimen

$Creep_{datum}$ = Creep at correction factor 1

$Creep_{rep}$ = corrected creep for volume/surface ratio of repair material

$Creep_{lab}$ = Creep at RH 85%, temperature 4°C in a specimen with a volume/surface ratio of 25mm.

In the example above, the modification is slight. This is unsurprising as the volume/surface ratio of the repair material (its depth) is just twice that of the test specimen.

## *4.6 Modifications for field shrinkage*

Kong & Evans[101] describe the effect of volume/surface ratio on relative shrinkage in concrete as follows :-

$$\mu = 10.779e^{-0.005(volume/surface)} \qquad \text{Eq. 4-4}$$

where μ = relative shrinkage against datum specimen

This relationship can be used to find the ratio between shrinkage of the laboratory specimen and shrinkage that would be expected of that same material in the field with a known volume/surface ratio of the patch repair.

A standard shrinkage specimen for a repair material has the dimensions 25 x 25 x 285mm (according to ASTM C157[96]). Therefore, the volume/surface ratio of the repair material specimen is:

(25x25x285)/((25*25*2)+(25*285*4)) = 5.99 mm

If the planned repair patch has the dimensions 3m x 3m x 50mm, its volume surface ratio is (assuming only one face is exposed and the remaining faces are surrounded by the substrate):

(3000x3000x50)/(3000*3000) = 50mm

The relationship in equation 4-4 can now be used to determine the relative shrinkage of the laboratory specimen:

$$\mu = 10.779e^{-0.005(5.99)}$$
$$\mu = 10.46$$

Similarly, it is used to determine the relative shrinkage of the field repair:

$$\mu = 10.779e^{-0.005(50)}$$
$$\mu = 8.39$$

In summary,

Relative shrinkage of ASTM specimen = 10.46

Relative shrinkage of planned repair patch = 8.39

It is also shown by Kong and Evans[101] that:

Relative shrinkage in specimen / Relative shrinkage in planned repair = ratio of specimen shrinkage to in-situ shrinkage

Therefore:

10.46/8.39 = 1.247

∴ ASTM specimen shrinkage = 1.247 * In-situ shrinkage

It was shown in section 4.4 that when temperature and relative humidity differences between the laboratory shrinkage and shrinkage in the field are allowed for, a free shrinkage in an ASTM specimen of material Shucrete 1 is 204 microstrain.

Therefore, it can be stated that the free shrinkage of the in situ repair patch is:-

204 / 1.247 = 164 microstrain

## *4.7 Properties of the substrate*

In order to accurately predict the performance of a patch repair, it is necessary to know the properties of the substrate concrete with which it will interact. A core must be taken from the substrate concrete and its compressive strength and modulus of elasticity determined in the laboratory.

Considering the case of the site at Edinburgh for *shucrete 1* repair:

Height of Core: 250mm

Diameter of Core: 100mm

Strength: 58 MPa

Elastic Modulus: 28 GPa

These values, determined through laboratory testing also require correction to account for a number of factors.

Neville[82] gives details of the necessary correction factors to account for difference in height/diameter ratios of core samples (Figure 4.8):

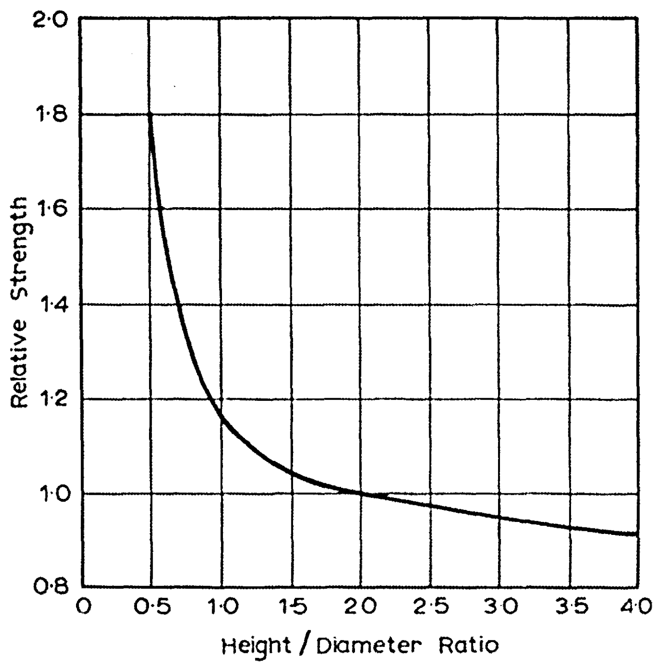



**Figure 4.8 Correction factor for height / diameter ratio of concrete cores**

Using the relationship in Figure 4.8, and the height and diameter of the concrete core taken from the site in Edinburgh:

Height/diameter of core =

250/100 = 2.5

Therefore, from Figure 4.8, the relative strength = 0.95

Therefore, the modified strength of core = 0.95 x 58 = 55.1 MPa

This core strength requires conversion to cube strength. *Kong & Evans*[101] provide this simple modification.

Cube strength = cylinder strength / 0.8

Therefore, for the substrate concrete at the Edinburgh site:

Cube strength of the substrate = 55.1 / 0.8 = 68.88 MPa

## 4.8 Development of properties

The development of the key properties in the repair material, with time, can now be tabulated. These properties are: tensile strength, shrinkage, elastic modulus and the resulting 'strain transfer' from the repair patch to the substrate. As the elastic modulus of the repair material gradually increases with the curing period, it is desirable for this value to become higher than the elastic modulus of the substrate concrete. In this way, a transfer of shrinkage strain can occur from the repair material into the substrate, leaving a reduced restrained shrinkage strain in the repair material (section 3.3). The amount of strain transferred into the substrate is governed by the relationship[58]:

$$\lambda = \frac{\left(E_{rep}/E_{sub} - 1\right)}{0.0032} \qquad \text{Eq. 4-5}$$

where $\lambda$ = shrinkage transferred (%)

$E_{rep}/E_{sub}$ = ratio of elastic modulus of the repair material to elastic modulus of the substrate.

Equation 4-5 has been developed empirically from wide ranging field data and incorporates the cumulative effects of creep and shrinkage.

Table 4.10 uses the information presented in Chapter 3 to determine the development of properties in the material Shucrete 1 when applied to a 3m square, 50mm deep repair patch in Edinburgh in December. The patch is located on a reinforced concrete abutment and the defect is within a full face area of substrate concrete (i.e. it does not continue around a corner).

**Table 4.10 Development of properties with time (days) of repair material Shucrete 1 and transfer of shrinkage strain to the substrate**

| Time after repair application (days) | Tensile strength (Mpa) | Free shrinkage (microstrain) | $E_{rep}$ (Gpa) | $\frac{E_{rep}}{E_{sub}}$ | Shrinkage strain transferred, λ (%) |
|---|---|---|---|---|---|
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0 |
| 2 | 2.16 | 24.5 | 13.58 | 0.485 | 0 |
| 4 | 3.09 | 45.8 | 20.14 | 0.719 | 0 |
| 6 | 3.60 | 64.1 | 24.00 | 0.857 | 0 |
| 7 | 3.79 | 72.5 | 25.40 | 0.907 | 0 |
| 8 | 3.93 | 80.3 | 26.55 | 0.948 | 0 |
| 12 | 4.33 | 107.2 | 29.71 | 1.061 | 19 |
| 14 | 4.46 | 118.6 | 30.75 | 1.098 | 31 |
| 15 | 4.51 | 123.8 | 31.19 | 1.114 | 36 |
| 20 | 4.71 | 146.5 | 32.83 | 1.173 | 54 |
| 21 | 4.74 | 150.5 | 33.08 | 1.181 | 57 |
| 28 | 4.90 | 173.9 | 34.38 | 1.228 | 71 |
| 50 | 5.12 | 218.8 | 36.26 | 1.295 | 92 |
| 100 | 5.27 | 261.9 | 37.56 | 1.341 | 100 |
| 150 | 5.32 | 280.2 | 38.02 | 1.358 | 100 |
| 200 | 5.34 | 290.4 | 38.25 | 1.366 | 100 |
| 250 | 5.36 | 296.9 | 38.39 | 1.371 | 100 |
| 300 | 5.37 | 301.4 | 38.49 | 1.375 | 100 |
| 350 | 5.38 | 304.7 | 38.56 | 1.377 | 100 |
| 400 | 5.38 | 307.2 | 38.61 | 1.379 | 100 |

Note: Equation 4.5 which determines the percentage of shrinkage transfer can yield both negative values and values above 100%. Therefore, the minimum practical shrinkage transfer value is 0% and the maximum value is 100%.

### 4.8.1 Consider day 14

In order to explain how the values in Table 4.10 were determined, the procedure used will be demonstrated for day 14 (shown shaded in Table 4.10).

#### 4.8.1.1 Tensile strength, day 14

In accordance with Table 4.1, the 28 tensile strength of *Shucrete 1* is 5 $N/mm^2$

Using equation 3-17:

$$\frac{f_t}{f_{t28}} = \frac{t}{2.7975 + 0.9216.t}$$

Therefore, from equation 3-17, the tensile strength at any day can be calculated. Consider day 14.

$$f_t = \frac{14}{2.7975 + 0.9216 * 14} * f_{t28} = 4.46\,\text{MPa}$$

#### 4.8.1.2 Shrinkage, day 14

The 28 day shrinkage strain of *Shucrete 1* (modified to allow for climate) is 164 microstrain (section 4.6). This value represents the expected shrinkage of Shucrete 1 in Edinburgh in December, where RH = 85% and temperature = 4°C. The size of the repair patch has also been taken into account (3m x 3m x 50mm).

Using the equation 3-8:

$$\frac{\varepsilon}{\varepsilon_{28}} = \frac{t}{12.292 + 0.5017t}$$

The shrinkage at any day can be calculated. Consider day 14.

$$\varepsilon = \frac{14}{12.292 + 0.5017 * 14} * \varepsilon_{28} = 118.6 \text{ microstrain}$$

#### 4.8.1.3 Elastic Modulus:

The 28 day elastic modulus of *Shucrete1*, in accordance with Table 4.1, is 34 GPa.

Using equation 3-20:

$$\frac{E}{E_{28}} = \frac{t}{3.2637 + 0.8725t}$$

The elastic modulus at any day can be calculated. Consider day 14.

$$E = \frac{14}{3.2637 + 0.8725 * 14} * E_{28} = 30.75GPa$$

In addition to the determination of properties of Shucrete 1, it was shown earlier in the section (equation 4-5), that the amount of patch repair free shrinkage which is transferred into the substrate concrete can be determined as follows:-

Shrinkage transfer at day 14 using equation 4-5 gives:-

Modular Ratio = $E_{rm}$ / $E_{sub}$ = 30.75 / 28 = 1.098

Where $E_{rm}$ = elastic modulus of Shucrete 1 at day 14.

$E_{sub}$ = elastic modulus of substrate concrete (section 4.7)

Using equation 4-5:

$$\lambda = \frac{(1.098 - 1)}{0.0032} = 30.7\%$$

Therefore, on day 14, 30.7% of the total shrinkage strain (after relaxation through creep) will be transferred into the substrate (this value is rounded to the nearest integer in Table 4.10).

Table 4.10 shows the properties of *Shucrete 1* and the amount of strain transferred from the repair material into the substrate concrete at different ages from 1 to 400 days.

## *4.9 The effect of creep.*

It was shown in Chapter 3 (section 3.3) that any tensile stresses which develop in the patch repair due to free shrinkage of the repair material being restrained at its interface with the substrate concrete will be relaxed through the action of creep. The following section outlines a procedure for determining the effect of creep at a certain age in days.

### 4.9.1 Unit Creep

The unit (specific) creep of a material is the creep that occurs due to a loading of 1 N/mm$^2$. As outlined in section 4.5, creep specimens tested in accordance with ASTM 512 are first cured for 28 days, then loaded at a constant stress/strength ratio of 30%. Therefore, the calculation of Unit Creep is not straightforward. Ordinarily, to calculate Unit Creep, the total Creep will be divided by the total stress applied over the period of loading. However, the laboratory Creep test does not subject the specimen to a constant load as Figure 4.9 demonstrates.

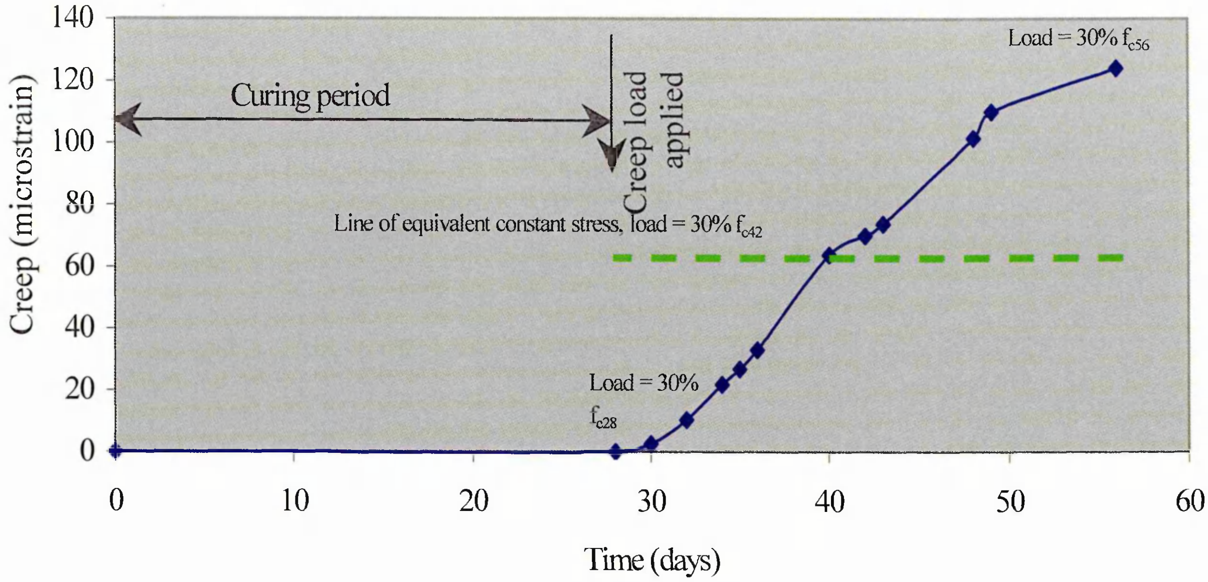



**Figure 4.9 Life of Creep sample from casting**

Figure 4.9 shows how, at day 28, the creep specimen is loaded. On this day, the load applied is equal to 30% of the 28 day compressive strength of the specimen material. This applied load is increased periodically until the final load is equal to 30% of the 56 day compressive strength of the material.

It is common, when determining Unit Creep values, to simply divide the total creep by 30% of the 28 day compressive strength. But it is clear from Figure 4.9 that *Shucrete 1* was subjected to a constant stress/strength of 30% throughout the 28 day testing period, hence the applied stress would have increased with age in order to maintain the 30% stress/strength ratio. It, therefore, needs to be determined if the common practice of determining unit creep by dividing total creep by 30% of the 28 day compressive strength yields results that differ only negligibly from a more accurate practice of dividing total creep by 30% of the average compressive strength of the material over the 28 day test period. To make the comparison, a mid-loading value for compressive strength is used, the 42 day compressive strength (represented by the line of 'equivalent constant stress' in

Figure 4.9). This line represents the average stress to which the laboratory specimen was subjected over the 28 day period between day 28 and day 56.

Therefore, it can be assumed that *Shucrete 1* achieved its 28 day creep value (the creep test was conducted for 28 days) under a constant stress equivalent to 30% of its 42 day cube strength.

In accordance with Table 4.1:

$$f_{c28} = 30\text{N/mm}^2$$

Therefore, using equation 3-11, the 42 day compressive strength of Shucrete 1 can be determined.

$$f_{c42} = \frac{42}{4.4994 + 0.8876 * 42} * f_{c28}$$
$$f_{c42} = \frac{42}{4.4994 + 0.8876 * 42} * 30$$
$$f_{c42} = 30.16 \text{ N/mm}^2$$

Therefore, it is shown that 42 day compressive strength of the material is just 0.5% higher than the 28 day strength. Hence using the 28 day compressive strength to determine Unit Creep is acceptable practice.

Therefore, the unit creep of Shucrete 1 can be determined;

$$30\% \; f_{c28} = 0.3 * 30 = 9 \text{ N/mm}^2$$

From section 4.5.4, the 28 day creep value of Shucrete 1 obtained at a 30% stress/strength ratio and modified to account for climate, time of application and specimen size is known:

| | | |
|---|---|---|
| Creep (modified) | microstrain | 488.2 |
| Stress/strength ratio | | 30% |

It is known that the creep specimen in the laboratory was loaded at a constant stress of 9 N/mm².

Therefore, the 28 day creep in compression at 1 N/mm$^2$ = 488 / 9 = 54.2 microstrain.

Where 54.2 microstrain is the modified specific creep value of the material 28 days after loading, having been loaded 28 days after casting. This figure requires further modification in accordance with section 4.5.1, which recommended an increase in specific creep of 183% to account for the fact that when the patch repair is applied, the repair material is subjected to stress immediately, i.e. at age 0 day.

Therefore, the 28 day creep in compression at 1 N/mm$^2$ for material subject to immediate loading after the application of the repair patch, $C_{U28}$:

54.2 * 1.83 = 99.3 microstrain

### 4.9.2 The effect of creep at day 2:

In order to calculate the creep in Shucrete 1 at 2 days after repair application, firstly the stress it has been subjected to must be established.

On day two, from Table 4.10, the free shrinkage of the material would be 25 microstrain. However, at the repair / substrate interface the repair material, Shucrete 1, is unable to shrink freely and tensile stresses develop. Hence the stress in the material at day 2 can be simply determined as:

$$\begin{aligned}\sigma_2 &= \varepsilon_{sh2} * E_{rep2}\\ &= \left(24.5*10^{-6}\right)\left(13.58*10^{3}\right)\\ &= 0.333 N/mm^2\end{aligned}$$

Where $\sigma_2$ = tensile stress in repair material at day 2

$\varepsilon_{sh2}$ = free shrinkage of repair material at day 2

$E_{rep2}$ = Elastic modulus of repair material at day 2

The values of $\varepsilon_{sh2}$ and $E_{rep2}$ are given in Table 4.10.

After repair application, tensile stresses in a repair patch (at the interface) gradually increase with time as the repair material continues to shrink and the elastic modulus increases. However, constant stress values are required to calculate creep at each age. Hence, at day 2, it can be assumed that the equivalent constant stress the material has been subjected to during its in-service lifetime is equal to half of the stress at day 2.

$$\sigma_{EC2} = \frac{\sigma_2}{t}$$
$$= 0.333/2 = 0.166 N/mm^2$$

Where $\sigma_{EC2}$ = equivalent constant stress applied to material over its first two days in service as a patch repair.

t = number of days repair material has been subject to equivalent constant stress

∴Equivalent constant stress in repair material at day 2 = 0.166 $N/mm^2$.

It is, therefore, assumed that the material will exhibit identical creep strain on day 2 as if it had been loaded with a constant stress of 0.166 $N/mm^2$ from the time of repair application. The unit (specific) creep value determined for Shucrete 1 earlier in this section corresponds to an applied stress of 1 $N/mm^2$ for 28 days. It was shown in equation 3-5 (hyperbolic expression) that the unit creep value of a repair material at any age can be obtained by using the 28 day unit creep value of the material. As the unit creep of Shucrete 1 at 28 days age is known, the actual creep (in microstrain) expected in the material at 2 days age can be determined using the following expression:

$$C_2 = \sigma_{EC2} C_{R2} C_{U28} \qquad \text{Eq 4-6}$$

Where $C_2$ = Creep of material at day 2

$C_{R2}$ = Ratio of unit creep at 2 days age over unit creep at 28 days age (from equation 3-5)

$C_{U28}$ = Unit creep in repair material at 28 days age after application

The expected Unit Creep ($C_{U28}$) of 99 microstrain is the creep which would occur over 28 days at a constant stress of 1 N/mm$^2$. Multiplying this value by the ratio between unit creep at 28 days age and 2 days age (equation 3-5) will give the creep which would occur over 2 days at a constant stress of 1 N/mm$^2$. This in turn, when multiplied by the average constant tensile stress the repair material has been subjected to between day 0 and 2, becomes the amount of creep that has occurred in the material at day 2.

We can obtain the ratio of 2 day creep to 28 day creep by using this relationship (equation 3-5):

$$\frac{C_2}{C_{28}} = \frac{t}{7.0162 + 0.701t}$$

Therefore,

$$\frac{C_2}{C_{28}} = \frac{2}{7.0162 + 0.701 * 2} = 0.238$$

Using unit creep in this way assumes that creep characteristics are the same in compression and tension (section 3.2.3.4.1)

Therefore, using equation 4-6:

$$C_2 = 0.166 * 0.238 * 99.3 = 3.9 \text{ microstrain}$$

Therefore, if the total shrinkage strain of the material is restrained at the interface, then the net tensile strain after relaxation due to creep becomes:

Strain at day 2 = shrinkage at day 2 – creep at day 2

= 24.5 – 3.9 = 20.6 microstrain.

However, if, as determined above, the actual strain in the repair material at day 2 is 20.6 microstrain, then the actual stress will be different from the 0.166 N/mm$^2$ assumed earlier. This stress, and subsequently the other variables can thus be recalculated iteratively.

$$\sigma_2 = \varepsilon_{sh2} * E_{rep2}$$
$$= \left(20.6*10^{-6}\right)\left(13.58*10^{3}\right)$$
$$= 0.278 N/mm^2$$

$$\sigma_{EC2} = \frac{\sigma_2}{t}$$
$$= 0.278/2 = 0.139 N/mm^2$$

Thus, a more accurate value of the creep at day two can be ascertained:

$$C_2 = \sigma_{EC2} C_{R2} C_{U28}$$

$C_2$ = 0.139 * 0.238 * 99.3 = 3.3 microstrain

Strain at day 2 = shrinkage at day 2 – creep at day 2 = 24.5 – 3.3 = 21.2 microstrain.

Again, the actual strain at day two is different from that assumed, hence this process is repeated iteratively until values for creep, strain and stress show no significant change during further iterations. At that stage the value of the strain represents most accurately the actual performance of the repair material.

Generally the strain after a series of iterations is between 90% and 99% of that before the iterations. A high creep value reduces the strain by 10% through the series of iterations; though a lower creep has little effect.

The final step required to complete the process of understanding the performance of the material at day 2 is to calculate the actual stress at day 2 after the relaxing effect of creep (as distinct from the average stress, which is calculated to aid determination of creep).

$$\sigma_2 = \varepsilon_{sh2} * E_{rep2}$$
$$= \left(21.1*10^{-6}\right)\left(13.58*10^{3}\right)$$
$$= 0.287 N / mm^2$$

After the first iteration, the strain in the repair was 21.2 microstrain. When computerised, a series of iterations were performed speedily and the final strain was 21.1 microstrain (shown in the above calculation).

### 4.9.3 The effect of creep at day 4:

The procedure for calculating the effect of creep on the strain at day 4 is similar to the above example for calculating creep at day 2, except for the method of calculating the average stress in the material.

For the purposes of deriving a general method applicable to all other ages, day 4 represents the age at which the amount of creep is being determined (the current age), and day 2 represents the previous age at which the amount of creep was determined (the previous age) i.e. the incremental step from day 2 to day 4. In this way any non-linear increase in stress can be allowed for.

$$\sigma_4 = E_4\left(\varepsilon_{sh4} - C_2\right) \qquad \text{Eq. 4-7}$$

$$\Delta\sigma_{2,4} = \sigma_4 - \sigma_2 \qquad \text{Eq. 4-8}$$

$$\sigma_{EC\,2,4} = \sigma_2 + \frac{\Delta\sigma_{2,4}}{2} \qquad \text{Eq. 4-9}$$

$$\sigma_{EC4} = \frac{\left(\sigma_{EC\,2} * t_{\sigma 1}\right) + \left(\sigma_{EC\,2,4} * t_{\sigma 2}\right)}{t_{\sigma 1} + t_{\sigma 2}} \qquad \text{Eq. 4-10}$$

Where $\sigma_4$ = stress in repair material at 4 days age

$\Delta\sigma_{2,4}$ = increase in stress from age 2 days to age 4 days

$\sigma_{EC\,2,4}$ = Equivalent constant stress in repair material between days 2 and 4

$\sigma_{EC\,2}$ = Equivalent constant stress between time of repair material application (day 0) and previous age (day 2)

$t_{\sigma 1}$ = Number of days under which the repair material endured associated stress

$t_{\sigma 2}$ = Number of days under which the repair material endured associated stress

$C_2$ = final creep at previous age (day 2) after the series of iterations

$\sigma_{EC\,4}$ = Equivalent constant stress between time of repair material application (day 0) and current age (day 4)

In Equations 4-7 to 4-10 the creep at day 2 ($C_2$) is considered to be the creep that has already occurred and hence it can be immediately deducted from the known shrinkage at day 4.

Figure 4.10 shows how, in effect, the average equivalent constant stresses from days 0 to 2 and from days 2 to 4 are further averaged to find the equivalent constant stress from days 0 to 4.

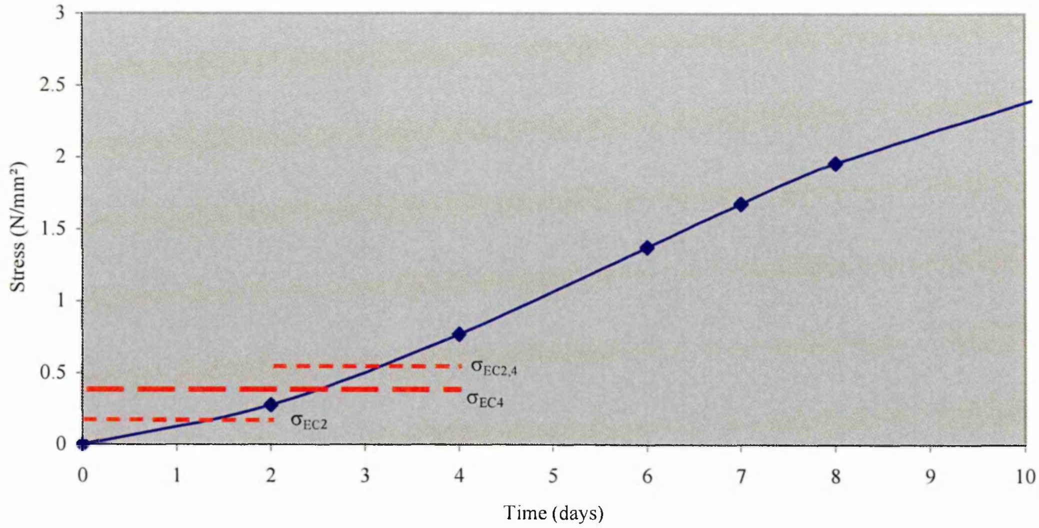



**Figure 4.10 Determining equivalent constant stress**

Using the data in Table 4.10 and substituting into Equation 4-7 gives:

$$\sigma_4 = 20.14 * 10^3 \left( (45.8 - 3.4) * 10^{-6} \right) = 0.853 N / mm^2$$

where $C_2$ (creep after iterations) = 3.4 microstrain

Substituting for $\sigma_4$ and $\sigma_2$ = 0.287 N/mm² (from section 4.9.2) into equation 4-8 gives:

$$\Delta\sigma_{2,4} = 0.853 - 0.287 = 0.563 N / mm^2$$

substituting for $\Delta\sigma_{2,4}$ in equation 4-9 gives:

$$\sigma_{EC2,4} = 0.287 + \frac{0.563}{2} = 0.572 N / mm^2$$

Substituting for $\sigma_{EC2,4}$ into equation 4-10 gives:

$$\sigma_{EC4} = \frac{(0.166 * 2) + (0.572 * 2)}{4} = 0.37 N / mm^2$$

Where 0.166 N/mm² = the equivalent constant stress from days 0 to 2 (section 4.9.2)

Determining the creep of Shucrete 1 at a constant stress of 0.37 N/mm² will give an accurate representation of the actual creep which has occurred under the varying stress to which the repair has actually been subjected over the period of four days after application. This stress can now be used to calculate the creep at day 4, and subsequently the strain. This strain can then be used to determine the reduced stress (due to relaxation) and the process of determining the actual creep can be continued iteratively until a stable value is found.

When conducting the iterations, the creep term from the previous time increment, $C_2$ (in the case above, the day 2 creep of 3.4 microstrain) is not subtracted from subsequent iterations as in the first operation – as its effect has already been included (that operation is only performed in the first calculation – deducting the creep already known to have occurred reduces the number of iterations required).

Hence, creep in the repair material at 4 days age, $C_4$, is given by:

$$C_4 = \sigma_{EC\,4} * C_{R4} * C_{U28} \qquad \text{Equation 4-11}$$

Where $\sigma_{EC4}$ = the equivalent constant stress from ages 0 to 4 days from Equation 4-10.

$C_4$ = creep in repair material at 4 days age

$C_{R4}$ = Ratio of unit creep at 4 days age over unit creep at 28 days age (from equation 3-5)

$C_{U28}$ = Unit creep in repair material at 28 days age after application from section 4.9.1

Substituting into Equation 4-11 gives:

$$C_4 = 0.31 * C_{R4} * C_{U28}$$

$$C_{R4} = \frac{C_4}{C_{28}} = \frac{4}{7.0162 + 0.701 * 4} = 0.407 \text{ (equation 3-5)}$$

$C_4 = 0.31 * 0.407 * 99.3$

$C_4 = 12.6$ microstrain

The values of Equivalent constant stress and Creep in the above equations are those obtained after a series of iterations (highlighted in green in Table 4.11).

When the process of iteration no longer significantly changes the values of creep and stress, the final actual strain at day 4 can be determined:

Strain at day 4 = shrinkage – creep = 45.8 – 12.6 = 33.2 microstrain

## 4.10 *Transfer of strain to the substrate.*

### 4.10.1 Consider day 14

On day 14, from Table 4.10:

$\varepsilon_{sh} = 118.6$ microstrain

Using the procedure outlined in section 4.9, the creep strain at 14 days, $C_{14}$, is calculated by a series of iterations to give:

$C_{14} = 73.6$ microstrain

Therefore, net Strain in the repair = 118.6 – 73.6 = 45 microstrain

Table 4.10 shows that, on day 14, using equation 4-5, 30.71% of the shrinkage strain in the repair material is transferred into the substrate concrete. Therefore:

Strain transferred to the substrate = 45 * 0.3071 = 13.80 microstrain

This amount of shrinkage strain is transferred from the repair material into the substrate concrete, therefore:

Residual restrained shrinkage strain in the repair material at day 14

$= 45 - 13.80 = 31.2$ microstrain.

The maximum tensile strain in the repair material (at the interface with the substrate) at each day is calculated using the method outlined above and is plotted alongside the tensile strain capacity of the repair material such that the performance of the material can be demonstrated graphically (a typical example is given in Figure 4.11). Using the equations that describe the development of repair material properties with age (equations 3-18 and 3-20), the tensile strain capacity with age is calculated and is plotted graphically up to a period of 300 days. The maximum tensile strain developed in the repair material due to restrained shrinkage and creep is also plotted in Figure 4.11. Due to the necessity for accounting for creep relaxation in the calculations (for maximum strain in the material), the increments in age at which creep is calculated have to be regular and small. This means that the technique used for predicting the tensile strain in repair patches requires a computer program due to the large amount of iterative calculation necessary.

The software was developed to resolve all the necessary equations, producing an output of two data sets: the tensile strain capacity of the repair material, and the restrained shrinkage tensile strain that occurs in the material. The software compares these sets of data and can inform the user if the tensile strain capacity of the repair material is exceeded. If the tensile strain capacity is exceeded, the time after application when the material will fail by cracking is also graphically determined.

Table 4.11 presents the performance of material *Shucrete 1*, at selected ages, over a 400 day period as provided by the calculations using the computer program.

The row 'stress 1' is highlighted to indicate that the calculation has included a deduction for creep which is already known to have occurred at the previous age at which calculations were performed. For example, at day 8, to determine 'stress 1', strain in the repair material is taken as the known shrinkage (from Table 4.10) minus the creep calculated at the end of day 7 (31.89 microstrain). This deduction is only carried out for the determination of 'stress 1' – the first cycle of a number of iterations. The penultimate row is highlighted to demonstrate that the figure includes strain transferred to the substrate concrete.

**Table 4.11 Performance of material *Shucrete* 1**

| Day | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 | 28 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tensile strength (N/mm²) | 0.00 | 2.2 | 3.1 | 3.6 | 3.8 | 3.9 | 4.3 | 4.5 | 4.5 | 4.7 | 4.7 | 4.9 | 5.12 | 5.27 | 5.32 | 5.34 | 5.36 | 5.37 | 5.38 | |
| Shrinkage (microstrain) | 0.00 | 24.6 | 45.8 | 64.1 | 72.4 | 80.2 | 107.2 | 118.5 | 123.8 | 146.5 | 150.5 | 173.9 | 218.80 | 261.86 | 280.24 | 290.43 | 296.91 | 301.40 | 304.68 | 3( |
| E (kN/mm²) | 0.00 | 13.6 | 20.1 | 24.0 | 25.4 | 26.6 | 29.7 | 30.8 | 31.2 | 32.8 | 33.1 | 34.4 | 36.26 | 37.56 | 38.02 | 38.25 | 38.39 | 38.49 | 38.56 | 3 |
| $C/C_{28}$ | 0.00 | 0.24 | 0.41 | 0.53 | 0.59 | 0.63 | 0.78 | 0.83 | 0.86 | 0.95 | 0.97 | 1.05 | 1.19 | 1.30 | 1.34 | 1.36 | 1.37 | 1.38 | 1.39 | 1 |
| Shrinkage transfer (%) | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 19.06 | 30.71 | 35.61 | 53.89 | 56.66 | 71.16 | 92.14 | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 | 1( |
| Restrain shrinkage stress | 0.00 | 0.17 | 0.36 | 0.53 | 0.56 | 0.62 | 0.93 | 0.90 | 0.90 | 1.08 | 1.02 | 1.19 | 1.25 | 1.38 | 1.46 | 1.51 | 1.54 | 1.56 | 1.58 | 1 |
| creep | 0.00 | 3.94 | 14.45 | 27.87 | 32.60 | 39.22 | 72.03 | 74.07 | 76.80 | 102.07 | 97.97 | 123.73 | 147.81 | 177.85 | 193.95 | 203.55 | 209.83 | 214.25 | 217.52 | 22 |
| Strain (shrinkage – creep) | 0.00 | 20.66 | 31.30 | 36.26 | 39.84 | 41.03 | 35.15 | 44.48 | 47.00 | 44.45 | 52.50 | 50.14 | 70.99 | 84.00 | 86.29 | 86.89 | 87.08 | 87.15 | 87.17 | 8 |
| stress 1* | 0.00 | 0.14 | 0.30 | 0.46 | 0.55 | 0.61 | 0.77 | 0.87 | 0.90 | 0.98 | 1.02 | 1.07 | 1.24 | 1.38 | 1.46 | 1.51 | 1.54 | 1.56 | 1.58 | 1 |
| creep 1 | 0.00 | 3.31 | 12.20 | 24.62 | 31.82 | 38.45 | 59.22 | 72.05 | 76.41 | 92.14 | 97.75 | 112.09 | 146.44 | 177.55 | 193.84 | 203.49 | 209.80 | 214.23 | 217.50 | 22 |
| strain1 | 0.00 | 21.30 | 33.55 | 39.51 | 40.63 | 41.79 | 47.96 | 46.49 | 47.39 | 54.38 | 52.71 | 61.78 | 72.36 | 84.30 | 86.40 | 86.94 | 87.11 | 87.17 | 87.18 | 8 |
| stress 2 | 0.00 | 0.14 | 0.31 | 0.48 | 0.55 | 0.61 | 0.83 | 0.88 | 0.90 | 1.02 | 1.02 | 1.12 | 1.24 | 1.38 | 1.46 | 1.51 | 1.54 | 1.56 | 1.58 | 1 |
| creep 2 | 0.00 | 3.41 | 12.66 | 25.31 | 31.90 | 38.53 | 64.12 | 72.42 | 76.44 | 95.99 | 97.77 | 117.31 | 146.73 | 177.59 | 193.85 | 203.50 | 209.80 | 214.23 | 217.50 | 22 |
| strain 2 | 0.00 | 21.19 | 33.09 | 38.82 | 40.54 | 41.71 | 43.06 | 46.13 | 47.36 | 50.53 | 52.70 | 56.56 | 72.06 | 84.27 | 86.39 | 86.94 | 87.11 | 87.17 | 87.18 | 8 |
| stress 3 | 0.00 | 0.14 | 0.31 | 0.47 | 0.55 | 0.61 | 0.81 | 0.88 | 0.90 | 1.00 | 1.02 | 1.10 | 1.24 | 1.38 | 1.46 | 1.51 | 1.54 | 1.56 | 1.58 | 1 |
| creep 3 | 0.00 | 3.39 | 12.57 | 25.17 | 31.89 | 38.53 | 62.24 | 72.35 | 76.44 | 94.50 | 97.77 | 114.97 | 146.67 | 177.58 | 193.85 | 203.50 | 209.80 | 214.23 | 217.50 | 22 |

| Day | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 | 21 | 28 | 50 | 100 | 150 | 200 | 250 | 300 | 350 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| strain3 | 0.00 | 21.21 | 33.19 | 38.97 | 40.55 | 41.72 | 44.94 | 46.19 | 47.36 | 52.02 | 52.70 | 58.90 | 72.13 | 84.27 | 86.39 | 86.94 | 87.11 | 87.17 | 87.18 |
| stress 4 | 0.00 | 0.14 | 0.31 | 0.47 | 0.55 | 0.61 | 0.82 | 0.88 | 0.90 | 1.01 | 1.02 | 1.11 | 1.24 | 1.38 | 1.46 | 1.51 | 1.54 | 1.56 | 1.58 |
| creep 4 | 0.00 | 3.40 | 12.59 | 25.20 | 31.89 | 38.53 | 62.96 | 72.37 | 76.44 | 95.07 | 97.77 | 116.02 | 146.68 | 177.58 | 193.85 | 203.50 | 209.80 | 214.23 | 217.50 |
| strain 5 | 0.00 | 21.21 | 33.17 | 38.94 | 40.55 | 41.72 | 35.79 | 32.00 | 30.50 | 23.72 | 22.84 | 16.68 | 5.67 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Actual stress | 0.00 | 0.29 | 0.67 | 0.93 | 1.03 | 1.11 | 1.06 | 0.98 | 0.95 | 0.78 | 0.76 | 0.57 | 0.21 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

*First iteration calculation

E = Elastic modulus

C = Creep at age $x$

$C_{28}$ = Creep at age 28 days

## 4.11 Tensile strain capacity

A simplistic strain capacity value can be obtained using tensile strength and elastic modulus.

$$\varepsilon_{cap} = \frac{f_t}{E}$$

Therefore on day 4, using the values for Shucrete 1 given in Table 4.10:

$$\varepsilon_{cap4} = \frac{3.09 * 10^6}{20.14 * 10^3}$$

$$\varepsilon_{cap4} = 153 \text{ microstrain}$$

This tensile strain capacity determined at each age is shown in Table 4.12:

**Table 4.12 Development of Tensile Strain Capacity in *Shucrete* 1 (microstrain)**

| day | 0 | 2 | 4 | 6 | 7 | 8 | 12 | 14 | 15 | 20 |
|---|---|---|---|---|---|---|---|---|---|---|
| tensile strain capacity | 0 | 159 | 153 | 150 | 149 | 148 | 146 | 145 | 145 | 144 |

| day | 21 | 28 | 50 | 100 | 150 | 200 | 250 | 300 | 350 | 400 |
|---|---|---|---|---|---|---|---|---|---|---|
| tensile strain capacity | 143 | 142 | 141 | 140 | 140 | 140 | 140 | 140 | 140 | 139 |

The data presented in Table 4.11 and Table 4.12 represents the performance of the repair material. It can be used to determine if Shucrete 1 is a suitable repair material for the repair patch in Edinburgh in December. The development of tensile strain in the repair material with time (Table 4.11 – penultimate row) can be now be plotted alongside the development of tensile strain capacity in the repair material with time (Figure 4.11).

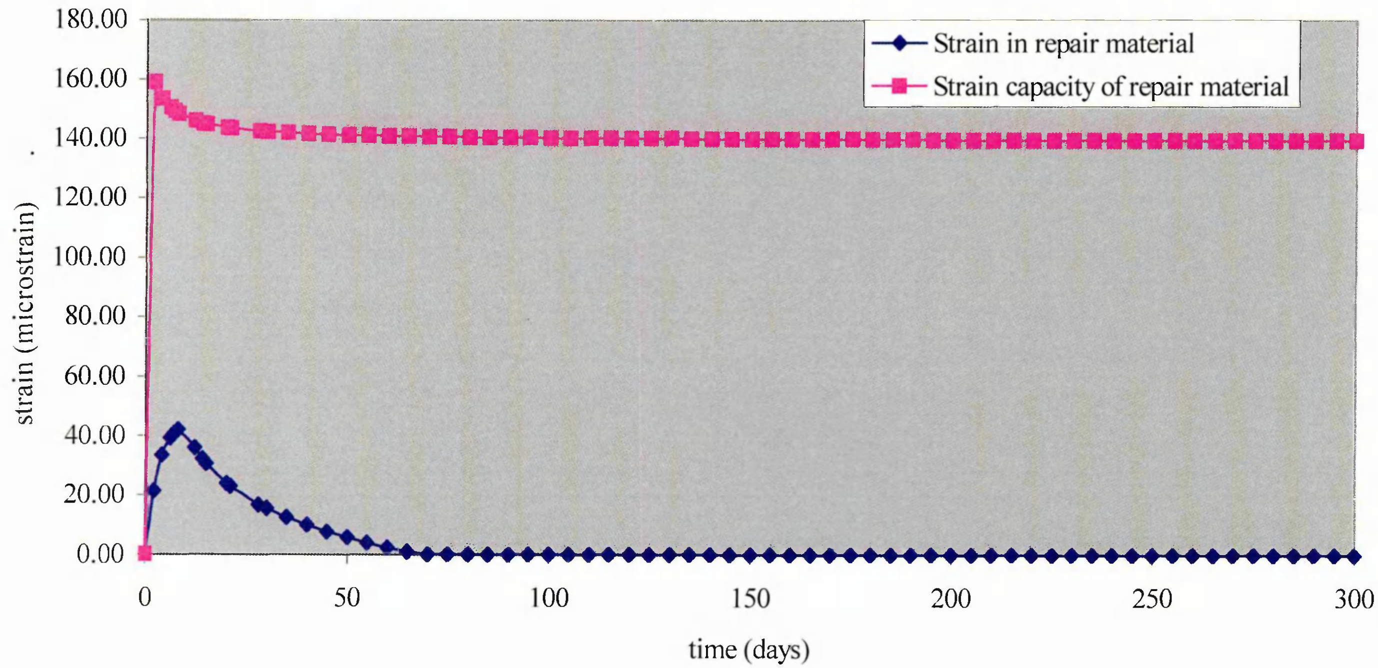



**Figure 4.11 Performance of material Shucrete 1**

Figure 4.11 shows that using *Shucrete 1* for this repair situation would result in a successful repair. The tensile strain capacity of the material will not be exceeded by the tensile strain that arises through restrained shrinkage. Therefore, the material will not crack. It will be visually amenable and it will protect the reinforcement it covers. It will also be able to share load with the surrounding substrate.

The strain which occurs in Shucrete 1 due to restrained shrinkage is represented by the blue line in Figure 4.11. Typically, for any repair material and substrate combination, as the material begins to shrink, it is restrained at the interface by the substrate, this restraint causes tensile stress in the repair. As a result of this stress, a natural relaxation through creep occurs. In the theoretical example detailed in this chapter, the shrinkage strain (and, therefore, restrained shrinkage tensile stress) continues to grow up until day 15. Around day 15, the developing elastic modulus of the repair material has become as stiff as the substrate concrete. When this happens, the shrinkage strain in the repair material can begin to be transferred into the substrate concrete. Initially, strain is transferred from the repair material into the substrate in relatively small amounts, then more and more of the strain is transferred until, when the elastic modulus of the repair material is 1.32 times the elastic modulus of the substrate concrete, all the strain in the repair material caused by restrained shrinkage is transferred into the substrate concrete. On day 50, the modular ratio has reached the optimum value of 1.32, and all restrained shrinkage is transferred.

There are a number of factors which could have caused Shucrete 1 to fail: a lower tensile strain capacity, a lower elastic modulus, or the substrate concrete having a higher elastic modulus.

## 4.12 Estimation of Creep using shrinkage data

### 4.12.1 Introduction

Often, when selecting repair materials, only limited information of their properties will be available . The property that is least likely to be provided by the manufacturer is 'creep'. As discussed in this chapter, creep will relax any strains that appear in a repair material, and all concrete repair materials will exhibit the beneficial effects of creep to some extent. Where no information on creep is provided by a repair material supplier (which is frequently the case), the software created in the project will make a conservative estimate of the creep, based solely on the shrinkage properties of a repair material – which generally will be provided by the manufacturer.

Data on the relationship between creep and shrinkage has been collated from eight sets of references in literature. For each reference, it was necessary to check if the tests to determine shrinkage and creep were performed under similar conditions. If this was not the case, modifications are made to the results. The preferred conditions are 28 day Shrinkage determined at 20°C and 55% relative humidity; creep specimens cured for 28 days, then subjected to 28 days of loading at a stress/strength ratio of 30%.

It will also be noted if the repair material was polymer modified.

### 4.12.2 O'Flaherty[58]

This data was obtained using the same materials for which the software routines in the previous chapter were determined. As such the values were obtained under the recommended conditions of:

28 day Shrinkage at 20°C and 55% relative humidity.

28 day Creep at 30% stress/strength ratio.

The creep and shrinkage data are listed in Table 4.13:

**Table 4.13 Creep and free shrinkage data for repair materials[58]**

| Material | Shrinkage (microstrain) | Creep (microstrain) | Polymer modified |
|---|---|---|---|
| G1 | 560 | 294 | yes |
| G2 | 1200 | 342 | yes |
| G4 | 310 | 546 | yes |
| G5 | 1030 | 1088 | yes |
| G6 | 920 | 1070 | yes |
| L1 | 500 | 680 | - |
| L3 | 320 | 580 | - |
| L4 | 580 | 428 | - |
| L5 | 450 | 434 | - |
| S1 | 600 | 408 | - |
| S2 | 630 | 456 | - |
| S3 | 440 | 586 | yes |
| S4 | 270 | 368 | - |

See Table 3.13 for more detailed information about the constituents of the repair materials listed in Table 4.13.

### 4.12.3 Mangat & Limbachiya[56]

The data given in Table 4.14 were obtained under the recommended conditions of:

28 day Shrinkage at 20°C and 55% relative humidity.

28 day Creep at 30% stress/strength ratio.

**Table 4.14 Creep and free shrinkage data for repair materials[56]**

| Material | Shrinkage (microstrain) | Creep (microstrain) | Polymer modified |
|---|---|---|---|
| A | 320 | 500 | - |
| B | 300 | 550 | - |
| C | 440 | 1000 | yes |

Material A was a blend of Portland cement, graded aggregates (maximum size 5mm) and additives to impart controlled expansion. Material B was a mineral based cementituous material with no aggregate sized particles or additives. Material C was a single component cementituous mortar incorporating microsilica, fibre reinforcement, and styrene acrylic copolymer.

### 4.12.4 Mangat & Azari[106]

The data listed in Table 4.15 were obtained under the recommended conditions.

28 day Shrinkage at 20°C and 55% relative humidity.

28 day Creep at 30% stress/strength ratio.

**Table 4.15 Creep and free shrinkage data for repair materials[106]**

| Material | Shrinkage (microstrain) | Creep (microstrain) | Polymer modified |
|---|---|---|---|
| $a_0$ | 400 | 620 | - |

### 4.12.5 Evans [107]

The data listed in Table 4.16 were not obtained under the recommended conditions. They require modification. The data was obtained under the following conditions:

Between 193 and 200 day Shrinkage at 20°C and 55% relative humidity.

28 day Creep at 25% stress/strength ratio.

**Table 4.16 Creep and free shrinkage data for repair materials[107]**

| Mat. | Shrinkage (microstrain) | Shrinkage at age (days) | Estimated 28 day shrinkage (microstrain) | Creep strain (microstrain) | Creep strain at age (days) | Estimated 28 day creep (microstrain) | Stress strength ratio (%) | Estimated 28 day creep at 30% stress strength ratio (microstain) | Polymer modified |
|---|---|---|---|---|---|---|---|---|---|
| Col. 1 | Col. 2 | Col. 3 | Col. 4 | Col. 5 | Col. 6 | Col. 7 | Col. 8 | Col. 9 | Col. 10 |
| 0 | 870 | 197 | 491 | 599 | 90 | 467 | 25 | 560 | Concrete |
| 15-1 | 796 | 200 | 448 | 824 | 90 | 642 | 25 | 770 | yes |
| 25-1 | 703 | 199 | 396 | 936 | 90 | 729 | 25 | 875 | yes |
| 25-2 | 806 | 197 | 455 | 580 | 90 | 452 | 25 | 542 | yes |
| 25-3 | 801 | 197 | 452 | 627 | 90 | 488 | 25 | 586 | yes |
| 25-4 | 660 | 193 | 373.2 | 515 | 90 | 401.2 | 25 | 481.40 | yes |

Material *15-1* achieved its free shrinkage of 796 microstrain at 200 days (Table 4.16). The 200 day shrinkage value is converted to the required 28 shrinkage ($\varepsilon_{28}$) using Equation 3-8 in section 3.4.2.4:

$$S_r = \frac{\varepsilon}{\varepsilon_{28}} = \frac{t}{12.292 + 0.5017t}$$

$$\frac{796}{\varepsilon_{28}} = \frac{200}{12.292 + 0.5017 * 200}$$

$$\varepsilon_{28} = 448 \text{ microstrain}$$

This 28 day value of shrinkage is given in Table 4.16 (column 4 – material 15-1). The material achieved its creep value of 824 microstrain at 90 days. This was converted to the required 28 creep using Equation 3-5 in section 3.4.2.2 as follows:-

$$\frac{C}{C_{28}} = \frac{t}{7.0162 + 0.701t}$$

$$\frac{824}{C_{28}} = \frac{90}{7.0162 + 0.701 * 90}$$

$$C_{28} = 642 \text{ microstrain}$$

This value is given in Table 4.16, column 7.

This Creep value was achieved at a stress/strength ratio of 25%. This figure requires further modification to determine creep at 30% stress/strength ratio. Using a linear relationship between creep and stress/strength ratio (section 4.5), gives :-

$$C_{28:30\%s/s} = \frac{642}{25\%} * 30\%$$

$$C_{28:30\%s/s} = 770 \text{ microstrain}$$

Where $C_{28:30\%s/s}$ is the 28 day creep achieved at a 30% stress strength ratio.

### 4.12.6 Limbachiya[108]

The data listed in Table 4.17 were obtained under the recommended conditions of:

28 day Shrinkage at 20°C and 55% relative humidity.

28 day Creep at 30% stress/strength ratio.

**Table 4.17 Creep and free shrinkage data for repair materials[108]**

| Material | Shrinkage (microstrain) | Creep (microstrain) | Polymer modified |
|---|---|---|---|
| qa | 530 | 500 | - |
| qb | 630 | 530 | - |
| qc | 860 | 1010 | yes |

### 4.12.7 Poston, Kesner, McDonald, Vaysburd[68,76]

The data are listed in Table 4.18 and were not obtained under the recommended conditions. They require modification. The curing conditions of samples were:

28 day Shrinkage at 20°C and 50% relative humidity.

Creep specimens were loaded for one year and results given as specific creep (creep per 1 $lb/in^2$)

**Table 4.18 Creep and free shrinkage data for repair materials**[76]

| Material | Shrinkage 50%RH (microstrain) | Shrinkage 55%RH (microstrain) | 28 day creep (microstrain) | Polymer modified | Material |
|---|---|---|---|---|---|
| 1 | 178 | 165 | 568 | - | Portland cement mortar |
| 2 | 201 | 187 | 1026 | - | Portland cement concrete |
| 3 | 339 | 315 | 2474 | yes | Polymer modified concrete |
| 4 | 293 | 272 | 1342 | - | Portland cement concrete |
| 5 | 305 | 283 | 870 | - | Portland cement mortar |
| 6 | 429 | 398 | 1426 | yes | Polymer modified mortar |
| 7 | 479 | 445 | 2892 | yes | Polymer modified mortar |
| 8 | 391 | 363 | 1773 | yes | Polymer and fibre modified mortar |
| 9 | 429 | 398 | 1218 | - | Portland cement concrete |
| 10 | 1779 | 1652 | 2015 | yes | Polymer modified mortar |
| 11 | 301 | 280 | 704 | - | Portland cement concrete |
| 12 | 258 | 240 | 1928 | yes | Polymer modified mortar |

Conversion of shrinkage

In accordance with section 4.4 for materials cured below 70%RH, each 1% reduction in RH will increase shrinkage by 2%. The shrinkage specimens were cured at a relative humidity of 50%. A 20 percentage increase would bring this value up to the datum relative humidity of 70%, and result in a reduction of 40%.

For material 2 (Table 4.18):

$$\varepsilon_{RH=70\%} = \frac{201}{1.4}$$
$$\varepsilon_{RH=70\%} = 144 \text{ microstrain}$$

Furthermore, a 15 percentage point reduction in relative humidity would provide the required relative humidity of 55%. This 15 percentage point reduction would effect an increase in shrinkage of 30% (in accordance with 4.4).

$\varepsilon_{RH=55\%} = 144 * 1.3$
$\varepsilon_{RH=55\%} = 187$ microstrain

Conversion of creep

Table 4.18 provides the creep at one year for specimens subjected to a constant loading of 1 lb/in$^2$.

For material 3[68]:

1 year specific creep at 1 lb/in$^2$ = 1.8 microstrain

1 year specific creep 1 N/mm$^2$ ($C_{U365}$) = 1.8 * 145 = 261 microstrain

Where 145 is the factor to convert from creep at 1 lb/in$^2$ to creep 1 N/mm$^2$

In accordance with Equation 3-5 in section 3.4.2.2:

$$\frac{C_{U365}}{C_{U28}} = \frac{365}{(7.0162 + (0.701 * 365))}$$

$C_{U365} = 261$ microstrain

Therefore,

$$C_{U28} = \frac{261}{365/(7.0162 + (0.701 * 365))}$$

$C_{U28} = 188$ microstrain

$f_{c28}$[68]= 6360 lb/in$^2$

$f_{c28} = 6360 / 145$ N/mm$^2$

$f_{c28} = 43.9$ N/mm$^2$

Therefore:

The 28 day creep at 30% stress/strength ratio, $C_{28:30\%}$:

$C_{28} = (0.3*43.9)* C_{u28}$

$C_{28} = (0.3*43.9)* 188$

$C_{28} = 2474$ microstrain

## 4.12.8 Emberson & Mays[61]

The data are listed in Table 4.19 and were not obtained under the recommended conditions. They require modification.

495 day Shrinkage at 20°C and 50% relative humidity.

495 day Creep specimens.

**Table 4.19 Creep and shrinkage data for repair materials[61]**

| Mat. | Shrinkage @ 495 days (microstrain) | Estimated Shrinkage @28 days (microstrain) | Creep @495 days (microstrain) | Estimated Creep @28 days (microstrain) | Polymer Modified |
|---|---|---|---|---|---|
| D | 200 | 105 | 230 | 165 | yes |
| E | 420 | 221 | 1600 | 1144 | yes |
| G | 440 | 232 | 400 | 286 | - |
| H | 280 | 147 | 960 | 687 | - |
| I | 210 | 111 | 580 | 415 | yes |

Material D was an SBR-modified cementituous mortar, E a vinyl acetate modified cementituous mortar, G an OPC/sand mortar, H a high allumina cement mortar and I a flowing concrete.

## 4.12.9 Neville[104]

This reference provided the creep versus shrinkage relationship of 52 concrete mixes of normal strength. Some values were not obtained under the recommended conditions:

365 day Shrinkage at 20°C and 50% relative humidity.

365 day Creep at 50% s/s ratio.

Data from this reference was modified in accordance with the procedures used in this section. Data from the eight references described above are plotted in Figure 4.12, which should be read in conjunction with Table 4.20.

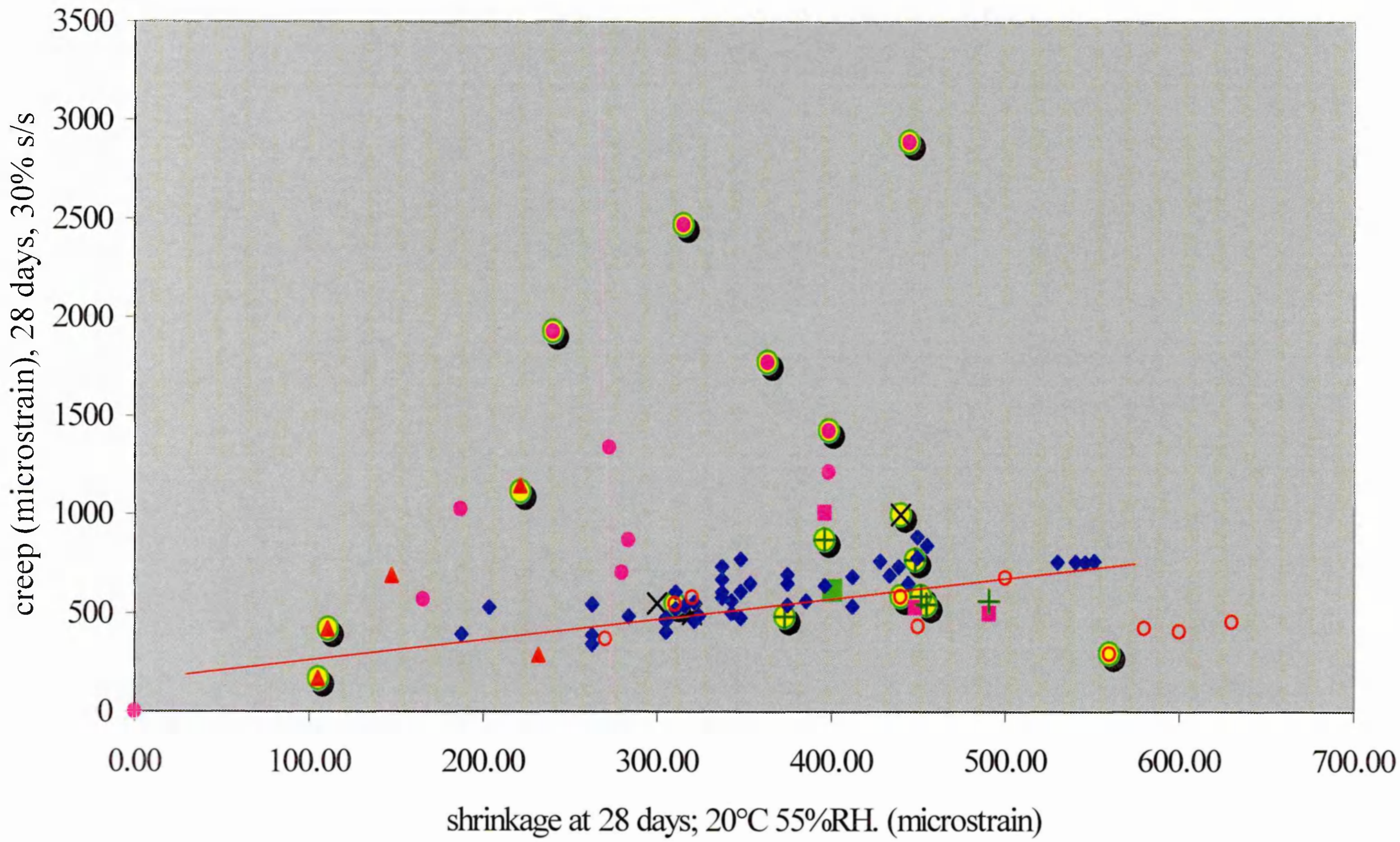



**Figure 4.12 Relationship between shrinkage and creep**

**Table 4.20 Key to Figure 4.12**

| Symbol | Material Type | Reference |
|---|---|---|
| ○ | Repair materials | O'Flaherty[58] |
| ◉ | Polymer modified repair materials | O'Flaherty[58] |
| ■ | Repair materials | Mangat & Limbachiya[56] |
| ■ | Repair material | Mangat & Azari[106] |
| + | Concrete | Evans[107] |
| ⊕ | Polymer modified Repair materials | Evans[107] |
| × | Repair materials | Limbachiya[108] |
| ⊗ | Polymer modified repair materials | Limbachiya[108] |
| ● | Repair materials | Poston et al[68] |
| ◉ | Polymer modified repair materials | Poston et al[68] |
| ▲ | Repair materials | Emberson & Mays[61] |
| ▲ (circled) | Polymer modified repair materials | Emberson & Mays[61] |
| ◆ | Concrete | Neville[104] |

The red line in Figure 4.12 will be used to determine the creep properties of a repair material when only the shrinkage properties are known. It is not a line of best fit but has been positioned so to provide a conservative estimate of creep (because the relaxing effect of creep is beneficial in reducing tensile stress which arise in repair patches). If a line of best fit is added to the graph, a coefficient of variance in the region of $r^2 = 0.4$ is the result. This shows that the link between shrinkage and creep is, at best, tenuous – especially when all the data is considered. In particular, the polymer modified repair materials from Poston et al[68] exhibit creep at levels far higher than the prescribed relationship would predict. It is of note that polymer modified materials from the other references do appear to follow the prescribed relationship with a reasonable degree of accuracy. On balance, Figure 4.12 shows that the prescribed relationship between creep and shrinkage is adequate for the purpose it is required for. Therefore, in accordance with Figure 4.12, the minimum creep at 28 days due to a 30% stress/strength ratio will be estimated using this relationship (shown by the red line)

Creep = (Shrinkage*1.1)+20 Equation 4-12

Clearly, on occasions, some materials will exhibit a creep greater than the equation estimates. This will only serve to relax the strain in the material further. It is, therefore, accepted that if manufacturers do not provide creep data, it is more prudent and accurate to assume a conservative value for the creep than may occur in a material instead of ignoring creep altogether and neglecting creep relaxation.

## 4.13 Summary of guidelines for selection of reinforced concrete repair materials.

The procedure detailed in this section will assess the performance of a repair material selected to repair reinforced concrete. The procedure examines the development of properties in a repair material with time. The procedure needs to be performed at regular time intervals. For example, in assessing the performance of a concrete repair over 400 days, the calculations need to be performed at a minimum of every five days, and every two days during the first fifteen days. This is to take account of the effect of relaxation through creep. Thus, the procedure lends itself solely to the application of a computer program, which will speedily perform the necessary iterations. The steps are as follows:

1 Obtain properties of repair material

Obtain these key properties of the repair material from literature provided by the manufacturer:

*1.1 Key Properties*

- Compressive Strength at 28 days ($N/mm^2$)
- Tensile Strength or Modulus of Rupture at 28 days ($N/mm^2$)

1.1.1 If Modulus of Rupture is provided then:

Tensile Strength = Modulus of Rupture / 1.7

- Free Shrinkage at 28 days (microstrain)
- Elastic Modulus at 28 days ($kN/mm^2$)

*1.2 Optional Properties*

If creep information is not provided by the repair material manufacturer, refer to section 1.2.1. The required information is:

- Creep strain at 28 days (microstrain)
- Stress / strength ratio endured by creep sample

### 1.2.1 Estimating Creep

The following equation will estimate the 28 day creep of the repair material based on its 28 day shrinkage value:

Creep = (shrinkage * 1.1) + 20 Equation 4-12

## 2 Obtain properties of substrate concrete

Extract Core from substrate concrete.

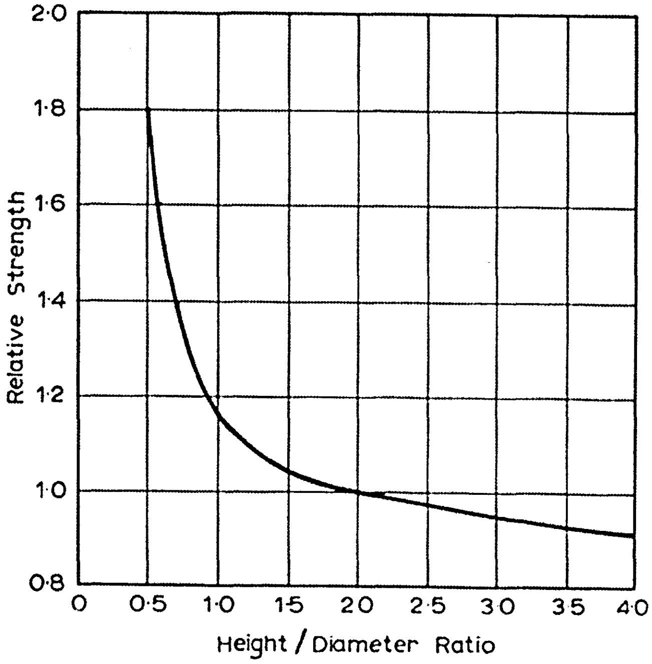
Using standard test methods determine:

Compressive Strength ($N/mm^2$)

Elastic Modulus ($kN/mm^2$)

### 2.1 Height/Diameter ratio modification

If the laboratory have not applied the height/diameter modifier for relative strength, it can be done using the graph below:




Strength of core = Unmodified strength * Relative Strength factor (N/mm$^2$)

2.2 Core to Cube modification

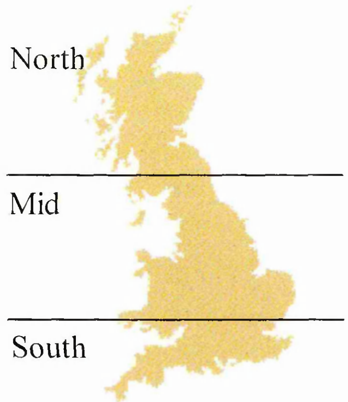
Apply the modification for conversion of core strength to cube strength

Cube strength = Core strength / 0.8

3 Climate Modification

3.1 Using the following table and map, obtain the local climatic conditions of the proposed repair:

| | Dec - Feb | Mar - May | Jun - Aug | Sep - Nov |
|---|---|---|---|---|
| North | 4° | 9° | 15° | 11° |
| Mid | 5° | 10° | 16° | 12° |
| South | 8° | 11° | 17° | 14° |




| | Dec - Feb | Mar - May | Jun - Aug | Sep - Nov |
|---|---|---|---|---|
| North | 85% | 70% | 70% | 85% |
| Mid | 85% | 70% | 70% | 85% |
| South | 85% | 75% | 75% | 85% |

3.2 Modify shrinkage by temperature

Assuming the shrinkage property was obtained by a standard test, it can be modified to obtain the shrinkage at a datum temperature of 15°C:

$$Shrinkage_{15°C} = \frac{Shrinkage}{1.08}$$

3.2.1 Obtain the difference in field temperature and datum temperature:

$$g = \frac{field\ temperature\ \text{-}\ 15°}{100}$$

3.2.2 Calculate the field shrinkage modified for temperature:

$$Shrinkage_{f(temp)} = shrinkage_{15°C} * (1 + g)$$

3.3 Modify Shrinkage by relative humidity

The measured shrinkage can be modified to the shrinkage at a datum relative humidity of 70%.

$$shrinkage_{RH70\%} = \frac{Shrinkage_{f(temp)}}{1.4}$$

3.3.1 Obtain the difference between field RH and datum RH

$$h = \frac{(\%RH \text{ at time of repair} - 70\% \; RH) * 3}{100}$$

3.3.2 Calculate the field shrinkage modified for relative humidity

$$Shrinkage_{f(temp\&RH)} = shrinkage_{RH70\%} * (1 - h)$$

3.4 Modify creep for temperature

Assuming the creep property of the material was obtained by a recommended test (curing samples at 20°C), the creep can be modified to obtain the creep at a datum temperature of 15°C.

$$Creep_{15°C} = \frac{Creep_{20°C}}{1.0625}$$

3.4.1 Calculate the field creep modified for temperature

$$Creep_{f(temp)} = Creep_{15°C} * (1 + (1.25 * g))$$

3.5 Modify creep for relative humidity

Calculate creep correction factor for RH at which test on sample was conducted (assume 50% if data unavailable)

$$k_{rhlab} = -0.007(RH\%_{lab}) + 1.3$$

where $RH\%_{lab}$ = Relative humidity whilst curing creep specimen

Calculate creep correction factor for RH at time and place of application

$$k_{rhfield} = -0.007(RH\%_{field}) + 1.3$$

Creep modified for relative humidity:

$$Creep_{f(rh+temp)} = \frac{creep_{f(temp)}}{k_{rhlab}} * k_{rhfield}$$

3.6 Modify creep for relative test specimen and repair sizes

Determine creep modification factor for repair

$$k_{sizerep} = -0.3057\ln(d) + 2.8375$$

where d = depth of repair (mm)

Determine creep modification factor for test specimen:

$$k_{sizelab} = -0.308\ln(v/s) + 2.6367$$

where v/s = volume/surface ratio of test specimen (if unknown use 25mm [ASTM C 512])

Determine creep modified for relative sizes of specimen and repair:

$$Creep_{rep} = \frac{creep_{f(Rh+temp)}}{k_{sizerep}} * k_{sizelab}$$

4 Volume/Surface ratio shrinkage modification

4.1 Obtain Volume / Surface ratio of laboratory specimen from which shrinkage data was measured.

If unknown use

(25x25x285)/((25*25*2)+(25*285*4)) = 6 mm (volume/surface$_{lab}$)

4.2 Determine Volume / Surface ratio of repair (volume/surface$_{field}$)

4.3 Calculate the relative shrinkage of both laboratory and field materials.

$$\mu_{lab} = 10.779e^{-0.005(volume/surface_{lab})}$$

$$\mu_{field} = 10.779e^{-0.005(volume/surface_{field})}$$

$$j = \frac{\mu_{lab}}{\mu_{field}}$$

$$\varepsilon_{28} = \frac{Shrinkage_{f(temp\&RH)}}{j}$$

5 The development of properties

The development of certain properties and interactions in the repair material can be plotted. Properties should be determined and tabulated on days; 0, 2, 4, 6, 7, 8, 12, 14, 15, 20, 21, 28 then a maximum of every 5 days for as long as the properties need to be determined. Leaving a gap between ages at which the calculations are performed will result in the procedure not taking account of the constant relaxing effect of creep on the tensile strains

5.1 Development of Tensile Strength on days t = 0 to 400

$$\frac{f_t}{f_{t28}} = \frac{t}{2.7975 + 0.9216.t}$$

Where $f_t$ = Tensile strength on any day, t.

$f_{t28}$ = Tensile strength of repair material at 28 days

t = time in days

5.2 Development of shrinkage on days t = 0 to 400

$$\frac{\varepsilon}{\varepsilon_{28}} = \frac{t}{12.292 + 0.5017t}$$

Where $\varepsilon$ = shrinkage strain on any day, t.

$\varepsilon_{28}$ = shrinkage strain of repair material at 28 days

t = time in days

5.3 Development of Elastic Modulus on days t = 0 to 400

$$\frac{E}{E_{28}} = \frac{t}{3.2637 + 0.8725t}$$

Where E = Elastic Modulus on any day, t.

$E_{28}$ = Elastic Modulus of repair material at 28 days

t = time in days

5.4 Shrinkage Transfer on days t = 0 to 400

As the Elastic Modulus of the repair material develops, some shrinkage strain may be transferred from the repair material into the substrate concrete.

5.4.1 Determine Modular ratio on days t = 0 to 400

Modular Ratio = $E_{rm} / E_{sub}$

Where $E_{rm}$ = Elastic modulus of repair material at day t.

$E_{sub}$ = Elastic Modulus of substrate concrete.

5.4.2 Determine transfer of strain (%) on days t = 0 to 400

$$\lambda = \frac{\left(E_{rep}/E_{sub} - 1\right)}{0.0032}$$

where $\lambda$ = shrinkage transferred (%)

$E_{rep}/E_{sub}$ = Modular Ratio

NB. $100 \geq \lambda \geq 0$

6 Unit Creep

Unit creep is the creep per load of 1 $N/mm^2$

6.1 Determine load applied to laboratory creep sample

Creep specimens are loaded at 30% of their 28 day strength.

Applied load ($N/mm^2$) = 30% $f_{c28}$

6.2 Determine Unit Creep

Unit $Creep_{initial}$ (microstrain) = $Creep_{rep}$ / Applied load

6.3 Modify Unit Creep to allow for early age loading

Unit Creep = Unit $Creep_{initial}$ * 1.83

7 The effect of Creep

To calculate the effect of creep the stress at day t is required.

NB. In the following equations, referring to the 'previous' day, means the previous day by way of increment. For example, if the current day is day 300, then the previous 'day' was day 295 (using the maximum increment of 5 days)

$$\text{Stress}_t = E_t\left(Shrinkage_t - creep_{t(prev)}\right) \text{.......(1)}$$

$$\text{Stress}_{EC(t \text{ to } t(prev))} = \text{Stress}_{t(prev)} + \frac{\text{stress}_t - stress_{t(prev)}}{2} \text{.......(2)}$$

$$\text{Stress}_{EC(o \text{ to } t)} = \frac{\left(\text{Stress}_{EC(prev\ o\ to\ t)} * t_{prev}\right) + \left(\text{Stress}_{EC(t \text{ to } t(prev))} * (t - t_{prev})\right)}{t} \text{......(3)}$$

Where $\text{Stress}_{EC}$ = equivalent constant stress

t = current day

$\text{Stress}_{EC(prev\ 0\ to\ t)}$ = Result of final equation (3) from previous t

t(prev) = previous day (by increment)

$$\frac{C_t}{C_{28}} = \frac{t}{7.0162 + 0.701 * t}$$

$$\text{Creep}_t = \text{Stress}_{EC(o \text{ to } t)} * \frac{C_t}{C_{28}} * \text{Unit Creep}$$

8 Calculating Strain in the repair material for t = 0 to 400

$\text{Strain}_t = \text{shrinkage}_t - \text{creep}_t$

A degree of relaxation through creep has occurred, this has had an effect on the strain in the repair material, and henceforth an effect on the stress the repair patch is subjected to. Therefore, section 7 is re-calculated replacing term (1) with:

$$Stress_t = E_t\left(Strain_t\right)$$

The process is repeated iteratively (steps 7 and 8) until there is little discernable difference in strain from one iteration to the next.

The values of *creep and stress* from the final iteration should be stored for use in the calculations for the next day increment. The final value for *Strain* is a key value, $\text{Strain}_t$.

9 Calculate transfer of strain to the substrate for t = 0 to 400.

If the elastic modulus of the repair material is higher than that of the substrate concrete then some of the strain will be transferred from the repair material into the substrate.

$\text{Strain}_{trans,t} = \text{Strain}_t * \lambda$

$\text{Strain}_{total,t} = \text{Strain}_t - \text{Strain}_{trans}$

10 Calculate tensile strain capacity of repair material from t = 0 to 400

$$\varepsilon_{cap,t} = \frac{f_t}{E_t}$$

11 Determining the performance of the repair material.

Plot $\varepsilon_{cap,t}$ and $\text{Strain}_{total,t}$ against time.

If the line $\varepsilon_{cap,t}$ against time, is exceeded by the line $\text{Strain}_{total,t}$ against time, then the repair material will fail at the intersection point.

If the two lines do not intersect then the repair material will not fail.

## 4.14 Conclusion

A broad opinion of research concerning the performance of concrete repair materials has been examined. The conclusion of this review is that current practices in reinforced concrete repair do not adequately take into account the necessity to control dimensional compatibility between the repair material and the substrate concrete. Additionally the importance of ratios of elastic moduli between substrate and repair are neither understood nor utilised by the majority of practitioners.

A procedure has been developed which predicts the development of critical tensile strains in the repair material. By comparing these tensile strains with the tensile strain capacity of a repair material (the development of which has also been researched), it is possible to predict the success or failure of a repair material. It is also possible, should a material fail, to predict the period after curing at which this will happen.

This procedure, if correctly implemented could reduce levels of failure in reinforced concrete repairs.

The procedure has been incorporated into the computer program developed in this project.

# 5 Decision making in the expert system for reinforced concrete bridge repair.

## *5.1 Chapter Objectives*

- Create expert system for concrete repair
- Develop a simple, expeditious method to assess severity and extent of defects based on cumulative expert knowledge and experience

## *5.2 Introduction*

An experienced engineer has the ability to diagnose defects exhibited by concrete and to recommend suitable remedies. Importantly, the ability to assess the significance of the extent and severity of defects, and to be minded of these in making repair recommendations, is a crucial part of the engineer's expertise. A review of expert systems for concrete repair in Chapter 2 of this thesis found that existing systems give generic repair advice which does not account for either the extent or the severity of particular defects.

The importance of a concrete element to an overall structure should be an important factor in the process of making decisions for repair. The central pier on a bridge can be considered a crucial element, upon which an averagely sized area of spall of reasonable depth might be considered very significant to the overall well-being of the structure. A wingwall on the same structure, with a similarly sized yet very deep spall, might not have important structural or durability implications for the overall structure, and should be treated accordingly. These are examples of decisions that an engineer will be making subconsciously as soon as he/she begins inspecting the structure, and yet, this most basic information will need to be entered into a computer in order for it to be able to make even the simplest decision. With the benefit of sight, engineers begin making subconscious decisions based on their knowledge immediately upon introduction to a defective structure - a distinct advantage for the engineer over a computer. With this in mind it can be said that there is a need to ensure fast and efficient entry of data into the expert system. Importantly, the elicitation of knowledge from experts, and the subsequent development of the expert system will take a simplistic approach to decision making.

### 5.2.1 Expert systems

Expert systems can be broadly described as computerized advisors that attempt to imitate the reasoning of experts in solving problems. Expert systems are also known as knowledge based systems.

There is no single code or set of guidelines for concrete repair, therefore gathering domain knowledge in the field is challenging. Best practice guidelines are dispersed amongst papers, regulations and instructions. Advice generated by the expert system has been collated through a literature review (Chapter 2) and interviews with field experts.

It is held as crucial that an effective and simple method of enabling an expert system to assess the severity and extent of a defect be developed for this work.

The aim of the expert system for reinforced concrete repair, is to create a software tool that, when given data on a bridge and its defects, can analyse the data, recommend a testing regime, recommend repairs, and finally recommend the most suitable repair material in accordance with the advice in this thesis. This calls upon the software to simulate the roll of an engineer in decision making.

### 5.2.2 Determining the severity of a defect

A key requirement of an intelligent expert system is the judgement of the severity of a problem it may have to diagnose. A problem discovered in some existing expert systems is the lack of advice offered on when to repair, and if repair is necessary. As an example, the appearance of a typical carbonation induced crack on a concrete element may well be cause for concern, although if the crack is negligible in length when compared to the scale of the element itself, this concern may well be misplaced. Existing expert systems appear to show little concern for this problem and will often give the same advice for the above example, as would be given for severe corrosion over an entire element.

A technique has been developed which provides the expert system with sufficient data to make decisions on the scale of defects, this part of the system is used in conjunction with a series of diagnostic knowledge bases to give the user a full picture of the nature and severity of any defects. The technique aims to make fast accurate decisions, without the need for laborious questioning or advanced mathematics.

## 5.3 Data input

The Highways Agency, and an increasing number of local authorities, store bridge information electronically. The Highways Agency's Structures Management Information System (SMIS) has replaced paper as a means of storing data from bridge inspections. However, SMIS, in common with most bridge management systems, stores text information. It was identified in the development of the expert system that a text interface is not a sophisticated use of available technology and an alternative form of user interrogation was developed.

Generally, reinforced concrete bridges are built to standard arrangements of decks, columns or piers, abutments, and wingwalls. With this in mind, it was decided that a fully graphical interface between the user and expert system would provide users with the type of modern software interface with which they are familiar, as opposed to a text based system that may seem old fashioned.

To enable the expert system to determine cause and severity, it is necessary for the program to also have information about the structure. Therefore the user is invited to create a three-dimensional representation of the structure or element upon which he/she requires the expert system to generate advice. This is done by the user being guided through a 'wizard' – a short on screen routine which automatically generates a typical reinforced concrete bridge based on a number of stored standard arrangements which the user can tailor. Alternatively the user can build a structure element by element. Examples of this process are shown in Figure 5.1 to Figure 5.3. Figure 5.1 shows the screen that immediately follows the request to construct a new structure.

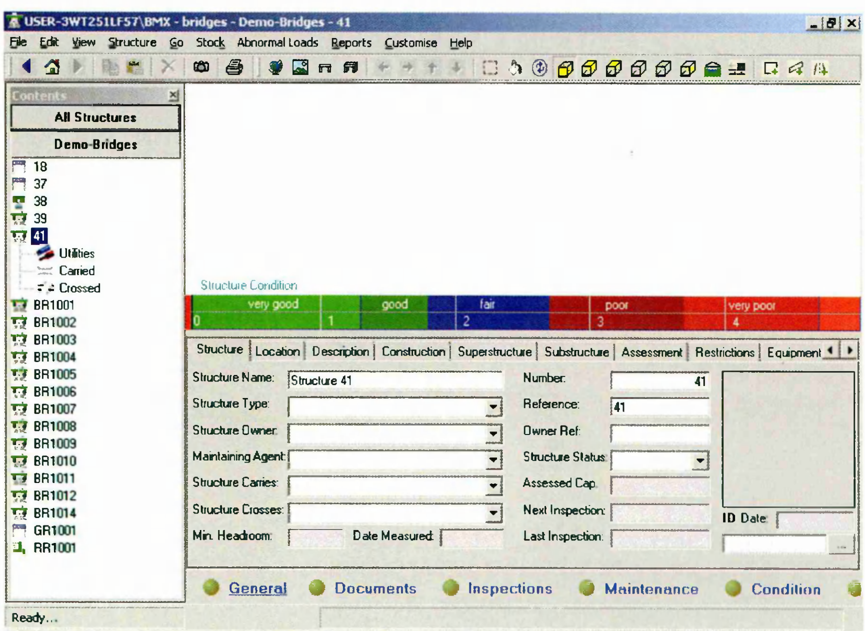



**Figure 5.1 New structure inserted**

In Figure 5.1, the program has been instructed that user would like to inform the system of a new structure on which a defect has been encountered. The user has indicated that he/she wishes to add a reinforced concrete deck. This is drawn onto the screen provided as shown in Figure 5.2.

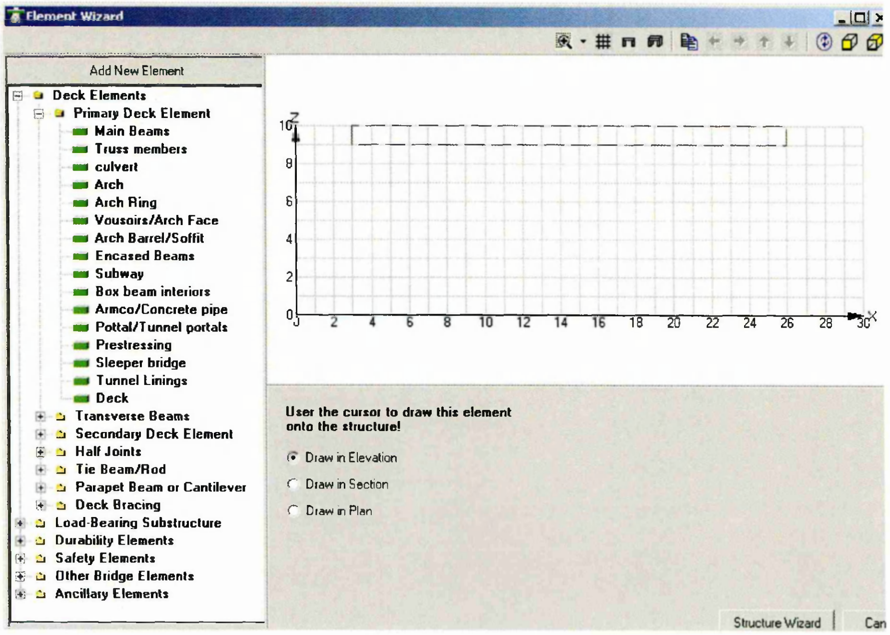



**Figure 5.2 Inserting a deck element**

In Figure 5.3 small bank seats are added under the deck. At this stage, the expert system has a fair representation of a simple short span accommodation bridge. The process has taken one minute, and the program is ready to being acquiring information on defects.

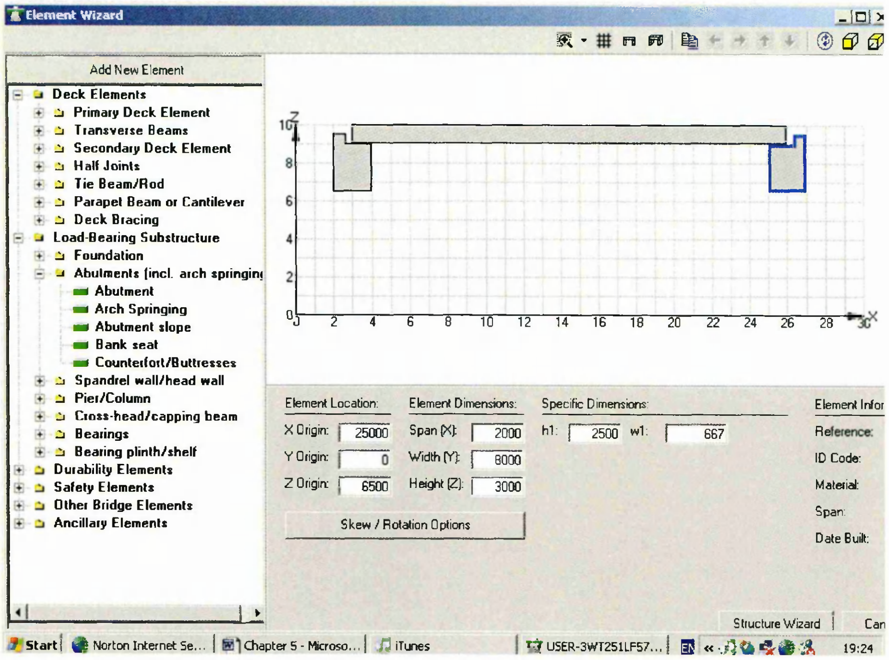



**Figure 5.3 Adding bank-seats**

It is not necessary for the expert system to know precise dimensions of a structure, as such a degree of accuracy will not affect the system recommendations. However, a reasonable representation of an element of a structure will enable the expert system to make sound judgments about the extent of defects.

## 5.4 *Diagnosing concrete defects in an expert system*

### 5.4.1 Beginning the process

Concrete defects can be grouped into four categories:

- Spalling
- Map-cracking
- Structural cracking
- Miscellaneous defects

These categories were discussed in Chapter 2.

It can be reasonably expected that even a user with a basic knowledge of reinforced concrete could distinguish (with some guidance) between these four types of defect; in the program developed it is assumed that this is the case.

In order for the process of defect diagnosis to begin, the user must graphically add a representation of the defect on to the concrete element entered into the program (section 5.3).

After this stage the expert system knows the approximate size of the affected element, the general type of defect and the approximate size of the defect. Thereafter, the four general defect types are treat differently by the program. This basic information will form the data which will be fed into the knowledge bases for advice.

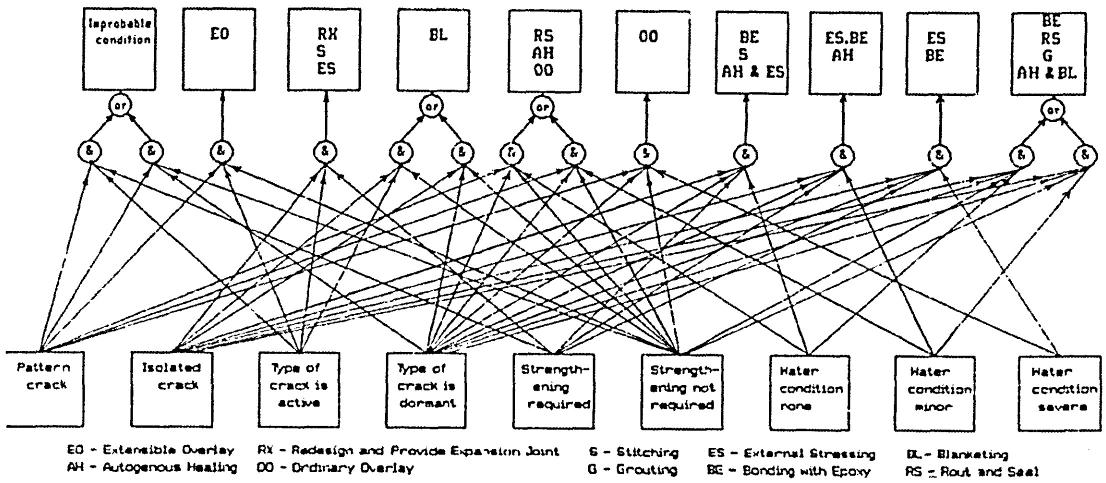
## 5.4.2 Constructing expert systems

### 5.4.2.1 Knowledge elicitation

Knowledge elicitation is the collection of domain knowledge. It is conducted in consultation with domain experts[39,47]. The domain experts in the field of concrete repair are generally civil engineers. This process of acquiring domain knowledge from experts is conducted through a series of interviews. These interviews can be done formally before a panel of experts or with individual experts. For the construction of this expert system, informal workshops were conducted with a series of experts.

### 5.4.2.2 Knowledge representation

In order to represent elicited knowledge, Kalyanasundaram et al[39] used a technique involving the formation of *knowledge nets* (Figure 5.4). They conceptualise the knowledge involved in repairing cracked concrete and place it into a static knowledge net. This knowledge net is then programmed into an expert system shell.




Sample Knowledge Net for Suggestion of Repair Method for "Cracking in Concrete"

**Figure 5.4 Knowledge net**

Variations of knowledge representation are similarly based on the creation of semantic networks (or flowcharts). Rajeev and Rajesh[47] take a more basic approach with the use of *instance nets* (Figure 5.5).

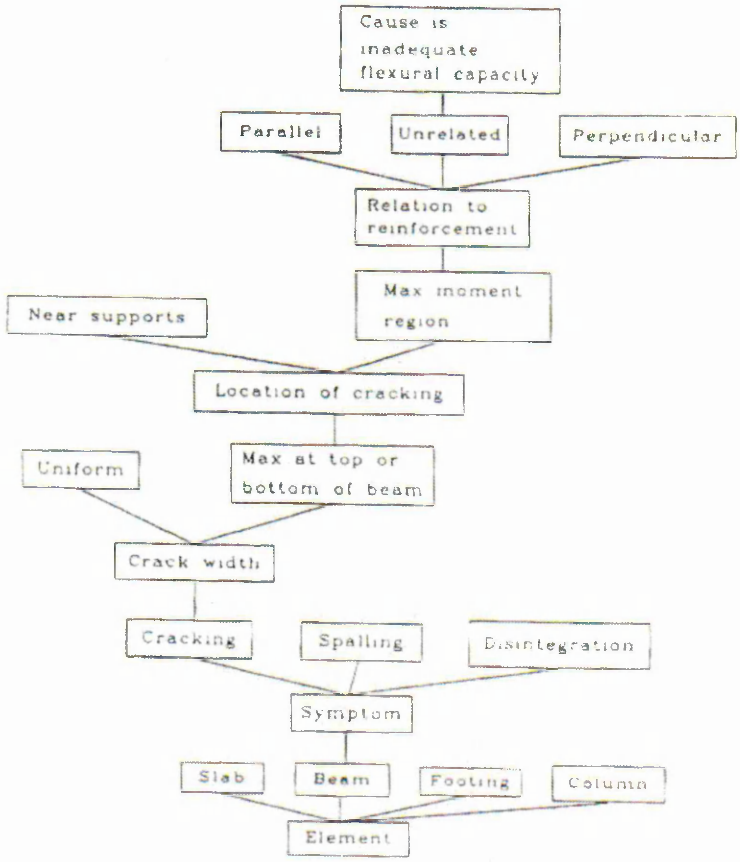



**Figure 5.5 Instance net**

#### 5.4.2.3 Coding

The knowledge engineer is provided with two ways of coding the knowledge obtained, and producing the inference engine which will eventually make decisions. Firstly, the engineer could construct an expert system in its entirety. That is the inference mechanisms and entire software components (the knowledge and the brain are both formulated). Although this method can allow the engineer to tailor the engine to the specific requirements of the domain, it requires an expert programmer. The second method is to use an expert system shell. Expert system shells contain inference engines prepared and ready for the input of objects and rules (the brain is acquired, the knowledge is formulated). An expert system shell can facilitate the structuring of knowledge, the control of the inference strategy, and some shells enable the design of the user interface[109]. The primary purpose of the project is to create an expert system for reinforced concrete repair and not merely to pursue innovative methods of creating expert systems. Therefore, a review of expert system shells was conducted, and a shell called 'Acquire' was obtained for the purposes of representing (in a knowledge base) information obtained from the expert panel through workshops and interviews.

It was established during initial interviews that whilst an expert system shell could function adequately to determine the cause of defects, such shells are not necessarily suitable for determining severity and extent of defects, particularly not in concrete repair situations where a single element could contain a large number of defects. Therefore, it was established that some form of traditional software programming would have to be used to determine the severity and extent of defects on an element. Thus, a key aspect of the development of an intelligent advisor for concrete repair is that a traditional expert system shell, through knowledge bases and an inference engine, will be used to diagnose the causes of concrete defects. This will work in tandem with a traditional software program that will mathematically assess the severity and extent of the defects. Throughout the process of concrete repair i.e. from diagnosis to repair material recommendation, the two aspects of the system will work together. The overall aim being to use the software tools available with as much simplicity as possible to create the desired intelligence.

### 5.4.3 Developing the knowledge bases

As a result of initial interviews, it was identified that there existed a need for five distinct knowledge bases (KB) in order for an expert system to be able to allow a user to fully diagnose any defect and take that defect through to the same conclusion which an expert would arrive at – some form of action, be it "repair", "monitor", or "do nothing". The required knowledge bases identified were:

- Diagnosis of cause of spall defects
- Diagnosis of cause of pattern crack defects
- Diagnosis of cause of structural crack defects
- Recommendation of testing regime for element
- Recommendation for repair methods

It was identified that these five knowledge bases would be employed at different stages in the framework of the overall system as follows:-

User enters structure geometry

User enters defect geometry and information

**Spall KB** or pattern **crack KB** or **Structural crack KB** used to determine probable cause of defects

Severity and extent of defects determined

Effects of all defects on single element assessed

Testing regime recommended by **Testing KB**

Cause of defects confirmed by test results

Repair information provided by **Repair KB**

Suitable properties for repair material recommended (see Chapter 4)

Bridge condition assessed

The knowledge bases and other routines developed to enable these ten stages to be performed by the expert system will be integrated into a commercially viable bridge management system.

For each knowledge base, the expert panel were asked to identify any criteria that could be used to assess the cause of a defect.

#### 5.4.3.1 Knowledge base for spalls

Beginning with the knowledge base for the identification of spalls, the experts listed all the factors that might influence their decision as to the cause of any spalling:

- Shape
- Size
- Depth
- Age of element
- Location of element in relation to splash zone of vehicles
- Reinforcement exposure
- Depth of exposed reinforcement (cover)
- Reinforcement condition
- Evidence of staining
- Evidence of seepage
- Proximity of element to carriageway (likelihood of impact damage)

In the expert system shell (Acquire), each factor which could determine the cause of the spall is called an 'object'. Each object has a 'range'. Ranges are the units of measurement over which objects are measured – either textual or numerical. For example, the range for the object 'Age of element' is a numerical value in years ranging from zero to infinity, the range for the object 'Splash Zone' (the object whose range is set to determine if the element is within the splash zone of vehicles) has the textual range 'yes' or 'no', importantly, the value 'unknown' can also be chosen. Similarly, the object 'reinforcement exposure' has values: unknown, none, low, medium, high. Although some objects, such as 'reinforcement condition' have ranges that comprise of natural language identities such as low, medium and high – in the expert system developed the user is seldom asked to make these kind of assessments, as that requires expert knowledge. Although the expert system shell requires to know the value for 'reinforcement condition' to make accurate decisions – it is provided by the user only indirectly, the actual value is inserted at the data input stage using photographs of different severities of reinforcement corrosion to guide the user . This close interaction between the diagnostic expert system shell and the numerical data input program written in a traditional computer language is a key relationship in the overall performance of the expert system.

All the data required by the expert system to make a decision is entered by the user at the data input stage. Figure 5.6 to Figure 5.10 show the procedures necessary for the expert system to obtain all the information required for it to be able to make decisions about the cause of spalling. Figure 5.6 shows the structure constructed earlier in this chapter – a central pier has been added.

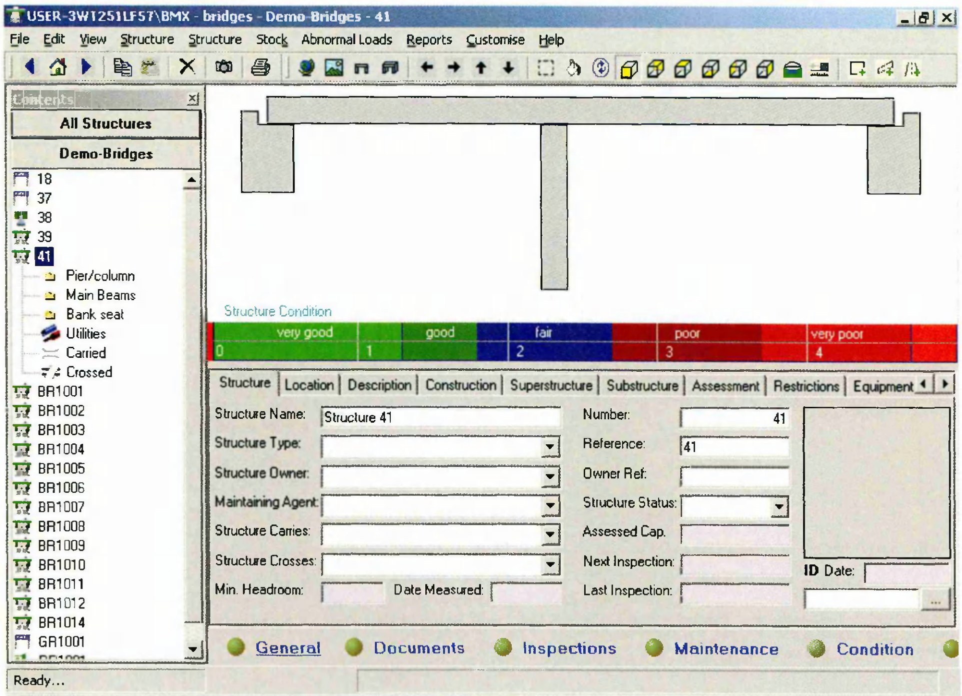



**Figure 5.6 Structure for defect study**

Assume a spall defect has been encountered on the central pier. In Figure 5.7, the central pier has been selected with a double click. An unwrapped view of the pier is shown, and it is onto this view that the spall defect will be added.

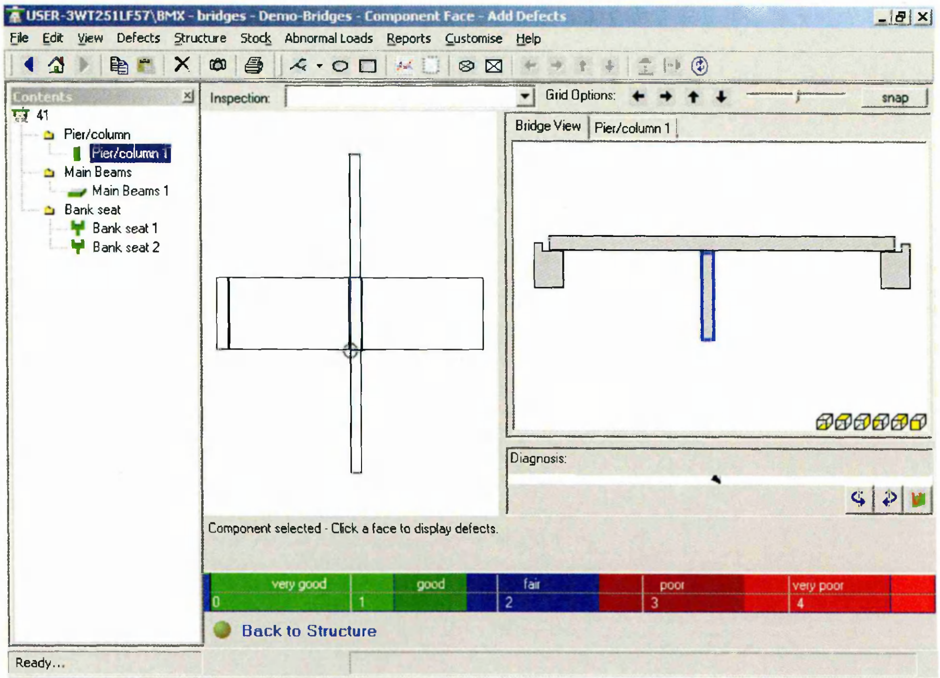



**Figure 5.7 Central pier selected for study**

In Figure 5.8 a rectangular defect has been added by selecting the 'add defect' icon and dragging a box onto the unwrapped pier as shown. At this stage the system does not know the type of defect that has been added.

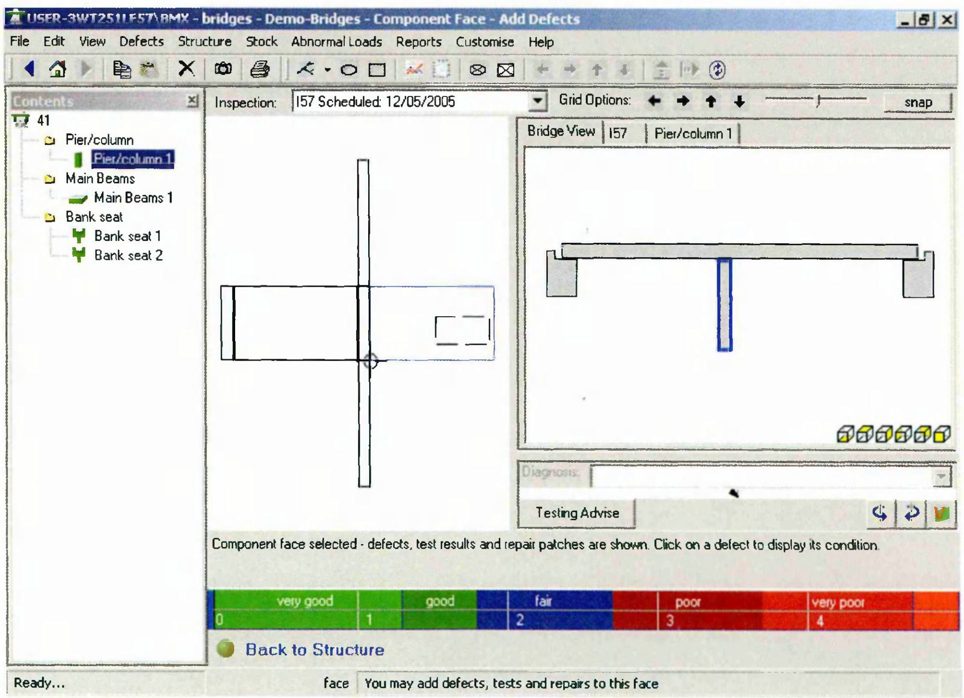



**Figure 5.8 Adding rectangular defect**

In Figure 5.9 the user informs the system that the defect added is a spall, by highlighting the spall icon as shown.

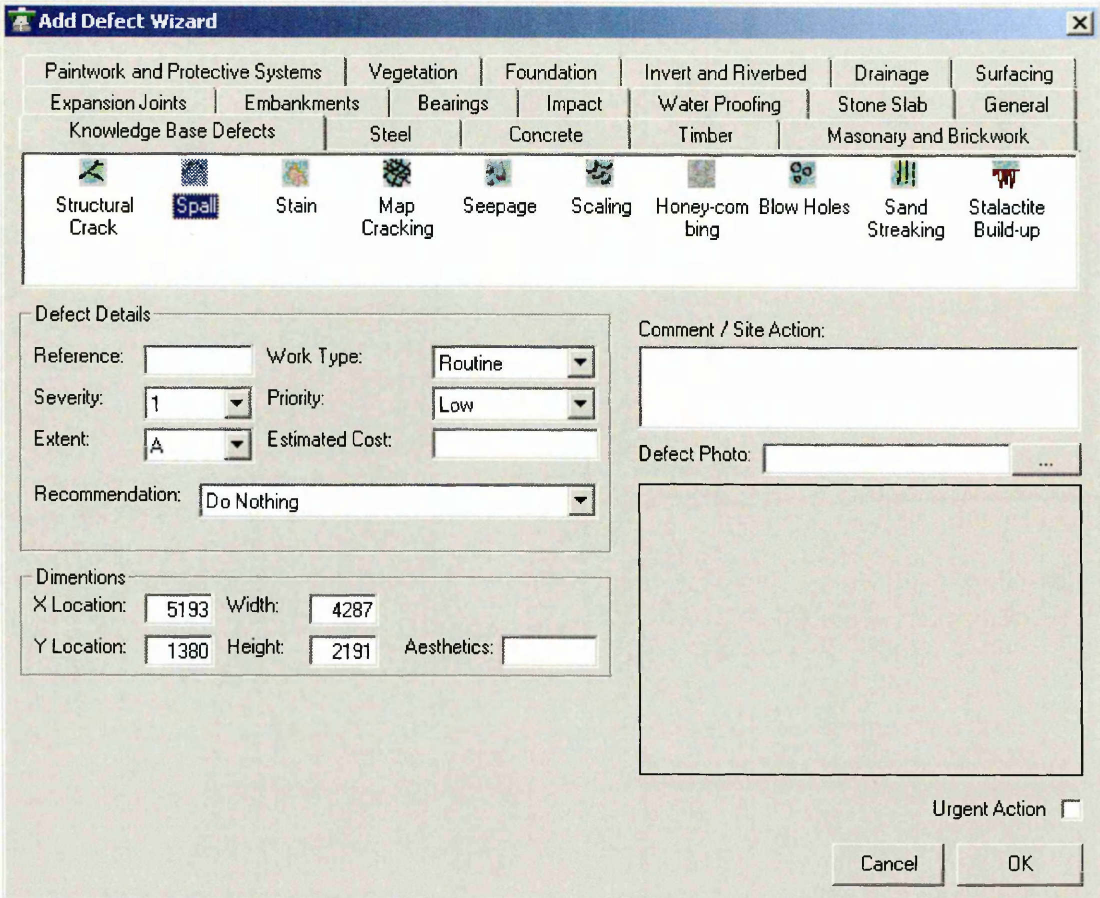



**Figure 5.9 Adding a spall defect**

In Figure 5.10 the user adds further details about the spall, such as its shape, depth, and if the defect is within the splash zone of vehicles.

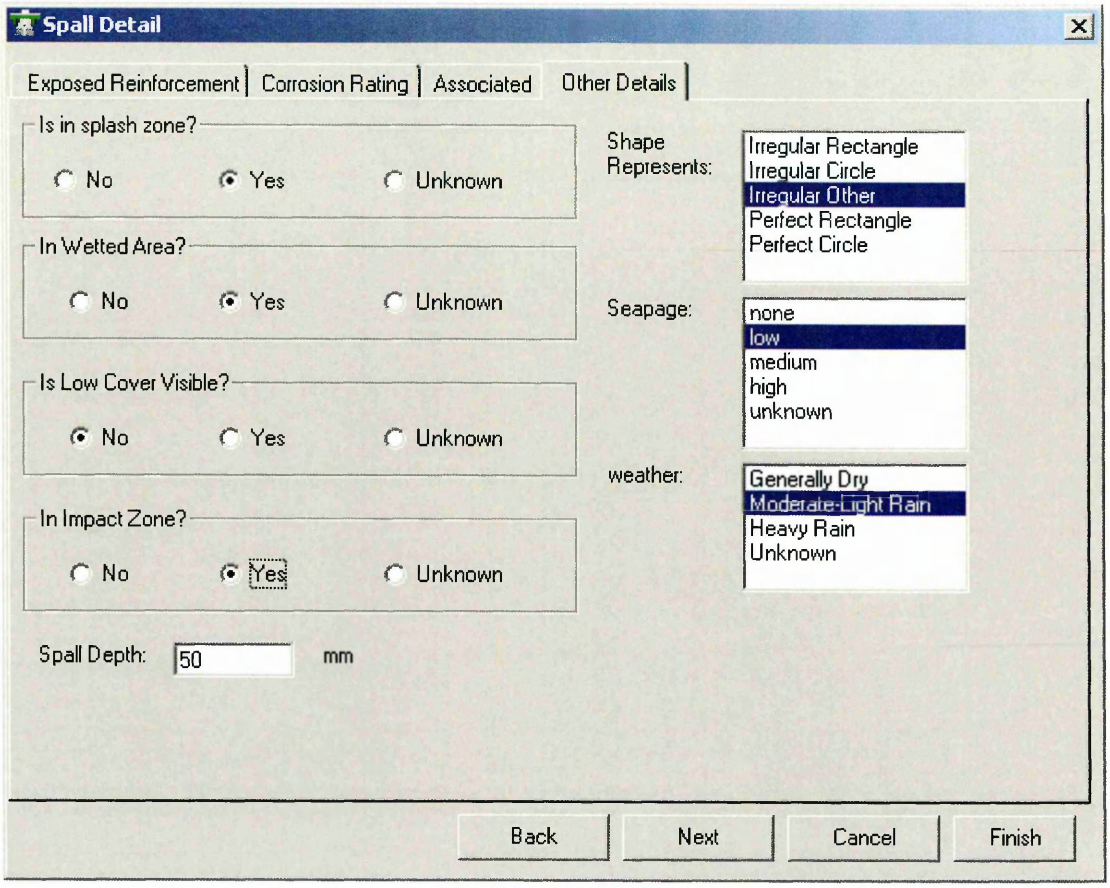



**Figure 5.10 Adding spall details**

Figure 5.11 shows how the user is expected to grade the condition of exposed reinforcement. Having initially gone through the data input procedures, and informed the system that the spall exhibits exposed reinforcement; the user is presented with a graphical interface. Using this interface the user judges, via comparison, how badly corroded the reinforcement is and matches this to the examples shown. An internal setting in the program, decided by the expert interviewees, determines that a rating of 20% or less sets the object 'reinforcement exposure' to 'low'. A rating of between 20% and 60% sets a value of 'medium' and anything above that 'high'.

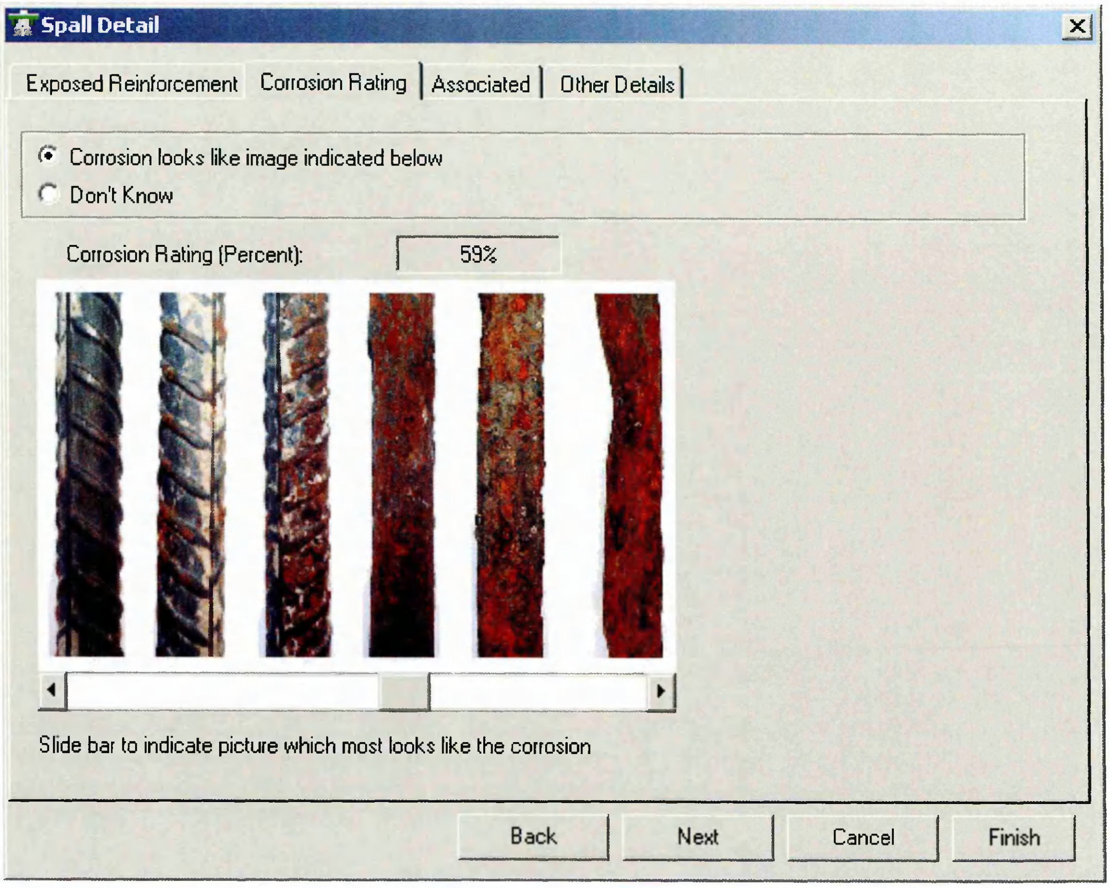



**Figure 5.11 Inputting corrosion amount**

The user may not be able to provide values for all the objects that the knowledge base requires to make its decision. In these cases, the value of these unknown objects is set to 'unknown'. Once all data has been entered and all the objects' ranges set. The knowledge base is ready to 'fire'.

In order to make expert decisions the expert system uses 'rules' which were created during the expert interviews. The expert system shell uses two different methods to set rules. Figure 5.12 shows a premise rule which the knowledge base for spall defects uses to set the object 'cause carbonation'. In the expert system, input objects have their ranges set by the user data and output objects have their ranges set by rules. ‘Cause Carbonation’ is one of a number of output objects. The collection of rules forms the knowledge base.

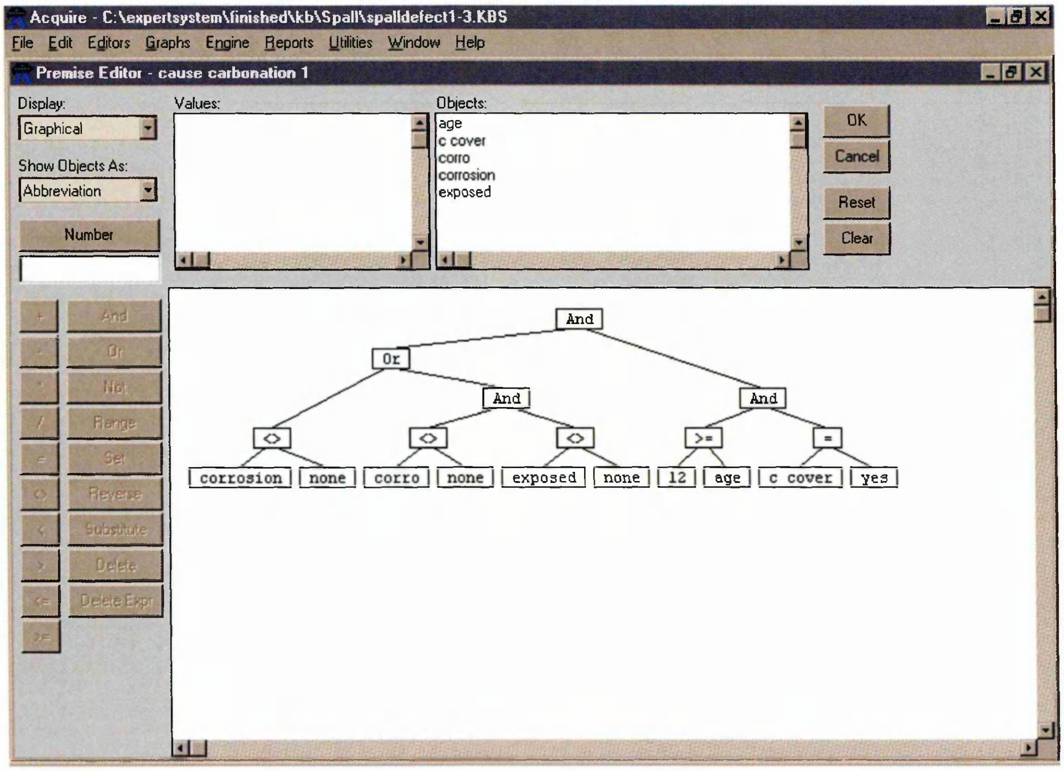



**Figure 5.12 Carbonation premise rule**

The rule in Figure 5.12 uses abbreviated object names. It sets the output object ‘cause carbonation’ to the value ‘low’. The diagram shows two criteria, both of which must be met in order for this rule to set the value for ‘cause carbonation’ to low. Firstly the bridge age must be less than 12 years, and the object ‘low cover’, which is set from the user entered data, must be set to ‘yes’. Secondly, if there is no exposed reinforcement, there must be evidence of corrosion (‘corrosion’ object not equal to ‘none’) or, if there is exposed reinforcement, it must exhibit some form of corrosion. If these conditions are met, the rule ‘cause carbonation’ will be set to ‘low’ and this result will be passed on to the user at a later time. The basic premise of this rule is that carbonation is not expected in a young bridge – however, if the cover is low, and if there has been corrosion of the reinforcement, there is a small chance that carbonation could be the cause. Hence the object 'cause carbonation' is set to 'low'. The rule shown in Figure 5.12 is one of a number of different rules which affect the object 'cause carbonation'. Other rules set the object's value to 'high' and 'medium'. If none of the conditions in the various rules for 'cause carbonation' are met, the value of that object will remain at 'none' – i.e. the knowledge base does not think the cause of the defect is carbonation.

Figure 5.13 shows all the rules in the spall knowledge base. Each rule is built up from expert opinions.

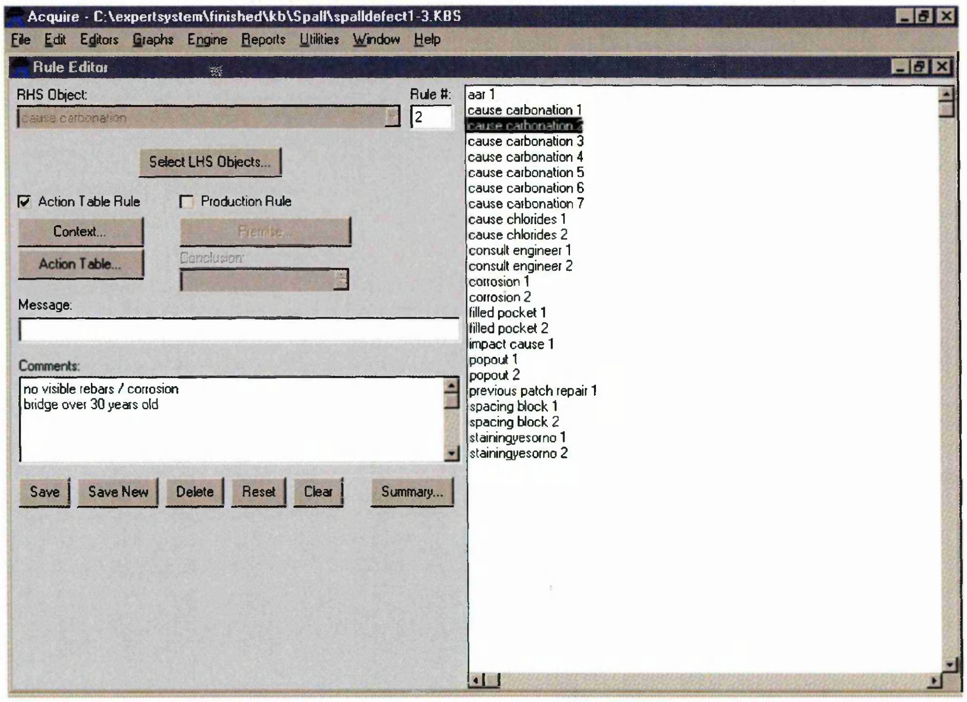



**Figure 5.13 Rules in the spall knowledge base**

The second type of rule that the knowledge base uses is an 'action table' rule. Action table rules begin with a context. Figure 5.14 shows the context for an action table that might set the 'cause carbonation' object. In accordance with the context, the knowledge base will only use this action table if the bridge age is less than 30 years and the spall exhibits no exposed reinforcement. For other possible scenarios such as the bridge age being over 30 years and the spall exhibiting exposed reinforcement, other rules have been constructed. A full set of 'cause carbonation' rules are constructed in order to enable the system to give advice about any combination of data.

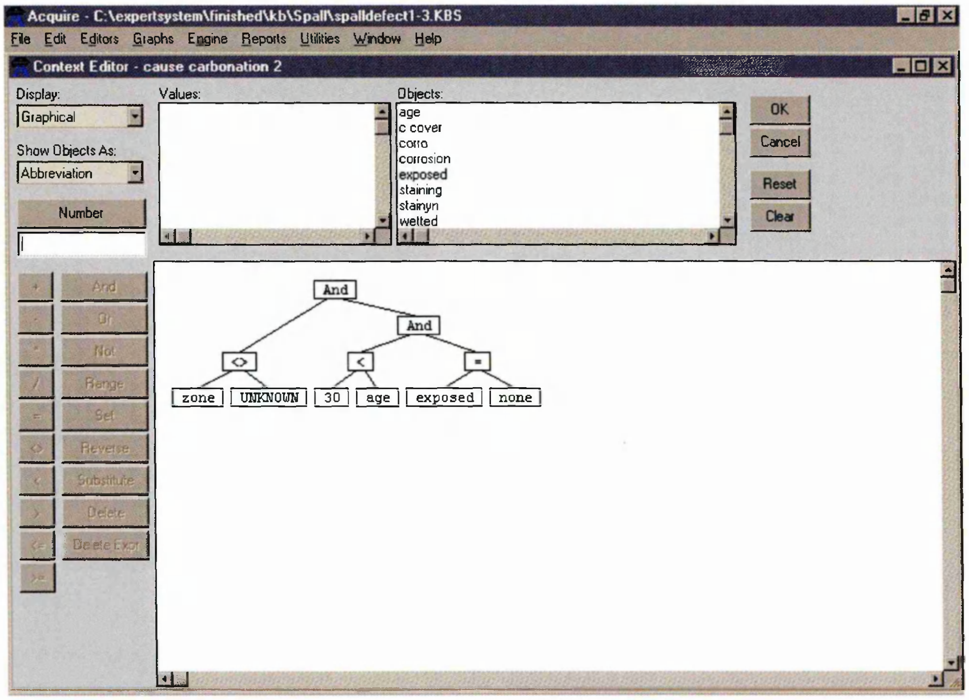



**Figure 5.14 Context  for carbonation action table rule**

If the context for the action table rules has been met, then the rule itself comes into force. Figure 5.15 shows the action table for a 'cause carbonation' rule. Each column of the action table is headed with the name of an object. In the rows below, the different values of that object appear. For example the object 'corrosion', which represents evidence of corrosion (such as staining) in the absence of exposed reinforcement, has its values; low, medium, high, and unknown listed below the column heading. Importantly, enough rows appear in the table to compare every possible combination of the object values in the table, and this can be seen in Figure 5.15.

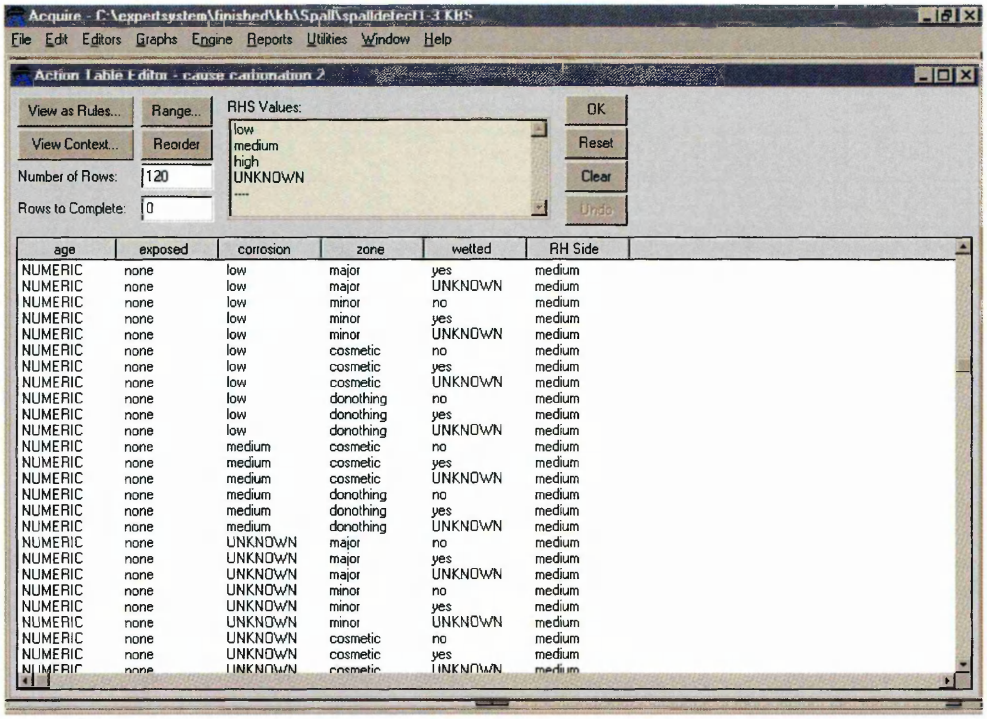
Because of the context, the column headed 'exposed' is always set to 'none' in this action table. The column headed 'age' is set to 'numeric' although the rule only fires if the bridge age is under 30. The other columns have variable values. One hundred and twenty rows exist in this table so that all object values can be compared. In the rightmost column the value for the output object (cause carbonation) is set, by the expert, to either 'low', 'medium' or 'high'.

Taking the first row as an example, and bearing in mind the context of this rule – that there is no exposed reinforcement and the concrete is younger than 30 years old:

- Low evidence of corrosion. e.g. the user would have indicated, at the data input stage, that there was evidence of mild corrosion staining.
- The spall is in a wetted area.
- The object 'Zone' represents the category of defect that this spall has been placed into by the traditional computer program – the area of the expert system where severity and extent are assessed. This variable is set to either: 'major', 'minor', 'cosmetic', or 'do nothing'. The method by which this is assessed is described in detail in section 5.5.

The expert judgement for this combination of values is that there is a medium chance that the cause of the spall is carbonation of the concrete. The object 'cause carbonation' is set to 'medium', and this will be reported to the user at a later stage.

This type of judgement is made for every combination of object values in the action table. It can be seen that using the action table allows a great number of combinations of object values to be assessed quickly. Premise rules and action table rules are used together to ensure that all eventualities can be assessed by the knowledge base.




| age | exposed | corrosion | zone | wetted | RH Side |
|---|---|---|---|---|---|
| NUMERIC | none | low | major | yes | medium |
| NUMERIC | none | low | major | UNKNOWN | medium |
| NUMERIC | none | low | minor | no | medium |
| NUMERIC | none | low | minor | yes | medium |
| NUMERIC | none | low | minor | UNKNOWN | medium |
| NUMERIC | none | low | cosmetic | no | medium |
| NUMERIC | none | low | cosmetic | yes | medium |
| NUMERIC | none | low | cosmetic | UNKNOWN | medium |
| NUMERIC | none | low | donothing | no | medium |
| NUMERIC | none | low | donothing | yes | medium |
| NUMERIC | none | low | donothing | UNKNOWN | medium |
| NUMERIC | none | medium | cosmetic | no | medium |
| NUMERIC | none | medium | cosmetic | yes | medium |
| NUMERIC | none | medium | cosmetic | UNKNOWN | medium |
| NUMERIC | none | medium | donothing | no | medium |
| NUMERIC | none | medium | donothing | yes | medium |
| NUMERIC | none | medium | donothing | UNKNOWN | medium |
| NUMERIC | none | UNKNOWN | major | no | medium |
| NUMERIC | none | UNKNOWN | major | yes | medium |
| NUMERIC | none | UNKNOWN | major | UNKNOWN | medium |
| NUMERIC | none | UNKNOWN | minor | no | medium |
| NUMERIC | none | UNKNOWN | minor | yes | medium |
| NUMERIC | none | UNKNOWN | minor | UNKNOWN | medium |
| NUMERIC | none | UNKNOWN | cosmetic | no | medium |
| NUMERIC | none | UNKNOWN | cosmetic | yes | medium |
| NUMERIC | none | UNKNOWN | cosmetic | UNKNOWN | medium |

**Figure 5.15 Action table for 'cause carbonation'**

The screen grabs shown in Figure 5.12 to Figure 5.15 are not at any stage shown to the user in the finished program. They are internal screens used to construct the knowledge bases and to set the output objects. Once output objects are set they can be presented to the user in the user interface as required.

### 5.4.3.2 Knowledge base for pattern cracking

For the purposes of this system, 'map cracking' is defined as any distinct patch of concrete that is affected by cracking. Single cracks are indicative, often, of some form of structural cracking – these are handled by a separate knowledge base.

During interviews, experts were asked to list any features that may affect their diagnosis of an incidence of pattern cracking. The following were identified:-

- The appearance of the pattern cracking
- The age of the bridge
- Efflorescence and its colour and form

- Seepage
- Evidence of corrosion
- Type of element and alignment
- Is the defect in the splash zone of vehicles
- Is the defect in a wetted area

There is a noticeable difference between the method the expert system uses during data input to obtain pattern cracking information against that it uses to obtain spalling information. Figure 5.16 shows how, after the user has drawn a patch of pattern cracking onto an element, the user is requested to choose an image which best represents the type of cracking they have encountered.

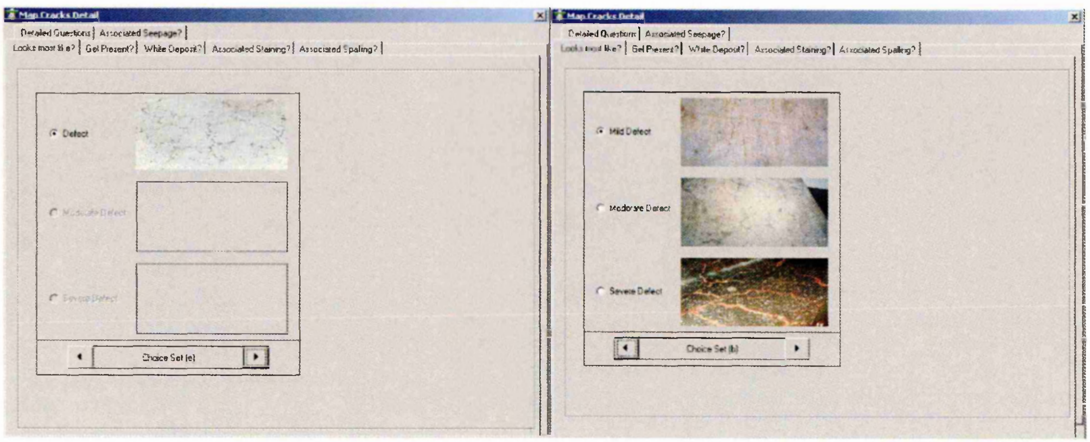



**Figure 5.16 Selecting a representative map-cracking image**

This method negates the need for the user to choose from a list of textual descriptions of the defects. Each one of the available map-cracking images is typically representative of cracking induced by one of the following ailments:

Chloride corrosion

Carbonation corrosion

Alkali aggregate reaction, alkali silica reaction etc.

Freeze-thaw damage

Plastic Shrinkage

Crazing

Drying shrinkage

Once the user has selected a defect image, he/she enters other relevant information. After this the knowledge base is ready to make decisions. The knowledge base to determine the cause of map-cracking was constructed in the same way as that which determines the cause of spalling. An object called 'image' has an alphabetical range from 'a' to 'i' where each letter represents one of the images the user selected to describe the appearance of the pattern cracking. All the objects are used in the formation of premise rules and action tables to diagnose the cause of the defect.

#### 5.4.3.3 Knowledge base for structural cracking and single cracks

For the purposes of the expert system, any single crack which is not catered for by one of the representative images in the pattern cracking data input prompt is handled by the structural cracking knowledge base.

Many large individual cracks are caused by corroding reinforcing steel, which causes cracking along the length of the reinforcing bar. Therefore, many individual cracks will not actually be caused by structural effects, if a crack added by a user is suspected of being caused by corrosion, and therefore is non-structrual, the user will be informed of this.

At the data input stage, the user draws on the element a crack, as shown in Figure 5.17. Following this the user is requested to provide as much additional information as possible, such as crack width, associated staining or seepage etc.

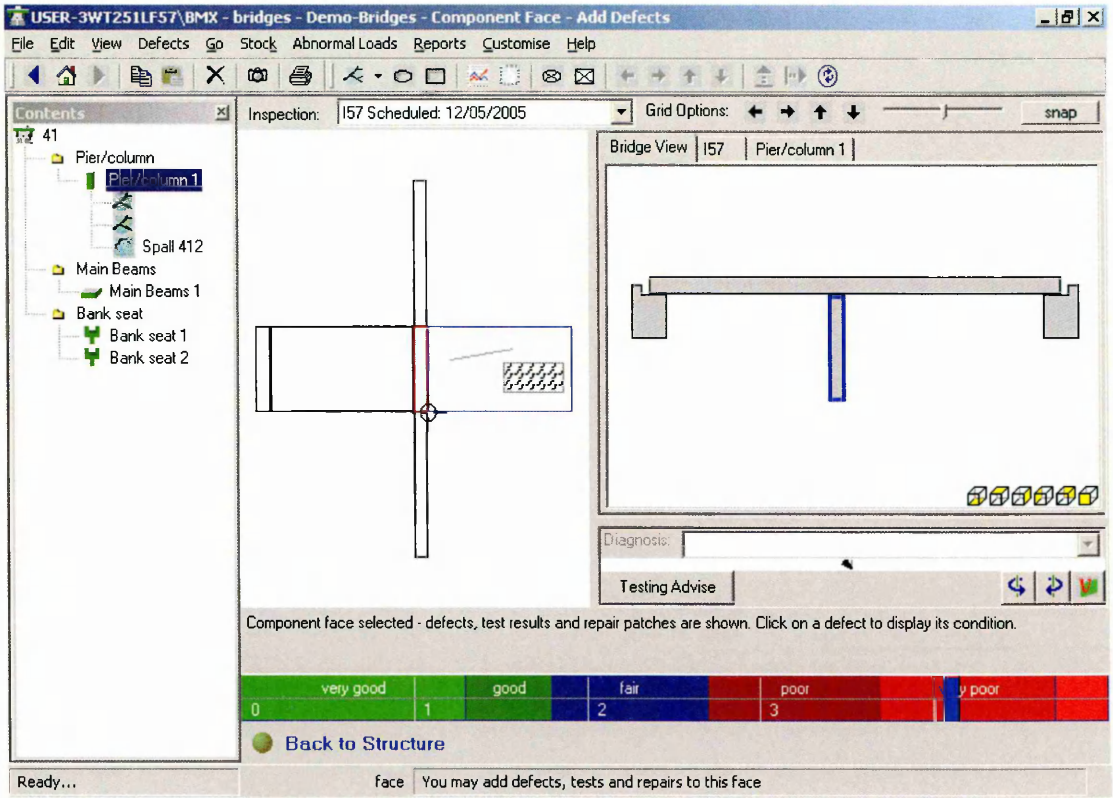



**Figure 5.17 Adding a structural crack**

The data gathered is then passed through the knowledge base and, in accordance with the techniques outlined in the previous section, knowledge bases determine the likely causes. However, due to the difference in geometry and type, this form of cracking is handled differently to pattern cracking and spalling.

Any element face into which the user adds a structural crack could be inclined. As any reinforcing steel is almost universally placed parallel and perpendicular to the edge of the concrete element it makes up, the first check the system makes is to see if the crack entered by the user is parallel or perpendicular to the reinforcing steel. There are two knowledge bases used to assess structural cracking. The first knowledge base examines the information, looking particularly for evidence of corrosion and cracks being parallel or perpendicular to the element edge, and decides if the crack is caused by reinforcement corrosion or structural effects. If the crack is caused by corrosion, then the straight line added by the user is amended automatically to a rectangular spall region 150mm wide (150mm being the typical distance between reinforcing steel centres). At this stage the defect, which was initially a 'structural crack defect' but is now a spall defect, is passed over to the spall knowledge base for assessment as in 5.4.3.1.

## 5.5 Determining the severity and extent of defects

It has been recommended in this thesis (Chapter 2) that in order for an expert system for reinforced concrete repair to be intelligent, it must be able to measure the severity and extent of defects.

Boam[110] suggests that there are six possible actions that can be taken as a result of a defect that has induced corrosion or left reinforced concrete susceptible to corrosion.

- Do nothing
- Reduce corrosion rate
- Repair visible defects
- Carry out major repairs
- Apply cathodic protection
- Replace affected element

For the purposes of the expert system, these options have been simplified into four categories into which any individual defect can be placed. An intelligent expert system must be able to place any defect into one of these categories:

1) Do nothing

The significance of the defect in question is of small importance. Durability is not affected. If left un-repaired, there will be no detriment to the concrete element.

2) Cosmetic

The defect considered had caused damage to the exterior appearance of the concrete element. There is a long term durability risk. The defect is noticeable and aesthetically unpleasant. It should be considered as 'in need of repair' either for aesthetic purposes, or for purposes of arresting any further deterioration which could lead to more severe defects.

3) Minor

The defect is significant. The protection the concrete provides to the reinforcing steel has been compromised and the defect will progressively worsen unless remedial action is taken. The defect is aesthetically unpleasant. Repairs should be undertaken, although the defect is not of a sufficient nature as to require urgency.

4) Major

The defect is so severe in its nature and magnitude that there is either an immediate loss of safety against collapse, or, even if the element is structurally stable, public confidence in its performance is compromised. Repair should be undertaken immediately.

Each individual defect will be placed into one of these categories by the expert system. However, once all the defects prevalent on an element have been entered into the program, at that stage the system should make a decision on the overall action to be taken on the element as a whole.

In order to be able to place any defect into one of these categories, the expert system requests information (typically):

- Proportional size of defect (i.e. size of defect patch relative to size of element it affects)
- Depth of defect (for spalls)
- Width of defect (for large cracks)
- Type of surface cracking (for pattern cracking and surface deterioration)
- Is reinforcement exposed (how much, how badly deteriorated etc.)
- Moisture condition
- Evidence of corrosion.

### 5.5.1 Determining the size of a defect

It has been identified that a key requirement of an intelligent expert system is the judgement of the severity of a problem it may have to diagnose. Existing expert systems reviewed lacked the intelligence to offer advice on when and if to repair.

The theoretical maximum area of an element that can be covered by a single defect, in terms of percentages, is 100% - the minimum is obviously 0%. This information is calculated automatically by the program when the user defines the defect patch (Figure 5.8). At the early stages of data input, the program knows only the type of defect (spall, pattern crack or crack) and its size (in terms of element coverage, from 0 to 100%).

A technique has been developed which allows the expert system to place any individual defect into one of the four repair categories.

The technique devised employs a horizontal axis to represent the percentage of the element covered by the defect. The four possible zones into which a defect can be placed are positioned, in order, onto the axis as triangles with their apexes at pre-determined points, as shown in

Figure 5.18.

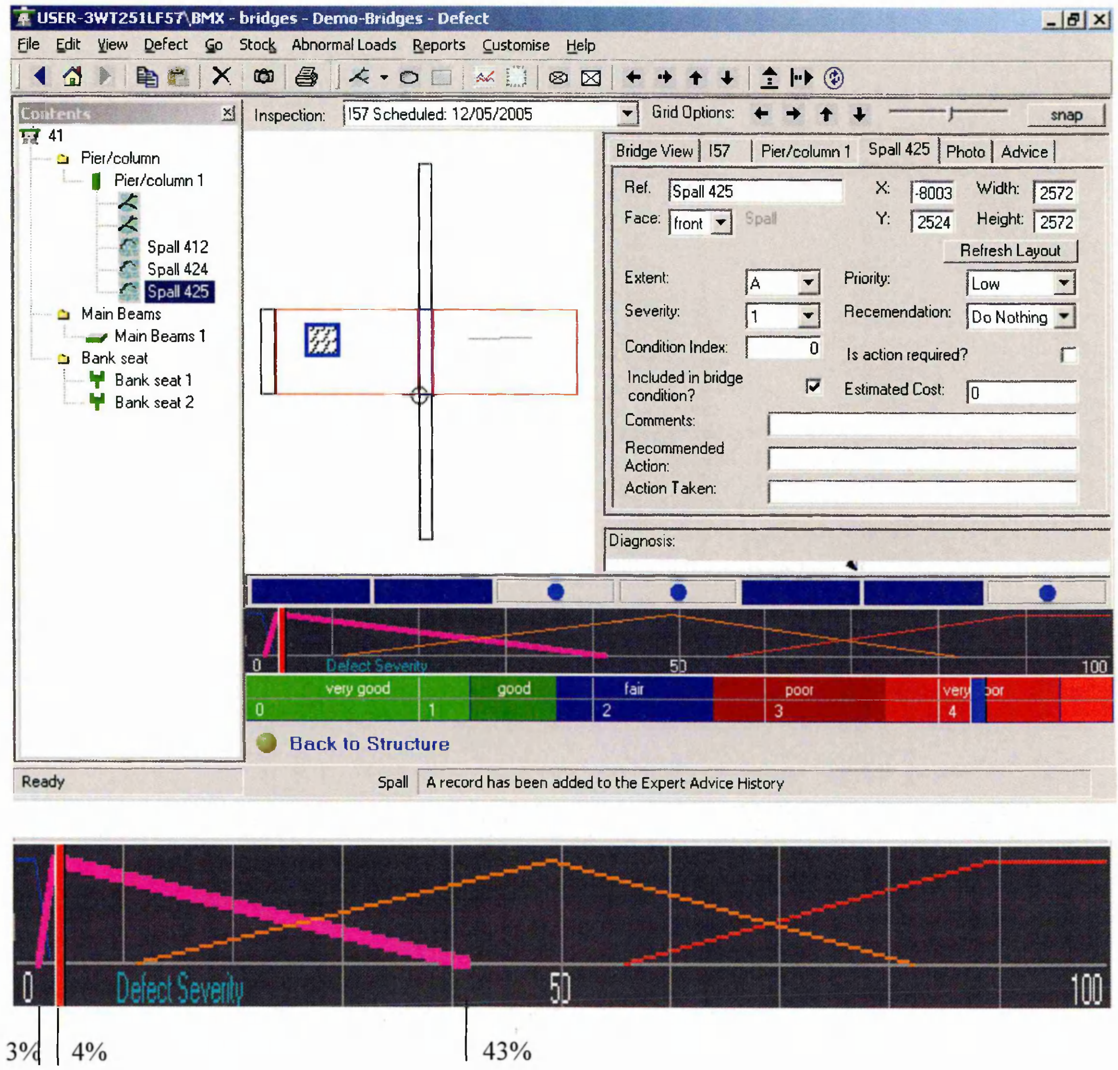



**Figure 5.18 Repair zones for 32mm deep spall**

In

Figure 5.18, the spall has been added to the unwrapped pier element as shown, and the system has been informed that the spall is 32mm deep. Using the techniques developed in this chapter, the expert system positions four triangular zones on the display (shown enlarged below the figure), from left to right, these zones represent the four categories into which each defect is placed. For example, the 'Do Nothing' zone covers the region from

0% to 4%. The Cosmetic zone covers the region from 3% to 43% (in order to represent uncertainty in the decision making process, the zones overlap, creating fuzzy boundaries.) The horizontal scale equates to the defect size as a percentage of the overall element area. The size of a defect added in

Figure 5.18 is 4% (of the element it affects), the defect is, therefore, a 'Cosmetic' defect. The position of the apex of the zones changes depending on the information added by the user, for example, in Figure 5.19, the depth of the spall (

Figure 5.18) has been altered to 50mm – a more serious defect. As a result the zones have shifted, and the defect size (4%) now falls into the 'Minor' repair zone. This information, which represents the current thinking of the expert system based on the information it holds about the defect, is always visible to the user. As the program's information is increased by further data input, the user sees how its opinion is affected as the zones move.


**Figure 5.19 Spall depth 50mm**

The location at which the diagonal arms of the zone triangles intersect dictates the width of coverage of the horizontal axis over which the zone falls. This position of the intersection changes depending on the amount of information the system is supplied with. For example, immediately after a spall defect is added to the program, the system knows two pieces of information – the fact that the defect is a spall, and the size of the defect as a percentage of the overall element size. Due to the limited information, the arms of the zone triangles cross close to the apexes, and as a result the overlap between zones is large. This effect represents the uncertainty and fuzziness. The fact that the expert system's information is limited is represented by the large overlaps between zones, and as a result defects could fall into the fuzzy areas between zones, i.e. a defect could be classes as both a 'do nothing' defect and a 'cosmetic' defect. The program's ability to determine which zone to place a defect into when it falls into these fuzzy areas is discussed in more detail in section 5.6.

### 5.5.2 Map cracking defects

There are two key factors which dictate the severity of a pattern cracking defect: its coverage of the element, and the pattern cracking image which the user selects to represent the defect (Figure 5.20 to Figure 5.25). In addition to this information, the user is asked to judge the cracking and place it into one of three bands: mild, moderate and severe. For example, if the user selects Image 2, Figure 5.21 to represent the defect, the system knows that the image represents corrosion induced cracking caused by chloride ingress. The user is then requested to grade the cracking. However, for some of the images, such as crazing, it is not considered applicable to divide the defect into these categories in such a way.

During a series of interviews, the industrial experts (concrete repair practitioners) were asked to select images which best represented the different causes of pattern cracking. Subsequently, the experts were presented with the definitions of the four categories into which an individual defect will be placed by the expert system (section 5.5). Precise definitions of the categories were agreed amongst the panel.

The image representing chloride induced corrosion cracking was presented before the panel (Image 2, Figure 5.21). The question was posed. "On average; for moderate cracking of the type shown in the image; how large would the pattern cracking patch be, as a percentage of the element area, in order for this defect to fall into the 'Do Nothing' category?" This question was followed by some discussion and the drawing of diagrams showing, to scale, how large a defect covering 5%, 4%, 3% etc. of a typical pier would look. The experts chose the value 2%.

This exercise was repeated for the other three zones, then further repeated to cover mild cracking and severe cracking for this particular image. From these sessions the resulting table was formed (Table 5.1):

**Table 5.1 Position of zone apexes for chloride cracking image**

| | Nothing | Cosmetic | Minor | Major |
|---|---|---|---|---|
| mild | 2.5 | 4.5 | 10 | 17.5 |
| moderate | 2 | 3.5 | 9 | 16 |
| severe | 1.5 | 3 | 8 | 15 |

For example, the vertical red band in Figure 5.20 represents the size of the defect. In this example, the user has drawn a considerable defect on the element, the defect covers 30% of the element area. The system already suspects the cause is chloride corrosion because the user selected the 'chloride corrosion' image (Image 2, Figure 5.21), the zone positions have, therefore, been set in accordance with the expert recommendations and the defect is well inside the 'major repair' zone. This is as would be expected for a defect covering (in this example) 30% of the element area.

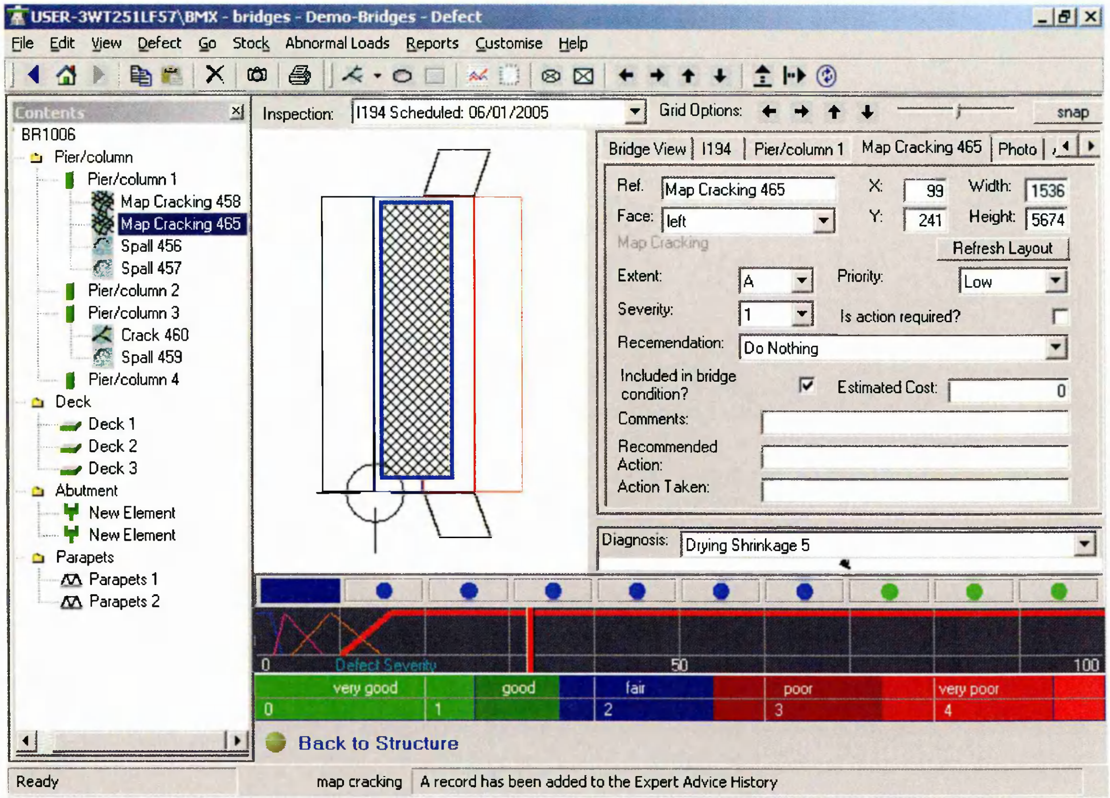



**Figure 5.20 Large map-crack. User selects 'chloride corrosion' image**

The following figures (5.21 to 5.25) show the images the users can select from to represent pattern cracking defects. Importantly, the images are not titled. Titles could prejudice a user's choice of image. Within the computer program, the images are identified as numbers.

**Figure 5.21 Images 1 & 2**

Image 1 represents carbonation cracking. Image 2 represents chloride cracking.

**Figure 5.22 Images 3 & 4**

Image 3 represents cracking from freeze thaw cycles and image 4 is typical of the early stage of AAR.

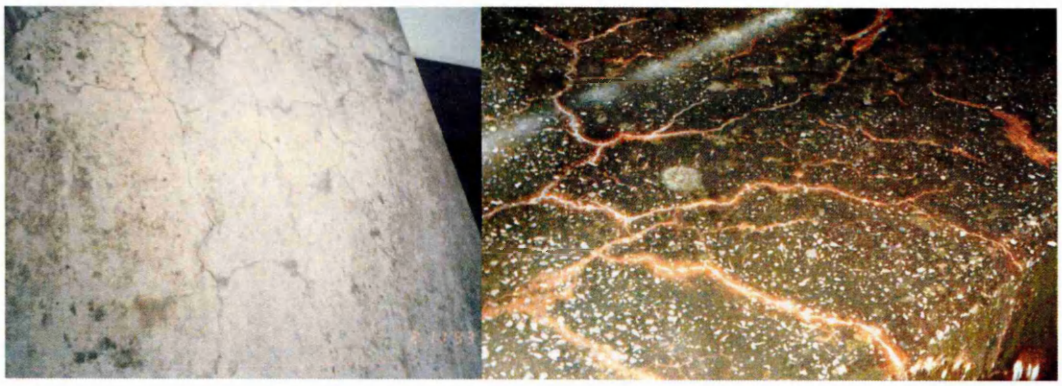


**Figure 5.23 Images 5 & 6**

Images 5 and 6 are more advanced forms of AAR, with image 6 showing the gel-like deposits which form around AAR cracking.

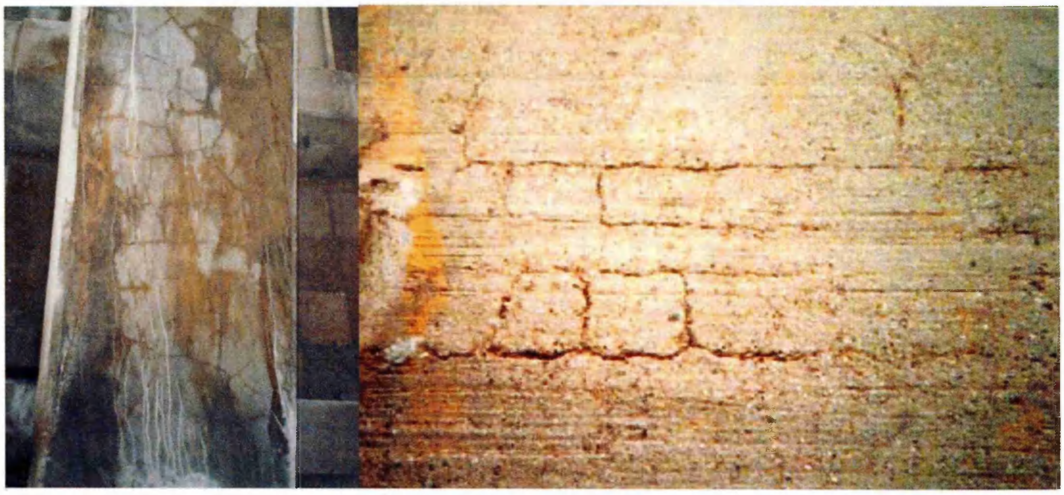


**Figure 5.24 Images 7 & 8**

Image 7 shows drying shrinkage in a new repair. Image 8 shows plastic settlement.

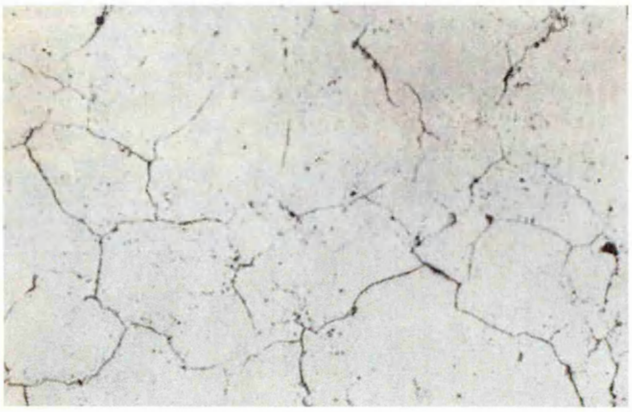


**Figure 5.25 Image 9**

Image 9 shown crazing.

Indications of the scales at which the images should be viewed are provided. The image selected by the user, and additional information provided, determine which of the tables derived by the expert should be used to establish the position of the severity zones.

This exercise to determine zone apex positions was repeated for each of the different pattern cracking causes and is shown in Tables 5.2 to 5.5.

**Table 5.2 Position of zone apexes for AAR**

| Image | nothing | cosmetic | minor | major |
|---|---|---|---|---|
| mild | 2.5 | 12 | 21.5 | 37 |
| moderate | 2 | 11 | 20 | 35 |
| severe | 1 | 10 | 17.5 | 32 |

**Table 5.3 Position of zone apexes for Freeze Thaw damage**

| Image | nothing | cosmetic | minor | major |
|---|---|---|---|---|
| mild | 17 | 62 | 90 | |
| moderate | 14 | 57 | 85 | |
| severe | 11 | 53 | 79 | |

Table 5.3 shows that regardless of the extent of a freeze-thaw defect, it cannot be considered as a major repair.

**Table 5.4 Position of zone apexs for Plastic Shrinkage, Crazing, and Drying Shrinkage**

| Defect | Image | nothing | cosmetic | minor | major |
|---|---|---|---|---|---|
| Plastic Shrinkage | - | 9 | 64 | 98 | |
| Crazing | - | 20 | 98 | | |
| Drying Shrinkage | moderate | 7 | 64 | 93 | |

**Table 5.5 Position of apexes for carbonation induced cracking**

| Image | nothing | cosmetic | minor | major |
|---|---|---|---|---|
| mild | 3 | 9 | 16 | 23.5 |
| moderate | 2 | 8 | 15 | 22.5 |
| severe | 1 | 7 | 14 | 21.5 |

#### 5.5.2.1 Secondary zone positions

At the first stage of questioning, when the experts were asked to consider the zone apex positions, their decisions were based on the three pieces of knowledge that the expert system would have at the initial stages of decision making: the pattern crack size as a percentage of the overall element size, the image selected by the user to represent the cracking, and (for some images) a textual description the user was asked to select rating the severity of the cracking. The experts were told that no further information on the defect was available at that stage, but importantly, the defect could have associated features such as staining and spalling. The experts were asked to factor into their judgements, assumptions based on their past knowledge about what other factors might be affecting the typical defect.

At the second stage of interaction, the user may enter additional information about the defects, such as:

- Amount of staining associated with cracking
- Amount of spalling associated with cracking
- Amount of seepage associated with cracking

Each of these factors is judged by the user, using example images to inform their selections. These factors are judged on a scale from 0 to 100. With 0 representing no staining (or no spalling, no seepage), and 100 representing what the experts would assume as the worst incidence of staining etc. which could possibly be associated with the defect.

The expert panel was presented with the following question (with diagrams and graphs to assist).

"If after the first stage of data input, you judged the apex of the 'Major' zone to be at 10 (a very severe defect). How severe would you expect corrosion staining to be (on a scale of 0 to 100) for you not to change your opinion regarding the position of the apex?"

The experts discussed their answers. If the user indicated that corrosion staining at the defect was rated as 100%, then the defect would be worse than the experts had assumed at the initial stage – if this was the case, the position of the apex of the Major zone would change, perhaps from the previous 10 to 8. As a result, smaller defects will fall into the Major repair zone. Figure 5.26 and Figure 5.27 demonstrate this graphically for a spall defect (crack defects also use this technique).

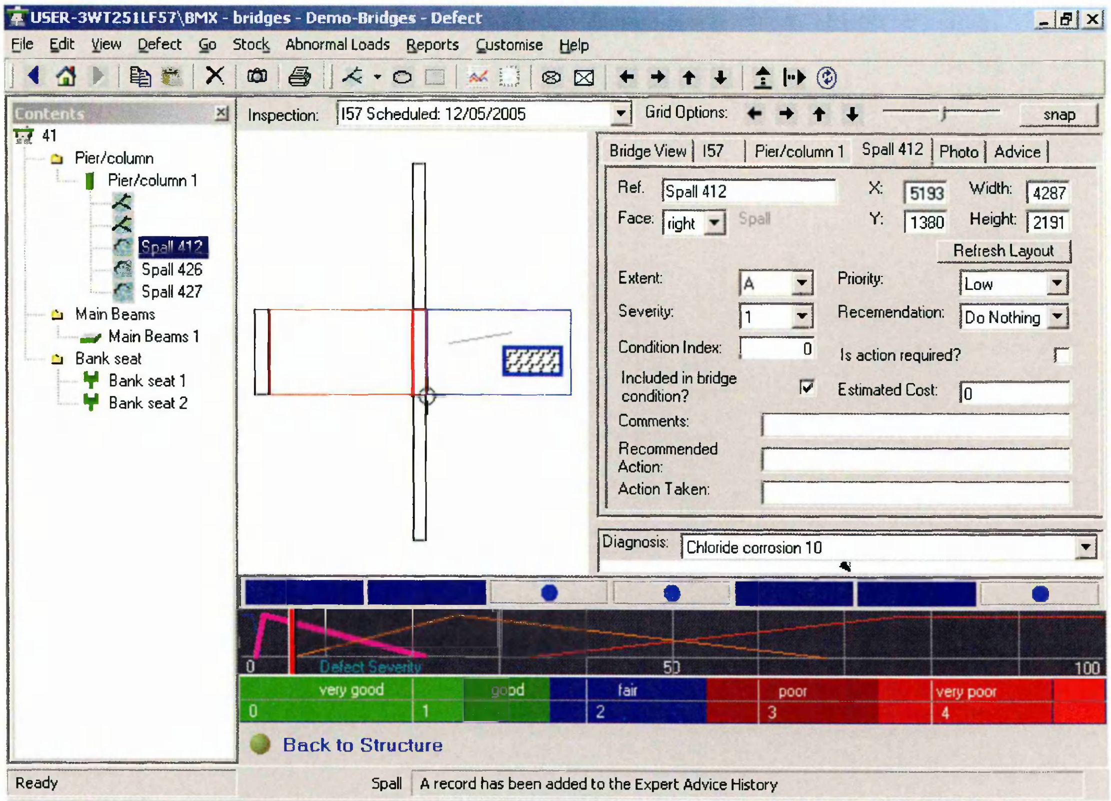



**Figure 5.26 40mm deep spall**

Figure 5.26 shows a 40mm deep spall. The apexes of the zone triangles have been positioned in accordance with section 5.5.3. The spall covers approximately 5% of the element – as a result it falls into the Cosmetic repair zone. In Figure 5.27, the user has added additional information – that 100% of the reinforcement is exposed and severely corroded. As a result of this extra information, using the technique outlined below, the zone apex positions shift, and the defect is now rated as a Minor repair.

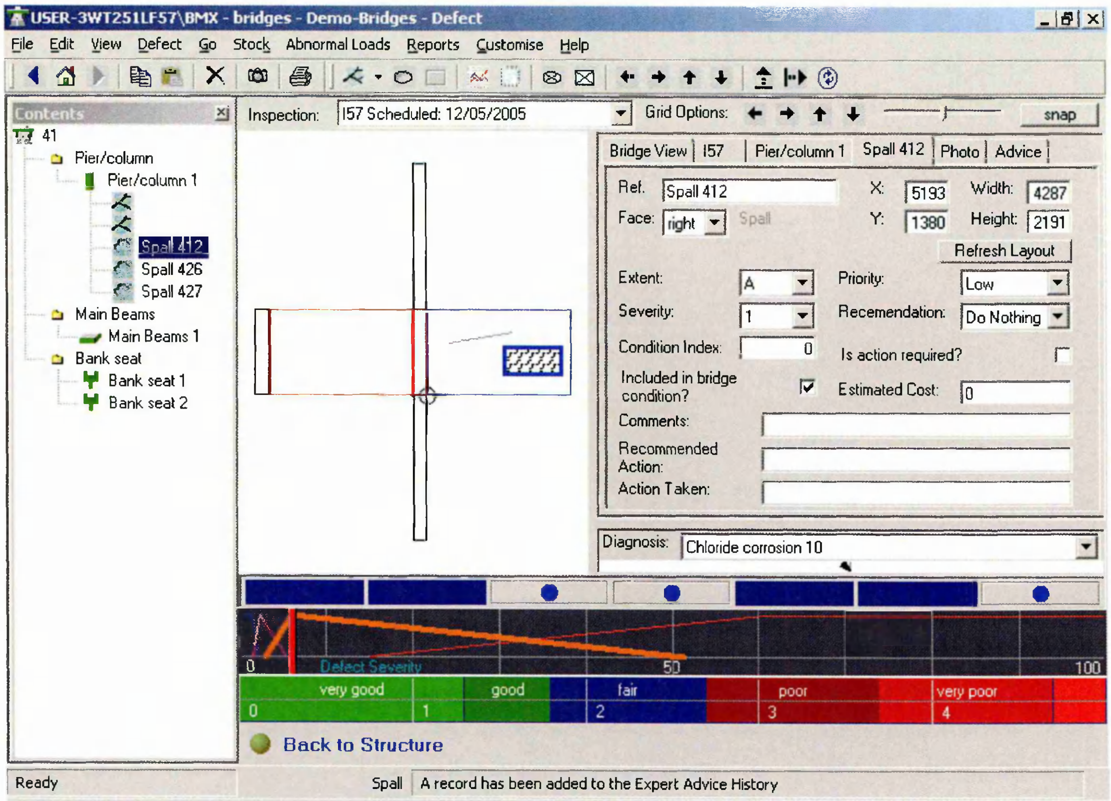



**Figure 5.27 40mm deep spall with exposed, corroded reinforcement**

If the user indicated that there was no corrosion staining associated with the defect, then the experts agreed that the defect would not be as serious as they had assumed, in this case the apex of the Major zone might move, for example, from 10 to 14. As a result, larger defects might not fall into the Major zone, but may be rated as Minor defects. Finally the experts agreed that for the question asked, a factor of approximately 37.5% would not make them change their initial opinion about the severity of the defect and, therefore, the position of the apex of the Major zone would not change.

The initial question was repeated, again for the Major zone, but this time it was assumed that after the first series of data input, the apex of the zone was positioned at 30%. The experts repeated the exercise and decided that a secondary effect from corrosion of 33% would not make them change the apex position of the major repair zone from its initial position of 30%.

Furthermore, the process of questioning was repeated for the three other repair zone categories. The experts' opinions were plotted graphically, this is shown in Figure 5.28.

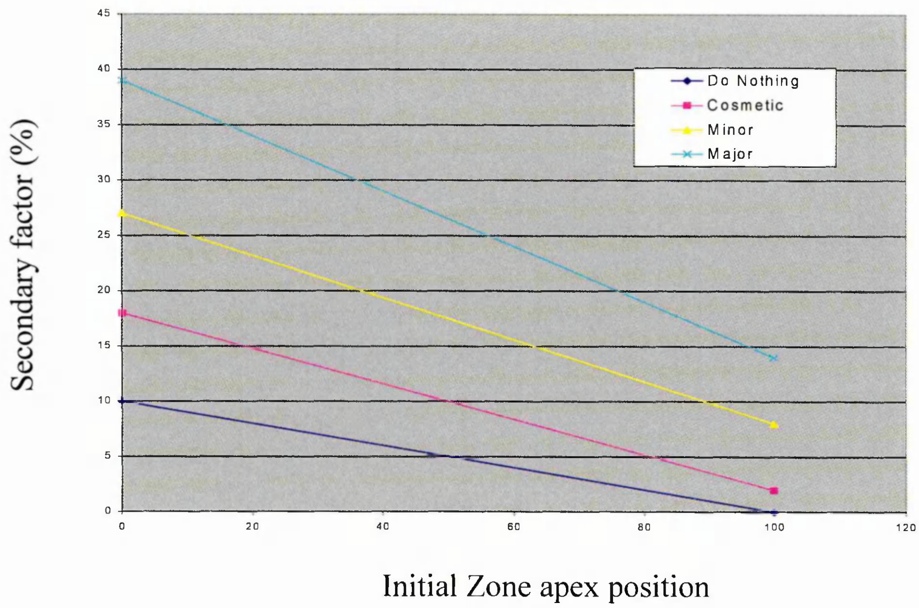



**Figure 5.28 Secondary zone movement graph**

In effect, the lines in Figure 5.28 represent how severe the experts expected the defect to be when they were presented with only limited information. For example, after the initial data input stage, the expert system has two pieces of information: the size of the defect in relation to the size of the element, and the representative image chosen. It is these two pieces of information which the experts used to position the zone apexes (beginning Table 5.1), however, whilst knowing only these two pieces of information, the experts made conscious judgments about the likelihood of the presence of other indicators of concrete distress (staining, corrosion, exposed reinforcement etc). Therefore, when deriving the zone positions the experts anticipated the presence of these 'secondary effects'. Henceforth, it is possible for a particular defect to exhibit secondary effects to a lesser or greater degree that those initially estimated to be present. The experts' assumptions, now obtained graphically, can be used to amend the initial positions of the apexes of the zones as further information becomes available. If a defect exhibits worse secondary effects than allowed for in the initial zone positions, then zone positions shift, and, for example, where a defect may have fallen into the Minor repair zone it would thereafter fall into the Major repair zone.

The lines in Figure 5.28, can be represented by their slopes and intercepts as shown in Table 5.6.

**Table 5.6 Secondary zone movement constants**

| | Slope | Intercept |
|---|---|---|
| Do Nothing | -0.1 | 10 |
| Cosmetic | -0.16 | 18 |
| Minor | -0.19 | 27 |
| Major | -0.25 | 39 |

The following method is used to amend the zone apex positions when secondary information becomes available. Each zone apex will be moved by an amount equal to the difference between the assumed severity of spalling, cracking, staining etc. and the actual severity entered by the user (multiplied by a factor).

For example, say the user has selected the 'Carbonation cracking' image and rated the cracking as moderate. The apex of the Major repair zone is set to 22.5 in accordance with Table 5.5. The user goes on to rate the amount of staining at 75%. Using Table 5.6, the amount of staining the experts anticipated would be present can be determined as follows:

Expected amount of staining = slope * apex position + intercept

= (-0.25 * 22.5) + 39

= 33.4%

It was agreed with the experts that the zone apex should be moved a distance equal to the difference between the anticipated secondary defect severity and the value entered by the user (multiplied by a factor).

Difference between anticipated and actual staining = 75 – 33.4

= 41.6%

This figure is then multiplied by a factor depending upon the particular secondary effect. Under initial review, these factors perform satisfactorily at 0.2. Though there was agreement amongst the expert panel that these factors should be calibrated during field testing of the program (Table 5.7).

**Table 5.7 Adjustment factors for secondary zone movement**

| Secondary effect | Adjustment factor |
|---|---|
| Staining | 0.2 |
| Spalling | 0.2 |
| Seepage | 0.2 |

Therefore the total zone apex adjustment for the example given:

Adjustment = 0.2 * 41.6 = 8.3%

As the defect is more serious than anticipated, the zone apex is adjusted thus:

New apex position = Original position – 8.3

22.5 – 8.3 = 14.2

This operation is completed for all four zones.

### 5.5.3 Spall defects

Each spall defect entered into the expert system by a user will be placed into one of the four repair categories. Whereas pattern cracking uses information on images selected by the user to define the apex positions of the zones; zone apex positions for spall defects are determined, initially, based on spall size and depth.

A question was asked of the expert panel. "If the size of a spall was 2% of the element area, what would the depth of the spall have to be in order for you to class the defect in the 'Do Nothing' zone?" The expert panel used graphs and diagrams to reach a decision, and the process was repeated for different spall sizes and different zone types. This session of questions delivered the graph shown in Figure 5.29.

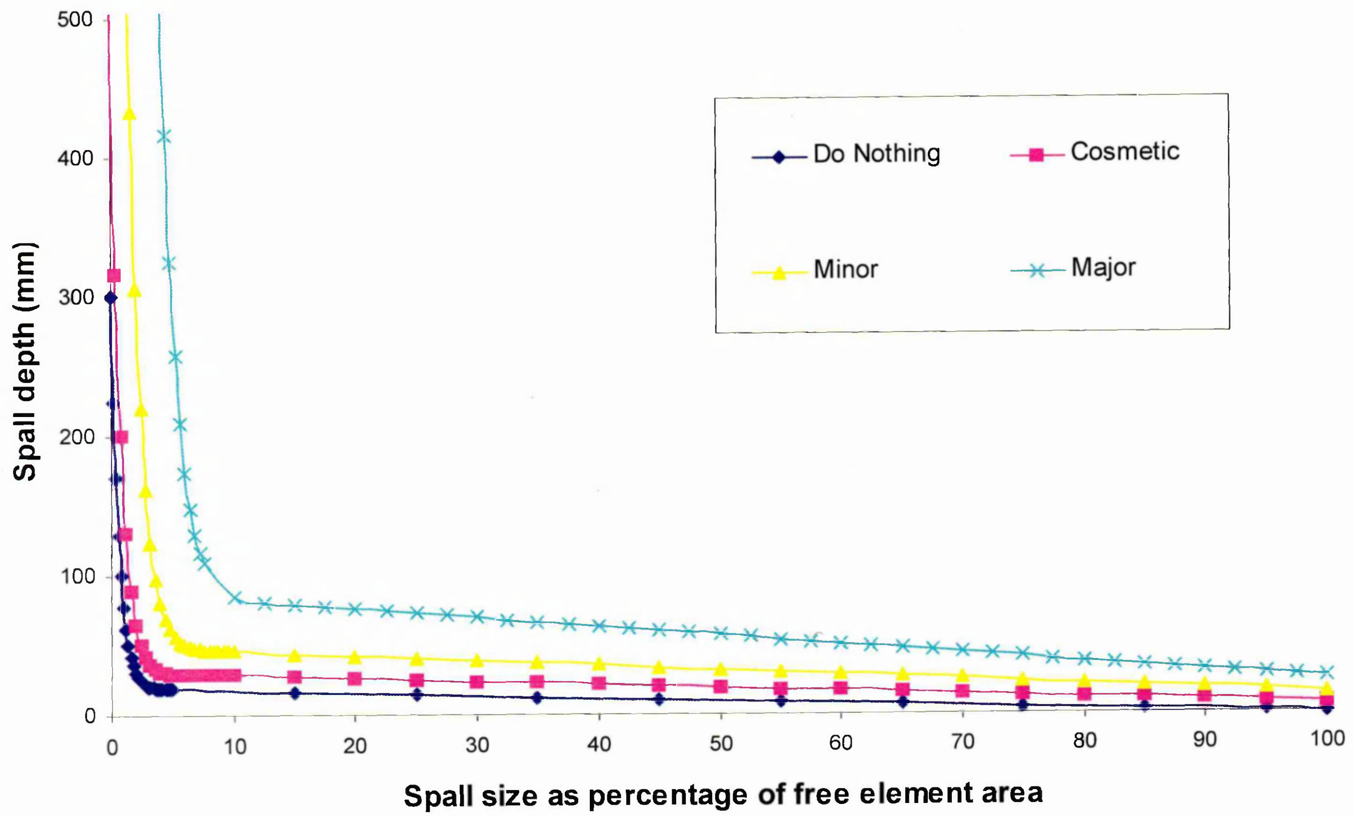



Figure 5.29 Experts' spall size/depth zone positions

When plotted graphically, the interaction between spall size and depth as determined by the expert panel could not be represented mathematically by one curve – it requires a curve and a line. Therefore, for a spall, each zone in Figure 5.29 is represented by a line showing the experts' answers to the question posed previously. In turn, these lines are represented mathematically by two equations; one equation representing the straight line portion of the zone line, and one representing the curved portion of the zone line. Separate equations were fitted to the curved and straight parts of each line. Depending on the size and depth of a spall, the program uses these equations to determine the positions of the apexes of the four zones which categorise the severity of the defect.

For each repair zone, there is a position along the y axis where the line and curve definition of the relationship between depth and size (for each zone) meet (Table 5.8).

**Table 5.8 Position where decision line and curve meet**

| Zone | Intersection (spall depth mm) |
|---|---|
| Do Nothing | 18 |
| Cosmetic | 28 |
| Minor | 45 |
| Major | 80 |

If the depth of the spall defect as entered by the user is lower than this intersection point, then the straight line relationship can be used to determine the apex position of the required zone. The constants describing the straight line portions of the relationships between zones, spall depth and size as determined by the experts are shown in Table 5.9.

**Table 5.9 Equation of line to determine zone apex positions for spalls**

| Zone | Slope | Intercept |
|---|---|---|
| Do Nothing | -0.1907 | 18.4837 |
| Cosmetic | -0.2277 | 30.2789 |
| Minor | -0.3343 | 48.4303 |
| Major | -0.6344 | 88.4455 |

For example, say a spall defect is added to the expert system, and the user informs the system that the defect is 25mm deep. To determine the apex location of the Cosmetic zone, the program first determines if the depth of defect is less than or greater than the intersection point.

Intersection point, Cosmetic = 28. Depth of defect = 25. Therefore, 25<28 and straight line relationship can be used.

Apex of Cosmetic zone = (Spall depth – intercept) / slope

= (25 – 30.279) / -0.228

= 23.15

Therefore, the Cosmetic zone apex is positioned at 23.15 (meaning that 25mm deep spalls covering 23.15% of the element area are classified as 'cosmetic defects')– any other zones whose apexes can be positioned by the straight line portions of the relationships derived by the experts also have their apexes located by this method.

If the depth of the spall defect as entered by the user is higher than the intersection point, then the curved portion of the relationship line can be used to determine the apex position of any zone as required. The constants describing the curve line portions of the relationships between zones, spall depth and size as determined by the experts are shown in Table 5.10.

**Table 5.10 Constants (K and S) for equations of curve for determining zone apex positions for spalls**

| Zone | K | S |
|---|---|---|
| Do Nothing | 1.55 | 282 |
| Cosmetic | 1.3 | 485 |
| Minor | 1 | 1923 |
| Major | 0.8 | 11340 |

Continuing the previous example, where a spall defect is added to the expert system, and the user informs the system that the defect is 25mm deep. To determine the apex location of the Do Nothing zone, first determine if the depth of defect is greater or less than the intersection point.

Intersection point, Do Nothing = 18. Depth of defect = 25. Therefore, 25>18 and the curved line relationship can be used.

$$\text{Apex of Do nothing zone} = \left(\frac{\log(\text{Spalldepth} - \text{intersection})}{-K}\right) - \left(\frac{\log S}{-K}\right)$$

$$= \left(\frac{\log(25-18)}{-1.55}\right) - \left(\frac{\log 282}{-1.55}\right)$$

$$= 12.29$$

The apex of the Do Nothing zone would be set at 12.29. This indicates that, because of the low spall depth, the defect is not considered too severe.

### 5.5.3.1 Secondary zone positions

As the program gathers more information about the nature of a defect, its decision about the severity of the defect becomes more intelligent. Say a defect is entered into the system and its size is judged at 2% of the overall element area. Then the user enters the defect depth (for a spall), the system knows how spall depth affects the positioning of the 'severity zones'. If a depth of 70mm is entered, the zone positions might place the defect in the 'Cosmetic' zone; an indication that the defect requires attention. As already discussed, the position of zones has been predetermined based on the opinion of the concrete repair experts. However, zones continue to shift as additional information about the defect is entered until finally the information collection is complete.

**Shifting zones depending on exposed reinforcement**

For every spall defect, the system assumes horizontal and vertical reinforcing steel is present (either exposed or still embedded) below the substrate surface. The program calculates the total length of reinforcing bar within the defined area of the spall, assuming a 200mm spacing between bar centres both horizontally and vertically. The program then equates this total length of reinforcement within the boundary of the spall defect with a variable known as the 'maximum possible exposed reinforcement'. Next, using a graphical technique, the user informs the program the actual length of exposed reinforcement and the program converts this to a percentage of spall reinforcement exposed, based on the maximum possible that could have been exposed, computed by the expert system.

This information on the amount of exposed reinforcement is used to move the decision zones in the same way as secondary factors moved 'map cracking' zones in section 5.5.2.1.

The expert panel was presented with the following question (with diagrams and graphs to assist).

"If after the first stage of data input, you judged the 'Major' zone to be at 10 (a very severe defect due to a high spall depth). How much exposed reinforcement would you expect (on a scale of 0 to 100), for you not to change your opinion regarding the position of the apex?"

The experts discussed their answers. If the user indicated that exposed reinforcement was rated as 100%, then the defect would be worse than they had assumed at the initial stage – if this was the case, the position of the apex of the Major zone would change, perhaps from the previous 10, to 8.

If the user indicated that there was no exposed reinforcement associated with the defect, then the experts agreed that the defect would not be as serious as they had assumed, in this case the apex of the zone might move, for example, from 10 to 14. As a result, larger defects might not fall into the Major zone, but may be rated as Minor defects instead.

This exercise was repeated for different zone apex positions and the three other different zone types. A graph (Figure 5.30) was developed very similar to that used for pattern cracking and eventually it was decided that the same graph was applicable for both cases.

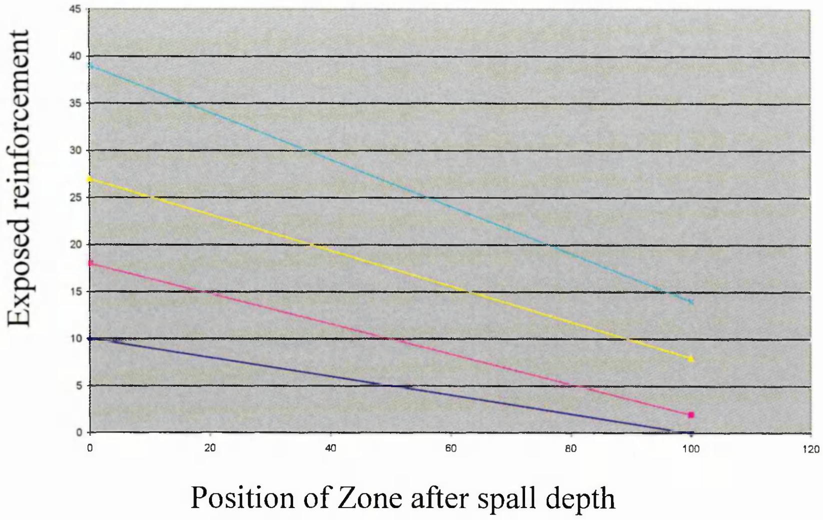



**Figure 5.30 Secondary movement of zone for spalls**

In effect, the lines in Figure 5.30 represent how severe the experts expected the secondary factors affecting a defect to be when presented with only limited information. For example, for a spall depth of 50mm, the initial question asked of the experts was how large a defect should be to be classed as a major repair. However, at this stage, they were told the spall could have other factors which may affect the judgement of severity, but at the first stage of data input they would have to assume how serious those other factors might be – factors such as the amount of exposed reinforcement and the degree of corrosion of the reinforcement – therefore, they made assumptions when initially determining the apex position of the zones. Those assumptions, now obtained graphically, can be used to amend the positions of the apexes of the zones as the additional information becomes available through additional user data input.

The lines in Figure 5.30, can be represented by their slopes and intercepts as shown in Table 5.11.

**Table 5.11 Secondary zone movement constants**

| | Slope | Intercept |
|---|---|---|
| Do Nothing | -0.1 | 10 |
| Cosmetic | -0.16 | 18 |
| Minor | -0.19 | 27 |
| Major | -0.25 | 39 |

The following method is used to amend the zone apex positions when secondary information becomes available. Each zone apex will be moved by an amount equal to the difference between the assumed amount of secondary defects and the actual amount entered by the user (multiplied by a factor).

For example, say the user has informed the system that the spall depth is 50mm. The apex of the Major repair zone is set to 60.6 in accordance with Table 5.8. Say the user goes on to rate the amount of exposed reinforcement at 75%. Using Table 5.11, the amount of exposed reinforcement the experts anticipated would be present can be determined:

Expected amount of exposed reinforcement = slope * apex position + intercept

= (-0.25 * 60.6) + 39

= 23.85%

It was agreed with the experts that the zone apex should be moved a distance equal to the difference between the anticipated amount of exposed reinforcement and the value entered by the user (multiplied by a factor).

Difference between anticipated and actual exposed reinforcement = 75 – 23.85

= 51.15%

This figure is then multiplied by a factor. Although the amount of exposed reinforcement has been used in the example, other secondary defects can affect zone positions (staining, condition of reinforcement, seepage). Therefore, the amount of zone movement determined (the difference between expected and anticipated secondary defects) is factored depending on the particular secondary effect. After initial reviews, these factors were set as shown in Table 5.12. Though there was agreement amongst the expert panel that these factors should be calibrated during field testing of the program (Table 5.12).

**Table 5.12 Adjustment factors for secondary zone movement**

| Secondary effect | Adjustment factor |
|---|---|
| Amount of exposed reinforcement | 0.5 |
| Condition of exposed reinforcement | 0.5 * percentage of exposed reinforcement |
| Seepage | 0.2 |
| Staining | 0.2 |

Therefore, the total zone apex adjustment for the example given:

Adjustment = 0.5 * 51.15 = 25.58

As the defect is more serious than anticipated, the zone apex is adjusted thus:

New apex position = Original position – 25.53

60.6 – 25.53 = 35.07

This operation is completed for all four zones.

It is of crucial importance to the development of the system that although the adjustment factors in Table 5.12 were estimated by the experts as accurately as possible, all were in agreement that these figures should be calibrated during field trials of the software.

The effect of the condition of the exposed reinforcement has to be factored by the amount of reinforcement that is actually exposed. For example, if the amount of exposed reinforcement is rated by the user at 50, and the condition of exposed reinforcement is rated at 50 (still with a spall depth of 50mm):

Expected amount of exposed reinforcement (Major) = slope * apex position + intercept

= (-0.25 * 60.6) + 39

= 23.85%

Difference between anticipated and actual exposed reinforcement = 50 – 23.85

= 26.15%

Adjustment for amount of exposed reinforcement (Major) = 0.5 * 26.15 = 13.075

Expected condition of exposed reinforcement = (-0.25 * 60.6) + 39

= 23.85%

Difference between anticipated and actual exposed reinforcement = 50 – 23.85

= 26.15%

Adjustment for condition of exposed reinforcement (Major)= 0.5 * percentage exposed reinforcement * 26.15

= 0.5 * 0.5 * 26.15

= 6.54%

Total adjustment for Major zone = 13.075 + 6.54 = 19.62

New position of Major zone apex = 60.6 – 19.62 = 40.98.

Importantly, if the user was also to specify that there was associated staining (or any other secondary defect), the apex position used in the equation to determine the expected amount of staining is the original apex position before the effect of any other secondary defects affecting the zone apex position was implemented.

#### 5.5.3.2 Spalls of unknown depth

There may be occasions where the user can identify a spall outline but the depth of the spall is not clear. The expert panel recommended that the program should accommodate this possibility. For example, a person inspecting the central pier of a motorway bridge may be able to see a spall, but unable to judge its depth accurately until such a time when traffic management can be arranged to allow close up inspection. In these cases, the expert panel was asked to position the severity zone apexes in a similar fashion to how pattern cracking apex positions were determined. The user is asked to judge the probable cause (Table 5.13).

**Table 5.13 Position of zone apexes for chloride ingress and carbonation spalls (unknown depth)**

| Spall cause | nothing | cosmetic | minor | major |
|---|---|---|---|---|
| Chloride ingress | 1 | 2.5 | 4 | 6 |
| Carbonation spall | 2.5 | 5 | 8 | 12 |

### 5.5.4 Structural cracks

For each of the three main types of defects that can affect reinforced concrete, two factors are used to determine the initial positions of the severity zones. For pattern cracking these were an image chosen by the user and the size of the pattern cracking patch. For spalling, spall size and depth. Structural cracking uses a similar method – it uses two variables, crack size and crack width, to set the initial zone positions.

For a typical pier element like that shown in Figure 5.31 the program knows the pier height, x. It also knows the total unwrapped length of the pier, y, as shown in Figure 5.32.

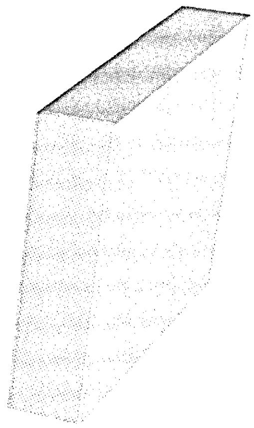


**Figure 5.31 Typical pier element**

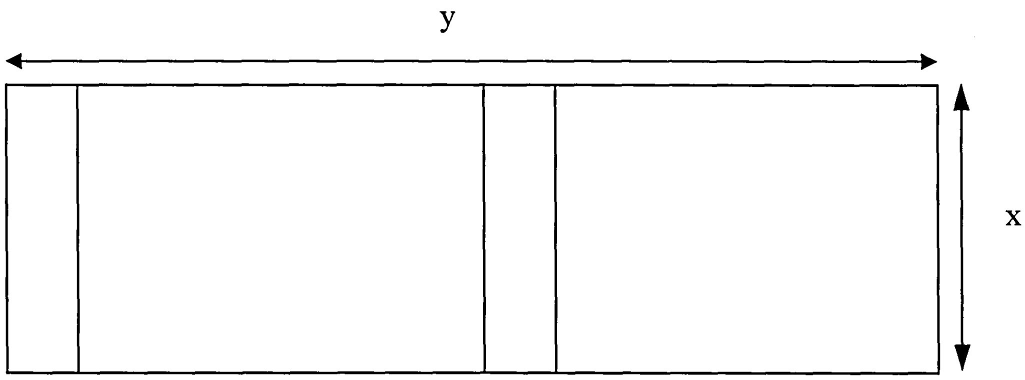



**Figure 5.32 Unwrapped pier**

When the user adds a crack onto the element (as in Figure 5.17), the system considers the crack as if drawn on a canvas of dimensions x, y (Figure 5.33).

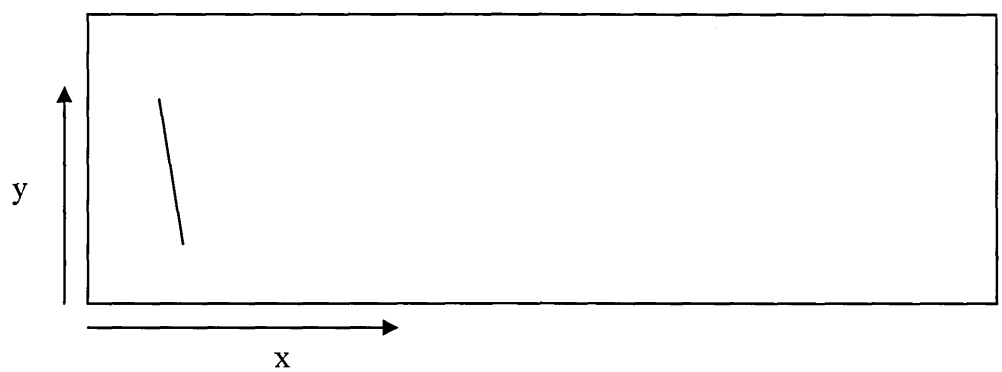



**Figure 5.33 Element canvas for crack size determination**

The expert system projects the crack onto the edges of the unwrapped element in order to obtain the dimensions p and q, as shown in Figure 5.34.

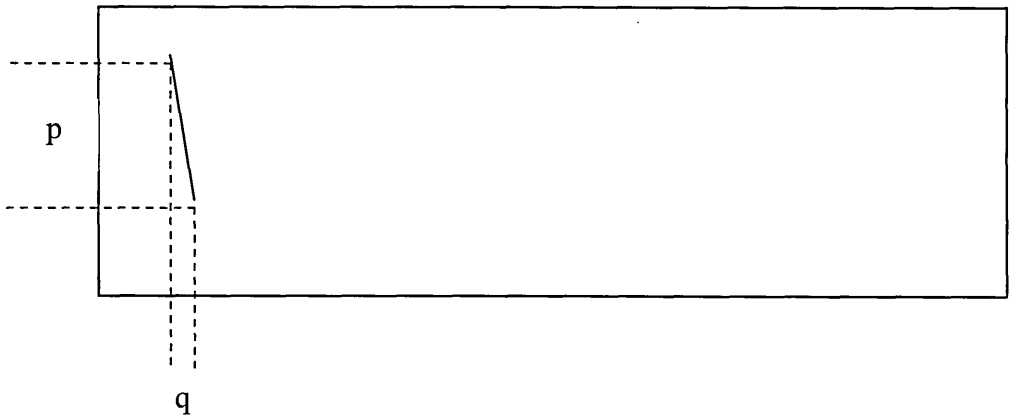



**Figure 5.34 Projecting crack onto element edges**

The program determines the variable, size of crack using the following expression:

(p+q) / (x+y) *100 = size of crack.

It was decided during a series of interviews with experts that the width of a crack and the depth of spall could in some way be related, in terms of how seriously they affect an element. A question was asked of the experts. "If all you knew about a spall was its depth of 40mm (no information about the extent of the spall itself), how wide would a structural crack have to be for you to be as concerned about the crack defect as you were about the 40mm deep spall defect?" Although the experts were in agreement that the premise of the question was unusual, they clearly understood how the information they were providing was being used in the expert system. They reached the conclusion that, in the absence of all other information, they would be equally concerned about a 40mm deep spall and a 4mm wide crack. It was established through similar questioning, that this factor of 10 could generally be employed to relate the seriousness of all spall depths and crack widths. It was agreed that the factor should be calibrated during field trials of the system. The equations determining the zone apex positions of spalls (section 5.5.3) were examined to test their applicability to assessing the severity of structural cracking. Using the graphs and equations in section 5.5.3, the variable 'size of crack' replaced 'size of defect' (which was measured as a percentage of the overall element area). The variable 'spall depth' is replaced by the width of the crack multiplied by ten. The position of the apexes of the zones are then determined exactly in accordance with the procedures for spalling. Similarly, zone movement through the addition of secondary defects is governed in the same way as for spalling.

### 5.5.5 Miscellaneous defects

Miscellaneous defects require no diagnosis by the expert system. They are defects which a laymen could reasonably be expected to identify, and their cause is generally effects of workmanship.

#### 5.5.5.1 Blow Holes, Sand Streaking

If the user of the expert system noticed an area of blow-holes or sand-streaking, they would add a patch of blow-holes or sand-streaking to the element. The apexes of the zones are positioned in accordance with Table 5.14.

**Table 5.14 Zone apex positions for blow-holes or sand-streaking**

| Image | nothing | cosmetic | minor | major |
|---|---|---|---|---|
| - | 20 | 98 | | |

### 5.5.5.2 Honeycombing

Similarly, areas of honeycombing observed by the expert system user would be added directly onto the concrete element. Honeycombing in large volumes can be regarded as serious, hence for this defect all four severity decisions are possible. The position of the zone apexes is determined in accordance with Table 5.15.

**Table 5.15 Zone apex postions for honeycombing**

| Image | nothing | cosmetic | minor | major |
|---|---|---|---|---|
| moderate | 1.5 | 5.5 | 10.5 | 18.5 |

## *5.6 Uncertainty in deciding severity.*

Any single defect, be it spall, pattern cracking or a structural crack, can at any time (within the expert system) be in two distinct states:

**Complete** – all information has been entered for that defect. For example, for a spall, the user has added the spall to the element on screen (so the expert system is aware of the spall's size and location), has entered the depth, shape, amount and condition of exposed reinforcement and also informed the system if the spall is within splash zones or close to trafficked carriageways.

**In Progress** – The defect is currently being entered into the system. For example, for a spall, the user may have entered the spall size and depth but no further information.

Once a defect is '**Complete**' it is at this stage that the system calculates the defect's effect on the 'Element condition' (section 5.7) and also calculates a variable called 'Uncertainty' based on the completeness of information offered. Also at this stage, the knowledge base will diagnose the defect.

The key area where uncertainly becomes important in the expert system is when a defect, on the severity graph, overlaps between two severity zones. For example, the defect in Figure 5.19 lies in both the 'Cosmetic' and 'Minor' severity zones. The fact that defects can lie in two zones is considered a benefit of the technique developed and not a drawback as it simulates the decision making process of a human expert whose judgement gets confirmed as more data about a flaw becomes available. Assessments of defect severity are not purely scientific, and often a degree of estimation based on expert judgement is employed in decision making.

If a defect does lie in two zones, this represents the fact that the system is not certain about the severity of the defect, and, it has still to make a decision.

The case shown in Figure 5.35 is taken as an example. Figure 5.35 shows an enlarged diagram of a defect falling into two repair zones, say Do Nothing and Cosmetic. (The display of the zone positions shown to the user is necessarily small within the software).

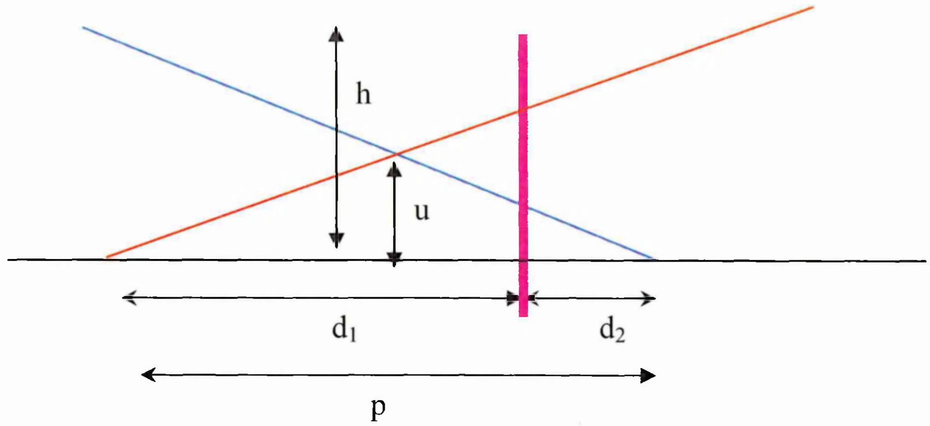



**Figure 5.35 Handling uncertainty**

Where h = height to zone apex

u = height to zone intersection

p = width of zone overlap

$d_1$ = distance of defect into rightmost zone

$d_2$ = distance of defect into leftmost zone

In order to determine into which severity zone to place the defect, the expert system first determines the variables:

$d_1/p$ = % distance of marker into Cosmetic zone

$d_2/p$ = % distance or marker into Do Nothing zone

The difference between these two variables helps to determine the dominant zone

$$(d_1/p) - (d_2/p) = V$$

The height h, is not in itself significant in the technique to determine severity zone positions. However, the ratio between the height h and the height of the zone intersection u, is important. A combination of V and u will decide the zone classification of a defect falling within two zones.

The possible range for the variable V is between 100 and -100.

The value u represents the height at which the arms of adjacent zone triangles intersect. It is measured on a vertical scale from 0 to 100. The apex of the zones is positioned at 100 units above the baseline. Initially, for spalls, pattern cracks and structural cracks the zone intersection value (or the uncertainty), u, is 60. That is, the lines intersect 60 units above the baseline. This is the value of u immediately after a defect has been added. This value of u begins to reduce as more information is added, and the expert system can be more certain about its decision. The value of u is reduced when new information is entered according to the Table 5.16 :

**Table 5.16 Change in uncertainty as data is added**

| Information entered | Reduction in u (units) |
|---|---|
| Spall depth | 5 |
| No exposed reinforcement | 15 |
| Exposed reinforcement | 5 |
| Condition of reinforcement | 5 |
| Width of crack | 5 |
| Splash zone question answered | 5 |
| Staining question answered | 5 |
| Seepage question answered | 5 |

As data is added to a defect, the value of u changes. The value of h is constant at 100. Figure 5.36 shows the graph which finally decides into which severity zone a defect lying in two zones should be placed.

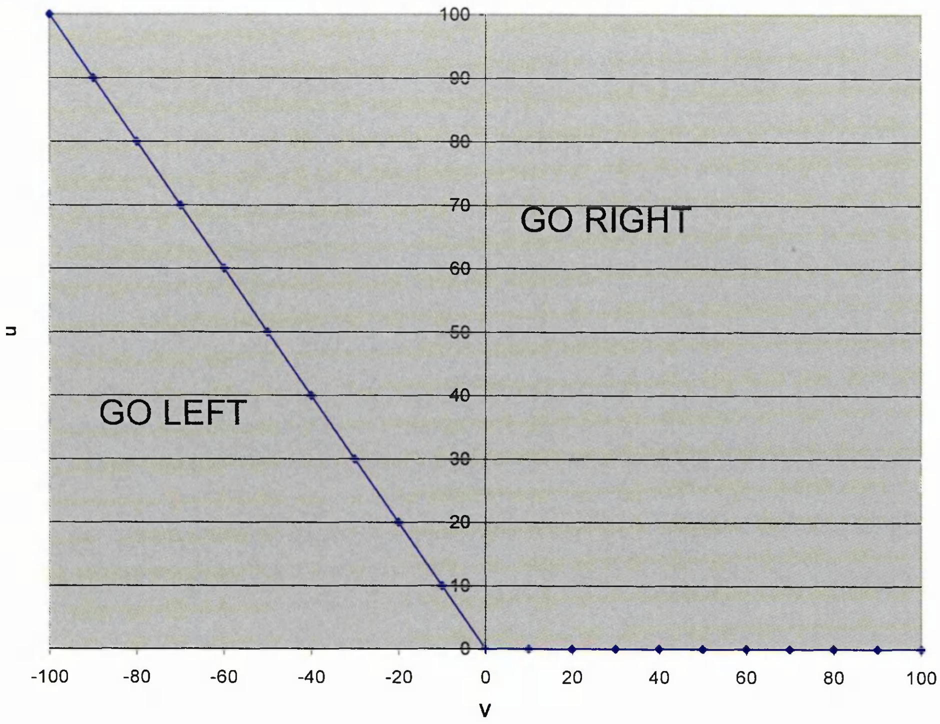



**Figure 5.36 Zone decision graph**

For example, say the two zones into which a single spall defect falls are Do nothing and Cosmetic. Say the defect is 30% into the Cosmetic zone ($d_1/p = 30$) and therefore 70% into the Do Nothing zone ($d_2/p = 70$).

$$V = 30 - 70 = -40$$

After the defect was initially entered the value of u is set to 60%. But say the user has entered the spall depth and informed the system that there is no exposed reinforcement.

In accordance with Table 5.16 this gives a 20 point reduction in uncertainty, hence:

$$U = 60 - 20 = 40$$

Using the graph in Figure 5.36, when u = 40 and V = -40, the decision is borderline. In this instance the expert system is conservative and selects the worst case. Hence the defect is placed in the Cosmetic classification. The final classification (which, if a defect lay within two zones is determined using this technique) is utilised by the knowledge bases (e.g. the action table rule in Figure 5.15)

The expert system uses the equation of the line in Figure 5.36, the equation of the line is

u = 1 * -V.

### 5.6.1 Types of uncertainty

Two distinct types of uncertainty have been identified which are addressed by the expert system these are called expert uncertainty and system uncertainty.

#### 5.6.1.1 Expert Uncertainty.

If the system has been presented with all the information it needs in order to make a good decision about the severity of a defect, on occasions, due to the nature and extent of the information provided, it may still be unclear into which severity zone a defect should be placed. Instances can arise where even the expert would be unsure; the engineer could be certain about the nature of the problem, but uncertain about which option to choose. Under the technique developed in this project, such Expert uncertainty can represented as horizontal uncertainty - as it falls between two zones, this represents a region where even an expert might have some conflict over how to rate a defect.

#### 5.6.1.2 System Uncertainty.

Upon inspecting a defect, an engineer gathers some information immediately and concurrently; size, geometry, location, staining, seepage etc. This is the information on which the engineer will immediately build up an impression about the defect. No expert system can take this human approach – expert systems receive information piecemeal, and it is this function where the greatest contrast between the expert's approach and that of the expert system is seen.

The system takes in information more slowly (although the intake of information can be seconds apart, this is clearly slower that the instantaneous intake of a human). So the system clearly takes longer to make its decision. It has passages of time when it knows it does not yet have the complete picture and that more information is about to be received. However, in the developed system, the user can still see how each piece of information is affecting the final decision. As each further piece of information is entered, the system becomes more certain about the accuracy of its decision. This is system uncertainty.

System Uncertainty is indicated on the vertical scale as the height of the intersection of adjacent zones. As more information is entered into the system, the intersection of adjacent zones lowers.

## 5.7 Contribution of each defect to element severity

Each defect on an element must in some way contribute to the overall condition of the element. A technique has been developed that allows the effect of each individual defect on an element to be assessed.

A panel of concrete repair experts were asked the following question "Consider that the condition of an element can be rated between zero and one hundred. With zero being an element without defects and one hundred an element so severely affected by defects that it is structurally and aesthetically redundant. If a single defect was diagnosed as being caused by chloride ingress, and that defect covered 2% of the element area, how much would this degrade the element condition on the scale of zero to one hundred?"

The unanimous answer to this question was that the effect on the element condition depends on the severity of the defect. Therefore, a method was developed to numerically rate the severity of each defect.

### 5.7.1 Effect of pattern cracking on element severity

In order to determine the severity of a pattern cracking defect, a user selects an image which best represents the defect (section 5.5.2). The user may also be required to judge the severity of the defect in comparison with the image they selected. It is after this stage that the positions of the zone apexes are set in accordance with section 5.5.2. Say the user has selected the image which represents chloride induced corrosion and rated the defect as moderate. The position of the apexes which the expert system then sets can be said to be the apex positions of the typical moderate chloride corrosion pattern cracking defect. The user may go on to add additional information and zone apex positions may change, however, the original zone positions before the additional information was added will need to be referred back to at future stages.

To begin the process of determining the effect of a pattern cracking defect on the element it affects, firstly, the expert system must determine how far the defect marker (representing defect size) falls into the assessed severity zone of the defect. In the example of Figure 5.37 (which shows a prototype of the severity marker), the user has inputted into the system all the necessary information pertaining to the pattern cracking defect.

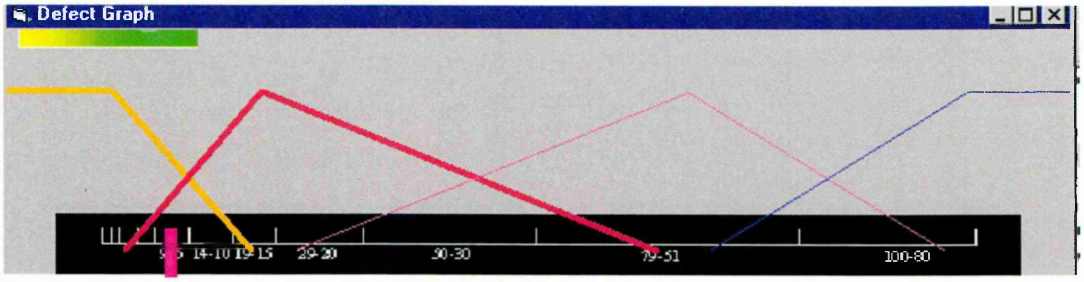



**Figure 5.37 Determining effect of defects on element**

The defect falls into the 'Cosmetic' severity zone. Next the system calculates how far the defect is into that zone. For example, the cosmetic repair zone begins at 10 and ends at 70 (a defect which is not particularly severe), and the defect size is at 15 (so the defect covers 15% of the element surface area). The total width of the zone:

Cosmetic Zone width = 70 – 10 = 60

Percentage of marker into cosmetic zone = (15-10)/60 = 8.3%.

Next, consider the position of the zone apexes after the initial stages of data entry (when the system knew the image selected by the user to represent the pattern cracking defect and the user judged severity rating). These zone apex positions were set in accordance with Table 5.1 and they represent, in the opinion of the expert panel, the zone positions for the average moderate pattern cracking defect probably caused by chloride ingress. Thereafter, take the distance that the defect was calculated to in the Cosmetic zone (8.3%), and determine what the size of the defect would be if it was 8.3% into the Cosmetic zone for the average moderate pattern cracking defect caused by chloride ingress.

The expert system determines the initial zone position of moderate chloride corrosion, i.e. where the Cosmetic zone arms initially hit the axis based on Table 5.1 . These points are 2.5 and 6.5. Therefore, the width of the Cosmetic zone at this stage is 4.

The system determines that 8.3% into a zone 4 wide = 4 * (8.3/100) = 0.332.

Therefore, the system determines a variable called the 'effective size of defect'. A very severe defect covering 5% of an element could be said to be equivalent, in terms of its effect on the element, to an average defect covering 10% of the element. This concept defines the variable 'effective size of defect' well.

Taking the distance the moderate chloride pattern cracking defect falls within the Cosmetic zone after all information has been added (8.3%) this is then compared to a defect 8.3% into the Cosmetic zone after the initial zone positions were set in accordance with Table 5.1 – the equivalent average defect.

Size of equivalent average defect = $Z_s + (D_a * Z_{sw})$

Where $Z_s$ = Position where left hand leg of severity zone in which the defect has been classed intersects with the horizontal axis for the average defect.

$D_a$ = distance defect falls into severity zone after all information was entered

$Z_{sw}$ = width of zone at initial stage

Size of equivalent average defect = 2.5 + (0.083 * 4)

= 2.832%

Therefore, for the given example, the defect size is 15% of the element surface area, but because the defect is not severe, the zones reflected this by spreading out to the right. The calculations determined that an average defect covering 2.832% of the element would have been as severe as the registered defect (which was less severe than average, but covered 8.5% of the element).

With a technique for comparing defects of any size and severity with an 'average defect' developed a question was once again asked of the expert panel. "Consider that the condition of an element can be rated between zero and one hundred. With zero being an element without defects, and one hundred an element so severely affected by defects that it is structurally and aesthetically redundant. If a single defect was diagnosed as being caused by chloride ingress, and that defect covered 2% of the element area, how much would this degrade the element condition on the scale of zero to one hundred?" The experts were now clear that every defect was to be given an 'effective size', balancing the actual size of the defect so it is represented by an average defect of a size judged to have a comparable effect on the overall condition of the element as the defect under consideration.

The experts arrived at an answer, and the question was repeated for different defect sizes, with the intention of graphically representing the experts' decisions. The relationship formed could be modelled by a simple equation. Thus, the need for literally hundreds of rules (e.g. if the defect is 50% of the element then the effect on the overall element is a condition rating of 30% etc.) is replaced by an equation.

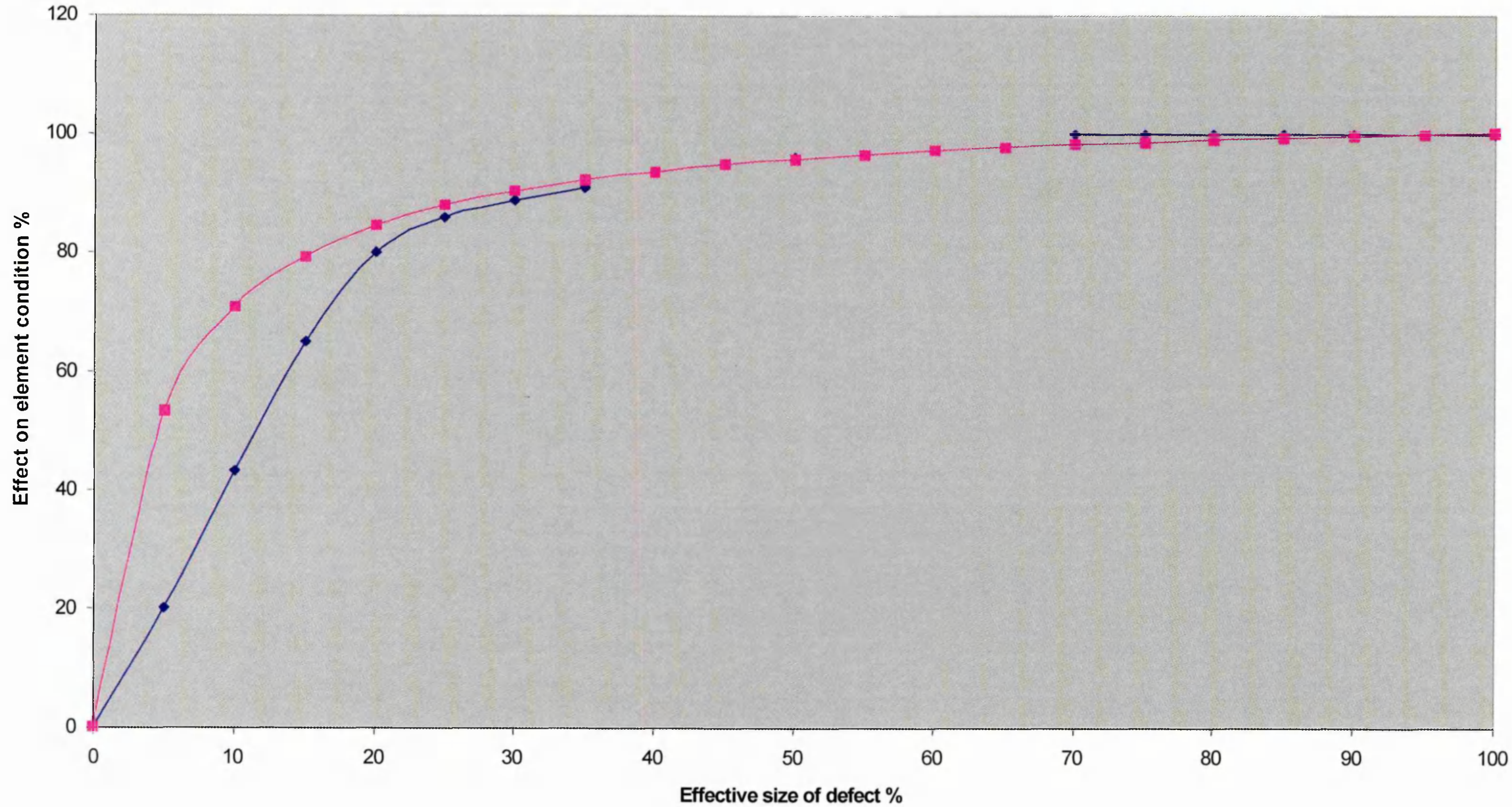
A graph was produced of the expert answers and is shown in Figure 5.38.

In Figure 5.38, the blue line represents the expert opinions on how the size of a chloride cracking defect affects the condition of an element. The magenta line represents the mathematical model of the experts' opinions. This colouring system is adopted for all similar graphs.

The equation which models the magenta line is

$$\text{Effect on element condition} = \text{Effective size} / (0.0465 + 0.0095 * \text{Effective size})$$

The figures 0.0465 and 0.0095 are constants for chloride induced cracking and, like many variables developed for the program, could be calibrated to enhance their performance during field trials of the system.




Effect on element condition = Effective size / (0.0465 + 0.0095*Effective size)

**Figure 5.38 Determining the effect of chloride cracking on element condition**

The experts' explanation of the form of the graph is that the effect of chloride cracking defects on the element condition increases approximately proportionally with the increase in defect size until a point is reached where the element is seriously affected by chlorides, beyond this point an increase in the amount of chloride defect covering the element does not affect the element condition at the same rate, as the defects already present were very serious and more of the same does not radically change the outlook for the element.

Therefore, for the defect in the previous example.

Size of defect = 8.5%

Severity zone decision = Cosmetic

Knowledge base decision = Chloride corrosion

Effective size of defect (equivalent size of average defect) = 2.832%

Effect on element = 2.832 / (0.0465 + 0.0095*2.832)

Effect on element = 38.58%

Therefore, at this stage, the following actions have taken place:

- User adds pattern cracking patch, system determines size
- User selects representative image and adds additional information
- System judges severity as 'Cosmetic'
- Knowledge base judges defect as 'Chloride corrosion'
- System calculates effective size of defect (size of equivalent average defect)
- Chloride corrosion graph (Figure 5.38) used to determine effect of defect on element condition

Therefore, the condition of the element is reduced from 0 to 38.58%.

It is important to note that the graph (and the other graphs which will be developed shortly) works cumulatively. If another chloride induced cracking defect is encountered, its effective size should be added to the effective size of previous chloride cracking defects in order to find the total effect of all chloride cracking defects on the element condition. This total effective size should be used to judge the effect these defects have on the element condition as whole.

For example, consider the previous defect and an additional defect:

Defect 1 – effective size = 2.832%

Defect 2 – effective size = 5 %

Cause of both defects = Chloride cracking

Total effective size of chloride cracking defects = 7.832%

Effect on element severity = 7.832 / (0.0465+0.0095 * 7.832) = 64.77%

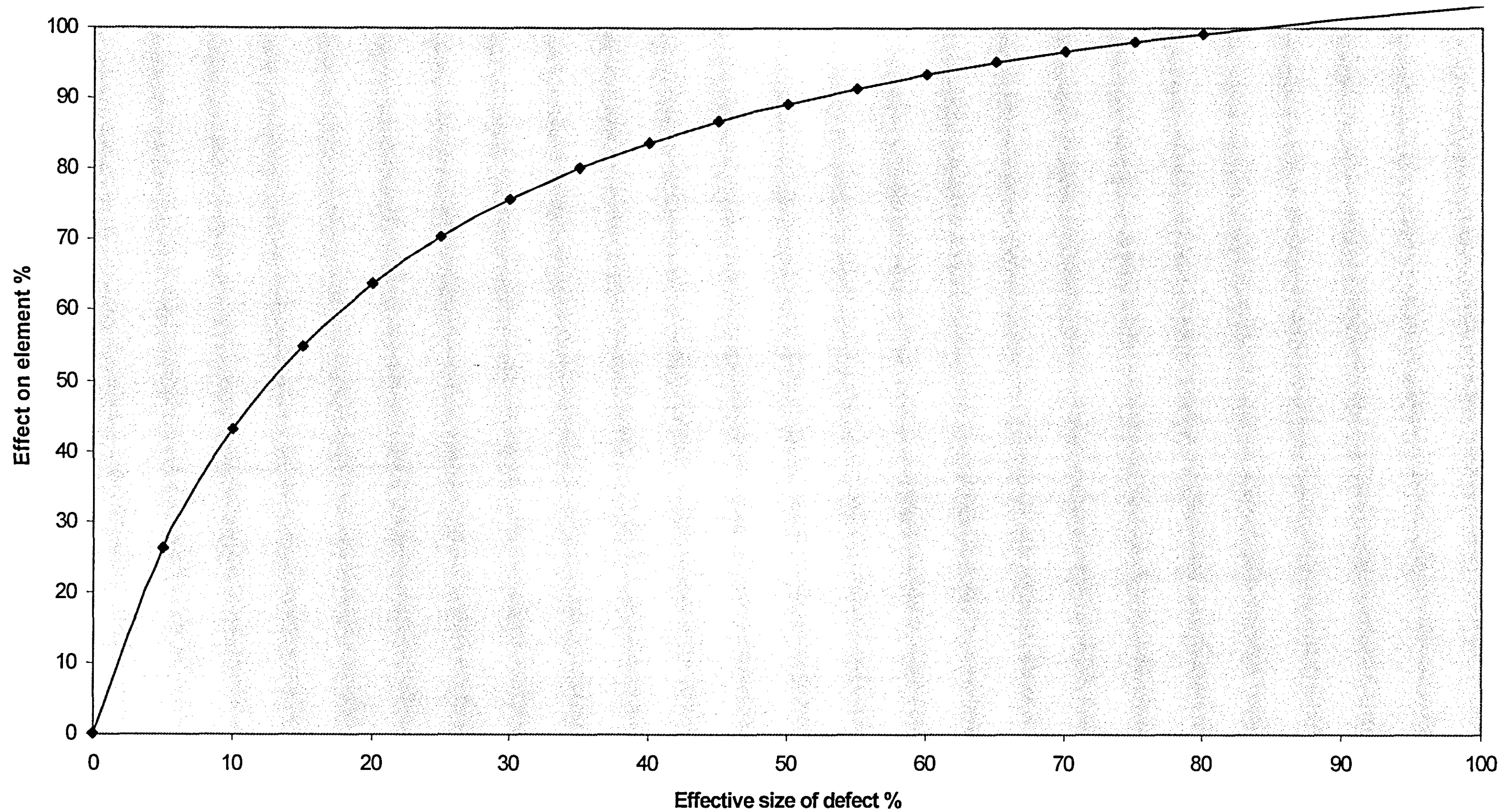
For purposes explained later in this chapter, the influence of each individual defect upon the element severity must be distributed between the two defects based on their effective sizes.

Influence of defect 1 on element condition = (2.832/7.832) * 64.77 = 23.42%

Influence of defect 2 on element condition = (5 / 7.832) * 64.77 = 41.35%

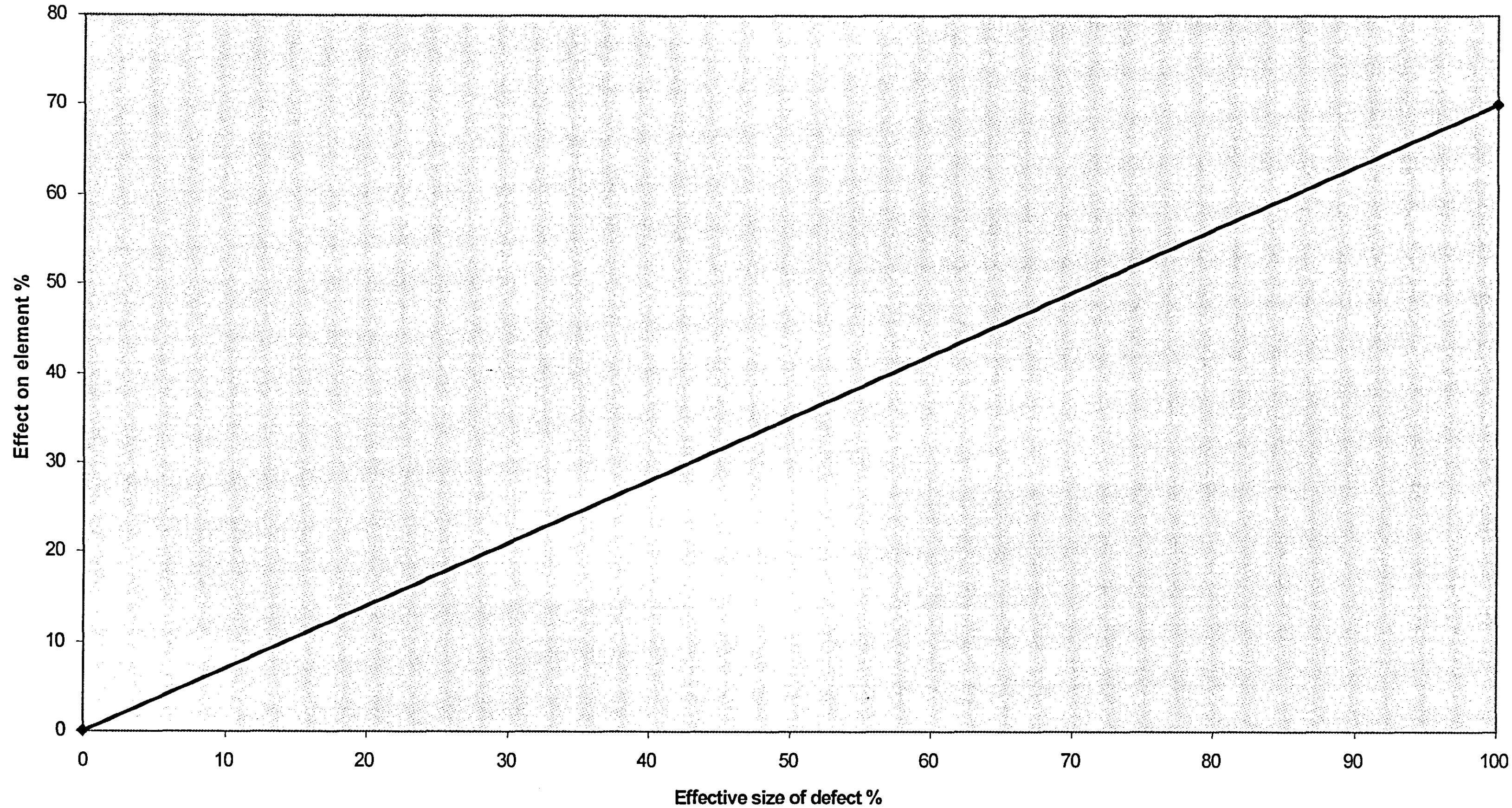
Therefore, it can be said that, of the element condition of 64.77%, 23.43% was caused by defect 1 and 41.35% by defect 2.

So far in this section, it has been explained how the effect of chloride pattern cracking on element condition has been assessed. Using the same technique, graphs to assess the effect of defects on element condition for the other forms of pattern cracking were determined (Figures 5.39 to 5.44).



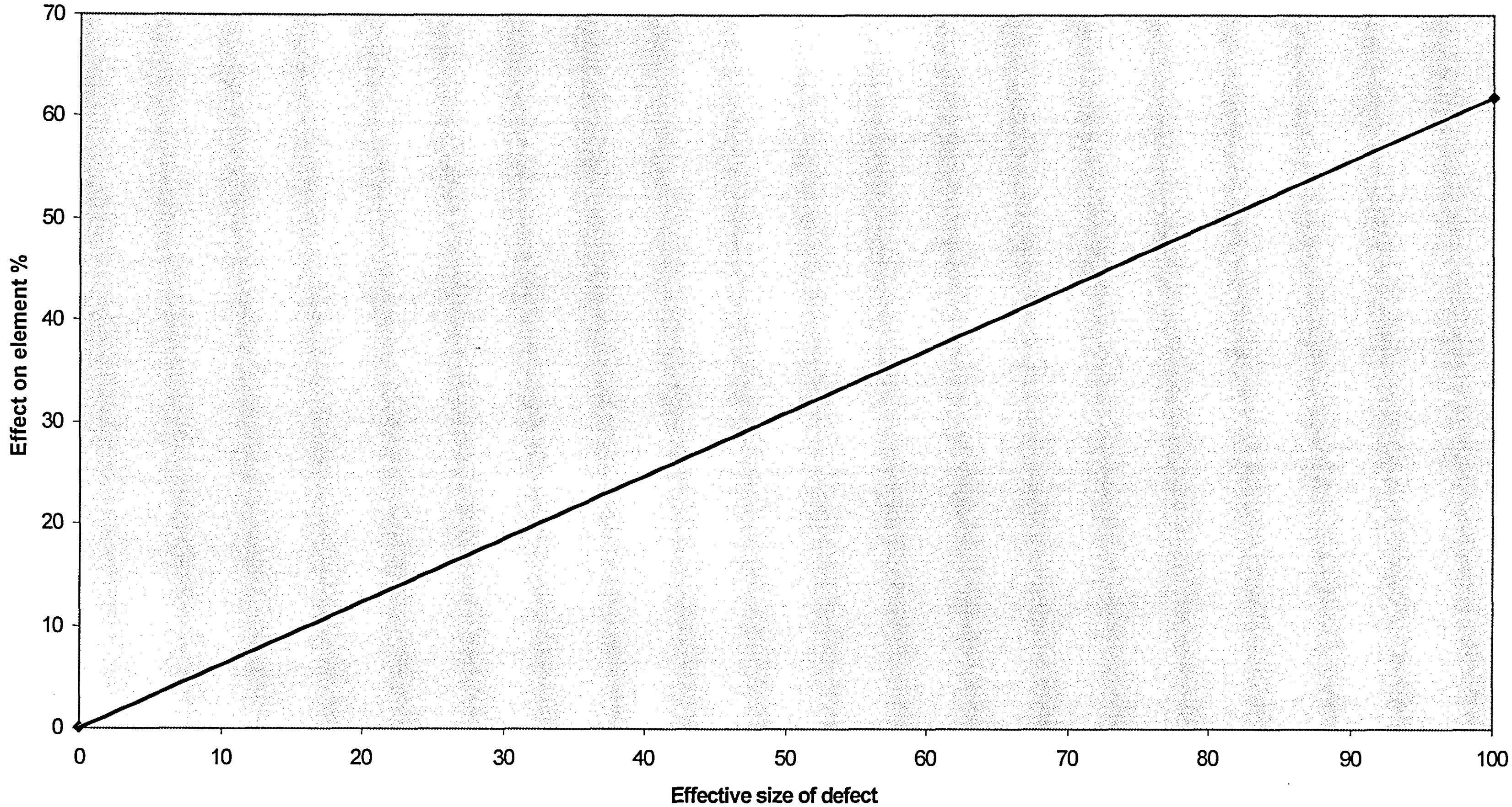

Effect on element condition = Effective size / (0.1506 + 0.0082*Effective size)

**Figure 5.39 Determining the effect of AAR cracking on element condition**



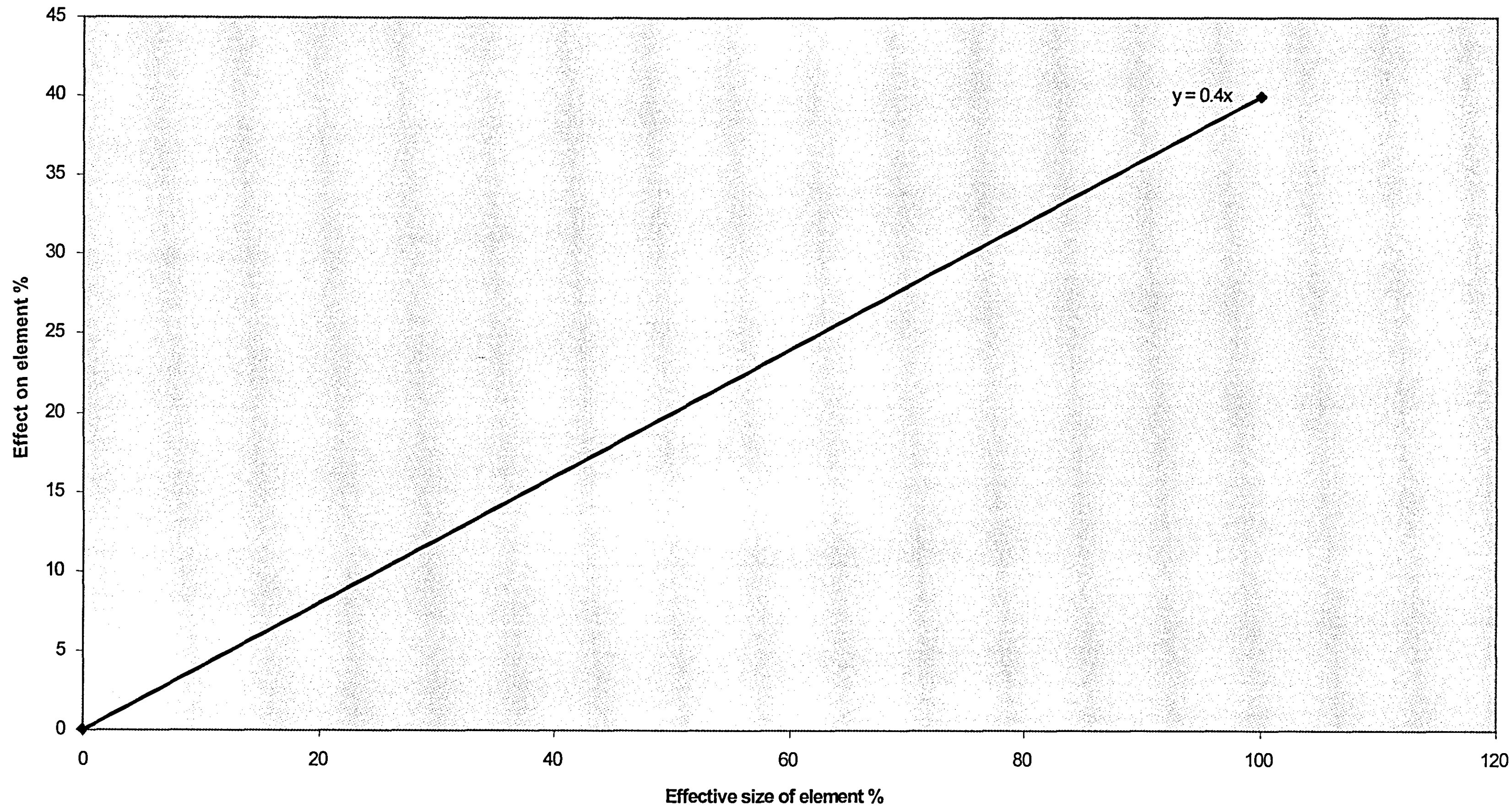

Effect on element condition = Effective size * 0.7

**Figure 5.40 Determining the effect of frost cracking on element condition**



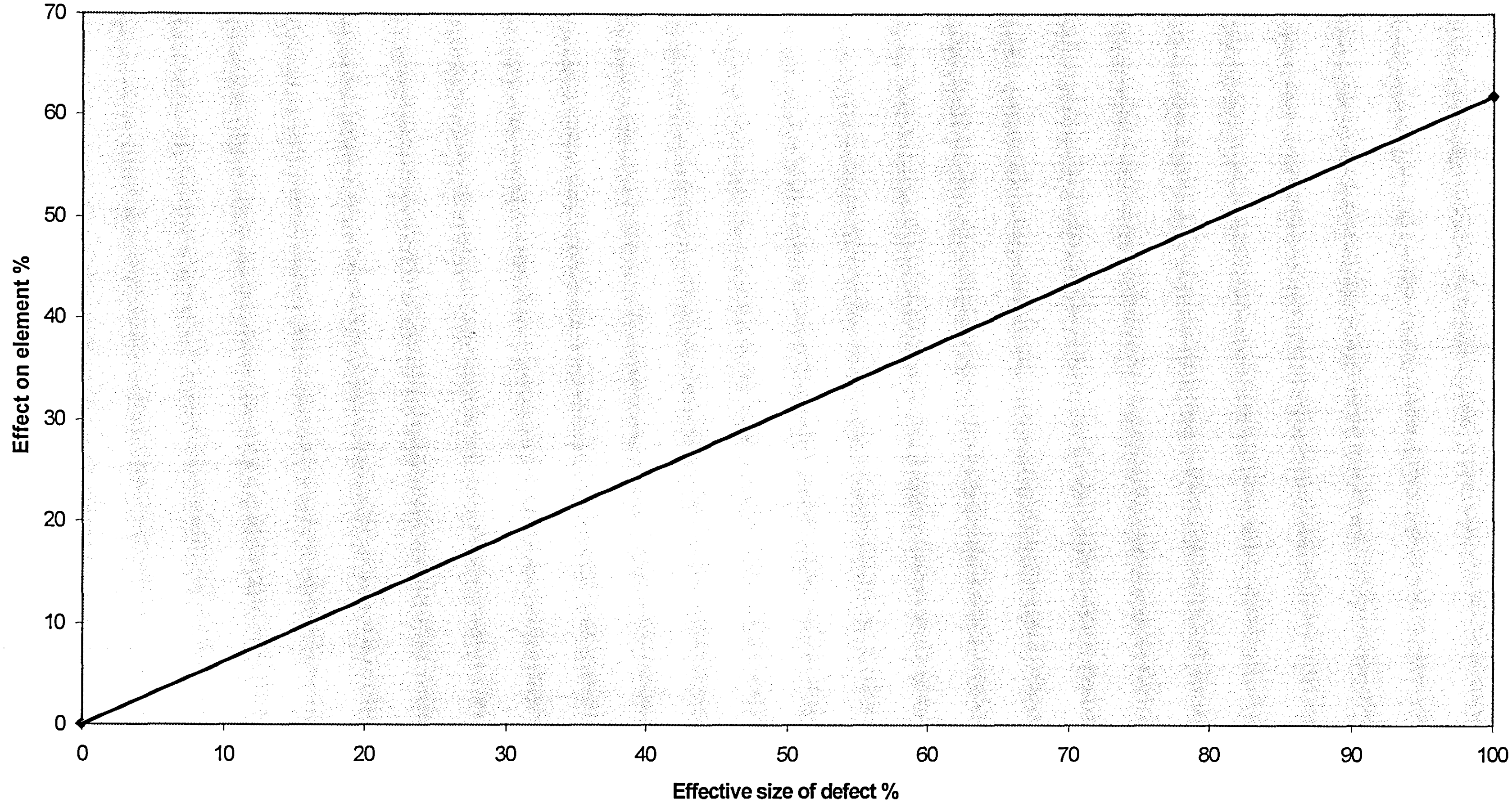

Effect on element =Effective size * 0.62

**Figure 5.41 Determining the effect of plastic shrinkage cracking on element condition**



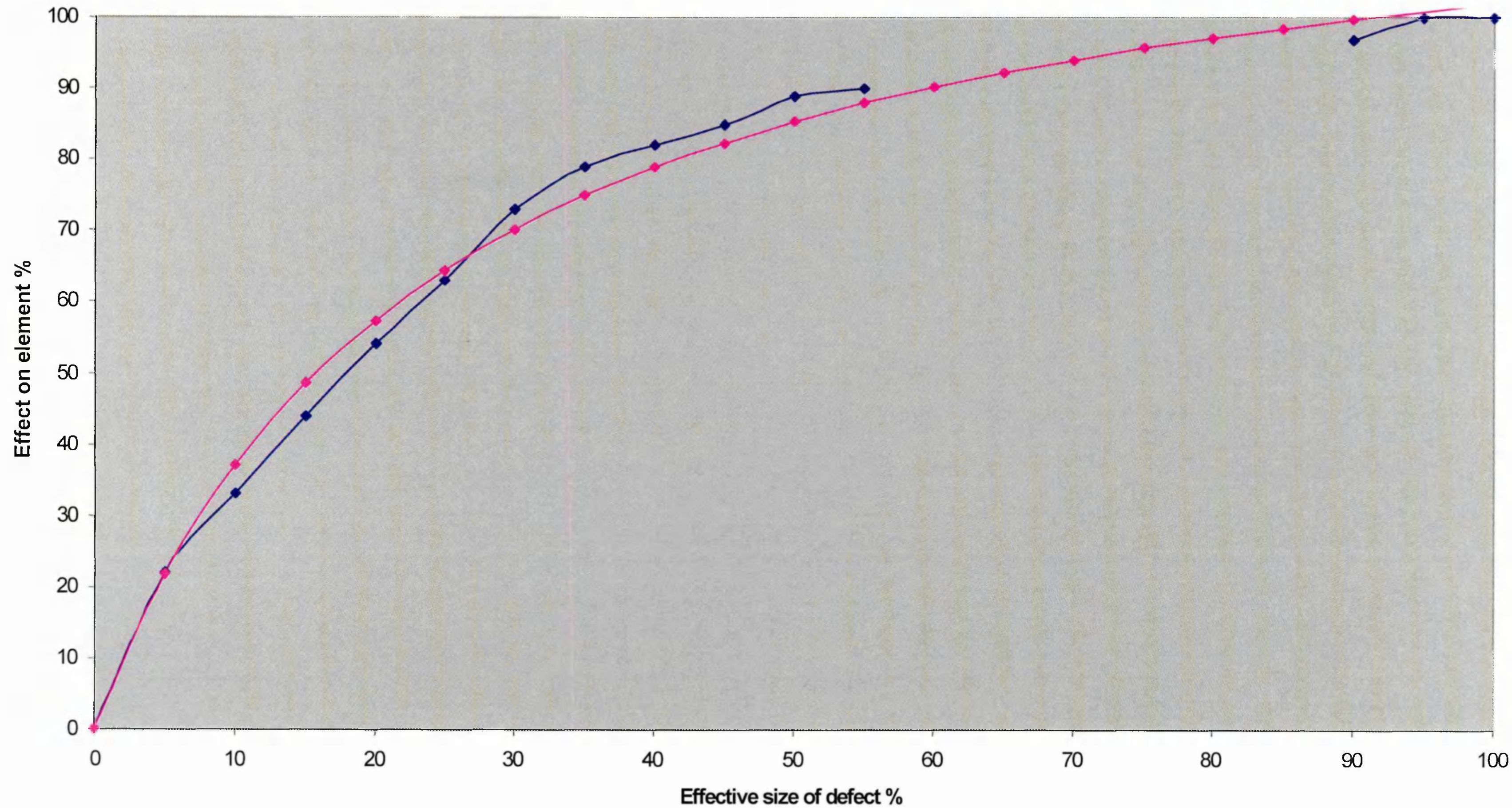

Effect on element = Effective size * 0.4

**Figure 5.42 Determining the effect of crazing on element condition**




Effect on element = Effective size * 0.62

**Figure 5.43 Determining the effect of drying shrinkage cracking on element condition**



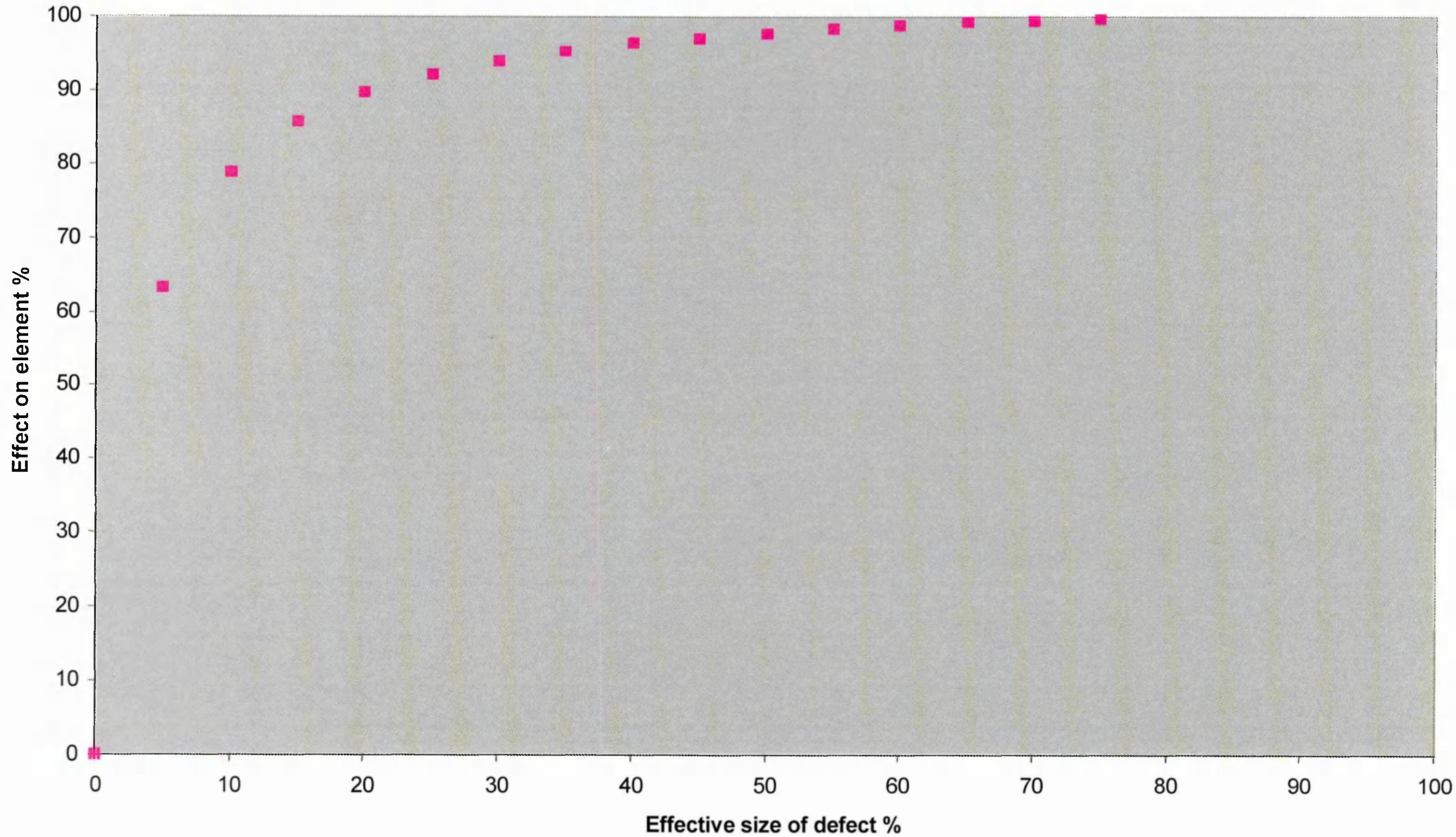

Effect on element = Effective size / (0.1919 + 0.0079*Effective size)

**Figure 5.44 Determining the effect of carbonation cracking on element condition**

### 5.7.2 Effect of spalling defects on element severity

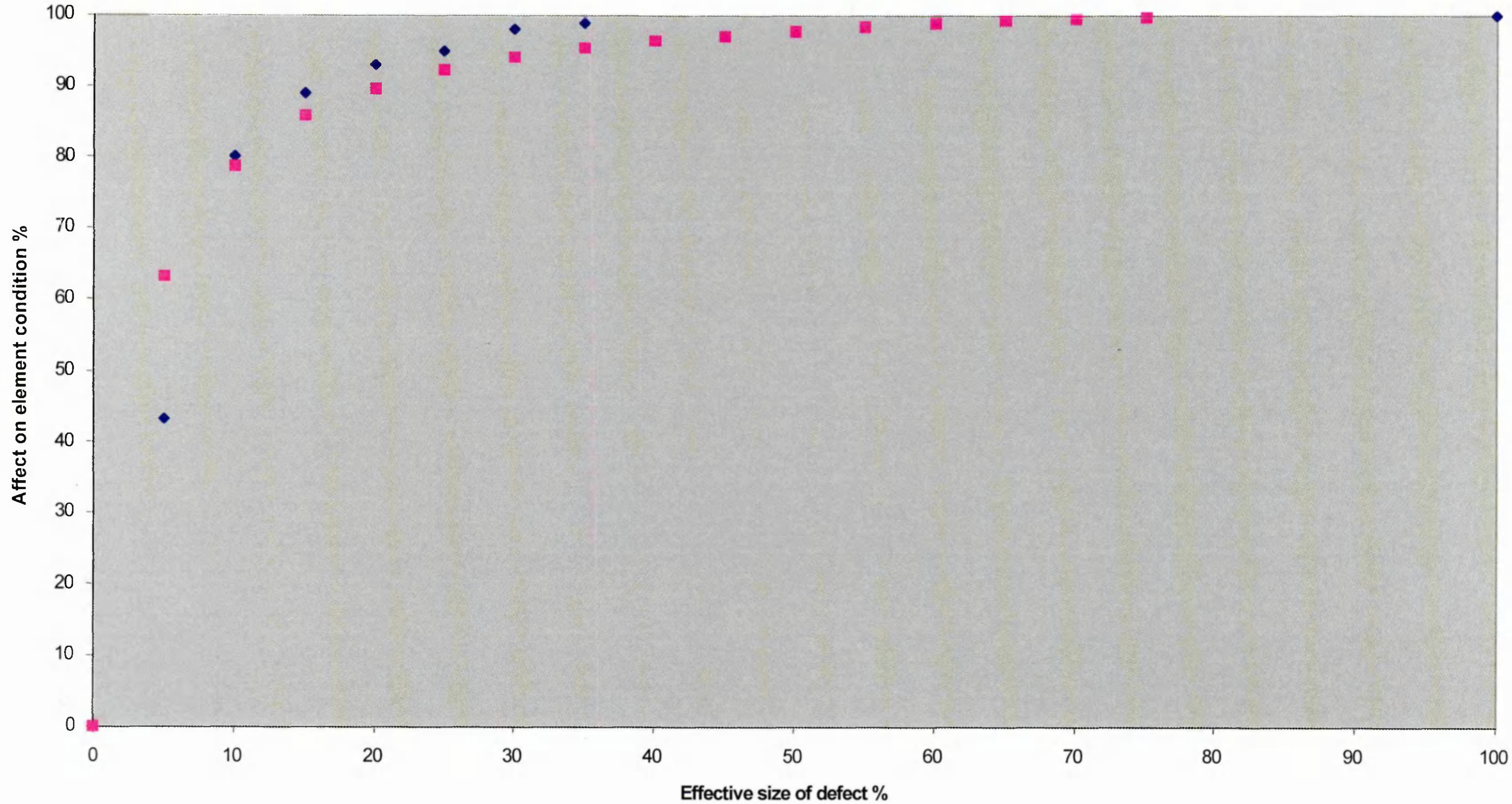
The position of the severity zone apexes for spall defects is governed by the technique outlined in section 5.5.3 – apexes are positioned based on spall depth. However, the position of the apexes can change depending on additional information entered by the user. The expert system remembers the positions of all the zone apexes after the initial data input phase (spall size and depth only) as this represents the position of the zones for an 'average' defect. As with the technique for pattern cracking, the distance a defect falls into the zone it is adjudged to be in, after all the information has been entered, is converted into a figure based on a defect falling into the same zone by the same amount when the zone apexes were in their original positions. Creating an 'effective size' - the equivalent size that an average defect would be to have the same effect on the condition of the element as the defect in question.

Graphs were developed, using the same technique outlined in section 5.7.1, to determine the effect on element condition of spall defects. It should be noted, however, that both spall defects and pattern crack defects can be caused by similar ailments, for example: chloride ingress, the progress of carbonation, and AAR. If one element is affected by both pattern cracking and spalls, and the cause is the same ailment, then for the purposes of determining the condition of the element, the spall graphs (Figure 5.45 to Figure 5.47) will be used. The premise being that if some pattern cracking has already spalled on the element, the existing pattern cracking is likely to spall soon, and should therefore be treated as spalling, which the system considers is more severe than pattern cracking.



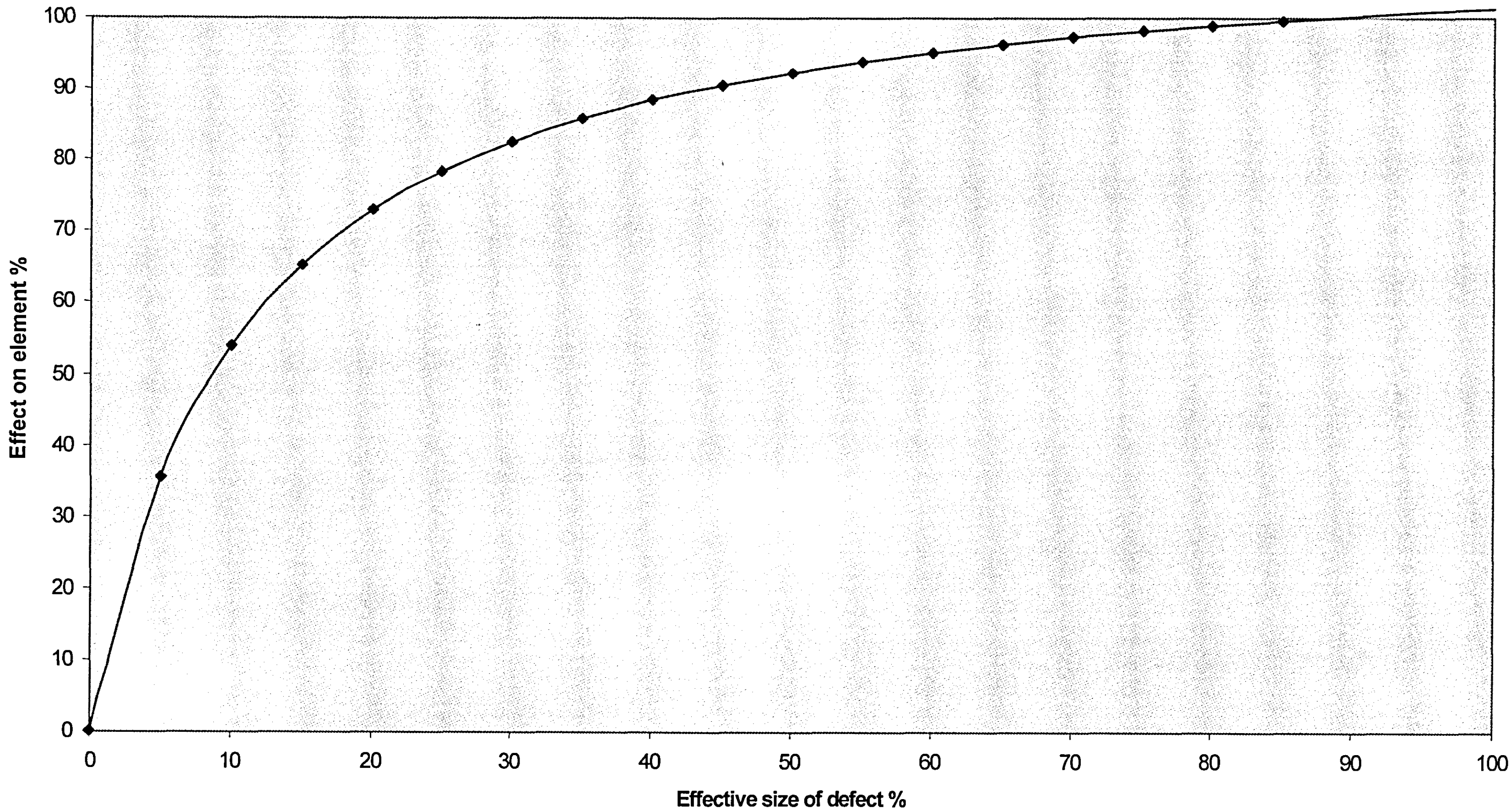

Effect on element = effective size / (0.031 + 0.0096*effective size)

**Figure 5.45 Determining the effect of AAR spalling on element condition**




Effect on element = Effective size / (0.031 + 0.0096*Effective size)

**Figure 5.46 Determining the effect of chloride spalling on element condition**




Effect on element = Effective size / (0.0965 + 0.0089*Effective size)

**Figure 5.47 Determining the effect of carbonation spalling on element condition**

For example, take two defects:

| Defect | Knowledge base decision | Effective size |
|---|---|---|
| Map cracking | Chloride | 11% |
| Spalling | Chloride | 5% |

Because at least one spall is prevalent, Figure 5.46 is used to determine the overall effect on the element.

Effect on element = Effective size / (0.031 + 0.0096*Effective size)

Effect on element = 16 / (0.031 + 0.0096*16)

= 86.7%

In effect, 16% of the entire element is covered by what the experts judged to be an 'average' defect. In reality, 40% of the element could be covered by less than average defects, or 5% by very severe defects. The end result takes all these factors into account and the user is aware that this defect is very severe.

### 5.7.3 Effect of structural cracking on element severity

A 'structural crack' entered by the user is assessed by a knowledge base to determine if the crack is indeed a structural crack, or a crack caused by corrosion. Cracks caused by corrosion will be treated as pattern cracking.

Genuine structural cracks, in the same way as spalls and pattern cracking, have their severity zone apexes set to an initial position based on a minimum of factors – crack size and crack width. The apex positions at that stage represent the position for an 'average structural crack'. The effective length of a structural crack is determined using the same processes used to determine effective size for spalling and pattern cracking as outlined in the previous sections.

Currently, this relationship is used to determine the effect of structural cracking on an element:

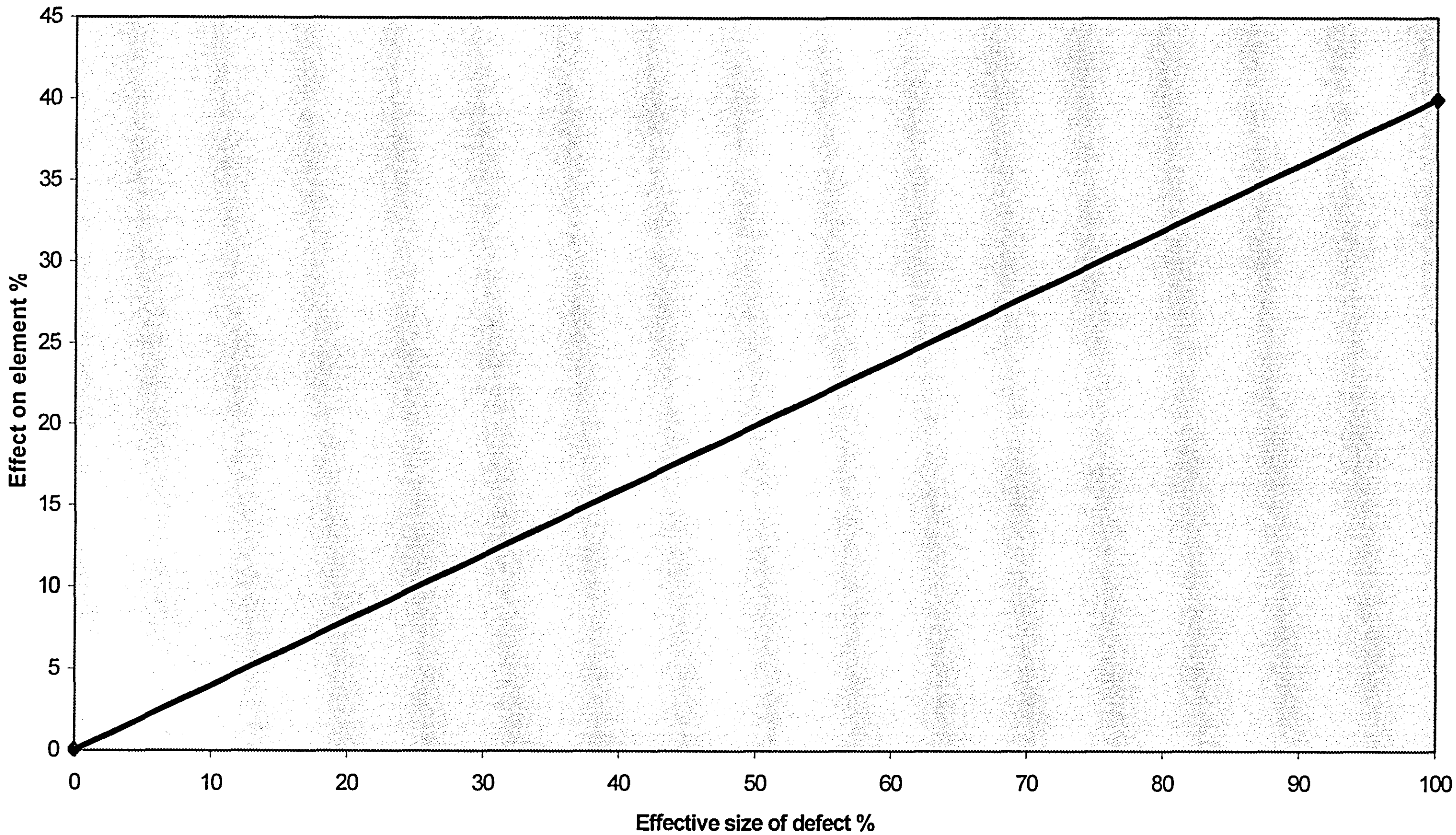
Effect on element = (a * effective length + b)

Where a = 1 and b = 0

This particular relationship will be revised and updated after field trials.

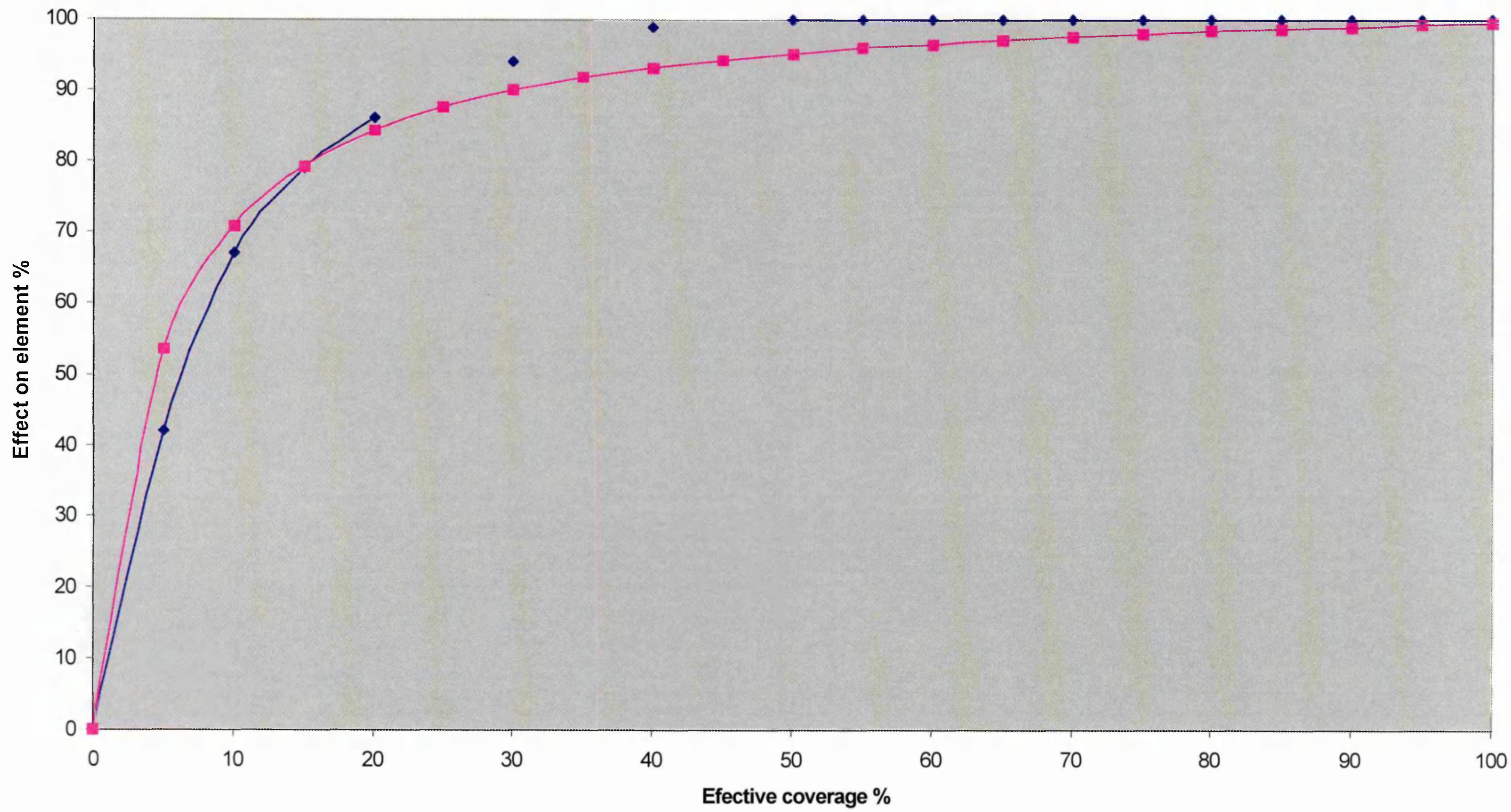
### 5.7.4 Effect of miscellaneous defects on element condition

Miscellaneous defects are sand streaking, blow holes and honeycombing. They are identified by the system user and not by the expert system. In order to determine the effect they have on the severity of an element, their actual inputted size can be used directly with the graphs and equations determined by the expert panel in Figure 5.48 and Figure 5.49.




Effect on element = size * 0.4

**Figure 5.48 Determining the effect of blow-holes and sand-streaking on element condition**




Effect on element = size / (0.0454 + 0.0096*size)

**Figure 5.49 Determining the effect of honeycombing on element condition**

## 5.7.5 The Element Graph

In addition to rating the severity of individual defects, it is an important requirement of any intelligent expert system in this field to be able to assess the overall condition of an element affected by multiple defects. Figure 5.50 shows how, when an element is selected, an element condition indicator is shown at the bottom of the screen.

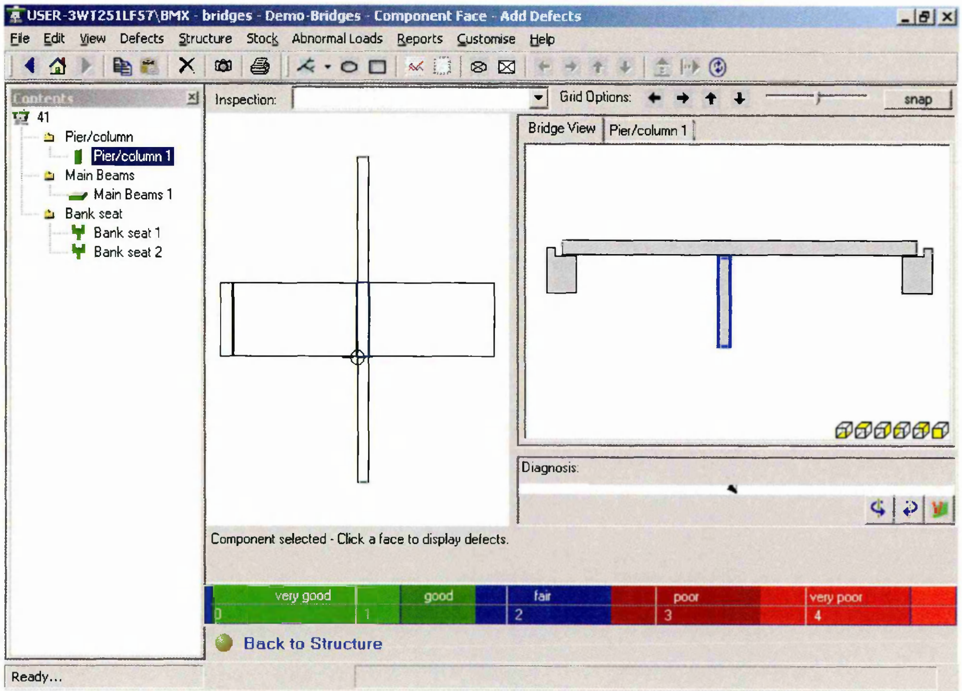



**Figure 5.50 Element condition indicator**

The marker on the Element Scale is a band, and the width of the band represents the confidence the expert system has in its prediction of the element condition. Hence, the width of the band represents 'element uncertainty'. The width of the band changes as more defects are added; the change is dependent on the severity of the defect being added, its uncertainty and the current element uncertainty.

Element uncertainty (the width of the element band) is calculated with the following formula:

$$E_{un} = [h_1 * D_{u1}] + [h_2 * D_{u2}] + [h_x * D_{ux}]......$$

Where $E_{un}$ = Element uncertainty

$D_{ux}$ = Defect uncertainty (see section 5.6)

$h_x$ = $R / (E_a)$

Where $E_a$ = Element condition after addition of defect

$R$ = Effect of current defect on element condition*

*Say three chloride caused spalls of identical severity and size are input into the expert system. The first spall may cause the element condition to increase to 20. However, the second spall would probably only increase the element condition to 30, and the third spall to 36. This is in accordance with the technique developed in section 5.5. However, each spall individually is responsible for one third of the final element condition of 36. Therefore, as the third spall is submitted, the variable R would be 12 although the actual increase in element condition could be much smaller.

For example, say a user adds a spall to a pier. The spall sets the element condition to 22 due to its effective size of 7%, and has an uncertainty of 30%.

$h$ = $22 / (22)$

= 100%

$E_{un}$ = $[1 * 30]$

= 30%

Say a second defect is added, with an effective size of 4% and an uncertainty of 40%. Assuming the causes of both defects are the same. The effective size of both defects combined is therefore 11% and this sets the element condition to, perhaps, 30. By pro-rata of effective sizes, the first defect is now responsible for:

$$7/(7+4) * 30 = 19$$

and the second defect for

$$4/(7+4) * 30 = 11$$

Reworking the first defect:

$$\begin{aligned} h &= 19 / 30 \\ &= 63.3\% \\ E_{un} &= [0.633 * 30] \\ &= 18.99\% \end{aligned}$$

and including the second defect

$$\begin{aligned} h &= 11 / 30 \\ &= 36.7\% \\ E_{un} &= [0.367 * 40] + [\,(1 - 0.367) * 18.99] \\ &= 14.68\% \end{aligned}$$

Total element uncertainty = 18.99 + 14.98 = 33.67%

Each time a new defect is added to the system and diagnosed, a new series of calculations for each defect is automatically conducted.

Currently, the width of the element marker is equivalent to one tenth of the element uncertainty, and is shown on a scale from 0 to 100. For example, if the element uncertainty is 33.67%, then the width of the element condition marker is 3.36, and the centre of the marker indicates the current element condition.

## *5.8 Testing*

Referring to the expert system framework in section 5.4.3: at the testing stage the cause of each defect has been diagnosed, and the severity of the defect has been assessed. In addition, the element itself has been given a condition rating based on the defects affecting it. Therefore, at this stage the testing knowledge base is ready to work. This knowledge will recommend the types of testing which should be conducted to confirm the findings of the expert system so far. Firstly, the information gathered by the program needs to be collated in the form which the knowledge base requires it. Structural cracking does not require chemical testing, the only testing the structural crack knowledge bases may recommend is monitoring the crack. This usually involves measuring if the crack is opening, closing, or static.

As a result of the output of the knowledge base to diagnose defects, some defects can be diagnosed as having multiple causes. For example, a spall in an old bridge with low cover in the central reservation of a motorway with a considerable degree of corrosion will be reported (by the knowledge base) as having a high probability of the cause being carbonation and a high probability that the cause is chloride ingress.

An operation called 'probable cause distribution' is undertaken. For example, say DEFECT 1 is diagnosed as 'high chloride' (a high chance it was caused by the ingress of chlorides) and 'medium carbonation'.

Each natural language assessment by the knowledge base has a value. High = 1, Medium = 0.75, LOW = 0.40.

Therefore, for DEFECT 1:

Chloride = 1

Carbonation = 0.75

The likely contribution of each cause is converted into a percentage

Likelihood that chloride caused defect = A /(C)

Where A = Numeric value of natural language qualifier appended to cause

C = total of all natural language qualifiers for all possible causes of defect

For DEFECT 1

Chloride = 1 / (1 + 0.75) = 57.1%

Carbonation = 0.75 /( 1 + 0.75) = 42.9%

This procedure is repeated for each pattern cracking or spall defect affecting an element. For example, assume an element (Column 3) is affected by 6 defects, a combination of spalls and pattern cracking. The process shown above is conducted for each defect, and a table is constructed as shown in Table 5.17.

**Table 5.17 Example defects on 'Column 3'**

| Defect | Carbonation | Chlorides | AAR | Early AAR |
|---|---|---|---|---|
| 1 | 71 | 29 | | |
| 2 | | 100 | | |
| 3 | | | 100 | |
| 4 | | 71 | | 29 |
| 5 | | 100 | | |
| 6 | | 100 | | |

Say the element condition rating due to all these defects, following the procedures adopted in section 5.7, is 30%.

The contribution of each individual defect to that 30% condition rating is determined by the expert system in accordance with the method outlined in section 5.7.1. This process is shown, for the given example, in Table 5.18.

Let EC = element condition

**Table 5.18 Contribution of each defect to element condition**

| Defect | Contribution to EC | Contribution to EC % |
|---|---|---|
| 1 | 10 | = 10 / 30 = 33.3 |
| 2 | 5 | 16.6 |
| 3 | 2 | 6.7 |
| 4 | 2 | 6.7 |
| 5 | 6 | 20 |
| 6 | 5 | 16.6 |
| Total | 30 | 100 |

Now, considering DEFECT 1, it has contributed 33% to the element condition. In accordance with Table 5.17 the defect has a 71% probability of being caused by carbonation and a 29% probability of being caused by chloride ingress.

By multiplying these values by the 33% contribution of defect 1 to the element condition, the figures the knowledge base needs are determined - the contribution of each cause to the element condition. These are shown, for the given example, in Table 5.19.

**Table 5.19 Determining primary causes of element deterioration**

| Defect | Carbonation | Chlorides | AAR | Early AAR |
|---|---|---|---|---|
| 1 | 71% x 33% = 23.4% | 29% x 33% = 9.6% | | |
| 2 | | 100% x 16.6% = 16.6% | | |
| 3 | | | 6.7% | |
| 4 | | 71% x 6.7% = 4.8% | | 1.9% |
| 5 | | 20% | | |
| 6 | | 16.6% | | |
| Total | **23.4%** | **68%** | **6.7%** | **1.9%** |

A full version of Table 5.19 would contain columns for all the possible defect causes. It can be seen that the primary ailment affecting the element is chloride corrosion, carbonation is also a factor. It is likely that the small defects suspected of being caused by AAR are also caused by chlorides, although the expert system will make an assessment on this.

At this stage the testing Knowledge Base can be provided with the information it needs i.e.:

Element

Total area affected by carbonation as %

Total area affected by chlorides as %

Total area affected by AAR (including early aar) as %

Early AAR as %

Plastic shrinkage as %

Impact as %

Freeze thaw as %

Drying shrinkage as %

Crazing as %

Internally, the expert system uses databases to store the data generated by the user input and the output of diagnosis knowledge bases. Tables such as those shown in Table 5.20 and Table 5.21.

**Table 5.20 Typical storing of knowledge base diagnosis for pattern cracking**

| Defect | aar | carbonation | chloride | crazing | Drying shrinkage | Early aar | Freeze thaw | Plastic shrinkage |
|---|---|---|---|---|---|---|---|---|
| 1 | high | | medium | | | | | |
| 2 | | high | | | | | | |
| 3 | | medium | | | | | medium | |

**Table 5.21 Typical storing of knowledge base diagnosis for spalling**

| Defect | Aar | Carbonation | Chloride | Impact | Prev Rep | popout | Spacing block | Filled Pocket |
|---|---|---|---|---|---|---|---|---|
| 1 | | high | | | | | | |
| 2 | | high | medium | | | | | |

When the figures in Table 5.19, and the additional information, can be provided to the testing knowledge base - it uses the principles of objects, ranges and rules to examine the evidence and deliver a conclusion. Importantly, the knowledge base has rules to recognise the situation where there may be two distinct and separate ailments affecting an element – for instance AAR and chloride corrosion.

## 5.9 Repair advice

Referring to the expert system framework in section 5.4.3, the expert system and the system user now have the information required to recommend repair advice. When asked for repair advice for a particular defect, the user is reminded of the probable causes the diagnosis knowledge base recommended. If more than one cause was recommended, the user is invited to select the cause of the defect in accordance with the results of the testing. In the absence of testing the user is recommended to select the defect with the highest natural language operator, i.e. high probability of chloride caused to be selected before medium possibility of carbonation. However, the repair knowledge base will accept a cause of both carbonation and chloride ingress at this stage.

### 5.9.1 Advice for spalls and pattern cracking

Once the user confirms the cause of the defect, an object in the repair knowledge base called 'defect' is set. The range of this object is:

Chloridecarbonation

Carbonation

Chloride

Crazing

EarlyAAR

ActiveAAR

InactiveAAR

Blowhole

Drying Shrinkage

Freezethaw

Honeycoming

Sandstreaking

Plastic shrinkage

Other

Therefore, the 'defect' object has to be set to one of these variables. The laboratory report following AAR testing will identify the status of that particular defect. The defect 'Other' encompasses small defects such as filled pockets which have spalled away, tie wires, and popouts.

The repair knowledge base has two input objects, i.e. two objects whose values are set by the expert system. These are 'defect' and 'severity', with 'defect' being the cause as discussed above, and 'severity' being the severity zone into which the defect fell.

Rules have been constructed, by the experts, which relate combinations of causes and severities to twenty-nine separate pieces of repair advice.

The example in Figure 5.51 shows a premise rule in the repair knowledge base that will set the output object 'drying shrinkage 2' to yes. Only one of the twenty-nine output objects, for each defect, can be set to yes – there can be no conflict. When the expert system detects that the output object set to 'yes' was drying shrinkage 2, it will search for the piece of advice which corresponds to that defect.

The example shows that if the defect is drying shrinkage, and if the severity of the defect is Minor or Cosmetic, the object will be set to yes. Table 5.22 shows all the twenty-nine pieces of repair advice. The expert system, in the case of this example, would return the text under 'drying shrinkage 2'.

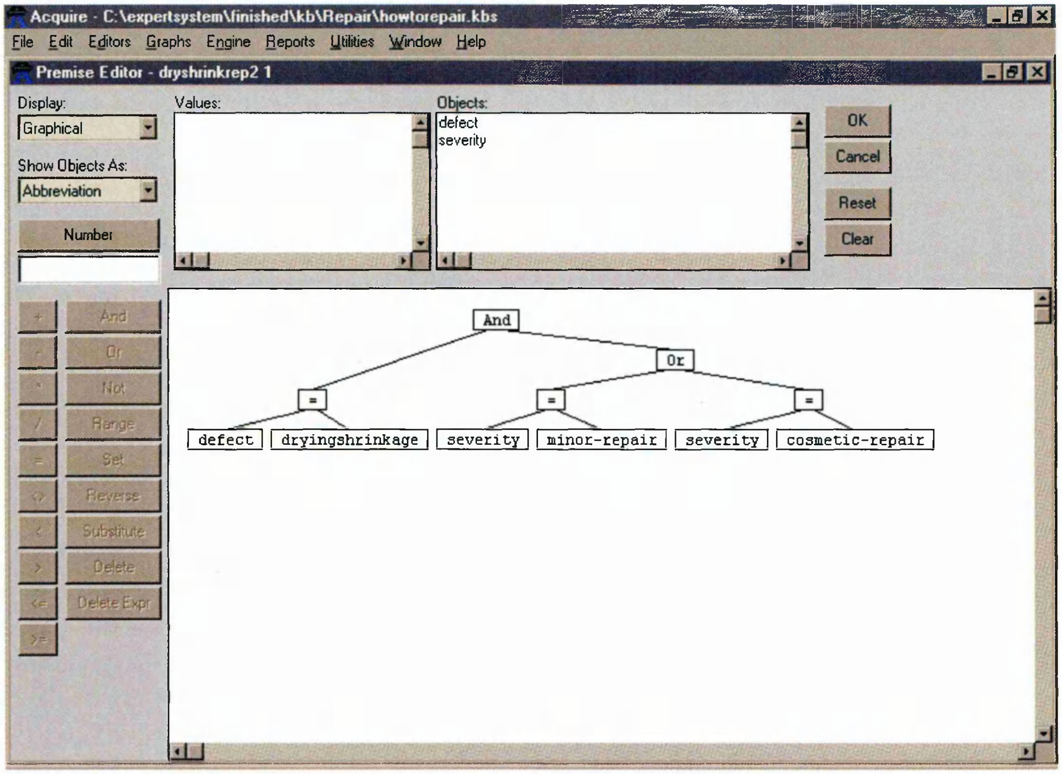



**Figure 5.51 Drying shrinkage repair rule**

**Table 5.22 Repair advice**

| Knowledge base output | Advice |
|---|---|
| Chloride carbonation 1 | This defect is caused by both carbonation and chloride ingress.<br>It's not serious enough to warrant break out and repair.<br>Repair with cementituous mortar. |
| Chloride carbonation 2 | This defect is caused by both carbonation and chloride ingress.<br>Break out and repair. |
| Crazing 1 | This minor defect is CRAZING.<br>No further action is necessary.<br>This defect is not serious. |
| Crazing 2 | This defect is CRAZING.<br>It is not essential that the defect is repaired.<br>If it is aesthetically unacceptable, the defect could be filled with a cementituous mortar. |
| AAR 1 | This defect could be caused by ALKALI AGGREGATE REACTION.<br><br>No immediate action should be taken. Monitor this defect. |

| Knowledge base output | Advice |
|---|---|
| | Note: This type of cracking is visually similar to FREEZE/THAW DAMAGE.<br>AAR cracking is deep, FREEZE/THAW cracking is shallow. Tapping with a hammer will confirm the diagnosis.<br>Additionally, the presence of a white gel in the cracks, or evidence of a white gel (which may have been washed away), would confirm AAR as the cause. |
| AAR 2 | This defect is inactive ALKALI AGGREGATE REACTION.<br>The reaction between cement and aggregate appears to have discontinued.<br>Undertake a structural survey of all elements effected.<br>If elements are structurally acceptable: leave or coat for aesthetic reasons. |
| AAR 3 | This defect has been confirmed as active ALKALI AGGREGATE REACTION<br>Seal the surface cracks<br>Apply water repellent impregnations (silane monomer)<br>Monitor closely. |
| AAR4 | This defect is confirmed as inactive ALKALI AGGREGATE REACTION.<br>Undertake Epoxy resin injection to restore integrity.<br>Coat the surface for aesthetic purposes. |
| Chlorides 1 | This defect is caused by Chloride Ingress.<br>It is not serious enough to warrant break out and repair.<br>Repair with cementituous mortar. |
| Chlorides 2 | This defect is caused by Chloride Ingress.<br>Break out and repair. |
| Plastic shrinkage 1 | This defect is PLASTIC SHRINKAGE.<br>No repair is necessary.<br>Leave. |
| Plastic shrinkage 2 | This defect is PLASTIC SHRINKAGE.<br>Fill cracks with resin (possibly by making a resevoir) and fill. |
| Plastic shrinkage 3 | This defect is PLASTIC SHRINKAGE.<br>Cut out and repair. |
| Freeze 1 | This defect is FREEZE/THAW DAMAGE.<br>No action is necessary at this stage. |
| Freeze 2 | This defect is FREEZE/THAW DAMAGE.<br>Remove loose material, fill cracks with cementituos mortar |
| Freeze 3 | This defect is FREEZE/THAW DAMAGE.<br>Break out and repair. |
| Honeycombing 1 | This defect is HONEYCOMBING.<br>No repair is necessary. |

| Knowledge base output | Advice |
|---|---|
| Honeycombing 2 | This defect is HONEYCOMBING.<br>Break back to sound concrete and reinstate. |
| Blow-holes 1 | This minor defect is BLOWHOLES<br>Leave this defect. It is not serious. |
| Blow-holes 2 | This defect is BLOW HOLES.<br>Fill these holes with a cementituous mortar.<br>If necessary, coat for aesthetic reasons. |
| Carbonation 1 | This defect is caused by Carbonation.<br>Repair using cementituous mortar. |
| Carbonation 2 | This defect is caused by carbonation.<br>Break out effected concrete and repair.<br>The system will recommend the correct repair material properties. |
| Sand-streaking 1 | This defect is SAND STREAKING.<br>No further action is necessary. |
| Sand-streaking 2 | This defect is SAND STREAKING.<br>Remove the defect and reinstate. |
| Drying shrinkage 1 | This defect is DRYING SHRINKAGE.<br>It is not of sufficient severity to warrant any action.<br>Leave. |
| Drying shrinkage 2 | This defect is DRYING SHRINKAGE.<br>Inject the cracks with epoxy resin.<br>Consider coating for aesthetic reasons if necessary. |
| Drying shrinkage 3 | This defect is DRYING SHRINKAGE<br>Open out cracking with router, re-repair with cementituous mortar.<br>Consider coating for aesthetic reasons if necessary. |
| Other 1 | This small defect can be left, or filled with cementituous mortar. |
| Other 2 | Fill with cementituous mortar. |

Defects whose repair advice recommends break out and repair of the substrate concrete can use the expert system to automatically recommend repair material properties based on the techniques developed in Chapter 4 of this thesis.

For large scale defects, and seriously debilitated concrete, an engineer will be required to make an economic assessment of the relative merits of breaking out and replacing defects, and electrochemical remediation techniques.

### 5.9.2 Advice for structural cracking

As discussed in section 5.4.3.3, there are two knowledge bases for structural cracking. The job of the first knowledge base is to ascertain if corrosion is the cause of the crack – if it is, the pattern cracking knowledge base takes over. The other task of the first knowledge base is to recommend action.

A typical piece of action recommended is monitoring the crack. Through monitoring the user checks to see if the crack is:

- Active widening
- Active opening and closing
- Dormant
- Closing

Certain rules will cause the recommendation of the first knowledge base to be 'Leave – no action required'. However, the recommendation can be to monitor the crack *and* check for corrosion. In this case, the system has been unable to decide if the crack is caused by corrosion or structural effects.

For minor cracks, the first knowledge base may recommend repair by rout and seal with no need for further tests or monitoring.

After the monitoring stage, a second structural cracking knowledge base is utilised. The second structural cracking knowledge base requires certain pieces of information – some supplied by the engineer and some by the expert system:

- Width of the crack

- Results of the monitoring
- Moisture condition of crack
- Is strengthening required

Importantly, it is the responsibility of the engineer to find and eliminate the cause of the structural cracking. However, if a crack is actively widening, or actively opening and closing, the knowledge base may recommend redesigning an expansion joint at the crack location.

There are eleven separate pieces of repair advice that can be generated by the second knowledge base for structural cracking. All viable repair options for a particular crack will be presented to the user. These pieces of repair advice are shown below.

<u>External stressing</u>

Consider external stressing for the repair of this crack.

It is recommended for moving and opening fine cracks.

Most useful on long members: beam, deck, parapet.

<u>Stitching (dogs)</u>

For use to re-establish tensile strength across cracks. Drill holes either side of crack and resin fix 'staples' made of reinforcing steel.

<u>Rout and seal</u>

Consider for static and moving 1mm cracks. Enlarge the crack, fill and seal with suitable joint sealant.

Redesign and provide expansion joint

Active cracks where strengthening may be required.

Active cracks can be routed out and filled with flexible sealant. Narrow cracks may be sealed with a flexible face seal.

Grouting

For dormant cracks up to several millimetres in width.

Extensible overlay

Use for moving cracks on flat horizontal surfaces.

Bonding

Bonding with Epoxy (crack injection) / cement mortar / microfine cements / resin

For static, Fine cracks (sub 1mm). Cracks as narrow as 0.05mm can be bonded using epoxy injection. Only apply to static cracks (or remove the cause of crack movement/growth)

Blanketing

Active or dormant cracks not requiring strengthening.

Autogeneous healing.

This natural crack repair process can occur in the presence of moisture and in the absence of any tensile stresses.

It could be practically applied for example, to close a dormant, thin crack, in a situation where moisture was present.

However, if the amount of water passing through the crack is large, this will wash away the lime deposits which would otherwise heal the crack.

Ideally the crack will heal in the presence of stationary moisture, either from natural sources or contrived.

Ordinary overlay

Used to treat static cracks on flat horizontal surfaces.

Often using a heavy coat of epoxy resin or an overlay of polymer modified cement.

# 6 Review of the expert system for reinforced concrete bridge repair

*Chapter Objective*

- To present and review the software that incorporates the techniques and routines developed in this thesis

## 6.1 Introduction

During development of the research, it became necessary for the software to be able to understand the dimensions of certain bridge elements in order to make decisions. It became apparent that the software being developed could be integrated into a 'Bridge Management System' which would not only function as an expert system for concrete repair, but also as a software inventory for storing bridge stock information. The software engineering company assisting in this research have taken prototype software developed by the author for the expert system and material property specification systems, and re-coded this software to produce software cosmetically acceptable in a commercial market and using more sophisticated database and software language techniques than those available to the author during prototyping. This chapter generally shows screen-grabs from the developed commercial software (www.bridgemanagementexpert.com).

Although a bridge management system is a commercially viable commodity, an expert system for concrete repair would be an untested commercial product. The collaborating software developers were, therefore, keen to maximise the saleability of the end product of this research through the provision of a bridge management system in addition to an expert system and repair material property specification program. Therefore, the ability to store detailed structure information has been added to the overall software by the collaborating software organisation. This database system works seamlessly with the expert system capabilities of the software. Fortuitously, many features developed as a result of this research, such as the need for the three dimensional representation of concrete elements, work in harmony with the bridge management database developed by the collaborating software engineers.

## 6.2 Structures Management

The bridge management system is a database in which details of an organisation's bridge stock can be recorded and managed. The bridge management system into which the expert system for reinforced concrete repair is embedded can store and manage data for thousands of structures. At its simplest level the program can store the name of a structure, its location, the features crossed by the bridge and other such important but basic information. Exploiting the full functionality of the bridge management system will allow the user to store the shape, sizes, materials and condition of all the elements of the bridge as well as photographs, reports and very detailed data. Figure 6.1 and Figure 6.2 demonstrate the bridge management system.

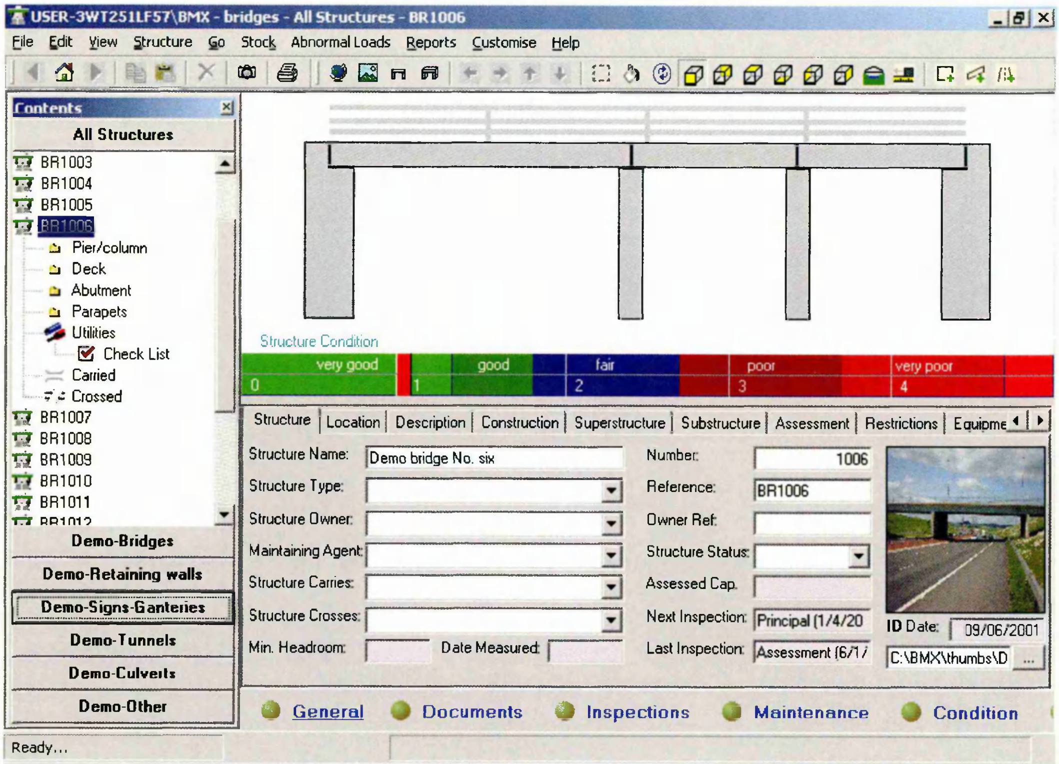



**Figure 6.1 Structure management**

Figure 6.1 shows a typical database screen. In the left window is a list of structures that have been entered into the database. When highlighting a structure in the left window, its diagrammatic representation appears in the right window, along with a digital photograph of the structure, and access to all the data about this bridge which the user may have entered in the system. The bridge management system – the database which stores information such as the structure name, its location and the time of the next scheduled inspection, is the work of the collaborating software engineers. However, information generated by the expert system – such as condition ratings of the concrete elements, is obviously also stored in the database.

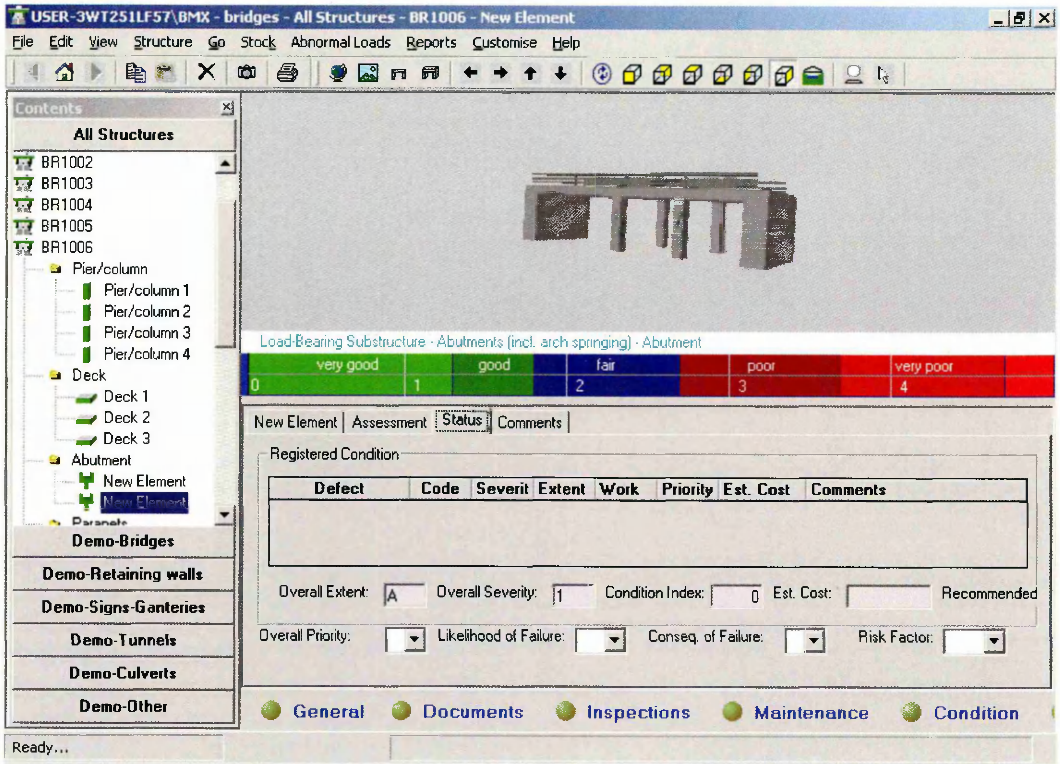



**Figure 6.2 Alternative views of structures**

Figure 6.2 shows an alternative view of the structure created by the user. The bridge management system has a broad range of functionality:

- Prioritising bridge maintenance
- Storing photographs
- Planning inspections
- Record keeping
- Abnormal load route planning

## 6.3 Element and Structure creation

In order to allow the expert system to make judgements concerning the severity of defects affecting elements, it is necessary for the program to have information about the size and shape of individual bridge elements. For example, the importance of a spall of $1m^2$ is dependent on the overall size of the element it affects. In a slender leaf pier such a spall could be extremely important, whereas on a wide and tall abutment its seriousness would be far less. These kind of decisions can only be made by an expert system when it has information which will allow it to compare the relative sizes of the element and defect.

Figure 6.3 shows the 'structure creation wizard' that allows the user to quickly insert a typical reinforced concrete structure into the database.

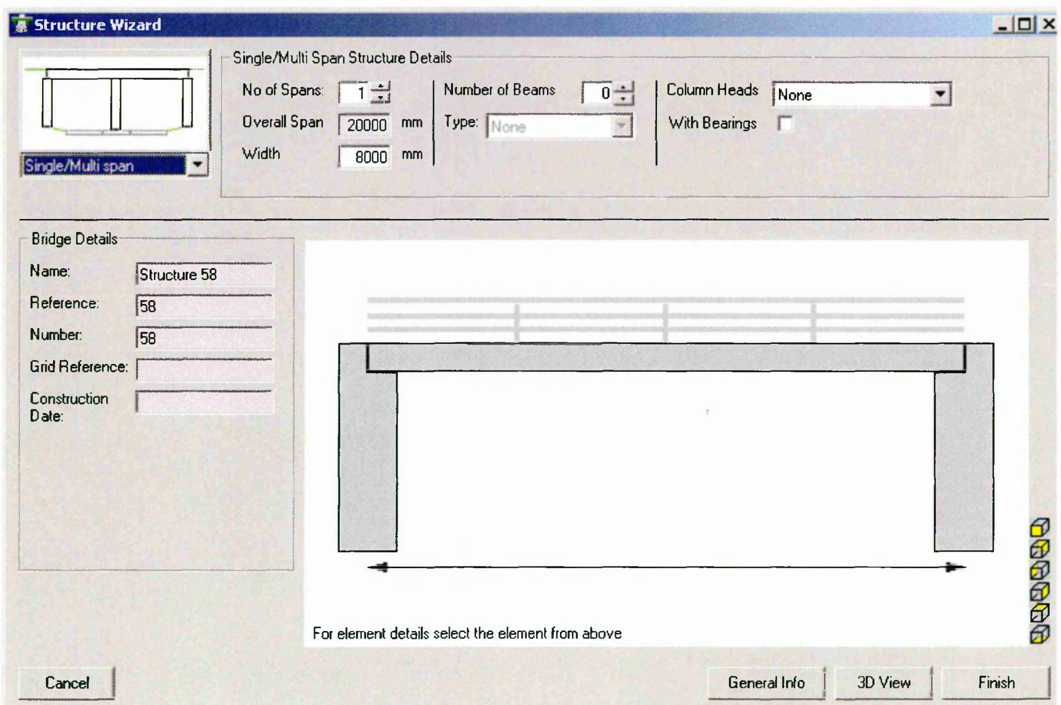



**Figure 6.3 'Structure creation wizard' single span bridge**

Figure 6.3 shows how a structure can be created quickly and easily. The structure type is selected first, then the number of spans, the span lengths and the structure width. The finished structure can be easily modified further.

Figure 6.4 shows the 'Structure creation wizard' again, this time showing the creation of a three span bridge shown in 3D.

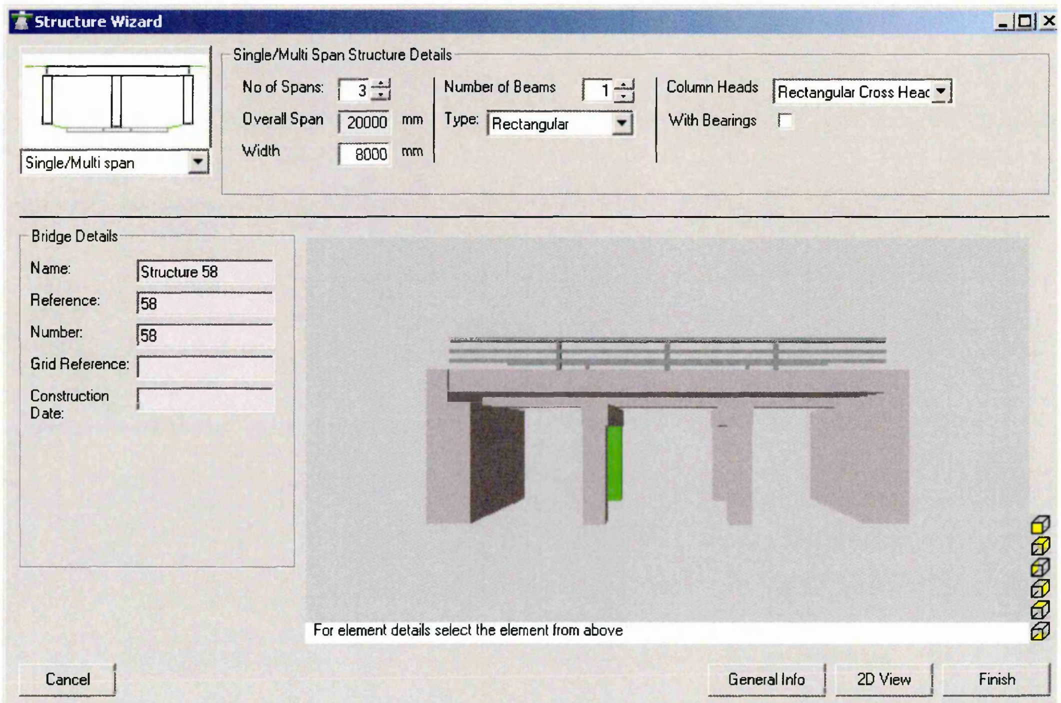



**Figure 6.4 'Structure creation wizard' three span bridge in 3D**

Once such a structure is inserted its geometry can be quickly and easily amended to match that of the structure being modelled. It is important to note that the routines developed in this thesis are not sensitive to slight differences between the actual and modelled geometry of the structures being assessed. It is important for the expert system to have only a reasonable indication of the relative sizes of elements and defects, and as such careful precision is not necessary when element sizes are being entered into the program.

An alternative method for entering the layout of a reinforced concrete structure into the system is to place elements individually as shown in Chapter 5 Figure 5.1. This method is suited to more unusual structural forms for which the structure creation wizard makes no provision.

### 6.3.1 Example of structure creation

A structure with a rhomboidal articulation arrangement over its piers could be considered an unusual concrete highway structure. Such a bridge is shown in Figure 6.5.

**Figure 6.5 Unusual motorway bridge**

Figure 6.6 shows how the user would begin to insert such a structure as shown in Figure 6.5 into the program. Firstly the user would use the standard menu and click ‘insert new structure’ from the ‘File’ menu. This action will automatically show the ‘Structure creation wizard’. In the case of the complicated structure shown in Figure 6.5, the structure creation wizard will have no suitable template to model the structure. The user closes the structure creation wizard window and is presented with a blank diagram window, as shown in Figure 6.6. The user’s next action is to click ‘insert element’ from the ‘structure’ menu.

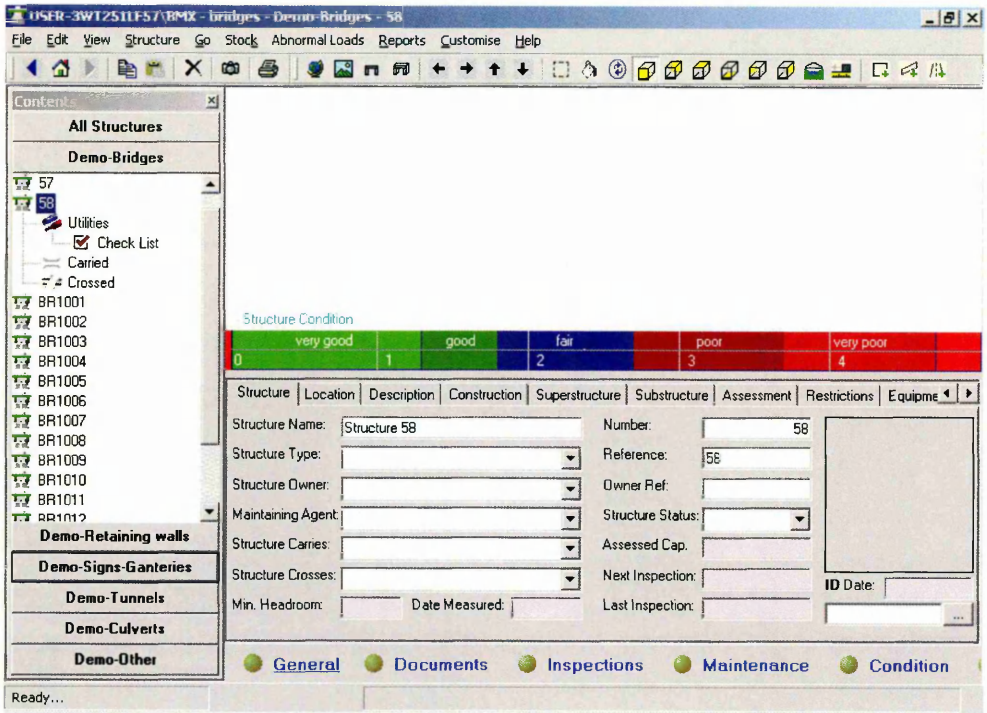



**Figure 6.6 Blank diagram window**

From here the user is presented with a menu giving a large variety of bridge elements, a beam is selected and is drawn onto the screen, generally at the required dimension. This operation is shown in Figure 6.7 and Figure 6.8.

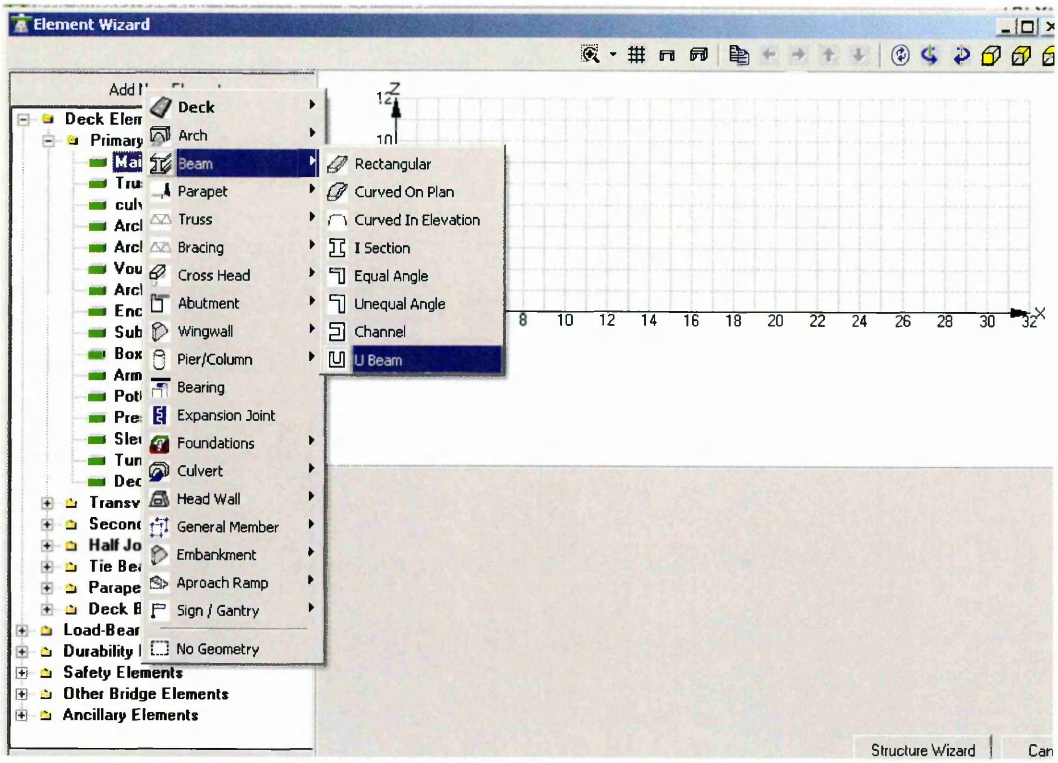



**Figure 6.7 Inserting a beam**

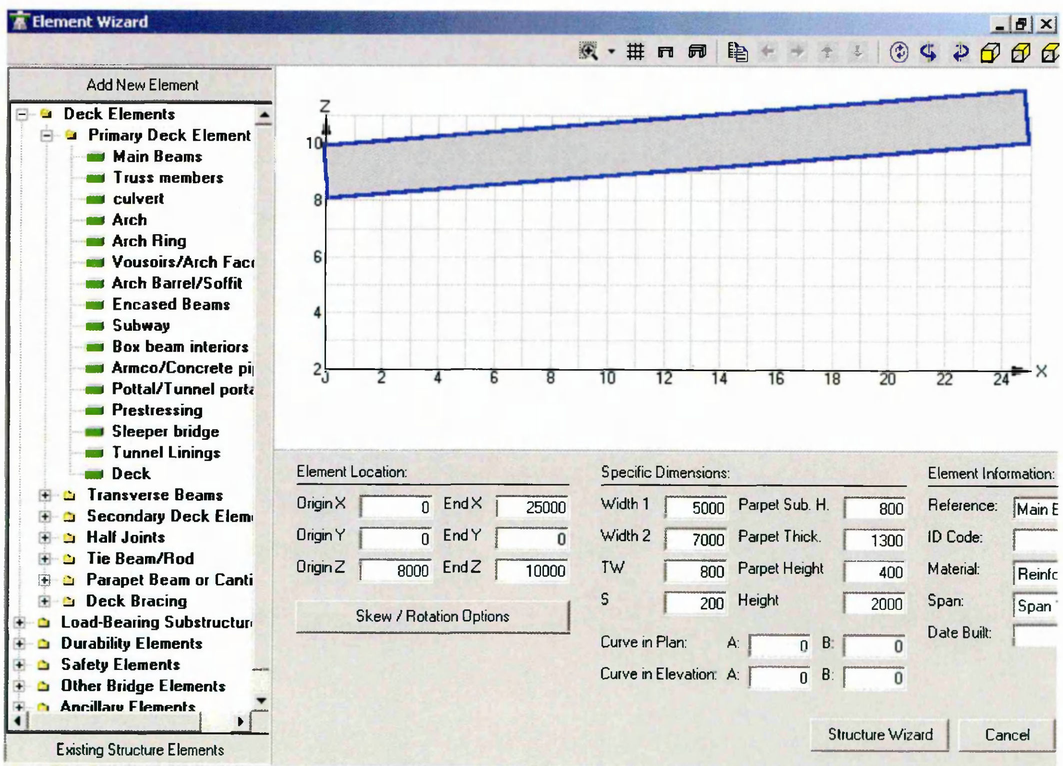



**Figure 6.8 Beam once inserted**

Further beams are added using the same technique. Figure 6.9 shows the insertion of the third and final beam. The span lengths, and the elevations of the start and end of the beam can easily be amended.

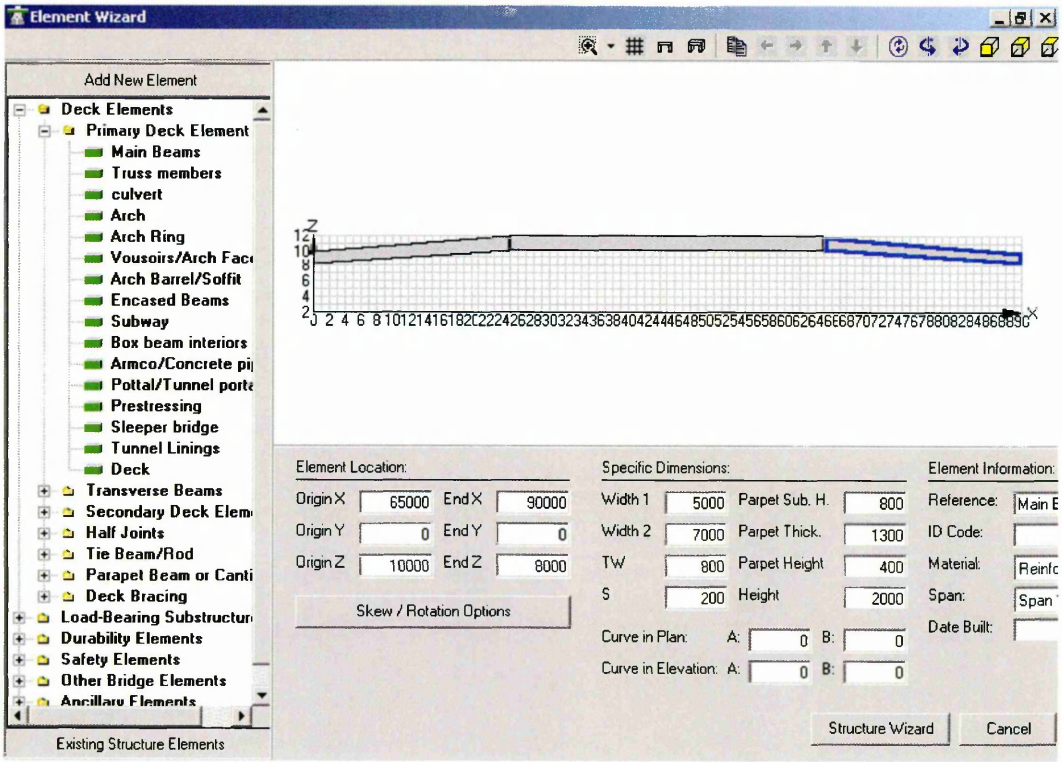



**Figure 6.9 Three beams inserted**

Finally the piers are inserted using the same techniques previously outlined, this is shown in Figure 6.10.

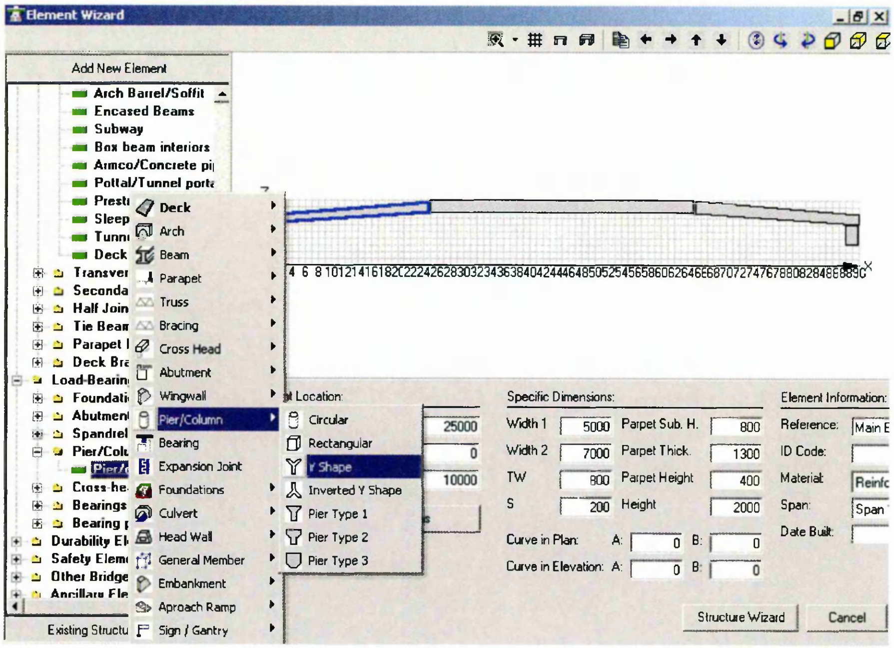



**Figure 6.10 Insertion of pier**

The program contains the full functionality that is expected of modern windows based software. For example, the first pier inserted can be cut and pasted to create the second pier. This is shown in Figure 6.11.

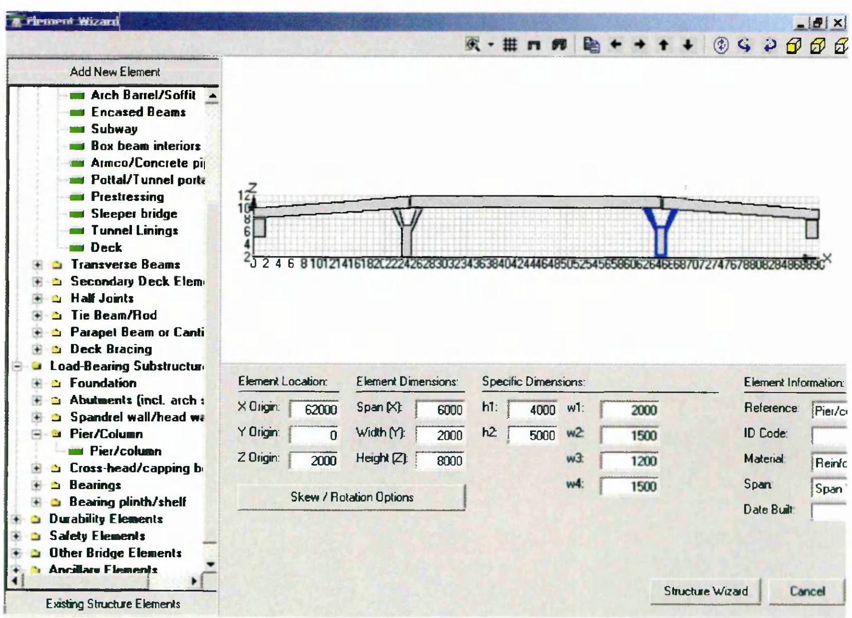



**Figure 6.11 Copying an existing pier**

## *6.4 Defects – input, assessment and diagnosis*

The types of defect that the expert system can recognise and diagnose were discussed in Chapter 5.

For the purpose of entering defects into the expert system, two general groups of defects are considered, specifically patches and cracks. Clearly, a crack will be caused by either structural or corrosion effects, whereas a defect patch will rarely be caused by structural reasons. Entering either type of defect onto an element is fast and simple.

In Figure 6.12, the user has highlighted a column from the bridge view window. The program unwraps the shape of the element, and the user enters an elliptical defect by pressing the 'add elliptical defect' icon and using the common 'click and drag' technique to add the defect of the required size in the required position on the element. The premise behind this technique is to enable the user to be able to describe the general shape of the defect – the user may add a defect which is generally square, or generally elliptical.

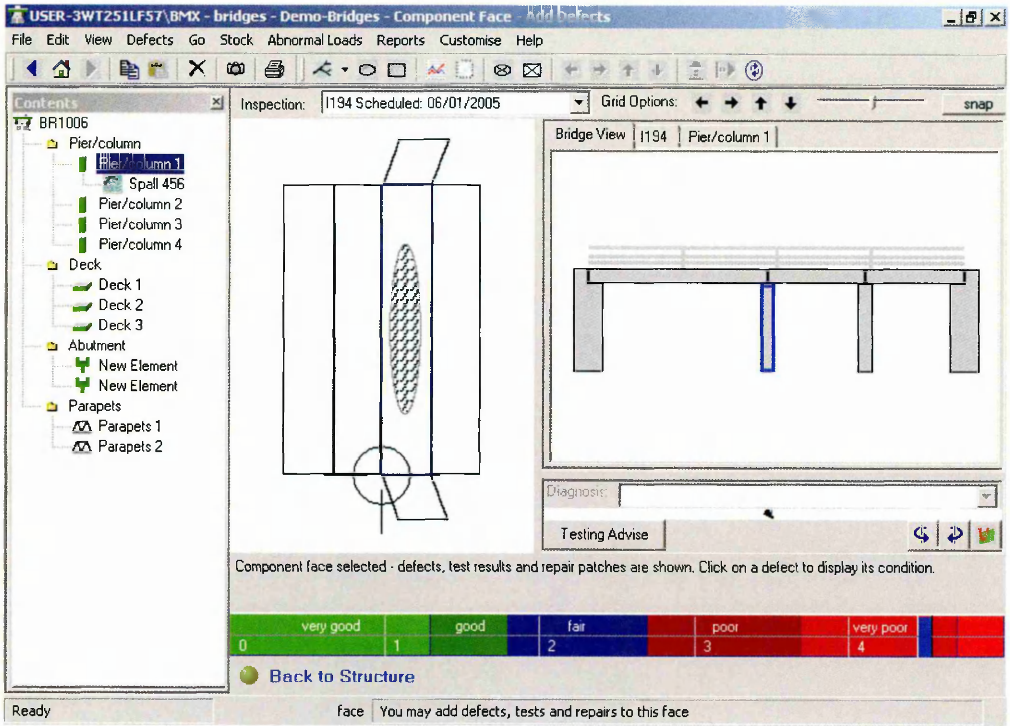



**Figure 6.12 Entering an elliptical defect**

The screen view shown in Figure 6.13 is presented to the user once a square or elliptical defect has been added to an element. On this screen, the user categorises the defect as either spall, stain, map cracking, seepage, scaling, honeycombing, blow holes or sandstreaking.

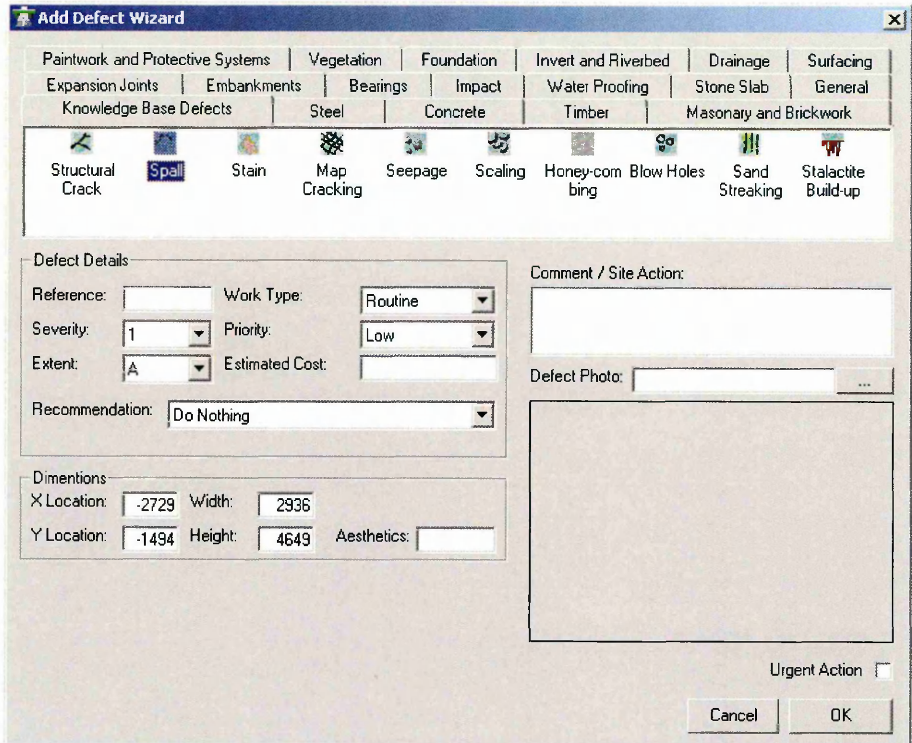



**Figure 6.13 Classifying the defect**

Should the user categorise the defect as a spall, the 'spall detail' window, shown in Figure 6.14 will appear, and the user is requested to enter detailed information about the defect.

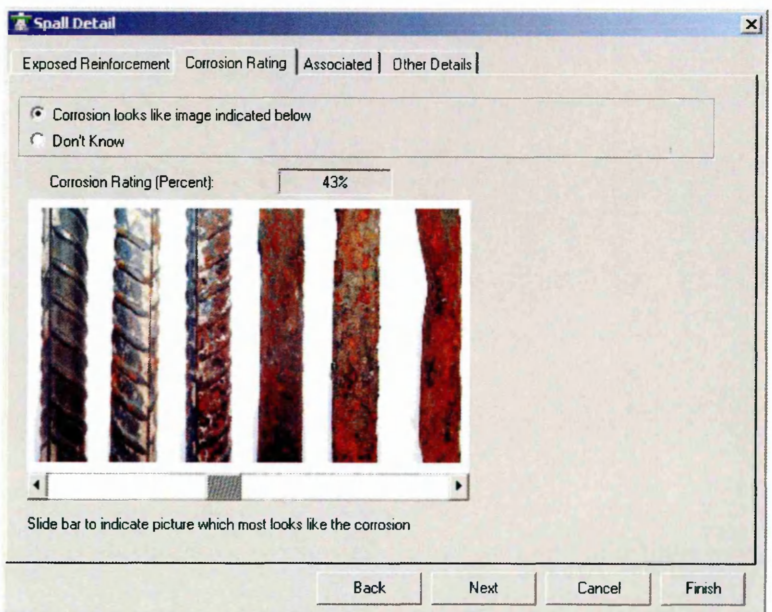



**Figure 6.14 Entering corrosion information**

Figure 6.14 shows the 'corrosion rating tab' in the 'spall detail window'. The user is requested to judge the general condition of any exposed reinforcement on a scale of 0 to 100, with a series of images to enable consistent results.

Figure 6.15 shows, once again, the 'spall detail' window, although this time with the 'other details' tab selected. In this area the spall depth and other information can be added. It is also possible in this view to give more information about the shape of the spall. For example, had a rectangular defect been added, the user could set the shape to 'perfect rectangle' – the knowledge base would recognise that the defect was likely to be a failed previous repair.

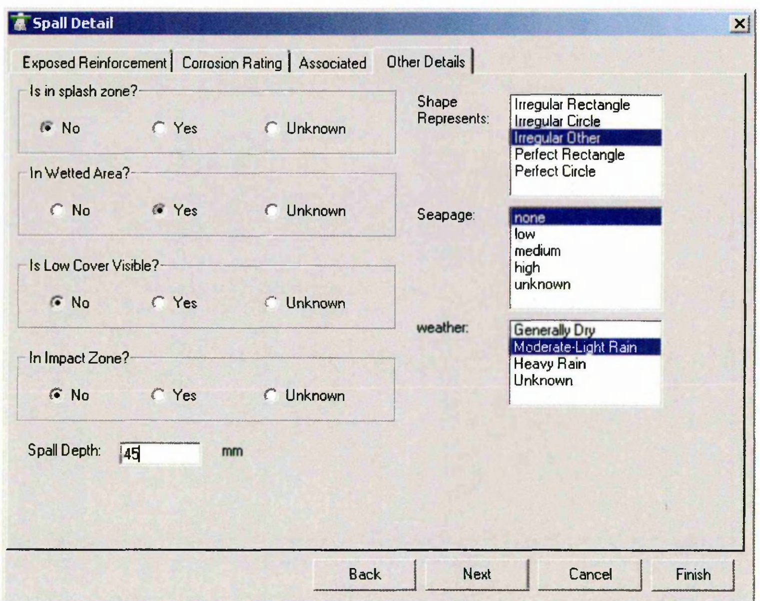



**Figure 6.15 Entering more spall information**

Figure 6.16 shows the status of the element after the spall information has been added. As discussed in the previous chapter, the vertical red band on the severity scale represents the defect size. With a spall depth of 45mm entered, the system has judged this defect as a 'minor repair'. However, the defect falls between the zones 'cosmetic repair' and 'minor repair'. The system has chosen the most severe zone because, on this occasion, the user chose to leave a lot of spall detail set to 'unknown' (such as the amount of exposed reinforcement). Therefore the system uses the techniques developed in the previous chapter to place the defect in the most severe zone.

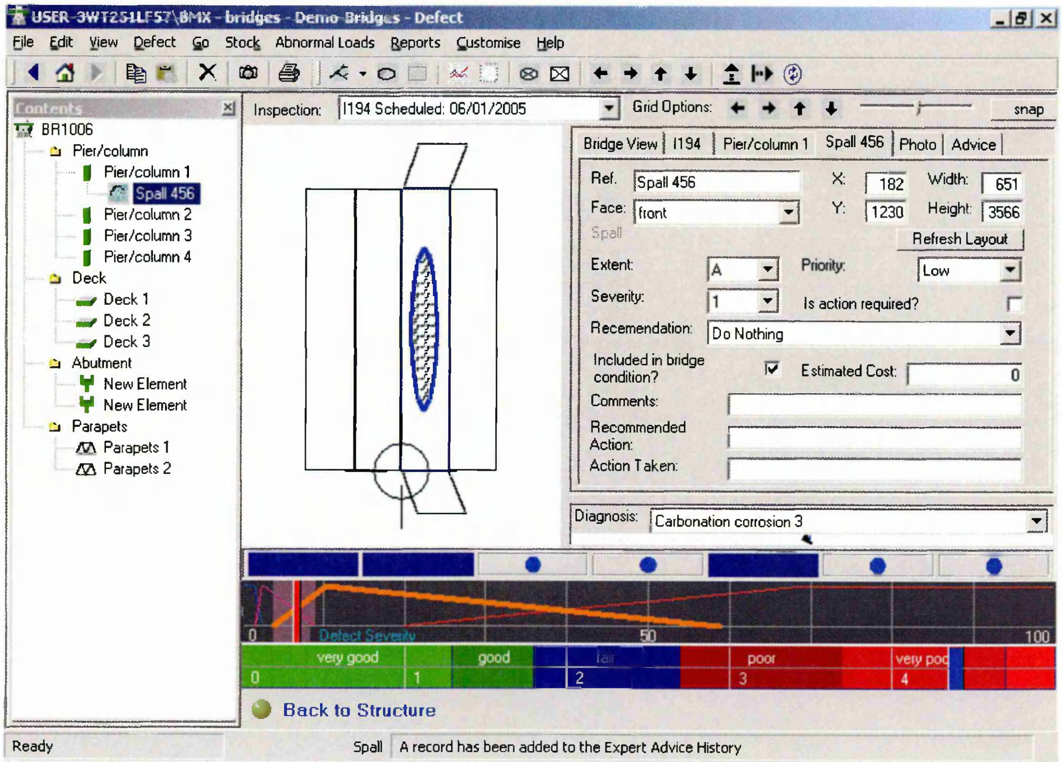



**Figure 6.16 Judging spall severity**

Figure 6.17 shows the insertion of a generally rectangular defect onto a different face of the same column, in the same way as described previously. This shape will represent a map cracking defect.

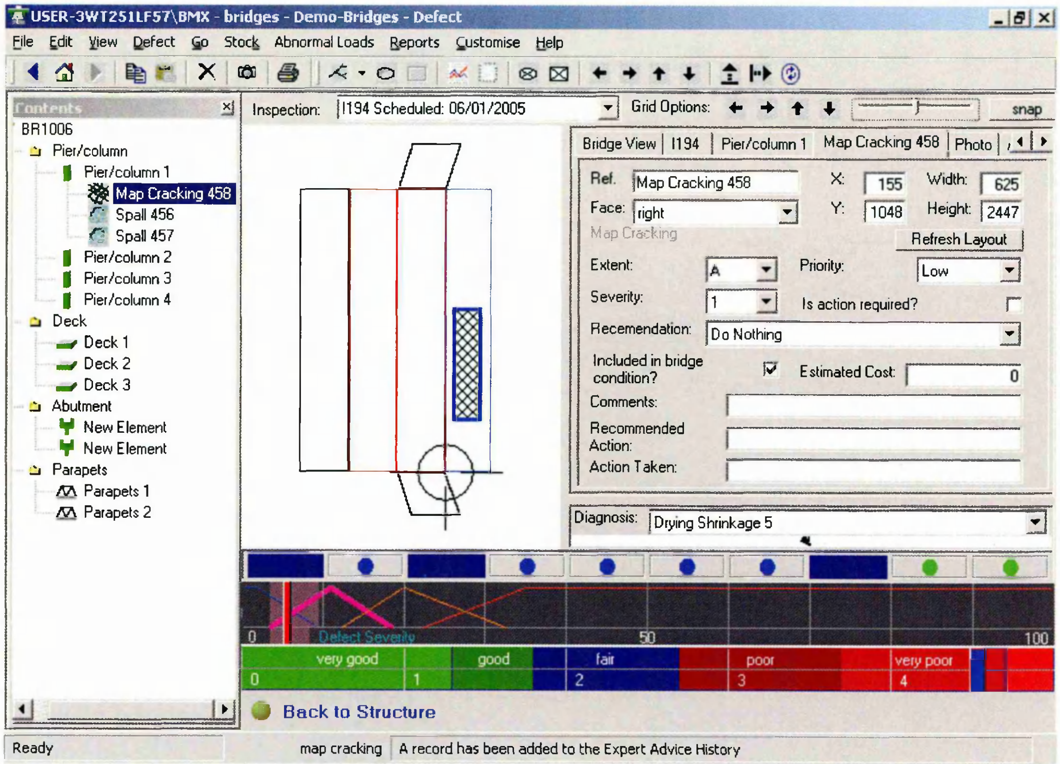



**Figure 6.17 Entering a map-cracking defect**

Figure 6.18 shows the 'map crack details' window. If the user identifies a defect as map cracking (Figure 6.13), this window will appear. The user can scroll through a series of images in order to identify the one which best represents the defect being entered. The example in Figure 6.18 shows alkali-aggregate reaction at varying stages of developing, although, importantly, the user is at no stage told which type of defect the image represents. Once the user has chosen the most representative image, they are returned to the element screen view, and, in the same way as shown previously for a spall defect, the element condition is given a rating by the program (Figure 6.19). In this example the user chooses the severe AAR image. There are six other tabs through which other information about the defect can be entered.

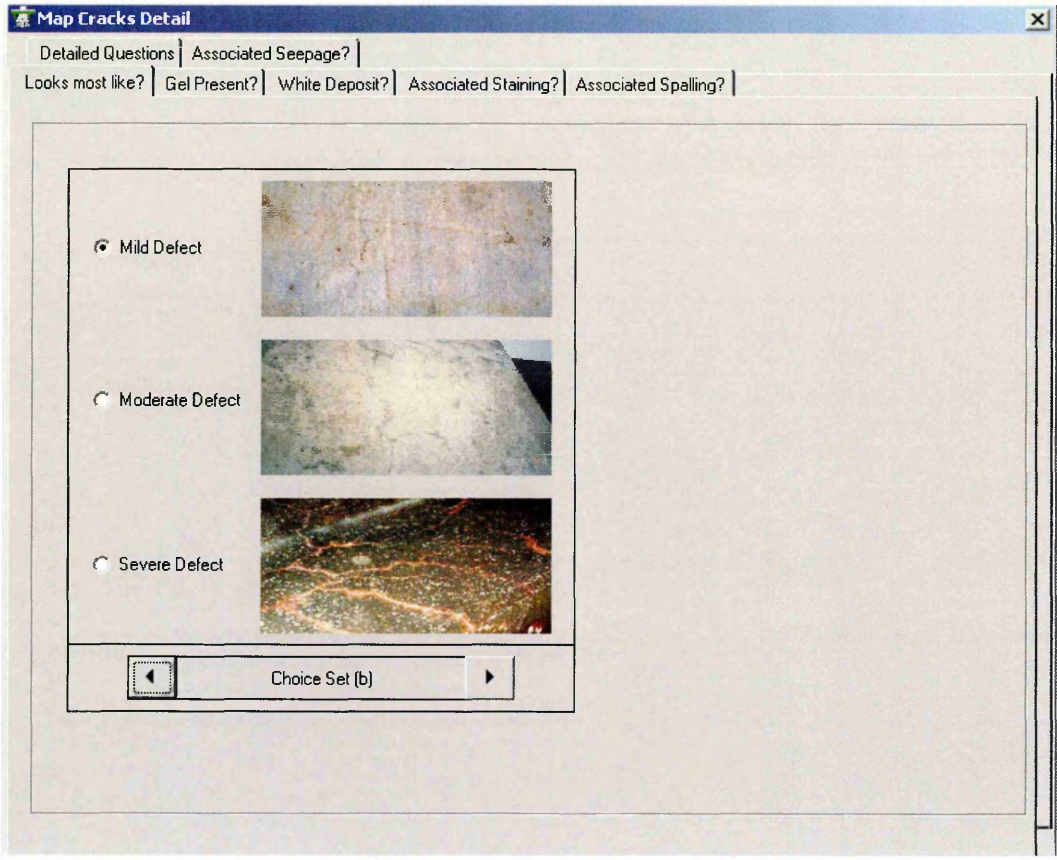



**Figure 6.18 Choosing a representative image**

In Figure 6.19, the user has clicked the 'advice' tab over the rightmost window. This causes the knowledge base to run. In this simple example, as would be expected, the knowledge base advice is that the chances of the cause being 'early AAR' is low, and the chances of the cause being 'AAR' is high. The knowledge base can make more sophisticated judgements for other defects which are less simple to judge – such as the differences between freeze-thaw damage, chloride corrosion and carbonation corrosion. It is quite possible for a user to select the 'freeze-thaw' image and for the system to still give advice that the defect is possibly caused by, for example, chloride corrosion – depending on the additional information entered by the user.

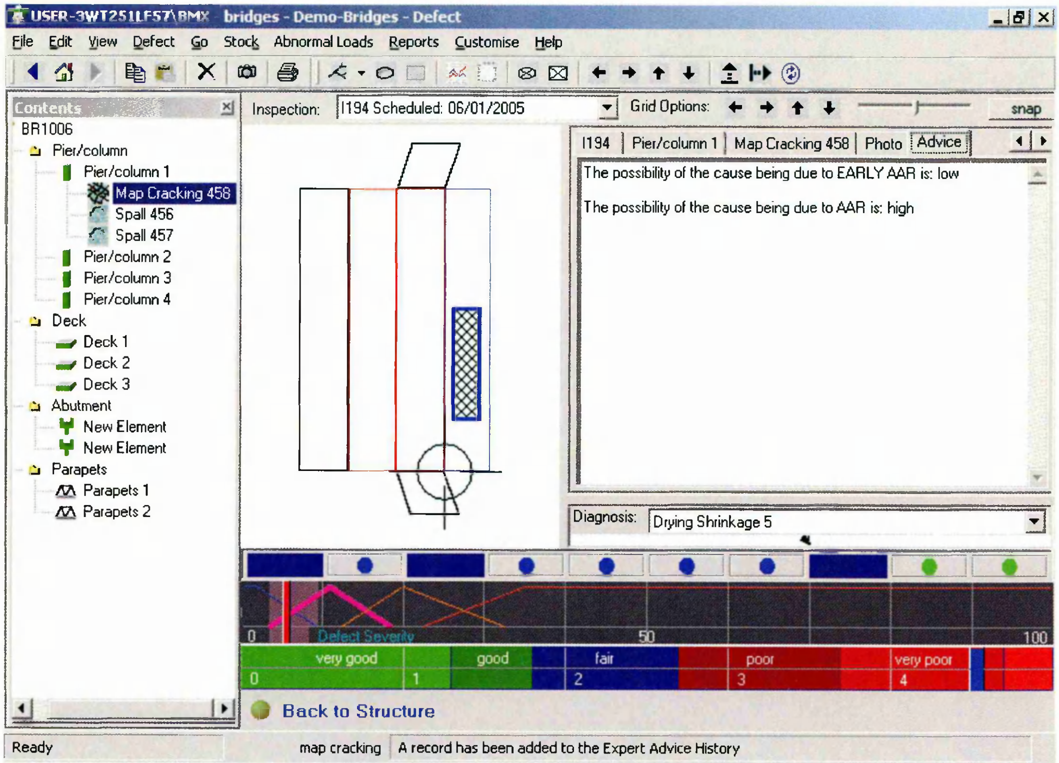



**Figure 6.19 Viewing expert system advice**

It is possible to obtain a full report showing the decision taken by the expert system for any defect. Figure 6.20 shows the knowledge based objects, and the values given to them by the program for the defect in question.

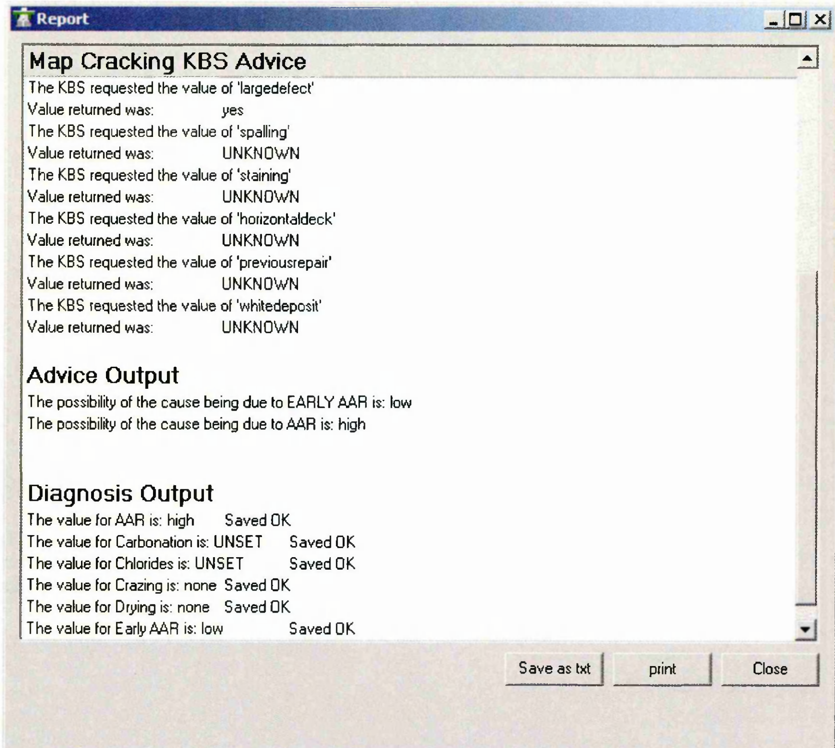



**Figure 6.20 Detailed knowledge base output**

The system provides the user with a good amount of functionality for locating and examining defects. In Figure 6.21, a 3D view of the current column is shown, both the defects entered can be seen.

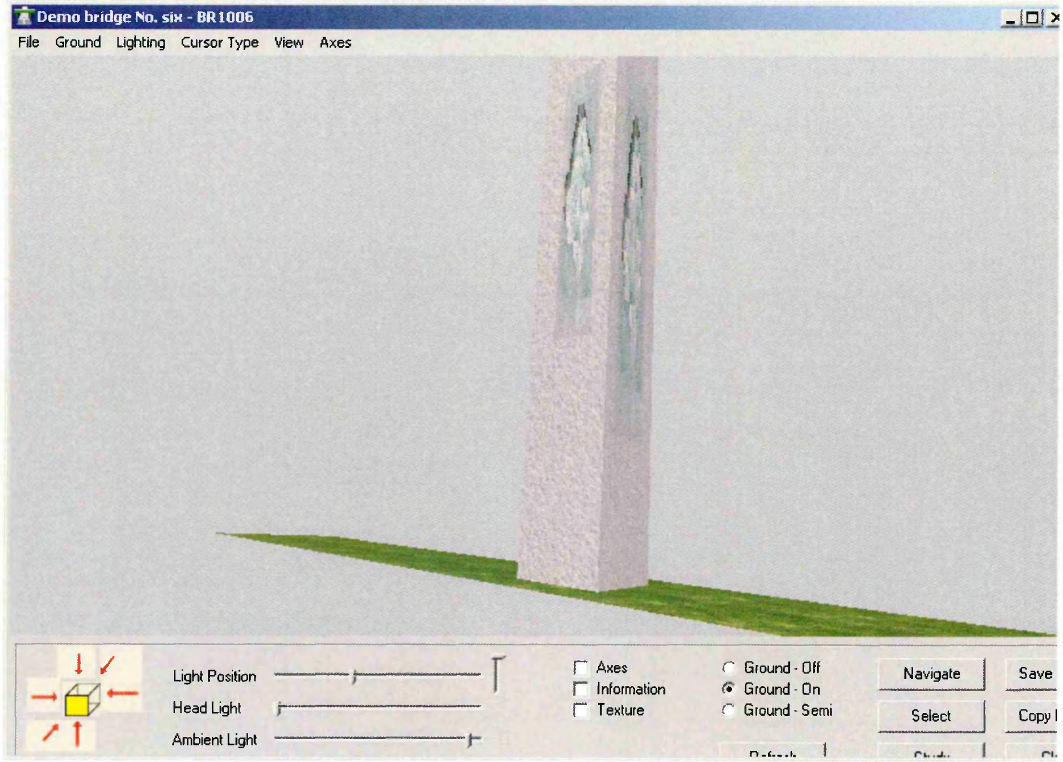



**Figure 6.21 3D view of affected column**

Once a straight line crack defect has been input into the program, the decision making processes of the expert system begins. The initial decisions of the expert system can be seen immediately in the zonal severity classification area (lower right – Figure 6.22). The system will then present a window requesting further information about the crack, such as its width, associated staining etc. Knowledge bases then give recommendations for the crack and information is delivered back to the user.

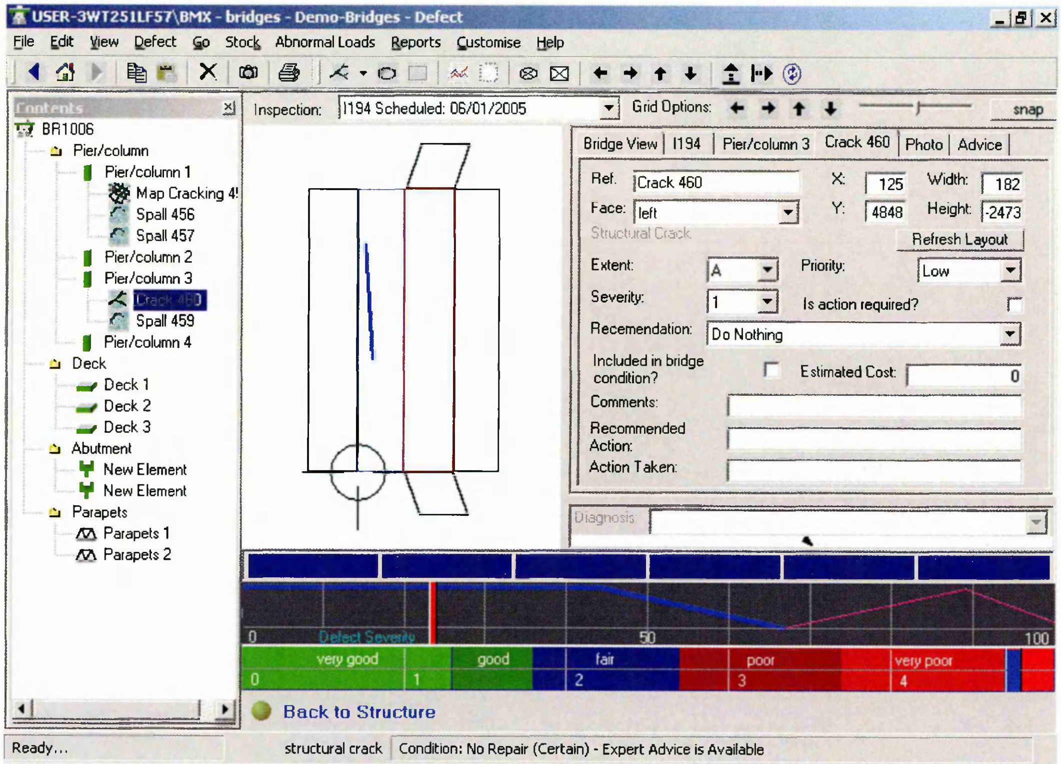



**Figure 6.22 Entering a crack defect**

## 6.5 Elements – Testing and repair

As soon as all the defects affecting an element within the program have been added, the system is ready to run the testing knowledge bases. The example shown in Figure 6.23 is for a column with two significant defects. The first defect, map cracking, was judged as being caused by either freeze-thaw action or carbonation corrosion. The second defect, a spall, was judged as being caused by chlorides. Once the user is satisfied that all the present defects have been entered into the system, the 'Testing Advice' tab is clicked. This prompts the system to run the testing knowledge bases, and the advice shown in Figure 6.23 is generated.

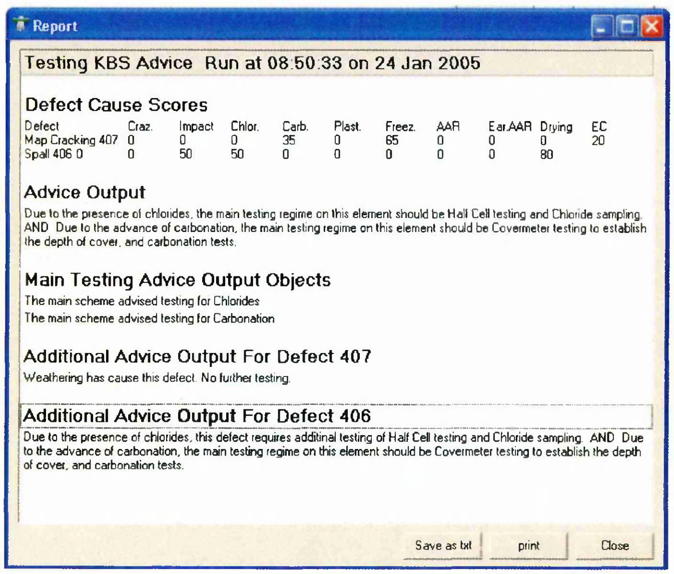



**Figure 6.23 Element testing advice from the knowledge base**

The main testing knowledge base looks at the element as a whole, and its advice, based on the causes of the defects present, is to test for chlorides and carbonation. Thereafter, each defect is examined individually to see if its cause falls under the advice of the main testing knowledge base. In the example of Figure 6.23, defect 407 (map cracking defect), does not require any additional testing over and above that prescribed for the element as a whole. Similarly, defect 406 (the spall), falls under the general advice for the element. However, should any of the defects be diagnosed as having a considerably different cause than the element as a whole, specific testing advice for those defects would be generated.

Once the testing for the element has been undertaken, repair of the structure, if necessary, can commence. Figure 6.24 shows repair advice for a wide crack on a flat horizontal surface, having been attributed a width of 20mm. The advice of the system is to repair the crack with an overlay.

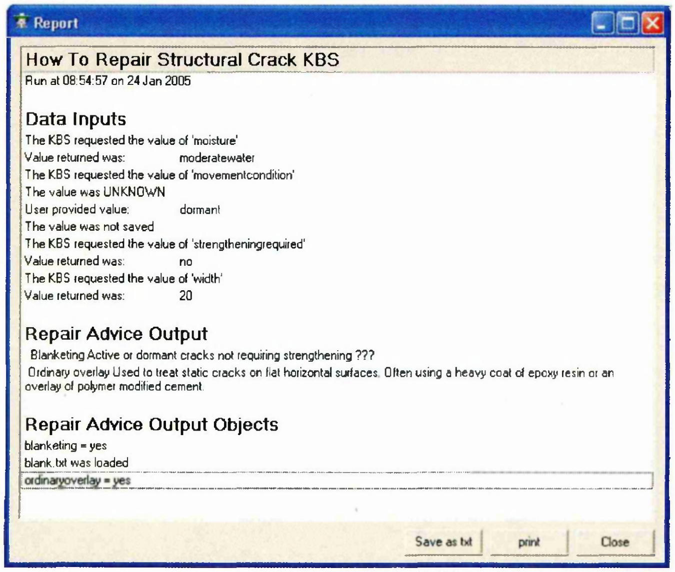



**Figure 6.24 Element repair advice from the knowledge base**

Figure 6.24 is the area of the program where the different types of repair advice shown in Table 5.25 (Chapter 5) are displayed. For many significant defects, this advice would read 'break out and repair'. If this advice occurs, the repair material selection routines can be employed.

## 6.6 Repair material selection

The repair material selection operation of the program is the area in which the key development work of this thesis is employed. Once an inspection has been completed, and the user has confirmed the findings of the knowledge bases (or corrected their findings should test results have proved them wrong), the advice of the repair knowledge base may well have been to break out and repair the affected concrete. Should this be the case, the system user is required to indicate the extent of the patch repair that will be carried out. Once this action has been completed, the repair material selection procedure begins.

Figure 6.25 shows the database of reinforced concrete repair materials and their properties at 28 days age. These commercially available materials come ready programmed into the software and the user has the opportunity to add an unlimited amount of further materials.

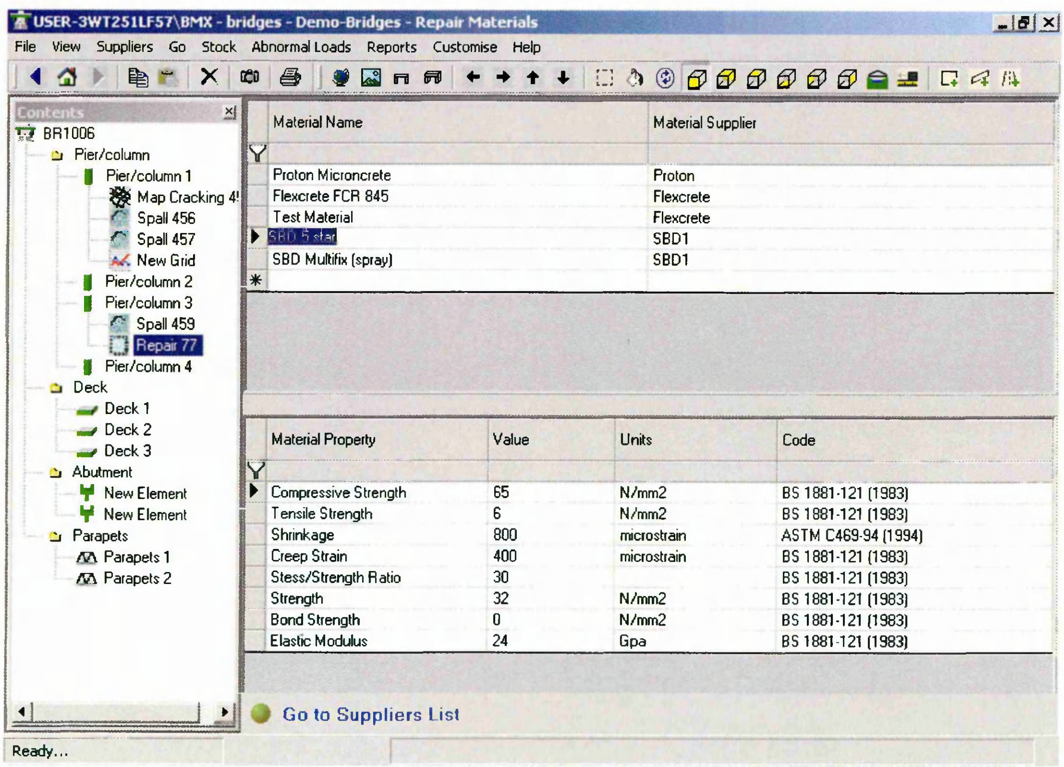



**Figure 6.25 Manufacturers' test data for repair materials**

When ready to proceed, the user is asked to enter the substrate information, as shown in Figure 6.26. The system is already aware of the size of the repair. The bridge location should have been entered by the user at an earlier stage (during population of the structure database) and, therefore, the software is ready to determine geographical climate effects. The remaining unknown data is gathered in the 'substrate properties screen', specifically substrate compressive strength (N/mm²) and elastic modulus (kN/mm²). The height and diameter of the core is required in order to apply the relevant factors and the scheduled date of the repair will allow the climate effects to be determined correctly. Upon clicking OK, the performance of all the repair materials in the database is assessed for the repair. The results are specifically tailored for the size of the repair, the strength and elastic modulus of the substrate, the location of the bridge, the date the repair will be undertaken and the size of the core taken from the structure. Repair on different structures, in different places at different times, will produce different results. The example shown here is for the spall in Figure 6.16, in Edinburgh, with the additional details from Figure 6.26.

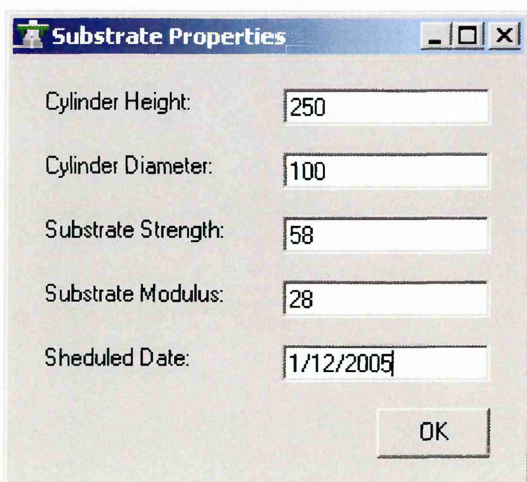



**Figure 6.26 Substrate information**

Of the five repair materials in the database, using the techniques developed in Chapter 4 of this thesis, three would perform adequately. Figure 6.27 shows the three successful materials listed in the top left window.

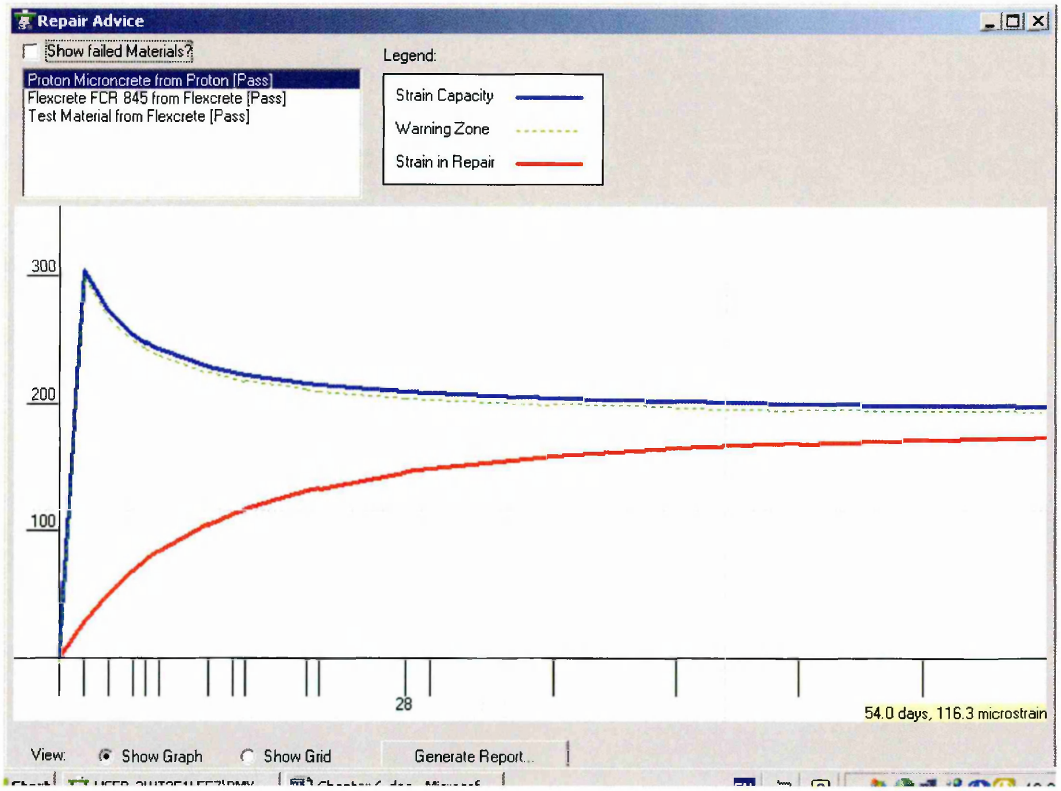



**Figure 6.27 Performance of Proton Microconcrete**

The first successful material, shown in Figure 6.27, is Proton Microconcrete. The blue line represents the strain capacity of the repair material. As the material shrinks, the restraint to this shrinkage at the interface between the repair and the substrate causes tensile strains, shown by the red line. These tensile strains continue to increase up to 200 days before they plateau at a value of approximately 170 microstrain. The strain capacity, being 200 microstrain at 200 days, is greater than the strain in the repair material and, therefore, the material performs successfully although perhaps, in this case, the margin of success is less than desirable. The green dotted line represents an additional factor of safety, materials with restrained shrinkage strains above this line will be classed as failed.

Figure 6.28 shows the performance of the Flexcrete material. This material performs in a noticeably different manner to that in Figure 6.27. At approximately day 12, the elastic modulus of the repair material becomes greater than that of the substrate concrete. Consequently, some of the shrinkage strain in the repair material in transferred into the substrate concrete in accordance with equation 4-6 (Chapter 4). As the elastic modulus of the repair material continues to develop, more and more of the developing shrinkage strains are transferred into the material until, as approximately day 50, the repair material is stiff enough to transfer all its strain into the substrate. This material would, therefore, be a very safe material to employ in this situation.

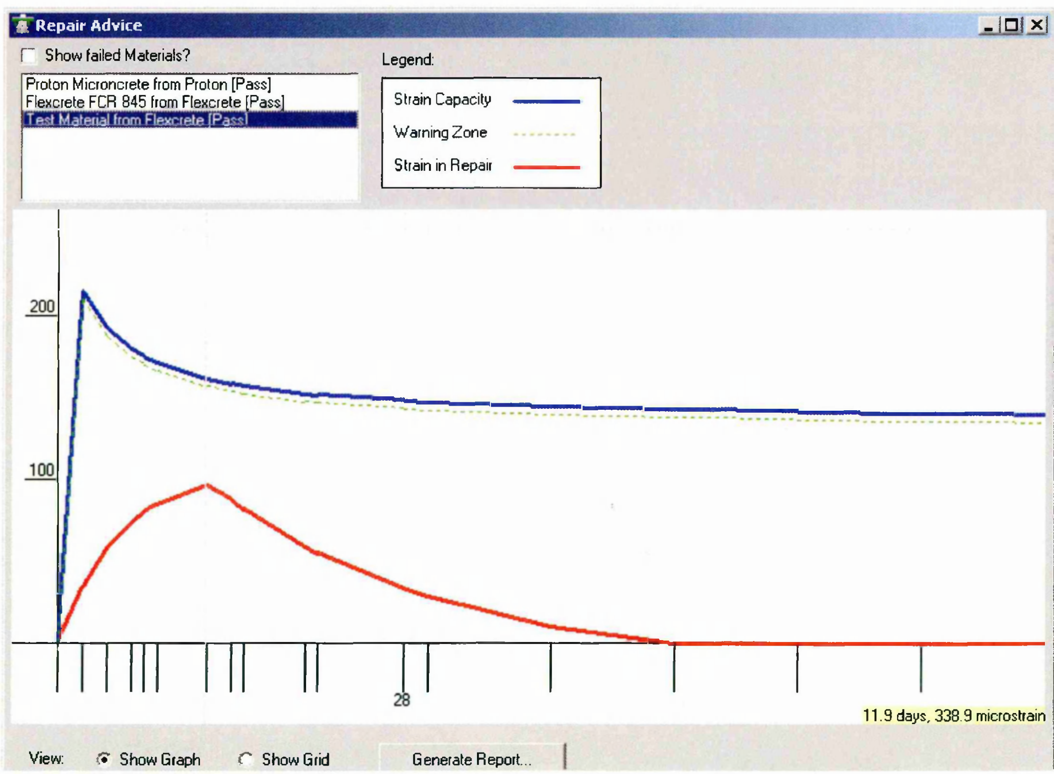



**Figure 6.28 Performance of Flexcrete material**

If the user checks the box in the top left hand corner of the screen, the failed materials will also be displayed. This is shown in Figure 6.29.

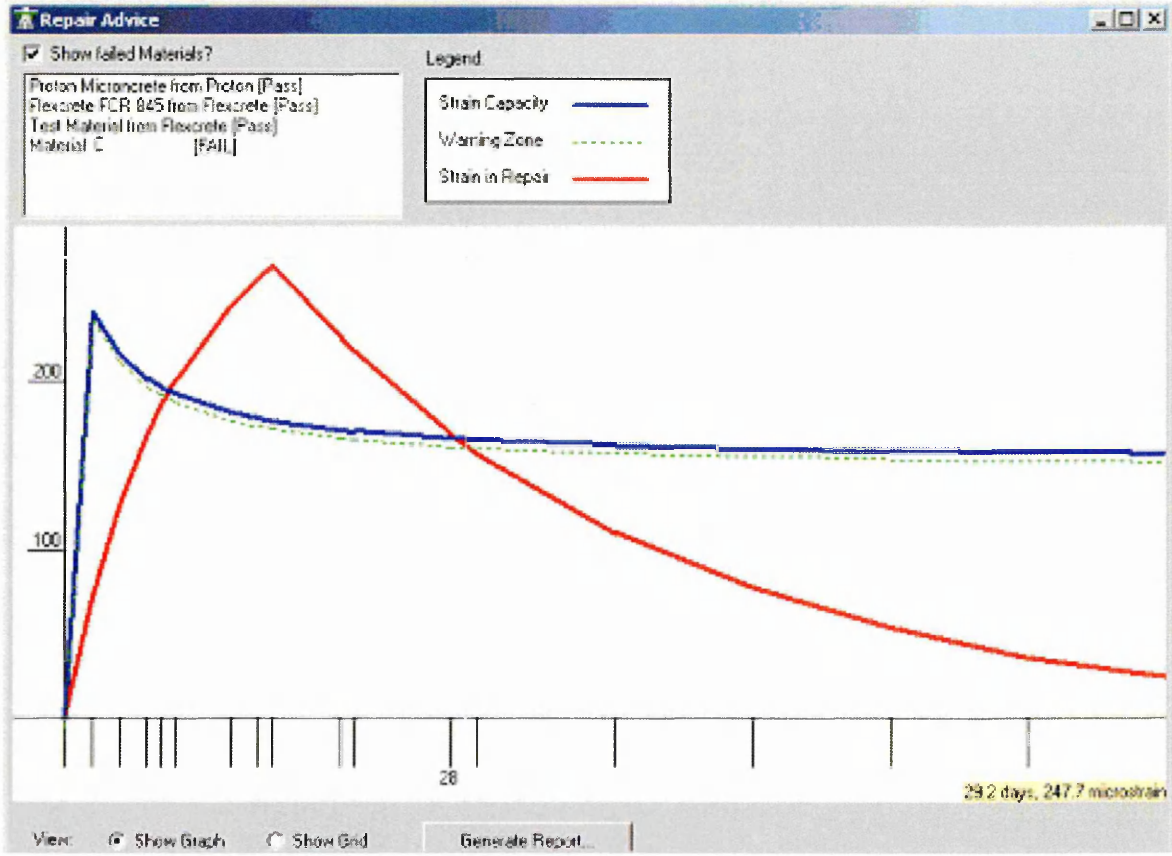



**Figure 6.29 Failed material**

Material C in Figure 6.29 has a very high elastic modulus, and quickly becomes much stiffer than the substrate concrete. However, the material has a high shrinkage, and before the repair material has become stiff enough to transfer restrained tensile strains into the substrate, its strain capacity has already been exceeded. The material fails at approximately day 10. Thereafter the shown performance must be disregarded. This material would obviously be avoided for the particular repair situation which generated the shown result.

## 6.7 Summary and Conclusion

The commercial partners in the research presented in this thesis have built a bridge management system around the thesis recommendations. The bridge management system is a database through which an organisation's bridge stock can be organised and managed from inspection through to prioritisation and maintenance.

In order for the expert system to have the necessary intelligence to make useful decisions, there was a need for the program to gather geometrical and geographic information about the structure being examined, which led to the development of the interface that allows users to assemble structures on the screen from their basic elements. This feature is also used for the efficient inputting of defects, allowing the system to gather the information it needs to make decisions based on the relative size of elements and defects.

As the user adds information about the nature of defects, the various knowledge bases begin their decision making processes and their findings are displayed to the user. The expert system performs a number of key functions:

- Diagnoses the cause of a defect
- Rates the severity of the defect on a scale from 0 to 100
- Rates the condition of an element based on all the defects affecting it
- Offers recommendations of which tests to perform on an element based on all the defects affecting it
- Offers repair advice for each defect on an element, based on the recommendations of the knowledge bases which preceded the repair advice stage
- Chooses repair materials which will perform adequately should any defects require to be broken out and repaired

The purpose of the system is to act as an intelligent advisor at all stages of the concrete repair process, from defect identification through to repair. The created system achieves this task.

A simple and effective method for determining the extent and severity of reinforced concrete defects has been developed. Structures can be quickly modelled within the software, and defects can be added onto the modelled structures. As a result, the software is immediately aware of the extent of defects.

In conjunction with experienced concrete repair practitioners, a system has been developed to allow the software to place any of the three key defect types (spalling, map cracking, structural cracking) into one of four 'decision zones'. These decision zones define the four likely repair categories for a defect, namely, 'do nothing', 'cosmetic repair', 'minor repair', 'major repair'. Primarily, the position of these zones is decided using key factors. For example, the key factors for a spall are size and depth; for map-cracking the key factors are size and the defect cause suggested by the visible cracking. Thereafter, zone positioning is altered depending on secondary factors. For example, secondary factors for a spall may be the amount of reinforcing steel exposed by the spall, the condition of the steel, or the amount of staining and seepage associated with the spall.

Knowledge bases use information such as the repair zone into which a defect has been placed, the defect's shape, its proximity to the carriageway and the bridge age, to decide likely defect causes. Additional knowledge bases examine all the defects affecting an individual element, and recommend testing regimes to confirm the defect causes. Finally, a third layer of knowledge bases recommends how to repair the defects.

# 7 Conclusions

The repair of reinforced concrete is the subject of broad ranging research due to the ubiquitous maintenance requirements of reinforced concrete in all environments.

The stated aim of this research was twofold:

- To examine, and develop further, existing state of the art research into compatibility between reinforced concrete repair materials and the substrates on which they are employed. Furthermore, to prepare a method for determining the long-term insitu performance of repair materials, which, when employed in a software system would enable identification of repair materials suitable to perform in the situations required.
- To prepare an expert system for concrete repair, to work in conjunction with the repair material selection system, which will output intelligent advice at all stages of the reinforced concrete inspection and repair process.

In the software developed, the relationship between traditional computer programming to assess the severity and extent of defects, and less traditional expert system techniques for decision making works seamlessly and effectively. The simple methodology of expert system development devised in this research could be employed with similar effect in many others areas particularly where complex objects can be placed into sets (such as the way reinforced concrete defects are placed into the four repair decision zones). The practice adopted was to identify two key factors in decision making, for example, to determine the severity of a spall, the size and depth of the spall were the key factors. Experts were asked to relate these two factors graphically. This allowed the experts' opinions to be represented with numerical equations, providing a sound initial estimate of the severity of a defect. This initial estimate can be refined by considering additional factors, also represented numerically based on expert opinions.

Thus the method of knowledge elicitation developed in the research to assess reinforced concrete defects is effective in incorporating cumulative knowledge of practising experts. This approach was considered a great benefit as it allowed the expert system to be developed within the timeframe available, ensuring adequate time for other key aspects of the research to be undertaken. The methodology adopted has produced sensible and reliable results. It provides a relatively simple and practical approach for expert system development in the field.

The performance of reinforced concrete repair materials is a field of research with a broad range of varying opinions amongst experts. In particular, expert opinion varies regarding which properties of materials are important to specify. This research has reviewed and considered the spectrum of opinion and has adopted the premise, proven through field testing, that elastic modulus, shrinkage and creep are the crucial properties for the performance of concrete repair. Strain in a reinforced concrete repair material is affected by the growth of these properties, their effects on one other, and interaction with the substrate. These phenomenons have been incorporated into a routine which can predict the growth of tensile strains in repair materials with time, and make comparisons against the tensile strain capacity of materials. The method developed to predict the performance of reinforced concrete repair materials is based on the measured field performance of a variety of materials. It adequately models tensile strain resulting from restrained shrinkage in repair patches. The procedure allows an accurate assessment of the performance of a repair material to be made. Thus selection of materials can be made in an auditable and scientific way, rather than on an ad-hoc basis.

The routine developed has been incorporated into a software program. Lengthy iterative calculations are performed by the software and users are graphically informed when and how unsuitable materials will fail. Accordingly, materials shown to perform well can be selected, and engineers can justify the choice.

Over-arching the repair material selection software, and the inspection and repair expert advice software, is a structures management system, which seamlessly ties together the research outlined in this thesis. The software will advise engineers on the cause of a defect, this advice can be confirmed by testing. A testing regime will also be recommended by the system. The software advises the engineer on how to repair a defect, and will filter out repair materials that are suitable for use from those which are not.

During interaction between the software and the practitioner, the opportunity is always available for engineers to make the final decision themselves, either in consultation with more experienced colleagues or through a review of the available literature. Moreover, a practitioner can be reassured that if his opinion agrees with that of the expert system, then the opinion of the vastly experienced panel of experts interviewed in the preparation of this research would also agree. Therein is the overall goal and originality of the software. To take the cumulative knowledge of the concrete repair practitioners, and the models developed for long term concrete repair material performance, and to accurately represent these in a responsive, adaptable computer program.

# 8 Further Work

## *8.1 Field testing and calibration of the expert system*

A rigorous assessment of the performance of the expert system in the field should be conducted. The results of such an assessment would be used to calibrate the expert system's performance so as to achieve maximum accuracy of diagnosis of defect and assessment of severity. This can be done through liaison with practitioners who use the system on site in genuine situations. If necessary, expert system rules can easily be amended to incorporate any revised opinions that arise from examining the expert system's performance.

It is important to assess how engineers agree with the severity ratings generated by the system for concrete elements and individual defects. The system has been developed in such a way that clear differences of opinion between experts and the system, in the field, can be reported to the software suppliers and easily remedied by modification of the many constant factors used to describe the collaborating experts' opinions.

## *8.2 Field testing to assess the performance of the concrete repair material property selection system*

The software components within the expert system that recommend optimum properties for repair materials should be tested in the field. The program can be used to select repair materials that will perform adequately – the success of materials selected by the system will demonstrate the veracity of the routine developed. However, it would be advantageous to be able to specify materials which the software shows will fail; how accurately the software predicts the time of failure of these repairs would be a good judge of its performance.

## 8.3 Prioritising the repair of bridges and bridge elements.

Some bridge elements are more important, when determining the condition of the overall structure, than others. For example, a severely corroded wingwall may have little impact on the performance of the structure as a whole, whereas a mildly affected central pier may be very significant. Because of the many different types of bridge design, a good deal of research may be necessary to endow the expert system with the intelligence necessary to recognise the importance of individual elements. However, if this task were completed, both prioritisation of element repair, and an accurate overall structure rating, would be relatively straight forward to develop. Comparing overall structure condition could be used to prioritise the repair of bridges, although, again, some bridges are more important than others. Decisions on which bridges to repair are not related solely to their condition, but also to factors such as location, use, the likely consequences of further deterioration, factors such as funding, value management and even local politics.

## 8.4 Expanding the expert system capability

This thesis has been concerned specifically with concrete defects and repair. However, the bridge management system which over-arches the software tools developed herein can manage all variety of bridge types: steel, concrete, masonry arches, culverts etc. Expert systems to diagnose defects and recommend repairs on other types of structure could be prepared. As concrete structures seem to be subject to more maintenance difficulties than other structures, it is conceivable that the development of expert system for the other structural types could be more straightforward. The logical methods for assessing extent and severity developed in this thesis could also be employed in these additional modules.

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## *9.3 Publications*

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