THE DESIGN LIFE OF STRUCTURES

Edited by G. Somerville

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Preface

There has been growing interest in the service life of structures due to a perceived lack of performance in structures built in the last 40 years or so. With that in mind, the British Group of IABSE organized a colloquium in July 1990 at Pembroke College, Cambridge, UK, which, in effect, was a workshop on the feasibility and practical application of design life principles to structures.

All sectors of the construction industry were represented, including private and public sector owners, local authorities, architects, engineers, contractors, material specialists and universities. The scope related mainly to bridges and buildings, and the subjects covered included design concepts, detailing, loads, performance requirements, predictive modelling, material specifications, workmanship and maintenance. Other papers covered seismic design and long-life structures. Additionally, there were contributions on how related industries (e.g. offshore, marine, nuclear and aircraft industries) coped with design life. Five of the 33 invited delegates were from Europe and North America.

In all, 29 invited papers were presented in four sessions, with reporters preparing extensive summaries of the wide-ranging discussions that took place in each. This book contains both the papers and the reports, together with a final summing-up, which gives the main conclusions from the colloquium, as well as indicating the way ahead in developing a structured approach to give better in-service performance.

G.S.

Vorwort

Angesichts der als unzureichend wahrgenommenen Leistungscharakteristiken von Gebäuden und Bauwerken, wie sie in den vergangenen 40 Jahren errichtet wurden, hat das Interesse an deren Nutzungsdauer zugenommen. Vor diesem Hintergrund organisierte die britische Sektion von IABSE im Juli 1990 ein Kolloquium, das im Pembroke College an der Universität Cambridge, GB, abgehalten wurde. Hierbei handelte es sich faktisch um einen Workshop zur Frage der Durchführbarkeit und praktischen Anwendung von Nutzungsdauer-Prinzipien auf Gebäude und Baulichkeiten.

Sämtliche Sektoren der Bauindustrie waren vertreten, unter anderem private und öffentliche Eigner, Kommunalbehörden, Architekten, Ingenieure, Bauunternehmer, Spezialisten für Baustoffe und Universitäten. Thema der Veranstaltung waren in erster Linie Brücken und Gebäude, und zu den behandelten Sachgebieten zählten unter anderem Design-Konzepte, Detailausarbeitung, Lasten, Leistungsanforderungen, Modellkonzepte für die Ausarbeitung von Vorhersagen, Spezifikationen von Baustoffen, handwerklich-technische Ausführung und Instandhaltung. Seismisches Design und auf langfristige Nutzung ausgelegte Baulichkeiten bildeten den Gegenstand weiterer vorgelegter Papiere. Zudem befaßten sich Beiträge damit, wie verwandte Industriezweige (z.B. Off-Shore-, Atom- und Flugzeugindustrie und der mit der Seefahrt befaßte Sektor) die Problematik der Nutzungsdauer von Baulichkeiten zu lösen versuchen. Funf der insgesamt 33 geladenen Delegierten kamen aus Europa und Nordamerika.

Im Rahmen von vier Zusammenkünften wurden 29 Referate gehalten und detaillierte Zusammenfassungen der weitreichenden Diskussionen, die sich an die Presentation jedes Papiers anschlossen, wurden ausgearbeitet. Das vorliegende Buch enthält sowohl den Wortlaut der Vorträge als auch die Diskussionsberichte. Die jeweiligen Zusammenfassungen führen die wichtigsten, aus dem Kolloquium resultierenden Schlußfolgerungen auf und weisen den künftig einzuschlagenden Weg bei der Entwicklung eines strukturierten Ansatzes, um bessere In-Service Leistung zu realisieren.

G.S.

Preface

Il a été observé un intérêt croîssant en ce qui concerne la durée de service des ouvrages en raison d'une perception de l'insuffisance des performances des ouvrages construits su cours, environ, des 40 dernières années. En gardant ceci à l'esprit, le Groupe britannique d'IABSE a organisé un colloque en juillet 1990 au Pembroke College, Cambridge, R.U., lequel constituait de fait un atelier de travail portant sur la faisabilité et l'application pratique des principes de durée d'étude des ouvrages.

Tous les secteurs de l'industrie de la construction étaient représentés, et parmi ceux-ci citons les propriétaires des secteurs public et privé, les collectivités locales, les architectes, les ingénieurs, les entrepreneurs, les spécialistes des matériaux et les établissements universitaires. La portée a principalement intéressé les ponts et les bâtiments et les sujets traités ont incorporé les concepts d'étude, les études de détail, les charges, les impératifs en matière de performances, la modélisation prédictive, les cahiers des charges des matériaux, l'exécution et la maintenance. D'autres communications ont porté sur les études sismiques et les ouvrages de longue durée. Il a par ailleurs été présenté des communications concernant la manière dont les industries connexes (par exemple les industries de l'offshore, marine, nucléaire et aéronautique) prennent en charge la durée d'étude. Parmi les 33 congressistes invités, cinq d'entre eux venaient d'Europe et d'Amérique du Nord.

Il a été présenté au total 29 communications invitées lors des quatre sessions et des résumés complets des discussions de large portée ayant eu lieu dans chacune d'elles ont été préparés. Cet ouvrage contient aussi bien les communications que des rapports ainsi qu'une récapitulation finale, exposant les principales conclusions tirées du colloque et indiquant la voie è suivre en matière de mise au point d'une optique structurée dans le but d'obtenir de meilleures performances en service.

G.S.

Prefacio

Se ha venido mostrando un creciente interés en la duration de estructuras en servicio, debido a la aparente falta de rendimiento de estructuras construidas durante los últimes 40 an~os, aproximadamente. Ello hizo que, en julio de 1990, el grupo británico IABSE organizara un coloquio, que tuvo lugar en el Pembroke College, Cambridge, Reino Unido. Dicho coloquio fue, en realidad, un seminario sobre la viabilidad y aplicación práctica de los principios de la duración del disen~o a las estructuras.

Asistieron al mismo representantes de los distintos sectores de la construcción, incluyendo propietarios de los sectores público y privado, autoridades locales, arquitectos, ingenieros, contratistas, especialistas en materiales y universidades. El seminario trató principalmente de puentes y edificios, habiéndose estudiado materias tales como conceptos de disen~o, detalles, cargas, requisites de rendimiento, modelado predictivo, especificaciones de materiales, mano de obra y mantenimiento. En otras ponencias se habló de disen~o sísmico y duración prolongada de estructuras. Además, se recibieron comunicaciones sobre la manera con que se hacía frente a la duración del disen~o en industrias afines, tales como la prospección petrolífera, marítima, nuclear y de aviation. De los 33 delegados invitados, cinco procedian de Europa y Norteamérica.

En total, se presentaron 29 informes invitados, distribuidos en cuatro sesiones, y amplios resúmenes sobre las serias discusiones que tuvieron lugar en cada una de ellas fueron preparado. El presente volumen contiene las ponencias y los informes, junto con un resumen final, en el que se presentan las principales conclusiones del coloquio y se apunta la tónica a seguir en el desarrollo de un enfoque estructurado, que permita proporcionar un mejor rendimiento en servicio.

G.S.

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Introduction

In July each year, the British Group of IABSE holds a colloquium at Pembroke College, Cambridge, which is named after Dr William Henderson, the first president of the Group.

In 1990, the event took place on 16–18 July, and the topic under discussion was The Design Life of Structures'. The programme was put together by an Organizing Committee consisting of:

G.Somerville (British Cement Association)	Chairman
D.A.Holland (Department of Transport)	
D.W.Quinion (Chairman, British Group, IABSE)	
K.Sriskandan (Mott MacDonald Group)	
R.J.W.Milne (Institution of Structural Engineers)	Secretary

The first four named individuals also acted as Chairmen and Reporters to the various Sessions.

The nature of the colloquium is unique, in that attendance is by invitation only. Approximately 30 delegates (some of whom must come from overseas) are expected to contribute a Paper. Each Paper is taken as read, with only a short introduction by the author, and considerable importance is attached to the lengthy discussion periods in each Session. In this volume, each discussion has been summarized by a Reporter and is presented at the end of each of the four Sessions.

In recent years, there has been growing interest in the service life of structures, due to a perceived lack of performance in structures built in the last 40 years or so. It has been suggested that there should be a shift in emphasis in design, with more attention given to 'performance in service life'—possibly based on design life concepts. However, the design life approach is still at an early stage of development, and there is not universal agreement on how it should be applied—or even if it is necessary to do so—to produce structures with improved in-service performance.

The purpose of this colloquium was to take a good hard look at the application of design life concepts to structures. With that in mind, the organizing committee invited representatives from all sectors of the construction industry, including owners, designers, contractors, local authorities, universities and research and testing organisations—and including a mixture of disciplines (architects, engineers, quantity surveyors, material specialists, etc.). It was further decided to arrange the colloquium in four Sessions, having the following themes:

(1) Overview of service life prediction
(2) How particular industries cope with design life
(3) The present state-of-the-art
(4) What happens next—the way ahead.

In addition to inviting attendees to contribute a paper on a relevant topic within one of the four themes, the organizing committee identified six key questions which they wished to be addressed during the colloquium; hopefully, following discussion and debate, consensus would be reached on at least some of these key issues. The six questions were:

(1) What do we understand by the term/concept of design life?
(2) Can we/should we, consciously design, detail and build structures for a specified (albeit 'notional') design life?
(3) How do we develop the concept in practical terms? What should the design life be for different categories of structure or component?
(4) Do we have the knowledge, and the design and construction techniques, to tackle this at this time?
(5) What are the real factors involved—practical and technical? What are the financial implications? What are the advantages—why should we do it?
(6) Do we really need to develop these concepts at all in order to do 'better', or does the answer lie in improving existing methods, developing greater awareness, stepping up training and making a greater commitment to quality?

Finally, I would like to record my grateful thanks to all the participants and to the hard-working members of the Organizing Committee, who contributed so much to what was a very successful event.

G.S.

Part A: Overview of service life prediction

Design life and the new Code

J.STILLMAN

The points made in this paper are drawn largely from work done in revising the 1950 Code of Practice on Durability (UK) and I trust they will serve to introduce many of the topics associated with this subject.

The new draft attempts to assemble the many simple ideas about durability in an ordered manner to provide a framework within which clients, designers, manufacturers and contractors can discuss and agree policies. Comments have been received and are now being dealt with, and I am hopeful that a final draft will be achieved before the end of the year. It will now be in two parts, the first a Code containing firm recommendations, and the second a Guide with supporting information: I will refer to it simply as 'the Code'. It was finally decided to publish the whole document as a British Standard Guide (May 1991).

In the draft the Committee had the help of an excellent consultant Mr Sylvester Bone RIBA, who has made a major contribution to the project. It covers building and structural engineering but not roads and bridges.

The need for a new initiative

My task now is to persuade you as to the value of the durability concept, and to ask you as engineers and architects to add yet another condition to be satisfied in design.

The 1950 Code was well conceived but ahead of its time and not widely adopted. In my opinion we now have enough data to allow the concept of design life to be made part of general practice. One must be careful not to make it sound too easy: my experience has been that the necessary technical and research information comes in too many forms from too many sources. The bibliography for instance lists 103 publications which must be obtained from some 25 different places. Keeping up with the state-of-the-art is therefore difficult. It is also too expensive, especially for the many small architectural practices. Structural engineers are perhaps in a better position: it seems to me that they are already more conscious of durability and they have the advantage of working with a more limited palette of materials.

Many papers to this and other conferences are concerned with how technically to achieve a more assured or longer life for particular materials or components. There is a current demand for better quality in all products and EC product standards will include a reference to an economically reasonable life. BSI have produced a Standard on Reliability covering a wide field, BS 5760, where reliability is another word for durability.

Philosophically, in predicting service life the assumption must be that buildings are designed according to advised good practice, and that materials and workmanship are inspected and up to standard. In this case deterioration depends on external and internal environments, including interstitial condensation and wear, with rates of deterioration estimated by reference to experience or tests.

Unfortunately from time to time premature failures hit the headlines, where the details of design, quality of materials and workmanship have not been good enough. It is impossible to ignore this problem. The Code therefore includes a section describing some common causes of failure in recent years. There is an item for instance about the dangers of carbonation of exposed *in situ* reinforced concrete and another on the galvanized coating of steel.

Definitions

Building life means different things to different people at different times. Definitions are therefore very important and I will use a number to discuss the roles of the parties concerned.

DURABILITY: Ability of the building and its parts to retain their performance under the effect of agents over a given period

SERVICE LIFE: Actual period during which no unacceptable expenditure on maintenance or repair is required

REQUIRED SERVICE LIFE: Specified period for the service life, e.g. as stated in the client's brief for a project

Responsibilities

It is clear that the client has to endorse a requirement for a particular service life as part of the brief, and no doubt this will normally follow discussions with the designer.

Are we asking the designer to put yet another rope round his (or her) neck? I do not think so: the Code makes it clear that predicting durability is not an exact science and figures arrived at for the predicted life of a building and its parts will often be no more than an informed guess.

Even if it is no more than an approximation, and qualified by the physical conditions of the project, I believe that it can form the basis for an understanding which is helpful to the designer in enabling adequate finances to be sought to meet the objectives, and helpful to the client in reducing the risks of

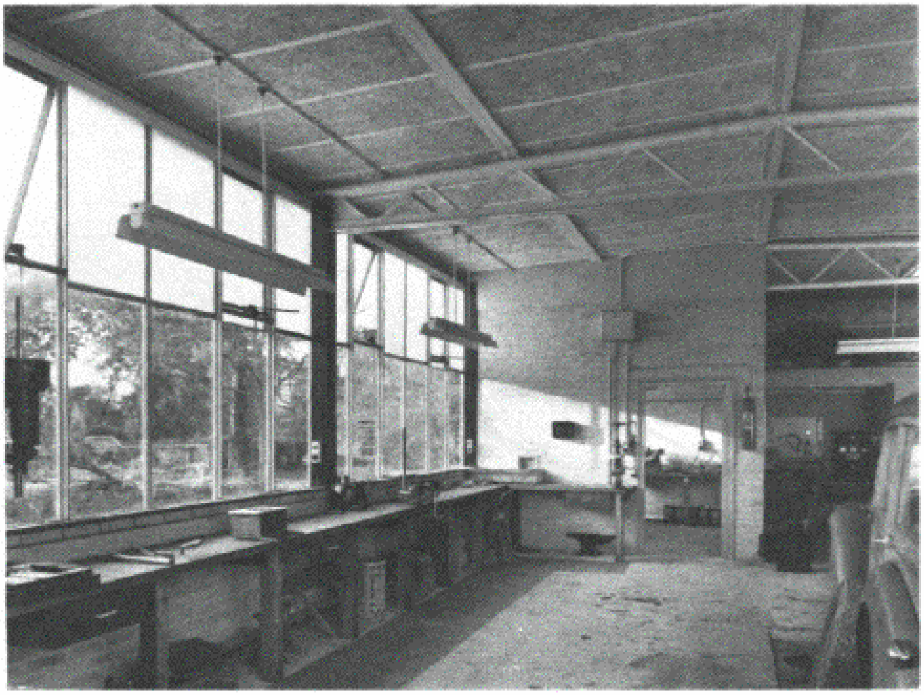

Figure 1 Have the materials stood the test of time? The architects had no reply when asked this question by their clients when this farm workshop was being built in 1955. They had specified some of the promising materials of the post-war period: short bored piles, flint lime bricks, light steel trusses, woodwool decking, three layers roof felt and aluminium patent glazing with some laminated insulating glass. Fortunately it has survived 35 years with only the roof felt requiring replacement in 1989 and it is still in use as a workshop. Nowadays the durability question should be settled at the briefing stage. Architects: Stillman and Eastwick-Field.

disappointment where for instance components may prove to have shorter lives than had been anticipated. It must be accepted that the conditions may not be static and one wonders what will be the effect of the threatened climatic changes of which we seem already to have foretastes, and the growing problems of atmospheric pollution.

Tables

The client is recommended to select from a table, summarized as follows:

Categories of required life

1. Temporary	As agreed, period up to 10 years	Site huts, exhibition buildings
2. Short life	Minimum period 10 years	Temporary classrooms
3. Medium life	Minimum period 30 years	Industrial buildings, housing refurbishment
4. Normal life	Minimum period 60 years	Most public sector buildings
5. Long life	Minimum period 120 years	Civic and other high quality buildings

Design life: period of use intended by the designer

This then is the designer's response to the client's required service life. It must develop into a more detailed statement covering in addition to the basic structure, materials and components subject to periodic replacement, repair or maintenance.

Components should be classified		Examples
1. Replaceable	Will not last the life of the building	Floor finishes and M&E components
2. Maintainable	Will last with treatment the life of the building	Doors and windows
3. Life long	Will last the life of the building	Precast cladding panels

Further definition of maintenance

1. Repairs only	As required	Broken windows, cracked slates
2. Scheduled maintenance and repairs	At regular intervals as BS 8210	5 yearly repainting of joinery
3. Condition based maintenance and repairs	To follow regular inspections as BS 8210	Contract maintenance of lifts or quinquennial inspection of churches

Design life data sheet

To assist the designer in keeping track of all these factors the format of a 'design life data sheet' is provided. Following a note on the basic information and environmental data, the building elements are listed with columns to enter categories of components and maintenance levels.

Example of design life data sheet entry

Data: Building Cat 4 (normal life at least 60 years) to be built on exposed site in East Anglia

External finishes:		
Roof membranes	Material	mastic asphalt
	Component category	Cat 1 Replace after 25 years
	Failure mode	F: no exceptional problems

Is this extra effort really necessary?

There are those who hold that correctly designed and constructed buildings, which are properly maintained, will last indefinitely. Experience shows that this is not so. Modern buildings rely heavily on hard materials needing copious movement joints with mastic or plastic seals. Steel and aluminium need protective coatings of limited life to resist corrosion and there is an increasing content of plastic materials with no proven track record.

The use of new materials joined together in new ways, and periodic changes in design objectives for example for energy conservation, introduce new risks. More persuasive argument is provide by Dr William Allen in his paper 'Root causes of degradation' to a conference on the same subject at the Institution of Civil Engineers in 1984.

Design life prediction

Armed with the data sheet statements the designer has the difficult task of specifying suitable forms of construction and components. As well as the particular conditions to which the building will be subjected, design life must take into account expected standards of workmanship and future maintenance.

PREDICTED SERVICE LIFE: Period predicted for the service life, from experience or test.

Much of the information required by the designer must come from manufacturers. Sometimes this can be supported by guarantees or warranties. For new materials Agrément Board certificates are a valuable source. BRE and other research bodies make test information available, and there is a general body of knowledge derived from experience when using traditional materials. In each case the predicted life of a material or component and the jointing involved must be set against the agents that are likely to cause its deterioration. In many cases it is a combination of two or more agents that are the most dangerous; for instance wind and rain, wetting and freezing, or dampness and fungus. The new Code sets out a list of all such agents and gives reference information on the effects of the most important.

To help designers make useful comparisons, manufacturers are asked to make statements in the form of predicted life for a period of years or range of years in specified conditions, pointing out any known dangers, e.g. the need to use special quality bricks in parapets, or the need for correct priming before making mastic seals.

For the future

Is it possible to improve on the broad brush methods described above? The Code describes an American 'decision tree procedure' reproduced from ASTM E6 32 which envisages accelerated ageing tests to be compared with degrading in practice, leading to mathematical models and predictions of service life. No doubt such procedures are used in research institutes here.

Mention is also made of 'criticality and hazard evaluation techniques' which are used in other industries and discussed in BS 5760. Derived from these, it is recommended that components should be categorized according to the anticipated mode of failure, a procedure which may show up cases where small defects could have disproportionate consequences, for example brick slips falling from tower blocks.

Classification of failure modes

A	Danger to life
B	Danger to health
C	Costly repair
D	Costly because repeated
E	Interruption of building use
F	No exceptional problems

Life cycle costing

In the long run no doubt decisions should be made with knowledge of costs. Is it better to cover ones flat roof with cheaper membranes to be replaced every 15 years or more expensive asphalt every 25 years? I am glad to see that papers on cost in use are to follow.

For the present, predictions of design life must rely largely on the designer's judgement, taking into account what data are available and the context of the project. Nevertheless by bringing the subject of durability into open discussion I believe there will be improvements both in mutual understanding, and in building performance.

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The client's view—the public sector

H.P.WEBBER

Introduction

There are probably relatively few cases where a client specifies an unequivocal design life for construction work which he commissions. This may simply be lack of sophistication, but more likely a commercial developer may plan to sell the building at an early stage, or in the absence of reliable data on service life and maintenance costs of various forms of construction, a client may take the easy option of accepting contemporary norms of quality. The latter may also apply in the public sector although for some types of public sector facilities a probable service life can be foreseen.

Not all facilities are designed with the intention that they should have a long life. Special industrial or operational facilities often have a known period for use and, in city centres, rapidly increasing values sometimes make demolition and replacement economic after quite short lifetimes. However, such cases are the exception, and for most of the UK building stock, through-life operation and maintenance costs are important factors in determining the economic lifespan.

At the other extreme, the exceptional cases of modern buildings of such note or aesthetic quality that preservation is justified beyond an economic period are, of course, a prominent but tiny minority. However for the central government client this may not be so and a significant proportion of buildings may be expected to remain in use far longer than typical commercial, industrial or residential property. This is not to claim that public sector architecture has a record of particular aesthetic success, although there is a steady flow of well regarded work. Rather, it recognizes that public buildings often have semi-permanent institutional functions and a prominence that conditions the opinions of generations who grow up accepting a building's familiarity. There is also the well known tendency for contemporary work to be reviled but loved and preserved by later generations. A good example of this is the Palace of Westminster which received heavy public criticism at the time it was built.

Clients and their requirements

When considering design life the following types of client may be distinguished:

(a) A commercial developer intending to sell at an early stage.
(b) A property investment institution usually letting on full repairing leases.
(c) An owner occupier or a public sector body building non-specialist premises such as offices which could be sold in the market.
(d) A public sector or industrial organisation building special purpose facilities which would not find a ready market.

The owner occupier and public sector bodies at (c) share with the investment institution at (b) an interest in owning premises which are robust and kept in good condition to sustain their value so that assets may be realized at any time. Although (b) will not carry normal maintenance costs, he retains an interest in sound construction, not least because tenants may disclaim maintenance liability if latent defects are discovered.

The commercial developer at (a) is likely to have a clear requirement for design down to a price, as may some of the public sector or industrial organizations at (d), but this category will also contain public institutional buildings with long expected lifetimes where additional money spent achieving durability can be shown to be cost effective.

Both the property investor and the public body, at (b) and (c), respectively, when dealing with office buildings will probably make investment decisions on the basis of 60-year leasing or occupation, whereas for industrial and commercial premises of the out-of-town portal frame type, 25 years is common. There are also now many cases where investment institutions have chosen to reclad and extensively refurbish office buildings after only about 30 years in order to retain a quality image for the property and obtain premium rents.

The range of the design life problem

The range of useful life expectations for buildings of different types, or the same type for different clients, is clearly very wide. Table 1 is therefore a very simplified summary.

What is design life?

The question must be asked, whether it is practicable to design a building or its structure for a given life. In general the designer will have in mind a useful life, subject to reasonable levels of maintenance and he will recognize that no

Table 1. Some examples of typical useful lifespans

	Lifespans (years)
Temporary buildings, timber huts, portable cabins etc.	1–10
Airfield pavements	15–20
Operational military facilities related to service life of a weapon system	20
Maritime structures	40
General office industrial and residential buildings	60
(but heating/air conditioning installations)	15–20
(and cladding)	30–60
Prisons and court buildings	100
Heavy civil engineering structures, bridges, etc	120
National institutions and monuments	up to 200
The British Library	250

structure is expected to be entirely maintenance free during its design life. Materials are expected to deteriorate from weathering, wear and tear or fatigue, but appropriate maintenance should enable the required life to be achieved, as indicated in Figure 1. Broadly speaking, most types of deterioration progress steadily with the passage of time and it is usually not possible to define a point at which a facility ceases to be serviceable. There are, however, some causes which could render a facility suddenly unviable or which could present the threshold for much more rapid deterioration. Examples are:

(a) *Serviceability thresholds*
- Statutory obligations
- Health and safety matters
- Primary operational or functional requirements

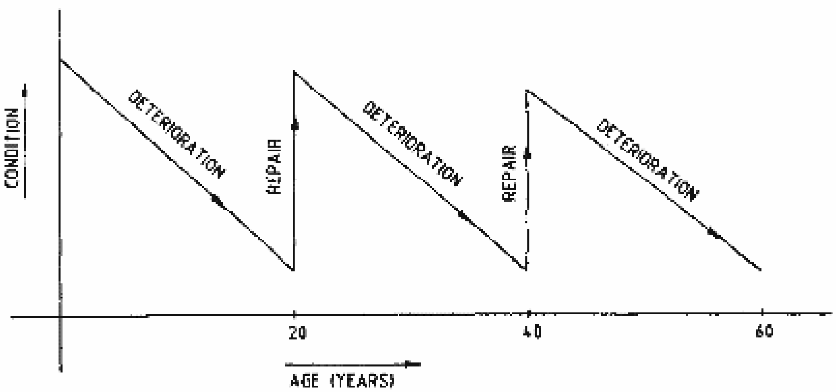

Figure 1 Deterioration and maintenance over time. After Browne.

(b) *Thresholds of rapid deterioration*
- Concrete reinforcement corrosion as a result of carbonation, etc.
- Failure of weather envelope
- Weathering and embrittlement of bituminous based materials, plastics, etc.
- Fatigue life

Taking the example of an office building with a design life of 60 years, the designer will recognize that fundamental parts of the building such as the structure and, usually, the cladding must have the potential to last the full period. Other items, such as M&E installations will be expected to need replacement every 15 or 20 years and superficial finishes and fittings may need frequent refurbishment.

When designing for the public sector client who builds with the intention of owning and operating the facilities throughout its design life, total through-life costs can be taken into account when choosing and specifying materials.

When selecting products three main possibilities occur:

(a) Information is available for a clear and informed choice to be made between cheap short life products or more expensive more durable ones, allowing the designer to choose how many times the items will need replacement in the building's life. e.g. a flat felted roof or a pitched tiled roof.
(b) For some products the available norm will have a well known maximum life and the designer must provide for expected routine maintenance or replacement, e.g. boiler plant.
(c) Many proprietory products are sold with little data on long term performance and the designer must select with prudence, bearing in mind the mixed record of success which novel proprietory products have had in construction.

PSA has published Cost in Use Elemental Tables which provide data concerning capital costs, cleaning costs, cyclical maintenance and renewal costs and the probable costs of remedying defects in design and construction. The cyclical maintenance and renewal costs given in the tables include all maintenance of a periodic nature which could be anticipated by the designer when choosing a component. The rates and life expectancies are based upon recommendations made by manufacturers and building professionals and they are related to normal ageing processes.

Investment appraisal

Structured investment appraisal including discounted cash flow analysis provides the framework within which through-life costs can be examined to facilitate correct investment decisions and permit higher initial standards where these are justified.

A formal investment appraisal will set out the objectives and proposed expenditure with the alternative ways of meeting them. The cost benefits of the alternatives are then compared. An appraisal will follow a sequence of basic steps:

(a) Define the objectives
(b) Consider the options
(c) Identify the costs and benefits of each option
(d) Discount the costs and benefits which can be valued in money terms to bring them to their net present value so that they may be compared on a common basis
(e) Weigh up any uncertainties in connection with the estimated costs
(f) Note any facts which may affect the selection of options (such as the building user's preferences)
(g) Select the best options

This approach which is common for all types of public sector investments contrasts with that of a developer, expecting to sell the completed building or let on a full repairing lease, who will look for low initial capital costs with little consideration for maintenance costs thereafter.

Feedback

For many years PSA has operated a system of feedback from its maintenance, design and site control staff to identify common defects and maintenance problems. The people responsible for the PSA General Specification have taken the feedback reports into account in making judgements on the cost effectiveness of amended or strengthened specification requirements, including mandatory quality assurance for some products and services. The result is a published library of specification clauses for building and civil engineering with an explicit policy of conservatism to achieve minimum through-life cost by balancing initial quality and long term maintenance. Some aspects of that experience are described briefly below.

Concrete deterioration

The PSA feedback system has identified reinforcement corrosion caused by concrete carbonation as a widespread problem with a high annual maintenance cost which can be expected to continue. Corrosion from chlorides in the maritime environment, road salt and calcium chloride is also significant but on the Government estate, sulphate attack and alkali silica reaction are not significant.

Carbonation The neutralization of concrete alkalinity with consequent steel depacification, as a result of the permeation of atmospheric carbon dioxide, penetrates below the concrete surface according to the following relationship:

$$\text{Depth of carbonation} \propto \sqrt{\text{time}}$$

Thus if concrete cover is doubled, reinforcement corrosion is delayed by 4 times. On the other hand reinforcement fixing errors which reduce cover can seriously reduce a structure's life. Figure 2 illustrates the rate of carbonation.

Table 2 compares the minimum cover requirements of BS 8110/85 with those of draft Eurocode No. 2. The figures for minimum cover given in the table under BS 8110 are the nominal covers specified in the standard, less the 5 mm tolerance which BS 8110 permits for bars up to 12 mm.

Draft Eurocode covers tend to be less than in BS 8110. As an example, the Eurocode would permit 20 mm cover in humid conditions where for grade C30 concrete, carbonation could reach the reinforcement in some 30 years. However,

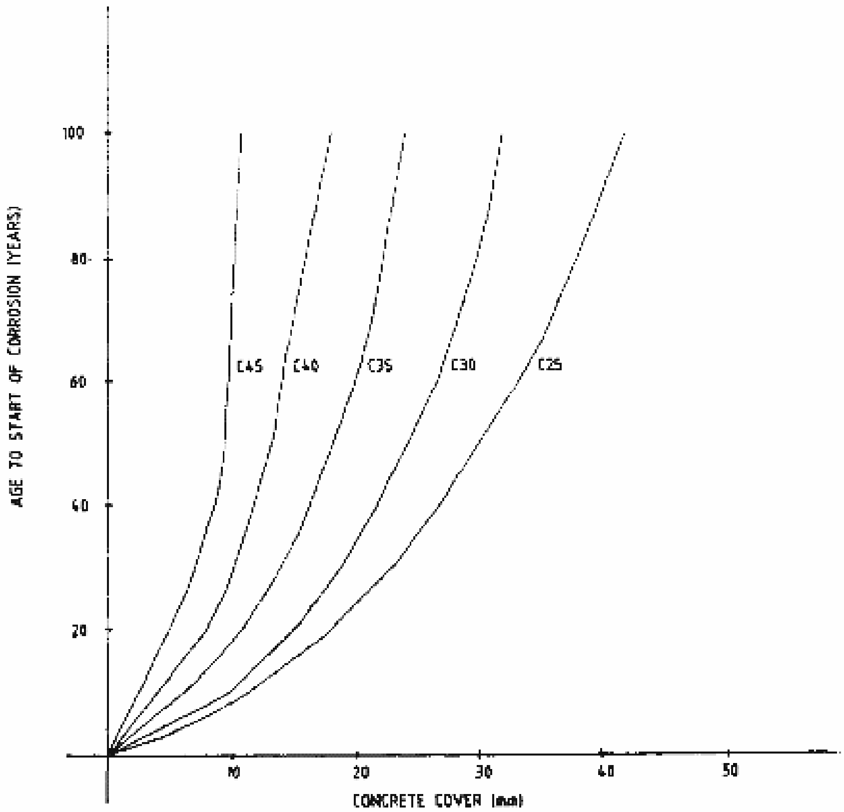

Figure 2 Carbonation: time to corrosion start.

Table 2 Minimum cover to reinforcement: BS 8110 and Eurocode 2 compared

Draft Eurocode 2: design of concrete structures		BS 8110/85: structural use of concrete				
Exposure class	Minimum reinforcement cover (mm)	Conditions of exposure	Minimum cover (mm) (= nominal cover − 5 mm)			
			C30	C35	C40	C45
1 Dry	15	Mild	20	15	10/15	10/15
2a Humid without frost	20	Moderate		30	25	20
2b Humid with frost	25	Severe			35	25
3 Humid with frost and de-icing salts	40	Very severe			45	35
4a Seawater without frost	40	Very severe			45	35
4b Seawater with frost	40	Very severe			45	35

Eurocode No. 2 does also require the minimum concrete cover to be determined with regard to exposure conditions and concrete quality to ensure a continuing alkaline environment surrounds the reinforcement, although it provides no guidance on achieving this. The Eurocode covers are 'boxed' and can therefore be modified in the British standard.

The marginal cost of buying grade C40 concrete rather than C30 is only £2–3 per m^3 and it is for consideration whether the higher grade concrete should be specified for durability as a matter of routine.

EC construction products directive: durability

It is of interest to note that the draft interpretative document on mechanical resistance and stability under the Construction Products Directive requires the following:

- Product standards and technical approvals should indicate a recommended working life for the product
- Products should have a working life at least equal to the working life of the project or they should be replaceable, by procedures planned in the design stage and taking account of cost effectiveness
- Works should be designed for an intended working life
- Design codes shall indicate minimum required durability characteristics for a range of working lives

Stainless steel reinforcement

As shown in Table 1, the British Library which is now under construction near St. Pancras, is an example of a public institutional building with a very long expected life. Durability of all parts including the structural concrete has therefore received careful attention. Concrete mixes have been designed with a view to durability under carbonation and sulphate attack and thin concrete sections such as precast units, lintels, etc., have stainless steel reinforcement.

Two types of austenitic stainless steel bars for concrete reinforcement are described in BS 6744. Type 304 S31 has good corrosion resistance in most situations but type 316 S33 contains molybdenum and is more suitable where chloride levels are high. The cost of type 304 and type 316 are, respectively, 7 and 9 times the cost of carbon steel. Stainless steel is therefore uneconomic for use in normal reinforced concrete situations but may be required in exceptional circumstances where good corrosion resistance is vital.

Cladding panels in high rise buildings can be difficult to repair and falling pieces of spalled concrete present a safety hazard. The code of practice for cladding panels, CP297, specifies type 316 for fixings and lifting hooks. To prevent problems which could occur from dissimilar metals, type 316 should also be used for reinforcement in such cases.

Maritime structures

Concrete

Mass concrete gives excellent service in the maritime environment if dense and impermeable. To achieve this, aggregate must be hard, clean, well graded and well shaped to achieve a good degree of workability with a water/cement ratio of 0.42 or less. PSA generally specifies cement content of 400 kg/m^3 for concrete exposed to the maritime atmosphere or splash zone, and 350 kg/m^3 for concrete permanently under water.

Reinforced concrete has a life unlikely to exceed 40 years in maritime structures, notwithstanding the recommended minimum cover of 75 mm to all steel.

These recommendations are significantly more conservative than the corresponding figures in Eurocode No. 2 and the draft of ENV 206, the European pre-standard for concrete, which, respectively, require 40 mm cover and 300 kg/m^3 of cement with a water/cement ratio of 0.50.

Structural steel

The first measures against corrosion of structural steel in the maritime environment can be taken at the design stage by avoiding recesses or pockets where water and debris can lodge and, as far as possible, the steel should be accessible on all sides for inspection and treatment against corrosion.

A cost effective solution for steel in maritime works is to design to allow for loss of section due to corrosion, but coatings such as coal tar epoxies can be used and may be applied above mid-tide level to delay corrosion. When applied to new works they will delay corrosion for the first 10 years of a structure's life.

For *in situ* work below mid-tide level protective coatings such as tapes and other proprietory systems are available for underwater application. Cathodic protection is not a first choice as it gives no visible or audible sign that it is out of action and frequently during biannual inspections it has been found to have been switched off for safety reasons when explosives or fuel were being moved and not switched on again.

Timber

Timber in maritime engineering is used mainly for fenders and groins, but tropical hardwoods are now only specified where they are essential, and then only from renewable sources. Generally hardwoods are required because of their resistance to marine borers and fungal decay; also in applications such as fender piles, the structural properties of the timber are important and good flexural strength is needed together with a high degree of impact resistance.

Greenheart has a life of 40 years or more in exposed maritime situations and timber structures are relatively simple to repair. Many 19th-century stone maritime structures were founded on timber piles and in such situations the timber is commonly found still in good condition.

Cast iron

Cast iron was used extensively for maritime structures in the 19th century but it is now generally near the end of its useful life after about 100 years.

Stone

Subject to adequate pointing maintenance, stone has a very good life expectancy in maritime structures. It is aesthetically attractive and gives excellent service in dock walls, revetments, breakwaters, coastal defences, etc. There are good examples in the 19th-century granite and Portland stone basins and dry docks of the naval dockyards, all still in excellent condition. However, structural failure if it occurs in maritime masonry structures, can be sudden and catastrophic.

General

Some older maritime structures now suffer as a result of high berthing loads of modern ships and additional depth requirements which may lead to overdredging. Scour from modern propellers and bow thrusters may also be a problem.

Military structures

It is ironic that the massive hardened structures designed to shelter aircraft and personnel from attack have quite short design lives to match the weapons systems concerned, but by their very nature these heavy structures can be expected to survive for very many years. In practice, designers will recognize the relative continuity of key defence establishments and can expect that facilities of this sort will be refurbished and adapted for future generations of weapons systems. Nevertheless, cost planning will often be based on design lives of the order of 20 years.

Seismic risks

For structures such as the shiplifts, dry docks and jetties which will serve submarines with nuclear propulsion systems, all conceivable risks to structural safety which could occur during the design life must be taken into account. Thus the structures are designed for safety in the worst conceivable seismic events and checked against disproportionate failure under even more severe loading. This prudence, which of course reflects public concern for nuclear safety, contrasts with the statistical approach to design loading exemplified by wind loading of conventional structures which relates the magnitude of the design load to the probability of it occurring during the design life of the structure.

Airfield pavements

An airfield pavement can be a surprisingly complex structure to analyse as a result of the build-up different layers of construction over many years. Unlike a road designed for millions of repetitions of standard axles, an airfield pavement is designed for relatively few repetitions of very heavy loads and high tyre pressures. Failure may occur from overload, fatigue or, more commonly, from weathering of bituminous materials and joints. High standards of specification and site control are therefore adopted to maximize the life of bituminous airfield pavements which is typically 15 years for porous friction course and 20 years for Marshall asphalt. PS A has an active research and development programme looking at improved binders and other techniques to extend pavement life between resurfacing.

Conclusion

Public sector clients share many design life problems with the private sector, for example in conventional office or accommodation buildings, but the government civil and defence estates contain many examples of buildings with both short and exceptionally long operational expectations. The government estate is also unusual in that it provides an opportunity to build up information on the through-life costs of various types of construction and the cost effectiveness of improved initial specifications.

The client's view—the private sector

M.S.FLETCHER

Introduction

The assumption in the title is that the private sector has or needs a different approach to design life from the public sector.

The long term needs of the two sectors are basically the same; i.e. to provide a structure with a known capital cost, a known expenditure on maintenance during the expected life of the structure and an assessable loss of revenue from the revenue producing operations in or on the structure whilst the maintenance is being carried out.

The difference in approach between the two sectors is likely to be in the method of financing and the effect of this upon those purchasing the structure. However through the evaluation process and purchasing control and in spite of the diligence of public sector employees, current thinking of some economists is that one gets better focused decisions if the man who ultimately is responsible sees his firm's net asset value depressed, or loses his job or causes his firm to go into liquidation if his decision is incorrect.

For the purposes of this paper the part of the private sector under consideration is that which purchases and then occupies a structure for its own use using borrowed money which is to be repaid from the profits of the activity carried out on or within the structure.

Finance

The capital sum which is borrowed has to be repaid to an agreed repayment schedule which starts after completion of the structure or after an agreed grace period during which repayment is deferred. The commitment to repay is independent of the profits achieved.

A very good example of finance of the UK government private sector infrastructure projects is that adopted for the estuarial crossings such as Dartford Bridge on the Thames. A concession company has the task to finance, design, construct and operate the bridge for an agreed period. The concession company has 'pinpoint' equity, i.e. a very small amount of equity, but has the ability to attract investors such that the capital required can be borrowed. The lenders are firmly committed at the time when the concession company tenders, subject of

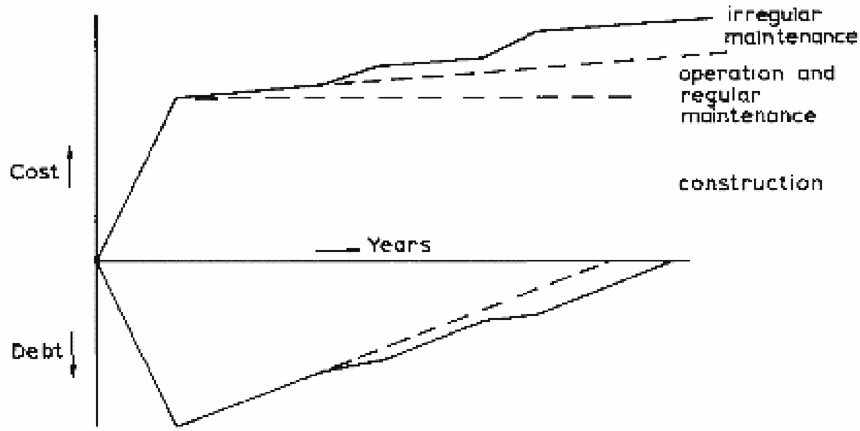

Figure 1 Debt profile in real terms.

course to the project not failing in its administrative and legislative procedures prior to the start of construction.

A typical graph of such a financial arrangement is shown in Figure 1. It represents the debt profile throughout the life of the loans. At tender stage the preparation of such a graph involves an assessment of income each month after the revenue producing activity is started, together with appropriate sensitivity testing of the assessment. In the case of an estuarial crossing, the assessment involves the amount of traffic and mix of vehicles at year of opening, the rate of growth, the rate of change of tolls, the possible change of traffic mix, the generation of new traffic and the influence of any identified developments, e.g. the intention to build another nearby crossing in the future.

The assessment of the amount and rate of expenditure involves three phases; preparation design and construction; management and operation; regular and irregular maintenance of the structure. The estimate of construction cost and the rate of expenditure is similar to a conventional construction tender. The estimate of management and operational cost during the concession period is similar to that in any other business enterprise involving manpower, equipment and materials. The estimate of regular and irregular maintenance is the contribution to the finance profile of the project which is influenced directly by the design life of the structure. The influence is in two ways, the cost of maintenance and the loss of revenue whilst the maintenance is being carried out. The uncertainty which arises in the minds of potential investors when an engineer introduces the need for irregular maintenance is considerable. It would be even greater if the investor had sufficient engineering knowledge to appreciate how sparse the available data are to make an assessment of this cost.

Life of the structure

The 'design' life of bridges in the United Kingdom, according to BS 5400, is 120 years. This is a typical period for the conventional type of bridges covered by the Standard. However, similar periods are adopted for unconventional structures, e.g. studies in Norway for submerged floating tube structures to carry highways have indicated a required lifetime of approximately 100 years and studies for a major suspension bridge over the Straits of Messina have concluded that the conventional service life of the structure would be 200 years.

The private sector initially is concerned with the 'financial' life of the structure, i.e. the period over which the borrowed money must be repaid. Typical periods estimated for Dartford and Severn Bridges are between 20 and 25 years and for the Skye Bridge currently being tendered for in Scotland, the maximum concession period is 30 years. For the proposed Straits of Messina bridge the concession period for financial evaluation is 40 years.

Both the financial life and the design life of a structure vary depending upon funding source, geographic location and type of activity to be carried out in or on the structure. The above figure for bridges should not be assumed for other types of structures which should be subject to individual assessment.

The difference between the financial life of a structure and the design life of the structure indicates that at the end of the financial life the structure has a substantial residual value. The amount of residual value is determined by:

- The quality of the design
- The quality of the materials and components employed
- The quality of the construction
- The incidence of environmental or applied loadings which were not adequately known or assessed at the time of design
- The quality of regular and irregular maintenance

In all cases the owner or concessionaire of a private funded project requires of his engineer, in addition to his normal design and construction duties, to provide a costed schedule of regular and irregular maintenance and a sensitivity upon that cost. He also requires the engineer to achieve a residual value of the structure after the finance life has ended which is appropriate to its use at that time.

Regular maintenance

For an estuarial crossing regular maintenance is a common concept, washing tunnel walls, cleaning gullies, repainting road markings, cleaning and replacing lighting lanterns, etc. It is reliable in its cost prediction and knowledge is available. It is significant how little of that knowledge is published.

Irregular maintenance

This is the aspect which worries the private sector. The trade press carries many stories of the need for substantial expenditure during the life of a project. Such events are news. Many people are aware of the substantial irregular maintenance carrried out on Severn Bridge but very few people are aware of the smaller amount carried out on the other west country suspension bridge at Tamar. There is a need for engineers to publicize their successes more. Papers are written at the end of construction; perhaps the institutions should insist that the author of a paper describing a structure writes two follow up papers 10 and 20 years later to describe the structure's performance in service. The private sector purchaser would find this very helpful. Is the engineering profession brave enough to do so?

Reduction of irregular maintenance involves fundamental design brief and specification decisions in the initial stage. Firstly the client has to specify any likely change of use either during the financial life or during the design life. One of the most dramatic examples of change of use is bridge live loading. The increase in the number of heavy goods vehicles in the traffic stream on UK roads has caused the need for much irregular maintenance. It is the client's task to predict change of use and to control change of use. If it cannot be predicted, there is little point in specifying long design life criteria, even more so if you do not add spare capacity for unknown increases in loading, either by an allowance initially or by designing at concept stage a feasible strengthening scheme for implementation when such an increase occurs. The private sector may be more receptive to such an approach than the public sector.

Irregular maintenance also depends upon the specification of components with a known service life. Planned replacement at predetermined intervals has not yet been developed for such items as bridge bearings, expansion joints and the stays of cable stayed bridges, although in the last 15 years, more emphasis has been placed on the initial design incorporating a planned method of making such a replacement. Components with longer service guarantees would be welcomed and would certainly be worth paying for.

Specification of major materials is also a problem, because the evaluation between initial cost and reduced maintenance or greater residual value is inadequately supported by data and in fact by inadequate engineering research into long term problems. A client wishing to purchase a tunnel reads that concrete segments in the Channel Tunnel have uncoated reinforcement and concrete segments in the Storebaelt tunnel have epoxy coated reinforcement. A private sector client wishes to be certain that the specification has taken into account service life, particularly the financial life, and that the implications of an error in this assessment are made clear to him. It must not be assumed that the private sector wishes to buy the cheapest product. It is frequently in its interest to buy the best available.

Workmanship

One of the difficulties in civil engineering is that the organizations who supply components are often not the people who install them in the structure. The achievement of planned design life is more influenced by successful workmanship during installation than any other aspect. The most secure way to obtain high quality workmanship is for the client to employ trained and experienced observers full time on the site who are independent of the financial pressures of the constructor. Many clients in the private sector do not accept this, some constructors try to persuade clients that it is not so; it remains however a safeguard clients should not ignore.

Innovation

The emphasis upon the design life tends to restrict the use of innovative techniques, designs and materials in construction. A private sector client wishes to place the responsibility for the use of new designs and materials onto this designer. This seems to be a correct approach. The requirements for the proposed Skye Bridge in Scotland are that the designer carries out the annual inspection of the bridge throughout its financial life and reports upon its condition. This is an excellent means of ensuring that those who introduce innovation are also responsible for the consequences.

Conclusion

The private sector wishes to ensure that the life of the structure and its components has had its financial implications fully assessed. Adequate data are often not available for the engineering profession to provide this assessment. It is the task of the designer to explain this to his client.

The private sector is not only interested in the cost of maintenance during the financial life. It has a considerable interest in the residual value. The Forth railway bridge, a private sector project, has been in service approximately 75 years after the end of its financial life. Its residual value was substantial.

The concepts of financial life and costs of maintenance need further awareness within the design professions and the construction industry. The private sector may be willing to pay a higher initial cost over the financial life of a structure if it ensures more certainty in the cost of irregular maintence and more certainty in the residual value.

Design life in practice

A.STEVENS

This review is intended to represent typical practice in the design of the life of building structures. But it is not based on extensive research. It has to be seen as a personal view, hopefully more or less characteristic.

More reservations about this topic note the separation in our thinking about structure in buildings, which are more often protected, and structure in bridges, which are almost always exposed. Moreover, it is helpful to make a further distinction between building structure on the one hand, and buildings as a whole, which also embrace building services, finishes and fittings on the other.

That having been said, let us examine the words 'Design Life in Practice' as they might apply to building structures. A number of expectations arise. 'Design' might import a measured choice between alternatives on the balance of advantage, taking account of conditions. 'Life' might signify quality or possibly length; and the word 'practice' might imply a body of experience from which one could learn as a basis for advance in the future.

However in reality, such expectations are not satisfied in everyday practice in the design of building structures. Design decisions, directed towards assuring the duration over which the structural framework will continue to perform satisfactorily, are based on a limited amount of simple data.

It is usual to consider the life of structure in buildings to procure that its duration should be normal. However in my experience there is little consensus about what 'normal' might mean. One interpretation could be that building structures will perform more or less as they have in the past, and as owners and lessess have come to expect. If pressed, most structural engineers might concede 'normal' to be a serviceable life to about 50 to 100, say, 75 years, given that there is some (unspecified) amount of maintenance and minor repair.

It is significant that the target service life of structural works is rarely specified. Equally rarely does the maintenance manual for the building prescribe a programme of inspection and routine maintenance for the structure. Since important sections cannot be inspected or maintained without major works, this is understandable. Consequently, it may be reasonably (but in fact, wrongly) implied that inspection and maintenance will not be required during 'normal' life.

Some variation to this approach may be found in buildings where extensive refurbishment can be expected on a regular cycle, say 25 years. In such cases, it may be possible to argue that internal dry-clad steelwork needs no corrosion protection, where its condition can be regularly if infrequently reviewed, and, if necessary, remedial measures taken.

There are sometimes special cases for which a much longer life is sought. There may be buildings in which the contents or the operation accommodated is so valuable, that disruption for maintenance repair or even relocation, would be unacceptably expensive. On the rare occasion longevity is required then the target duration changes from normal to eternity, such being the accuracy of the design process.

Infrequently, structures with otherwise desirable attributes have a service life much shorter than 'normal'. Architectural fabrics in lightweight structures are an example. Those instances are recognized by users to be unusual, and so their replacement several times during the life of the building becomes acceptable. Such acceptability could only apply to an element that was economically replaceable.

Possibly the most difficult decisions about useful life arise during the refurbishment of existing structures. Usually the age of the building for refurbishment will have exceeded 'normal' life expectancy. Much of the fabric will be a clear case for replacement or at least specialist preservation. But the framework and foundations frequently present the designers with difficult problems.

Normally the structure of the building appears in good shape. Refurbishment would not otherwise have been proposed. Where discovery proves otherwise, repair and replacement are a purely technical problem.

It is where it is necessary to predict the remaining life of masonry, concrete or reinforcement, whose condition can only be quantified by tests and investigations carried out on samples taken from the whole, that uncertainty arises. Whereas engineers are happy to rely on a statistically small sample in new buildings, no such confidence can be placed on less comprehensive statistics for a unique existing structure, for which there will be no comforting precedence.

Thus although there will exist the results of a full-scale long term test of durability in precisely applicable environmental conditions, apparently available for assessment of further serviceable life, the designer has problems. He knows less about the properties and condition of the building materials than if he had specified them new. He is unable to find out about them comprehensively and reliably. Those uncertainties have to be balanced against the performance of the structure over its previous history, provided that conditions in the future do not differ markedly from the past and that any local deterioration in serviceability can be effectively remedied.

Nonetheless, it is the very enlightened and experienced owner who perceives the structure of his building as a declining asset. Most, not having consulted their advisers, would see the structure lasting forever. It comes as an alarming revelation to find that it is possible, though happily not probable, for the framework of their building to deteriorate to the point where it could be no longer fit for use, and where costly repairs are unavoidable.

That is not a criticism. Where those responsible for buildings are not fully informed about them, it is usually because they have not found it necessary to be. In common with the rest of us, those who run buildings base their approach on their past experience and what they know of others in like situations.

By the same token, it is rare that designers will recommend or developers will demand that the options for the durability and quality of building structures should be reviewed and measured decisions taken, on the basis of comparisons of cost, or, better still, value in use.

As we have seen, the approach to the design of the duration of structures for buildings is simplistic compared, for example, with procedures that are normal in decisions about mechanical and electrical services. Building services are seen and accepted to be of limited life, consumable. Consequently capital/operating costs are of interest both to potential owners and lessees and therefore developers. Thus investigation of the life and cost in use of building services arise from market demand. Not so for the structural framework. The deterioration of structural framework to the point where repairs or maintenance are essential to preserve life expectancy, is seen by those responsible for funding building works as the exception. They are often perceived to be the results of poor decisions at the design stage, and indeed, sometimes are so.

Why should this be? One reason is that the experience of the consumer is often confined to few instances, maybe just one. He may not know whether his experience is typical or unique, and may feel he can draw no general conclusions. Experience of new buildings of occupiers, and even owners, is commonly infrequent, perhaps once in a lifetime.

The phasing of decisions for the design of buildings tends to dilute attention from critical, longer term issues. Those involved in design, in development and construction of building structures are interested in capital cost and how that might be minimized. They are not moved by arguments that might speculate on the possibility for the reduction of costs of maintenance and repair in the future, if it would mean an increase in the capital cost, in the here and now. This will be particularly so where the prospects for return on capital, for example, an improvement in rent potential, might be very uncertain.

Nonetheless, the building will be unsaleable and therefore of little value where the term of life of its structural framework can be called in question. Expectation of useful life must be demonstrably 'normal'.

On the other hand, those with the responsibility for the operation, maintenance and repair of buildings, who consequently might be interested in operating and running costs, appear on the scene far too late when any decisions they might have wished to influence, even if they knew how, have long since been taken. In any case, experience of one isolated and possibly unique instance may not be sufficient to influence design decisions for building structures generally.

In short, the presence of interested, informed parties motivated to optimize the combined capital and operating costs for building structures and the occasion for appropriate design decisions, are not coincident. As a consequence there is little general market demand for design life advice about structures in buildings on the basis of standard of service and value.

However, if we are honest, that is not the only reason. Another explanation for the lack of demand, could be a lack of supply of a convincing product.

We have examined the simplistic approach to design life in building structures. Whereas it may not appear very scientific, it has a sound engineering basis. In circumstances where there is uncertainty over a wide range of choice, there is often security at the extremes.

For example, given the spectrum of material properties, methods and quality of construction, behaviour of the finished product, conditions in which it will operate and the variations in those conditions with time, then it is impractical to predict the design life for building elements with accuracy. In that light, the decision of the designer to make sure, on the basis of the best evidence available, that the useful life of the structure shall not be less than a certain and long period, is seen to be at least reasonable, if not wise.

We may suspect, on grounds of our experience in other design areas, that a simplistic approach may be more expensive than one more refined. But on the evidence available, we cannot be sure.

Seeking improvement in our predictive capacity, with increased knowledge, we must allow for the possibility that we may be faced with what, in the computer world, is known as combinatorial explosion. By that is meant that to cope with all the possible combinations of the variables in the equation could well overload resources.

However, until designers can produce convincing arguments in support of their ability to predict accurately, given the type of building and the conditions in which it is required to serve, then general interest in design life of building structures will remain at its present level.

Furthermore, were such capability available it would be necessary to erect compelling financial arguments on its back. Those that pay today will need to be persuaded about tomorrow's benefits, and who would enjoy them.

Were it to be found that no arguments for financial cost advantage could be sustained, research and development in pursuit of evidence would have been in vain.

If our potential clients have yet to become interested in such issues, there are at least two reasons for designers and particularly engineers, to be interested now. First, we ought to be seeking to advise, or at least becoming able to advise, on the real costs of the building, that is, capital and operating costs, so that total cost could be optimized. The reasoning behind this proposition is pragmatic. When advice is not comprehensive it is often in the long run, when the implications become apparent, unsatisfactory to the consumer, and therefore to his adviser.

Second, we would like to be more certain about the probable outcome of our technical decisions, particularly those about materials, for which we have design responsibilities and liabilities; particularly so where it could be to project advantage to propose elements whose service life is abnormal or subnormal, and where deviation from the norm could require greater, certainly more, evidence to convince. In short, it will always be in the designer's interest that his advice should be complete, and correct.

Understandably, the comfort and happiness of designers is not of great concern to the general public, let alone building owners. There have to be other and good reasons that might convince the man with the funds. Before investment in research and development can be forthcoming it will be necessary to convince that useful results are achievable and furthermore, that their application in building projects will be financially beneficial.

Our problem is seen to be lack of information, proven decision rules and a reliable design base. Since I know that designers do not have resources, individually, either to mount or coordinate the implied mammoth exercise for gathering information, if it is to happen it will need to be paid for by someone else. But who?

On the one hand we have those who develop and operate buildings. But without surety of outcome, the chances of support from this direction are remote. There is the Government, funding through research projects both in industry and the academic world. I believe we shall hear what success there has been in this field. But even so, will it be enough? Which brings us to the purpose of the Conference. What might we seek to achieve?

One important aim must be to consider our ability to cost the life-cycle of buildings generally, and the building structures in particular, accurately enough to make comparisons of options effective. That capacity for cost estimation would comprise:

(1) Making accurate and reliable estimates of the capital cost of a variety of types of building structure, each capable of satisfying function requirements, and operating conditions, and
(2) Making accurate and reliable estimates of the net present value of the costs of inspection, maintenance and repair of each option, during its useful life

What we seek are the data, statistics of building materials and their performance, that will inform the generation of a greater variety of alternatives for design life than is currently available, and that will afford the means to compare them reliably. When that should become possible it will remain to be seen whether or not there is financial advantage to be gained by choosing between the opportunities so afforded. What I am hopeful that the Conference can provide is an educated assessment of the prospects for that enterprise, as a first step in what appears to the uninitiated to be a long journey.

A performance approach to design

K.H.WHITE

Introduction

'Today's problems are a result of yesterday's decisions'—this was the theme of a recent application by Arup Research and Development, the Building Research Establishment and Strathclyde University for research funding under the SERC/DOE LINK Programme on Construction Maintenance and Refurbishment. In the context of this conference on Design Life of Structures it would perhaps be more appropriate to say 'today's decisions may avoid tomorrow's problems'.

This paper reviews the research we have carried out for BRE on the subject of Building Performance and Costs-in-Use and gives the main conclusions and recommendations that have resulted therefrom. Buildings of course include the supporting structure, and perhaps the work we have done can be applied to structures in the broader sense, whether they are of concrete, metal, masonry or timber, or a composite of any of these. The application is demonstrated using a hypothetical motorway bridge.

Performance of an item in the context of our research may be defined as the provision of one or more functions over a period of time, subject to the work necessary to maintain the item in near to perfect condition up to the point where total replacement is a better option than repair or minor part replacement. In most cases, life expectancy of an item is an arbitrary period based on common sense and experience.

The first study

The starting point of our first study was why was so little life-cycle costing being done in practice when the method was an accepted form of cost evaluation. Was it perhaps impractical or did the results have little impact on design?

The first study (1985–1986) was entitled Cost Effectiveness of Design Decisions and it examined the design of an existing office building and the associated capital cost and revenue costs for an 8–year period of occupation. The basic design options considered at the time of the original design were evaluated by a retroactive life-cycle cost technique to gauge whether the actual building as built was the most cost effective solution.

Two main conclusions were reached, the first that had the design team been armed with life-cycle cost appraisals when discussing design options with the client, the client might have been persuaded to choose different solutions which appeared to reduce both capital and revenue costs to some extent.

Secondly, had the design team attempted to implement a life-cycle cost approach, there was little or no information available at that time on maintenance operations, part replacement cycles or anticipated life of materials on which to found a sound discussion of the economic results of the design with the client.

The second study

The starting point for our second study, derived from the recommendations of the first study, was that a performance and cost-in-use data base was essential to implement a life-cycle cost input to design as part of the normal design process.

The second study (1986–1987) entitled 'Performance/Cost Data Base' therefore examined the needs of different disciplines for such a data base, the form it might take and the type of data that it should contain. Interviews were held with a range of clients, design consultants and premises' managers and, although the type of data output and the reason for requiring data naturally varied across such a broad spectrum of potential users, there was common agreement that such a data base would be extremely useful in each particular activity of briefing, design and building management.

It was considered that a performance/cost data base would have strong support from building clients and professional consultants. The data base would contain data relating to labour and material in operating and maintaining buildings and replacing materials, plant and equipment, i.e. the data required to implement a life-cycle cost approach. In addition it would contain the physical attributes of materials, variations of usage and the interactions between materials, and between materials and the environment, that affect life expectancy of a building and any part of it, including obviously the structural element.

The second study also outlined a total system for the feedback of cost-in-use information to the designer and feedforward of design information to the premises' manager, and recommended that a defined vocabulary on performance and cost-in-use and definitions of performance criteria were developed. This would allow explicit statements on design life requirements to be made in briefing dialogues and for manufacturers to provide designers with unambiguous statements on the performance of their products.

The third study

The starting point of the third study was that no data base could contain all information about all materials, so what information was essential and how could one define the methodology of its use.

A simple existing hi-tech factory was modelled in economic terms covering both capital and revenue costs. Principal design elements were then varied and performance schedules with specific criteria and data were prepared for the base building and for each variation. Once this was done, the maintenance and cyclic replacement cycles were defined and the capital and revenue cost changes assessed.

By comparing the performance values, life expectancies, operational and maintenance requirements and part replacement cycles, choices could be made between design options and on what was on offer from manufacturers taking account of the varying ratios between initial investment costs and on-going running costs.

The cost-in-use output indicates that whilst there are differences in the costs of day to day maintenance as between one design option and another, such cost differences, all other aspects being equal, would not necessarily be seen by the client or the designer as significant enough in themselves to sway the design decision to one option or another.

This is not say that day to day costs are to be ignored, but what emerged as a much more significant aspect was the capital re-investment pattern (Figure 1).

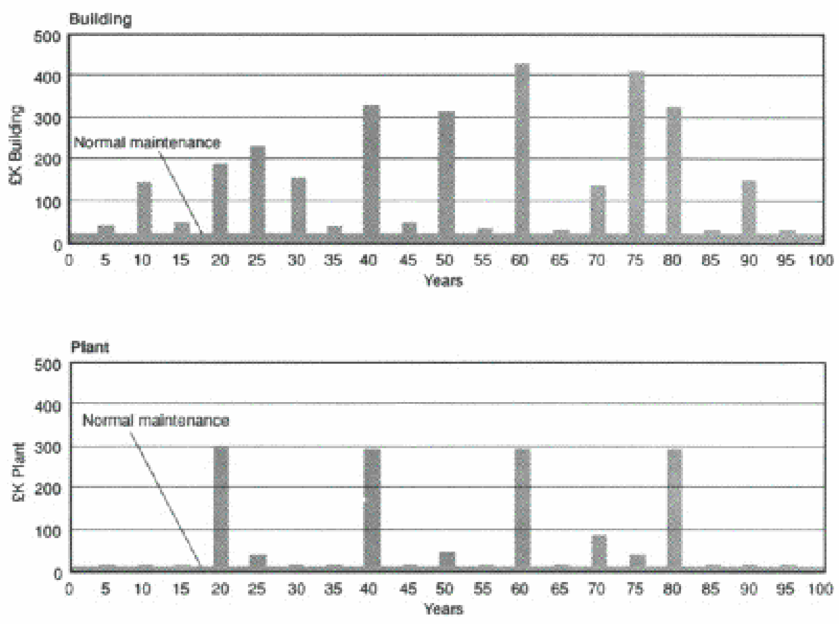

Figure 1 Re-investment cycles for building and plant at today's cost.

Capital is required to replace items or to replace significant parts of items at the end of their working life or due to premature failure. Naturally the less maintenance that is done, the greater the risk of premature failure and the more often will the building owner, or lessee on full repairing lease, have to provide capital unexpectedly to replace major parts of the building.

It was seen therefore that the definition of expected life and the definition of maintenance necessary to achieve that life is of crucial importance to building clients, owners or lessees, designers and premises'/facilities' managers. A recent report [1] indicates that the proper management of property as operational assets is of increasing interest and concern to owners and lessees.

Performance approach

In order for performance and costs-in-use to be properly considered in building design, it is considered that a performance oriented design approach has to be adopted right from the briefing stage and form an integral part of design development.

For a performance approach to design to be adopted and to succeed, it must be simple, easy to use, be adaptable to a multiplicity of situations, and as far as possible, be built on existing practice. The recommended procedure is as follows.

Briefing stage

The client with the design team prepares an outline Building Performance Profile (Figure 2) appropriate to the scheme, and defines the client's strategy for maintenance, and the seriousness of premature failure of any element to his business (and, of course, health and safety).

Design stage

The design team prepares System Performance Profiles in a similar manner as for the building for each system to an extent dependent upon its importance to the building and the impact of premature failure. The design team discusses these with the client and agrees them as the basis of detail design development.

System/material selection or specialist tendering

The design team selects the materials and systems with all the appropriate performance characteristics and with the appropriate anticipated life. With information on maintenance and part replacement, costs-in-use can be evaluated and comparisons made.

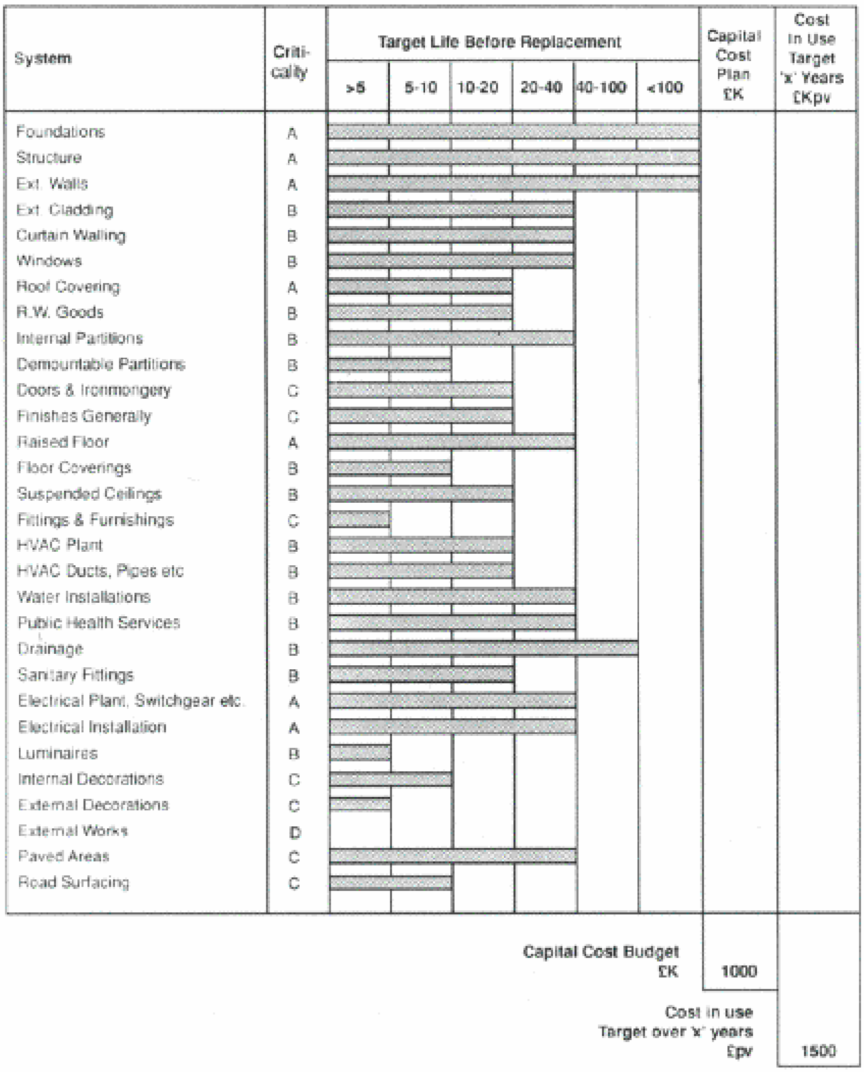

Figure 2 Building performance profile. The question of criticality can be addressed by a simple grading system. (A) Highly critical: failure causing cessation of operation and disruption during remedial work. (B) Critical: lowering working efficiency, remedial work out of normal hours. (C) Not critical: requiring remedial work but not immediately essential. (D) No work. Other definitions may be developed but these are given to illustrate the performance profile.

The design team would obtain such information from:

- Manufacturers'/Suppliers' technical literature
- Published test results from a recognized laboratory
- Feedback from existing buildings
- Tendering on performance specification, with tenderers responding by supplying full O&M details and anticipated life expectancies related to offer

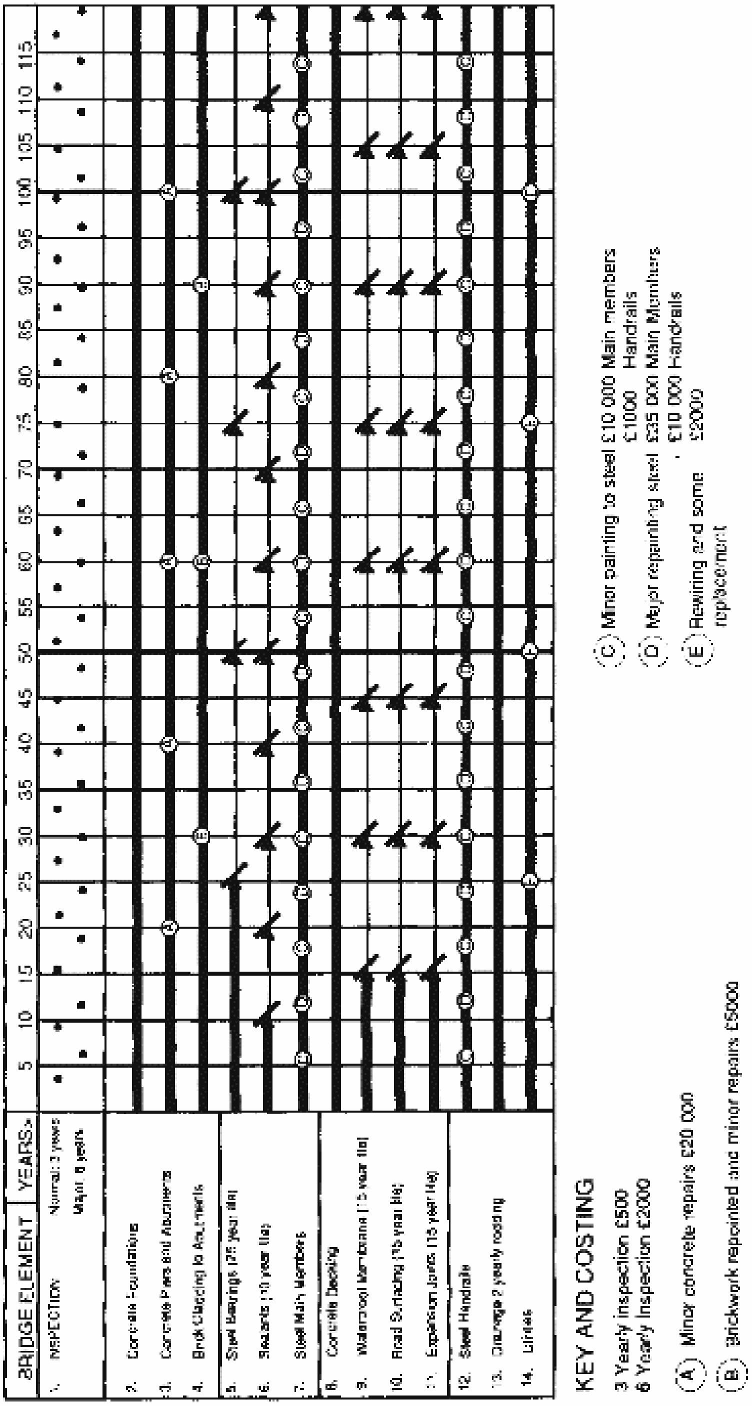

Figure 3 Performance management plan for target design life of 120 years.

- A building performance and costs-in-use data base containing information from all above sources (LINK CMR research project)

Handover and occupation

The system and material performance profiles and costs-in-use forecasts (based on O&M and part replacement details) become part of the normal O&M manual. A separate version with information appropriate to facilities managers is bound separately to be available to future lessees.

Application to structures

To demonstrate the application of the performance concept to structures, a notional motorway bridge has been analysed. The Department of Transport defines the required design life as 120 years, but very few parts of any bridge will last 120 years without maintenance, repair or replacement.

The hypothetical Performance Management Plan given in Figure 3 helps the designer to take account of the implications of deterioration likely to arise over the passage of time. In addition, with the type of financial evaluations given in Table 1 and illustrated in Figure 4, the designer is able to compare the economic consequences of one design option with another. Given the intended design life of 120 years, costs are expressed both as discounted Present Value and at 'Today's Cost' to give both a comparative investment basis and a hard cash basis for decision makers. It must be recognized that long term discounting compresses differences in Present Value terms.

The purpose of the Performance Management Plan is to provide a simple but effective design tool at an early stage in design to focus the attention of both the client and the designer on the maintenance strategy and the work and consequent cost required to achieve a given design life. The performance profile should be fed forward to set up the planned maintenance as intended by the design, and be used by maintenance engineers as the basis for feedback on actual results, a need noted recently by others [2].

To return to the questions at the beginning of this paper, the life-cycle costing technique is a useful tool but it can only be regarded as a means of evaluating expenditure over time to determine the notional investment required. The primary design tool is the performance profile which brings the subject of performance and maintenance to the forefront throughout design and subsequent use. I suggest that the combination of the performance profile, a performance and costs in use data base and the life-cycle cost technique would make a very powerful tool indeed.

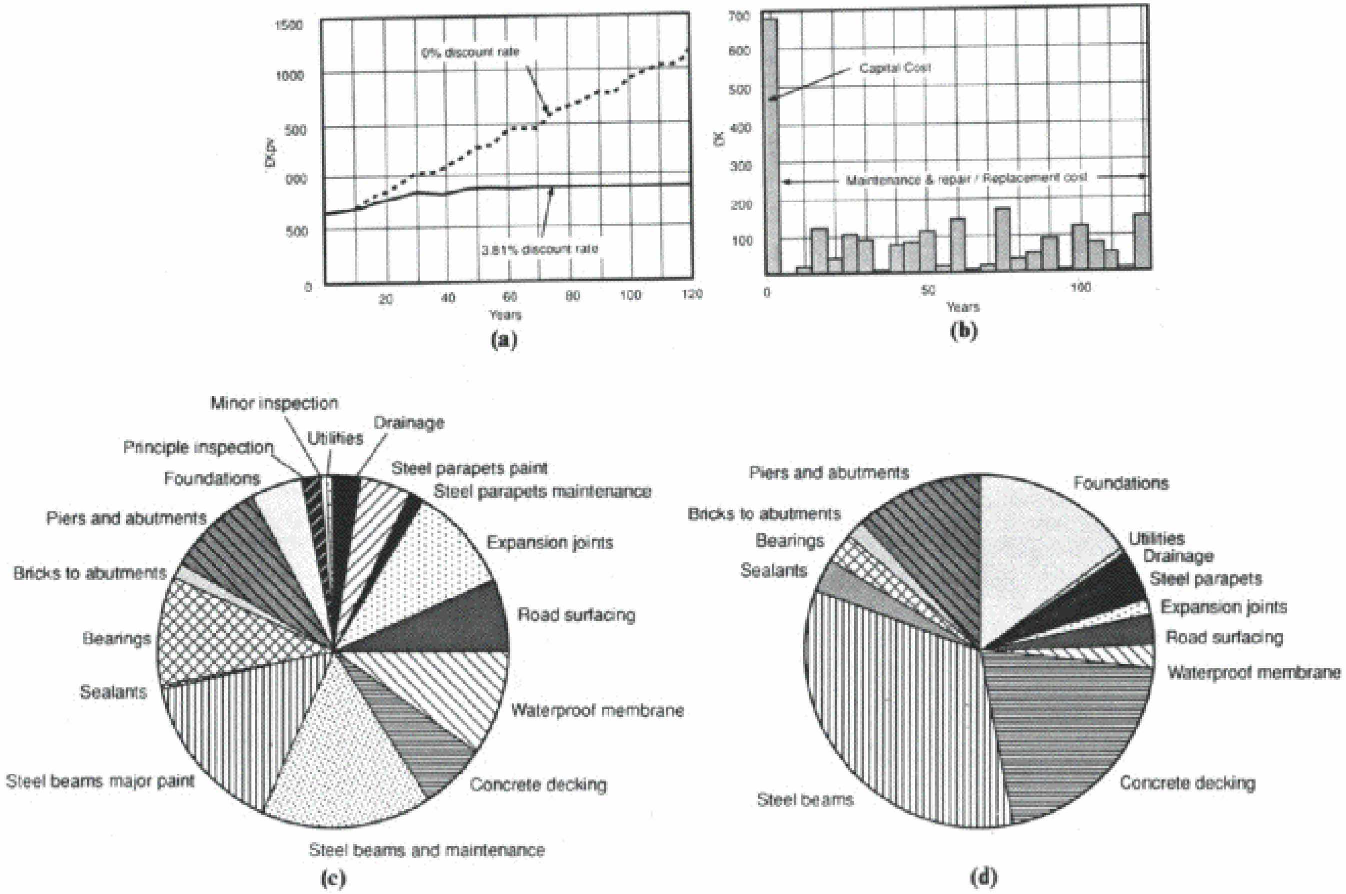

Figure 4 Illustration of financial implications used in compiling Table 1. (a) Cumulative present value, (b) Five yearly cash flow at today's values at zero discount rate, (c) Capital cost and maintenance (today's cost), (d) Capital cost.

Table 1 Motorway bridge life cost basis[a]

Element	Capital cost (£K)	Repair/replace cost (£K)	Life to repair/replace (years)
Minor inspection	0	0.5	6
Principle inspection	0	2.0	6
Foundations	100	0.0	120
Piers and abutments	80		
Repairs		20.0	20
Bricks to abutments	10		
Repointing		5.0	30
Bearings	20		
Replace		50.0	25
Sealants	2		
Replace		1.0	10
Steel beams	225		
Maintenance		10.0	12
Major painting		35.0	12
Concrete decking	135	0.0	120
Waterproof membrane	15		
Replace		25.0	15
Road resurfacing	28		
Resurface		15.0	15
Expansion joints	12		
Replace		25.0	15
Steel parapets	25		
Maintenance		1.0	12
Painting		10.0	12
Drainage	7		
Rodding out		1.0	2
Utilities	4		
Part replace		2.0	25
Total	663		

[a]Total Cost over 120 years at Today's Cost is £2 213 000. Cost amortized at 3.81% discount rate gives a Present Value of PV£929 047.

Benefits

The principal benefits of implementing the performance concept are seen to be:

- Clients are provided with the means to brief designers on performance required, or state their needs so that designers can respond
- Designers can ensure that clients understand the limits of expected performance at a cost, and can respond to the client brief with clear statements
- Designers and manufacturers can select and provide materials appropriate to the client's needs
- Designers can demonstrate that they have exercised skill and care in the selection of materials

- Manufacturers would be able to market and sell products with statements on life expectancy and O&M requirements to their competitive advantage
- Premises and facilities managers and maintenance engineers, can be briefed on the anticipated running costs and the re-investment pattern likely to be required during the life of the building or structure
- The demarcation line between design liability and product liability would be more clearly drawn
- Early harmonization with the performance approach and economic working life requirements of the EC Construction Products Directive due to be ratified in the UK in 1991.

Acknowledgements

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Loads and load effects

W.W.L.CHAN

Introduction

Load and resistance equally affect structural reliability but the determination of load values used in design has attracted little interest from design or research engineers. In contrast, the enthusiasm for research and advances in knowledge of the strength of materials and structural behaviour has continued unabated.

This has led to modern codes of practice of increasing complexity to determine structural resistance more 'accurately', but with relatively little refinement on the treatment of loading.

This background paper summarizes the present situation on loading, discusses some aspects which affect design life and makes suggestions for improving loading specifications in structural design. It is mainly related to loading for buildings.

It should be mentioned that Bill Henderson took a keen interest in this subject, having from 1977 to 1979 chaired the Institution of Structural Engineer's Code Servicing Panel on Rationalization of ?-factors and the preparation of its First Report to the Institution's Structural Codes Advisory Committee [1].

Man-made loads

Dead loads

Upper and lower limits of dead loads are relatively simple to determine provided that reasonable safety margins are incorporated to allow for tolerances in dimensions, materials densities and, where relevant, moisture content. Dead loads should in most circumstances be considered to increase with design life because of maintenance and repair which often involves adding new material.

Imposed loads

Imposed loads as tabulated in loading codes [2] have remained substantially unchanged in value and format for several decades. Typically, tables of single 'nominal' values of uniformly distributed and concentrated loads are given for certain occupancy classes, followed by rules for load reduction factors for structure elements supporting large areas of floor or numbers of storeys.

Apart from very few research engineers interested in loading, most engineers do not know how the tabulated loads compare with reality, but are comfortable in the belief that they are 'safe' because their application has resulted in generally trouble-free structures (at least for ultimate strength). This apathy appears to result from the lack of published papers on loading which appeal to practising engineers, expensive and tedious nature of load data collection, the general unsuitability for laboratory experiment and the lack of commercial incentive which, by contrast, so strongly stimulates research into the resistance of competitive structural materials.

Few engineers appear to realize that field surveys and mathematical modelling of imposed loading have been carried out and published in various countries since the 1930s, with the United Kingdom amongst the important contributors [3].

The main parameters which have been surveyed are sustained loads, representing the static contents and sometimes including people loads if occupied for long periods of time, and intermittent loads representing mainly people loads and sometimes including temporary storage. People loads may be calculated based on the extent of crowding appropriate to the occupancy type; Borges and Castanheta [4] have shown that crowd loading varies between 2 and 7 kN/m^2. Surveys in different countries have produced significant differences in loading intensities, but most of the differences can be attributed to different survey and data classification techniques; when these are eventually adjusted to a more unified basis, the following picture of imposed loads may be expected:

- Load intensities and variability for various occupancy classes can be harmonised at least between European, North American and Australasian countries
- Coefficient of variability of imposed loads range from 30% to 135% based on current data presentation
- Imposed loads tend to increase in intensity with design life mainly from re-arrangement of fittings and storage when changes of occupancy occur
- Load intensity reduces with larger areas (well known from code load reduction rules)

Thus data exist to codify loads on a 'characteristic' basis, which is an essential form of load input to the design equation for the limit state format, but so far this has not been done. Although the data are by no means adequate, it should be possible to complete the missing parts by engineering judgement within the framework of acceptable modelling techniques. It is hoped that the drafting of Eurocodes on Actions may provide a new forum for this approach.

Table 1. Calculated unfactored dead and imposed loads for typical traditional and lightweight buildings

Building	Load ratio imposed to total
1. Two storey house, imposed load 1.5 kN/m^2	
a. With timber frame rendered walls	0.60[a]
b. With traditional cavity brickwork walls	0.27[a]
2. RC frame building, imposed load 5.0 kN/m^2	
a. With lightweight concrete composite steel deck floor and curtain wall	0.53[b]
b. With in situ concrete floor and cavity brickwork wall	0.28[b]

[a]Loads on wall foundation.
[b]Loads on edge beams on each storey.

In future, it is also hoped that more up to date techniques can be used to make load surveying more comprehensive, more efficient and less inconvenient to the occupier. Hitherto nearly all surveys have been carried out manually with little or no instrumentation and on a single arbitrary point in time basis. It would be helpful if a selected number of new buildings could be installed during construction with electric floor load sensors so that continuous recordings can be made over long periods.

At present, buildings are sometimes designed for greater imposed loads than those necessary in the specified occupancy conditions, so that more flexibility is incorporated for future change of use, normally claimed to incur insignificant cost penalty. This may be true where the imposed loads are a small proportion of total load, as for example in a conventional reinforced concrete frame office building. However, with lightweight construction—a design trend which is certain to continue—the cost penalty will be more significant as the ratio of imposed load to total (dead plus imposed) loads increases due to reduced deadweight. Table 1 calculations made by the author show the large increase in the proportion of imposed load comparing traditional construction with lightweight construction.

Geophysical loads

Wind and snow loads used in design are now based on statistically derived values associated with return periods and locational variations. Being so based, characteristic values may be derived and may be varied for the desired design life. In contrast to man-made loads, more expenditure and effort is put into data collection of geophysical loads as part of a wider range of climatological data for different purposes besides structural applications, such as shipping, aviation and agriculture. In addition to extreme value loads for the ultimate strength limit state, data are now, or becoming, available for loads for the serviceability state.

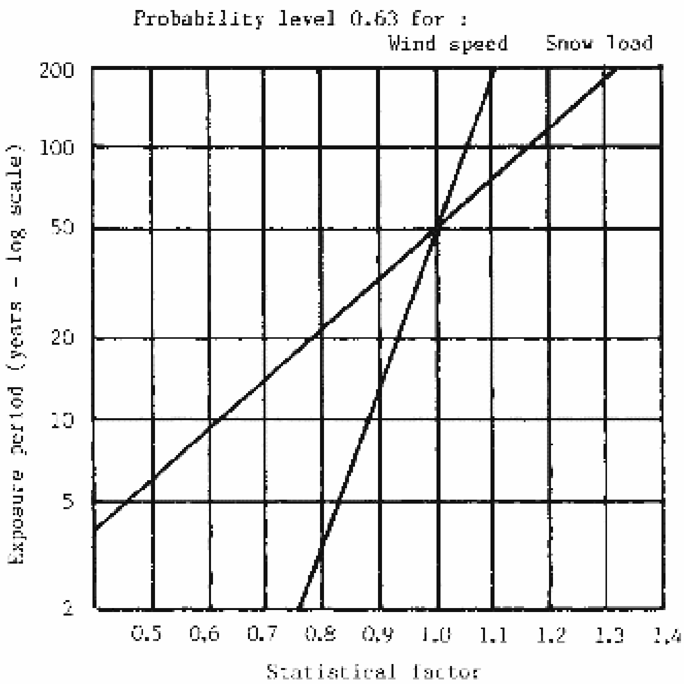

Figure 1 Statistical factors for wind speed and snow load in UK codes.

For lightweight and slender structures, wind load data are now available in special codes to check oscillation and damping, for fatigue and serviceability limit states.

Seismic loads are not normally considered in the United Kingdom except for high risk structures and will not be discussed here.

There should be a safe minimum cut-off limit when load reduction factors are applied to short life structures, to safeguard against *excessive* overstress and deformation or failure in the event of the higher long life load occurring during the shorter design life period. This consideration is not currently being given sufficient attention. Figure 1 shows the current exposure period statistical factor in UK wind and snow codes normally based on the probability of exceedance of 0.02 in any one year (or an average return period of 50 years) for wind and snow loads.

Limit state format

Limit state format for design requires loads to be determined on a 'characteristic' basis, but so far this has not been formulated particularly for imposed loads. The present system merely adopts the 'nominal' loads previously used in the 'permissible stress' basis of design, and the same value of γ_f is applied regardless of different variability of load type. This is not only illogical and anomalous but a backward step, particularly as we have the means of producing agreed characteristic values. The example is often quoted, under present rules, of having to apply the same γ_f value to an imposed load for a hydrostatic head of water with zero variability, compared to an imposed load for say a floor with large variability, which is clearly anomalous. In overall terms, it can be said that engineers have little idea of the real safety margins involved in design.

Prescribing characteristic loads

In a report shortly to be issued as a Published Document [5], BSI Technical Committee CSB/54 on loading in buildings proposes that, instead of the present system of using single values of γ_f for loads, each variable load type should be given two pairs of values for direct use in design. No separate γ_f factors would be required in design, any factoring which takes into account the variability of the load type being included in the prescribed load pairs. The two pairs would separately relate to the ultimate strength and serviceability limit states and each pair would consist of:

- For ultimate strength
  - The maximum extreme value
  - The minimum extreme value
- For serviceability
  - The maximum severe value
  - The minimum severe value

In essence the extreme values represent the combination of sustained and intermittent loads, and the severe values represent most of the sustained loads and a relatively small component of intermittent loads where relevant to the particular load type. In the case of geophysical loads, the sustained component of load is small or could be taken as zero.

It is hoped that within the forum of drafting the Eurocode on Actions, this proposal will be given consideration and support.

Load effects

Ultimate strength limit state

Structural collapses, although dramatic and affecting public opinion when they occur, are rare. They are mainly the result of gross design error or faulty construction but seldom from real lack of engineering knowledge nowadays. Despite this, research in structural behaviour is almost exclusively directed towards ultimate strength of materials and structural forms.

Ultimate strength investigation is attractive to the researcher because it is a clear and explicitly definable limit state and provides unlimited scope to advance the theory of structures.

The result has been that codes of practice have become exceedingly detailed in design procedures for ultimate strength often for even quite simple structures, in combination with crude loading data, surely an unbalanced, uneconomic and illogical situation.

Serviceability limit state

It is widely known that costly serviceability failures are far more common, although they seldom involves loss of life and injury and therefore attract less public attention. Yet the research effort on this limit state and the design guidance available in codes and other sources are extremely limited.

Unlike ultimate strength, many criteria concerning serviceability are not quantitative (e.g. appearance) and often involve combination of structural and non-structural factors (e.g. crack width and durability) or multi-material conditions (e.g. brickwork combined with steel and concrete frames) or comfort thresholds (e.g. sway, deflection, vibration) or time effects (e.g. creep) or cyclical loads (e.g. thermally induced displacements). Loading for serviceability limit state is by nature more complex compared to loading for ultimate strength related to an extreme event.

Separate codes are generally written by different Standards committees for each structural material and rarely, if ever, for combination of materials. Advice on design for serviceability in all codes has always been scanty. γ_f values of 1.0 have been typically assigned to published 'nominal' loads for serviceability in most design codes but the real significance has not been fully questioned. Often this has resulted in, for instance, unsatisfactory overcambered beams against factored loads which do not occur for sustained periods, if at all.

Lightweight prefabricated structures and cladding often contain a diversity of materials, more connections, and greater sensitivity in reversible and irreversible movements from environmental changes than in traditional construction. Serviceability problems will therefore become even more important in the future.

Conclusion

Loads are as important as resistance in structural design, but have received relatively little attention, particularly the prescription of imposed loads in buildings. Load survey data and load modelling techniques are available, although limited, to put all loads on a statistical characteristic value basis. New proposals are to be published shortly to prescribe factored characteristic loads for both ultimate strength and serviceability limit states.

Too much emphasis has been given to ultimate strength, despite the low incidence of structural failure. Too little attention has been given to serviceability despite the high incidence of failure.

The increasing use of lightweight prefabricated structures highlights two loading aspects affecting the adequacy of design life for current and future structures: the higher cost penalty for adopting greater than necessary imposed loads to provide flexibility of change of use; and greater risks of serviceability failures.

Loads generally increase with longer design life. For short life structures, load reduction should be made with care in order to prevent excessive overloading or failure in the event of higher loads from a longer return period occurring within the short design life.

Acknowledgements

The author is grateful to Mr C.J.Judge, Building Research Establishment, for his help and advice on loading publications and constructive comments during the preparation of the paper.

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Some thoughts on the application of design life principles in practice

J.SCHLAICH and M.PÖTZL

The need for life principles in practice

To consider buildings and structures as objects which should last forever and therefore should be as good as possible is obviously as unsatisfactory as the request derived from the school of thought that we must always try to do better and apply any available knowledge. It leaves the designer with the undefined and uncomfortable situation of not knowing when it is enough, while aware that he is producing a chain of very unequal links without being able to assess which one will fail after which period of time.

In order to prevent the lifetime of the whole from being terminated by the loss of one single link, which would otherwise be an extremely wasteful approach, the request for accessibility and replaceability was introduced, which certainly helps here and there, but should by no means be considered as the remedy or panacea, as shall be discussed later.

This may, in view of the very comprehensive paper of Somerville on this subject [1], be enough to shortly characterize the situation as it exists in practice and to explain why it is worth making efforts towards a more rational approach in predicting the design life of buildings and structures.

Two categories of effects affecting design life

What catches the eye as we look at the possible causes and factors which obviously allow buildings to terminate their life within a finite period of time, and contemplate the possible concepts of a prediction, is the striking similarity with earlier discussions on structural safety. Following the initial euphoria about a modern and rational concept on a probabilistic and stochastic basis that was to fully replace the 'oldfashioned' empirical and deterministic approach towards safety, it is now generally accepted that there are two groups of measures [2]:

- Those which can be derived by means of probabilistic tools, because the parameters can be correlated and there is sufficient statistic evidence to translate in rules and regulations whose observance can be supervised but which make sense only if those measures which are to avoid human errors or accidents have become effective or, as Schneider formulates it: 'Safety theory may start only after real errors have been ruled out' [3].

Similarly, as regards design life, we can identify

- Effects of continuous degradation or of life exhaustion, where the correlation between the attack and the deterioration with time is either known already or has a good chance of becoming known after some further research work. With respect to these phenomena, a theoretical design life assessment or prediction is in principle possible. In some such cases it is even possible to provide (or else omit) means to the effect of lengthening (or shortening) the design life on the basis of a cost versus lifetime relation. In principle these effects are related to the special material used.
- Effects of sudden, unexpected appearance as a result of human error and which are more related to the individual building or structure as a whole. Therefore, their appearance and whether or not there is this negative influence on the lifetime depends on the skill and experience of the designer, or the consent with his client and on the quality of the work, whereby the latter, of course, also holds true for the first type of effects.

Whereas we shall come back to the second category later on in this paper, some well known examples for the first category are listed here:

- Corrosion of the reinforcement of structural concrete:
  - time for carbonation to reach the reinforcement as a function of concrete cover thickness, cement, curing, density, water/cement-ratio, environment
  - time for corroding reinforcement of structural concrete to become critical, i.e. causing loss of strength, spalling of concrete etc. as a function of humidity, steel quality, coating
  - time for open cracks to cause corrosion as a function of type and width of cracks, and of the environmental conditions.
- Corrosion of structural steel:
  - time for wearing of paint as a function of…
  - time for wearing of a galvanized surface as a function of thickness of zinc layer and of…
- Stress corrosion of high strength steels as a function of alloy, height of stress,…
- Ageing of wood and other material used for structures such as coated fabric, GRP as a function of time and agents
- Fatigue of members and joints:

– high cycle fatigue excluding (theoretically) any fatigue failure

– low cycle fatigue and fredding, relating the number of probable cycles during a certain lifetime to the permissible number of cycles.

Though this list is not complete it is noticed that there is only a limited number of examples that fit into this category and that they are in all cases concerned with the material itself (i.e. structural concrete, steel, wood, plastics, glass etc. as such), but not with the structure or building as a whole.

Looking at some of the recent research work on this sort of predicting service life of materials, especially so of concrete, one wishes to express the hope that the same may happen some day here as did with structural safety, when myriads of sometimes unreadable papers finally gave birth to a very practical and educationally sound approach: the partial safety factors. From a practical point of view, a similar outcome from the work on material degradation as a function of time and related to the different parameters correlating the material on the one hand and the actions and agents on the other, would be most welcome. There is a good chance that this may happen, especially if the dialogue initiated by this colloquium is maintained.

A concept for predicting design life

In view of the fact that a termination of lifetime of the material used for a structure really means the end of the structure as a whole, a clear concept towards design with respect to a limited or prescribed lifetime could be:

- Choose the required building life according to the specific building's role or significance, e.g. following [4], or its owner's request and satisfy it by
- Selecting the materials, their composition and protection for this building in its specific environment in such a way that they fulfil the required lifetime and by
- Designing and constructing the specific building or structure as a whole in such a way that its overall lifetime does not fall short of the lifetime of its materials and fulfil this requirement at a minimum of cost.

This of course may be only one concept amongst many others and may be suitable mainly for structures with directly exposed structural materials such as bridges, while other concepts—which will certainly be proposed during the colloquium—may be more useful for buildings.

This concept accepts that any building or structure is a prototype and has a limited lifetime but requests that ageing should be a continuous and controlled degradation process of the building's exposed material. It leaves us with the question of how to handle the second category of effects as mentioned earlier, so that they will not interfere with the material's continuous degradation, resulting in discontinuity or even sudden descent.

Since this question obviously addresses all which we call skill, creativity, experience and dedication of the designer and builder, there cannot be a concrete answer to it—fortunately as we might say, because if so we could replace the engineer by an apparatus or device. Nevertheless we may try to discuss some points of the optimal building or structure, which lasts at least as long as the material it is made of, or which could be called the degrading building or structure.

Means towards the continuously degrading building or structure

The role of the conceptual design (Entwurf)

A commonplace fast must be brought back to mind: the quality and the economy of a structure in the widest sense are above all prescribed and governed by its conceptual design. This is not to exaggerate the role of the designer or to underestimate the necessity of excellent workmanship or to let the contractors evade their responsibility, but it is a fact that a large number of deficiencies which are attributed at first sight to errors or negligence on site can be traced back to the design. We all know numerous examples of this, from the most trivial case of overcongested reinforcement, which does not allow the concrete to penetrate, to such monumental failures as the partial collapse of the Berlin Congress Hall. Of course, the latter in fact collapsed due to corrosion of the prestressing steel holding back the edge beams. But this corrosion cannot be attributed to a material deterioration but resulted from a misinterpretation of the load bearing behaviour of the stress ribbon [5].

It is extremely disquieting to observe that the majority of structural designers these days appear more inclined to react than to act, to cure or heal than to create. To follow this up with an example, just compare the number of papers written on crack control as a reaction to durability problems, to those proposing crack prevention by providing for flexibility as a response to restraints.

One cannot design and work with a material which one does not know and understand thoroughly. Therefore, design quality starts with education. This is the best investment for quality. Structural engineering is a practical profession and therefore no student should be admitted without a practical training, no university curriculum should fail to include courses in sketching, drawing, modelling. Students must be taught how to live with computers, but to use them only after getting approximate results by rule of thumb calculations, and to keep good company with their future architect colleagues. It also calls for translucent and consistent design concepts for reinforced and prestressed concrete. They can be derived only from physical models, and we must get away from empiricism. The strut-and-tie-models could bridge the gap which is liable to open between engineers in practice, who need handy tools, and those researchers who only believe in computer results, by satisfying both: the practitioner will use it in daily design work and the researcher will derive from it the input which he then elaborates on his computer [6].

A better understanding of the behaviour of reinforced concrete and of the context of the structures made from it, will certainly make us aware that we have directed our efforts in the past too frequently to factors of secondary importance: it makes no sense to fight the shear battle for years if the savings in stirrups are negligible in comparison with the outcome of other design parameters. It makes no sense to calculate crack widths with extreme accuracy if their influence on durability can only be measured in broad terms. It makes no sense to infinitely refine an FEM analysis if the material properties or the geometrical imperfections originate from an uneven building site.

However, it does make sense to apply imagination to design and structural detailing. Quality and lifetime extension thus achieved does not cost, but results in savings.

Robustness through design

We should not care so much for structures that withstand or resist, but that survive; we should not so much try to compare action and reaction, load and resistance, but try to conceive the structure as a whole and in its environmental context during its life, such that it survives even events it was not analysed for, with robustness [7] and without additional cost.

An example may illustrate that: it has become a habit to predominantly use box girders for bridges. For reasons of fabrication, they must be, say, between 2.5 and 4.5 m high. So the standard spans range between 30 m and 80 m, much longer than necessary in most cases, where 15–25 m would suffice to meet the functional requirements; not to speak of the aesthetic improvement (Figure 1). These shorter spans are the domain of the slab or of the open TT-girder. Their smooth, open sections are better than filigree, undulated and hollow ones. They permit a better control of concrete cover, prevent thermal shock, are easier to inspect and maintain and last but not least, need fewer or no construction joints. A pure slab is superior to an open T-section, and the latter to a closed box girder. The slab, due to its low bending stiffness may be connected homogeneously to the supporting columns and thus require no bearings. Why, if flexibility requires it, should these columns not be made of steel and be directly bolted to the concrete slab and concrete foundations [8]? Such structures, no question, look light and translucent, which is unfortunately frequently confounded with fragile or even brittle. Robustness and ductility or flexibility are not opposing, but complementary! That the opposite may be true was deplorably demonstrated by the collapse of the Cypress-Viaduct during the recent Californian earthquake. This heavy looking structure was brittle due to the wrong choice of the statical system by providing unnecessary hinges, by poor detailing of the frame corners and by too little transverse reinforcement.

Design must also foresee robustness towards execution errors on site. So if a column is loaded by bending in addition to its usual axial forces and thus needs non-symmetrical reinforcement, it may be better to provide the additional bars on either side than to risk that they are placed on the wrong side.

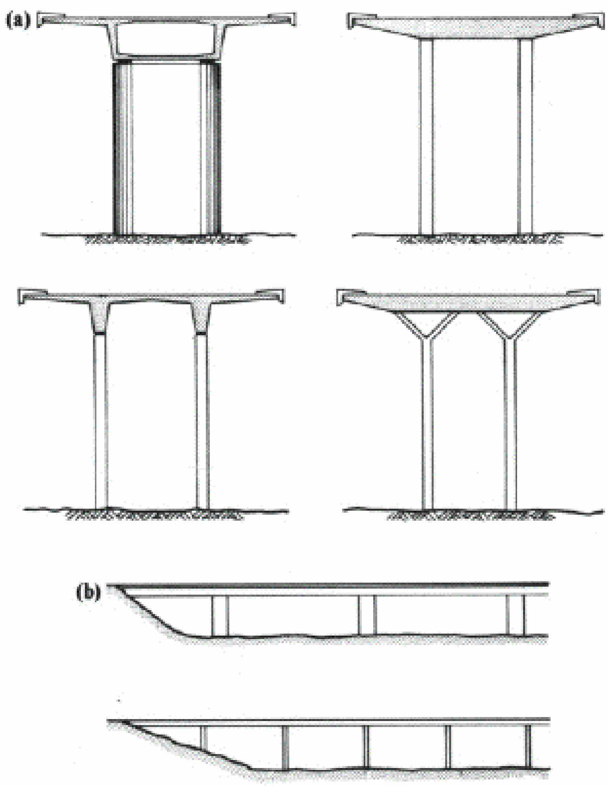

Figure 1 (a) Some cross sections of highway bridges, (b) Influence of the span lengths on the form of a valley bridge.

When selecting the materials (concrete, steel, wood, plastics), we should be governed only by the question whether their specific properties are appropriate for the given purpose, not by affiliation to a lobby. It is a pity that most civil contractors and also many university institutes are material-oriented and not simply construction-minded. The joint use of different materials in one structure, a hybrid solution, promises better results. The composite girder for long-span cable-stayed bridges is often superior to the pure concrete or the pure steel girder. High-rise buildings erected in steel and encased in concrete are the most economical. Box girder bridges with concrete top and bottom slabs and steel webs open new possibilities. The quality of concrete itself is best brought out if the design does not deprive it of its monolithic nature. The best joint is no joint, the best bearing is no bearing. If we know and utilize the ductility and ability of reinforced concrete to compensate for forces due to settlement or temperature effects, we will approach this goal. Latest research, such as on the effect of confining reinforcement on rotation capacity, or on the capability of concrete to transfer forces across cracks with the help of aggregate interlock, is really useful for a move in this direction.

Concrete can be freely shaped on site, independent of a shop. Instead, we have distorted its character by imitating, for purely short-sighted economic reasons, the shapes and production methods which are typical for steel and wood: plain and straight members, cut in pieces and joined again, boring and clumsy—and quickly deteriorating.

Hundreds of examples could be given showing how the lifetime of structures depends on design and detailing and how it can be extended if existing knowledge and imagination are employed. A wealth of such examples can be found in CEB-Bulletin No.182 [9].

On life extension through replacement of components

One natural way towards lifetime extension is to replace shorter life components [4]. Typical examples are the pot-bearings or the externally applied prestressing tendons for bridges. Paints or plastic covers, which themselves need be renewed after some years, are proposed to ensure durability of concrete.

It is agreed, for instance, that the railings and kerbs of a bridge subject to intensive salt spray must be replaceable without having to build a whole new bridge, and the same applies for the cables of a cable-stayed bridge. The paint, plaster and cladding of a house are other obvious examples. However, a word of warning must be spoken against carrying such tendencies too far. A building or a structure should first of all be designed and built to last. Such is its character as against that of a machine. It is what the user expects of it. If the trend continues, the designer will concentrate on replaceability instead of quality. If negligence has no serious consequences, it will be encouraged. If the surface of concrete is the painter's job, the concrete contractor will not care. The outcome will be a disaster both under the aspects of quality and of lifetime. Buildings and structures will become a stockyard of individual spare parts which need not be compatible because they can be exchanged. The external prestress appears not only to solve but also to evoke some problems. The writers believe that a combination of regular post-tensioning plus some external cables, which will also prove an asset to strength, is a better solution. This has been used for the new Danube-Bridge near Bratislava, CSFR. In Germany, though the single-cell box girder for 6–lane highways was developed there, it is forbidden today in favour of two individual box girders on separate piers, in order to be able to repair or replace one half of the bridge with the traffic running on the other. For railways, standard-type simply supported beams are prescribed for the new high-speed trains, which are replaceable overnight. The result is unsatisfactory, not only from an aesthetic point of view but also with respect to durability: these structures will not only be repairable, they will indeed need repair. This sort of thinking is conservative, going in the wrong direction.

Termination of life through nominal change of user requirements

It has to be accepted that here and there, the life of a structure of building terminates due to a radical change of its user's requirements. But very often the designer could have prevented such a situation by thinking ahead. Typically this is the case if a structure has been built to serve predominantly one single requirement but which is nevertheless of only insignificant influence on the forces. The satisfaction of this requirement then governs the lifetime, but is of no influence on economy. If now the effect of this single requirement on the structure is uncertain or varies extremely, only nominal additional costs will be incurred in strengthening the structure with respect to its resistance to this factor, and thus its value or lifetime will be increased, perhaps dramatically.

Two examples may illustrate this. A young industrialist builds his first factory. The structure is a frame, mainly there to support a crane running on two rails, which rest on corbels (Figure 2). For economic reasons, a crane is chosen with a capacity just adequate for current production requirements. Accordingly, the consultant designs the corbels, columns and foundations for this load. The firm flourishes and after some time the industrialist wishes to install a crane of greater capacity, but he cannot because the corbels will not tolerate the additional load. The columns and foundation, on the other hand, could take it easily because they are designed for a most unlikely superposition of wind, snow and crane loads, whereby the latter is the least significant. The whole building has become obsolete because its designer did not foresee the eventuality, in view of which he should have overdesigned the corbels, and only the corbels, with the effect of negligible additional cost but a tremendous advantage for his client. One might argue that the client, if he were asked, might not permit overdesigning the corbels at his expense, because it is not required for safety reasons or by codes. An engineer, however, must try to understand the psyche of his client and should not ask him, if he cannot expect

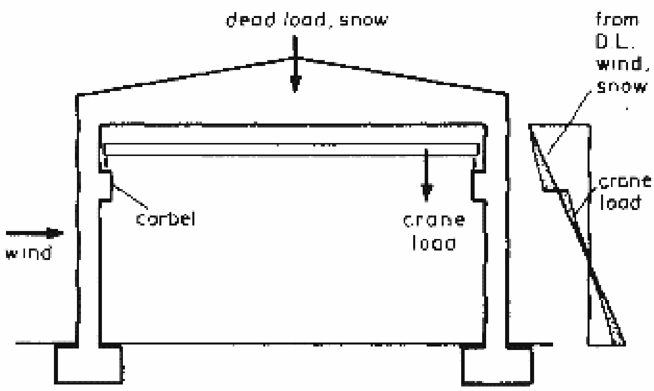

Figure 2 Moments in the columns of an industrial building.

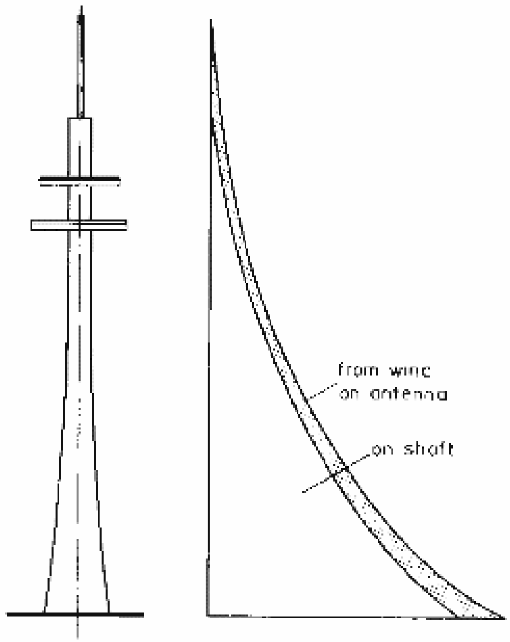

Figure 3 Moments in a TV tower due to wind.

the right answer. Engineering is more than following prescriptions; it demands some imagination.

A TV station needs a cylindrical antenna, height 15 m, diameter 1.2 m, on top of a concrete mast, height 200 m (Figure 3). The predominant load is wind, but the contribution of the antenna itself to the wind moments is of significance only at the topmost part of the mast, further below it disappears as against the contribution of the mast itself. This is even more the case for the vertical loads. Therefore, a far-sighted designer will give some extra strength, which costs almost nothing, to the top of the mast and will thus improve its quality by preparing it for the day quite likely to come when his client wants to have a longer antenna installed; that is what the mast is there for. On the other hand, the client's representative himself will not ask for that extra strength right from the beginning, because he does not know the financial consequences and is afraid of being held responsible if the extension should not be required. Sometimes, engineers must assume other people's responsibilities.

Closing remarks

The authors admit that thinking consequently in lifetime categories is quite new, but that time has come to do it more consciously. As with structural safety considerations this can only partially happen, and with respect to the material as such on an analytical basis, but much depends on the capability of the designer to think ahead and to avoid the sudden and unexpected descent of the remaining lifetime by an imaginative conceptual design, good detailing and quality control.

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Overview of service life prediction—materials issues, including QA and certification

J.JUBB

The prediction of service life, particularly where long periods of 50–100 years are involved, is hazardous. All the information available that will help in making the prediction must be systematically gathered, interpreted in the particular context in hand, supplemented by further research where necessary and finally, have applied to it the judgement of an experienced assessor.

A systematic approach is essential and becomes more so when new materials, products or systems are involved, because the central plank of practical experience is diminished and the uncertainty increased.

A clear understanding of the agencies which can cause deterioration must be obtained and the manner and extent to which they will have an effect, singly and in combination, assessed.

Simulative testing, properly correlated with service conditions, allow minimum life predictions with reasonable confidence.

Introduction

Each structure is unique in some respects and this poses questions as to whether a chosen design life can be achieved with reasonable certainty. The relationship between materials forming the structure, the components fabricated from them and the overall structure is often complicated and this complication is compounded by the variability in the design of the structure, the environment in which the structure is built and required to operate and the people and their activities it must accommodate in service.

The range of questions posed and the ease with which they are answered depends on a number of important factors, the central one of which is the amount of directly relevent experience of similar structures available. When new materials and processes are introduced, such experience diminishes. The variety in the structures already referred to indicates the need for caution in assessing whether a chosen design life will be achieved.

In assessing the suitability of materials (and components) for use in structures, or anything else for that matter, an intimate knowledge is required of the materials, the way in which they are incorporated in the structure and the conditions they will experience in service. It is unusual if all of these are available with absolute certainty, and sometimes the level of uncertainty can be significant. A systematic and cautious approach is called for if an economic and effective choice is to be made.

The use of new, or relatively new, materials increases the uncertainty and requires additional precautions, which normally take the form of technical investigations (with attendant costs); this is especially the case where structural safety is concerned. This paper will concentrate on this aspect of new materials; it can be demonstrated, however, that the underlying approach remains the same for all materials, but the formality and level of investigation increases, or should increase, in the case or new materials.

The BBA approach

The work of the BBA, over a period approaching 25 years, has been the assessment of innovative materials, components and systems for use in construction. The Agrément Certificates issued are intended to give useful information on the performance of the subject when installed and used in a particular form of construction. This involves ascertaining all the performance characteristics of the product when new and, additionally, predicting over what period of time these characteristics will be maintained. The latter is generally the most difficult part of the BBA's work, but it is made easier by virtue of the fact that only a single product or material, of defined specification, manufactured and used in defined circumstances, is considered at any one time. It is notable that where the cost of the product controls the composition from day to day, rather than there being a fixed specification, Agrément Certification is not possible.

The assessment of service life involves a study of the changes that will occur in the material due to environmental agencies (natural and man-made) as well as the changes produced by non-environmental agencies (see Table 1). With new materials, the accuracy of the prediction will depend critically on how accurately the environmental and non-environmental agencies that will produce changes in the material can be defined and simulated in tests. It is mainly because of the lack of knowledge of the quantitative levels of the agencies likely to cause deterioration that organizations involved in life predictions of building materials have in the past spent most of their time attempting to correlate accelerated laboratory tests with samples exposed naturally. Success in this work has been somewhat mixed, including cases where a correlation has been possible and the subsequent results in service satisfactory, cases where a correlation has been obtained but the subsequent service life prediction has been inaccurate, and many cases where a correlation has not been possible. This has tended to result in a body of opinion that accelerated ageing tests cannot be relied on to predict

Table 1. Factors to be taken into account in service life prediction of materials

	Product life					
	Environmental agencies			Non-environmental agencies		
(nature)	Chemical	Physical	Mechanical	Chemical	Mechanical	Biological
	Oxidation	Loss of volatiles	Freeze/thaw	Processes/activities	Loading	Damage (mechanical
		Solvation etc.	Wind fatigue/overstress	Cleaning	Abrasion	Life support etc.
	Carbonation			Spillage	Impact	
(form taken)	Hydrolysis		Thermal fatigue/overstress	etc.	etc.	
	Thermal degradation					
	UV degradation etc.		Wetting/drying fatigue/overstress etc.			

the service life of building products. This is not the BBA view, but it does emphasize the point that a life prediction programme must be properly designed and the results interpreted in the light of the factors which may influence the performance of the product in a defined service situation; supported where possible by theoretical studies and experience of relatable products and conditions. The programme will often involve a series of different tests, each of which supplies pieces of information which, taken together, enable an assessment to be made. The results may not be applicable to the use of the product in other service situations and the test régime chosen may not be suitable for any other material that is to be used in the same service situation. Certainly, great caution would be required in any extrapolation made.

Although in theory it should be possible to define the ultimate life of any structure and the role played by the constituent materials, difficulties arise, however, in the length of time required to do this and the cost involved. It is important to note that it is considerably easier, and cheaper, to predict whether a material will reach a defined target life, than it is to predict how long the material will last. This is often a problem with building products, the target life being largely undefined. With structures it is very important indeed to have a well defined target.

The life achieved may be influenced by a renewable or non-renewable protective system. The protection may be essential to the life of the product or be the means of extending the life beyond that normally envisaged. The protection may be total or it may be only partial and intended to slow the degradation processes. The service life assessment becomes one which includes assessing the efficiency of the protection, the risk of damage in transportation, installation and service and possibly the praticability of replacement of the protection and the end of its useful life.

Most BBA material or product assessments involve a likely minimum life. In the case of structures, where a target service life is defined in advance, a prediction becomes one in which an assessment of whether the target will be achieved or not. The prediction of minimum life is extended in the form of further assessment of how the product will behave following the minimum period; with emphasis being placed on the likely mode of failure. Any maintenance required to help in achieving and/or extending the life is defined.

The minimum life is generally the time to first failure and often represented by the period to which it is reasonable to extrapolate the results of accelerated ageing tests. In all cases account will also be taken of:

- Experience of relatable materials and/or conditions
- Theoretical studies (for example involving chemistry, physics, statistics)
- Torture tests (extreme conditions)
- Judgement

By its nature, and bearing in mind the subjects assessed are in some way innovative, the predicted minimum life will be somewhat conservative.

Agencies likely to cause deterioration

Environmental agencies

Environmental agencies may potentially affect the long term properties of a material and the reactions they may create are given in Tables 2 and 3. From a knowledge of the chemistry and physical nature of the material, a decision on which of these will be critical for the material in question can be made. However, difficulty exists in quantifying the effects of these agents over the envisaged lifespan of the material for a given function. In some cases the available meteorological data or data derived from it in codes of practice, for example, can be used to develop a test régime. The available data on wind loads enable assessment of the effects of wind fatigue for individual situations, for particular zones or for all possible situations in the country over specified periods of time with reasonable confidence. Similarly, the data on rainfall and the associated British Standards on the driving rain indices give at least an indication of how wet materials may become and over what period of time they may remain wet in different situations on the building and in different zones of the country.

Unfortunately, the data in two critical areas, UV radiation and thermal radiation, are not available to the same extent. With UV radiation the problem is lack of measurements reported on sufficient sites over a long enough period of time to establish maps or even a sensible idea of the levels of radiation that are likely to occur in practice. There are more thermal radiation data available, but

Table 2. Main environmental agencies (natural and man-made)

	External	Internal	Underground
Heat	Solar (thermal) radiation Radiant loss	Internal heating Solar gain Localised hot spots, heating Lighting equipment	(Little variation in temperature)
UV	Solar radiation		-
Water	Precipitation (solid/liquid) Humidity	Processes Cleaning Spillage	Soil water
Chemical	Oxygen (ozone) Carbon dioxide Industrial pollution Marine atmosphere	As for External plus Processes Cleaning Spillage	Variable pH Variable chemical content Pollution
Wind	Wind loads	Wind loads	–

Table 3. Environmental agencies: possible deteriorating effect

	Reaction		
Agent	Chemical	Physical	Mechanical
Heat	Thermal degradation Acceleration of chemical reactions	Loss of volatiles	Freeze/thaw Thermal fatigue Phase changes
UV	Degradation of polymeric materials	–	–
Water	Oxidation Carbonation	Creep (under stress)	Expansion/contraction
Wind	-	–	Fatigue

it is difficult to convert these data into actual temperature profiles of a material in service.

Temperatures and the changes in temperature are the most important agencies in causing deterioration of materials. Not only may these cause chemical changes due to thermal decomposition of the material (e.g. thermal degradation of PVC), physical changes due to loss of volatiles (e.g. plasticizer loss) or by creating phase changes within the material (e.g. salt crystallization within ceramics) but it will also affect the rate of decomposition due to chemical reactions such as oxidation or those due to UV radiation. In addition, the continuous changes in temperature, particularly where composite structures are involved, or porous materials in the presence of water, produce mechanical stresses which can cause failure.

The information that is required to predict the likely effect is:

1. The maximum temperature achieved in service
2. The minimum temperature achieved in service
3. Either:
   – the temperature/time distribution over the life-span of the product, or
   – the mean reaction temperature over the life span of the product (the continuous temperature that will achieve the same amount of degradation over the same timespan as would occur under service conditions)
4. Maximum daily temperature variation
5. Mean daily temperature variation
6. The number of times over the lifespan of the product the temperature will fall below a critical value, i.e. 0° C.

Items 1, 2 and 3 give information used to predict the degree of deterioration due to chemical or physical processes. Items 4, 5 and 6 give information used to predict the possible mechanical damage to a product.

The temperatures that will be achieved in service due to solar radiation will at any one time be the product of the ambient temperature and the temperature rise created by the solar radiation. The amount of solar radiation received will depend on the location of material and its orientation and slope in relation to the solar radiation.

The available solar energy depends on the hours of bright sunshine, the opacity of the cloud cover, the time of the year and the latitude of the site. The temperature achieved by the product in service will, apart from the above, depend upon the colour, the reflectivity, the thermal properties of the material and the wind conditions.

Tables 4 and 5 give the results of continuous temperature measurements for a period of one year obtained at the BBA site on roof coverings at different orientations and slopes with different forms of thermal protection. It can be seen from these measurements that differences in the orientation, slope and protection result in significant differences to the temperature profiles and temperature fatigue of the material, and hence a different predicted life. These measurements

Table 4. Measurements on black roofing material (12 months)

	North 30° slope	Horizontal 0° slope	South 30° slope	Ambient
Maximum temperature (°C)	57	72	78	32
Mean maximum daily temperature (°C)	26	34	40	15
Minimum temperature (°C)	−15	−15	−17	−3
Mean minimum daily temperature (°C)	3	2	3	6
Annual mean temperature (°C)	10.7	11.7.	12.4	9.3

Table 5. Temperature measurements on flat roofs: different thermal protections

	Thermal protection			
	None	White paint	6 cm gravel	Invert roof
Maximum temperature (°C)	70	46	37	31
Mean maximum daily temperature (°C)	33	19	15	20
Minimum temperature (°C)	−17	−16	−2	16
Mean minimum daily temperature (°C)	2	2	9	19
Annual mean temperature (°C)	13.3	9.6	12.0	19.9

only apply to one site; the variation that occurs on different sites within the United Kingdom is unknown. If reasonably accurate predictions of service life are to be obtained, either many more measurements of actual service temperatures are required, or a means of converting existing meteorological data into material temperatures is needed.

Non-environmental agencies

The non-environmental agencies that have to be considered include all those other factors imposed on the material during its service life that may cause deterioration. They are in general the stresses that are imposed by users in the various activities of living, working and playing, including:

- Permanent loading
- Fatigue loading
- Impacts
- Abrasion
- Chemical effects

It is difficult to tabulate all the agencies that may have an influence in all the different situations where construction materials are used. It becomes very much easier, however, when a specific material for use in a particular situation is considered. For example, domestic floor coverings are subject to:

- Abrasion from foot traffic
- Indentation from static furniture
- Indentation from moving furniture
- Indentation from rocking chairs
- Indentation from dropped toys, household tools and utensils
- Water for cleaning; spillage
- Chemicals for cleaning; spillage of normal household effects.

In this case, the levels of stress and their frequency are invariably ill-defined. They will often vary with the type of building, the type of activity and the number of occupants. Even different rooms within the same building may have different stress levels, e.g. bathrooms and kitchens, generally have a requirement for a greater resistance to water than the living room.

Animals, insects and plants may also cause deterioration of materials. In the case of animals and/or insects the material may be used by them for nesting or to support life; both adversely affecting the material's ability to function. In the case of plants, it may be necessary to establish whether the material, in the defined conditions of use, will encourage and/or support life, with the possible need to provide long term biocidal treatment, as in some cases with timber.

Available tests

Tests may be put into two categories, predictive and indicative.

Predictive tests. Where the rate of reaction of a material to a given condition is controlled by the laws of thermodynamics, it is possible to extrapolate test results to longer periods of time and to convert behaviour under accelerated test conditions to behaviour that would be obtained under service conditions. Chemical reactions, thermal degradation and some physical changes are of this type. Unfortunately in many circumstances, the interest is not in the rate of the chemical or physical reaction but in the resultant change in the mechanical properties of the material. These changes are not necessarily proportional to the rate of chemical or physical change (for example, failure of an adhesive due to hydrolysis) and instead of following the rate of reaction it is often necessary to follow the change (degrade) in mechanical properties against time of exposure to an aggressive environment. The time of exposure required under test depends on the service life target and the degree of acceleration that can be used in the test. It is essential that the test is carried out over a sufficiently long period of time to establish with reasonable certainty that a sudden, unpredictable collapse of the material will not occur.

Acceleration of a test by performing the test at elevated temperatures or at increased concentration of the aggressive media should only be undertaken after it has been established, perhaps by a preliminary investigation, that the acceleration does not produce reactions different in nature to those which would occur under service conditions.

It is always reasonable to carry out an accelerated test at the maximum service temperature or at the maximum concentration of the aggressive media that is likely to occur in service, although these conditions may only occur over a short period of time in the life of the material (see indicative tests below).

Different reactions and/or different rate of reaction sometimes occur with materials under stress and where this is a possibility, the tests must be carried out with the material in such a condition. It may be reasonable to use the maximum stress that will occur in service, but higher stresses should not be used before the possible effects of this are established.

Indicative tests. These are tests which usually cannot be accelerated but may employ the most unfavourable service conditions. The results of the tests cannot usually be extrapolated to show behaviour that occurs following an increase in the test time.

The tests are two types: performance type tests such as freeze/thaw, fatigue, abrasion, impact resistance, etc. and tests which measure a physical property of the material which, related to prior knowledge of it, gives information on how the material will behave in practice. For example, for ceramic materials, the compressive strength, partial or total porosity, pore size, distribution and permeability, all give information which separately or combined may be used to assess the likely performance of the material in service.

Points that have to be remembered are:

1. The performance test must reproduce as accurately as possible the service conditions, i.e. a wind fatigue test would simulate a three second wind gust.
2. A test which has been demonstrated as being useful in indicating the behaviour of one material may not be suitable for use with other materials or even for use with the same material in other environmental conditions.
3. Any acceleration of the tests should only be undertaken after it has been established that the acceleration does not produce effects that are different in nature to those that would occur in service.
4. The tests may have to be repeated on aged materials if the results of environmental ageing show that significant changes in properties will occur.

Service life predictions

The accuracy of any service life prediction depends upon:

1. A rigorous analysis of the environmental and non-environmental agencies that the product will be exposed to in service.
2. The development of a comprehensive test programme to obtain information on the effect of those agencies which cannot be eliminated by a knowledge of the chemistry or physical nature of the material
3. An informed interpretation of the test results
4. Assessment of any other relevant data
5. Judgement on the available data based on experience

In very few instances are the service conditions known accurately enough to allow a simple extrapolation of accelerated ageing tests to predict the ultimate service life of a material. Almost invariably, only the extremes of the service conditions are known and the time spent at any particular condition between these extremes is unknown. It is impossible in this situation to establish the ultimates service life of a material accurately. However, it is often possible to establish a minimum life by simulating in tests the most severe service conditions and comparing the results with naturally exposed samples to obtain an acceleration factor. Another approach is to establish and examine the slope of a deterioration curve against time. If, after a sufficiently long period under test at or above the most severe service conditon, there is either no significant change in the material's properties, or the properties are still well above those required in service, then again a minimum life can often be assigned to the material. If under these conditions there is a significant and continuous change in the properties, then there is a risk that, in service, it will not achieve the required minimum life and either the material has to be restricted in its use to a less aggressive environment or, if this is not possible, rejected.

Conclusions

The assessment of the service life of a material requires consideration of changes due to environmental and non-environmental agencies to which it is exposed in service. An accurate life prediction can only be made if all the service conditions can be accurately defined.

Quality assurance and certification

From the foregoing text, it will be clear that there may be many uncertainties to be accepted and dealt with in predicting the service life of structures. It follows, then, that any opportunity to reduce uncertainty should be firmly grasped, leaving aside the possible legal and financial implications of not doing so. Reasonable assurance that the design/specification of the structure is achieved in practice can be readily obtained through conventional (standardized) practices. The BS 7500 series and the related CEN and ISO standards provide a suitable basis in most cases. Note that the Standards relating to quality management systems leave conformity to specification (material, product or structure) still to be dealt with; for example, through kitemarking where appropriate Standards exist or Agrément in cases involving innovation. The quality management system can extend beyond materials and products into design and site procedures and this can make a further contribution to ensuring that the intended design and specification for the structure are achieved in practice. The procedures on their own provide only a framework within which committed and efficient staff can achieve the desired, practical results.

The question as to whether formal certification—of the management system, materials specification and product performance (including service life prediction)—is required, where the client allows an option, is very much a matter of the circumstances applying in particular cases, taking account of the novelty of the structure and the circumstances in which it is designed, constructed and has to fulfil its function. It can generally be agreed, however, that the discipline involved in adopting such formal procedures can be beneficial. Where third party certification is involved, a further benefit, spreading, if not shifting, of responsibility for the structures can be achieved. For the future, it seems likely that for structural applications, third party certification of materials and components will be required under the Construction Products Directive, thus removing an option currently available in many cases.

For efficient companies, quality assurance is a way of life and third party certification a formality; it is important, however, that the formality confers on the company a commercial benefit and does not become simply a burden. For inefficient companies, quality assurance and certification are nettles they will be under increasing pressure to grasp if they wish to continue competing, let alone winning, in business.

The contractor's contribution to design life

R.B.WOODD

Introduction

The design life of a structure is the intended minimum life with reasonable maintenance. The contractor's contribution to the actual life of a structure may be good or destructive; but his contribution to the 'design life' is through the practical feed-back he provides into the design from his experience. Design modification, or even over-design to cater for human fallibility is as valid as allowance for corrosion.

The presumption is that the end of a structure's life is signalled by material failure; but more often than not it is a change of use or attitude (i.e. an aesthetics rather than a material factor) which signals the end of a building's life. Buildings are usually demolished because of dated looks, inadequate floor to floor height for better servicing, or loss of original use rather than because the fabric is seriously worn.

Design life then has two extremes, form and materials; and these have a common element workmanship. Generally public buildings like town halls, law courts, churches and grandstands can be seen as having extended lives and therefore merit better form, better materials and better workmanship; other buildings may have less quality or more economy in all three areas! For certain buildings, e.g. industrial, flexibility may well be the most important single function giving another aspect to be considered if the building is to have a long life.

The same elements apply to structures such as bridges, dams and tunnels except that here the life expectancy is usually greater and the abstract element is usually one of design assumptions. Good looks seldom seem to play a part.

However with these types of structures the consideration of maintenance by the designer should be a major factor. Is it or is it not likely to be necessary? Should maintenance access be designed in? What can be done to ensure material survival by over-specification if necessary, where maintenance access is unlikely? These questions need to be addressed with experience and honesty and may then shape the structure.

It is said that 'a chain is only as strong as its weakest link'. There are a great number of potential weak links on the construction site because nearly everything in building is a prototype. Even if it has been used extensively before, it has not been used in this location, set of circumstances and with this person installing, feeling as he does today.

We know only too well from both major and minor examples how much the human being contributes to the whole that results at the end of the construction process. Examples are:

- Liberty ships splitting on accidental weld lines
- Reinforcement misplaced during concreting
- Idealistic detailing of system building cut away to make erection 'possible'

The design and materials may be marvellous of themselves but a little bit of man's input is unavoidable and fraught with risk.

The contractor's role

The contractor is the manager of the site-based physical human input to construction. It is fashionable in this age of litigation to believe that everything should be perfectly performed, and this thinking comes from the mass production spectacles we look through. I remember the dissatisfaction of looking through trays of handmade tourist statues, silks or metal work in India for a perfect example. However much I told myself they were handcrafted on a limited budget, I still hoped for the soulless perfection of machine made.

Buildings and structures are handmade and assuming 25% of the cost is site labour then a £10 million structure involves 100 man years of site work. There will be a lot of opportunity for human wandering in that. What sort of examples are to be addressed? Some instances are:

- Reinforcement offset during concreting
- Bolted connections not tightened
- Excessive or inadequate application of mastic sealant
- Inadequate preparation of surfaces before over-laying
- Rain reaching sensitive materials

These are not all the result of carelessness. Ignorance or practical difficulties can contribute just as much.

In the normal situation the high safety factors both in design and detailing can give a false picture regarding the quality of site workmanship actually commonly achieved, or practically achievable on a site.

So people may be imperfect, it is the contractor's task and contribution to manage them in such a way that their imperfections are contained, spotted and put right. In other words to provide good workmanship. Human error is no plea under the contract even if this is only as realistic as saying that ignorance is no plea under the law (when the law is only discovered by being repeatedly tested in the Courts). The contractor's final contribution is to take on board the risk of the workmanship of his employees.

Does good workmanship stand alone and apart from design? Good design should recognize human fallibility and site practicalities. It is in the sensible emphasis to the designer of this on the one hand, and control of site workmanship on the other hand, that the contractor makes his contribution to the 'design life'. It is quite clear that bad design and/or bad workmanship are hard put to make a long life building. But good workmanship can mitigate poor design and bad workmanship can destroy good design.

Workmanship

So what contributes to good workmanship? A few items are:

- Clear sensible and understandable requirements (specification)
- Good communication
- Adequate tools and environment
- Adequate time
- Good motivation and supervision (management)
- Proper experience and training

Clear and realistic specifications

Most specifications are written as an aim, not a genuine requirement to be enforced, with the result that contractors in pricing anticipate that 'sensible' relaxation is realistic. This is a slippery road. Inevitably the detailed specifications cover the tried and tested materials. The tricky new skills are often the province of specialist subcontractors and covered by a single performance requirement.

Good communication

To know what you are supposed to be doing and why is a very good start to getting it right.

Adequate tools and a proper environement

This is relatively straightforward and usually within the scope of management but often the designer lives in an ideal world, e.g. specifying high shrinkage lightweight insulation blocks for the inside skin of external cavity wall construction with the requirement they must not be allowed to get wet but need to be built in February, March and April. The result is often that a degree of chance combines with a degree of risk taking in the quality of the finished product.

Adequate time

This will vary with each individual but in every case a rush is counterproductive as is a stop/start régime. The Friday car typifies the problem.

Good motivation and supervision

Man management courses remind us all how important good motivation is and that the motivation that produces a good job is usually more than just money. Supervision rightly forms part of this motivation.

Proper experience and training

Training is an important issue which receives great lip service but do we now sometimes train for training's sake not with any serious intent to get a job done better. The training that comes from experience gained in doing the task is very much better than anything that comes out of the training school.

These aspects are input items to the quality situation and in general are the province of management in setting the context within which the operative can achieve. The next stage is to try to police the workmanship.

Quality control

The old fashioned method of policing was management and trade discipline combined with experienced quality control. This essentially meant autocratic managers with experienced operatives and informed supervision. This is no longer seen as an available recipe and Quality Assurance has crept in as the currently perceived solution transferred across to the site from the arena of mass production.

If the old system worked in the past has it been found wanting or have circumstances changed? Probably some of each, but perhaps the latter is the more significant.

What does site based QA provide? It provides a régime of rigid discipline in that instructions or procedures once set down must be adhered to and a *laissez-faire* attitude is stamped out by third party audit.

This discipline in writing procedures and ensuring they are carried out is a good thing and clearly works extremely well for major happenings, repetitive production items or a transparent thing like a design. Clearly in construction the management framework, e.g the methods and procedures to be used and around which safety depends, is transparent and many of the components are mass produced. So all these can readily be subjected to QA discipline with advantage; but what about the site workmanship, the care and expertise that lead to durability? These remain in part outside the scope of QA to police. QA can only require a person to be suitably experienced and monitor some of his output at fixed points. It does not 'help' the poorly trained operative to lay bricks, vibrate concrete or lay asphalt better whereas it does help his manager to plan, organize and monitor better the work in his charge.

The need is for people who have genuine practical experience combined with the motivation and conscience to put it into practice. A major contribution required from contractors is producing these people. It is not for others to produce them as a contracting resource because this suggests that classroom training would be adequate.

Contractual organization

Currently the management role is in ascendance to cope with the construction aspects of finance, sub-contractor coordination and man management to achieve fast programmes, i.e. all the short term immediately perceived needs. The layers of management needed to run a contract with a management contractor employing a main contractor employing a subcontractor is mopping up in paperwork the people who used to be engineers. In the short term this means experience may be spread too thinly and too remote for comfort. Seniority comes before experience is gained.

The last 10–15 years have seen great changes in construction organization and change usually takes time to settle down before any distilled good emerges. But it is wise to remember that the game of competitive tendering is only calculated to give the work to the wisest and best in an idealized 'gentleman's world' full of 'experienced contractors'.

The perceived opportunity for gain does a great deal to encourage unrealistic optimism and self-assurance from both client and contractor. The contractor's contribution should therefore be self-restraint in only undertaking what he can resource competently.

Buildability

It is clearly sensible that the designer ensures (irrespective of the specifications and drawings) that the important elements of the design are recognized and realized in the construction. Similarly it is the task of the contractor to try to ensure that the human elements of workmanship are recognized in the design. A prime example is the matter of tolerances for fixing external cladding systems where too tight an allowance is as poor design as expecting instant answers to design queries is bad contracting.

The contractor should contribute to the design process the constant reminder of practicality or 'buildability' and the designer should be willing to listen and accommodate this input.

Another aspect of practicality is the control of changes. Theoretically the contractor can cope with changes by proper organization but the reality is that there is always a persistence of vision which leads to uncertainty and error, i.e. non-buildability. Managing change is commonly considered to be part of the contractor's role when perhaps it should rather be resisting and controlling change.

Design responsibility

Only the practical aspects of construction are considered because I have taken the view that any design passed to the contractor, whether in the form of performance specifications, specialist sub-contractors, construction joints or temporary works remains a part of 'the design' as a whole. As such it should not merit different treatment from the rest of the design. However there will be practical matters such as concrete mix design, curing methods, mortar consistency and steel erection procedures where the contractor will contribute his own practical knowledge to the end result.

Conclusions

The contractor's contribution to the design life of a structure is through the input into the design of his experience of what is required to ensure that the design intent is achieved on site. To encourage this assurance the contractor needs to

(a) Set up a framework for providing managers, supervisors and operatives with both training and experience.
(b) Input practicality into the design detailing.
(c) Control design development and ensure adequate construction time.
(d) Only take on work he is competent and resourced to plan and supervise.
(e) Use self-imposed disciplines such as quality assurance where appropriate: This may be by imposition on others.

Where design life is important the client should take these matters into account when choosing his contractor. This may mean recognizing both cost and time factors.

Summary of presentations and discussions

D.A.HOLLAND

Mr Stillman hoped that the new UK Code, which was now out in draft for comment, would be completed by the end of the year. Although the Code excludes large civil engineering works, one of its key functions is expected to be to improve communications by the use of standard methods in discussions of durability. Thus, the definitions in the Code are important, in particular that of service life by which was meant the actual life during which no unacceptable expenditure on maintenance or repair is required. The client's required service life and the predicted life of a product are brought together by the designer when determining the design life. Tables in the Code provide guidance in the selection of a required life, the classification of components and the definition of maintenance. Recording these for each element of a building on a data sheet provides a statement of the conditions attaching to a particular choice of design life.

Commenting on indemnity and liability, he drew attention to the trade off between the more secure prediction by way of design life and the additional cost of more robust construction.

Closing his introduction, Mr Stillman argued for innovation, rather than the conservatism born of fear of premature failure, and the environmental benefits of more durable buildings.

Mr Webber reminded delegates that not all structures in the public sector were required to have a long design life. He cited military facilities and office accommodation for the civil estate as examples. On the other hand, the Government estate had a very large number of buildings with a much longer life than that common in the commercial sector: examples included the Houses of Parliament, Truro Crown Court and the QE II Conference Centre.

Some structures lasted longer than expected. The glass house at Kew was a good example of the assumption that long life required a reasonable level of maintenance. This particular building was well into the second peak of its maintenance cycle. Another example, Woolwich prison, had survived because of its robust construction. It, like many of its Victorian counterparts, was now undergoing an extensive refurbishment programme. In other cases, the disposal of robustly built structures at the end of their design life could prove uneconomic.

Mr Webber referred to maritime facilities and the need for conservatism in their design, particularly when selecting covers for reinforcement. He quoted the examples of the trident jetties and the RC dam across the mouth of the dry dock housing the Mary Rose.

Mr Webber's final illustration was of an airfield runway where typically a design life of 15 to 20 years was required of the bituminous materials, after which resurfacing would be needed. Binders that would allow the interval before resurfacing to be extended, were now being sought.

In Mr Fletcher's view, the private sector client was someone who commissioned a structure and obtained his revenue from activities within and around it. A number of risks are of concern to clients; procedural, completion, construction cost, operation and maintenance, and irregular maintenance. Of those, irregular maintenance, and in particular change of use, could have the most significant effect on the payback period. Although change of use was the client's responsibility, the prediction of irregular maintenance is a designer's responsibility. Similarly, the designer is responsible for the risks in materials and techniques, and for explaining to the client the implications of innovation. He was also of the view that the designer should be responsible for inspection during the financial life of a project.

Mr Fletcher hoped that the colloquium would provide some of the answers to the evaluation of residual life. In the private sector, where financial life was more important than design life, the residual value of a structure was of particular significance.

From a practitioner's point of view, current design methods are effective in the sense that they generally satisfy customer expectations on the life of structures. However, as pointed out by Mr Stevens, clients are not very aware or sophisticated, which may account for the fact that the life of structural works is rarely specified or discussed between a prospective building owner and his designer. Designers and clients do not share a common understanding about the expected life of building structures. The designer has insufficient information to enable him to devise a form of structure with material ingredients that will have a life predictable within narrow limits of time or cost. For most owners, the capital cost of a building will dominate its design. That makes sense if no further costs are expected during the life of a building. It is often the case that owners have insufficient experience of the range of performance of building structures for the question of maintenance and repairs to become much of an issue with them. Interest in design life will stay at its present low level until designers can produce convincing arguments about their ability to accurately predict life and performance as a basis for evaluating one design against another. To meet that challenge requires the collection of performance data and statistics, a task that may be theoretically possible but economically impractical.

The design process depends on the confidence of the designer and the client in their relationship. The early involvement of the client, both at the conceptual stage and during design reviews when the arguments and reasons for conceptual design decisions can be explained, would eliminate a lot of the surprises that can occur later on in the design process. In the discussion following these four papers, it was recognized that the designer's interest would depend on his financial involvement for what he has been commissioned to do. Nevertheless, it is important, for feedback and experience, that involvement with the project should continue after completion. This is particularly important if the designer is to be responsible for making clear to clients the limits of his knowledge and the reliability of the materials he is choosing.

An interesting parallel was drawn with shipbuilding, where the rules are maintained by classification societies rather than by means of updating Codes of Practice. The close involvement of surveyors to check designs, oversee construction and workmanship, and in the resurveying and reclassification of ships, ensured the feedback of experience and performance, and maintained the links between owners and classification societies.

Doubts were expressed about the responsibility of the designer for decisions on durability. Designs are not made for a 120 year design life but in accordance with standards of uncertain quality. These are drawn up by British Standards committees, which contain a variety of differing interests rather than having any long term responsibility.

The question of the designer's liability was raised: in particular, whether by designing explicitly for a life of, say, 120 years, his liability should extend for a similar period. A number of differing views were expressed, ranging from those advocating a limited period of liability of, say, 10 to 15 years, to those suggesting a designer could carry liability throughout the whole life of a structure, but only when the designer's role involves continuing inspection and recommendation. Others argued that the designer was not responsible for the actual period of a design life, but simply used it as the period over which the statistical analysis of applied loads was carried out.

Another contributor argued that designers should be more conscious of what they are doing when deciding to build or construct in a particular way. The process of deciding on a design life and designing to it, is not compatible with the use of Codes of Practice, which are not always up to date, in terms of the performance of different materials over a period of time.

Another speaker was surprised to learn that design life was such a woolly concept. Systems for assessing and certifying design life had been available through BBA for some 25 years. The certification process for both individual components and structures was a means of recording conclusions drawn from technical evidence and experience. Those conclusions were subject to legal tests of negligence.

The concept of design life has to be handled carefully in practice, otherwise lawyers could have a field day. At best, it is an intelligent guess and not an accurate forecast. It is a means whereby the designer and the client can gain a better understanding of their respective expectations, and it brings into the open the need for maintenance and the replacement of components.

Another speaker pointed out that designers do not guarantee the life of a structure; what they are doing is all that can be done professionally at the present time to ensure the life will be as long as that required. It is important that those limitations and qualifications are properly understood. Engineers cannot take responsibility for events over which they have no control.

It was suggested that the liability of all parties should be set at five years, provided a special durability assessment of a structure was made at the end of that period. If the structure passed the assessment, the supplier and the contractor would be freed from any further liability. This speaker also suggested a special building damage fund. This would apply in the public sector and would require a levy of 1% of construction costs, half of which would be used for the durability assessment, the other half to cover the repair of damage that was undetected by the assessment.

The need to distinguish between technical service life and economic or financial service life was highlighted. Economic evaluations can only be made in present day terms: comparisons are only valid as relative measures to select between alternative solutions. They cannot be considered to be valid in the long term. An example of this present day relativity (with obvious political implications) was provided by eastern Europe, where design concepts are identical with those used elsewhere in Europe except that safety provisions have been lowered systematically by 10%. Only time will tell which approach is the correct one.

Mr White drew attention to the difference between a client's expectations, that a structure should last for ever, and the reality of the consequences of inadequate maintenance. In his opinion this highlighted a common failing, the lack of consideration of a structure's performance throughout its life. Performance, allied to the expenditure over time required to maintain that performance, provided the basic concept of whole life costing. This is a technique used by very few; yet without such an approach there is no rational means of communication between a designer and a supplier, and no reliable means of testing in-built material performance assumptions.

He proposed a building performance profile, developed by the client and the design team. From this a performance management plan is derived to take account of the implications of deterioration over time, and a discounted cash profile, permitting comparison of alternatives in whole life cost terms, is established. The role of the client is essential if design life, and more particularly whole life performance, is to be achieved. It was equally important that every project should have a performance management plan.

In the absence of Dr Chan, his paper was introduced by Dr Somerville. Drawing lessons from the paper, Dr Somerville pointed out that, although both loads and resistance affect reliability and performance, comparatively little work had been done on loads, but much on resistance. What had been done on loads was inconsistent in the use of nominal values for imposed loads, and statistical data for wind and snow. He thought that, in relation to loads a greater effort was called for. At present, for durability, codes tended to mirror the nominal load approach for imposed loading, by offering simple classifications of the overall environment.

One area worthy of further research was that of the interaction between macro- and micro-climate. The classic example was de-icing salt. There are, however, different types of interaction: for instance, maritime areas where the abrasive effects of physical impact and chemical attack formed an integral part of the general environment.

In summary:

- Not enough is known about loads
- What information we do have is not structured for design purposes
- We are still at an early stage in recognizing the various types of interaction involved
- We need to move forward on the loads front, irrespective of the development of design life concepts.

Dr Schlaich drew a parallel between the present debate on design life and earlier ones on structural safety where two groups of measures, probabilistic tools and avoidance of human error, were identified: safety theory started after real errors had been ruled out. He argued that the prediction of a theoretical design life was possible. Many phenomena are amenable to probabilistic analysis: others depended for their avoidance on the skill and experience of the designer and builder, and on the reliability of the methods they used.

Implicit in the concept for predicting design life was the acceptance of a limited lifetime, the selection of appropriate materials, and designing the structure as a whole such that its overall lifetime does not fall short of the lifetime of its materials. The ageing process is to be seen as one of a continuous and controlled degradation.

The avoidance of sudden unexpected errors or failures relies crucially on the quality of the conceptual design, and on detailing. This requires the proper education of engineers and a healthy suspicion of black box formulae and rules, as well as awareness of material development and the quality of workmanship.

The watchwords in the search for a defined design life are robustness, replaceability and forethought: the latter being of particular importance where change of use could be a client's requirement.

Mr Jubb explained that, for the last 25 years, BBA had been developing routines for assessing the service life of materials, components and systems. The ultimate goal was to ensure that, at the end of the service life, material components have the characteristics anticipated. This required an understanding of the agencies affecting materials, and the limits of variability when dealing with specific cases. This presented a challenge which could be met at least in part by simulative and accelerated testing; experience was also an essential ingredient. Behaviour under extreme conditions could be examined using specially designed 'torture' tests. At the practical end, on-site operations have to be reliable if the criticality of the installation process to the performance of components is not to be put at risk.

Service life prediction is based on a range of appropriate tests, but under the routines established by BBA, tests are only one ingredient. They can only contribute to the extent that there is a correlation between test conditions and service life conditions.

Finally, Mr Jubb commented that quality assurance was a useful basis for monitoring performance, and provided a reliable framework for feedback.

Mr Woodd argued that the contractor's contribution was a total one, deriving from the need to understand fully what was intended to be achieved. He was critical of unnecessary complexity, which should be designed out, and of designs requiring suspect methods or materials. What was needed was a little bit more money from the client and a bit more care from the designer. However, the intention must not be to make designs too refined; to do so is not in most people's interest.

Mr Woodd questioned the processes of selection and supervision of contractors. The principles that he advocated were to resist greed, to input practicality, to provide and use experience, to employ available resources, and to control design development. A contractor's offers are worthless if he lacks substance.

The discussion following these papers was based on two aspects of service life which had been singled out; the first, based on statistical evidence and experience, was related to the performance of materials and the surface of structures; the second, which included sudden deterioration or unexpected events, related to the whole structure.

In response to concerns about responsibility and liability for accelerated testing, it was pointed out that neither BBA nor manufacturers have any problem identifying appropriate tests. Publication of test results and assessments implies liability in the case of negligence, although it should be remembered that an Agrément certificate, for instance, applies to uses claimed by the manufacturer. Members of the Agrément Union had agreed standards to be applied to materials and systems. Confidentiality, particularly of composition of materials, was recognized as important if innovative engineering was not to be discouraged. However, this placed some constraints on the form and duration of performance certificates. It was suggested that quality assurance should provide a better framework for dealing with all aspects relating to materials and workmanship.

Certification was seen as placing a potential limit on the usefulness of structures, if it is unable to accept change in adapting existing facilities to new uses. Formal investment systems also make such flexibility of use difficult. On the same theme, reservations were expressed about the more widespread adoption of the design and construct approach, where it was considered unlikely that anything other than what had been specified would be provided, without explicit instruction. It was pointed out that in English law fitness for purpose is implicit; design life liability might apply there as well. In many cases it had to be the responsibility of the creative designer to take decisions for the client in anticipation of likely changes of use, the two cases quoted in Dr Schlaich's paper being good examples.

Views differed about the approach of clients to the question of change of use. Some individual clients only want what is perceived today, although others were prepared to adapt and stretch an initial design. Building clients have been receptive to arguments about catering for the future on the grounds of residual value. Here, value engineering has an important role in assessing and promoting viable options.

The example of the adaptability of some old bridges was cited, where the loading effects which they now carried were some three times higher than those they had been designed for. On the other hand, it was pointed out that, in general, lightweight structures were less adaptable. The possible exception was that of steel structures where load capacity could be increased by, for example, adding cover plates. In these cases extra strength, in terms of whole life costs, could be achieved by additional provision some time after they were constructed.

The essential role of site control was commented on, particularly with respect to the sensitivity of materials to site conditions or operations.

Innovation in both materials and methods is dependent on the management of operations. Here speed and money have a role to play, although it must be remembered that speed without money is inversely proportional to quality. Nevertheless, if systems existed to ensure that quality was achieved in practice, performance would be expected to improve. This bore on organizational mechanisms.

It was recognized that BBA was reducing the risk that a product would fail. Nevertheless, treating a complete structure as a product raised the question of what BBA routines for components could be adopted and adapted for whole structures.

Attention was drawn to the third party testing and assurance systems in France and Belgium, although there was a cost (1 1/2% of building costs) for insurance, and a requirement that the design and construction processes were independently checked. Although this was being introduced into the UK, doubts were expressed about how popular it would be, and whether people would pay for it. It did, however, provide a quality control system.

The DTp Technical Approval system was cited as a QA system for complete structures.

It is possible to apply the BBA routines to a great number of material combinations, but care is needed regarding combinations or novel elements.

The role of installation processes and quality assurance were identified as critical issues for some products. BBA can indicate the role of approved installers where experience should reduce the possibility of inadequate performance.

There is a lack of acknowledgement of the many failures leading to reduced performance: de-icing salts and Ronan Point were given as two examples. Here the totality of life was a problem worth investigating, including the influence of factors outside the normal procedures of design and construction. Data should be collected on what works in practice, as well as on what does not.

Durability audits could be applied to structures to assess whether the design will achieve the desired design life whilst changes can still be incorporated, and later to assess early performance of the completed work in relation to the design life.

B How particular industries cope with design life

What can we learn from marine structures?

J.C.CHAPMAN

General

Let us first formulate some of the questions and considerations which need to be addressed in respect of land structures. Then we can discuss what might be learnt from marine structures.

The specification of design life requires that durability must be quantified, that is time-dependent phenomena must be considered and assessed; this of course is already done in many instances. The significance of the current interest in these matters seems to be that durability should be specifically addressed in all construction.

The circumstances in which it is feasible or desirable to specify a design life, the methods of specifying and assessing life, and the contractual obligations resulting from the specification, are matters which will doubtless be discussed at the symposium.

Is there a useful distinction to be made between design life and durability? Perhaps it is that design life refers to the structure, and durability refers to the components and protective coatings. Some components and coatings can be replaced, perhaps many times, in the life of the structure. A given design life may be achieved either by the use of durable components, requiring little maintenance, or by less durable components requiring more maintenance.

Land structures form part of the shared environment, so the public exercises some control over the location, size, purpose, functional standards, facilities and appearance of structures. Appearance depends on opinion, quality of design and on quality and durability of materials. A short life structure will usually need to be cheap, and is unlikely to meet the highest quality standards.

Quality standards vary according to location. Lower standards are accepted in designated 'industrial' areas. Industrial structures may be process structures, or protective enclosures for processes. In areas designated as industrial, the lifetime requirement of the process is allowed, subject to various stipulations, to dictate the design life of the structure.

Buildings can be made portable, either as integrated modules, or as components. Such buildings might be called multi-life. Buildings with adjustable spaces have a greater life potential. In principle, office buildings should achieve a longer life than buildings for manufacture, because headroom requirements are more uniform.

Housing is the largest element in the built environment. Quality and durability should be the aim.

Variety is an essential element in environmental acceptability, so technical optimization which results in uniformity should not be the objective. Uniqueness, which is the limit of variety, has a special attraction, which does not necessarily connote aesthetic merit.

Ancient structures are venerated. Antiquity requires durability and worthiness. What is the contemporary contribution to heritage? Will our successors be grateful only that our structures were not too durable? We enjoy the past, should we not pay something for the future?

In buildings the structure is usually protected from the external environment. The cladding, fittings and services are most prone to deterioration. The role and influence of the structural engineer in these respects are secondary. Integrated design teams provide more scope for diverse talents.

High quality, long life construction contributes to the conservation of materials and energy.

Requirements and influences affecting life

The required life is related to the life of the process or function. These are subject to changes in demand, obsolescence and deterioration. Long life implies durability. Quality is consistent with built—in durability, which leads to high first cost and low maintenance costs. High interest rates, combined with low capital allowances and tax-deductible maintenance costs, encourage poor quality and insufficient durability. They mortgage the future.

If the life requirement is not indefinite, the owner of the structure will specify the required life. Examples are:

- Fixed offshore structures, where the life depends on the estimated life of the oil or gas field
- Nuclear reactor structures, where the life of the process equipment is limited
- Ship structures, where machinery and other equipment deteriorates and the trading demand changes
- Bridge structures, where the specified life is related to foreseen replacement capability (though more logical justifications might be adduced).

On the other hand property owners (especially home owners) will wish to assume that the life is indefinite, and mortgagees will need to be assured that the life exceeds the term of the mortgage by a sufficient amount.

The public interest requires high quality long effective life construction, that is the construction system should be conservant in respect of energy and materials. Conservant construction can be achieved by various means. Quality, durability and maintainability are fundamental. The ability to adapt, dismantle, relocate, or recycle, might also be important.

Responses affecting life

Corrosion in its various manifestations is the largest, most pervading, and least tractable response phenomenon affecting durability. Corrosion depends on the process environment, on the ambient environment, on, protection, and on maintenance. Corrodible parts may be accessible for maintenance, or inaccessible.

Certain corrosion processes can be quantified on the basis of assumed rates of corrosion. Examples are the annual thickness loss for unprotected steel in seawater, and the life of galvanizing depending on thickness and the nature of the environment.

There are accepted levels for contaminants in concrete which are thought to give an indefinite life in protected environments, but there is no accepted basis for quantifying life, as a function of composition and environment. The difficulties arise from the variability of natural materials, the non-uniform distribution of detrimental constituents, the problems of sampling, lack of data on the effects of service temperature and humidity, and lack of data on the relation between composition, environment and life.

If steel is protected against corrosion by metallic coating, the coating may cause embrittlement, depending on initial defects and on the environment. The durability of the coating depends on the environment.

Recognized procedures exist for the calculation of fatigue life. Although these procedures are inexact in respect of both loading and response, they do provide a basis for the quantitative specification and estimation of fatigue life.

In engineering structures the magnitude, frequency and sequence of loading are uncertain. The detailed distribution and magnitude of stresses in welded connections, for known loads, are also uncertain. The quality of welds and associated defects are difficult to control and to detect.

A small reduction in stress leads to a relatively greater increase in fatigue life. In general it will be prudent to design for a rather long nominal fatigue life. Fatigue prone land structures are generally designed for lives in excess of 100 years. As welded structures have only been in use for about 50 years, we have no service confirmation of the overall adequacy of current design procedures. We do however have some experience of premature fatigue failures. We must therefore rely principally on experimental evidence, which is not comprehensive, and usually relates to very simple connections. Creep in structural materials at ambient temperatures and service stresses can usually be accounted for approximately by assuming a reduced stiffness. Time-dependent ground movements are influenced by many factors, both natural and mancaused.

In general, we have little knowledge of the correspondence between assumed design loadings and actual loadings, or between assumed environmental conditions and actual conditions. In general therefore, satisfactory service experience can only provide confirmation of the overall adequacy of design methods.

Ship structures

The ship has much in common with a building, but it should also float in all but the most unlikely circumstances and it must be propelled at an optimum speed, which depends on its function, at optimum efficiency.

The Royal Institution of Naval Architects is principally a single product, multi-discipline Institution. In this respect it has something in common with the Royal Aeronautical Society. The former has extended its interests to offshore structures, the latter to aerospace. These Institutions differ from other engineering institutions, which are either multi-product, multi-discipline, or multi-product, single discipline.

The naval architect, who is an engineer, is responsible for the total design of the ship. He is assisted by mechanical and electrical engineers and by interior designers. The naval architect must concern himself with the functional requirements of the vessel, with the optimization of hull form in respect of stability, resistance and sea keeping, and with structural design. Thus the naval architect (like the land architect) has prior concerns with non-structural considerations, and structural form is determined principally by internal and external functions.

Maximum stresses due to cargo and to waves are of approximately equal magnitude. The structure will be more resistant to wave action in some loading conditions than in others. The loading on the ship is in some respects controllable by the owner and the master:

- The magnitude and distribution of the cargo
- Bad weather can to some extent be avoided
- Speed and handling are at the master's discretion

Weather avoidance depends on the judgement of the master, or company policy, and on the accuracy and reception of forecasts.

The anticipated life of a ship may typically be twenty years. The structure is subject to a spectrum of wave loading of which there is much knowledge, and to internal loading on which limits are set, and which is recorded in the ship's log. Actual weather conditions are also recorded.

Structural design is based on the rules of a Classification Society and the design is checked by the Society. A surveyor of the Society is present during construction and the Society makes periodic checks on the condition of the structure during service and on repairs carried out.

There is therefore a feedback of experience through design, fabrication, construction, operation, maintenance and repair, to the design rules, which does not exist for land structures. In particular there exists a potential data bank of quantifiable fatigue performance. Fatigue cracks and fatigue initiated fractures do occur and are repaired. Much of the structure is readily visible and the crews have a rather personal interest in inspection, maintenance and repair. It cannot however be said that the standard of maintenance is always as high as the circumstances would appear to merit.

The approach to fatigue cracking differs from that for land structures, where the design calculations are nominally aimed at avoidance of cracking for the specified lifetime. It also differs from that for offshore structures, for which extensive fatigue calculations are routinely undertaken. Ship design is generally based on limiting stress levels and on avoiding avoidable discontinuities, though fatigue studies may be undertaken for novel structures. The following reasons may be offered in support of the empirical approach to fatigue in ships.

(a) The design-performance feedback provided by the classification society system allows the empirical rules to be validated by experience.
(b) The rules are specific to particular classes of ship.
(c) The ship is constantly attended by the crew. The ship can be taken out of service periodically for maintenance and repair. Fatigue damage can be repaired at the same time as damage due to other causes.
(d) A ship is subject to several other hazards such as fire, collision, grounding, foundering, explosion, cargo handling, in addition to structural failure, which is therefore more readily accepted as a normal hazard by owners and insurers.
(e) Fatigue cracking does not usually lead to failure, unless the crack initiates brittle fracture. Surprisingly, Grade A steel is still allowed outside certain parts of the ship, such as the mid-length region.
(f) Although wave bending moments can be predicted with reasonable accuracy, much greater uncertainty attaches to local wave effects and to green-water loading on deck structures.

Whereas service performance is in principle well documented, some total losses occur for which the cause is never discovered.

The classification society system also has some disadvantages. There is strong competition between societies and it seems that a society is liable to lose business if its rules are more stringent than the norm. Also, there is a natural reluctance to introduce more stringent rules than hitherto, because of the implication for existing ships designed according to the previous version. Nevertheless some such adjustments are made.

There is of course room for improvement in standards of design, construction, operation, and maintenance. But it should be said that to design, build and operate these enormous steel structures, which are subject to severe, complex and variable natural forces, is a daunting task, and would be much more so without the experience which the classification societies provide.

The effects of uncertainties of loading and stress distribution can be reduced by using strain measurements as an aid to operation. Strain gauges at key locations were in fact installed in the first 900 ft long container ships. For the early voyages, the masters were instructed to reduce speed if the strain exceeded a certain level. The requirement was relaxed as confidence grew.

Bending moment estimates are supported by strain measurements which have been made on several ships. The gauges measured and counted the number of times the given levels were exceeded, with sufficient intervals to construct moment spectra.

Whereas a 20 year life is usually envisaged, the guarantee period is usually about one year.

Offshore structures

These may be fixed, vertically tethered, anchor tethered, dynamically positioned, or jack-up. A remarkable variety of such structures have been built in recent years, with relatively little background experience. Unlike ship structures, design has largely been *ab initio,* and in general, successful. The offshore industry has accumulated much experience of advanced structural and fatigue analysis, of wave loading and of assessing uncertainties.

Lessons for land structures

It would clearly be beneficial if a performance data bank could be created for land structures. With the advent of Eurocodes, it would be appropriate for this to be introduced under EC auspices for all structures designed according to those codes. A building performance data bank does apparently exist in the United States. Their experience of legal and allied difficulties, costs and funding, acquisition and dissemination of information, should be relevant and useful.

If access to records could be made available, much could be learnt from a systematic study of the prevention, incidence and repair of fatigue cracking and corrosion in ships and offshore structures. The whole-life system of specification, contractual arrangements, design, construction, quality assurance, operation, maintenance, repair and recycling, as practised for various classes of structure, could provide a useful field of comparative study. Each system has evolved to suit the ownership and circumstances surrounding each class of structure, but each system might benefit from adjustment in the light of experience in other fields.

The advantages and disadvantages of the Classification Society approach to design have been discussed. Might it be possible to impart some of the advantages to the current system for land structures? The lack of continuity and provenance, and the differences of approach to identical problems according to the current composition of committees, might in principle be minimized by the employment of expert staff in the construction division of CEN.

In principle the proposed system of Eurocodes should remove some of the unnecessary variations; whether intentions will be realized remains to be seen. Eurocodes still rely on volunteer committees, with limited input from practitioners, with the added difficulty that membership is allocated according to nationality and not on the basis of complementary contributions.

It is perhaps difficult to imagine that officialdom will solve these problems. Could CIRIA, CA, SCI take a lead in strengthening the industrial input?

Questions to be addressed

The briefing document posed the following questions.

What do we understand by design life?

The specification of design life with any associated contractual obligations, having regard to the conditions of service, including those controlled by the owner, and ambient conditions; obligations for maintenance; the durability of each item provided by the contractor would need to be included; the protection and corrosion of numerous materials; the specification of load history and environment and the determination of fatigue life and rate of corrosion; depletion of resources and recycling; energy consumption and environmental damage; a potential paradise for lawyers.

Should we design for a specified life?

In many fields design life is already specified, but in housing, for example, owners and mortgagees will need an indefinite life, subject to reasonable maintenance. The answer to the question depends on circumstances.

What should the design life be?

The intended or nominal life can only be decided by the owner, having regard to financial and tax constraints and influences, the construction regulations, to comparative costs and to advice given. Current financing and tax arrangements in the United Kingdom encourage poor quality, wastage of resources and degradation of the built environment. If there is no carrot we need more stick, in the form of quality based building regulations.

Do we have the knowledge?

We have some knowledge of material responses to loading and the environmental attack. Some owners are able to declare a nominal life and will agree a basis for estimating expected life. In many cases it will be necessary for the nominal life to be indefinite.

What are the real factors?

Some of these have been discussed.

Do we really need these concepts?

The concept is already accepted for some classes of structure. The question of durability should be specifically addressed for all construction, but it will not always be possible or even desirable to specify a design life. Where the public interest requires improved quality, this can be implemented through the building regulations. That is, in some instances, quality standards are preferable to performance standards.

Conclusion

We do need to pay specific attention to quality and durability, which need to be specified and evaluated with regard not only to commercial criteria but also to the public interest. To achieve this we require accepted criteria for standards and methods which are deemed to satisfy given levels of quality and durability.

Durability of a structure is a function not only of durability of materials but also of maintenance and repair, and of the environment. Quality and durability are linked but are not synonymous. Inasmuch as behaviour under known conditions, and the actual service conditions, are both subject to uncertainties, it might be that reliability methods which incorporate judgements, might usefully be employed to rationalize and standardize durability assessments. Where the quality and durability are both of a high standard, the structure is more likely to be granted a long life.

The degree of durability which is required depends on the class of structure. Where a finite life needs to be specified on commercial grounds, this can be done and is done. In many instances the life must be specified as indefinite or perhaps as a minimum period, but quality (which implies relative durability) can be improved by regulation. Our long term experience of durability is limited to traditional materials. Perhaps accelerated testing can give useful information, at least on a comparative basis.

In many instances commercial mechanisms will not ensure the quality and durability which is required for the public interest. We already have public interest regulations covering safety and health, insulation, location and appearance. We now need to tighten existing regulations on quality. Regulations or fiscal incentives could encourage conservative construction.

It seems that there should be a Eurocode on durability in construction. This could then be supplemented as necessary in the various documents covering different materials, products and structures. Perhaps the new version of CP3 Chapter 9 will provide a model for a European code. Perhaps the revised version should not be restricted to building. Performance data should be fed back to the specification and design processes. The term 'conservant construction' encapsulates the desirable qualities which have been discussed in the foregoing.

Offshore and marine structures

R.O.SNELL

Introduction

The Oil Industry has built approximately 4000 offshore and coastal platforms in the last 50 years. Deepwater rough sea platforms have become routine within the last 15 years. These structures are each just one part of a complex industrial system built to extract, process and export oil and gas, of which the structure typically accounts for about 20% of the total cost.

Initial platform designs were developments of piled jetty type structures in Lake Maracaibo and the shallow waters of the Gulf of Mexico. Platform structural design techniques steadily improved as they extended into deeper water in the Gulf but reservoir characteristics and drilling costs were the driving force in field economics thus structures and facilities tended to be standardized rather than optimized.

The North Sea developments commencing in the late 1960s created the need for optimized structural performance as the high permeability reservoirs and harsh deepwater environment caused facilities' costs to dominate the economics of oilfield developments. Recent, very deep water, Gulf of Mexico platforms are also facing the same economic drive. It is the design life of optimized one-off structures that will be the subject of this paper.

Optimization in oil industry terms does not necessarily mean a highly elegant design. It means achieving fitness for purpose at minimum cost.

Design life and duty

The design life and duty of an offshore structure are mobile targets. It is not generally appropriate to consider them entirely separately as it is quite common for structures to be subject to substantial changes in loading during their life.

The design life is subject to changes due to:

- Oil or gas price
- Taxation
- Reservoir actual performance compared to original predictions
- Additional adjacent discoveries developed by wells tied back to the existing platform

- Enhanced oil recovery techniques
- Infrastructure usage, e.g. becoming a pumping platform for new fields

The design duty is subject to change due to:

- Load additions or reductions arising from changes in use
- Revisions in the Regulations against which acceptability is judged; platforms in the UK sector of the North Sea are recertified on a 5 yearly basis against the latest criteria, not those in practice at the time of original design
- Revisions in environmental loading predictions, most notably in the wave climate, as knowledge of an area improves relative to that available at the time of design

Examples of changes in service life and duty to illustrate the mobility of these parameters are:

(a) *West Sole*: Developed by BP in the late 1960s and initially considered to be a 30 year life field. Present perception is that the field may have an economic life up to 50 years. Design loads have increased due to upgrades of topsides equipment and correction of original underestimates of wave loading.

(b) *Forties*: Developed by BP in the mid 1970s as a 30 year life field. With substantial load additions at least one platform is expected to see service for approximately 40 years. An improved understanding of the wave climate since original design has shown a reduction in extreme wave height but a 40% increase in the overall number of waves.

(c) *Buchan*: Floating production system developed by BP in the early 1980s by converting a Pentagone drilling semi-submersible originally built in 1973. Initially the field was expected to have a 2–8 year life depending on the performance of a complex reservoir. Now evaluating capability to extend life beyond 1995.

(d) *Green Canyon 29*: Floating production system developed by Placid which commenced production in 1990. Shut down and abandoned after a few months due to reservoir performance below prediction.

With the exception of the last example the general trend of increased life and duty is common throughout the industry.

Definition of failure

The structural components of an offshore platform fall into two quite separate areas:

(a) The topsides structure, which is that at or above lower deck level, sees a predominately static loading from an accurately known deadweight (weighed to ±2% accuracy) and a well defined live load.
(b) The supporting substructure, a piled steel jacket, concrete gravity-based structure, or the pontoons and columns of a semi-submersible, sees a combination of accurately known load from the topsides and self-weight and a cyclic environmental live loading of a much less accurately defined nature. For many members in the substructure the cyclic environmental load is substantially in excess of the deadload, resulting in stress reversals.

Failure of a topsides member would most likely arise from local overload due to incorrect use, accidents leading to blast and/or fire, or human error in design and fabrication. Failure of a substructure member would most likely arise from fatigue, human error in design or fabrication, ship impact or local overload due to impact by an object dropped during supply operations. The consequence of overload from environmental loading would most likely be local failure and load redistribution rather than immediate overall collapse.

For both substructure and topsides, some components are controlled by the short term load case during transport to site and installation, and standardization of member sizes results in only a limited number of members being fully utilized. Typically a steel jacket substructure would have some members with an AISC utilization under the controlling loadcase close to 1, but the majority of the member utilizations in the range 0.85–0.95.

The definition of failure used for design is exceeding code acceptability. For assessment after construction it is a more complex definition and depends on the perspective of the assessor.

Operator

The operator defines failure as the point when the cost of maintenance required to sustain an acceptable margin of safety makes continued production uneconomic. In the high fatigue loading environment of an offshore platform, approximately 5.7 million load cycles per annum in the North Sea, maintenance cost is primarily related to fatigue damage. An optimum balance between initial capital cost and operating expenditure is the design objective. A 2–3 times life multiple for fatigue design using design curves based on the mean minus two standard deviations of laboratory test data is about the optimum.

Accident scenarios such as an explosion or major fire are considered with the aim of containment until personal evacuation has been achieved. In such analyses the design life is a single accident event.

Regulator

The UK Regulatory Authority, the Department of Energy, requires recertification on a 5 yearly basis which, if the structure has been modified or the design load increased, involves demonstration of acceptability to the latest edition of their design guidance and codes. This is a difficult target for old structures. The Norwegian Petroleum Directorate follows a similar principle. The US Minerals Management Service do not use recertification thus compliance with the code current at the time of design is adequate. The principal code, API RP2A is currently on its 18th edition in working stress format and is now in draft in Load and Resistance Factor format. Some of the changes between editions have been quite fundamental.

Accuracy of design calculations

The oil industry has committed substantial funds to offshore structure research. It is normally undertaken on a cooperative basis by the principal operating companies with the results directly influencing revisions of codes and regulations. Design contractors and certifying authorities serving the operators actively support the research effort.

This research, in combination with in-service experience and improved computational techniques has substantially improved the reliability of design predictions and has made possible reductions in the weight and cost of structures over the last 15 years. It has also made it possible to reassess many existing structures demonstrating a capability to meet increased loadings to the latest regulatory requirements. The change in the probability distribution of loading and resistance as design knowledge matures is illustrated in Figure 1.

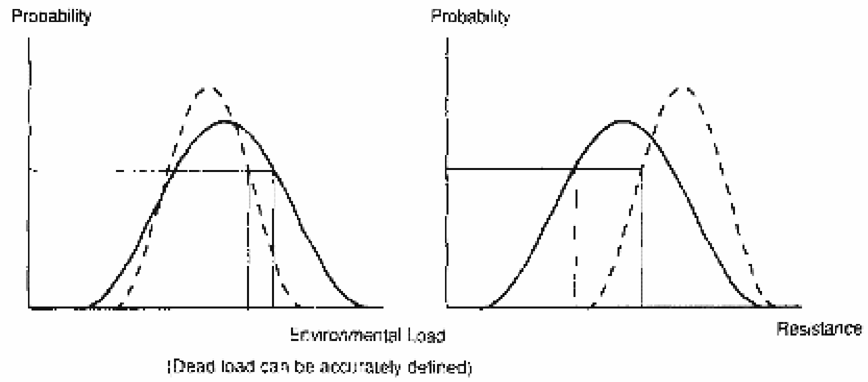

Figure 1 Impact of maturing design knowledge on reassessment of loads and resistance (— original design;– – – reassessment).

Some significant areas remain inadequately understood of beyond current practical computational capability resulting in uncertainty when calculating the risk of failure and design life of an offshore structure.

The principal areas of uncertainty and computational limitations are:

- Calculation of wave and current load
- Prediction of coincident extreme wave, current and wind values for a specific return period
- Computational ability to incorporate three dimensionality of wave loading
- Computational ability to predict successive individual member and joint plastic deformation events leading to overall collapse, hence true ultimate load capability
- Pile vertical and lateral stiffness
- Cyclic degradation of soil properties
- Fatigue of welded joints

This uncertainty is accounted for in the factors of safety used in design.

Oil companies compete in oilfield exploration and in refining and marketing, not in structural engineering safety margins where a major failure experienced by one company affects the whole industry.

Fabrication practice limitations

As with most structures the fabrication standard specified for an offshore structure is a compromise between cost and quality. Some of the earliest platform substructures built in the UK sector used Gulf of Mexico fabrication practice. Operators quickly found that the more demanding fatigue environment required a more rigorous standard if maintenance costs were to be contained. The industry responded by setting very high fabrication standards and since then has been closely questioning and, where possible, reducing requirements.

Very large tubular structure fabrication standards essentially cover the accuracy of assembly, weld quality and inspection.

Accuracy of assembly

Tubular joint fatigue calculations are very sensitive to stress concentration factor calculations used to predict local stress distributions. Tolerances in plate thickness, rolling, alignment with adjacent plate and individual member position in a complex subassembly can all have a significant effect on the stress concentration factor and hence fatigue durability of a joint. An extreme example of the effect of member position tolerances on a Forties structure joint is given in Figure 2.

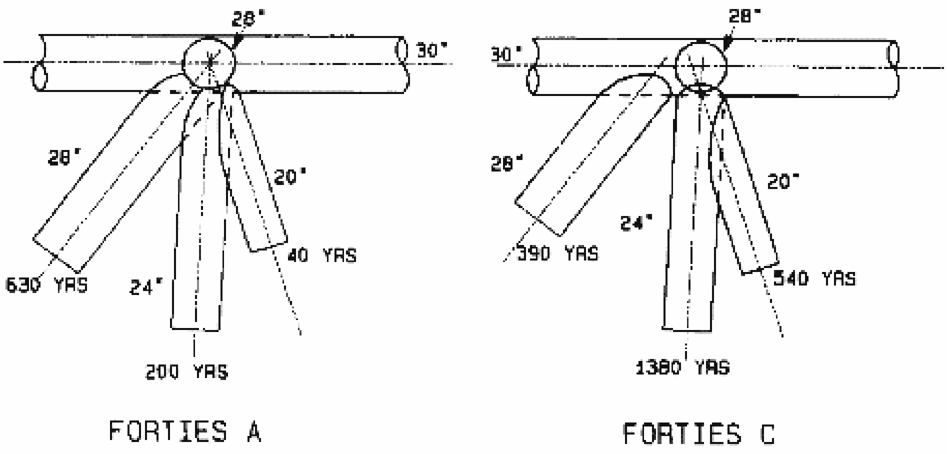

Figure 2 Extreme example of the effect of tolerances on fatigue life. Forties joint C3 at –80ft nominal level.

Weld quality

There is very little room for compromise in the quality of welds and material toughness for high fatigue environment structures. Most of the fabrication standards work in this area has concentrated on defining standard weld geometries and procedures. The most difficult task is the final closure weld undertaken when each member is installed in the overall structure. This is usually a butt weld made with access only from the external face. It is not normally possible to make such a weld without stress concentrations or defects on the inside face. Recognition of this limitation is very important when quantifying design life.

Inspection

Fabrication contractors serving the offshore oil industry have developed and maintain a very high quality of fabrication. They are aware that any reputation for quality compromise is commercially very damaging. Operators concentrate inspection on critical components rather than on all items, thus making substantial savings in fabrication supervision.

In-service performance

The in-service performance of offshore platforms has been monitored through a combination of inspection and instrumentation programmes.

Instrumentation

The Magnus platform, standing in 186 m water depth, 100 miles north east of the Shetland Islands, was monitored to obtain the response of the structure and piles to wave load. The results from selected extreme storm event during the period December 1983 to April 1986 have been analysed and the results reported by Webb and Corr [1].

The instrumentation, data gathering and analysis techniques used were state of the art. Whilst in overall terms the programme was a major advance, the true potential was not fully reached due to failure of some of the wave sensors. This resulted in a need to derive results from matching and comparing statistical descriptions of wave loading and response, rather than a direct input/output comparison.

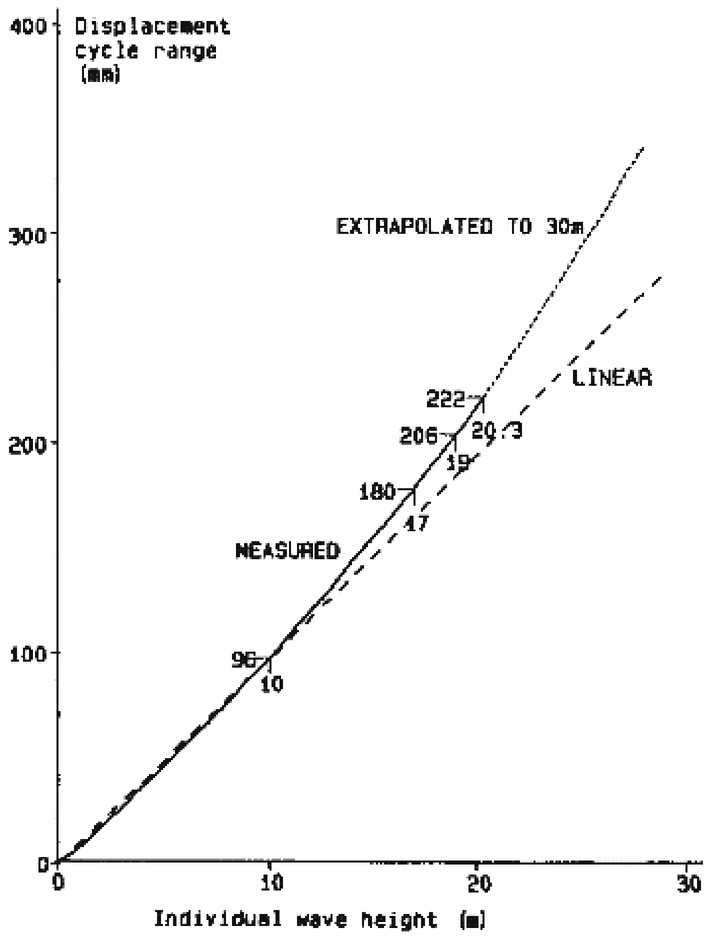

Figure 3 Wave and height displacement relationship.

The key conclusions from the programme were:

- Actual pile stiffness was up to three times greater than predicted, which reduced the natural period of the structure and the dynamic component of the load.
- The structure loading remained generally linear up to the maximum wave height experienced. Figure 3 shows the relationship between wave height and displacement. Although the very large diameter legs at Magnus make this structure more inertia dominated in modest waves than most platforms, it had been anticipated that non-linear drag loading would dominate in higher wave loadings. The highest recorded wave was 20 m compared with the design 26.1 m maximum height wave predicted to occur once during a 100 year period.
- The calculation process which aggregates individual member wave loadings substantially overestimated the overall load. Figure 4 shows a representative plot of actual load in a leg compared to design predictions.

From this monitoring programme it was concluded that the design duty could be

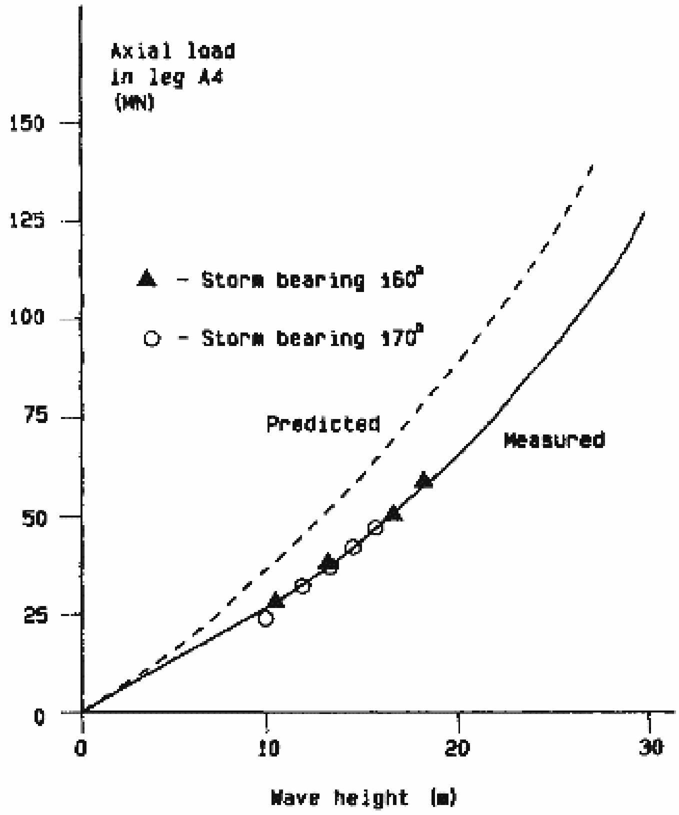

Figure 4 Wave height and axial load relationship.

increased and still meet the intended design life. An addition of four further wells was sanctioned.

Based on these results, and other complementary work, BP has reduced design specification inertia coefficients by 15% and drag coefficients by 17%. This still leaves a margin of approximately 50% between prediction and the measured results for this structure.

A new monitoring programme has commenced on the Tern Platform (Shell Expro) which uses similar techniques and systems to that pioneered on Magnus, but with the benefit of more design development and experience. The results should provide a very good basis on which further refinements of design parameters can be made.

Inspection

Platform substructure inspection is undertaken usually on an annual basis to monitor the overall structure condition and any specific welds identified as having some risk of fatigue damage.

The early generation of North Sea platforms suffered from a deficiency in the design of the oil well conductor bracing against vertical wave forces, and some had poor detailed design of appurtenances. These have generated a substantial inspection and maintenance workload which will continue throughout the life of the structures. In most cases maintenance costs on these structures have been sustainable because the reservoirs are performing well. We are now entering the declining years of production and the impact of maintenance cost on economic life is increasing.

The structures designed since the mid–1970s have largely avoided basic design deficiencies and do not have a significant number of welds theoretically likely to require repair within the field life. The inspection philosophy and technique used for these platforms is a form of condition monitoring designed to detect the random distribution of defects in closure butt welds and dropped object damage. Unlike classical fatigue damage which would show crack development commencing on the external face, cracks from fabrication defects in these welds are likely to propagate from the inside face. To date at least two major members on offshore platforms have been found to require repairs to defects in closure butt welds. The inspection philosophy with example applications is described in detail by Snell [2].

The primary technique used is the detection of flooding of the normally sealed dry tubular as a result of a through wall defect. This is the 'leak before break' concept which is good practice in pressure vessel design. The inspection is normally undertaken by a remotely operated vehicle. The interval between successive inspections is predominately a function of the fracture toughness of the material.

Should a member be found to have flooded, a full inspection of the welds by methods such as ultrasonics and metallic particle inspection is required to characterize the defect and plan for repairs.

Summary of in-service performance

The inspection and monitoring evidence to date has given confidence that the design and fabrication techniques used are reasonable, conservative in 'static' terms but not necessarily so in fatigue when practical fabrication limitations are taken into account.

Comparisons of the actual occurrence of fatigue damage with design predictions generally shows predictions to be over conservative. A factor of 10 on life is a general guide to the degree of conservatism. There are however sufficient anomalies, often but not always associated with the margins of fabrication tolerances, to make refinement in this area less rewarding than others.

Maintenance and repair

The maintenance and repair costs on offshore structures are dominated by the cost of access and diver support. A high quality subsea weld costs about £3 million. Grouted clamps cost about £1.5–2 million.

A considerable amount of analysis will normally be undertaken to examine alternatives including deferring repairs wherever possible. It is in this phase of platform analysis that the refinement of individual member design life is taken to its finest limits.

An example of this type of work is illustrated by the history of a cruciform tubular joint in the conductor bracing on two identical Forties platforms both installed in 1975. The original design underestimated the vertical wave forces with the resulting reassessed theoretical fatigue life in this particular joint being less than 1 year.

Inspection on an annual basis has been used to monitor deterioration and repair options evaluated. The first detectable defects were found 5 years after installation on one of the platforms. Profile grinding was used to remove the defects and delay propagation. Design work on repair options commenced which proved the technical feasibility of the stressed grouted clamp concept as a cheaper alternative to a weld repair. In 1985 the rate of propagation of the defect increased. As construction work to upgrade Forties topsides facilities was scheduled to commence in 1978, making access for repairs difficult, the decision was taken to implement a clamp repair in 1986, 11 years after original installation.

On the second identical platform the first significant defects were found in 1987 and showed further growth in 1988. Deep profile grinding, locally up to 20% of the plate thickness, was used to remove the defect and defer further

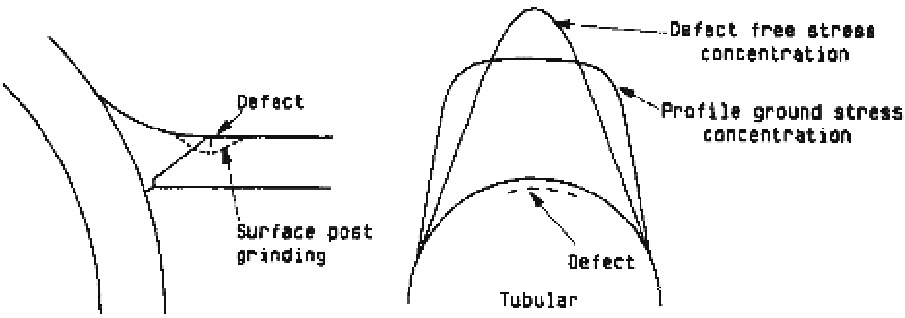

Figure 5 Profile grinding to control defect propagation.

growth. No defects have been found during 1990 inspection. R&D effort is in hand to develop significantly cheaper clamp designs than those used on the first platform.

The use of profile grinding, illustrated in Figure 5, to control defect propagation is of particular value in tubular structures because of the very sharp peak in stress concentration and scope for local load redistribution. Grinding up to 25% of the parent plate thickness has been used with success. It may be possible to go to greater profile depths locally.

Conclusion

The Oil Industry has made routine the design and operation of large offshore platforms in a climate of change in the required economic life, operating load and regulatory criteria. It has achieved this largely through a sustained commitment to research, in-service monitoring and maintenance. There are some major issues as yet inadequately understood which result in conservative design, particularly for extreme load cases. The design life in a rough sea environment is dominated by fatigue considerations with all structures requiring, as a minimum, a significant expenditure on inspection.

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Factors affecting the prediction of design life of structures in the nuclear reprocessing industry

G.W.JORDAN

Introduction

The nuclear scene in the United Kingdom has been set by Wilkinson [1]. In order to support this scenario, nuclear facilities must have a predicted design life to meet a number of criteria. An adequate design life of a structure is essential to maintain plant operation and process. Operational success has meant that structures are being reviewed beyond their 'normal' 25 year life and a 50 year life currently presents few problems.

Large capital investments have been made. Facilities contain large volumes of concrete and large tonnages of structural steelwork and there is a need to protect this asset. Safety is of paramount importance in the industry and serviceability complements this to maintain performance at any time in a structure's life. Accessibility for maintenance in irradiation or controlled areas may not be possible, hence an essential requirement for durability. Adequate protection of structures by coatings, for example, means ease of decontamination which in turn facilitates decommissioning by reduction in disposal of contaminated materials. The large amount of work and cost associated with decommissioning means consideration and accommodation of this aspect early in design.

British Nuclear Fuels PLC (BNFL)

Nuclear reprocessing involves 'wet' and 'dry' storage, both of which have similar demands on structure durability. At Sellafield, spent nuclear fuel is received and stored prior to chemical reprocessing. Wet storage options are favoured by BNFL. After reprocessing, waste products from Magnox (**Mag**nesium **no** **ox**idation) fuel element cans are encapsulated in cement and stored in stainless steel drums in a dry environment to await final repository disposal by UK Nirex Ltd (**N**uclear **I**ndustry **R**adioactive Waste **Ex**ecutive).

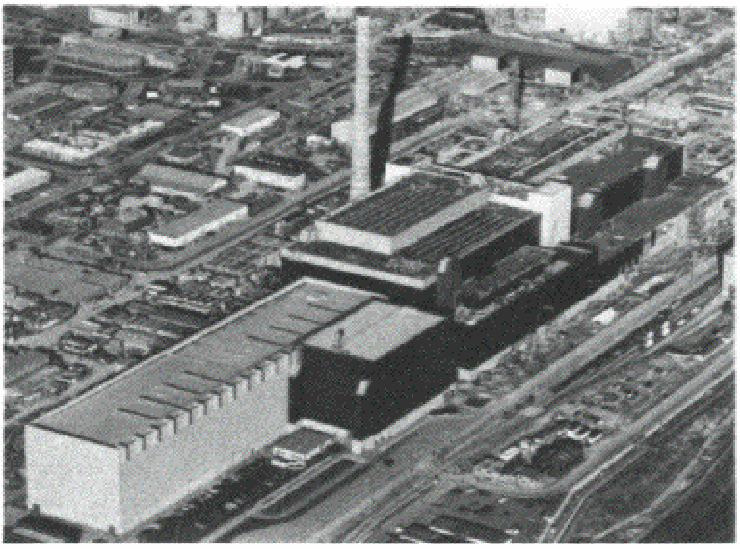

Figure 1 Thermal oxide reprocessing plant (THORP).

Aim of design

Knowledge is not yet adequate to allow concrete structures to be designed for a specific durability and life. Structures designed and built to the recommendations of BS 8110 [2,3] may normally be expected to be sufficiently resistant to the aggressive effects of the environment so that maintenance and repair of the concrete would not be required for several decades, i.e. a life before significant maintenance generally in the region of 50–100 years.

Safety is of paramount importance and must be complemented by serviceability requirements. Durability of structures has come to the fore in recent years and is being achieved by use of high quality materials and high standards of construction, the whole under strict quality assurance surveillance.

Design life

Somerville [4] has postulated a possible definition of nominal design life as 'the minimum period for which the structure is expected to perform its defined function without significant loss of utility and not requiring too much maintenance'. Service life [5] of a structure or component may be defined as 'the period of time after installation during which all properties exceed the minimum acceptable value when routinely maintained'. Serviceability is the capability of the construction to perform the function(s) for which it is designed and constructed. Durability [5] is the capability of maintaining the serviceability of a construction over a specified time.

As well as providing for normal operation requirements many nuclear structures are required to withstand effects of extreme environmental hazards such as earthquakes, wind and abnormal loadings, such as impacts from possible mishandled fuel flasks. Structures are required to perform against these effects at any time in their design life.

Prediction of design life

Beeby (personal communication) has indicated that such predictions are difficult but has suggested the parameters of 'time for carbonation front to reach reinforcement' (Figure 2) or 'initiation of corrosion by carbonation' (Figure 3) as indicative of the problem.

A typical concrete for a nuclear reprocessing facility exposed to severe

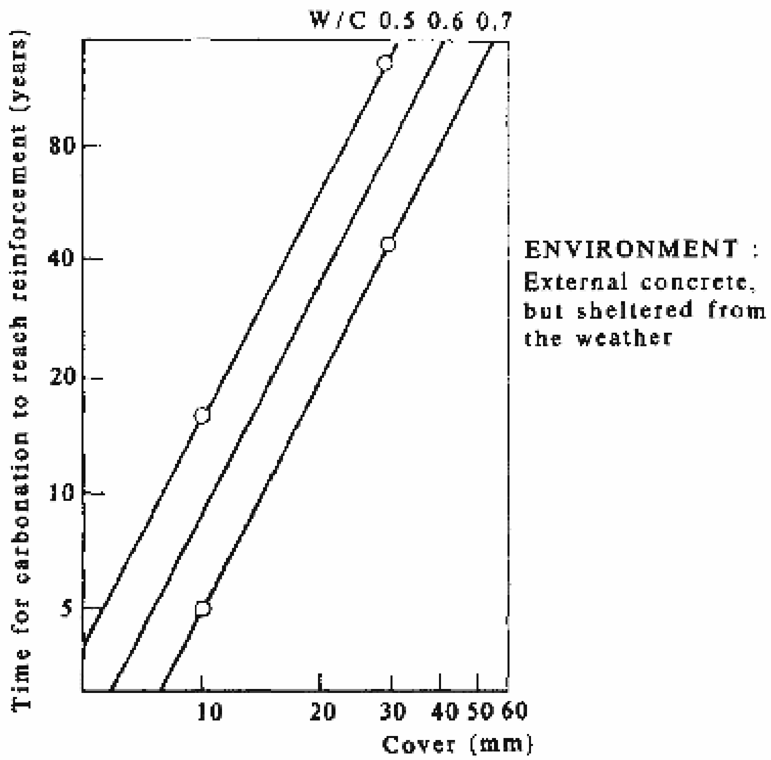

Figure 2 Time for carbonation to reach reinforcement. From [4].

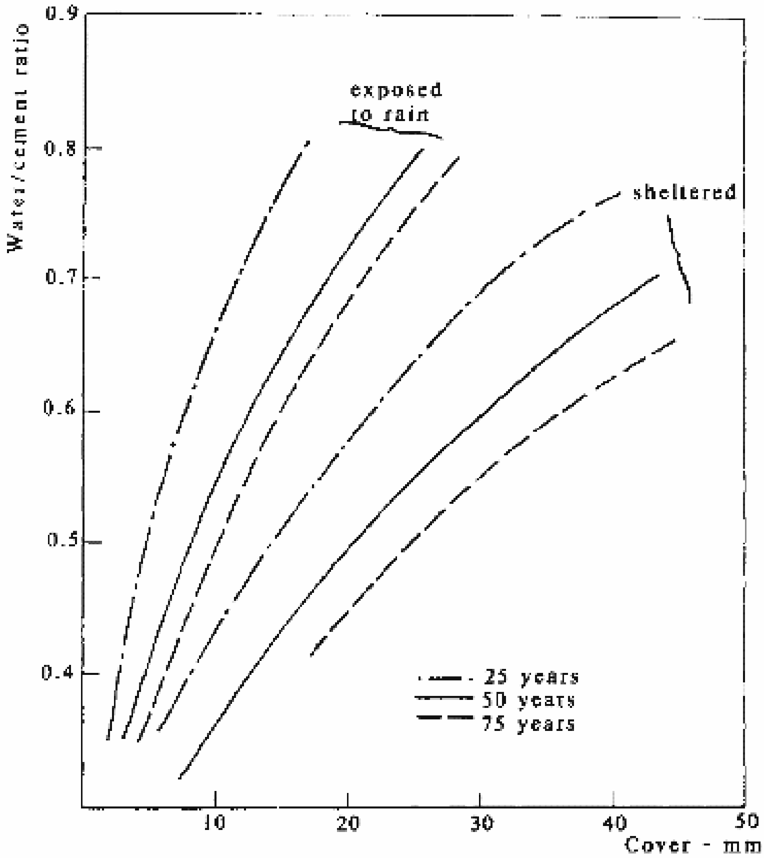

Figure 3 Relationship between water/cement ratio, cover and time to initiation of corrosior by carbonation in two external environments. From [4].

Nominal cover to bars	40 mm
Max free water/cement ratio	0.55
Minimum cement content kg/m³	325
Lowest grade of concrete	C40
Nominal max aggregate size	20 min
Exclusion of chlorides	

conditions, concrete surface exposed to severe rain, alternate wetting anc drying and occasional freezing, or severe condensation is as follows:

Design and construction to BS 8110 will result in a design life in excess of 50 years.

Factors affecting attainment of design life

Predominantly within the nuclear reprocessing industry these may be identified as:

- Environmental effects: Sellafield is in proximity to the coast
- Requirements of wet storage technology: nuclear fuel storage ponds
- Requirements of dry storage technology: waste storage facilities
- Irradiation degradation: sealants, PVCs
- Chemical attack: process nitric acid
- Concrete and steel are fundamental construction materials

Design and construction are carried out to BS 8110 and BS 5950 [6] but for features outside their scope special investigations are made, some of which are described in this paper. Concrete sometimes has a fundamental requirement of provision of shielding against radiation effects. Steel is employed for frameworks of building structures and support of mechanical plant.

Degradation process

These are numerous, some of which are:

- Reinforcement corrosion (with cracking and spalling of concrete)
- Carbonation
- Leaching (by water)
- Chemical attack
- Freezing of water
- Abrasion
- Temperature variations
- Chlorides
- Pitting corrosion in prestressing wires
- Alkali/aggregate reaction

Of these only the first four are considered in this paper.

Prevention of degradation

A popular practical concept such as the 4Cs rule for long life concrete is attention to **C**ement content, **C**over, **C**ompaction and **C**uring and is supported by Beeby [7, 8], by Pomeroy [9] and reported in [10].

For a water retaining structure such as a nuclear fuel storage pond [11] designed to BS 5337 [12] or more recently to BS 8007 the requirements correspond generally to Table 3.4 of BS 8110 for severe exposure.

Attention to cover is such that post concreting cover surveys are carried out. Use of pulverized fuel ash (PFA) and ground granulated blast furnace slag (GGBS) cement replacements indicate an improvement in durability. Use of galvanized, epoxy coated [13] or stainless steel reinforcing bars have not currently been found to be necessary in BNFL structures, on economic grounds.

Leaching

In unpainted ponds of water retaining concrete, some of the ions leached from the concrete, e.g. chlorides, silicates, can accelerate the corrosion of the stored fuel or its cladding. Although the concentration of these ions in the pond water is maintained at very low levels by the water chemistry control plant any increase in impurity levels would increase the burden on the purification process and increase the volume of active waste for disposal. This has led to the provision of painted coatings on pond wetted surfaces.

Coatings on concrete

The objectives of such coatings are:

- To prevent pond water degrading the concrete or corroding the reinforcement
- To reduce rate of leaching of ions from concrete into pond water
- To provide a surface which is smoother, easier to clean and decontaminable when maintenance is carried out
- To minimize the permeation of radioactive species into the concrete to permit easier decontamination and ultimately decommissioning; also the quantity of contaminated concrete to be disposed of after decommissioning is reduced
- To increase the reflectivity of the concrete to assist with viewing of handling operations

Such coatings are two pack solvent free epoxies applied to concrete by airless spray after blast cleaning. Service lives of greater than 25 years are expected.

In a less onerous environment of a waste product store although assumed to be subjected to 'severe' conditions of Table 3.4 BS 8110, it has been concluded that painting of the store walls is unnecessary to achieve a 50 year design life.

Coatings on mild steel

Jordan and Mann [11] give descriptions of coated sandwiched mild steel membranes that provide containment integrity against water leakage to the environment (Figure 4). Corrosion protection is afforded by application of solvent free cold cure polyurethane by airless spray subsequent to blast cleaning (to Sa 2 1/2) and priming of the metal substrate within 4 h. This to provide a design life in excess of 40 years.

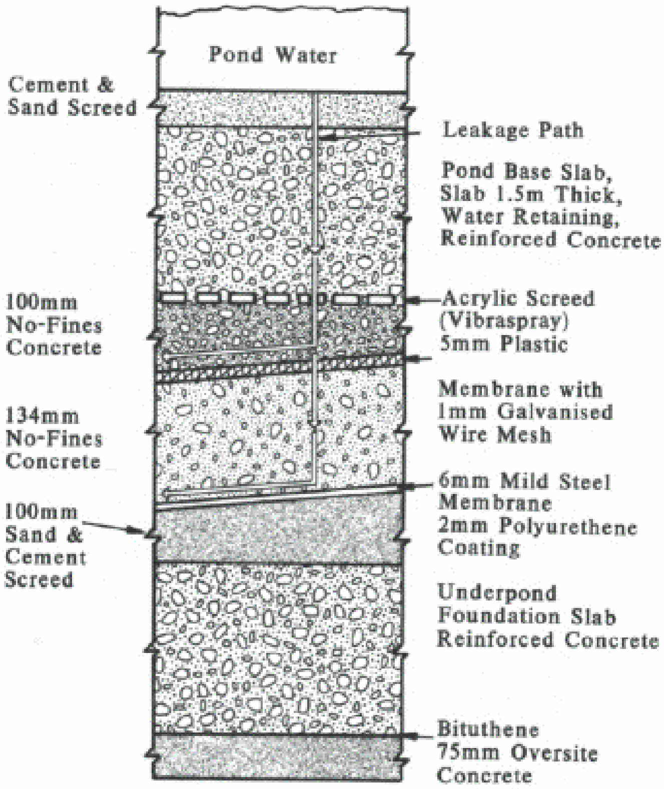

Figure 4 Section through receipt and storage pond foundation.

Cathodic protection

Cathodic protection can be afforded to reinforcing bars. Corrosion of reinforcing steel can be a result of chloride contamination of concrete, carbonation of concrete, or poor construction practice; notably lack of cover or a combination of these factors. This has not been used on BNFL structures, but evaluations are being made. Cathodic protection of mild steel membranes has not been attempted.

Chemical attack

Nitric acid is one of the principal chemicals used in various levels of concentration in the reprocessing cycle. This is normally transported between various plants in stainless steel pipes. Assessments have been made of effects on typical concretes indicating preferential attack on the cement matrix rather than the aggregate. Various protection measures varying from epoxy painting to acid resisting tiling have been identified.

Inspection techniques for detection of degration

The objective of this work was to develop non-destructive techniques which were capable of inspecting the condition of reinforcement in concrete structures. The method involves making a number of potential measurements (potential mapping) across a concrete surface using water coupled probes [14]. Where chemical activity is present, such as corrosion of reinforcing bars, the potential of the area will change relative to the surrounding enabling a semi-quantitative assessment to be made of the structure. Development has reached a stage where this is in semi-routine use.

Repairs

To mitigate against any effects of reinforcing bar corrosion due to leaks in water retaining structures and to provide an improved working environment, epoxy resin repairs of cracks have been undertaken successfully and without emptying the ponds of water.

Effects of irradiation

Irradiation degradation can occur on polysulphide and polyurethane sealants and water bars of PVC material. Metal shielding protection is afforded in some circumstances. The design life is assessed by tests on fully irradiated specimens, it being important to simulate the fully submerged pond conditions. Epoxy coatings tested have a good resistance to irradiation.

Structural steelwork

In recent years considerable improvements have been made both in paint systems and their application on prepared surfaces. Structural steelwork is blast cleaned usually after fabrication to Sa 2 1/2 standards and shop primed before delivery to site. In areas of long period to first maintenance, galvanizing and chlorinated rubber paint systems have been applied and for larger structures metal sprayed systems have been found acceptable. Corten steel has been used on exposed columns.

Cost estimates of protective systems have been made and include the cost of maintenance during the service life of the structure. In waste stores it has been necessary to consider a 50 year design life and the need for zero maintenance. This has led to stainless steel structures. For long term maintenance, costs of stainless steel compared with paint protective systems, has been found to be cost beneficial in areas of difficult access, e.g. storage pond roofs. Against this has been the need for careful evaluation of stainless steel integrity in such environments.

Ventilation stacks

As a final example of design life requirements steel ventilation stacks can be subjected to fatigue loading under some vortex shedding situations. Evaluation work has been done to determine fatigue life of stainless steel weld details.

Conclusion

This paper has presented an overview of work done in the last ten years to support safety standards demanded by the regulatory bodies for nuclear reprocessing facilities, and to protect BNFL investment. Much more extensive work will be necessary to provide confidence in the design life of the ultimate repository.

Acknowledgements

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Design life of welded structures

M.H.OGLE

Introduction

The invention of electric welding 100 years ago may not have seemed a very significant event to the structural engineer at the time. Indeed, the idea that a magician with a sparkling 3 mm diameter wand could wave it across a large structural joint and make it disappear would have been incredible even to a blacksmith, let alone a mason, bricklayer or carpenter. And yet for the metal working industry this dream actually came true. Today, many millions of tonnes of welded metal structures are built every year. There is also an increasing confidence in the reliability and durability of such structures.

This confidence has been hard won and there have been many serious setbacks on the way. The fact that the act of arc welding effectively undid all the carefully contrived metallurgical structure of the parent metal in the region of the joint was a major problem. This did not happen with riveted or bolted joints. Although certain successful industrial applications started to appear after the First World War it was not until the stimulus of the Second World War that a scientific appreciation of the potential dangers began to emerge. Never before had the integrity of the structure been dependent on such subtle and invisible factors. A damp bolt had never been a problem; why now a damp electrode?

The advantages of welding were most obvious to the power and process plant industries. The ability to provide strong leak tight joints in pipes and pressure vessels without the inefficiency and complexity of riveting, bolting, threading and forging was a strong incentive for development. The consequential insurance losses arising from some catastrophic failures was a strong incentive for research.

The high joint efficiency, the watertightness and smooth surface of a full penetration butt weld compared to a riveted lap joint was very attractive to both naval and merchant ship designers. After the Second World War there was a natural transfer of the technology into the structural steelwork industry, where high productivity and the widespread use of the cheaper fillet weld were common features. However, the shortage of steel at that time meant that development was slower than it might have been and 'maintenance free' concrete was extensively used for buildings and bridges.

There has now been a reversal in the confidence placed in these two materials and it is worth looking more closely at the recent history of welded construction to see how the various problems have been overcome in the last 50 years.

Some important milestones

Brittle fracture

In 1942 the picture of an almost new Liberty ship sitting in harbour in calm water with its back broken in two, was a blow to confidence in welded construction. Subsequent investigations showed that the presence of severe stress concentrations and tensile residual stresses arising from welds imposed greater demands on material toughness than with riveted construction. Considerable pressure was put on the steelmaking industry to produce steels with a guaranteed minimum Charpy value. In the United Kingdom it was not until 1956 that a standard for notch ductile (ND) structural steels was published. This in effect provided tougher versions of BS 15, the basic mild steel (Grade 43). It was not until the publication of BS 4360 in 1968 that tough high tensile and weldable structural steels (Grade 50) were standardized.

In 1966, BS 449 and BS 153, the UK design codes for steel buildings and bridges, respectively, produced maximum thickness limits for avoidance of brittle fracture. Their limit state successors BS 5950 and 5400, respectively have had slightly more stringent requirements. There have been no reported brittle fractures in buildings or bridges designed to any of the above codes.

It is of interest to note that in North America, in spite of the colder winters, the Charpy requirements appear to be less severe. However, following recent incidences of brittle fractures in heavy jumbo section welded splices in roof trusses this form of construction has been temporarily advised against until a more reliable material detailing specification can be drawn up.

The small additional material cost of providing adequate notch toughness is not a serious penalty and is certainly a good insurance policy. Modern welding consumables usually have little problem in matching the requirements of the parent metal.

As brittle fracture weakness is likely to show up early on in a structure's life (assuming regular reoccurrences of cold weather) the signs are good that this problem can be properly controlled by appropriate material selection and detailing. Hence it need not be a limiting factor on design life if the code requirements are followed.

Weldability

The most important milestone was the failure of the Kings Bridge in Melbourne, Australia in 1962. One span of the composite multiple I-girder highway bridge failed by brittle fracture of the tension flanges and webs. The fracture occurred because the fillet welds at the ends of the cover plates had fabrication cracks. This severe notch, together with the stress concentration and inferior toughness caused the failure. The root problem was one of too high a carbon equivalent and a lack of appreciation of the potential danger.

This prompted the steel industry to develop more weldable Grade 50 steels, which eventually appeared in BS 4360 and gave optional limits on carbon equivalent values (CEV) for various grades.

In 1974 BS 5135 was published which gave the world's most comprehensive data for selecting weld procedures to avoid hydrogen cracking, with direct cross–referencing with the CEV limits in BS 4360. Since that time the reported incidences of hydrogen cracking have been due to conditions being used outside the code recommendations.

BS 5135 gives guidance on the avoidance of solidification cracking in high heat input weld metal. It also gives guidance on material selection to avoid lamellar tearing. A recent survey showed that the incidences of lamellar tearing had reduced to a low level in the United Kingdom. One spin-off from the offshore industry has been the development of low sulphur Z quality steels at a reasonable price. Problems of lamellar tearing are more likely in sections, where materials are more susceptible than in plate.

Residual stresses

There are many sources of residual stress, and weld shrinkage is one of them. Yield magnitude tensile stresses can be generated, which can have an effect on fracture and fatigue. The balancing compressive stresses can be detrimental to buckling resistance, particularly in built-up members with longitudinal welds. The collapse of four box girder bridges in the early 1970s highlighted the importance of considering these effects, particularly on stiffened plate construction of medium aspect ratio.

Whilst it was not considered to be the prime cause of any of the collapses it was considered to have a significance effect on the ultimate compressive strength of certain elements. In the Interim Box Girder Rules, calculations had to be done to ensure that the residual stresses were kept to a low level by limiting heat input. This of course ran contrary to the aims of BS 5135 which wanted to ensure that sufficient heat input was present to avoid hydrogen cracking! The later bridge rules in BS 5400 have dropped the requirements and the strength rules assume a generous level of residual stress will be present.

Weld distortion

Like residual strength, out of flatness of plates, and out of straightness of stiffeners and struts can reduce compressive strength. It can also magnify stresses locally, which may particularly affect fatigue. The workmanship specifications for buildings and bridges (BS 5950 Part 2 and BS 5400 Part 6, respectively) give limits which are safe for the assumed design strengths in Parts 1 and 3, respectively).

With arc welding it is difficult and expensive to correct distortions to finer tolerances. However, the eventual use of power beam welding has shown great improvements in this respect.

Weld inspection

The quality of welds must be adequate for their required performance. Material properties of the joint are usually checked at the procedure approval stage. In the case of bridges, production run-off plates are often tested for butt welds in flange plates.

Welding discontinuities such as cracks are not permitted as they can cause both fracture and fatigue. Other discontinuities such as lack of fusion or penetration, slag and porosity may be permitted in small amounts in order to avoid unnecessary and harmful repair.

There are no British standards for specifying the appropriate quality for production welding in buildings or bridges, although the fabrication industry has produced one for buildings. This is a serious gap as it is essential for quality assurance purposes that the minimum quality is defined, particularly where fatigue is critical. Often the quality required for procedure tests is used, which is unnecessarily stringent for most production welds.

The methods of inspection are covered by British standards and the new BS 3923 for ultrasonic inspection is particularly detailed. This area is particularly active in the European standardization scene (CEN) at the present time.

Fatigue

It will be seen from the milestones mentioned above that the main factors which could reduce the margins against static failure of welded structures are generally well covered by standards. Hence the risk of early shortfall of the structure's life by one of these mechanisms should be very small.

The one mechanism of failure which does have a directly calculable relationship with design life is fatigue. Fortunately this is not likely to be a problem with normal buildings, and only parts of certain bridges are likely to be controlled by fatigue. Nevertheless if there is a deficiency in the design, the actual life can be a small fraction of the design life.

Premature fatigue failures have been reported in a number of welded bridges both in the United Kingdom and the United States. Most of these have been designed prior to the publication of adequate fatigue rules, which in the United Kingdom did not appear until 1962 in BS 153. Most of the failures involved cracking due to secondary stressing effects and did not constitute an immediate risk to the structure. In the United States cracking was more widespread due to the very much lower static design load. There have been no significant reports of fatigue failures from bridges designed since the UK fatigue rules were published. As this only represents a period of less than 25% of the design life (120 years), it cannot be treated as proof of assurance that the design life will be met in all cases.

One of the major problems with fatigue rules is to make a sensible estimate of the loading over the full life of the structure. These are clearly political problems of overtly anticipating future changes in loading legislation before their time. On the other hand it is essential that a safe estimate is made of both the magnitude of future fatigue loads and their frequency. If the fatigue loading turns out to have been underestimated the cost of future inspection effective enough to detect fatigue cracks in good time will be very substantial.

Current methods of life prediction are based on a very substantial volume of fatigue test data on welded joints collected over a period of 40 years. Whilst exact prediction will always be difficult, most design rules are based on a high confidence of survival (usually 95%) at the end of the design life. As the loading is subject to observation and unexpected increase in damage cannot suddenly occur without warning, it should not be necessary to provide the load factors necessary for protection against unexpected static overload.

Corrosion

Provided that the welded structures are not subject to aggressive chemical attack the use of welding on steel (or aluminium) structures should not have a deterimental effect on corrosion. No special protective treatment is required provided the weld area is properly cleaned. Compared to riveting or bolting the details should be cleaner and less likely to contain crevices for entrapment of water. In fact one of the advantages of welding is its ability to seal round all joints with a permanent durable 'filler'.

Where intermittent welding is used, codes usually recommend maximum spacings to ensure good contact is maintained so that rust cannot separate the parts (a pressure of about 30 N/mm^2 can be exerted by rust).

Welds are relied upon to seal tubular members against ingress of moisture. These are often single run fillet welds with stop-starts. Whilst there do not appear to be reports of corrosion problems in tubular members the difficulties of ensuring a perfect seal should not be underestimated. Welds have been known to burst due to freezing of trapped water.

Most structural codes do not have an overt corrosion allowance (except in the case of weathering steels), in spite of the fact that the corrosion is probably the most likely cause of deterioration. The most vulnerable component is probably the fillet weld whose throat is often substantially less than that of the parent plate. If corrosion does occur, the margins on fillet weld strength may reduce more rapidly than for the member itself.

Creep

Creep is insignificant in welded structural steelwork. Deflections may exceed the theoretical elastic values on first loading on account of residual stress.

Aluminium alloys

The comments above generally apply to welded aluminium alloy structures with the exception of brittle fracture, hydrogen cracking and corrosion, which are not problems with the common structural alloys (5000 and 6000 series). However, special attention has to be paid to softening of heat affected zones. CP 118, the structural design code, is being replaced next year by BS 8118 which will have an integrated weld quality specification.

Conclusions

Confidence in the long term integrity of welded structures is now high in spite of a number of serious failures in the past. These failures have resulted in research to understand the mechanisms, and in the development of a range of design, materials and workmanship standards to ensure that integrity is assured. There are many steel structures doing good service today which were built over 100 years ago. Provided that these design and manufacturing procedures are properly applied there is no reason why today's welded structures should not be doing good service beyond 2090.

The maintenance of masonry

J.HEYMAN

Introduction

Large masonry structures require more or less continuous inspection and maintenance, not to check their stability (for they are extraordinarily stable), but to make sure that the stone is not weathering too severely, that water is not penetrating, that cracks are not extending, and so on. Small 'cosmetic' defects are usually remedied immediately, but every so often (and there is some evidence for a 100–year cycle), more extensive structural restorations are undertaken. In England, for example, in the second half of the nineteenth century, many major churches were repaired by George Gilbert Scott; at about the same time, Viollet-le-Duc was active in France. The end of the twentieth century again sees many restorations under way.

Both Scott and Viollet-le-Duc did much more than merely replace decayed stonework. Both undertook major engineering work, and Viollet-le-Duc's experience and insight into structural behaviour is evident in many of the essays in the ten volumes of his *Dictionnaire raisonné*. In a certain sense, the structural action of masonry is very simple: in another sense, the architect or engineer faced with the restoration of a large cathedral may feel that he would like his common sense remedies to be placed upon some more scientific footing. This present article reaches no new conclusions, and certainly none that would have worried Scott or Viollet-le-Duc, but discusses how such a scientific base might be constructed.

Analysis of masonry buildings

It is both prudent and convenient to regard a masonry building as an assemblage of dry stones, some squared and some not, placed one on another to form a stable structure. Mortar may be used to fill interstices, but this mortar will have been weak initially, may have decayed with time and cannot be assumed to add strength to the construction. Stability of the whole is assured, in fact, by the compaction under gravity of the various elements: a general state of compressive stress will exist, but only feeble tensions can be resisted.

In particular, the shape of the construction will be maintained by the interlocking of the elements; in the case of stones with squared faces, friction forces must act on those faces if there is any tendency for sliding to occur in the fabric. Thus the vertical compressive forces due to gravity act as a kind of prestressing of the masonry, both maintaining overall stability and allowing inclined internal forces to be transmitted without causing either tension or slip.

The magnitude of the compressive forces arising from the self-weight of the material can be assessed quite easily. The stress due to self-weight at the base of a plain wall of uniform thickness is simply ph, where p is the unit weight of the material and h the height of the wall. It is of interest to compare this self-weight stress with the crushing stress of stone. Using rough figures (which, as will be seen immediately, are all that are needed), a crushing stress of a medium-strength stone might be 40000 kN/m^2, while the self-weight might be 20 kN/m^3. Thus, if the wall is built to such a height that crushing just occurs at the base, then the height h is found to be 2000 m. If as much as three-quarters of the wall is cut away at the base by windows, arcades, or the like, then the remaining one-quarter of solid masonry could still support a height of 500 m, an order of magnitude greater than the height of usual construction.

These figures imply that there is a very large factor of safety if the calculations are referred to crushing stress of the stone. Indeed, the most highly stressed elements in an ancient stone building will have average stresses not greater than 1/10 of the crushing strength of the material, and in fact, the main portion of the load-bearing structure will be working at less that 1/100 of the crushing strength; infill panels and subsidiary elements will be subjected to a 'background' stress of say 1/1000.

While the above calculations give order-of-magnitude values only, some significant conclusions may be drawn. A first (and minor) point concerns the level of 'background' stress. Apparently a very small compressive pre-stress is all that is necessary to avoid the dangers of sliding and general loss of cohesion of the masonry. If, then, the engineer wishes to apply some artificial consolidation to his structure, perhaps by post-tensioning horizontal prestressing cables, it is of some help to have a notional reference level against which to measure his pre-stress.

A more important conclusion, however, emerges from the order-of-magnitude calculations. If low *average* values of stress do indeed result from the calculations, then the whole theory of the structural action of masonry becomes part of the wider plastic theory of structures. The assumptions of plastic theory are well known, and need not be detailed at length; for the present purposes the requirements that deflexions should be small implies that there are no slender portions of the structure (such as columns or thin walls) which are liable to buckle. Under these circumstances, the very powerful 'safe theorem' applies. In simple terms, this theorem states that if it is possible to find a system of internal stresses in equilibrium with the external loading, and this system is satisfactory in the sense that there is no danger of crushing of the material, then there is complete assurance that the structure as a whole is safe.

The structural analysis of masonry then becomes something very different from that given by conventional elastic theory. In particular, the calculation of 'redundant' quantities by means of compatibility equations is not required; for example, precise foundation conditions do not have to be determined in order to determine displacements which are used in subsequent calculation. The engineer does not seek to determine the 'actual' state of the structure. Instead, all that he needs to examine is a single state of equilibrium, and he is entitled, indeed he is in some sense encouraged, to seek a favourable state of equilibrium; for example one in which stresses are assumed to be uniformly distributed through the masonry rather than to be low in places accompanied by peaks in others. (But there would be no need to check that 'actual' stresses are uniformly distributed.)

The plastic theory of masonry

A prime principle of restoration of masonry emerges; it is to ensure that the structure as a whole and individual parts of the structure are in a state corresponding to the assumptions of the simple plastic theory of masonry. This apparent inversion of the scientific process, in which theory usually explains practice rather than practice, as here, being required to fit theory, is really no more than good engineering sense. An analogy in steelwork construction would be that the material should possess minimum qualities of strength and ductility; a closer analogy would be the exact specification of the material for reinforced concrete construction.

As an example of the application of this principle, the simple mediaeval masonry wall may be considered. Such a wall, say 1–2 m thick and apparently solid, will usually consist of two thin skins (say up to 200 mm) of good quality coursed stone, with a rubble and mortar infill. In restoring such a wall, care must be taken to ensure that weak fill is strengthened if necessary so that it can bear with comfort the calculated equilibrium stress. If this is assured, then the actual strength of the fill does not matter; the engineer analysing the stresses is not obliged to distribute them in accordance with some notional idea of relative stiffness of skin and fill.

Having made the material of the structure conform to the assumptions of plastic theory, then the engineer can examine further consequences of that theory. He will be working entirely with sets of forces in the masonry, and will not be concerned in general with actual magnitudes of stresses. (There are, of course, elements of the structure where stresses must be calculated; the main crossing piers of a cathedral are such éléments.) What the engineer is concerned with is to ensure that he can indeed find force systems which lie within the masonry. If the forces were compelled to pass outside the masonry (as a line of thrust might be compelled to lie outside the thickness of a poorly designed flying buttress) then this would imply tensile stresses in the stone, and it has been noted that it is realistic to assume that masonry construction is incapable of carrying tension.

Thus the engineer seeks *compressive* force systems in equilibrium with the external loads, and by definition these forces must lie within the masonry; it is not the calculation of magnitudes of stress which is important, but whether the masonry is of the right *shape*. The question, in fact, becomes one of geometry. (As a simple example, it will have been noted that the gravity stresses in a uniform wall are independent of the thickness of the wall. If a thinnish wall blows over under the action of wind, it is because no system of compressive stresses can be found at the base, not even a line load along an edge of the wall with corresponding infinite stresses, to equilibrate both the gravity load and the wind load.) According to plastic principles, the problem of the analysis and design of masonry is properly one of geometry rather than one of stress.

A second principle of restoration of masonry is thus concerned with geometry. If a building has been standing for some little while, this is sufficient assurance, by the safe theorem, that its geometry has been satisfactory. Care must be taken in the restoration to ensure that there are no signs of continuing deformation; the geometry of the structure must be stabilized.

The two principles that have been enunciated as stemming from simple plastic theory, namely that weak infill structure should be strengthened and that geometry should be stabilized, may appear so self-evident as to render slightly absurd the application of high-flown theory. However, some of the conclusions which may be drawn from plastic theory are perhaps not so self-evident. One of these has already been hinted at: that the mere existence of a masonry structure is sufficient assurance that the structure is absolutely safe. The minimum period for a structure to stand might be a few minutes, or perhaps more realistically, to allow for the compaction of slow- setting mortar, say a year. If an existing structure is examined, and shows distortions and cracking, but no signs of continuing distortions, then this deformed existing structure is also absolutely safe. The mere presence of old cracks is not in itself alarming; indeed, it is natural to expect cracking to have occurred as a masonry structure has adapted itself to its environment. A study of this adaptation usually gives a clear indication of the reasons for the presence of the cracks.

The problem of settlement

One environmental change, where time scales may be a generation rather than a year, occurs in the soil supporting the structure. The conclusions so far have been based upon an examination of the masonry alone, but settlements, due for example to the consolidation of soil under foundations, may perhaps have an interaction with masonry. The question here is one of what is meant in conventional structural analysis by 'small-deflexion theory'. The usual assumption of structural theory is that deflexions are small enough not to affect significantly the equilibrium equations, which in turn means that overall geometry changes can be ignored. As has been seen, it is precisely the question of overall geometry which is of prime significance to the stability of masonry.

The kind of geometry change that can result from settlement may be illustrated by noting the behaviour of existing large crossing towers. Examination of initially horizontal masonry courses in the abutting nave, choir and transepts will indicate more or less gross distortions, showing that the tower has settled by as much as say 300 mm. This is both commonplace and straightforward; the four piers supporting the tower, themselves highly stressed, will require high bearing stresses from the soil for their support. Typically, the whole of the plan area of the tower at the crossing, might require a mean supporting stress of 1000 kN/m^2; this is high enough to ensure that consolidation and settlement will inevitably occur, but not so high as to finally distress a stiffish clay.

It is to be expected, then, that a tower will settle in relation to the surrounding portions of the structure, the 'soil-mechanics' time-scale for consolidation, that is, for the reaching of effective equilibrium, is a generation rather than a year. If the settlements are uneven, or otherwise lead to marked changes of geometry of the tower, then the whole tower is likely to fall. On the other hand once this risk period is safely past, and providing that there are no changes in the general condition of the soil, such as would be caused by alterations in the level of the water table, then the tower may well be deemed to be safe structurally.

This 'generation rule' covers a very large number of collapses, both in England and abroad For example, major collapses occurred at Winchester, Gloucester, Worcester and Beauvais (twice), all within a short period of completion of the work. There are one or two examples of survival for much longer periods; the mediaeval tower at Chichester, for example, fell only in 1861, and, at Ely, the crossing tower stood for two centuries before its collapse in 1322. For Ely an explanation may perhaps be sought in the effects of a fluctuating water table in the Fens, and there is evidence that a small start was made on drainage work in the previous century. Certainly an unexplained minor collapse at Ely (of part of the north transept in 1699) followed, by about a half a century, the main draining of the Fens.

In general, however, crossing towers which still stand, although they may betray large and relatively even settlement, usually show no signs of any movement having occurred in the last few centuries. In such cases, the course of action indicated to the restorer by the findings of simple plastic theory is to leave well alone. There are many practical examples of the successful application of this do-nothing principle; although those responsible may have been worried, at least they did not offend intuitive judgement by carrying out needless strengthening works.

Thus, to summarize, the following principles govern the restoration of masonry structures:

1. Loose masonry and rubble infill must be consolidated to a sufficient strength. The strength need not be that of the best close-coursed masonry, but must be such that calculated ambient stresses are relatively small compared with the strength.
2. If there is evidence of continuing distortion of the structure, or if it is known that permanent forces (such as thrust from roofs) might act to distort the structure in future, then steps must be taken to stabilize the geometry as a whole.
3. If there is no evidence of such continuning distortion, although permanent distortion may exist due to causes which no longer operate (e.g. settlement of foundations), then no action need be taken.

Aseismic design and the lifetime concept

T.P.TASSIOS

Introduction

It has been recognized that despite its considerable significance (notional as it may be) (see Figure 1) [1], the lifetime concept resists explicit modelling and numerical evaluation of its 'absolute' value. However, such a concept may be much easier to apply in comparing various design, exposure and maintenance circumstances; for such a relativistic use, absolute lifetime values are no longer necessary and the concept exhibits its potentiality without liability problems eventually produced because of its large uncertainties.

It has to be accepted that the fewer the 'ageing causes' (e.g. just CO_2, or just erosion and humid/dry cycles), the easier the handling of the concept. In other words, when several 'ageing causes' may act simultaneously on the structure, their synergetic effect becomes so complicated and uncertain that it is hard to speak in terms of lifetime 'prediction'. This is also reflected in the practical tables of classification of aggressive environments included in codes; synergy of pathological causes is rarely considered.

The situation seems to be more difficult to handle when accidental actions have to be considered, with or without environmental aggressivity. In this connection, it has to be remembered that design against accidental actions used to be based on deemed to satisfy rules rather than rational reliability methods.

In what follows, however, an attempt is made to discuss possible uses of the lifetime concept in the particular case of aseismic design.

A series of small quakes

A first possible way to think in terms of a lifetime of a structure exposed to earthquakes would apply in the case of slightly seismic areas where, with a rather abnormal recurring period, small to moderate seismic actions are expected. In such a case, the inherent overstrengths of traditionally designed and built buildings may offer sufficient instant safety margins against these recurring horizontal loadings. However the repetition of a large number of moderate cycles through decades (or centuries) may lead to an oligocyclic fatigue of some

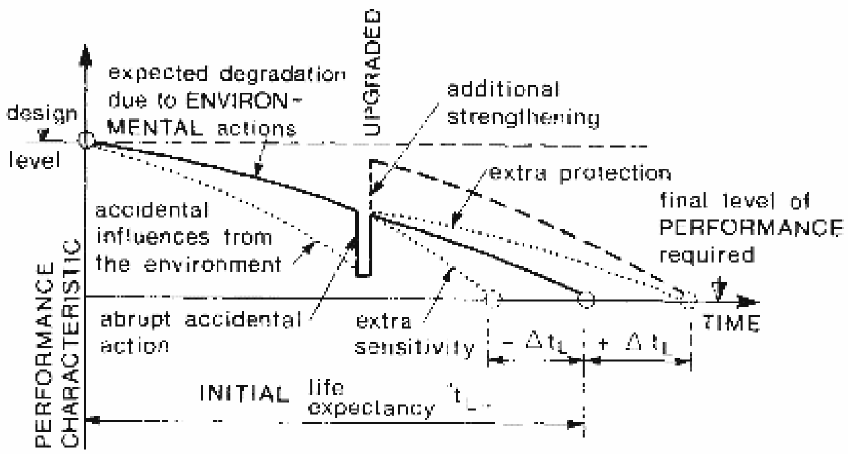

Figure 1 Lifetime is a very useful concept, especially in cases of compared circumstances.

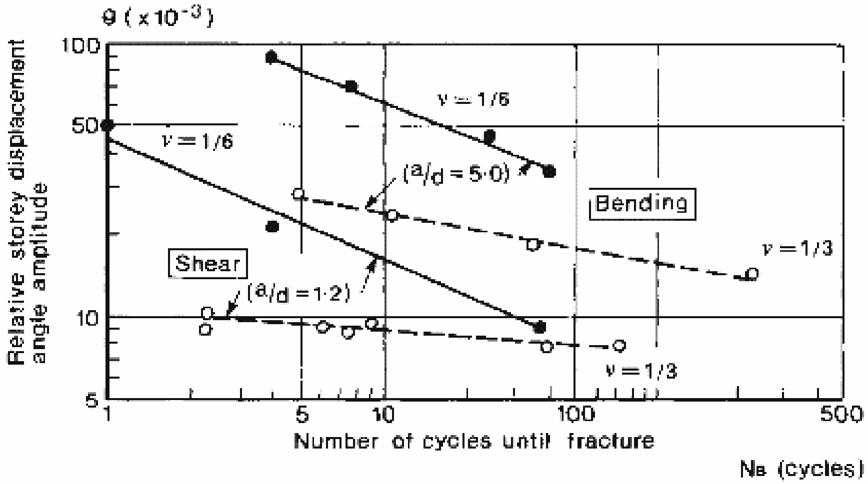

Figure 2 Relationship between relative storey displacement (angle amplitude ?) and number of cycles until fracture N_B [2], depending on shear ratio *a:d* of columns.

critical areas. Such would be the case of short R.C. columns without special confinement, as illustrated in Figure 2 [2].

Similar damage accumulation sensitivity has been observed in masonry buildings. If this is the case, then for an expected number N_0 of repetitions of given interstorey drifts (θ in Figure 2) produced by each major seismic event, recurring every T_0 years, one could have a rough estimate of a corresponding lifetime

$$L_t \approx \frac{N_B}{N_0} T_0 \tag{1}$$

where N_B denotes the limit number of repetitions of θ leading to fracture. Obviously, this is only a pseudo-quantitative approach; in real life the probabilistic nature of all quantities entering Eqn. (1) renders such a numerical evaluation almost impossible. However, it is possible to say that under the hypothetical conditions of Figure 2, low normalized values of axial load *(v)* may secure a considerably longer seismic lifetime.

Under strong earthquake conditions

In highly seismic areas, where seismic design has to face the imposed strong action effects directly, the lifetime concept may also be encountered (indirectly though) via the so called socially acceptable probability of 'failure'. The concept of the generalized cost function is useful in this connection:

$$K_G = K_0 + k_r P_f \tag{2}$$

where K_0=the capitalized investment and maintenance costs, k_r=total 'repair' costs (including non-monetary losses) and P_f=acceptable probability of 'failure'.

In order to minimize the generalized cost, suitable P_f -values would be sought. It is reminded that where the money is expensive and the 'cost' of human life is low, high P_f -values are considered as optimum. The opposite is true in rich countries. Thus, for the same seismic conditions, 'poor' countries may use lower seismic coefficients than 'rich' ones. By way of consequence, low and long seismic lifetimes are expected correspondingly.

Combined environmental and seismic actions

A third way to consider the lifetime concept in seismic areas is to examine the rather disproportionate effects of the synergy of aggressive environments and earthquake events.

First, one of the most decisive characteristics shaping the seismic behaviour of R.C. structures, i.e. ductility, is distinctively affected by aggressive environments. In fact, as far as reinforcement is concerned, corrosion may produce steel embrittlement, i.e. a drastic reduction of its elongation at rupture, without however affecting its yield-limit. (Obviously, surface oxidation, e.g. in marine environments, may directly affect the ultimate capacity of R.C. critical regions because of reduction of steel bars' cross sections.) Pre-stressed steel is also suffering a similar and possibly more dangerous embrittlement in some aggressive environments (Figure 3) [3].

On the other hand, spalling of concrete due to steel corrosion may lead to abrupt loss of bond, contributing to reduction of energy dissipation capacity. All in all, it is expected that the synergy of an aggressive environment with a future earthquake may lead to a drastic reduction of lifetime expectancy. In this

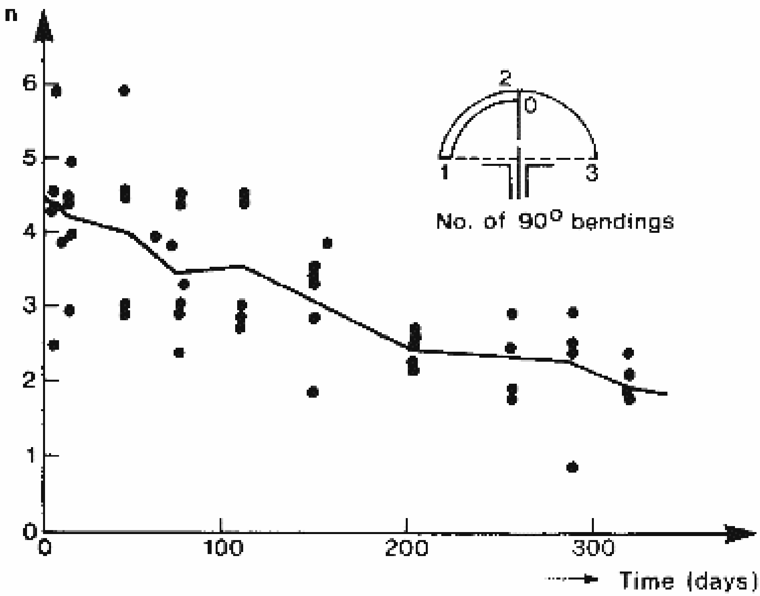

Figure 3 Gradual embrittlement of pre-stressing steel φ7 expressed as a reduction of the number of 90° bandings necessary to produce rupture. The steel was kept embedded in slightly corrosive soil (Tassios 1983).

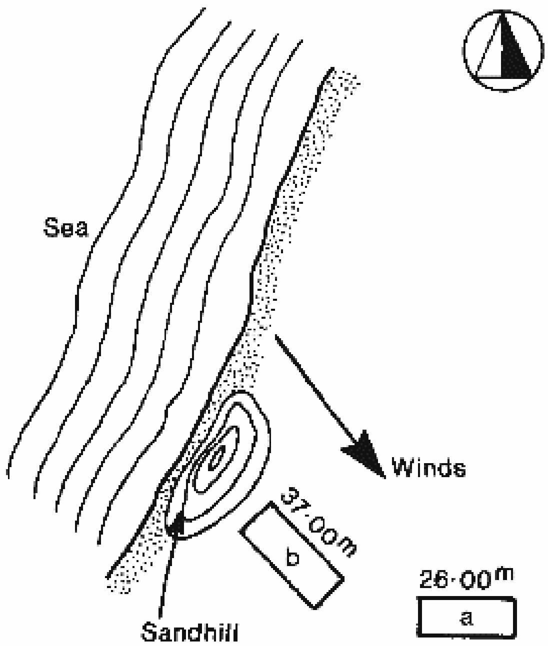

Figure 4 Building (b), better protected against the prevailing winds, has suffered less seismic damage than building (a).

connection, it is interesting to report the behaviour of two hotel buildings situated on the coast of Kyllēnē (west Greece).

Two 15-year-old structurally identical buildings (Figure 4), belonging to a tourist resort (Kastro Kyllēnē, west coast of Greece), were stricken by a series of earthquakes (M=5, 22 September and 30 September and M=6, 16 October, 1988). The damage to these specific buildings was mainly due to their flexibility; little real structural damage of the framework itself was observed. However, damage to building (a) was remarkably higher than that to building (b). In other words despite their structural identity, building (a) proved to be much more flexible than building (b) because of the differential level of previous damage produced by corrosion. Building (b) was well protected from the prevailing seawinds, since it was hidden behind a high sandhill. As an estimator of the level of previously produced corrosion damage, the total length of spalled concrete along the bars of columns was measured. The ratio of these damage lengths was found to be larger than 3:1 for buildings (a) and (b), respectively.

Thus, it was reasonably assumed that bond loss and reduction in concrete cross section after corrosion contributed to a considerable stiffness reduction of columns in building (a); the subsequent seismic damage was therefore higher. This is only one of the examples of possible synergies producing a shorter seismic life of a structure. However, modelling of such synergies is still missing, whereas field experience is not yet sufficiently rich for empirical rules to be established; but the rationale has been made clear and the way is open for future research along these lines.

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The Europe an approach to design life

S.ROSTAM

Durability versus service life

In recent years, concern with the durability of structures has focused much attention on 'how', 'why' and 'when' structures deteriorate.

The present costs of maintaining and repairing prematurely deteriorated concrete structures have in some cases reached levels threatening the welfare of nations or regions. As concrete is, and will remain to be, by far our most important and most widely used building material, a rational solution to the durability problem simply must be found.

It has been realized that the interaction between a structure and an aggressive environment inevitably will cause the structural material to age and decay. The concern now focuses on how the decay takes place and at what rate.

Introducing the rate of decay, i.e. the *time factor,* was a major step forward in the understanding of the durability of structures. This enables the non-quantitative *durability problem* to be treated in a quantitative way as a *service life problem*.

Concrete

One of the first clear conceptual treatments of the 'service life of structures' was presented by Fagerlund [1]. Subsequently the service life approach has been adopted by numerous researchers, international professional organizations, and has now reached the code drafting bodies. One such organization which has made a strong effort during the past decade to raise the level of understanding of the engineering level of the service life of structures is the CEB (Comité Euro-International du Béton). This work has been performed by the CEB-General Task Group No.20: Durability and Service Life. The work is reflected in the CEB Bulletins [2–6], and a novel chapter on *durability* has been introduced in the draft CEB Model Code for Concrete Structures 1990 [7] (MC 90) now in press. Part of these activities form the basis of this paper.

Specified service life

The service life generally expected from ordinary housing is of the order of 50–75 years. The codes and standards do not specifically state such values, but nevertheless the large general experience which usually exists on the durability of structures is reflected in the requirements of the National Standards.

The current British Code of Practice for bridges, BS 5400, specifically states that the expected service life following the requirements of the code is 120 years.

Major structures for which a required long service life has been specifically stated are:

- The Delta Storm Barrier in the Netherlands with a requirement of 200 years
- The Channel Tunnel between France and the United Kingdom with a requirement of 120 years
- The Great Belt Link, Denmark, with a requirement of 100 years

Service life calculations

In spite of these specific requirements no well established accurate methods for service life calculations are available, and the values represent *wishful thinking* more than rational design criteria. Nevertheless, by specifying such long intended service life a clear signal is given indicating that all efforts shall be made to apply present day knowledge in achieving as good, as durable and as robust a structure as possible.

Adopting the service life concept has opened our eyes for the need to develop engineering models for the deterioration of structures subjected to different types of aggressive environments.

Deterioration mechanisms for concrete

In view of the severity of the concrete durability problem it is interesting to notice that the number of really significant deterioration mechanisms can be reduced to four:

- Reinforcement corrosion
- Freeze-thaw bursting
- Alkali-aggregate expansive reactions (AAR)
- Chemical attacks (including sulphates)

The first destroys primarily the reinforcement and the other three primarily destroy the concrete.

Service life design concept

Objectives

The objectives of a service life design can be expressed as follows [7]:

> Concrete structures shall be designed, constructed and operated in such a way that, under the expected environmental influences, they maintain their safety, serviceability and acceptable appearance during an explicit or implicit period of time without requiring unforeseen high costs for maintenance and repair.

Achieving the objectives

The whole process of creating structures and keeping them in satisfactory use and service depends on a combination of the following:

- The *design provisions taken*
- The type and composition of the *materials* chosen
- The *workmanship* obtained
- The *maintenance strategy* and techniques foreseen
- The level of *quality assurance* selected throughout the whole process to ensure a satisfactory reliability

This involves not only technical aspects but certainly also economic considerations as well as the close and pre-planned cooperation between the following four involved parties:

- *The owner,* by defining his present and possible foreseen future demands and wishes, if any
- *The designer* (engineer and architect), by preparing design specifications (including proposed quality control schemes) and conditions
- *The contractor,* who shall follow these intentions in his construction works
- *The user,* who will normally be responsible for the maintenance of the structure during the period of use.

Any of these four parties may, by their actions or lack of actions, contribute to an unsatisfactory state of durability of the structure and thus cause a reduction of the service life. Also interaction between any two of the parties may cause faults which can have an adverse effect on durability and service life.

The service life should be obtained without relying on special additional protection needing frequent maintenance or replacement. However, in cases of especially aggressive environments special additional protective measures may be foreseen.

A design should also include provisions to ensure satisfactory weathering and ageing of exposed surfaces thus allowing buildings to *grow old gracefully*.

Design strategy

The design strategy should consider possible measures to protect the structure against premature deterioration. A set of appropriate measures (one or more) can be combined to ensure that the required service life is obtained with a sufficiently high probability. The different measures may act simultaneously in contributing to the protection, or one measure may be substituted by the next, once the former has been overcome or eliminated by the aggressive substance threatening the structure in question.

Multi-stage protection strategy

Service life design in practice may thus profit from a multitude of protective measures selectively chosen to cooperate in order to ensure the required service life with an acceptable level of reliability. This innovative design strategy is considered a *multi-stage protection strategy* which leaves the selection of the individual protective measures to the designer. Furthermore, the design shall wherever possible ensure adequate access to all parts of the structure, including voids and equipment, to allow for inspection and possible maintenance to be performed, i.e. the structure shall be available for inspection and maintenance.

CEB-FIP—Model Code 1990

Specific recommendations on Design for Service Life have recently been presented in MC 90, chapter 8 [7], to which reference is made.

Special protective measures

In especially aggressive environments special protective measures may be required to ensure a satisfactory service life. Examples of such environments would be splash zones and zones just above in saline waters, areas with direct exposure or splash from de-icing salts, concrete lined tunnels in saline environments.

Three examples of special protective measure are stated in the following:

Increased concrete cover with skin reinforcement

Two different strategies may be followed:

(a) The cover on the skin reinforcement is considered a sacrificial cover which may spall some time in the future if the skin reinforcement corrodes. In this case the skin reinforcement also acts as a sacrificial anode protecting the main reinforcement. The outer part of the cover is not taken into account in the load carrying capacity. Stiffness and restraining forces are calculated with and without this extra cover,

(b) The skin reinforcement is specially protected, e.g. by:
   (i) Epoxy-coating. It should be ensured that there is no electric contact between the coated skin reinforcement and the uncoated main reinforcement. By maintaining uncoated main reinforcement (coating of single bars), a future installation of cathodic protection is a valuable option, should this prove necessary some time in the future (multi-stage protection strategy).
   (ii) Selecting stainless steel. There is no restriction in electric connections to the main reinforcement.

Epoxy coated reinforcement

Fusion bonded epoxy coating of reinforcement may provide a long term reliable barrier against corrosion of reinforcement due to either carbonation or chloride ingress.

Coated bars have different bond and anchorage characteristics which should be taken into account in the design and detailing of the reinforcement.

Selecting epoxy-coated reinforcement (coating of single bars) rules out the possibility of installing cathodic protection in the future, should this become necessary. By coating prefabricated welded reinforcement cages by fluidized bed dipping, the valuable option of cathodic protection as a back-up facility is maintained.

Cathodic protection

Cathodic protection of reinforcement may provide a reliable protection against corrosion even in cases where very high chloride concentrations occur. Such protection may also be achieved for reinforcement where corrosion has started.

For structural parts in the air, anodes should be placed on the concrete surface either distributed or localized, and the protection based on an impressed current system. Several surface mounted anodes need a conductive overlay to ensure the current distribution to the reinforcement. The deadload of such overlays should be considered in the design.

Cathodic protection of pre-stressed structures is considered possible, but due to the increased risk of hydrogen embrittlement of the prestressing steel, care should be taken in the design and especially in the monitoring and operation of the system to avoid overprotection and hydrogen development at the reinforcement.

Initial electric bonding of the reinforcement in new structures will greatly facilitate the installation of cathodic protection some time in the future, should this prove necessary.

The Great Belt Link

Construction of the Great Belt Link between Funen and Zealand in Denmark is one of the largest transportation projects presently undertaken in Europe and the largest project in Denmark to date (Figure 1).

The complete link will be approximately 18km long and the construction costs will be approximately DKK 20 billion=US$ 3.0 billion (1990 prices).

The link will include three major structures:

- A twin bored railway tunnel under the eastern channel
- A combined road and railway bridge across the western channel
- A high level road bridge over the eastern channel.

The construction will take place in two stages with the opening of the rail link in 1994 and the road link in 1997.

The Eastern Tunnel

The tunnel will consist of 7.7 m diameter separate tubes for trains in each direction, with cross passages at 250 m intervals for mechanical and electrical equipment and emergency escape routes.

The spacing between the tube centres is 25 m. A section in the tunnel and cross passage is shown in Figure 2. All linings are segmentally bolted precast segments.

The approximately 8.0 km length of the tunnels is carried out as bored tunnels over the central 7400 m with short transitions of cut and cover tunnels at each end totalling 550 m.

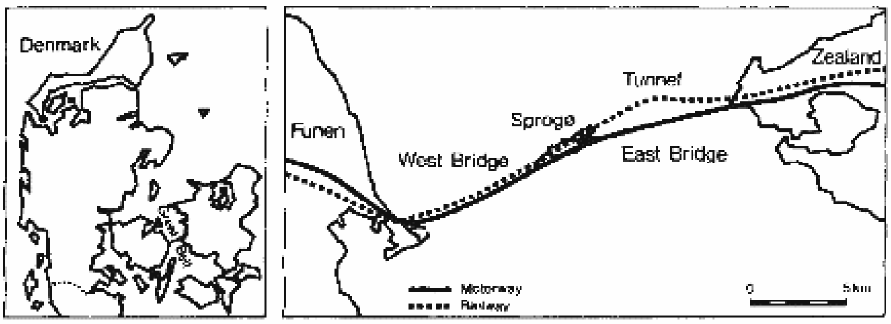

Figure 1 The Great Belt Link, Denmark.

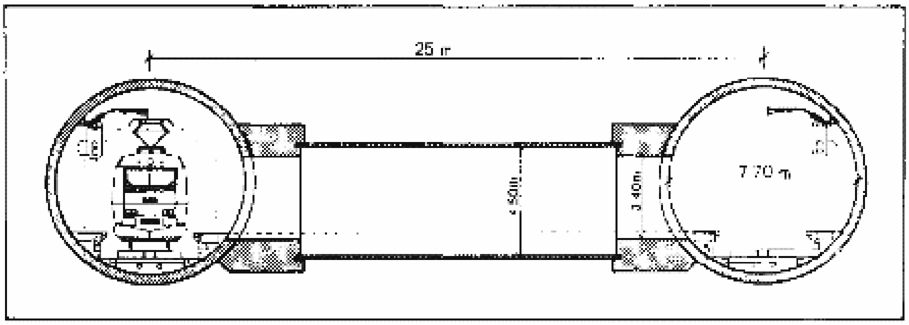

Figure 2 Eastern Railway Tunnel: cross section in bored tunnel.

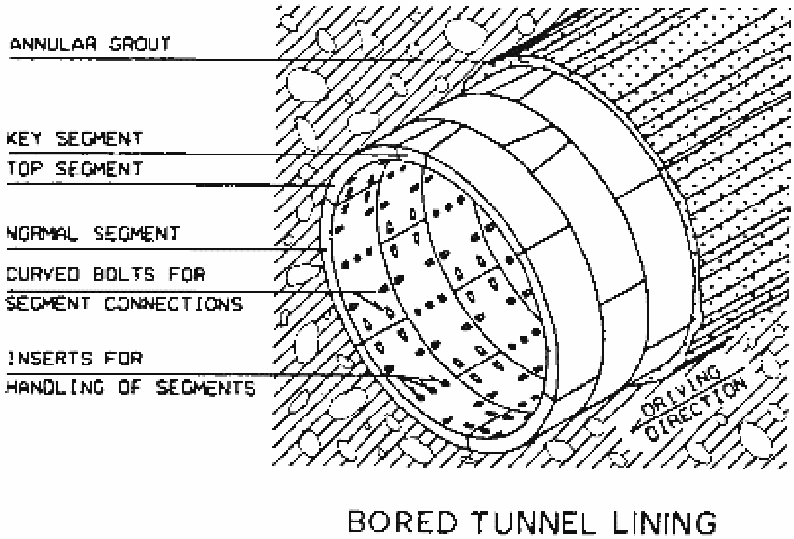

Figure 3 Precast segmental concrete lining with epoxy-coated fully welded reinforcement cages.

The main tunnels will generally be lined with a 400mm thick segmental reinforced concrete lining with six segments plus a key as shown in Figure 3. The lining is designed to be waterproof for the full hydrostatic pressure of 8 bar. Water tightness is obtained by use of gaskets between segments.

The design consulting engineers for the eastern railway tunnel are a joint venture of COWIconsult and Mott, Hay and Anderson International, the latter being part of the Mott MacDonald Group.

Lining durability

The lining of the bored tunnel consists of 1.65 m wide and 400 mm thick precast concrete segments.

High groundwater chloride and sulphate levels combined with a hydrostatic pressure means that dense high quality concrete is essential for the segments. In the light of growing evidence of deterioration of existing reinforced concrete lined tunnels in saline ground conditions, a comprehensive study was undertaken at an early stage of the tender design to establish criteria for the various components of the structure providing for minimum maintenance requirements.

A number of protective measures were considered of which coating to the external surfaces of the segments and epoxy-coating of the reinforcement cages were included as specified variants in the tender documents. On the basis of pricing, it was decided to use epoxy-coated reinforcement.

The coating constitutes part of the overall 100 years service life design approach adopted for the project. This is based on a multi-stage protection strategy providing successive barriers against incoming chlorides and sulphates. The four main barriers against chloride corrosion are:

- Annular grout filling between the lining and the soil
- Concrete composition including flyash, micro silica and OPC with a W/C-ratio of max. 0.35
- Epoxy-coated reinforcement
- The possibility of providing cathodic protection if corrosion should occur some time in the future. The cages are fully welded prior to coating which ensures electrical contact between all bars in a cage and thus makes cathodic protection a realistic future option.

The configuration of the reinfocement cages for the segments led to the use of the fluidized bed technique for epoxy-coating where the complete cage is coated by heating and dipping in a tank full of epoxy powder fluidized by air jets. The powder fuses to the preheated cage giving a uniform protective layer of epoxy.

Although used for coating pipes and valves in the pipeline industry, this technique has not been employed on prefabricated reinforcement cages before on any large scale. An extensive test programme was therefore carried out during the tender design both to confirm the practicability of the system in this application and to establish the effect of the coating on the bond to the reinforcement. It was also necessary to determine any difference in structural behaviour at the radial joints, where the welded mats are used to resist splitting of the joint. The testing programmes showed that the technique was feasible for the reinforcement cages and that only minor modifications were required to the structural design and detailing of the segments.

West Bridge

West of Sprogø the Great Belt Link combines in the 6.6 km long high speed railway and 4-lane motorway bridge from Sprogø to Funen.

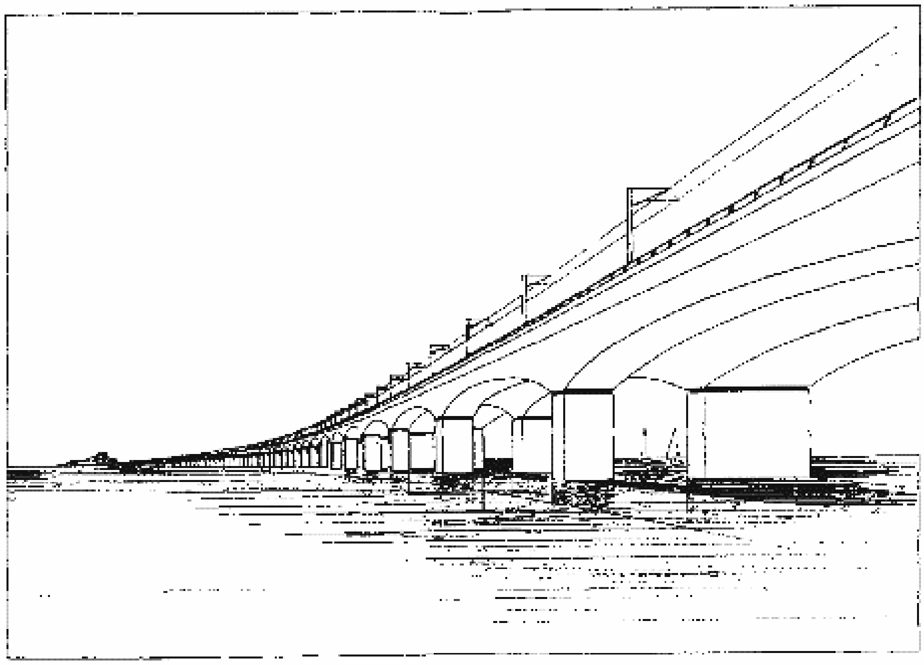

Figure 4 West Bridge: perspective view.

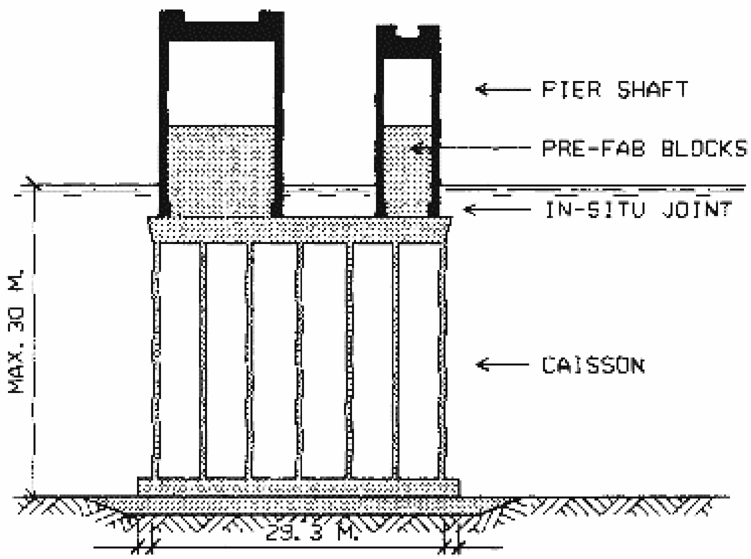

Figure 5 Precast concrete caisson and pier shafts.

Figure 4 provides a perspective view of the bridge alternative now being constructed by the European Storebælt Group.

The pier shafts have rectangular hollow cross sections with a constant wall thickness. The pier shafts are capped by a 2.5 m plinth. In the lower part of the pier shafts, concrete fill is provided by prefabricated blocks (see Figure 5).

The superstructure elements are 110.4 m long haunched box girders with depth decreasing parabolically from 8.70 m at the piers to 5.73 m at mid-span for the railway girder. The corresponding dimensions for the roadway girder are 7.34 m and 3.78 m, respectively. Each girder will be cast in a préfabrication yard. Figures 6 and 7 show sections in the bridge superstructure.

The installation of the bridge elements will be performed by means of a large U-shaped crane vessel with a lifting capacity of approximately 6500 tons. This vessel will place all 324 units, i.e. caissons, pier shafts as well as bridge girders.

Durability of pier shafts

When considering protective measures for the bridge piers standing in saline waters, the following options have been considered:

- Smooth, uncracked and impermeable concrete surface
- Rounded outward corners
- Minimum of construction joints on exposed surfaces
- Sloping horizontal faces where ponding may occur (alternative: surface coating)

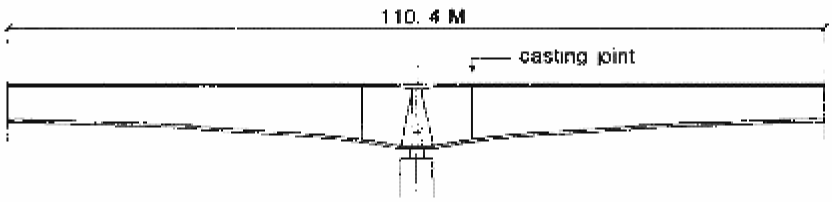

Figure 6 Precast concrete bridge superstructure: longitudinal section.

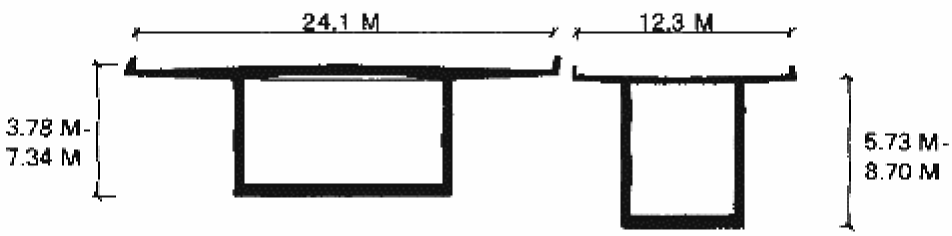

Figure 7 Superstructure: cross sections.

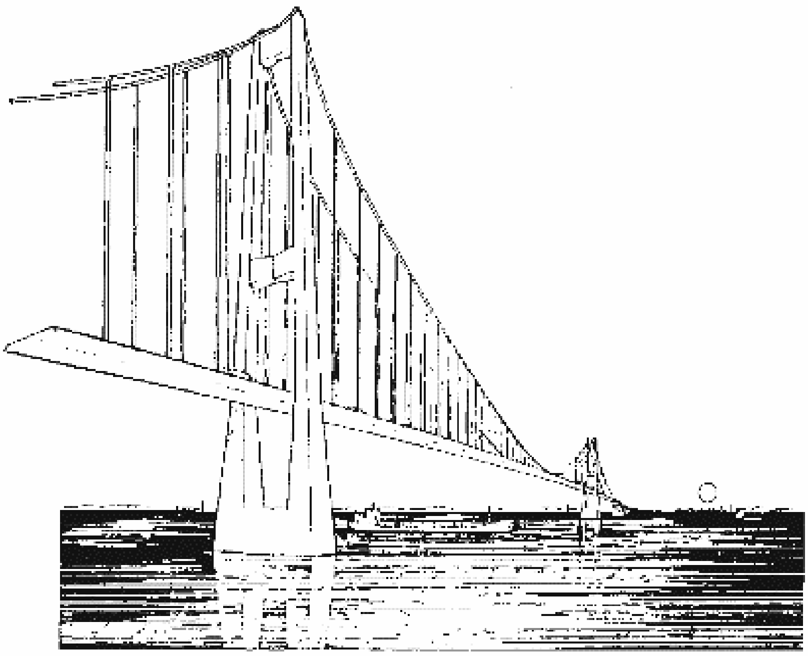

Figure 8 East bridge: perspective view of 1700 m record main span suspension bridge.

- 100 mm concrete cover with sacrificial skin reinforcement (corroding thus protecting the main reinforcement, and with subsequent spalling of its cover); alternative is non-corroding skin reinforcement of stainless steel or epoxy-coating
- 75 mm concrete cover with epoxy-coated reinforcement
- Liner in splash zone, e.g. stainless steel or surface coating
- Provision for future cathodic protection (electric bonding)

Each option has a different overall effect and degree of reliability, but, if a careful selection of a number of these options is made, very long durability may be expected.

Durability of superstructures

The following protective measures have been chosen:

- Rounding of outward edges (radius=50 mm)
- Concreted cover of 50 mm (+5 mm tolerance)

Furthermore waterproofing membranes will be used on decks.

East Bridge

The third major component of the Great Belt Link is the high level road bridge across the Eastern Channel. This will be a 1700 m record breaking span suspension bridge, and will be the most spectacular part of the link (Figure 8). It will be visible at great distance and the impact on the seascape will be considerable.

The main bridge will be a suspension structure with a main span of 1624 m and two side spans each of 535 m. The bridge Towers will rise 254 m above water level and may be constructed either in steel or concrete.

The tendet design has made the following provisions to ensure adequate durability for the most critical parts of the concrete substructure:

- Rounding of outward corners
- Main proposal: 100 mm cover, with a stainless steel mesh as skin reinforcement
- Alternative proposal: 75 mm cover, and epoxy-coated reinforcement
- Additional option: surface treatment of the concrete with water repellent silane

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Design life of concrete highway structures—the North American scene

D.G.MANNING

Introduction

The design life of a structure is specified in the relevant design code. It is a somewhat arbitrary figure, often based on the estimated time to functional obsolescence of the structure, which has relatively minor effect on the actual design of the structure. The basic requirement of design codes based on the limit states design philosophy is that the factored resistance shall exceed the total factored load effect at the ultimate limit state. The load factors applied to the loads and the performance factors applied to the resistances are determined using a calibration process based on the design life and this establishes the safety index for the structure. The assumption of a specific design life also determines such design parameters as the wind return period, the number of loading cycles for fatigue calculations, and, indirectly, the hydraulic capacity to be provided for river crossings.

In reality, the concept of design life is rather nebulous. The end of the life of a structure has little to do with its design life, but is a decision made by the owner on the basis of economics and functionality. The actual period a structure remains in use may be more or less than the design life and this gives rise to the term service life. Several definitions for service life have been developed. One of the simplest is that adopted by Committee 365 of the American Concrete Institute: The period of time for which the structure performs its intended function'.

Predicting the service life of concrete structures is a relatively new, but very topical subject which is being studied by committees in most of the major technical societies. This paper provides a brief review from the North American perspective, of the factors which affect the service life of new and existing concrete structures. Most of the examples are taken from highway bridges with particular emphasis on the effects of corrosion.

New construction

Whereas the assumption of a design life for a structure establishes the load effects with reasonable accuracy, the service life is determined largely by durability considerations which are difficult to quantify. This is mainly due to the interaction of the large number of factors with respect to material properties, construction details, exposure conditions, degradation mechanisms, and quality of maintenance, which affect durability. Two basic approaches are possible at the design stage:

1. Design the most durable concrete feasible and assume it will have the desired life
2. Develop performance criteria based on factors controlling the serviceability of a component or structure

The first approach can be charitably considered as an act of faith tempered by engineering judgement. The second is a desirable goal towards which significant progress has been made but much remains to be done before durability can be adequately defined as a serviceability limit state in design codes. When faced with determining the service life of a new concrete structure, a number of methods can be used. These are, in order of increasing complexity:

1. Estimates based on experience
2. A comparative approach
3. Accelerated testing
4. Stochastic methods
5. Mathematical and simulation modelling

The first method relies almost entirely on engineering judgement, whereas the second relies on deductions made from the performance of similar quality concretes in similar exposure conditions. However, the major disadvantage of both these methods is that many of the materials used in concrete today such as supplementary cementing materials and admixtures, and protective treatments used to enhance the performance of concrete such as surface treatments and coated reinforcement have a relatively short performance history. Accelerated testing is sometimes used to estimate the service life of new materials and can be very useful in predicting comparative performance and the mechanism of failure providing that the artificial environment does not induce forms of degradation which do not occur in the service environment. Stochastic methods rely on either reliability methods or a combination of statistical and deterministic methods to predict the probability of time to failure. These methods rely on the analysis of performance data without consideration of the mechanisms involved, in contrast to mathematical and simulation techniques which do require that the mechanism of degradation be understood. Simplified forms of some of the mathematical models for the more common forms of deterioration are shown in Figure 1.

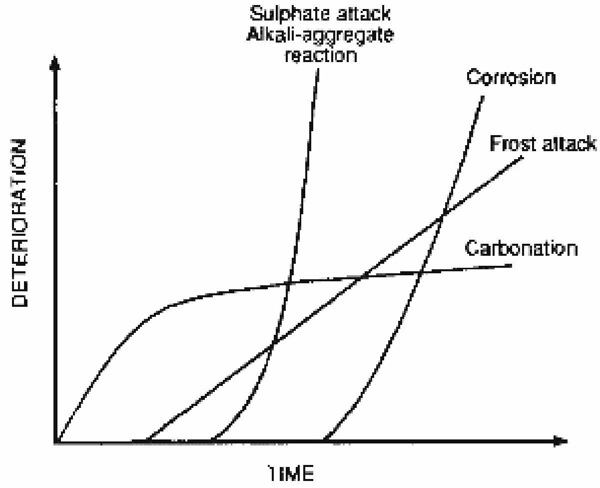

Figure 1 Simplified deterioration-time models for common mechanisms of degradation.

The models can be expressed mathematically as follows:

- Carbonation: $d = kt^{1/2}$
- Corrosion: ; $d = t_0 + kt^a$ a=0.5 for diffusion control, a=1.0 for reaction control
- Sulphate attack: $d = t_0 + kt^a$
- Alkali-aggregate reactions: $d = t_0 + kt^a$, a>1.0
- Frost attack: $d = t_0 + kt$

where d is the deterioration, t_0 is the initiation time, k is a constant (for a particular mechanism and concrete) and t is the time.

Current practice

While substantial progress has been made towards developing performance criteria and mathematical models, current practice relies on rules based largely on experience and, to a lesser extent, on accelerated testing. Perhaps the most significant aspect of the service life of concrete high-way bridges in North America in the past thirty years has been the poor corrosion performance resulting from underestimating the severity of the service environment and especially the effect of de-icing chemicals. As the observations of structures revealed that corrosion protection was inadequate, this resulted in a series of incremental improvements in concrete quality, concrete cover and positive corrosion protection treatments such as waterproofing membranes and epoxy-coated reinforcement as shown for the case of the Province of Ontario in Table 1. Additional quality assurance provisions were also introduced to ensure that the higher standards of construction were achieved. In retrospect, premature corrosion damage in exposed concrete deck slabs with shallow covers and

Table 1. Deck protection requirements in Ontario during the period 1958–1986

Year	Concrete strength (28 days)	Thickness of thin slabs	Cover to top mat	Hot mix	Other requirements
Pre 1958	3000 psi	7 in	min 1 in	Yes	Not air entrained. not waterproofe
1958–1961	3000 psi	7 in	min 1 in	Yes	Air entrainment introduced; silicon prior to paving
1961–1965	3000 psi	7 in	min 1 in	Yes	Mastic asphalt or glass fiber in an membrane
1965–1972	4000 psi	7.5 in	min 1.5 in	No	Two coats of linseed oil and kerose construction
1972–1975	4000 psi	7.5 in	min1.5 in	Yes	Hot applied rubberized membrane; added in 1974
1975–1978	4000 psi	7.5 in	2.5 ± 0.5 in	Yes	
1978–1981	4000 psi	7.5 in	2.5 ± 0.5 in	Yes	Epoxy-coated reinforcement in top
1981–1986	30 MPa	225 mm	70 ± 20 mm	Yes	
1986–	30 MPa	225 mm	70 ± 20 mm	Yes	Additional requirement for epoxy-c cantilever section of decks witho

subject to frequent applications of de-icing chemicals should have been predictable. However, the practices given in Table 1 were considered state-of-the-art in bridge design, and corrosion protection for private sector construction in similar exposure conditions lagged the public sector by about a decade.

A positive outcome of the deficient performance of highway bridges was the recognition of the need to define the service environment much more rigorously, not only for the structure as a whole, but for the individual components of the structure. This recognition of the microclimate, in addition to the macroclimate, has resulted in differing degrees of corrosion protection being provided to the various components of a bridge depending upon the severity of the exposure conditions of the component and other factors such as whether the component is pre-stressed. The concept is very similar to the 'zone defence' approach to the painting of structural steel in which large areas such as the web of a girder receive one level of treatment, areas such as the top face of the bottom flange where contaminants might accumulate and drainage is poor receive a greater degree of protection, and the edges of the flange receive the greatest amount of protection. The current requirements in Ontario are summarized in Table 2. For the most severe exposure conditions, multiple protection systems are used, such that redundancy is provided.

Recognition of the effects of microclimate also emphasized the effects of detailing practice on field performance. Measurements of the chloride ion content of deck soffits showed values below the threshold value for corrosion except where the soffit was exposed to surface run-off flowing through handrails or leaking expansion joints, and, in the case of twin bridges, open longitudinal joints. Parapet or barrier walls were found to offer very effective protection to the deck soffits. Corrosion problems were most acute for thick slab decks on a superelevation, in which case, the whole soffit could be contaminated. A new drip detail was developed on the basis of laboratory testing. Where open railings or longitudinal joints are used, design details were changed to require the use of additional epoxy-coated reinforcement, as shown in Figure 2. The number of coated bars adjacent to transverse joints was also increased.

Finally, there has been a recognition that some components of the structure, specifically expansion joints and some types of bearings, cannot be made sufficiently durable to perform satisfactorily for the design life of the structure. Accordingly, these components are designed to be replaceable. This approach is consistent with the concept of designing structures with three categories of components:

1. Components which, with little or no maintenance, have the same life as the service life of the structure, e.g. primary load-carrying components
2. Components which, with periodic maintenance, have the same life as the service life of the structure, e.g. deck with a bituminous surfacing or barrier wall with a concrete sealer

Table 2. Corrosion protection requirements for concrete bridge components in Ontario

Structure component	Minimum concrete thickness (m)	Maximum w/c of concrete	Reinforcing steel to be epoxy coated	Concrete cover to steel (mm)
Deck	225	0.45 (reinforced) 0.40 (post-tensioned)	Top mat[a]	70 ± 20 (top mat) 40 ± 10 (bottom bat)
Barrier wall	200 (top of wall)	0.45	All bars	70 ± 20
Beams: pretensioned concrete		–[b]	All bars within 3 m of a transverse joint[c]	$28 + 5$ $28 - 3$
Substructure components: Group 1 elements directly exposed to salt splash and/or roadway drainage		0.45	All bars within 100 mm of exposed face[d]	80 ± 20 (exposed face)
Group 2 elements indirectly exposed to salt	–	0.45	None	95 ± 20 (exposed face)
Group 3 elements not exposed to salt	–	0.50	None	In accordance with OHBDC[e]

[a]Epoxy-coated steel is also used for the bottom mat in the cantilever portion of thin slab decks for designs with open railings or longitudinal joints. Thick slab decks have additional requirements which are dependent on the superelevation. All decks have a waterproofing membrane, protection board and 80 mm hot mix.
[b]Specified by strength, typically 30 MPa at transfer, 40 MPa at 28 days.
[c]Additional requirements for exterior girders for designs with open railings or longitudinal joints.
[d]Components exposed to discharge from deck drains are treated individually.
[e]OHBDC, Ontario Highway Bridge Design Code.

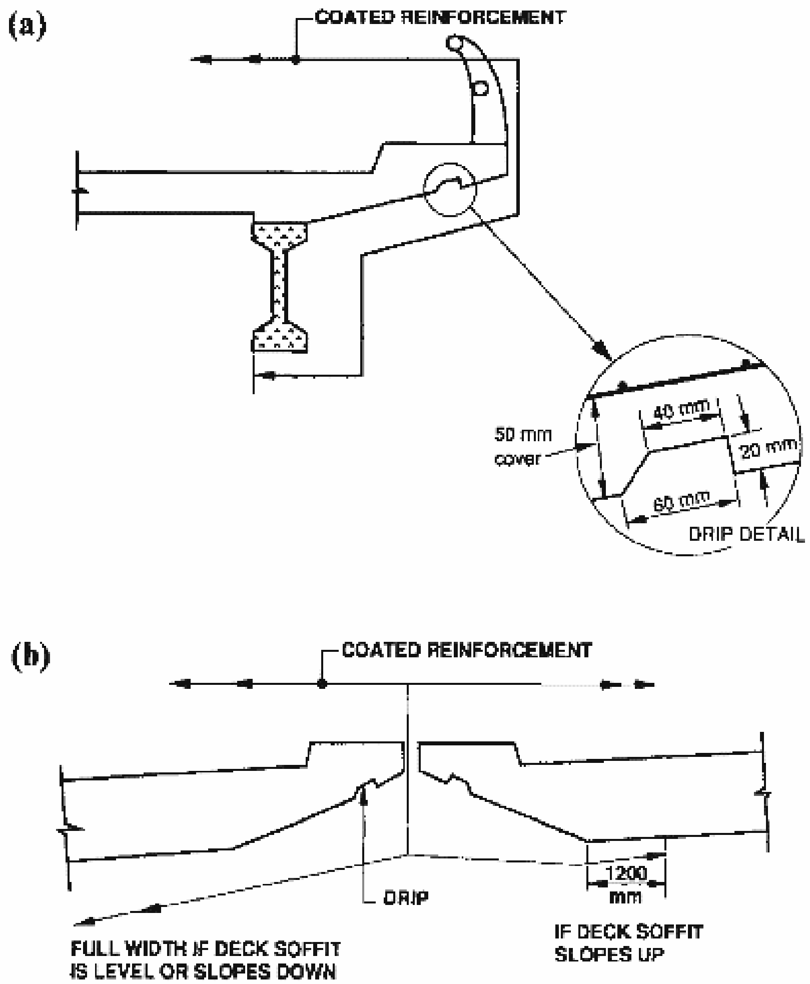

Figure 2 Examples of design details which recognize microclimate effects: (a) concrete girders and slab decks; and (b) thick slabs with or without voids.

3. Components which have a planned replacement period less than the service life of the structure, e.g. joints and bearings

Existing structures

Predicting the service life of existing concrete structures is much more difficult than for new structures. There are three essential questions which must be answered:

1. What is the existing condition of the structure?
2. What is the existing (and future) rate of deterioration?
3. What is the impact of items 1 and 2 on the service life of the structure?

Condition assessment

Techniques for surveying the condition of existing structures have improved considerably in recent years, especially for detecting the presence of corrosion activity. However, surveys which include a visual inspection, the detection of delaminations, measurement of corrosion potentials and the extraction of physical samples for laboratory analysis are both time-consuming and expensive. Consequently, there has been considerable pressure in North America to develop rapid, non-contact techniques, of which impulse radar and infrared thermography have shown the most promise. It is important to recognize that both techniques detect the presence of delaminations, which are usually the consequence of corrosion of reinforcement, and do not detect corrosion activity directly. As a result 'black corrosion' which is characterized by deep pitting and the formation of low iron oxides and hydroxides (in environments low in oxygen) without disruption of the concrete, would not be detected by radar or thermography. Thermography measures differences in the surface temperature and, since discontinuities impede heat flow, delaminations produce a measurable difference in surface temperature under certain conditions. Thermography is most effective on exposed concrete surfaces. The detection of deterioration by radar is based on reflections of a high frequency electromagnetic wave from discontinuities in the structure. Because most discontinuities are near the surface of the concrete, radar is most effective on bridge decks with a bituminous surfacing because interference between the radar echo reflected from the deck surface and that reflected from the discontinuity is reduced substantially (compared with an exposed concrete surface).

More recently, techniques which can be used to measure the rate of corrosion of embedded reinforcement have been investigated. Several candidate techniques such as those based on resistance polarization (sometimes called linear polarization), ac impedance, current transients and electrochemical noise are available, but resistance polarization appears to be the most promising for use in the field. The procedure is based on the Stern-Geary characterization of the polarization curve for corroding metals. A small potential, normally 10 or 20 mV, is applied and the relationship between potential and current is linear. The slope of the plot of potential versus current is known as the polarization resistance, R_p, and is related to the corrosion current by

$$i_{corr} = \frac{B}{R_p}$$

where

$$B = \frac{\beta_a \beta_c}{2.3(\beta_a + \beta_c)}$$

β_a and β_c are the anodic and cathodic Tafel slopes, respectively. The Tafel constants are difficult to measure in the field, but the constant B is relatively insensitive to the values of β_a and β_c and assumption of a value of B=40 is adequate for most practical purposes.

Experience with resistance polarization measurements in the laboratory and the field has resulted in the following guidelines for interpretation of the data:

- $i_{corr} < 0.20\ \mathrm{mA/ft^2}$: no corrosion damage expected
- $0.20 < i_{corr} < 1.0\ \mathrm{mA/ft^2}$: corrosion damage possible in the range of 10–15 years
- $1.0 < i_{corr} < 10\ \mathrm{mA/ft^2}$: corrosion damage expected in 2–10 years
- $1.0 < i_{corr} < 10\ \mathrm{mA/ft^2}$: corrosion damage expected in less than 2 years

Although these values are subject to change as more experience is gained in the use of the rate-of-corrosion measurements, the existence of both the measurement technique and a range of values which can be used to make engineering decisions on field structures is an indication of the progress which has been made in answering the first two questions posed with respect to corrosion in existing structures. However, answers to the third question, predicting the impact of corrosion on the service life of the structure, are much more elusive.

If the assumptions of general corrosion and a uniform corrosion rate are made, the corrosion currents measured by resistance polarization can be converted to section loss as follows: $1\mathrm{mA/ft^2}$ ($1.08\mu\mathrm{A/cm^2}$) = 0.49 mil/year (12.5μm/year) (1 amp h consumes 1.04 g of iron). In order to determine the impact of this loss of section on the structure, it is necessary to distinguish between the serviceability and ultimate limit states and to examine the stresses in the particular component which is affected.

Cracking, delamination and spalling of the concrete cover is usually a serviceability consideration which affects appearance and, for travelled surfaces, the rideability. However, other factors such as the loss of bond, an increase in the corrosion rate because reinforcement is exposed to the atmosphere and the danger associated with spalled concrete falling from vertical surfaces must also be considered.

The impact of the loss of section on the ultimate limit state of a component is a function of the location of the corrosion and the stress state in a member. For example, the impact on a lightly reinforced column may be minor, whereas the effect on a flexural member, especially if accompanied by pitting and loss of ductility, could be substantial. The subject is further complicated by the need to consider not only the impact on the component, but also on the structure as a whole in order to account for the possible redistribution of load effects.

Final remarks

While many of the factors which affect the service life of new and existing structures are understood in qualitative terms, there are considerable difficulties in expressing these relationships quantitatively. There is a need to develop practical models not only for aggressive actions, but also for the interaction of aggressive actions. For new structures, variability in workmanship, structural details and maintenance must be taken into account at the design stage. For existing structures, techniques for assessing the impact of deterioration on the individual components and on the structure as a whole are only in their infancy.

Summary of presentations and discussions

D.W.QUINION

Dr Chapman referred to the uncertainties concerning the durability of marine structures due to fatigue, the environmental conditions, undetectable defects in the materials as used and workmanship. Such structures are usually expected to perform for a defined period. He emphasized that potential problems can be overlooked in working methods which are taken for granted. Whilst the explosion in the Ronan Point flats was unforeseen, the possibility for a gas explosion existed by the presence of the gas supply, but there was no guidance available to the designer as to any necessary action for him to take. It is well known that most Middle East aggregates are unsatisfactory, but this is largely ignored in current competitive design and construction practices, which therefore overshadow design life and durability considerations.

He described the development of a series of 700 ft long ships without adequate research and the subsequent modifications to overcome the opening of fatigue cracks at welds where discontinuities occurred. This he compared with the research and development applied to the next class of 900 ft long ships, taking into account the performance of the 700 ft ships. He indicated how the non-replaceable elastomeric bearings supporting a massive arch truss for a building over a railway were given a reliable life of 200 years after accelerated corrosion tests indicated a possible 300 year life. He then described the collapse of a jack up platform, due to the development of unforeseen lateral movements when one leg jammed. Life is full of uncertainties and the problem is to think of what we have not yet thought about. We need some form of check lists to help to identify possible modes of failure.

Mr Snell remarked that in the United States, the factor of safety of 1 used on jack up platforms was inadequate. Offshore platforms are large expensive structures in an adverse environment, designed for a comparatively short but assured life. The initial shallow depth structures in the North Sea were typical of shallow Gulf of Mexico conditions and are now seen to have massive redundant members, but they are still performing well over 20 years later. Current designs deal with depths of up to 400 m and have to cater for return storm periods of 1 in 50 or 1 in 100 years and data on winds and waves are being assembled cooperatively by all interested parties. Platforms are supplying increasing numbers of wells and functions and are often required to carry additional top side loads. In many instances, the design predictions for load cycles and fatigue were inadequate or have been exceeded with consequences for the assessment of their remaining safe working lives. There is increased R&D to understand the behaviour of platforms, their individual components and welding. He described the relationships between wave heights and loads and the motion of a platform at deck level under wave conditions. He described the effect on durability of inaccuracies in member positions at welded multi-member joints and of the type of welded connection used. Fatigue is the main determinant of operating life and only with time and assessment of much data can this be predicted for long operational performance.

Mr Jordan described the facilities and design thinking applied to the £ 1.8 billion Thorp reprocessing plant, which is three times the length and twice the width of St. Paul's Cathedral. There are rigorous requirements for ensuring safety and predicted performance. He amplified the differing approaches for dry and wet stores in terms of concrete cover, treatment of reinforcement, and concrete linings to ensure durability and retain radioactivity. He described inspection and repair techniques in the context of the hazardous environment and the need to use remote handling facilities.

Mr Quinion presented some comments based on discussions with an ex-test pilot. Reliability is essential and features which would be subject to fatigue are designed out. Extensive testing is the basis of extrapolation of aircraft design features. Limits of expected durability lead to timely replacement of many components. Civil transport aircraft are bought for capital and operating cost effectiveness over an assessed working life. As with a car, the initial purchaser expects to sell in the second hand market, but maintenance and replacement schedules are rigorously applied. Military aircraft are designed for specific purposes and are not necessarily adaptable to a different role. Low level flight is very stressful by comparison with high level, and operational life is short. Military bridges are also designed for their function, which is lightweight and rapid erection, to minimize the problems of getting to site, and erection hours. They are optimized to give a predictable high performance of limited use and are withdrawn from service after receiving a designed working use. These bridges are uneconomic for use by contractors.

The discussions emphasized the need for objective observations of what is important and critical as a basis for subjective assessments. There is need to do much more monitoring of critical aspects and designing so that such monitoring can be undertaken. The importance of cracks has been emphasized for ships and is equally so for aircraft. With aircraft there is the requirement to achieve optimum use of materials and reduce weight. This approach was applied to buildings but is now seen as uneconomic for the lifetime of the structure. With general structures there is greater concern with the chemical aspects of durability and interactions with foundations. We need to identify which cracks are important, where the first significant cracks may be expected and how these are to be identified. It is impossible to inspect entire structures without a critical procedure.

Other industries appear to apply more attention in inspection, maintenance and routine replacement of components than we do in the construction industry. Should we do likewise and how do we promote a change of attitude? We should design structures with a knowledge of the problems and seek to minimize them. Certification bodies adopt the use of Design Manuals and use of regulatory inspections, but would that be inhibiting on the construction industry? We could do more in the development of procedures and good detailing to be provided in guide documents. We know that corrosion is a problem and we should avoid the use of non-inspectable details.

There seemed to be general agreement that corrosion and cracking are usually two of the main problems experienced by structures and their detection is not easy. They are not usually easy to treat once initiation is under way. Good details and working procedures will minimize their occurrence but is is important that critical members and areas be identified so that these receive priority in inspection. Means of access should be provided, wherever possible, to enable inspection and maintenance to be carried out at scheduled intervals. Guide documents to encourage good working practices are seen to be necessary.

Dr Ogle briefly traced the development of welded construction since its general adoption some 50 years ago. Liberty ships identified the problem of brittle fracture, the Kings Bridge problems of weldability of steels, box girder bridges identified problems with residual stresses and the Kieland platform the importance of fatigue. There is now a good range of tough steels to weld. Whilst good and adequate guidance appears to exist for guarding against fatigue failures, the history of performance is still short compared to the designer's expected life or to what could be hoped to be the working life of a bridge. Much attention is therefore directed to methods for predicting the remaining fatigue life of welded structures. Recommended procedures and details are being codified for steel structures and similar work is occurring for aluminium structures.

Professor Heyman described the wealth of experience available to us from centuries of performance by masonry and timber structures. Appropriate use was made in ages past of relatively small pieces of stone and of timber for longer members and this with good detailing had much to do with their longevity. He showed how the very low crushing strengths contributed to long life. It is important in old masonry structures to identify whether stability exists or they are moving. If it exists, then no interference is required, although monitoring of it should continue. Where movements are occurring, then the cause must be identified and stabilized at source.

Dr Somerville introduced the paper submitted by Professor Tassios, who was unable to attend. Lifetime expectancy must take account of degradation due to the many environmental factors that will affect any structure, and should assume that sudden accidental actions and their consequences are properly rectified immediately after such occurrences. The accidents will otherwise reduce lifetime expectancy, whilst upgrading may be used to extend it. It is important to recognize that combinations of different types of environmental deterioration with, say, seismic actions may produce effects which have not been predicted. Likewise, the accumulation of damage over time may result in a vulnerability or interaction within the structure, which may result in its behaviour characteristic being different to those predicted under seismic actions.

Professor Rostam emphasized the importance of details in determining the service lives of concrete structures. For instance, if the concrete cover to the reinforcement is reduced from 40 to 20 mm, then the time taken for carbonation to penetrate and reach the reinforcement is reduced from 100 to 15 years. The use of epoxy coated reinforcement is expected to substantially delay the onset of corrosion. Structures show no visual damage as cracking is being initiated and show it progressively as cracks are propagated. Design or service life is determined by the acceptability of accumulated damage. The analysis by Pederson of 100 000 cases of damaged concrete structures attributed 37% as being due to the design and 53% due to construction, with the remainder due to materials and other reasons. Designers frequently overlook the fact that the cost of predictable repairs is usually many times the cost of their prevention. Alternative bids for concrete structures should take into account their service lives and associated maintenance costs. It is possible to have a strategy to optimize the balance of capital and maintenance costs. The use of larger concrete covers should see the use of stainless steel mesh reinforcement within it to control surface cracking. The risk factors, associated with alternative bids, might be compared by measures of the linear or square metres of features which are recognized as being at risk.

Dr Manning took his paper as read and discussed the similarities and differences between the construction and three other industries. The nuclear industry has rigorous requirements for design, QA, inspection and maintenance, controlled by a strong regulatory agency. Service life prediction is a sensitive issue and inspection requirements may compromise durability.

In the aircraft industry, an order for new aircraft will include large numbers of replacement engines and other key components to be used at scheduled periods. All components, assemblies and finished products are tested for long periods and abnormal loadings. Failures are minimized by burn in, inspection and maintenance, scheduled replacements and redundancy of critical components. Scheduled replacements are aimed at just pre-empting the first likely failure.

In the automobile industry, the aim is to achieve long production runs and maximize profits. Warranties are linked to maintenance contracts. There is little incentive to increase service life, but a strong incentive to avoid premature failures and produce a uniform product. Some failures, which are not safety related, are tolerated whilst critical ones involve recall for correction. Testing is carried out for long periods with abnormal loadings under adverse environmental conditions.

Building structures are generally impossible to inspect properly but have similarities with nuclear, aircraft and automobile in containing a skeleton of critical components and the use of redundancy or multiple load paths. The other industries have:

- Expressed requirements for inspection and maintenance
- The use of warranties
- Strong regulatory agencies with jurisdiction for in-service performance
- Scheduled replacement of parts
- Identical (often) high cost products
- Economic incentive for uniformity of product
- Feasibility to test components and finished products under abnormal loading conditions
- Finite service life
- Vested interest by manufacturer
- Alteration in usage

The discussions were mainly directed at the achievement of better quality. It was suggested that only a small increase in the cost of reinforced concrete can produce considerable improvements in its service life. The selection of the contractor should be based on proven good performance and use of quality management techniques, so that the required quality, and hence durability, will be forthcoming. The first 10-15 years are critical to the durability of concrete structures and the problems are then apparent. It was suggested that 60% of the problems occur or appear in the first three years. A study of the progressive increases in the Code requirements for concrete covers over the last three decades does not show signs that this has had any notable improvement on durability. So how do we achieve longer lives from our structures? It was suggested that we are continually moving towards the limits of our knowledge and experience, but fail to give due recognition to the implications of these moves.

The aim appears to be the avoidance of premature death and, to achieve this, we are required to provide robust structures and to minimize, or improve, the effectiveness of joints and other features recognized as giving problems. In response to the question as to why the construction industry appears to have so many difficulties, several answers were offered. Our clients do not recognize the problems with service life, or achievement of durability, and hence the need to do anything better or spend more money to do so. The industry is mainly fragmented, with large numbers of clients, designers, contractors and suppliers with large and small businesses. It is divide and compete, rather than unite and improve. Factory methods were introduced post-war to improve productivity and quality in concrete work and were notably unsuccessful. It now seems that there is a new move to encourage the use of high quality concrete cladding units.

The need to introduce new topics into the university and college syllabuses, such as management topics, is difficult unless traditional subjects are dropped or the lengths of courses are extended. The introduction of Continuing Professional Development (CPD) provides the opportunity to break out from this restraint, so that education is planned to continue into employment and the present core teaching is extended into the years following graduation. There are great opportunities now available as CPD is generally adopted as a feature of professional life.

Good quality construction must come from the use of contractors who can demonstrate their ability to perform to the standard required for the types of construction involved. Such contractors will have established quality management systems and evidence of their successful use.

C The present state-of-the-art

Buildings: general

J.RODIN

The term design life of buildings implies that this is a conscious part of design. In my experience, this is rarely the case. Buildings, in general, last as long as their owners want them to last and are prepared to invest in them. However, it is clear that decisions taken by the client or the designer can have a profound impact on the lifespan of a building and the cost of keeping it in beneficial use.

Of great concern has been the unexpected short life of some quite significant buildings resulting from:

- Increasing land values making redevelopment attractive; going higher where permitted, or deeper to increase prime above-ground space
- Changing building function within a building too tightly designed to accommodate present day requirements
- Use of materials or construction methods that have not withstood the test of time
- Inappropriate estate and building design unsuited to social need.
- Skimping on space (particularly for services) design loading and materials

In some of these cases, the significant factors were either unforeseen or outside the designer's control. In most cases, however, it can be said that first cost was, and in many cases still is, a primary consideration.

Life-cycle costing, whereby the total cost of providing, running and maintaining the building is taken into account, is beginning to feature more strongly as a consideration particularly for those building elements which have a known shorter life or require regular maintenance. Nonetheless, except for very rare cases, the building designer does not embark upon his design of the building as a whole with any specific life period in mind. The tacit assumption is that the building will last indefinitely providing it is properly maintained and known short life elements are replaced as required.

In an unacceptably large number of cases, this assumption of indefinite building life has not been met in practice. Some of the reasons for this are listed above. The designer needs to address those that are within his control, in a more serious and conscious way than in the past. The design objective should be to eliminate potential causes of premature failure and to enhance those qualities which will extend building life, i.e. to achieve robustness in both structure and building use. Such robustness can often be achieved at minimal, if any, extra cost providing it is consistently part of both concept and detail design. For example, a higher floor loading in office buildings is now generally accepted even by the most commercial developers as a wise investment permitting more flexible use of the building, the extra cost being insignificant in relation to the total asset value.

For too long, emphasis on slender sections, for aesthetic reasons and to minimize material use, dominated design paricularly of the structure. Coupled with inadequate detailing and workmanship, this led to premature failure of key elements and sometimes buildings as a whole. In the light of this experience and in today's economy, the emphasis now is on optimization of the construction process rather than the material content. In this context, the provision of an extra 10 or 15 mm of concrete cover would cost next to nothing but might add significantly to the potential life of the structure.

Simple considerations such as these would lead to robust structures and buildings much more likely to respond satisfactorily to the unexpected and stand the test of time. I would define a robust building structure as one which:

1. Is systematically designed to cope with known hazards considering both risk and consequence
2. Is not unduly sensitive to:
   - marginal departures from the design assumptions
   - local defects or movement
   - environmental change
3. Is readily buildable and is not dependent upon perfect workmanship and compliance with the specification
4. Identifies and provides good access for all items requiring maintenance or inspection
5. Incorporates early warning signs of serious defect
6. Does not deflect or vibrate to an extent that alarms the occupants or disturbs their function
7. Permits remodelling to suit changing use

In this same context of robustness, ample plant room space, easy access for plant maintenance and replacement and extra storey height and ceiling space can add enormously to the building's potential to cope with planned or unexpected future use, and hence extend its useful life.

These remarks apply primarily to the basic planning of the building and its structure where it is my belief that the primary route to long life is through robustness as described above, rather than through life-cycle costing techniques. Exceptions to this would be planned short or extra long life buildings, or system building where there is great repetition over a large family of similar buildings.

However, life-cycle costing does have an increasing part to play in the design and selection of secondary components particularly those which are likely to be replaced several times during the life of the building as a whole. It is potentially of greatest benefit to those who own or lease large numbers of buildings and have reliable data on actual performance.

Maintenance and refurbishment of buildings absorb nearly half of the £34 billion UK construction market, yet there is very little coordinated data to confirm or otherwise whether this money is well spent. This is an area where similar problems are experienced by a very large but disparate collection of building owners and users. Given reliable data on the performance of original and replacement materials or components, life-cycle techniques could be put to good use to condition future design and optimize investment in maintenance or refurbishment. The LINK Maintenance and Refurbishment programme, a joint venture between the government and private sector, is focussing research funds into this area. Current priority topics are:

- Whole life costing: optimizing initial and maintenance expenditure based upon reliable field experience
- Updating and maintenance of building services
- Assessment and repair of concrete structures and elements
- Assessment, maintenance and repair of deteriorating materials or elements

Research projects already approved include sealant performance, flat roofing and recladding support systems; all items of widespread concern having common features across a large range of buildings. Another project recently approved will provide a framework for the development of a reliable data base on performance and costs in use. In due course, such work will provide more information from which life-cycle appraisals can be made for key building elements.

There will still remain uncertainties particularly regarding future inflation and discount rates, and there may be overriding political, economic or tax considerations which decide the issue. We will also be at the mercy of unforeseeable future events or factors which cannot be measured in financial terms. For example the investment in a quality long life cladding may, in the event, be proved wasteful if a ‘new look’ is required to meet changing fashion or market demand. Nonetheless life-cycle techniques do and will have their place. They will never replace common sense and sound judgement, but they will certainly be a valuable aid to such judgement particularly in choosing between options.

Design life and populations of building structures

J.B.MENZIES

Introduction

The construction of a large stock of modern building structures over the past few decades has been achieved by taking advantage of increasing standardization and much technical innovation and development. Particular structural designs incorporating recent developments have come rapidly into widespread use and large populations of similar structures have been built. They have generally proved successful; the uncertainties in structural design, construction and use have usually been offset sufficiently. There have been, however, a few collapses of buildings in service and a larger number of cases of buildings exhibiting structural deterioration before the expected life has been reached.

The quality of structural materials, components and connections and their ability to retain their structural performance in the environmental conditions of the completed building have been factors contributing to structural deterioration in some large populations of similar buildings [1–4]. The early deterioration of these modern structures was associated with the use of new or improved materials or of traditional materials in a different form or in a more aggressive environment than was recognized. In consequence, widespread inspection and remedial works have been required.

Innovation in construction is normally based on research and development but, however careful and thorough the necessarily short term development work, long term performance in service may not live up to expectations. A small risk of unsatisfactory performance in service will remain. Since standardization can lead quickly to widespread use it is desirable for techniques to be applied which limit the consequences for deterioration or other failure of structures in service. What techniques are or might be used? Is design life a helpful concept to apply to this situation?

The characteristics of populations of similar structures

Modern methods of building construction have relied more and more on factory production of materials and structural components. Such production has the capacity to produce large quantities of the same material or component. Their subsequent use in construction may result in a limited variety of structures.

Populations of buildings with strong similarities between individual structures are also created by common design and construction. More generally, a whole range of populations exists where individual populations are defined by some particular common feature or combination of features. Common features may range from the materials through to the complete structure and may also include aspects of design and construction, history and purpose. For example the following populations and subpopulations of building structures may be defined:

- Those built using a material to BS X
- Those designed to BS Code of Practice Y
- Those built using standard components Z
- Those built with structural system AB
- Those built by a particular contractor
- Those structures designed as school buildings
- Those structures designed to BS Code of Practice Y between 1960 and 1970
- Those structures constructed in England up to 1989

Consequences

The potential consequences of structural deterioration or other failure within a population will depend very much on the nature of the population which is identified. This may best be illustrated by examples,

Trussed-rafter roof collapse

In July 1976 the roof and parts of the walls of the sports hall at Rock Ferry Comprehensive School in Birkenhead collapsed suddenly [5]. The sports hall was an isolated structure with walls of cavity concrete blockwork construction and a roof structure of one-way spanning timber trussed rafters of pitch 17° and span 18.2 m.

The collapse was found to be caused by lateral instability of the trussed-rafter roof because transverse diagonal bracing was not provided. The unstable condition of the roof led to a progressive transfer of lateral load from the roof structure to the gable walls. When this load reached the lateral restraining capacity of the gable walls, one of them burst outwards and the whole roof collapsed. The trussed rafters used in the roof were found to be adequate to carry the vertical design loads provided that they were restrained in position. The design included provision for suitable restraint to the compression members in the trusses, but it did not include suitable diagonal bracing of the complete roof structure to prevent lateral movements of the trusses *en bloc*.

Once the cause of collapse had been established, the implications for other buildings had to be considered. The roof structure had an unusually long span for trussed-rafter construction which is very commonly used in housing. Wide publicity was given to the failure in order to alert owners of any similar structures to the potential defect and, although the shorter span trussed-rafter construction used in housing was much less prone to this form of instability and existing trussed-rafter house roofs were not considered to require remedial work, recommendations for the lateral bracing of trussed-rafter roofs generally were made more explicit by the industry and in codes of practice.

Pre-stressed concrete structures

A 12-year-old post-tensioned roof beam spanning 60 ft in a non-standard Intergrid structure collapsed in 1974 [2]. The collapse was found to be due to corrosion of the prestressing tendons within the ducts. High concentrations of chloride ions existed both in the ducts and in the main body of the precast concrete and the duct grouting was incomplete. Water had penetrated the duct coupling via the joint between the precast units and had enabled chloride ions from the main body of the concrete to migrate into the cavity within the duct. The tendons had then been exposed to chloride bearing water and eventually had become critically weakened by corrosion. No adverse reports of the condition of structurally similar buildings were received from owners who were contacted. It was concluded at the time that the collapse arose from an isolated instance of bad workmanship which, combined with other faults, had led to the collapse, i.e. a population of similar structures containing the same shortcoming was not identified.

This conclusion came under review in 1976 when deterioration of pretensioned columns in an Intergrid building was discovered [2]. Inspection of the population of these buildings revealed further deteriorated components associated with the presence of unexpectedly high chloride contents in the concrete and also in some cases with carbonation. Specific areas of susceptibility to deterioration in particular variants of the system were identified.

In attempting to establish a prognosis for the future condition of these structures it was not found possible to identify a level of chloride content in the concrete below which tendon corrosion will not occur in the normally expected building life of, say, 50 years. Corrosion had to be viewed as a risk in the future. Consequently, periodical inspection of these structures was required as a means of monitoring the risk.

The investigations of Intergrid buildings aimed to provide technical advice for the appraisal of these structures. This immediate population was clearly defined. However this experience raised the question of whether there were other populations of structures built during the same period which may be susceptible to deterioration associated with high chloride contents in concrete, carbonation or inadequately protected prestressing tendons. Related populations exist and publicity was used to alert structural engineers to the potential implications.

Limiting the consequences

For a population defined by a common structural component or system it might be argued that more caution should be exercised in design of components than for one-off construction in order to limit the potential consequences of an isolated failure. Generally the same design is used for both one-off and long runs of construction on the assumption that the greater development effort and the better quality control and inspection in 'factory' production will ensure a lower risk of failure. There appears to be no explicit consideration in design generally of limitation of potential consequences.

Variety in structures

One approach to limiting consequences in populations of buildings would be to encourage variety in the structures so that a failure or defect discovered in one building does not necessarily indicate that other buildings incorporating similar materials, components or design features are in jeopardy.

This approach would be difficult to achieve in modern building structures where uniformity in design and construction is encouraged by economic and technical requirements and recommendations through, for example, Codes of Practice. In particular the standardization of construction materials leads to their widespread use.

It is also doubtful whether a heterogeneous building stock would be as safe overall in terms of protection to life as a large population of uniform buildings where a failure would appear first in a tiny fraction of the population and so give warning of a safety hazard. However, whilst the warning will protect the lives of people using the building population, it may lead to the widespread economic and social consequences already referred to because the whole population is implicated by a single failure.

Signs of failure/deterioration

An alternative and more promising approach for the future which would minimize economic and social consequences, is to make the structural design in a uniform population of buildings such that structural failure or deterioration in an individual building will be manifest first on a local scale and thus inhibit the building's use. Essentially the user/maintenance inspector is alerted to the need for repair by the structure itself providing obvious signs of deterioration/distress, i.e. the structure feeds back information on its condition. In these circumstances the failure becomes a matter of maintenance by repair and replacement and there will be no requirement for drastic action in relation to other buildings in the population. However, whilst this may be an attractive concept it is not clear that there is adequate knowledge for it to be put into practice.

Some requirements for successful use of the concept can be identified. The starting point is that the safety and maintenance of both individual structures and populations of similar structures are beneficially affected if they are designed so that local distress/deterioration is obvious. Such structures automatically provide visual signs which will be recognized by occupants or maintenance inspection and lead to action to ensure repair. Since engineering inspections are expensive, preferable designs would be ones where structural distress is obvious to the occupants (the non-technical user) thus providing automatic performance monitoring by building occupiers.

Little attention is given in building design to ensuring that structural components can be visually inspected, repaired, or replaced with ease during the building life. Column, beam, wall, floor and roof structures are frequently clad to cover services and to provide decorative finishes. Since developments in structural materials and construction processes continue to take place and experience of their long-term performance in service is not available, improvements in design for ease of structural inspections and maintenance (together with similar improvements relating to the building envelope and services) may be desirable for the future. The major advantages would be that safer buildings would result and life-cycle costs would be reduced.

Visual inspection will only reveal local failure or deterioration in some types of structure. For example, provided that the surface of members can be seen, corrosion of structural steel is obvious since it leads to rust staining and disruption of protective paints. Likewise, corrosion of reinforcement in reinforced concrete usually causes obvious cracking and disruption at the concrete surfaces well before the structure becomes unsafe. Structural safety in respect of these types of deterioration can therefore be ensured by designing so that inspecion is possible.

Deterioration in some other types of structure is not visible. For example, corrosion of the highly stressed pre-stressing tendons in post-tensioned concrete beams may not produce any external visual signs before sudden failure occurs. Structures with a strain-stiffening load deformation characteristic or components made of brittle materials give little or no warning in the form of deformations when overload and failure are imminent. Maintenance of safety in situations of this type is more difficult although it can be achieved by the traditional approach, i.e. by making sure as far as practicable that deterioration will not occur and designing well within eventual long-term capacities. The load-shedding characteristics of the whole structure become more important in these situations.

Timescales

The signs (feedback of information) of deterioration/defect in a building structure will be more effective in limiting the consequences in similar structures in the population the earlier they appear in the building's life. Where signs occur during the construction of the first structure in a population, modifications can be made to avoid the problem in subsequent structures. This rapid feedback reduces the consequences to a minimum. At the other extreme, where the signs are not manifest until the whole population has been built and brought into service, the consequences will be greatest. In general the feedback of information on structural performance into specification and building control processes will be effective if it occurs in a period which is shorter than the duration of construction. The shorter the timescale of feedback the less are likely to be the consequences.

Design life

The application of the concept of design life to populations of building structures would need to take into account the behaviour of the materials and structure as the potentials for deterioration to begin to operate. If deterioration mechanisms are known to produce clear signs before structural safety and serviceability are significantly impaired, then a design life might be set based on direct extrapolation of experience. But a much more cautious view would need to be taken where deterioration may not be manifest. At the same time the susceptibility of deterioration to economic repair should have an influence on the establishment of a design life. These considerations might lead to a different conclusion for a population of structures than for an individual structure. A further complication arises, however, from the rapid and continuing changes in building technology and organization in the construction industry. These changes introduce uncertainties into predictions of future performance additional to those already present relating to environmental conditions and use of populations. The definition of a design life for a population of structures is therefore a complex task requiring further analysis.

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Bridges

B.RICHMOND

Introduction

Design life as a concept for bridges needs to be considered within the perspective of the complete range of the interacting factors that govern the design of a bridge. The range encompasses environmental matters, effects on third parties, geotechnical, functional including operational aspects and, of course, maintenance and durability. The 'Review of the SERC civil engineering research programme 1983–1988' in its foreword referred to the current concept of 'full life engineering performance' in order to encapsulate a wider theme of a similar nature. Therefore rather than seek to understand the term design life in a limited sense within the complete picture, it should be a term which reflects all the critical aspects of that picture. The structural performance aspect of design life, for example, then becomes one of a number of essential elements that make up the whole.

This should not mean that the importance of structural performance is diminished, quite the contrary, since bridge structures are particularly demanding areas of structural design. They are usually one-off designs in common with most civil engineering structures which exist only as untested prototypes for which the construction stage is often one of the critical loading cases. The completed design is also called upon to act in a demanding manner for all of its life in major bridges. This may be compared with the relatively low demands put upon most buildings. But, as part of the normal static environment of buildings and roads, a bridge is expected to have the very low risk to life and limb common to that environment. The continuing development of bridges using new materials and structural forms is, however, essential but consequently very demanding if undertaken with due regard to the very high safety levels required.

Thus the complexity of the design of bridges should be emphasized as opposed to attempting to distil from the many parameters involved a simplifying unification. Design life should, therefore, be seen as an addition of the dimension of time which thereby makes a rational approach to the methodology of design more accessible.

A brief review of the wider aspects of design by means of recent examples may help to clarify the above comments.

A19 Tees Viaduct

The Tees Viaduct is one of a number of bridges built in the period from 1965 to 1975 where no attempt seems to have been made to assess design life, however it might be defined. After a relatively short period of use Maunsell was commissioned by the Department of Transport in 1980 to examine and report on a number of aspects of the multiple steel plate girder bridge, including strength for increased loading, concrete deterioration, bearings and articulation and maintenance access. All of these areas show the importance of considering the design life in a very wide sense including of course the difficulties inherent in forecasting future requirements, e.g. loading. One aspect stands out, however, as being particularly noteworthy—maintenance.

The steelwork of the girders is stiffened and braced, producing maintenance problems which are accentuated by the access difficulties of a high level major road over both water and railways. Increasing labour costs and safety requirements further compound the problem. The solution to this complex and difficult situation was produced by a combination of a major technical innovation which virtually removed the problem and financial studies of the cost maintenance for the life of the bridge in various circumstances. The development of a glass fibre reinforced plastic membrane in the form of a cellular panel system suspended from the girders as a load-bearing floor was the technical innovation. The development of this lightweight floor required new design and production techniques to ensure that high structural performance could be obtained at relatively low cost. The floor provided two advantages:

- Enclosure, i.e. air carrying corrosive particles was not given free access to the inside of the girders
- Access, the load bearing characteristics of the floor enable access for future inspection and maintenance to be undertaken with ease

It is interesting to note that as GRP is a new material in the context of bridges particularly where British Rail's interests are affected, it was necessary for the scheme to be subjected to much more basic evaluations than if conventional materials had been used. For example, the fire resistance of the material, which is 60% by volume glass and therefore cannot burn, had to be demonstrated by special tests. Would aluminium be evaluated fully as a fire risk if used in certain bridge applications where its less than ideal fire characteristics might constitute a hazard? Steel, of course, is also suspectible to failure when affected by heat and is consequently protected in buildings. Even concrete can be subjected to major damage by fire as has occurred on occasions on motorway structures.

The second Severn Crossing

The second Severn Crossing presented an opportunity to approach the design in the complete sense from the earliest stages of a project, in contrast to the Tees Viaduct.

An appointment in 1984 to undertaken a study into the alternative crossings, considering both the site and form of bridge or tunnel as parameters required all aspects of the project to be evaluated.

By 1984 it had of course become apparent to all, user and owner alike, that there were major problems in many of the bridges built in the 1960s and 1970s which were significant sources of cost both directly due to increased maintenance and indirectly through delays to users. There was, therefore, a favourable climate of opinion towards an approach which puts increased emphasis on the function of the crossing and recognized the user explicitly in assessing the requirements of the second Severn Crossing. This resulted in a number of studies and investigations during the 1984–1986 phase which were pursued in greater detail in the following phase resulting in the preparation of tender documents for a 5 km bridge crossing at the English Stones, which were issued in April 1989.

It was of course particularly significant that the existing crossing had affected both the Department of Transport and users through extensive maintenance and strengthening and also through restrictions on traffic due to the effects of wind on wind sensitive vehicles. It is perhaps not surprising therefore that these areas were considered within the investigations but the large number of other areas which were also the subject of major studies and surveys, all with a significant bearing on design, illustrates the width of scope necessary within the concept of design life. The principal studies and surveys are listed below:

- Environmental studies
- Bathymetric surveys
- Geotechnical surveys
- Geophysical surveys
- Topographical survey
- Hydraulic: tidal model of estuary
- Hydraulic: numerical physical model of estuary including bridge piers
- Ship simulator studies of Shoots channel investigation
- Aerodynamic studies of main bridge including windshielding
- Full scale testing of a 'Luton' van in cross-wind with windshielding

- Ice and snow testing of windshielded superstructure
- Maintenance/durability of bridge superstructure including cable members
- Load factors for very long bridge structures including effects of indeterminacy, materials, non-bonded cables and span length
- Foundations and British Rail tunnel

It would not be putting it too strongly to say that the decision by the Department of Transport to include wide terms of reference in our brief made it possible to undertake the complete range of studies listed and also made it possible to arrive at the most appropriate solution, i.e. a bridge crossing at the English Stones. The importance of wind on vehicles and the potential effects of the bridge piers on navigation are two examples of important areas which could, if not resolved effectively and economically, have changed the type and location of the crossing. All the areas listed and many other considerations were also vital components of the process of determining the qualities of the crossing. Further details on those two will, perhaps, demonstrate the depth of investigation necessary in the wider concept of design life.

Windshielding of vehicles

The effect of winds on vehicles crossing bridges is an area where there is relatively little official data and yet over the last six years it has become increasingly clear from those who are dealing directly with a number of major bridges that it is an important phenomenon. It has, however, not been generally seen as being a design matter and has been accepted as an 'act of God'. The problems encountered on the existing bridge had, however, required the introduction of restrictions on traffic which were not favourably perceived by the users or the police who had to operate them.

The process of producing a means for virtually removing restrictions on traffic required a number of investigations and innovations. The financial advantages of reducing restrictions had to be evaluated by assessing the probabilities of critical wind speeds and the associated traffic costs over a long period. The technical problems, that required major innovations, were of two kinds:

- To devise and test a system to provide windshielding for wind-sensitive vehicles
- To ensure that a satisfactory system of windshielding was compatible with structurally and economically acceptable systems for the bridge superstructures.

Initial thoughts and studies suggested that a configuration of horizontal members forming an open ranch-type fence with a porosity of 50% and a height of 3 m would give effective shielding. There was, however, a major problem in proving its effectiveness as the effect of wind on a vehicle is a function of both vehicle velocity and vehicle dynamics as well as the aerodynamics of the windshield barrier/vehicle combination. Wind tunnel testing was not capable of simulating more than one of those three parameters at a time and although it was used in the initial studies, a more complete simulation was essential. The use of an actual vehicle travelling at appropriate speeds was recognized as being necessary for obtaining valid results of incorporating all the above parameters. It was then found that the installation at the MIRA test track, set up for assessing vehicle response to cross-winds, could provide the necessary wind effects.

A Rolls Royce Avon jet engine provides an air-flow which is distributed through a series of outlets to simulate the air-flow associated with a gust of wind. A very valuable programme of tests on a 'Luton' van showed that the windshielding barrier was effective in providing shelter to wind-sensitive vehicles.

The aerodynamic stability of the 450 m main spans of the cable-stayed bridge over the Shoots navigation channel would, of course, be adversely affected by the 3 m high windshields. Wind tunnel testing determined the combination of structural dynamics and aerodynamics necessary for safety which showed that the economical twin-plate girder approach would not satisfy the aerodynamic requirements. Another innovation overcame that difficulty. An aerodynamic fairing was devised which converted the twin-plate girder system to a satisfactory shape. A feasibility study showed that the A 19 cellular GRP envelope would produce a structurally sound and durable solution.

Bridge piers and navigation

Finally a brief mention of those who will continue to navigate a river after a bridge is built. It is a most important issue particularly in this instance where tidal flows and variations in depth produce critical conditions for the pilots of vessels up to 6500 tons. It was necessary to determine the size and position of piers from the results of a computer model of the estuary which showed a feasible crossing alignment. More complete results were then determined from a physical true to scale model of part of the estuary. This was used to produce a more detailed assessment of the effects of the new bridge on navigation. The importance of this issue and the need for the pilots to be able to make their own assessment resulted in a further assessment. The data base of a ship simulator was set up using the hydraulic data from the model together with the day and night images of the estuary including navigation lights, etc. Further data bases used included radar and depth for shallow water behaviour of the ship.

The simulator is simpler in its imagery than aircraft flight deck simulators but is essentially the same with controls, instruments and a visual display. The computer provides a real time response in which the complex hydrodynamic behaviour of the ship in varying currents is simulated. It was possible for pilots to take a ship through the Shoots channel in conditions before and after the bridge piers are constructed. The effectiveness of the simulator can be gauged by the pilots' reaction to the 'before' scenario. They found the experience very similar to actually navigating a ship in the Shoots channel under the same state of tide.

The simulator can provide guidance on the effects to be expected from changes in varying circumstances of wind, tide and accuracy of approach. It is also available for assessing future changed requirements and in an area quite separate from the functions of the bridge it may be seen as a most important tool, i.e. it contributes to the design life of the bridge in all its phases including construction.

Management strategy and maintenance

P.H.DAWE

Introduction

This paper gives a broad outline of the way in which the Department of Transport in the United Kingdom manages its stock of bridges and other highway structures on motorways and other all-purpose trunk roads. Whilst the experience of the Department will not be typical of the experiences of other authorities responsible for the care and maintenance of other types of structure, there will be certain common points and practices. In particular there will be similarities with other long life structures where it is recognized that the structures will require regular inspection and adequate maintenance if they are to fulfil their intended function for as long as necesssary. Like all authorities, and especially those in the public domain, the Department has only limited resources and there is a need for a management strategy to ensure that the bridge population is maintained at the required standard as efficiently and effectively as possible.

DTp bridge stock

The transport departments of England, Scotland and Wales are responsible for 9500 miles of motorways and other trunk roads. Although these represent only about 4% of the total road network in United Kingdom they carry 30% of all traffic and 60% of all heavy goods traffic. They are therefore heavily used routes and vital components of the country's road network. The Department of Transport in England owns about 8500 highway bridges and about 3000 other structures including retaining walls, sign/signal gantries and tunnels. About 75% of the bridges are concrete, 15% steel (mainly steel/concrete composite) and 10% masonry arches. They range from the Severn Bridge with a main span of 988 m, down to small culverts with spans of 3 m. Although a large number of the bridges are over a hundred years old, the oldest being originally built in 1185, the majority are modern bridges built within the last 20–30 years. The bridge stock is therefore not a homogeneous population and its management cannot be compared with the management of, say, a fleet of lorries or a stock of properties of similar age and composition.

National Structures Database

The key to any successful management operation is the availability of adequate and relevant information. The heart of the Department's management strategy for its structures is the computerized National Structures Database. This holds details of all the Department's structures, their location, construction and ancillary components. In addition, the data base holds information from inspection reports and details of maintenance expenditure. The data base operating system includes the facility for report writing, and simple report writing routines which allow the user to easily prepare lists of structures with common features and to analyse the information contained about the state of the structures as a whole, have been developed.

Inspection procedures

One of the main sources of information about the state of the Department's bridge stock is from the operation of the bridge inspection programme. This requires that a Principal Inspection, which at present consists of a close visual inspection, should be carried out on each structure every six years. A less intensive General Inspection is to be carried out every two years. The results of these inspections are recorded on standard forms and can be input into the data base. If a particular problem is identified, a Special Inspection may be called for. This will usually involve special testing, and even a detailed analysis of the structure if its load bearing capacity is in doubt.

Maintenance programme

The findings of the inspection programme are used to develop the maintenance programme for the following and subsequent years. The costs of the necessary remedial work are estimated and the work given a priority rating according to the following criteria.

- Category 1: Work necessary for the safety of the public or integrity and load carrying capacity of the structure.
- Category 2: Expenditure committed, e.g. work continuing from previous year.
- Category 3: Hybrid scheme where work will be part of a contract including road works, or where advantage can be taken of road closures for other reasons.
- Category 4: Work which would cost more than 10% more in real terms if delayed until the following year.
- Category 5: Work which would cost less than 10% more in real terms if delayed until the following year.

Although the basic criteria hint at the need to take economic considerations into account, they rely upon subjective decisions for their implementation. Moreover, since each agent authority, usually a county council, is responsible for drawing up its own prioritized list, there is room for considerable variation country-wide in the way in which the priorities are ascribed.

The programmes from each agent are used as the basis for the development of regional and national programmes, and the preparation of the annual bid for bridge maintenance funds, which is one of the many items considered by the Treasury under the Public Expenditure Survey. The actual amount of money eventually allocated by Treasury will determine the size of the bridge maintenance programme and the extent to which work on the various priorities can be accommodated. It should be noted that, although there is an element of a 'stitch in time' in the categorization of priorities, safety is paramount and there is also a recognition of the need to reduce delays to a minimum. Details of the bids and out-turns can be stored in the structures data base.

15 year rehabilitation programme

Over the last 5–10 years it has become apparent that increasing amounts of money were having to be spent on the maintenance of structures of fairly recent construction, particularly concrete bridges. Much of the increasing deterioration can be attributed to the widespread use of de-icing salt causing chloride damage. Other causes include carbonation, alkali-silica reaction, sulphate attack and frost damage. Deterioration can also be due to poor design and detailing, poor materials, poor workmanship or inadequate maintenance. The full extent of the problems affecting the Department's concrete bridges was revealed by the study undertaken by Maunsells. As a result of inspecting and testing some 200 randomly selected but representative concrete structures, they were able to estimate the level of maintenance funding likely to be needed to tackle the problems over the next 10–15 years.

Following the publication of a new Bridge Assessment Code in 1984, the Department decided to embark upon a programme of assessment and strengthening of its older short span bridges. There is some urgency to complete this work so that these bridges will not only be adequate for current traffic, but also be ready for the heavier 40 tonne lorries when they are introduced in 1999. A subsequent programme will deal with long span bridges, with spans greater than 50 m, where traffic loading has increased due to the increased numbers of heavy goods vehicles on the roads. It may also be necessary to look at some of the more modern short/medium span bridges which could be deficient in shear resistance.

In addition to the activities mentioned above, there is also a need to rectify deficiencies in certain structures where current design standards and specifications are not being met, mainly those involving safety and durability. The topics include such things as waterproofing unprotected bridge decks, dealing with some problems with pre-stressed concrete bridge decks, replacement of sub-standard vehicle parapets, and strengthening of piers and columns to resist higher impact forces.

Because of the need to tackle these various bridge problems in a planned and rational manner, the Department has developed a 15 year strategy for dealing with them. The length of the programme was determined by the need to tackle some of the safety related problems within a reasonable time, whilst avoiding too much disruption of the road network at any one time, and also by the need to even out the demand in resources. The present programme may be regarded as an increased effort in the area of steady state maintenance, combined with the work needed to bring some structures up to current standards. Even when the programme is completed, there will still be a need to carry on with the regular maintenance cycle for the structures which, by then, will have deteriorated.

Significance of bridge design life

The prescribed design life for a structure has limited significance as far as the actual design of the structure is concerned, except in fatigue analysis and, to some extent, in the provision for durability. It costs very little extra to design and construct a structure for 120 years rather than 30 years. In fact it is difficult to see how a structure such as a bridge could be designed to last for a set number of years. Does that mean that it is expected to remain in its as-built condition for that number of years, or does it mean that its condition gradually deteriorates so that at the end of its prescribed life there comes a particular point when it ceases to be usable? The main reasons why bridges are replaced are because of improvements to the road network, e.g. widening, realignment, bypassing, etc., rather than because of the state of the structures themselves.

The main significance for a prescribed design life is its implication for any future maintenance strategy. A short life could indicate that the structure will be expected to receive only a minimum amount of maintenance as and when necessary, whereas long life structures will be subjected to a regular cycle of inspection and maintenance. In essence, a long life structure does not have a well defined life but is expected to remain functional for as long as it is required, and for as long as it is economic to carry out the necessary remedial works. Unforeseen causes of deterioration, such as chloride damage, can mean that the assumed design life is not achieved. Because of the amount of traffic which the Department's bridges carry, their importance in the road network, and the disruption which any work causes, it is difficult to conceive that short life replaceable structures would be acceptable. Hence the Department's choice of long life backed up by regular inspection and maintenance.

Managing a stock of bridges

Managing a stock of publicly used structures such as highway bridges has its own particular problems. Because they are long life structures, it is difficult to foresee, at the time of design, the intensity of loading that they will be required to carry throughout their entire life. Design standards may change as the results of new research become available. Public expectations in terms of safety and reliability may change. Attempts to improve the movement of traffic in certain weathers may lead, as in the case of de-icing salts, to unforeseen deleterious effects on the structures themselves. Thus, in addition to the normal on-going maintenance required to keep any structure exposed to the elements in good condition, there may be the need for additional discrete programmes of work to bring the stock up to current standards.

As already mentioned, the Department's problems are exacerbated by the need to minimize traffic disruption on heavily used routes while any remedial work is carried out. Like all public authorities, the Department has only limited resources at its disposal for this work. There is therefore a great need to ensure that the money spent on bridge maintenance is spent in the most cost effective manner, taking account of traffic delay costs; and ensuring that repairs are carried out at the opportune time without compromising public safety.

Bridge management systems

The Department, like other bridge owners throughout the world, has recognized the need for the development of a comprehensive computerized Bridge Management system for the management of its stock of bridges. This will be based upon the existing National Structures Database, but will employ more rational procedures for the determination of priorities and the allocation of funds. These will be based upon the economic evaluation of various repair strategies and the determination of cost benefits. The advantages of a comprehensive system which can be accessed by all the various maintaining agents, is that it should lead to a much more consistent approach to the classification of deteriorated structures and the timing and type of maintenance activities carried out. An additional advantage of such a system is that it will provide the information necessary to justify the bids for bridge maintenance funding, in competition with the other demands upon the public purse.

It is realized that the success of such a system will depend to a large extent upon the quality of information fed into it. In particular it will be necessary to have reliable information about rates of deterioration, and reliable estimates for the costs of repair. It will be necessary to evolve the various maintenance strategies which are appropriate for the various types of structure or element.

The system will also depend upon the consistency and adequacy of the initial inspection reports.

A bridge management system should not be regarded as a black box which will automatically produce an optimized maintenance programme from a limited set of inspection information. At best, it can only be an aid in ensuring that maintenance work is carried out in the most cost effective manner. There will be various restraints such as the need to ensure that no undue restriction is placed on the road network at any one time. There will have to be some form of pre-dedication of a part of the budget to deal with cyclically recurring schemes, such as painting. Activities for which there is some political pressure, such as the assessment and strengthening of the older bridges, must also be accommodated and will not necessarily be prioritized on cost benefit criteria. Both the development and operation of the system will require the application of a considerable amount of expertise and professional judgement.

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Predicting future decay in concrete structures

J.G.M.WOOD

Introduction

Most concrete decay arises from the corrosion of the reinforcement. In rarer cases, sulphates attacking the cement paste, dimensional instabilities associated with the aggregates, and/or frost lead to a breakdown of the concrete matrix. This paper concentrates on the two dominant corrosion problems from carbonation and chlorides.

The alkaline environment of concrete, if free from significant chlorides, produces a stable passive condition in which the reinforcement is protected from corrosion. These conditions can be maintained for thousands of years so that the reinforced concrete will not be degraded by corrosion processes. This requires slowing the rates of ingress of carbon dioxide and chlorides so that they do not penetrate the cover to the steel. The background to these corrosion processes is summarized in BRE Digests [1] and, more fundamentally, by Page and Treadaway [2].

Carbonation

The natural carbon dioxide in the air, typically 0.04%, progressively neutralizes the alkalinity in concrete, from the surface inwards. Currie of BRE [3] has reviewed the processes and given typical data; Parrott of BCA [4] has done similar work in more detail. We know that in many bridge structures, carbonation depths do not exceed 2–3 mm after 20 years. From this we can predict (using penetration $= k\sqrt{\text{time}}$) carbonation depths of 5–7.5 mm at 120 years. Where the cover was specified at 30mm, and built with a tolerance giving a 25–35 mm range, there would be, in the absence of cracks, no carbonation induced corrosion problem at 120 years. One would expect the first signs of spalling to develop in perhaps 1400 years, unless world CO_2 levels change! However, in many existing building structures, carbonation depths reach 7.5 mm in 1 year and 15 mm in 4 years, at which point badly placed steel will start to corrode. Three or four years later the process of spalling starts scarring, and eventually weakening the structure. With such poor concrete, the influence of cracks is marginal.

Chlorides

The initiation of the deterioration process with chlorides is similar, provided precautions have been taken to minimize the initial chloride level of the concrete. (One notes with surprise that permissible levels of chloride in concrete ($Cl^- < 0.4\%$ of OPC) in the UK codes, e.g. BS 8110 [5], is the same as the level at which corrosion can be initiated.) In marine conditions it takes a period of years for chloride to penetrate the cover, then corrosion starts (see Figure 1). The period of initiation of corrosion can be estimated by predicting diffusion,

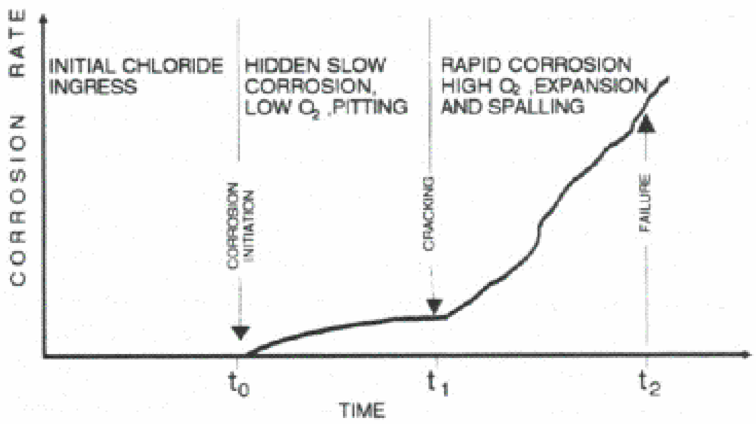

Figure 1 Corrosion development.

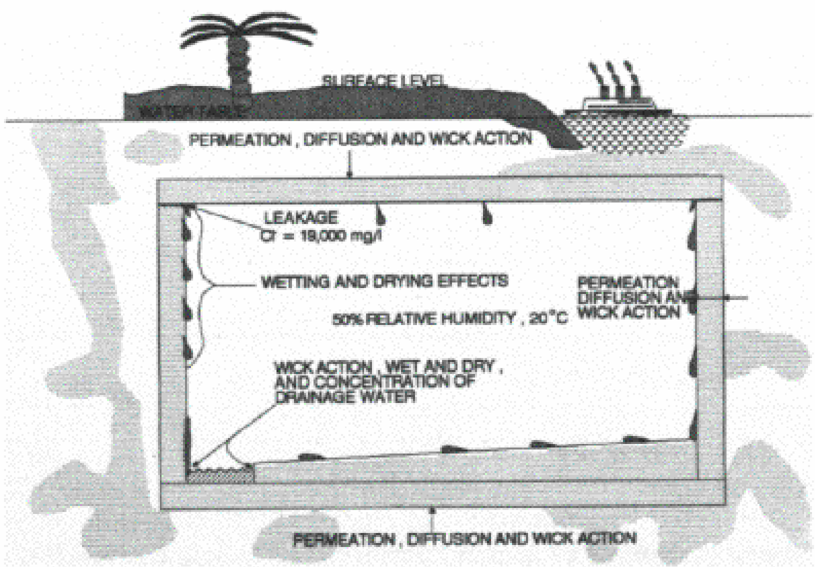

Figure 2 Chloride ingress mechanisms for buried structures.

wetting and drying, permeability and wick action processes [6] (see Figure 2). Then, two or three years after corrosion initiation, the magnitude of the corrosion product will be sufficient to crack, and then spall the concrete if sufficient oxygen is present. If oxygen is in short supply, corrosion will produce the less detectable, but more damaging pitting of the rebar. Once the concrete is cracked the corrosion process is accelerated.

Rates of ingress and corrosion

The rate at which carbonation, chloride ingress and corrosion develop is of vital importance for predicting the durability characteristics and maintenance requirements for buildings over their design life [7] and beyond. The environment and the quality of concrete can change the rate at which these processes proceed by several orders of magnitude. The durability clauses in codes reflect this poorly.

Corrosion rates can produce surface losses of rebar sections of up to 1 mm per year from general corrosion, and up to 2 or 3 mm per year in local pitting corrosion. The fundamental theoretical data from laboratory experiments in determining corrosion rates is blurred and erratic. On-site electrical corrosion rate monitoring gives chaotic and misleading results. However, selective exposure of areas of steel on actual structures using high pressure water jetting is starting to provide a good empirical basis for estimating the corrosion rate.

Temperature rise will increase chloride ingress and corrosion rates by a factor of 2 or 3 between UK (10–20°C) and Arabian Gulf (25–35°C) conditions. Hence the need for the excellent Bahrain Conferences [8] in 1985, 1987 and 1989. The cold conditions (5–15°C) of North Sea offshore structures therefore provide a misleading yardstick for simple generalized empirical extrapolation of deterioration, even in Southern England.

The relative humidity has a dominant effect on corrosion rates [9], while wetting and drying accelerates both chloride ingress, and the rate of corrosion.

The concentration of chlorides can rise from the dilute 19 000 ppm Cl^- in sea water to over 200 000 ppm where de-icing salts are spread or where sea spray concentrates by drying. Rain washing of salts off surfaces has a major beneficial effect, while ponding aggravates chloride ingress.

The coefficient of diffusion D for concrete at 25°C can range from 0.5 to over 500 x $10^{-12}m^2/s$ depending on cementitious type, water/cement ratio, compaction, etc. Similarly K, the permeability, can range from below 0.025 x 10^{-12} to over 250 x 10^{-12}m/s. These coefficients are more comprehensible as rate of ingress of chloride into initially salt-free concrete. With sea water at 25°C, a D of 0.5 x $10^{-12}m^2/s$ would give a penetration of cover to give chlorides above the corrosion threshold of 0.4% of cement, of 6 mm at 10 years. A D of 50 x $10^{-12}m^2/s$ gives a penetration of 22 mm at 10 years. Similarly, a 30 m head of water on a 300 mm thick wall with a permeability of $k = 0.025 \times 10^{-12}$ m/s would increase the penetration of chloride by a further 8 mm in 10 years.

Determining concrete characteristics

We have two reliable ways of determining *K* and *D* for concretes. One is back calculation from service performance. The other is short term (2–12 months) testing of laboratory or quality control concrete specimens. Mott MacDonald has used both. The bulk diffusion test [6] is particularly valuable as a cost effective laboratory and site test for chloride ingress. It accelerates diffusion ingress rates by a factor of 20. Normally tests are for 1–3 months, but they can be run for years. The chloride ingress profiles can be analysed and extrapolated using the program CHLORPEN.

Most laboratory test data on permeability or diffusion are for very short term 'rapid' tests lasting hours or days on young concretes. This 'rapid' laboratory testing is usually misleading and seriously underestimates the performance of concretes with pfa and slag which have a substantial improvement in resistance to chloride ingress with time as continuing hydration closes up the capillary pores [10].

Many manufactures of cement, admixtures and silica fume use the distortion inherent in early age 'rapid' tests to exaggerate the relative qualities of their products.

For carbonation, the recent work by Parrott at BCA [11, 12] and BRE [13] is improving our stock of knowledge for predicting behaviour, and provides a basis for upgrading our specification and quality control procedures in Standards, and in site practice.

Cracks

The influence of cracks on durability is a topic where lack of data and wishful thinking provoke controversy. The old view that cracks below 0.3 mm had little effect is true for poor quality concrete. For high quality concrete, recent UK and Japanese research [14] shows that cracks target and accelerate corrosion initiation. More research data must be a priority, as site data are starting to show how cracks are creating problems in structures.

Summary of state-of-the-art

The state-of-the-art may be summarized as follows:

1. There is a good scientific basis for understanding the processes, but a more limited basis for predicting the rates of these processes.

2. The techniques of numerical modelling used for structural analysis can be adapted for predicting durability. However, much of the data and some of the rate functions for this are lacking.
3. The rigorous *post mortem* of damaged structural members and the laboratory test data are starting to fill the gaps in the data needed for reliable modelling of future deterioration and its effects on the safety of structures [15, 16].

We are now applying this growing scientific knowledge and data using numerical modelling, for durability design and specification for major new projects [17], and for structures in unusually severe environments [18]. The other main application is to predict the future performance of deteriorating structures. This maintenance and repair work substantially enhances our data base for predicting deterioration as results of detailed testing and long term monitoring of concrete structures between 10 and 60 years old are built up.

However, the potential field of application is broader if we are to use the opportunities of Eurocodes, and properly assured quality control to restore concrete's reputation for durability.

Practical applications of prophecy

There are a number of stages at which we need to foretell the durability of a structure.

(a) Design and specification
(b) Construction quality control
(c) Handover at 1–3 years old
(d) Near the end of the decennial or legal liability period at 6–15 years
(e) When problems start
(f) When trying to upgrade durability by repairs, coatings, cladding, CP, etc.

Design specifications and quality control

The formalized BS system of quality control for concrete, whatever its merits for achieving strength, largely neglect the fundamental properties which determine the durability of the structure. The question, 'how deep will 30 N/mm^2 concrete carbonate in 60 years on a rain wetted column?', is not answerable. It may be 5 mm or 50 mm. The same uncertainty applies to the chloride ingress rates. The uncertainties are multiplied by the availability of slag, pfa and silica fume.

If we are to be able to predict, for design and in specifying materials, the

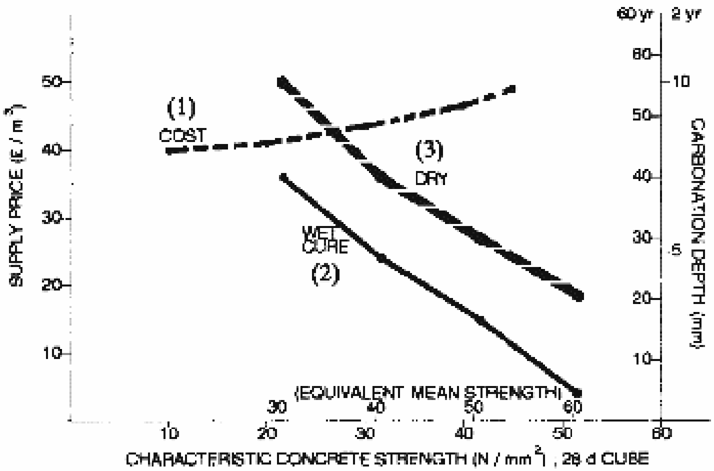

Figure 3 Cost vs. strength vs. carbonation ((1) carbonation depth vs. strength for dry curing; (2) carbonation depth vs. strength for wet curing; (3) strength vs. cost for OPC concrete, 20 mm aggregate, 50 mm slump).

durability performance to match the clients' brief, then we must go back to fundamentals and collect the necessary data. We must be able to determine that certain strength characteristics of a specific mix type, properly compacted with defined curing, will achieve sufficient resistance to carbonation and chloride ingress for certain defined environments. The process involves penetration rates of fractions of a millimetre per year into concrete by carbon dioxide and chlorides. We are naive if we think the tests that can be performed in a few days or a few hours will provide a reliable basis for extrapolation for decades.

For carbonation there is already a substantial body of data available on the relationship of cost, strength and carbonation rate [11–13]. Some of this [13] is summarized and extrapolated in Figure 3. It is relatively simple to ensure durability against carbonation, all we need to do is to specify 40 N/mm^2 concrete at £47/m^3 instead of 30 N/mm^2 concrete at £43.5/m3 and control cover.

Structures in service

Once a structure is in service, the rate of carbonation and depth of ingress of chlorides can be measured and extrapolated using data from rigorous sampling of carefully selected representative areas. The large scale analysis on a coarse overall grid on the whole structure delights test contractors and scaffolders, but the cost benefit is doubtful.

Sampling must be carefully related to the degree of dampness of the concrete, as the damper the concrete, the slower it carbonates. In contrast to the

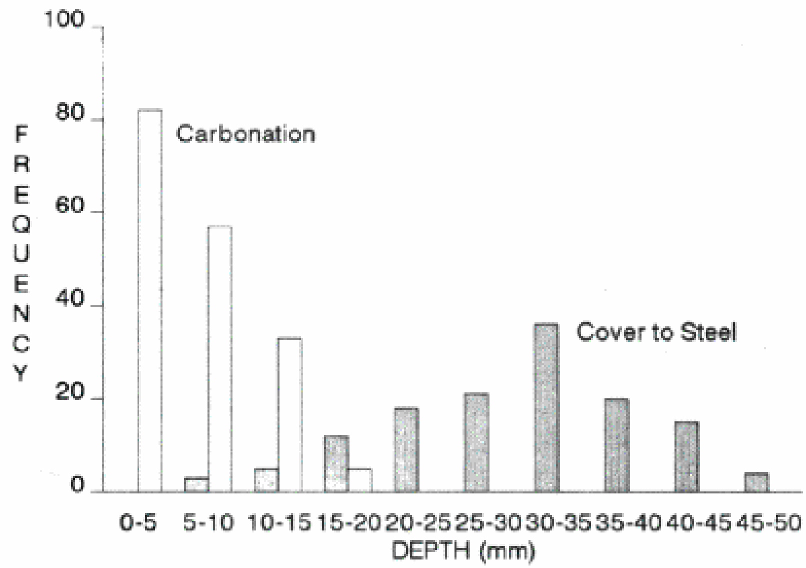

Figure 4 Carbonation and covermeter results.

recommendations of BS 8110, BS 5400 and Draft Eurocodes, the worst condition for carbonation induced corrosion is in concrete which wets and dries between 60% relative humidity and 90% relative humidity with occasional rain or condensation. This is dry enough to substantially accelerate carbonation rates, yet wet enough to develop corrosion relatively rapidly once the carbonation hits the steel. Wetter conditions postpone deterioration by slowing the carbonation. Dry internal conditions where carbonation will be faster are, in many structures, sufficiently dry to prevent the corrosion developing even though carbonation has gone past the steel. The carbonation depth data must be related to the histogram of rebar cover depths (see Figure 4). Poor control of cover is a more frequent cause of early bridge or building corrosion problems than poor concrete.

Predicting future trends in marine structures or structures where the exposure to salt is reasonably consistent can similarly be based on sampling representative areas. Once the structure has been in service for a year or so, precise profile grinding of the initial chloride ingress profiles in the concrete can provide a basis for extrapolation. As the structure matures, so the data for reliable extrapolation improve, and after 5 years in service it is possible to make realistic estimates of the range of times when chlorides will reach the steel in the structure and when corrosion will be initiated. Predicting the corrosion *rate* thereafter is more difficult. As with carbonation, a very selective sampling procedure based on a representative range of concretes, and a representative range of exposure conditions is critical for the cost effectiveness of this type of exercise. For each area, matching rebar cover histograms must be produced.

Bridges, as structures subject to road salt, are very much more difficult to predict. Much the worst and most damaging chloride ingress occurs where the waterproofing system on the road deck breaks down, permitting salt to accumulate and spread below the waterproofing, or to dribble down through expansion joints. There is a widespread lack of appreciation that chloride solutions from de-icing salts can be 10 or 15 times stronger than seawater, but may only be present for short periods of time. There is strong American evidence that even unwaterproofed bridges with plain concrete decks perform well if they have good cover and, more importantly, a good cross fall. This is because the rain freely washes the salt from a plain concrete deck, whereas it seeps down through, and is trapped below, a damaged waterproofing layer. The provision of falls, drips and drainage to carry salt away is crucial.

The spray problem from vehicles needs to be considered not only for bridges, but also for buildings adjacent to major highways. The greatest risk is on bridges with ledges where spray can accumulate but is not washed off by the rain. Perhaps more thought should be given to washing off structures after the winter. Once the bridges have been in service for three or four years, predictions can be made of when corrosion will initiate and then produce damage, but this assumes no change in exposure. From the degree of moisture, some idea of the type and severity of corrosion can be estimated on the basis of experience on other similar structures which have been opened up for repair and reconstruction.

However, where there is a changing condition (e.g. the deck surfaces re-waterproofed with material which is substandard), then the basis for extrapolation is destroyed. Of particular concern are the structurally important details, like half joints, where the structural cracks target chloride in to corrode highly stressed steel.

Construction to reduce future problems

A proper specification for durability, rather than just strength, and proper supervision of the materials and site construction practice, is important. QA is of little value without rigorous quality control with the proper test procedures. Cover meters are available. Tests for durability quality must be developed.

Where site supervision shows up structural defects due to badly placed steel giving inadequate cover, or due to poor quality compaction, rigorous remedial action is required. The only fully effective action is to reconstruct. Repairs and coatings cannot substantially improve poor construction as they have their own inherent durability problems [19]. Coatings, in particular, have lifetimes far shorter than those of good structural concrete. Once the structure is in service there can be some benefits to durability from coating and cladding [20]. The greater benefits are from proper maintenance of the waterproofing, drainage and expansion joints.

Some objectives for the 1990s

a. For all structural concrete
   - Rigorous control of cover; cover histograms to be supplied with as-built drawings
   - Control cube strength to be set above the strength requirement to limit carbonation rate for mix type, curing and environment
   - Readymix, precast and site concretes to be QA tested on a monthly basis using standard carbonation rate test (new test needed)

b. For concrete near highways, the sea or in car parks
   - Initial chlorides in concrete to be below 0.06% of cement
   - Control cube strength to be set to limit chloride ingress rate for mix type, curing and environment. Higher strengths at 28 days will be needed for OPC than ggbfs and pfa
   - Concrete to be QA tested on trial mix and monthly with bulk chloride diffusion test
   - 20% of bill of quantities for concrete, formwork and rebar to be paid only if wet curing is maintained for 7 days after the formwork is removed.

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Financial implications: refurbishment, replacement, insurance, privately funded projects

E.C.CHAPLIN

Introduction

During the past three to four years the construction industry has been debating and showing keen interest in the private funding of civil engineering projects, notably toll roads and bridges. This paper concentrates on this evolving form of investment in the United Kingdom using it as a vehicle to raise a number of design life issues which have wider financial impact throughout the industry.

Civil engineering projects have generally been designed to serve one purpose indefinitely, whereas buildings are likely to experience many changes of use throughout their lives. It is well known that increased maintenance and replacement is now a feature of the highway network. The underlying causes are principally greater traffic volumes and heavier axle loadings, with further increases to come, than those originally envisaged. In effect, the UK roads and bridges have now been subjected to changes in use with consequent implications on design life.

Such effects are encouraging fundamental changes in attitude towards the life expectancy of materials, maintenance strategies for structures, whole life cycle engineering techniques for infrastructure investment, as well as identifying the need to obtain reliable information on 'performance'.

Background to private funding

- Projects financed by funding institutions up to and after construction completion, the commencement of income generation
- Project maintenance financed from revenue
- Capital cost comprises construction cost as well as:
  - interest charges
  - enabling fees (for initial design and for obtaining Parliamentary powers)
  - commissions

  - legal fees
  - management costs, etc.
- Construction cost defined as sum of:
  - final design
  - construction cost up to commissioning stage
- Operational cost consists of:
  - maintenance/refurbishment costs
  - interest charges on debt
  - offset from revenue

Critical factors

In general:

- Capital cost=approximately 1.5×construction cost. Hence capital cost must be reduced to the lowest level because of high financing charges.
- Due to risks, funding institutions prefer to limit the finance loan period to between 15 and 20 years maximum. Such projects must be seen to be robust.
- Financial confidence in the project should improve with time after commissioning as revenue generation becomes apparent. Financial risks are therefore diminished and interest charges can be reduced by re-financing of loans in several tranches (say five year intervals).
- Concession period for project operation geared substantially to loan periods (currently continually reducing) dictated by the money market and level of required profitability. Hence to minimize financing costs, the concession period should be long or unlimited, thus permiting re-finance and regulated, if need be, by restrictions on the operator to safeguard the public interest.
- Planned maintenance costs and revenue should ideally both rise in parallel with RPI (retail price index) thereby offsetting each other. Hence any financing of maintenance costs is considered a lower risk area.

Risks and concerns

Risks and concerns perceived by promoters and the funding institutions are:

- Uncertainty in the movement of interest rates during the loan period
- Changing taxation requirements, dictated by Government
- Reversion of the project into public ownership, also dictated by Government
- Rate and magnitude of revenue generation, dependent on forecasting of future traffic volume

- Unforeseeable changes in use of the project, e.g. increased traffic loading
- Engineering uncertainties ill-defined at offer stage
- Little-used construction techniques

Impact on design life

In such a market economy approach, the concept of projects having a design life improved by a high initial investment and supported by a relatively low level of maintenance is not viable. Private promoters, affected fundamentally by the cost of front-end financing, would prefer relatively higher maintenance costs with a much lower level of initial investment.

In this context, does the design life of the project become theoretically infinite since individual structural elements, if suitably designed, can be maintained, refurbished or even replaced at intervals responding to usage demands? This policy would reduce 'crystal-ball gazing' to acceptable proportions and recognizes that all materials, however well used and maintained, cannot be expected to have an indefinite life. Perhaps instead of using the term design life for materials and components, the concept of design death has some merit, akin to planned obsolescence?

Nevertheless, a case might be made for a modest increase in the initial investment for, say, deep foundations difficult to modify at a later date, strengthened to cater for future loading arising from changed circumstances.

In privately funded projects, the scepticism of the financial and insurance markets towards unfamiliar construction techniques can be understood, a policy of restriction on untried or challenging ideas arising from the unknown with reliance on basic technology. However, frustrating innovation must surely stifle the breakthroughs needed to enhance durability and so reduce maintenance costs?

One recent suggestion is to adopt lower standard road construction materials rather than those currently required. Although a lower first cost may lead to a higher maintenance/replacement cost, this could be acceptable under a private funding arrangement, as well as winning environmental votes. But in the event of defects arising, either through design or workmanship, the insurance market would probably not regard this as a fortuitous event. The risk would be uninsurable since insufficient experience would be available to convince underwriters.

Conversely, the enhancement of road material specifications in excess of that currently required could improve design life but at a price which the private sector could not afford.

Why does the industry not carry out more investigation into performance and life expectancies through accelerated age testing of materials and structural components and so be able to better predict the maintenance strategies needed today, as well as relying on decades of experience? Would this give further impetus to the wider adoption of whole-life costing? Or are we as engineers, too fascinated with attempting to produce perfection with our materials?

Conclusions

Although the questions raised on design life have been presented in the context of privately funded projects, they can be addressed equally to traditional forms of infrastructure provision.

It is now evident that the industry has recognized the need to place greater emphasis on investment and expenditure profiles in relation to the life performance of materials and components. This emphasis will vary between clients in the public and private sectors, the former seeking the means to maintain accountability in the face of mounting maintenance costs and the latter, particularly in the building sector, requiring more accurate performance data. The advent of 'intelligent' buildings, the increasing attention to professional liability, decennial insurance and collateral warranties, not forgetting environmental issues, places an urgency on these investigations which hopefully will give us an improved expectation of reliability (structural and cost) and improve the public image of the industry.

Review of past experience: what to do (and to avoid) in the future

C.P.J.BARNARD

Introduction

The concept of design life for highway bridges was not recognized in former times and the idea is only now slowly gaining acceptance. It is as much the realization of the important part that maintenance plays in the life of a structure, as an essential post-requisite to the design and construction phase, that is leading to the development of the design life concept.

The past

Early bridges were built to serve only local needs, and such design as was carried out was certainly not to any recognized code or national standard. It is unlikely that the question of maintenance would have received much consideration in those days and it seems that maintenance of highway bridges was carried out only at infrequent intervals and sometimes then solely as a result of a court order.

Responsibility for bridge repairs lay with the inhabitants of a district or owners of neighbouring land and, as a consequence, often little was done until remedial works became absolutely essential. Frequently there was doubt as to who was responsible for repairs and so a decision had to be taken by a jury at Quarter Sessions. If the inhabitants were found liable, a rate was raised to pay for the cost of the repairs. Old court records point to this lack of maintenance since orders were made as often as every 10 to 20 years to repair the same bridge.

The present

It is encouraging that matters have improved significantly since then and there is now a better understanding of design principles. Designs are now carried out to a national standard for stresses and for loadings and, with these current standards, bridges are deemed to have a notional design life of 120 years.

Detailing of bridges is more satisfactory which, together with the present specification for construction, should lead to structures having greater durability than previously. Additionally, there are the benefits of improved materials and methods in carrying out works.

Can it now be said that we have in fact achieved our aim and that it is certain that bridges will survive for their full design life? This appears to be an optimistic view which is unlikely to be realized, judging from the number of structures at present in train for assessment and strengthening and those where major faults are already evident. So where have we gone wrong? The most likely reason is that, in setting the present standards and specification, there was an assumption that these codes would make bridges relatively maintenance free.

Unfortunately, this assumption has not proved to be sound and there are many examples where deficiencies in bridges have led to the need for premature action to keep them in good order. There are instances of inadequate design, or of designs taken to, or beyond, the limit of current knowledge. Many bridges are now overloaded from the enormous increases in vehicle loading and intensities. Poor construction details and specifications, together with low standards of construction, leave the bridges more vulnerable to attack from the ingress of water and aggressive agents and from early deterioration. The increasing use of de-icing salts and the premature failure of products and materials contribute to the current problems. All of these factors result in a need for more frequent and extensive maintenance and repair in a bid to extend the life of structures.

Design life

It is evident, therefore, that the concept of design life for a bridge is not an idea which can be isolated from other obligations, particularly that for maintenance. Maintenance is the one field which has been much neglected in the past, but where there needs to be concentration of effort in devising policies and strategies to ensure that the design life of the bridge is, in fact, achieved.

In addition, it is not helpful to find that with current funding levels there is an implied policy that the expected life of a bridge is not 120 years, but is more likely to be an indefinite life. This is a result of the limited rate for bridge renewal which is being supported at present. The OECD report on bridge maintenance in 1981 stated that, in the United Kingdom, the rate for bridge renewal was less than that necessary to meet the assumed design life of 120 years for modern highway structures. The figure quoted in the report was about 0.4% per annum, meaning that the average bridge had to last for about twice its assumed design life.

The figures for other European countries and the United States are roughly comparable and so the idea of a design life limited to even 120 years does not appear to be generally accepted in financial terms. The outcome of this is that an even greater emphasis has to be put on the need for maintenance.

But why is design life important and what factors make it a concept to be followed, rather than leaving maintenance and repairs to be carried out as and when they prove to be necessary? Some reasons are that:

a. Any bridge should have the strength and capacity to carry the load imposed from traffic for the whole period that it is in service.
b. If the bridge is able to do this, traffic routes over the bridge can be maintained. This is essential because of the ever greater dependence we have these days on road transport for the movement of goods as well as people.
c. With a planned design life one would expect there should be an overall cost economy in the managed approach to maintenance. This is an important factor since the majority of bridges are maintained in the public sector and therefore public funds are being used.

Once the principles of the design life for a structure have been accepted, the aim should be for the bridge to have a sound initial design and construction and have managed maintenance for the agreed life appropriate to that structure. This does not mean that all parts of the structure should last for the full period, but that any maintenance and renewal of parts of the bridge should be carried out at the appropriate time, according to the condition and expected life of the individual part in question.

The future

The means of achieving the goal of a full design life for highway bridges will vary depending on the stage reached in the life of the structure. In view of the low renewal rate, an important aim must be to conserve and extend the life of the present bridge stock and to improve the probability of a full life for new bridges.

The first steps have been taken to deal with existing bridges by the implementation of the present assessment and strengthening programme. This was initiated by the Department of Transport for trunk roads and motorways, but now it is a country-wide exercise. The intention is to identify bridges with substandard carrying capacity and to strengthen or restrict these, as appropriate, to enable relative freedom of movement, primarily for the 40 tonne lorries due to be permitted on the roads from 1999 onwards.

However, a lack of load carrying capacity is not the only problem in existing bridges. The next phase, which should run concurrently with strengthening work where possible, is to correct the many other deficiencies in bridges. These other shortcomings can be summarized either as omissions or as mistakes.

For instance, many structures lack deck waterproofing, or have an inadequate system, and most bridge parapets are below present containment standards. Effective protection is only now being included against the carbonation of concrete, chloride attack from de-icing salts and problems with the penetration of water and its subsequent disposal. Access to bridges for inspection and maintenance has rarely been given much attention.

Mistakes in construction details, specification and construction itself have led to the need for earlier remedial measures than should reasonably be expected. This can result from poor design details, such as, for example, the means to seal joints properly or the use of half joints in bridge decks; the latter could be the next major problem to solve. Alkali silica reaction in concrete should no longer be a problem in new bridges, but construction defects as basic as the cardinal sins of lack of cover, compaction and curing in concrete works are still with us.

Many materials and products have often not lived up to the claims made for them. Proprietary components, including deck waterpoofing and joints, parapets and bearings have proved to be deficient in too many instances. This is possibly from a lack of realization of the searching examination they get from water and de-icing salts, or from the damaging effects of present day levels of traffic. Even weathering steel has a question mark against its performance now that problems of salt contamination, crevice corrosion and the potential for corrosion fatigue cracking have been reported in the United States.

Correcting these faults will be expensive, for example recent reports suggest that up to £400 million needs to be spent on the Midland Link viaducts alone! However, what is equally important is not to repeat the mistakes in the future on new bridges. Much can be done at the design stage to incorporate features which contribute to easier maintenance.

Design for maintenance

Greater feedback of the details of current problems would be a good starting point in helping designers to detail and specify bridges which have the minimum maintenance requirements incorporated at little or no increase in cost. A maintenance audit helps to identify appropriate details, and the preparation of a maintenance manual is only now being included as part of the design and construction process.

Factors to be considered in design include accessibility and the need to minimize the amount of maintenance work. Accessibility to all parts of a bridge which require inspection and maintenance is essential, whether this is by means of access gantries, or is provided by adequate clearances around members. Maintenance needs can be reduced by the appropriate selection of materials, particularly more durable concrete and maintenance-free metals.

The elimination of vulnerable components should be considered by having, for example, continuous decks to obviate road joints. Components may have to be over-designed if they are costly to repair or renew. Those with a limited life should be simple to remove and replace. Regular, planned maintenance operations should be made easy.

Aggressive agents, particularly water and de-icing salts, should be restricted by adequate drainage, by the sealing of joints and by the waterproofing of exposed and vulnerable surfaces. A knowledge of current research into materials and allied subjects might be beneficial when considering alternatives, but care in the use of new materials is important.

Maintenance policies

An improved approach to maintenance is equally necessary to match the hoped for improved condition and design of existing and new bridges. Programmes of bridge inspections are routinely carried out and better record keeping and data bases are being developed.

However, monitoring the deterioration of defects is only at an early stage, and is not widespread. It is to be hoped that this part of the inspection process is extended to cover the overall condition of all bridges, even if it has to be selective at first, in order to produce a reliable rating system for maintenance priorities.

Elsewhere, detailed work is already being done on condition surveys of concrete bridge components by the US Transportation Research Board. Examination of components to check their properties and condition and the factors causing deterioration could lead to a better evaluation of the need for repairs or replacement and to predict component life.

When repair priorities and the evaluaton of the performance of bridges in service can be more readily determined, the information will provide the lead to a more productive management of bridges. It should enable informed decisions to be made as to the optimum time for remedial measures and the standard of maintenance to be expected. Benefits should be seen also in aiming for a policy of preventive maintenance and to seek the best returns on current expenditure by effective procedures in carrying out that maintenance.

Conclusions

To get to this ideal situation will be expensive, the assessment programme alone is expected to cost at least £1000 million, so it will not be done overnight, 15–20 years would be a more realistic target. Training and education are equally important to get the message across to those involved and research must play its part. But if these improvements in the strength and condition of existing bridges, in the design and construction of new bridges and in the business of managing the subsequent maintenance of all bridges can be achieved, then design life does have some meaning.

Summary of presentations and discussions

G.SOMERVILLE

The chairman opened this session by reminding delegates of the six key questions to be addressed at the Colloquium and asked authors to concentrate on these when introducing their papers. He also expressed a personal opinion that something had to be done to improve the performance of structures, since society took an unkind view of the construction industry's performance in the last few decades; there was also a need to reduce the incidence of litigation, arguments and general uncertainty about the performance required and how to achieve it.

In introducing his paper, Mr Rodin observed that the term 'design life' implied that this was a conscious part of the design process. In his experience this was not so; in general buildings lasted only as long as the owner wanted them to. However, some bad experiences with new forms of construction in the 1950s and 1960s had gone beyond that, with useful life being dictated by a lack of in-service technical performance. At the beginning of his paper, he had cited five reasons explaining why the life of some buildings had been unexpectedly short; three of these were within the domain of the designer but two were not, and were largely unforeseeable at the design stage.

Mr Rodin then stressed the seven points in his paper, which, in his opinion, would ensure a robust building if properly treated by the designer. He then added an 8th point—optimization of the construction process. All of these presented opportunities for adding extra touches for greater durability, and he argued that this was justified in financial terms, i.e. if the asset value was around £1000/ft^2, and the building could be rented at £60/ft^2, then the construction cost of £30–40/ft^2 could reasonably be increased by up to 10% with the strong possibility that the asset value would be substantially increased.

Dr Menzies began his introduction by defining what was meant by populations of buildings and gave examples from recent practice. He then showed slides illustrating failures which had occurred over the last 20–30 years. He stressed that the consequences of failure with populations of buildings were proportionally more severe, and possibly one solution was to introduce greater variety into buildings. Experience had shown that there had to be an in-built early warning system which indicated signs of deterioration well in advance of incipient failure. In drawing on past experience of system building to develop the concept of design life in the future, Dr Menzies felt that greater effort was required to 'get in right'– justified because of the repetition involved—since the risks and consequences of failure were substantial.

In introducing his paper on bridges, Dr Richmond stressed the need to take a very wide view in considering all the factors that could influence in-service life. Many of the points made by Mr Rodin about buildings also obtained for bridges, although, in general, the required performance for bridges was more demanding and for a longer period. Using two specific examples (the Tees Viaduct and the second Severn Crossing), Dr Richmond illustrated in detail the factors which had to be considered.

An extensive and lively discussion took place after the presentation of these three papers, covering both matters of principle and detail. The following points were noted:

1. There was possibly a need to distinguish between 'technical life' on the one hand (which would involve the selection or design of a system expected to last at least *n* years) and reliability aspects on the other (i.e. identification of, and checks on, the factors which could influence whether or not the chosen system would work as planned). This point also relates to papers presented earlier by Professors Schlaich and Tassios where two key comments were made:
    - i. the need to separate out the scientific consideration of steady-state degradation from factors due to real errors or deficiencies (Schlaich)
    - ii. accidents (or earthquakes) would require separate treatment (possibly qualitative) from deficiencies due (say) to corrosion (possibly quantitative) although the effects were often interactive (Tassios)
2. Considerable sympathy was noted for the view that structures should be classified very simply in terms of life, perhaps just as 'normal' and 'long life', although the point was made that specifying a life in years would put up a signal that longevity was important and required careful consideration in a particular case.
3. It was absolutely essential to develop a maintenance strategy at the design stage, and to make provision for inspection, maintenance and possible replacement. Some types of structure require more maintenance than others; the nature and location of parts of structures may make access virtually impossible, and hence these parts would require to be 'life long', since maintenance was not a practical option.
4. Structural performance and life are dependent on function, environment and other factors which are not determinable at the design stage. The aim of design might then be that the structure will last a long time (indefinitely?) provided that it is properly maintained, and known shortlife elements (those whose life is determined by procedures such as those used by the British Board of Agrément) are replaced as required.

Based on this premise, and notes 1–3 above, a modified version of Mr Rodin's 7-point plan might be as follows (to help develop a design life rationale):

i. Systematic design to cope with known hazards, considering both risk and consequence (possibly involving a design review/audit of durability, linked to the importance or criticality of the structure or structural element),
ii. Relative insensitivity to:
   - marginal departures from the design assumptions
   - local defects or movements
   - environmental change (both micro- and macro-climate).
iii. Buildability, and not total dependence on perfect workmanship and compliance with specifications. The establishment of minimum standards of workmanship and relevant specifications, together with appropriate (quantifiable) methods for checking that they are met, is perhaps *the* key issue in the reliability issue mentioned in note 1 above.
iv. Provision of good access for all items requiring inspection or maintenance. If this is not possible, then see note 3.
v. Incorporation of early warning visible signs of serious defects.
vi. Movement is limited, during the expected useful life, so that the function of the structure is unimpaired.
vii. Capability to allow some change in use.

The essence of this rationale is that there should be some redundancy, tolerance/flexibility in the system (robustness, in Mr Rodin's terms). A question then remaining is: is this sufficient in itself (provided that the seven items can be sensibly addressed in quantitative terms) or is it necessary to have a range of target lives (however expressed) to cover the full range of structures which the construction industry has to produce?

5. Following on from 4 above, various problems were raised which would inhibit future development of this rationale. These included:
   - Who takes the lead (or pays) in developing improved standards?
   - The need to 'sell' such a concept to clients (whose expectations would vary) who might well be aiming for a low initial investment, and would need to be made aware of longer-term benefits
   - The whole question of the legal liability of all the many parties involved in creating a structure
   - The general lack of knowledge, uncertainty and variability of the factors involved, including acceptable levels of performance requirements
   - The need to establish the sensitivity/vulnerability of different types of structure, and the influence of detailing (both of the structure and its fabric and fittings)
   - The interaction of many of the factors involved

6. On specific detailed points, there was discussion on the use of external, unbonded pre-stressing tendons to permit inspection (and replacement, as and when necessary), because of uncertainties about the adequacy of grouting. While recognizing the possibilities of such an approach, it was pointed out that the knock-on effects of any such proposal would require consideration, e.g. much greater reliability on the performance of the anchorages.
7. Still on detail, and still on concrete, it was suggested that a general improvement in providing resistance to corrosion could be obtained very simply by increasing concrete grade and cover. However, other contributors felt that such simple recipes should be evaluated in context. The prime performance requirement was to provide adequate resistance to corrosion due either to carbonation or chlorides. *All* possible options (including coatings to either the rebars or the concrete) should be evaluated, both in terms of effectiveness and of life cycle costing; taking steps to avoid, or reduce the intensity of the aggressive action might also be an economic option.

Following this discussion period, the remaining four papers in this session were introduced.

In introducing his paper, Mr Dawe stressed that bridges were very much 'long life' and that maintenance was crucial. Such structures were expected to remain functional for as long as required, and for as long as it was economic to carry out maintenance or other remedial works. He felt that prescribing a nominal design life had little practical significance on actual design procedures at present, due to the current state of knowledge. To illustrate that, and to stimulate discussion, he posed two questions:

- What is the difference between a bridge designed for 30 years, compared with one designed for 90 years?
- What is the residual life of a bridge after it has reached its nominal design life and how do you quantify it?

Mr Dawe was in favour of a simple classification system, in design life terms, and favoured 'short', 'medium' and 'long'.

Dr Wood was not in favour of a simple classification such as 'normal' and 'immortal'; he felt that greater precision was possible and cited computer modelling of chloride penetration as an example of where predictive modelling had developed significantly. He was critical of the present simplistic approach in Codes to environmental classification and stressed that this was an area where considerable work was required. He cautioned against undue reliance on very short-term tests in predicting performance, over (say) 120 years, with any confidence. Dr Wood was generally in favour of raising standards in specifications, as a recipe-type solution, and highlighted the section in his paper, dealing with 'practical applications of prophecy', which indicated different circumstances when the prediction of future performance might be required; each possibly requiring a different approach and answers to a different set of questions.

Mr Chaplin concentrated on financial aspects in the private sector. He began by saying that the cost of financing a project could be up to 50% greater than its construction costs. Most clients, in his experience, would wish to reduce the initial investment costs, while accepting higher maintenance costs, which could be partly offset against revenue. There was a real need to plan for obsolescence, and possibly to think in terms of 'design death' rather than design life. In general, clients do not like change, but preferred standard tried-and-tested solutions, which, Mr Chaplin admitted, worked against innovation and new ideas. He very much supported the techniques outlined earlier in Mr White's paper. Mr Chaplin further contended that no material was perfect, and hence the need for maintenance was inevitable; he argued that this favoured the lowering of standards in specifications. He ended by saying that private sector investment in structures was very much a 'bottom line' affair, aimed at minimizing all costs, and this would be a major factor in the decisions taken by clients.

Mr Barnard stressed that bridges were very much in the 'long life' category. Their prime function was to carry traffic continuously for their entire life, with minimum interruption. However, it must be recognized that loading specifications could well increase several times during the current nominal life of 120 years. Maintenance was of vital importance. In his experience, component faults (joints, bearings, parapets, drainage, etc.) were the most significant cost, and 'failure' of these could reduce the potential life of the structural elements in bridges.

The discussion which followed the presentation of these four papers was mainly on the same points which had arisen in the earlier debate after the papers by Rodin, Menzies and Richmond. Additional points which arose included:

- A 120 year life (currently the target for bridges) is virtually indefinite. Current knowledge would not introduce significant changes into any structure required to last (say) 70 years or 120 years. Therefore, structures should indeed be classed as 'normal' (up to 50/60 years) and 'long life' for anything beyond about 60 years. There was not total agreement on this, it being argued that knowledge was being obtained so quickly at present that finer distinctions could be made in the near future, should this be considered necessary in economic and performance terms. There was then detailed discussion on current progress with the predictive modelling of various deterioration mechanisms.
- Specifying different design lives might require the definition of different margins or factors of safety. Would we ever be in a position to do this, and how much erosion could we allow in these factors before a

structure was considered unsatisfactory, unserviceable or unsafe? Clients tended to want unequivocal answers to questions such as: Is the structure safe? Will it still be safe in *n* years? Can it be maintained/repaired to give another *x* years of service?

- Any structural approach, based on design life concepts, must involve a maintenance strategy, and the encouragement of clients to take a greater practical interest in their investment. Was this ever likely to happen, or would clients continue to think in bottom line terms, and to take decisions based on change in use rather than on structural performance in technical terms?

D What happens next—the way ahead

Determination of the residual safety and service life of old steel bridges and conclusions for the design of modern bridges

G.SEDLACEK

Introduction

A great many existing steel bridges were built in the last century. In Berlin nearly 500 steel bridges of the underground railway have been built up in the years 1890 to 1910 and since then have undergone several phases of repair or strengthening after damages in the world wars or due to changes of service requirements. For these bridges the question of the actual safety for modern traffic loads and the remaining service life is put forward.

Up to now, the Institute of Steel Construction in Aachen has expertised 38 old railway bridges in Berlin (Figure 1) [1, 2]. This paper describes a procedure whereby the residual safety and service life of old steel bridges may be determined and how a basis may be established on which economic decisions for further strengthening or replacement by a new bridge may be taken. The procedure is described by the example of one of the 38 expertised old railway bridges, the Anhalter-Bahn-Bridge in Berlin. It may however be applied to any other steel bridges.

Problem

The structural system and the cross section of the Anhalter-Bahn-Bridge is shown in Figure 2. The bridge is riveted and was built in 1890 as a part of the underground-line U 1 in Berlin (Figure 1) crossing the Landwehr-Canal. It was blasted in the Second World War. After the war it was repaired.

In view of the bad corrosion state of the bridge, the railway authority asked for an expertise that should answer the following questions:

1. Is the bridge sufficiently safe for actual service conditions?
2. If so, what is the expected residual life and what are the requirements for inspection and maintenance to ensure the expected residual life.

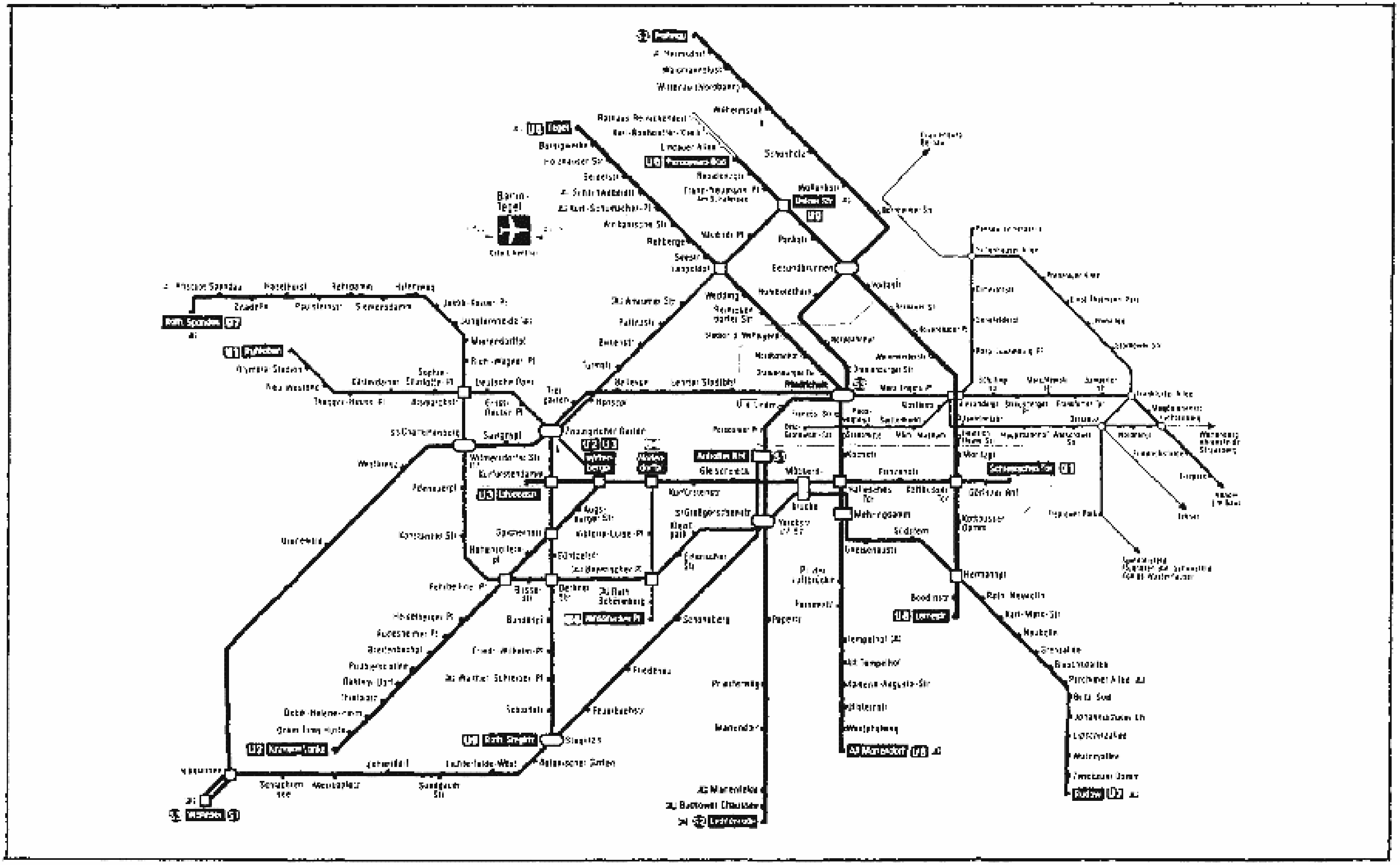


Figure 1 Bridges of the S- and U-train lines of Berlin.

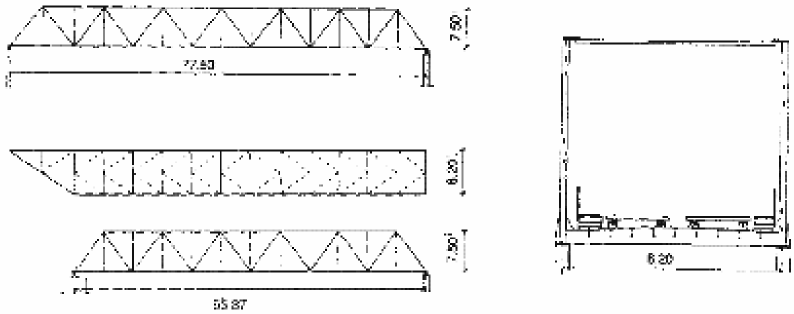


Figure 2 Structural system and cross section of the Anhalter-Bahn-Bridge in Berlin.

General conditions concerning 'brittleness' and 'ductility'

To answer these questions, a definition of different failure modes in view of 'brittleness' and 'ductility' is necessary. A structural member that may have small undetected cracks due to fatigue or other reasons may show different failure modes, which may be best distinguished by the example of a plate in tension with a central crack (Figure 3) that models a hole with small cracks on both sides:

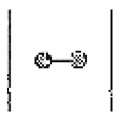
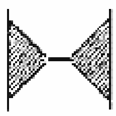
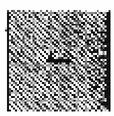
yielding pattern	failure mode	design values
	fracture before net-section yielding — brittle	applied stress distribution in the net-section + residual stresses + restraints
	fracture after net-section yielding — ductile	applied nominal stress distribution in the net-section
	fracture of the net-section after gross-section yielding — ductile	applied nominal stress distribution in the net-section

Figure 3 Definition of failure modes and the applied design values dependent on the ductility level.

1. Unfavourable failure is exhibited when fracture occurs before net section yielding with only local yielding at the crack tips. In this case all actual stresses in the net section comprising residual stresses, stress concentrations and the stresses due to other restraints have to be taken into account. This failure mode commonly is called 'brittle' failure.
2. If failure in structural applications occurs by failure after net section yielding, only the nominal stresses due to external loads in the net section are relevant, and notch effects, residual stresses and stresses due to other restraints may be neglected. This and the following mode are called 'ductile' failures.
3. A particular ductile failure mode, which may be required for plastic zones in plastic hinges or dissipative zones in seismic design, is achieved by fracture in the net section after gross section yielding. In this case the stresses from external loads only in the gross section are controlling the design of the member and the net section is capacity designed.

The failure mode is mainly influenced by the material, the temperature, the loading rate and the shape of structural member (state of stress).

For the safety assessment of old steel bridges of the kind of the Anhalter-Bahn-Bridge, in general the first and the second failure modes are relevant, as the assessment has to be carried out for design situation with low temperature.

General procedure

The following steps are necessary to determine the actual safety and residual life of the structure.

1. Establishment of a failure scenario, where the consequences of failure of the different bridge elements for different design situations are investigated. Those bridge elements are identified as vital elements, the failure of which would cause an immediate overall collapse. For the Anhalter-Bahn-Bridge the vital elements proved to be the tension chord and the struts (see Figure 4) as all other members are either redundant or stressed so little that they do not produce risks.
2. Vital elements which may fail by fracture due to tension loads are assessed in the following way:
   a. Several loading cases are determined with combinations of self-weight, traffic loads including dynamic impact and temperatures and with or without residual stresses and restraints depending on the expected failure mode. For these loading cases and a crack situation in the vital element that is assumed such that the crack sizes just reach the size of detectibility, the applied fracture mechanics action effects in terms of J_{appl}, see next section, are calculated.

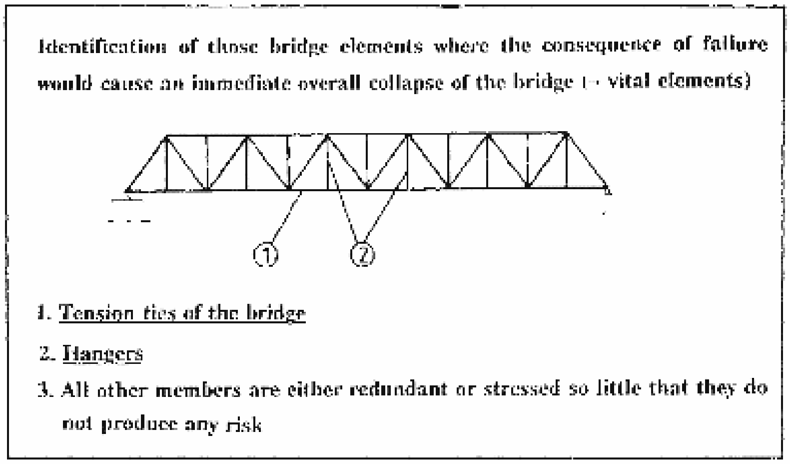


Figure 4 Vital elements of the Anhalter-Bahn-Bridge.

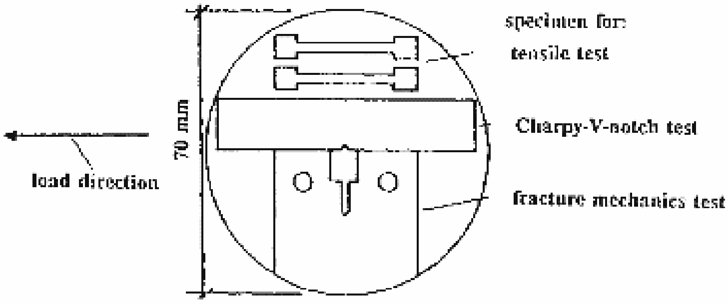


Figure 5 Miniaturized test element and available test specimen for different tests.

b. From miniaturized plate samples which are drilled from the vital elements at locations where they do not reduce the safety, the fracture mechanics material resistance in terms of J_{crit} is determined, which allows to carry out a safety check by

$$J_{appl} \leqslant J_{crit}$$

From this safety check and by varying the crack length assumptions, a critical crack length can be determined which indicates what amount of crack extension beyond the size of detectibility is tolerable.

3. From observations of the actual traffic situations on the bridge and extrapolation to future developments, a fatigue load is defined which allows us to determine the residual service time of the bridge on the basis of the crack propagation rate from the detectable crack to the critical crack size with a fracture mechanics model. The procedure developed in [3] is illustrated in Figure 6, in which the vital member

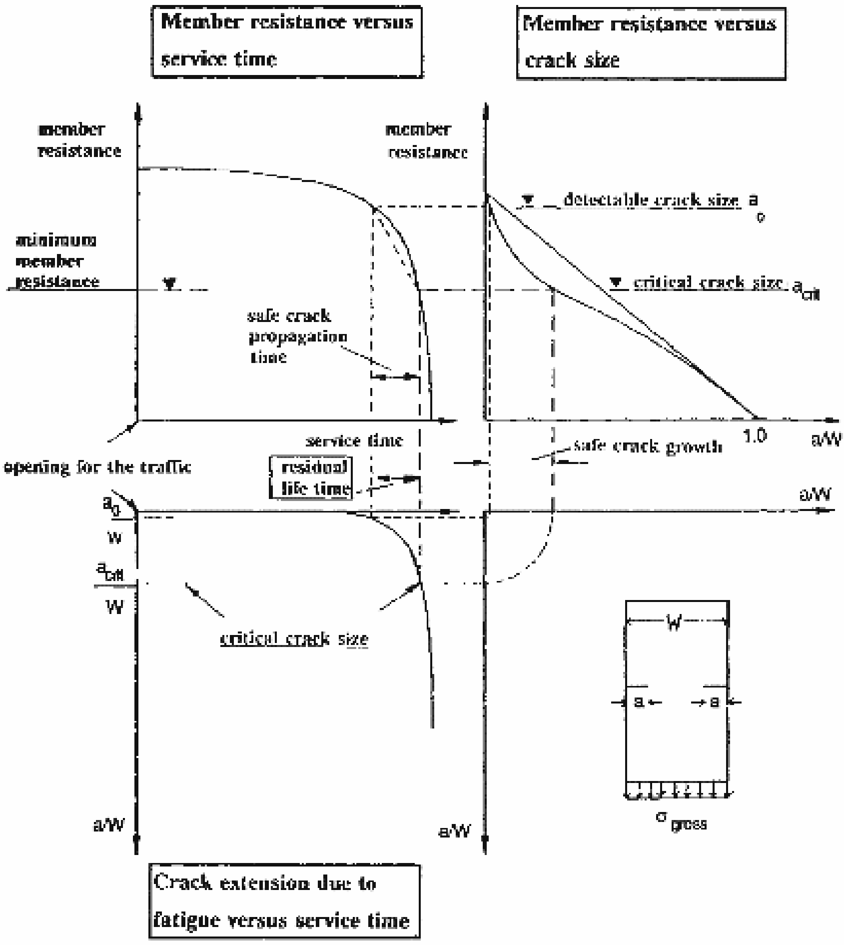


Figure 6 Connection between the member resistance and the crack sizes as a function of the service time.

resistance is plotted versus service life, due to fatigue induced crack propagation. It has proved its applicability and usability for different road bridges before it was used in Berlin [4, 5].

The main *advantages* of this procedure are the following:

1. It can be demonstrated that cracks with detectable sizes can be accepted without catastrophic consequences and no collapse (without prewarning) may take place. If this check is not positive, the member has to be strengthened with tough material or to be replaced before the next cold season (loss of toughness at low temperatures).
2. It can be demonstrated that the crack propagation from the detectable crack size to the critical crack size takes sufficient time, to allow for economic intervals of inspection. If this check is not positive, a

strengthening with tough material or a replacement should be considered.

3. In the case of both checks being positive, the inspections at safe intervals at the critical locations of the vital elements will allow the following conclusions:
   - As long as no cracks are observed, the structure is with sufficient safety fit for at least the service period up to the next inspection.
   - This statement can be repeated at each inspection up to the case when first cracks are found.
   - In the case where cracks are found, there is sufficient time left to react by replacing the members or the total bridge.

Justification of the procedure

The procedure explained above is based on the *J*-integral [6, 7] (Figure 7)

$$J = \int_{\Gamma} (w\delta_y - T)\frac{\delta_u}{\delta_x}\delta_s$$

where *w*=energy density, *T*=vector of stress, *u*=vector of displacement, G=integration path around the crack tip and d_s=element of the integration path.

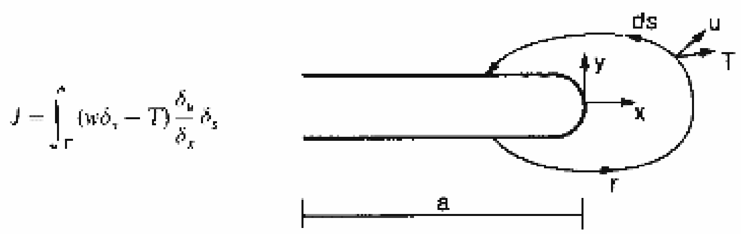


Figure 7 Definition of the integration path for a *J*-integral calculation.

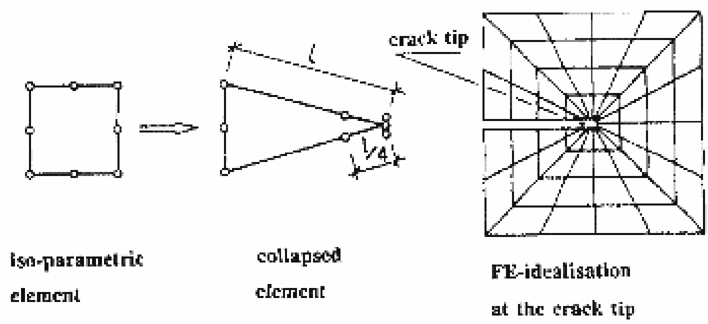


Figure 8 FE-elements and FE-grid for the calculation of J_{appl}.

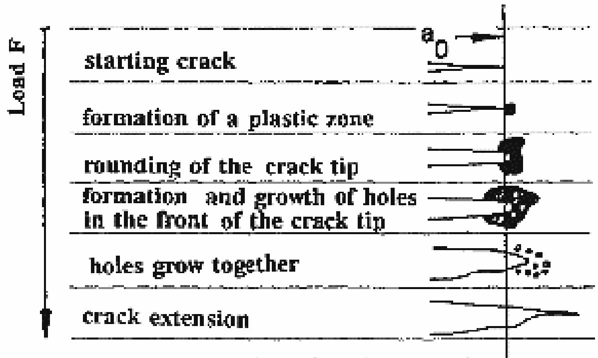


Figure 9 Procedure of crack propagation.

J_c, J_i [N/mm] temperature [-30°C]
21
42
57
45
16
34
51
100
111
29
74
16
40
233
31
154
63
104
127
81
98
61
69
129
70
45
135
40
27
145
$\bar{J}_c$ = 76.3 N/mm
σ_J = 50.25 N/mm

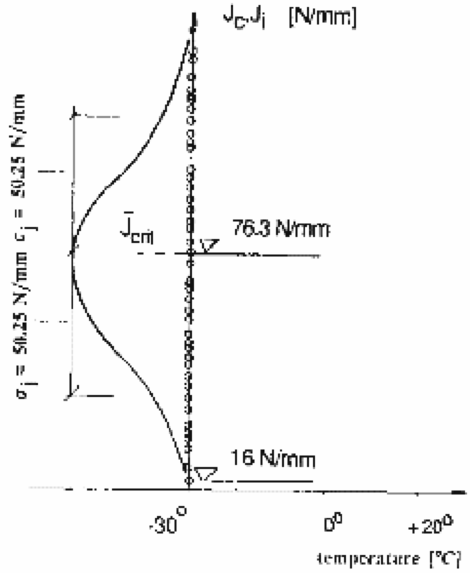


Figure 10 Critical material resistance values J_{crit} obtained from small scale test specimens.

The crack driving force in terms of the *J*-integral J_{appl} can be calculated by FE-analysis with a grid of collapsed iso-parametrics elements (Figure 8). The material toughness in terms of the *J*-integral J_{crit} can be evaluated from the fracture mechanics test specimen (see Figure 5) and can be interpreted as the energy which leads to crack tip opening just before crack extension (Figure 9). This represents a conservative assumption for the ultimate limit state, because it neglects the reserves that may be produced by stable crack extension after initiation.

The critical material resistance values J_{crit} obtained from a small scale test specimen taken from the tension chord and the struts and tested at a temperature T=–30°C are shown in Figure 10, the lowest value found was . $J_{crit} = 16$ N/mm

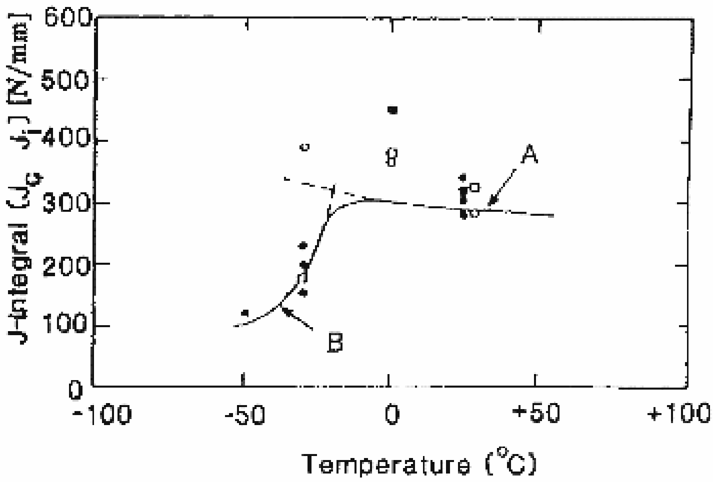


Figure 11 *J*-curve versus temperature (*A*=start of stable (J_i) crack extension values; B=start of unstable (J_c) crack extension values).

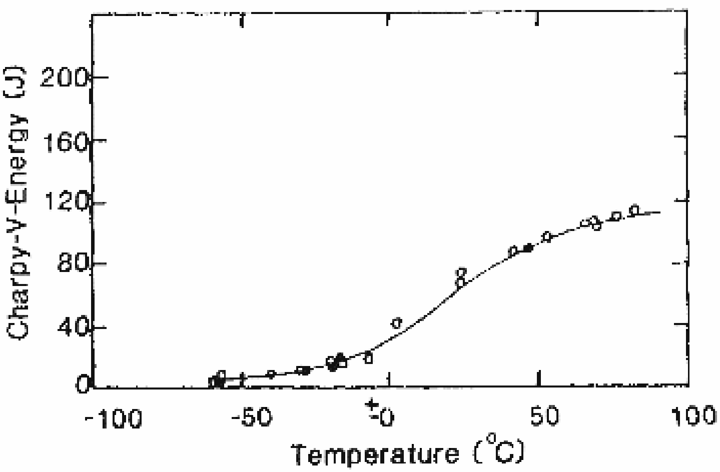


Figure 12 Charpy-V-energy versus temperature.

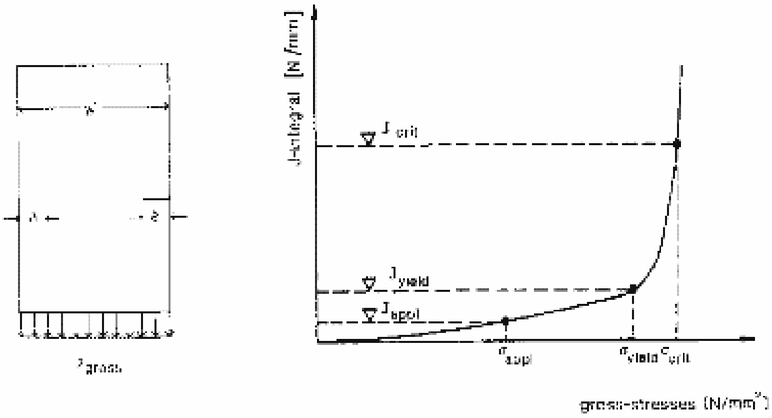


Figure 13 J_{appl} curve.

The $\frac{1}{2}$CT-10 samples allow the *J*-integral transition curve to be determined (Figure 11). There is no correlation to the Chargy-V-energy versus temperature curve (Figure 12) which for safety assessment can only be used qualitatively.

For a given structural member and a given crack location and crack size the curve of J_{appl} versus the applied gross stresses may be calculated based on the true stress-strain curve of the material coming out of the tension test (Figure 13). On this J_{appl}-curve J_{yield}, where the net section yielding is reached, and J_{crit}, where crack extension starts, are particular values. The case of brittle failure occurs if $J_{crit} < J_{yield}$, else net section yielding will occur before fracture is expected.

The reliability of the prediction of the ultimate resistance of structural members by the *J*-integral method has been proved by a series of justification tests with big plates [8, 9].

Application and results of the procedure

Tension chord

Figure 14 demonstrates the cross section of the tension chord close to the supports of the bridge and the model for the fracture mechanics assessment of this chord. The cross section of the other tension chords of the bridge are not critical in view of cracks because they have three or more lamellae in the flange and in the web.

In assuming that, according to test results, fatigue cracks would initiate in the parent metal under the flange of the angles starting from the hole of the rivets, a starting crack length is considered such that a detectable crack size of 5 mm comes out of the flanges of the angles. This assumption leads to a detectable starting crack size of *a*=155 mm (Figure 15).

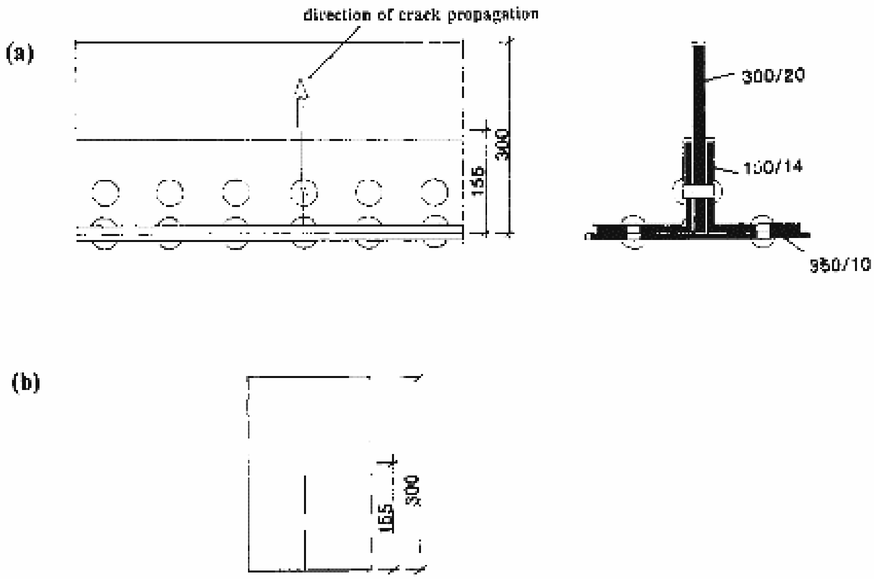


Figure 14 (a) Cross section of the tension tie. (b) Fracture mechanical model of the web lamella.

For the given true stress-strain curve the J_{appl}-curve was calculated as plotted in Figure 15. The relevant load case was represented by a temperature of -30°C which was combined with the applied stress due to self-weight, full traffic loads, residual stresses and all other restraints. This gives for the case of brittle fracture a total gross stress σ_{appl}=107N/mm². For this applied stress, the calculated *J*-integral J_{appl} is greater than the minimum *J*-integral of the material J_{crit}=16N/mm.

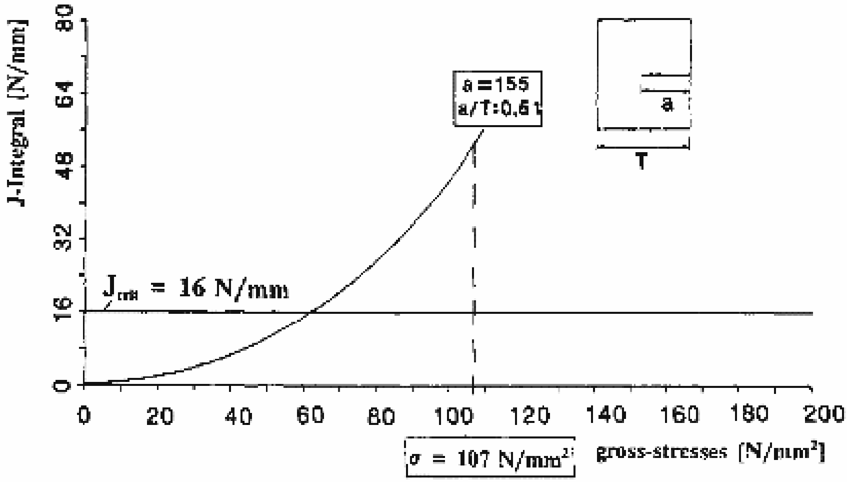


Figure 15 J_{appl} curve for the fracture mechanical model of the tension tie.

This safety check leads to the following statement: the cross section of the tension chord is not able to tolerate a detectable crack size. There is no possibility of detecting the crack before the tension tie will fail. If a crack in the tension chord occurs, the tension chord would rupture without prewarning by yielding because the failure mode of the tension chord is identified as 'brittle'. The conclusion of this safety check of the tension chords close to the supports of the bridge is that the cross section of the critical tension chords has to be strengthened with tough material before the next cold season.

Struts

The cross section of the struts and two alternative models (I, II) for the fracture mechanics assessment of the struts are shown in Figure 16. In assuming that fatigue cracks initiate in the parent metal under the heads of the rivets [10], a starting crack length is considered which represents the rivet hole plus two cracks at both sides perpendicular to the applied stresses, that have reached the edge of the rivet head (a=23 mm) (Figure 17). For the given true stress-strain curve the J_{appl}-curves for models I and II were calculated as plotted in Figure 18 and model I was chosen as relevant.

In Figure 19 several J_{appl}-curves are given starting from the basic curve for a=23 mm up to a=33 mm. The relevant load case was represented by a temperature of –30°C which was combined with the stress due to self-weight, full traffic residual stresses and all other restraints. This gives a total gross stress of σ_{res}=108.3N/mm^2.

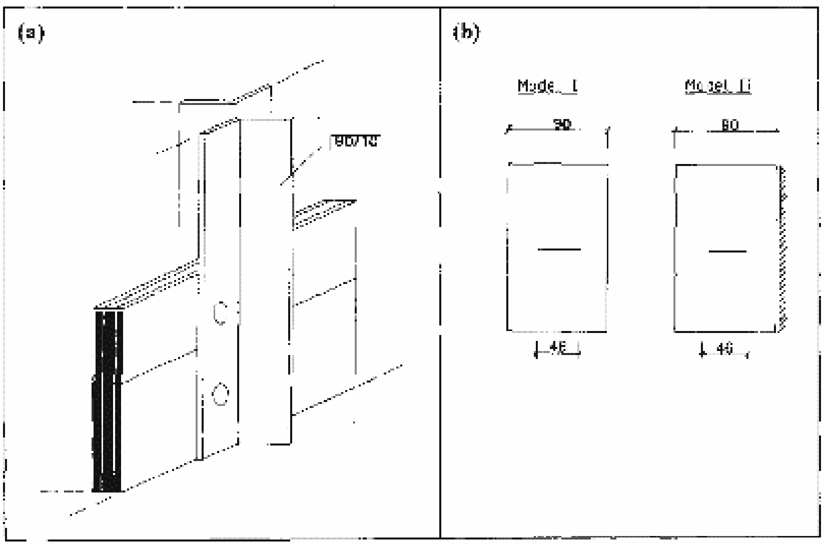


Figure 16 (a) Cross section of the hanger, (b) Two alternative models for the fracture mechanical assessment.

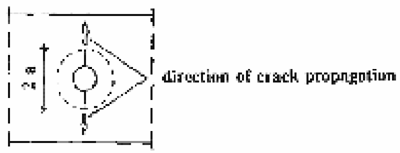


Figure 17 Definition of the starting crack length of the hanger.

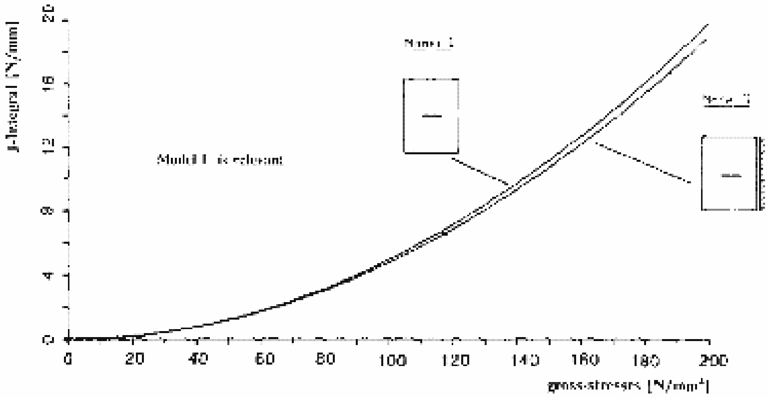


Figure 18 J_{appl}-curves for the fracture mechanical models I and II.

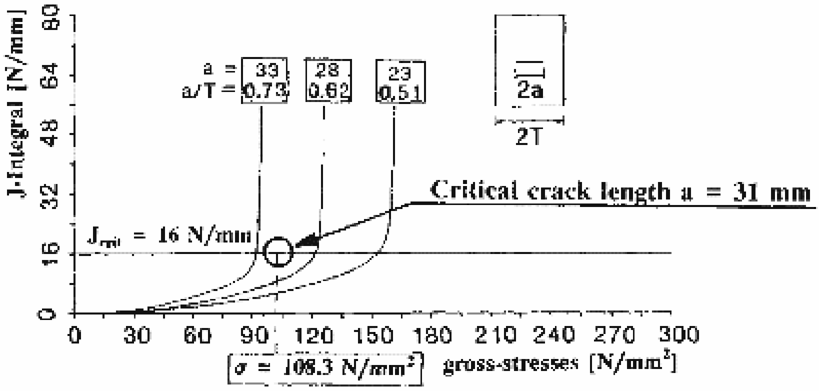


Figure 19 Determination of the critical crack size for the actual load situation.

The safety check $J_{appl} \leqslant J_{crit}$ for the strut (Figure 19) shows that for the minimum material value of $J_{crit\ I} = 16$N/mm as assumed in the determination of the stress state, the brittle failure mode is relevant and the tolerable crack growth length is a=31 mm, i.e. 8 mm crack growth beyond the starting crack size.

The crack propagation calculation is based on calculated traffic stresses of $\Delta\sigma$=69.3N/mm^2 or ΔK=710N/mm$^{3/2}$ with a number of 1360 cycles per day. The included dynamic impact factor is φ=1.21 as defined in the railway

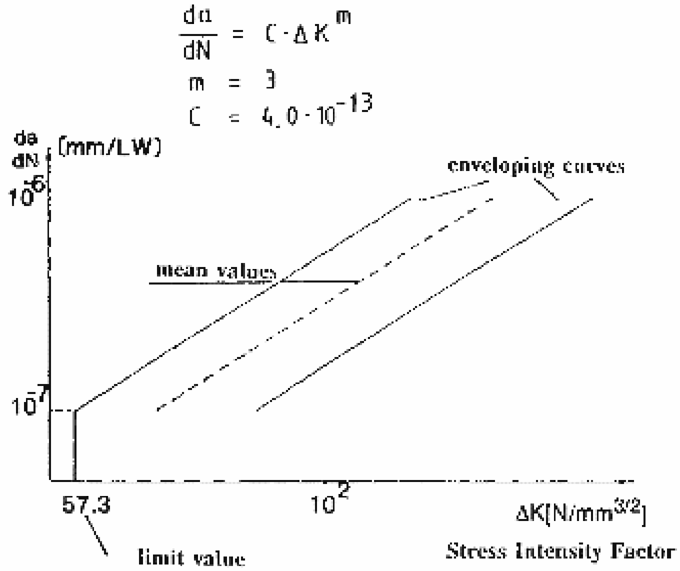


Figure 20 Determination of the crack propagation.

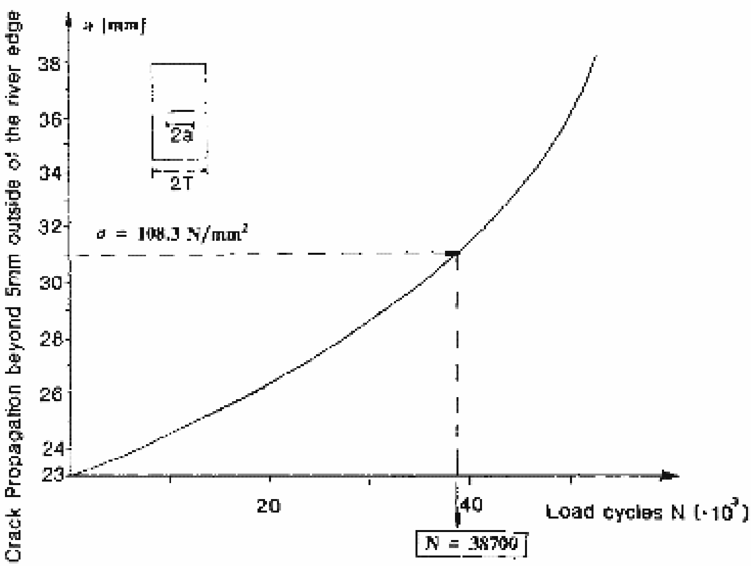


Figure 21 Crack propagation curve for the fracture mechanical model I of the hanger.

specifications which were valid for the Anhalter-Bahn-Bridge. The 'Paris' coefficients were taken as given in Figure 20.

The calculation of the crack extension yielded the curve shown in Figure 21. The calculated number of load cycles between the starting crack size and the critical crack size was calculated to be N=38 700. Based on an applied number of 1360 cycles per day, the safe residual life time of a cracked strut is

38700/1360=28 days. This time is too short for economic intervals of inspection.

Hence the struts of the bridge have to be strengthened with tough material or be replaced before the next cold season.

The final results from all checks including the aforementioned fracture mechanics assessment was:

1. The bridge complies with the present design codes but is not fit for use with sufficient safety when cracks will occur.
2. The critical member of the tension chords close to the supports and the struts have to be strengthened with tough material or replaced before the next cold season.

Conclusions

A fracture mechanics based procedure for the determination of the safety and the residual life of old steel bridges is presented, that avoids the uncertainties of predictions that are linked to *S-N*-curves. The method has already been applied to railway and roadway bridges, to guyed masts and rotors and is considered as a useful tool together with appropiate enginnering judgements. At present the method is being simplified and made more operational in order to be standardized in a code that could be used for the safety assessment of old steel bridges.

The conclusion for the design of new bridges could be that new bridges should fulfil the requirement of sufficient robustness, defined by tolerable cracks of sufficient size, that allow early detection and an additional crack growth time, that is compatible with inspection intervals and the time provisions for repair or replacements.

Acknowledgement

The works described in this paper were mainly carried out by the BVG in Berlin, Dr.-Ing. R.Hubo, Institute of Ferrous Metallurgy, Technical University Aachen and Dipl.-Ing. H.Eisel and Dipl.-Ing. W.Hensen, Institute of Steel Structures, Technical University Aachen. The fruitful cooperation between the BVG in Berlin and the institutes of the Technical University Aachen is highly appreciated. The construction works in consequence of the expertise were carried by Krupp Stahlbau, Berlin.

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The contractor's role

P.R.B.DAVIES

Introduction

The contractor's role in the building process for any structure is crucial to its success from time, cost and quality considerations. Design life of a structure is probably linked more closely with 'quality' than 'time' or 'cost'. Some observers may argue that a contractor's interest in providing high quality construction is always tempered by the commercial pressures of winning work, carrying it out and making a return for the company shareholders.

This is a somewhat negative view. The majority of contractors, certainly the major ones, have a lot of experience and hard won wisdom to bring to the construction team for a new structure.

This paper looks briefly at the concept of a specified life for structures, highlights the importance of drawing on experience and giving due attention to provision for maintenance. Finally the 'quality' consideration is discussed and some questions raised regarding the contractor's contribution to the design life concept.

Definitions

Design life is essentially longevity (but not always). Life is the ability to cope with original design parameters

a. Without maintenance
b. With maintenance either planned or as necessary.

A specified life

To build structures for a notional or specified design life is, it seems, entirely possible, though the client may well have to pay a premium to achieve such an objective. The premium may arise from

- A longer design process

- Specification of higher quality materials, e.g. stainless steel reinforcement
- A larger structure to give ease of maintenance access
- Construction requiring high quality workmanship with intensive quality control/quality assurance
- The structure having planned maintenance requirements of varying degrees through its design life

This list is general and purposely omits warranties and guarantees.

The notional life for building structures in the private sector is currently 30–40 years for the frame and foundations, with some developers taking the view that the building will need to be reclad after 20 years. Major internal refits are being considered on an 8–10 years cycle. Both the recladding and the internal refits are seen as necessary to keep the building 'fashionable' and technologically up to date rather than to replace 'worn out' life expired components.

Bridges and highway structures are not subject to the pressures of being fashionable and technologically up to date during their life. Commonly 120 years is regarded as the design life for a highway bridge. Certain components may be less, e.g. bridge bearings.

Learning from previous work

The majority of the United Kingdom's new highway structures have been designed and constructed over the past 35 years, mainly as a result of the enormous growth in freight and passenger vehicles and the need to provide strategic high speed roads for them to run on. A number of the early bridges that have done 25–35 years service will be known to have performed well whilst some have suffered problems of varying sorts.

Some questions therefore that we should ask:

- Have all the lessons from problem highway structures been reported to and digested by county and national road design authorities?
- Have the observations and lessons from looking at the good and the problem bridges been discussed with the design and contracting fraternity?
- Have the conclusions drawn been incorporated into current Department of Transport approved practice guides? For construction work the Roads and Bridges Specification is one such guide.
- Has an analysis of the problems found in the various types of highway structure been undertaken to allocate responsibility to (very simply) design or construction?

Within the private building sector, analysis of problems experienced after construction of the building has indicated that some 63% are caused by design, 11% due to materials and 26% due to construction workmanship. It is however too simple to say that the major risk therefore lies in design.

The essential point that we need to look at and discuss is: Have we learnt from past mistakes and applied the knowledge gained to designing, constructing and maintaining tomorrow's structures? If not, why not? Is it a failure (or contractual shyness) on the part of engineers to communicate? Ronan Point, Milford Haven Bridge and West Gate bridge were major calamities; the lessons learnt however quickly became enshrined in building or other regulations.

Maintenance of structures

All structures need to be maintained to some degree. The amount of maintenance attention is dependent on the use, complexity and location of the structure.

Historically on many types of structure, accessibility for inspection and maintenance work was given little thought. It is of the utmost importance that the designer considers, as his work progresses, the need to provide good accessibility to all parts of the structure which will require inspection and maintenance. The need for maintenance should also be considered as a design parameter.

Materials selection plays an important role in deciding future maintenance requirements and a risk study/cost benefit analysis is a helpful tool in reaching decisions. The designer may well justify the use of high cost reliable materials for high risk parts of a structure.

A further version of 'cost benefit' analysis is life cycle costing of the structure.

Limited life components, if they cannot be avoided in structures, should be designed into a location such that they may be replaced in a straightforward manner. If the structure has to be jacked up, e.g. to replace a bearing, the designer has a duty to incorporate strengthened points to which the jacks can be applied.

Quality and design life

Design life of a structure is undoubtedly strongly linked to 'quality' in the time, cost and quality triangle. Quality of design and details are important. Quality of the materials and the workmanship specified by the designer but obtained and managed by the contractor, are of paramount importance to the ability of the structure to meet its design life requirements.

Materials in themselves have arguably produced more problems than workmanship, e.g. alkali silica reaction, in the last 25 years. As engineers we hope we have overcome all these materials problems but undoubtedly there is something else lurking round the corner waiting to give us a nasty surprise.

Historically the role of the traditional contractor has been seen by many on the professional side of the industry as 'the guy who comes along with men and machines and, for a price against a set of specifications and drawings offers to build the structure (bridge, dock, building, whatever)' that is required.

Is this the right way to approach awarding the project, which may be a 120 year design life structure? Does the client get the best quality by this route? Some people will say 'Yes it is the best route, it has always worked well'. The auction or tendering method for selecting a contractor needs to be thoroughly reviewed. Contractors' experience is an asset that is not fully appreciated and therefore utilized for the benefit of a project by many clients and their design teams.

The contractor's role in the way ahead is very much to be a strong supportive member of the full project team. The management contracting approach for building a project is founded on the principle of the contractor being appointed on a fee basis (like the design team) and using his skills and knowledge to the client's benefit. The contractor's comment on the design and its details should be a matter of course even, undertaken not in the final 2–3 weeks to start of work on site, but almost from the outset of the design process.

What is proposed, it is realized, will pose problems in the contractor selection process, particularly perhaps for public sector works. These are challenges that have to be overcome.

Finally, the application of quality assurance and quality and quality control techniques to formal British Standards systems is gathering pace within the construction industry. Due to the industry's immense fragmentation there are learning curve problems to be overcome with QA/QC techniques. The successful application of QA/QC to the construction of a structure must surely be favourable for its longevity or design life.

The contribution of research to the development of design for durability

A.W.BEEBY

The future impact of research and development (R&D) on the way ahead for the treatment of design lives of structures is, of course, a subject for speculation rather than a factual presentation. To get some idea of where R&D might contribute and, possibly, the conditions for it being able to contribute significantly at all, it will be helpful to look back over the last 15 years to see what R&D has contributed so far in the reinforced concrete field.

In shear weight of papers produced, the contribution of R&D to the issue of durability and hence design life has been huge (see Figure 1 which indicates the number of publications on durability of concrete listed in the BCA data base over the last 14 years). Clearly the R&D community reacts rapidly and efficiently to the appearance of a problem. The number of papers being published annually by 1982 had doubled compared with 1976 and this number had doubled again by 1986.

The real question, however, is not the weight of paper that has been produced but the contribution that this work has made to our ability to deal with questions of service life.

As far as codes and standards were concerned, durability was not given much priority at the beginning of the period covered by Figure 1. However, CP110, published in 1972 included tables of minimum covers, concrete grades, cement contents and water/cement ratios for different exposure conditions. These tables indicated a requirement to increase the cover and the concrete quality as the environment became more aggressive. These tables in CP110 were largely developed from tables in CP116 published in 1965. Thus, at the beginning of the period, it was well understood in practice that durability could be ensured by the choice of an appropriate cover and concrete quality. Concrete quality was believed, in academic circles, to be largely a function of water/cement ratio but, from the practical point of view, since this could not be tested on site, the rules were dominantly written in terms of strength and cement content. These limits were assumed to ensure that, in practice, the water/cement ratio remained adequately low. The belief in the importance of water/cement ratio probably stems from the work of Powers [1] carried out on cement pastes in the 1950s which produced the relationship between permeability and water/cement ratio shown in Figure 2.

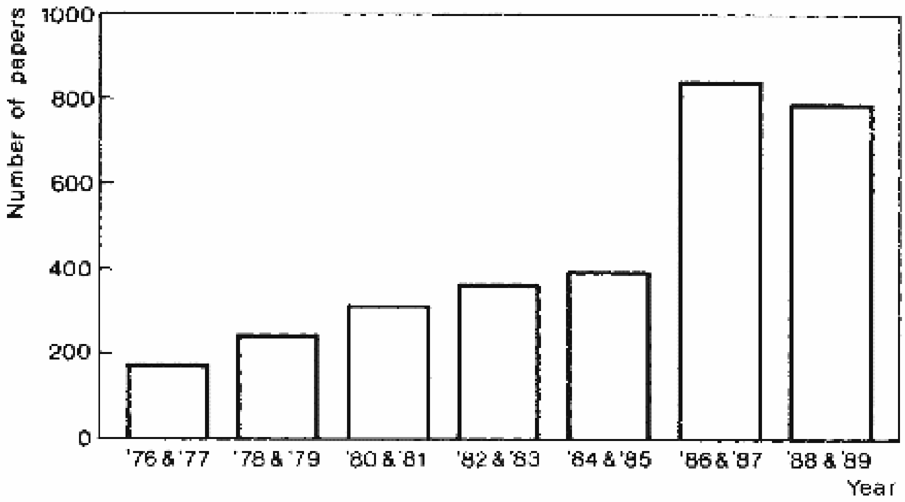

Figure 1 Number of papers published on durability of concrete over the period 1976–1989.

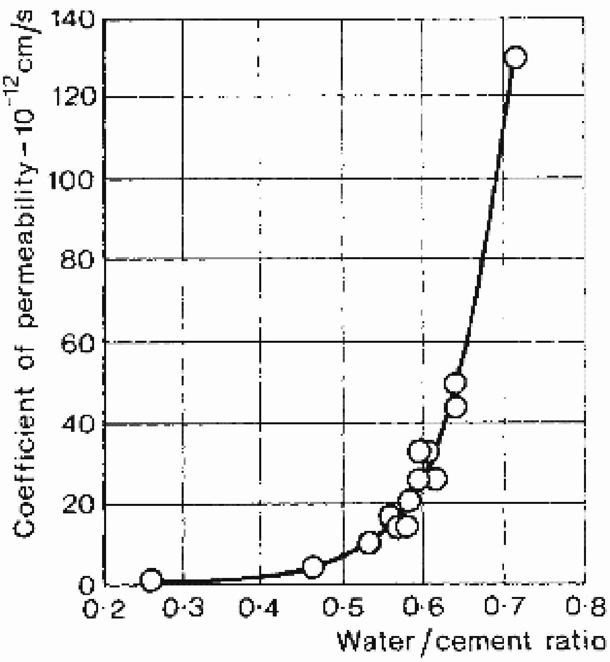

Figure 2 Relationship between permeability and water/cement ratio for mature cement pastes [1] (93% of cement hydrated).

How has 15 years of concentrated research effort changed this situation? The answer, as far as design is concerned, has to be that the situation is exactly the same. BS 8110, EC2 and, indeed, the CEB 1990 Model Code all include tables which reflect exactly the same understanding of durability, albeit with the figures and presentation adjusted here and there. Any publication aiming to give a general treatment of durability is still likely to feature Power's [1] curve prominently. How is it that research has apparently achieved so little despite the high level of interest shown in the problem over the last 15 years and the demonstrably huge research effort? If we can understand this, we may be able to see what is needed to ensure that future research is more effective, if effectiveness really can be equated with impact on practice.

The first point to note is that, over the period in question and, indeed, for many years previous to it, the science of durability has been in a phase of 'normal science' as defined by Kuhn [2]. Kuhn identified two basic cyclic phases through which a science passes. Most of the time a science is in the normal state but these periods of normal science are interspersed with short periods of revolution. In the normal state, the basic model underpinning the science is not questioned and scientific effort is concentrated on fleshing out this model. In the revolutionary phase, the model itself is discarded and replaced by a new model. The revolution opens up whole new areas for research and a science just after such a revolution is an exciting field to be in. The oddity about the science of durability is that the surge in interest in it is not related to any revolutionary change in our model of the problem but to the development of awareness within the construction industry of its importance and hence an increased availability of research funds in this area.

The second point to note is, again, related to the natures of science and engineering. Science is founded on measurement: it basically deals with matters which can be reduced to numbers. Engineering adopts a quantitative approach where it can, but recognizes that, in the current state of knowledge, this is not always possible. Codes contain many areas where rules which are considered to lead to results which will 'normally be satisfactory' are given. What 'satisfactory' is, is not defined but what it really means is that experience has shown that when the rule in question is applied, there are few complaints about the result. Such prescriptive provisions are not ideal but the job of engineers is to produce the best results they can with the limited information available, whatever its nature. Where quantitative information does not exist, rules based on qualitative experience provide a way forward. One of the unfortunate consequences of such rules is that they provide no means by which research (or science) can be brought to bear to improve them. Research can really only start to influence design when a quantitative framework for design has been accepted. The history of the development of design can be seen to be very closely related to the stages in which the requirements in successive areas of behaviour have been quantified and research has been enabled to act.

One can see this process at work clearly in the concrete design field in the United Kingdom. Prior to the development of limit state design, research was almost exclusively concerned with strength which was the only aspect of design where the criteria were defined quantitatively (defined loads and safety factors). This resulted in a steady improvement of our understanding of, and ability to predict, ultimate strengths due to bending, shear, torsion, bond etc. and also enabled developments such as plastic design, ductility and redistribution. Minimal research was carried out on serviceability because there was no way in which the results could be used. The UK code prior to 1972, CP 114, simply said that cracking would be satisfactory if the steel stress was below 230 N/mm^2 and that deflections would be satisfactory if the span/depth ratios were met. In the absence of any definition of 'satisfactory' there could be no link with quantitative research. Limit state design, as developed by the CEB and introduced into the United Kingdom first in CP 110 in 1972, with its definition of serviceability criteria changed this and crack width and deflection prediction became major areas of research activity.

Durability is still in the prescriptive phase in design codes; durability will be considered satisfactory if a certain specified cover and concrete quality is used. It hardly matters how much research is done, it cannot have an impact on design until a quantitative framework for durability design is accepted. This colloquium is proof that attempts are being made to develop such a framework, based on the concept of a specified design life. Again, it can be argued that the upsurge in research on durability arose from public interest leading to money being available rather than from any clear picture of how the research could be used to improve design.

It is not that the research carried out on durability has been of low quality or has been entirely valueless, but it has not been able to influence design. As a result of the work, we understand all the deterioration mechanisms much better than was the case 15 years ago. In many areas we are close to the situation where reliable mathematical modelling of some of these processes would be possible. All that is required is that the bridge of an effective quantitative design format should be built and the research community can then very rapidly provide the means to develop a sound quantitative design process.

In all that has been said above, the D of R&D has been ignored. Over recent years development has been more productive than research as far as practical assistance to construction is concerned. As examples, one can mention the development of epoxy-coated bars and the development of cathodic protection for concrete structures to a level where it is a practical option for maintenance. It can be argued, however, that even in this field, the lack of a quantitative design approach has inhibited developments. The major practical developments which have had a significant impact have been those which can be expected to completely avoid degradation. Such solutions can be considered logically and their economies assessed within the current design system. For example stainless steel does not corrode and hence the rules aimed at avoiding corrosion by providing suitable cover, etc. may be ignored. Epoxy-coated bars and cathodic protection aim to completely avoid or stop corrosion and thus fit into this category, though one may argue about whether they are likely to meet this aim. Developments which aim simply at improving durability are, however, much more difficult to justify in practice in the absence of any quantitative measure by which this improvement can be judged. The impetus to develop them is therefore reduced.

It has been argued that the lack of a quantitative approach to durability design has severely curtailed the ability of research to influence practice in this field. It might therefore be concluded that, once this is rectified by the acceptance of an approach based on design life, all our problems will be solved. This is not necessarily so, and a further criterion must be met before research can properly influence design. This is that the criteria defining satisfactory performance must not merely be quantitative, they must also incorporate usefully testable quantities.

For research to have an influence on design it must be possible for research work carried out in laboratories to be applied to practical situations. This is the case, for example, for design for ultimate strength. Research may be used either to develop design methods or to develop specific solutions to particular design problems. In each of these cases research can be used because the requirement of the design is that it should have a specified safety factor against failure. This is a quantitative requirement and one that can be tested under laboratory conditions. Exactly the same situation holds for research into serviceability; the requirements for crack width or deflection are testable under laboratory conditions and hence research may be used to develop design methods which will reliably satisfy these criteria. The testability of the predictions of a hypothesis is what sets science apart from mere speculation and this must be as true of research related to structural design as it is in any other field.

The question we now have to answer is: is the criterion that a structure should meet its functional requirements over its full design life a testable statement?

What is being asked is that the structure will still have a satisfactory safety factor, will not have excessive cracks or have deflected excessively in some 60 or more years time. In theory this is testable. We can set up tests which we can hope our children, grandchildren or great grandchildren may inspect in 60 or 120 years time and they will then be able to say whether or not the result was satisfactory. It is, however, not practically or usefully testable. Not only is it not practically possible to wait for that length of time for the results of an experiment but the results would then be irrelevant due to changes in technology over the intervening period. This suggests that design life concepts will not lead to the forging of any productive link between research and design for durability.

The picture for the future development of a sound, logical, scientifically based approach to design for durability seems rather black from the above discussion. It is not necessarily as black as it seems. If we wish to develop a design approach to durability which will enable research to play a significant role then we need to consider whether the problem can be formulated in a different way. In doing this we should not be blinded by the intellectual appeal of the concept of design life. Just to show that alternative approaches can be formulated, I will mention two possibilities. I am sure that ingenuity could lead to many more and possibly better suggestions.

Maintenance cost factor approach

The treatment of durability is often represented by diagrams similar to Figure 3. The structure is allowed to deteriorate to some limit at which time remedial action is taken to return its state to the starting level. The process is then repeated until the design life is met. In fact, the design life will be seen to be irrelevant since there is nothing stopping further cycles of remedial action being carried out and the life being extended. The 'life' in this model simply continues as long as the owner is prepared to continue the 'life support'. The durability of the structure could be defined as the ratio of the average annual cost of maintaining the performance of the structure within the acceptable performance band to the first cost of the structure. An appropriate target figure for this could be set at the design stage. An approach of this type is as clearly relevant to structures such as the Forth Bridge, whose life will continue as long as the painting is kept up as the concept of design life is irrelevant to such a structure.

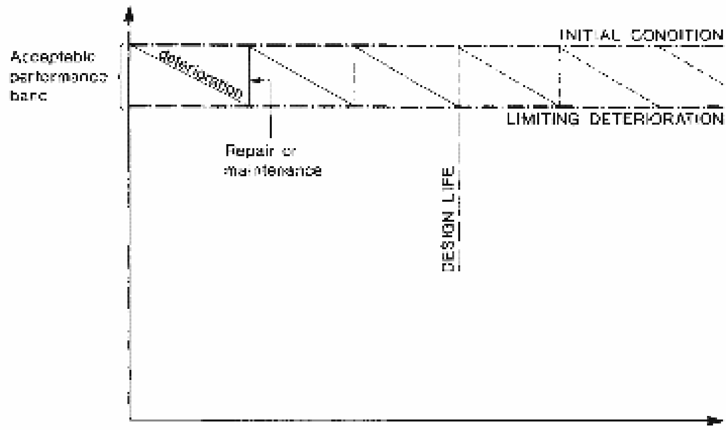

Figure 3 Schematic illustration of maintenance process.

Durability reference period on 'rate of degradation' approach

This approach is a possibility where a continuous degradation process is occurring, the progress of which can be measured. The idea is that, instead of requiring that the structure has not deteriorated to an unacceptable degree by the end of its design life, a much shorter period is defined (say 10 years) and a limit is placed on the progress of deterioration within this time. As an example, one might specify that carbonation should not have penetrated more than 40% of the cover in the first 10 years of life. In principle, this approach is aiming to assess and put limits on the rate of deterioration, which may be measurable, rather than the total time taken to reach a certain amount of deterioration which, as discussed above, is not usefully testable.

Conclusions

This brief paper highlights the very limited influence on design methods which research has so far had. It is suggested that this has arisen due to the nature of current design methods, which are prescriptive. It is also suggested that the development of design methods based on the concept of design life will not improve the ability of research to assist since the concept does not lead to usefully testable predictions. It is argued that, if research and development is to be constructively used in the development of design methods for durability, then some alternative concept to design life is necessary. Such alternatives are possible and equally logical.

In my view, our most pressing task in the field of durability research is the exploration of possible frameworks for design for durability which will permit and encourage the development of rational design methods. I hope that this seminar will serve as a catalyst to such developments.

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Design life of buildings: client expectations

J.G.BURNS

Introduction

Client expectations regarding 'design life' will vary with the type and use of structures that are the subject of engineering design. These expectations are most easily divided between public and private clients, and as Skidmore, Owings and Merrill's projects are primarily with the latter, this paper will be concerned with building projects for developers.

A few introductory remarks about these clients will be discussed first to better define the scope of this paper. Bridges and infrastructure works are primarily commissioned by public bodies and as such, they often have well-defined client briefs with respect to design life. Buildings, on the other hand, are primarily built by private clients that do not have well-defined expectations of a structural design life, and are often more concerned about the design life of other building components, such as building services and exterior wall. Developers typically have an investment period used to calculate the financial return on a property improvement, and must include the replacement cost of elements that have design lives less than this assumed period.

The question of adaptability seems to be a common thread to client expectations as it relates to buildings. These not only include renovations and adaptive re-use projects, but also new buildings. Why are relatively young buildings demolished prior to reaching their design life, while we renovate and adapt old buildings that have long since passed their anticipated lifespan? A building is usually demolished because it no longer serves its intended purpose, and cannot be economically renovated to serve another use.

Maintenance and inspection are expectations clients should have, and successful developers are aware of the importance continuing care of buildings has on the design life. This is not only an issue of physical deterioration of an asset, but also the need to maintain the aesthetic image of the building in order to maintain its value.

Design life of building structures

Most building developers do not have a clear notion of how long a building structure should be designed for, except that the minimum desired is the investment life of a building. With few exceptions, building structures should, when properly maintained, last forever. This is because most building structures are protected from environmental deterioration by their exterior envelope. If the envelope or skin is not maintained, a new exterior wall or roof may be needed earlier than expected, but the structure should be serviceable unless major deterioration occurs.

There are some exceptions to this 'buildings last forever' expectation, that building clients now are aware of. The most notorious are car park structures, sharing many of the problems of environmental deterioration and premature demolition associated with infrastructure projects. Clients are now more aware of the need for special design considerations when structures are exposed, and yet pressures to reduce initial building costs at the expense of future maintenance are ever present. Once again, successful developers are taking the lead in adding value to a building by reducing anticipated long term repair costs. When structures are exposed to the environment, clients now expect a life-cycle analysis, to weigh alternatives in terms of first cost and maintenance.

Another reason that building structures do not last forever, is that they collapse, or otherwise become unserviceable as a result of wind storm, earthquake or foundation subsidence. Clients expect that buildings meet the building regulations in this regard. Fortunately most codes have a minimum of one in fifty year return period for severe events, and this covers most investment periods that clients desire. Many clients will not spend more than minimum code requirements to obtain longer lifespans. Equally, additional money would not be spent on 'ductility' or on 'tying a building together' that some building regulations require. On the other hand, clients expect to be protected by following these regulations. And so these concepts that add structural capacity to resist the extraordinary event contribute to the expectation that structures should last forever. Unfortunately, many regions of the world have minimum acceptable codes that are lacking with respect to residual capacity. The predominant reason that building structures are demolished is not found in the environmental pressures described above. Buildings are replaced because they no longer serve the use they had been designed for, and a new use can more economically be accommodated in a new building than the existing. Many clients did not anticipate changes in technology that have made their office buildings obsolete in only a few years, with the advent of the electronic office. Other clients have purpose-built office blocks with inflexible plans that cannot now easily be leased to other tenants.

The design life of a structure is thus dependent on more than resisting heat and cold, water and chemicals, wind and earthquake; if it is adaptable, it is likely to have a longer life. Adaptability will be discussed further, but first client expectations of the design life of other building components are addressed.

Design life of building components

Unlike building structures, which clients expect to last forever, building components are often replaced several times over the life of a building. Exterior walls are covered with new skins, or completely replaced as architecture and the property market go through different building styles. Building services are upgraded or replaced to improve comfort, safety, energy efficiency and to accommodate new tenants. Elevators are very costly investments, especially for high-rise buildings, however they too must eventually be replaced as does all moving equipment. Internal walls are the most changeable, as they respond to continuing changes in use, at least in office buildings. One developer believes that drywall partitions are moved on average every 10 months.

With all of the above components so changeable, what does the client expect of the structural framework that supports these elements? The clients would like a structure that allows for flexibility and adaptability, but often are unwilling to spend additional money to achieve this in the initial design. This leads nicely into a discussion of adaptability.

Adaptability

Buildings that can accommodate change are more likely to have a design life of greater length than inflexible structures. Warehouses in the docklands built in the 19th century are transformed into offices and flats, while city office blocks built in the 1960s are being removed for electronic offices of the future. Clients are beginning to expect adaptability to be considered in the structural design of a building, given the greater uncertainty associated with the future property market.

Aspects that allow structures to more easily accommodate change include both spacial and loading flexibility. Long-span framing systems with uninterrupted space are better in this regard than bearing wall buildings and small structural bays. Adequate floor-to-floor height is essential to allow for future revisions to building services.

Sufficient loading is more difficult to quantify, as a balance must be reached between the minimum requirements of the building regulations and the probable maximum load that may occur over the life of a structure. A client's decision on an appropriate floor loading for a particular building is related to the ease that the floor system may be upgraded in the event of unusual future loads. Steel buildings are better in this regard than concrete buildings, and a client may opt for a steel building with reduced floor loading and sufficient capacity in connections, columns and foundations for future increases. Localized increases are easily accommodated with welded cover plates. Alternatively, a client may choose increased floor loading in selected structural bays for future tenants to locate extraordinary loading.

Adaptability is not solely a structural design issue. The full development team contributes to a successful scheme that integrates structure, planning and services, and involves the collaboration of the client, architect and engineers.

Maintenance and inspection

Clients are aware that buildings require continuing care to maintain the structure within its intended capacity and utility. Clients do need assistance from the design team on an appropriate maintenance and inspection programme. Exposed structures are particularly susceptible to corrosion and deterioration, as are structures that become exposed to moisture or other attack when protection systems break down. Buildings with passive systems for such reasons as vibration or earthquake control, or active systems for human comfort with respect to tall building acceleration require greater vigilance on the part of the future building operators.

Building structures are no different than any other building system or component that requires regular and expert care for continued service. The structural engineer must dispel the client's expectation that his structure will last forever without this care.

Case studies

Canary Wharf in the London Docklands has an excellent example of design life provision in the foundation systems utilized to support buildings over the existing dock water. Piles were designed for 125 years relative to corrosion. This value was reached in discussions with the district surveyor.

Building 11 at Broadgate in London has an exposed steel frame that serves as its dominant architectural expression. The epoxy-coating system utilized on the steel should last indefinitely with periodic inspection and repair. The client anticipates inspections at four year intervals, with touch-up paint where needed. The client has allowed for a complete repainting of the frame at twelve year intervals, primarily for cosmetic reasons rather than for protection.

The Economist Building in London required a major refurbishment in order for its owners, a newspaper, to comfortably enter the age of electronic publishing. In the course of upgrading the building services, the exterior wall was also cleaned and repaired. The exterior exposed concrete was investigated for chloride contamination and carbonation. A concrete coating was applied to arrest possible future deterioration. The client is aware that he must inspect the concrete periodically, and recoat the concrete in approximately 20 years. This building is an example of a 1960s building that was able to accommodate new technology, albeit with some difficulty, considering the restricted floor-to-floor height and services zones. In this case the client was able to extend the life of the building by accommodating change and by proper maintenance and repair.

County Hall in London is the subject of a major renovation and rebuilding scheme. The listed Riverside building, built in 1910 and extended in 1930, is to be retained with major internal demolition and reconstruction works. The building, originally designed for local government use, is to accommodate a hotel, housing, retail and a business/conference centre. The building is supported primarily by bearing walls, making it difficult to utilize as multi-tenant office space. The remaining buildings, the North, South and Island blocks are to be demolished to allow the construction of more flexible office accommodation. This development provides an example of the lack of flexibility associated with a purpose built building that leads to demolition when its intended use changes.

Rowes Wharf in Boston is an example of a mixed-use project, including offices, hotel, housing and retail. A steel frame was utilized throughout for all of these different uses for its flexibility in accommodating the variety of requirements. This flexibility should allow the structure to survive for a long life, even if the mix of uses is changed in response to the property market. The car park, built in the 'up-down' method below the adjacent Boston Harbour, includes carefully considered anti-corrosion measures. The client decided to utilize epoxy-coated rebar in the concrete slabs as well as an epoxy traffic topping. This particular client developed a building for long term ownership and operation, and thus the maintenance and care that is essential to a long structural design life is one of this client's expectations.

Summary

Client expectations regarding design life of buildings are not so well defined. In talking with developers, there is a general belief that structures should last forever, when properly inspected and maintained. It is other building components such as services and enclosure that are likely to be replaced during the life of a building. The ability of a structure to accommodate change over its life contributes in a substantial way to how long a structure is likely to survive. Examples of the above are illustrated with selected case studies.

Acknowledgements

The author would like to thank the following individuals for taking time to discuss their expectations: Richard Griffiths, Olympia & York; Paul Lewis, Stanhope Properties; Patrick Breshan, The Economist; and James M.Becker, The Beacon Companies.

Expectation and role of the client (owner)

D.A.HOLLAND

Introduction

The session to which this paper is contributed is concerned with what happens next, the way ahead. In many respects it anticipates discussions yet to take place on earlier papers. Nevertheless, it attempts to set out a general framework of some likely indicators common to design life considerations for most structures.

Expectation, the present

Before looking to the future we must first take stock of the present to establish the context within which the expectation and role of the client (owner) can be assessed. Earlier sessions have examined the concept of design life, both theoretically and practically, the latter with particular reference to several industries (shipbuilding, aircraft, nuclear, offshore, building, etc.) which can be expected to meet the needs of a wide variety of clients. Despite the variety, those clients have at least one thing in common; each is prepared to make a significant investment in the purchase of a structure. Owners of the different structures are all seeking a return on their investment although the nature of the return may well differ between them. Typically, a private owner will be looking for a monetary return whereas a public owner will be seeking benefits in kind rather than cash. Notwithstanding the differences, the client's expectation of the return on his investment, his confidence in the future and the risk he is prepared to accept will dictate the expectation he will have of design life. Thus, owners will expect their investment to earn more than the interest rate on a loan; they will also expect their purchase to outlast the minimum period needed for the investment to break even at that anticipated rate of return.

If value for money in terms of a return on an investment is one sought after characteristic, safety must be another. Just as there are economic risks that a prudent client will wish to minimize so there are risks that can threaten the integrity of a structure that have to be guarded against as far as possible. Doing so brings with it a cost, the extent of which will depend in large measure on the client's expectation of the duration of the service life of his structure.

In theory there is a trade off between initial cost and design life. The longer the design life, the longer the return period available for more extreme stochastic events and the greater the probability of their occurrence. The longer the design life, the longer the period during which a structure is earning an 'income'. However, the additional reserves required to safeguard the structure against more extreme events will add to the initial cost and hence extend the period during which the structure must earn its keep if the initial investment is to be recovered.

Financial conditions are not the only constraints of relevance; practical considerations are just as important. Ideally, the purchase should continue in service throughout its design life. However, practicalities are such that in most cases some periods of reduced levels of service while repair and maintenance operations are undertaken will be essential if safety and serviceability are not to be put at risk. As well as fair wear and tear, other time dependent effects, particularly loading (whether man-made or natural), and in many respects, the response of materials to loads, may require the assessment of the integrity of a structure and, if necessary, its strengthening. The owner will want to keep the associated non-productive, non-earning periods to a minimum. In achieving that, a balance has to be struck between initial cost and the cost of subseqent maintenance. Decisions here will, to a large extent, dictate the form and material content of a particular structural solution.

The uncertainties associated with forecasting time dependent effects and their consequences are such that it is not possible for a structure to have a determinate lifespan. The closest that one can come is to a statement such as:

> This structure is designed to carry safely a certain number of loading events; according to the best forecasts available, that number of events will take place during the given design life of the structure. However, the forecasts are subject to wide variation which could lead to a design life in practice significantly longer or shorter than assumed.

Expectation, the future

If the present is characterized by uncertainty what of the future? Perhaps the best that can be said is that many doubts will remain! Undoubtedly new materials, particularly those based on plastics and glass or carbon fibres and improvements in the performance of traditional materials (steel and concrete will for the foreseeable future continue to be the principal raw materials for the great majority of structures) will enhance durability and reduce maintenance needs.

Greater knowledge of the way structures and their component materials respond to loads and to the attrition of time dependent phenomena together with the development of built-in monitoring systems will lead to the better assessment of the residual life of a structure. That, together with the appropriate deployment of improved repair and strengthening techniques, will result in better targeting of a maintenance effort more closely directed towards prolonging the life of a structure. The use of knowledge based computer systems, allied to sophisticated monitoring techniques, will enable more consistent and certainly better informed decision processes to be established.

Taken all together these various measures will reduce a number of the uncertainties associated with whole life costs and will provide clients with a greater flexibility in determining the balance between the selection of design and material standards, initial cost and the expenditure that may be required subsequently if a predetermined design life is to be achieved.

The client's role, the present

A client has four key roles to play; promoter, specifier, manager and beneficiary.

It is the client's responsibility as promoter to clearly define his requirements and to secure the means of achieving them. No matter how certain he may be of the outcome of a project, a prudent client will carefully assess his project during the initial stages to identify those elements critical to its success and to set the boundaries for time and cost estimates. Analysis of the various risks, whether they be financial, technical, managerial, social or natural, will aid the direction of the requisite effort towards, if not minimizing then at least isolating and containing less certain events and establishing contingent processes. At the same time he will want to complete an initial assessment of likely benefits in order to inform investment judgements and aid the raising of finance.

As specifier, the client has to define the performance criteria that his project is expected to meet. Whether or not the client develops those criteria in detail or employs others to do it for him there are certain basic requirements that only the client can decide; the purpose for which the project is being undertaken, its location, its design life, maintenance levels and limits of liability. More knowledgeable clients may also wish to closely define the material and performance standards which experience dictates will, if met, provide the sought after level of service.

In exercising his managerial role, the client will have to decide how involved he wishes to become in the design and construction processes. The choice here will determine the appropriate terms of engagement of designer and builder.

Finally, as beneficiary, either direct or indirect (as, for example, in the public sector), the client will wish to ensure by means of suitable monitoring and maintenance regimes that the project continues to reach the levels of service required to safeguard the returns on his investment.

The client's role, the future

Looking to the future, the client's roles as promoter and beneficiary are unlikely to change; the first will remain essentially one of careful investment appraisal and the selection of an option that optimizes the probability of success, in both technical and financial terms, while minimizing risk; the second, that of beneficiary, will continue to provide the client with the hard evidence against which the success or failure of his project will be measured and, eventually, judged.

On the other hand, the means of achieving a project's objectives through the specification and management processes is likely to change. The speed of change will depend on clients' willingness to exploit the potential benefits of technological development and of greater management efficiency. Improvements deriving from technological change can be expected to increase durability and reduce uncertainty. Similarly, more efficient management aimed at minimizing waste in the use of both human and financial resources, possibly allied to quality assurance, will enable more to be done by fewer people with greater assurance.

Summary

Traditionally, a client's expectations of design life can perhaps best be summarized by durability, maintainability, serviceability, reliability and safety. Whilst those characteristics are unlikely to change in the future, the means of achieving them will depend on developments in material technology and construction techniques. Notwithstanding such changes and a client's willingness to encourage them or be receptive to them, the concept of design life is presented as an economic appraisal process in which critical success factors and risk analysis play a crucial part.

Whilst the client's traditional roles of promoter and beneficiary are unlikely to undergo any major change, technological and management developments will lead to greater certainty in the achievement of a predetermined design life.

Summary of presentations and discussions

K.SRISKANDAN

In his introduction Professor Sedlacek said that his paper was a contribution towards answering the question whether robustness and durability combined with requirements and capacity can be expressed in quantitative terms, particularly for steel structures. A method has been developed and used in the assessment of some 100 year old truss bridges on the Berlin Underground System.

The method consisted first of identifying by analytical means those critical members whose failure would lead to the failure of the whole structure. Having obtained the properties of the material (from micro-specimens cut out from each member), the time taken for fatigue cracks to grow from initiation to a size that would lead to failure of the member was calculated by fracture mechanics methods. From this it was possible to identify those members that needed to be strengthened or replaced, and other members which could be monitored at intervals which were less than the period for initiated cracks to grow to a size that would result in failure. The method had been applied to some 38 bridges.

In reply to various questions, Professor Sedlacek said that this was not a 'full reliability' method. It was a step by step method based on samples taken at the point and applying the *J*-integral. It would be difficult to apply this method to orthotropic decks which have spatial stresses. However, a start had been made on its application to military bridges with linear members and also to guyed masts. Whilst this was certainly the way forward for the assessment of existing bridges, the costs of fracture mechanics methods (about £500 for one *J*-integral) would make it expensive for new bridges.

Mr Davies said in his introduction that clients would have to pay a premium if they wanted a longer life for their structures. Recent research on faults in buildings has shown that design details accounted for 63%, faulty materials 11% and workmanship 26%. This highlighted areas to concentrate on, in addition of course to specifying better quality materials and workmanship.

Attention should be paid in design to the replacement of short life components. Design life is dependent on time, cost and quality. Life-cycle costing should be used in deciding whether to use better quality materials and/or components.

The contractor should not be considered merely as a third party who comes along and builds whatever has been designed by someone else. The contractor has years of experience which should be drawn on by the client and the designer. He should be considered as someone working on the side of the client in the project team.

Dr Beeby felt that it would be speculative to say what contribution research may make to the subject of design life. However, in looking back one can see that there has been a steady growth in research papers on durability. In fact, the growth between 1976 and 1990 has been almost exponential, with roughly 1000 papers having been produced in 1986/1987. The result as far as design is concerned has been absolutely nil, because all codes have stayed more or less the same.

The reason for the research having little or no impact is that the design rules have been prescriptive. There is no quantitative framework which could have been improved by research. The only quantitative figure in Codes is the design life which, because it is so far in the future, is not testable. Therefore, it is appropriate to reset the problem by specifying a 'durability reference period'. Instead of specifying a time to end of life, it would be preferable to specify the extent of deterioration in a shorter period of, say, 10 years, which if the rate of deterioration was known, would then equate to a required design life.

Mr Burns introduced his paper which he said was concerned with the expectations of private sector building developers. They seem to be more concerned with the life of other components such as building services and external walls.

More often than not, buildings are demolished prior to reaching their design life, because of change of use, and they cannot be economically rebuilt to serve another purpose. Clients would therefore like to have a structure that allows for change of use, but at minimum initial cost. Structures that have loading and spacial flexibility are likely to be more adaptable for change of use, and are therefore likely to last longer.

Mr Burns then illustrated some of his points using particular cases. The piles under Canary Wharf in London Docklands have a 125 year design life allowing for corrosion. The exposed structural steel frame of Building 11 at Broadgate has an epoxy protective system which, with periodic inspection and maintenance, is expected to last indefinitely.

The Economist Building in London, where the structure was refurbished for a change to electronic publishing was a case in point. The life of the structure was extended, albeit with some maintenance commitment with regard to painting contaminated concrete.

County Hall in London, which was purpose built for local government offices and is now a listed building, is being renovated at some expense to convert it to a hotel, housing, retail and a business/conference centre. Two adjoining buildings were demolished to allow the construction of more flexible office accommodation.

A final point made by Mr Burns was that, in the United States, more independent testing was employed by the client to oversee the work of the contractor.

Mr Holland described what he saw as the requirement of a public client. As an investor the client is accountable to Parliament. There are limits on what is affordable and investment appraisal methods have to be used in making the choices. These methods do require some definition of design life and to counter one of the questions posed by the Chairman, he said that it would not be possible to make any investment choices without the definition of a design life. In all of this, the client is seeking to reduce uncertainty.

Against all this background, he saw the client's role as four-fold. First, as a promoter, he defines his requirements and secures means of achieving them. Second, he is a specifier. While, in the past, there have been client specifications in the United Kingdom, the public sector will need to take note of 1992, and specifications are likely to become performance requirements rather than method statements. Third, as a manager, he will be concerned with organization, procurement, monitoring and meeting information needs. Fourth, as a beneficiary (owner), he has a maintenance role.

Finally, Mr Holland saw the client's role to create the environment within which his project has the best chances of success—by educating users of the limits of what is possible in terms of design life and getting their confidence.

In the discussion on the papers, questions were asked on what design lives were specified by public clients and to what extent politicians should be involved in the decisions. Mr Holland replied that, in the Department of Transport, roads are designed for a 40 year design life, which in effect meant that they were designed to take the number of standard axles that were predicted to run over them in the next forty years. A nominal design life of 120 years was specified for structures, although in practice the time dependency was checked only in relation to fatigue.

On the point of confidence, his view was that politicians were the ultimate decision takers, but they were of course told of the various options available and the consequences of choosing each. The speaker who asked the question commented that politicians and governments were in office for comparatively short periods and therefore their view of priorities could be different from that of engineers looking at the life of structures.

Another speaker commented that engineers were bound by codes which gave some indemnity. He felt that there should be a framework to step out of codes for long life structures.

The discussion then turned to quality control. One speaker commented that building contracts in the United Kingdom did not have independent testing. Another speaker commented that during a recent visit to Japan he found a contractor who was checking chloride content and in situ strength of concrete. In the United Kingdom, this type of control was not exercised nor would anyone know what to do if they found something wrong. He emphasized the need for better supervision and better record keeping.

Commenting on the North American experience, another speaker stated that they exercised a penalty clause for non-compliance. Client and contractor are finding that this was working well because the result was better testing. Another speaker commented that we should examine the Japanese practice to see if there were aspects that could be usefully introduced to the United Kingdom.

E Summary

Some final reflections on design life

G.SOMERVILLE

Introduction

As chairman both of the Organizing Committee and of the final Session (Part D), I was given the unenviable tasks of attempting to sum up and of pointing the way ahead. However, since it is a very broad subject, and the sense of the discussion to the individual sessions had been caught very well by the reporters, a summary seemed inappropriate.

What did seem appropriate was to reconsider the six key questions that had been posed to participants prior to the event (contained in the Introduction to this volume) and to assess whether or not some form of consensus had emerged. At the colloquium itself, the author attempted to do that, with the aid of some overhead projector slides, while seeking audience reaction to a series of questions and issues where a consensus view seemed possible. What follows is a more structured and extended written version of that final Session.

Do we need to do 'better'? Would a structured approach help?

The answer to both of these questions appeared to be 'Yes'. The general feeling was that lack of performance was on such a scale that something had to be done. While we could not clearly see the way ahead, enough information existed to make a start, and there was a need to be positive in developing design approaches which related more to functional needs and in-service performance with time, including coverage of maintenance and change in use.

Difficulties that hinder progress fall into two categories: technical and non-technical. Key words in the technical category were uncertainty, variability, sensitivity, interaction, and a general lack of knowledge. However, awareness was growing, performance data banks were filling up, and ideas/proposals for a structured system were emerging, if only at the embryo stage. Possibly, the introduction of Eurocodes would create a clear opportunity to bring in guidance on these issues, since design life was not solely a UK concern.

Non-technical matters might be of greater significance; those identified at the colloquium included:

- A general need to gain acceptance, particularly with clients, who would need to be made aware of the perceived benefits from integrating financial, functional and technical performance in life-cycle costing terms, involving both investment appraisal and value engineering.
- Financial matters: in particular, the wishes of individual clients on the breakdown between initial capital investment and subsequent maintenance costs.
- Legal liability matters: if a designer is working to a nominal life of x years, then what is his liability? Should there be a 'signing off' period after (say) 5–10 years, provided a durability assessment was done at that time.
- There were many parties involved in financing, designing, constructing and maintaining any structure; to meet defined objectives and obtain a consistent approach presented problems.
- The very broad questions of who should take the lead in any attempt to raise standards and to develop a more rational approach and how should it all be financed.

Do we need a target, represented by a nominal design life?

As a minimum, it was felt that the life of a structure required conscious consideration: however, views were mixed on what that meant at the detailed design level. On the one hand, the case was argued for simple concepts, only in terms of 'normal' or 'long life', with the development of reliable systems to increase the probability that those general targets would be met. Everyone agreed that 'design death' was to be avoided (i.e. premature failure, in technical performance terms, which would interfere with the client's financial and functional plans).

On the other hand, views were expressed that specifying a life in years would at least put up a marker that longevity was important and required careful consideration. Moreover, knowledge was increasing rapidly, and predictive modelling had reached the stage where greater precision was possible, if required, in practical and economic terms.

However defined, some form of target seemed desirable, but the feeling was that the term 'design life' gave it an air of precision which could result in unnecessary difficulties on liability. No alternative term was agreed, but phrases such as 'technical life', 'duration of use' and 'technical performance profile' were used frequently. The inclusion of the word 'technical' helps to make it clear that the aspect of performance, which is perhaps the main concern of the construction industry, has to be seen alongside functional and financial lives, which are the dominant concern of the client/owner at present. All three 'lives' would have to be considered harmoniously in the future by all the parties involved.

It was felt that such a target would be useful in a number of ways, including:

- The development of material/component specifications
- Assessing alternative design strategies in *comparative* as well as absolute terms
- In the short term, as a technique to develop balanced approaches to design based on lifetime concepts
- The introduction of the concepts of replaceability, maintainability and consideration of inaccessible parts
- Use of 'yardsticks of quality' along the lines of agrément assessments
- Formalizing already established techniques, e.g. the use of accelerated testing in establishing 'life' under fatigue loading

What are the issues to be considered, in developing technical life concepts?

It was suggested that the factors to be considered, and requiring varying degrees of attention, in moving forward would be:

a. Design concept and detailing
b. 'Loads'
c. Performance criteria
d. Factors of safety, margins
e. Design and detailing models (predictive models)
f. Material specifications
g. Workmanship
h. Maintenance

Very briefly, the points that emerged from the Colloquium, on each of these, were:

Design concept and detailing

- Avoid designs that are inherently vulnerable and sensitive to predictable damage or deterioration
- Build some flexibility into the design, i.e. the tolerance to cope with some change in environmental conditions, marginal departures from design assumptions, movements, etc.
- Consider redundancy (back up) in the system when designing for the consequences of known hazards

- Develop a strategy for maintenance/repair/partial replacement, which is flexible and can allow for some change in use
- Give attention to detailing for movement and moisture, e.g. articulation, drainage, shape of sections and elements, protective measures, etc.
- Provide access for inspection and maintenance
- Design concept should provide early warning of visible signs of serious defects
- Structures should be easy to build, without undue dependence on perfect workmanship
- In brief, provide robust structures

Loads

This was an area where a lot of work was needed, especially on:

- Definition/classification of macro- and micro-climates
- Interaction between different types of aggressive action
- Influence of architectural and engineering detailing on transport mechanisms for aggressive media
- A better feel of the variability involved, in order to derive design values for loads
- Consideration of interactions within the total structure when it has experienced some predictable deterioration and damage
- Establishing when predictive modelling of deterioration processes was valid and appropriate and when not, i.e. it relates to gradual deterioration, and cannot be expected to deal with unexpected events or accidents and the changes that these might effect on the gradual processes.

Performance criteria

Current structural Codes contain general performance criteria of the type: The structure should safely transmit the imposed and dead loads to the foundations', and 'Deflections should be compatible with the movements acceptable to other elements, including finishes, services, partitions, glazing and cladding'. Later in the Codes, methods are given for satisfying these general criteria.

Comparable performance criteria are required, in the context of durability and technical life. Here, the matter is further complicated by the time factor, by differing client needs even for the same type of structure, and by other factors, including future change in use, which are often unforeseen or outside the designer's control. Nevertheless, the issue of performance criteria has to be addressed, since, as with conventional design for strength and serviceability, it will not otherwise be possible to compare design solutions and choose the most appropriate.

While the evolution of performance criteria might be comparatively easy at the detail level (e.g. limiting crack widths, acceptable levels of corrosion), the broader performance criteria are more difficult to define, and there was no consensus at the Colloquium on how this should be done. For the record, the issues that should be addressed in this area include:

i. How long should different types of structure, performing different functions, be required to remain in service?
ii. In tackling item (i), should we try for refinement and nominate technical lives in years for a range of structures, or do we use a simple qualitative approach (e.g. normal, long life) and within that elaborate on the 'performance profile' technique illustrated by Mr White?
iii. Within the performance profile method, is it possible to classify structural elements as replaceable, repairable and life-long (as might well be done for services, glazing and cladding)? If so, should a factor be introduced to reflect the criticality of the element to the overall structural performance? Further, how should the expected life of replaceable elements be assessed?
iv. For any given type of structure, is it possible to develop an approach that is sufficiently flexible to allow the client to select from alternative financial strategies (e.g. low capital cost and high maintenance versus high initial cost and low maintenance)?

Factors of safety, margins

This represents an important element in conventional design, and is equally important in terms of technical life. There are several aspects as to how these might be used in practice.

i. Conventional usage of margins to take account of uncertainty and deviations in loads, predictive models, etc., as well as variations in material properties. This implies a numerate approach, based on limit state principles and the derivation of design values, while systematically dealing with known hazards.
ii. There is a broader issue—variations in the inherent vulnerability of different types of structural form. Does each type have to be identified and different margins assigned as in (i), or should this be dealt with by building into the design itself a level of redundancy or robustness which will cover the full range of structural form which is likely to be used in practice?
iii. Whatever framework is evolved to permit a more structured approach,

reliability issues will have to be introduced, both in detailed design and construction (including supervision). As a very simple example, if a minimum value for concrete cover is specified, there should be a procedure for ensuring that it is met. The existence or otherwise of such procedures should influence the margins and factors of safety that are used, and, crucially, will have a major impact on the 'as-built' quality of the final structure.

Design and detailing models

Design models relate to prediction of deterioration under specific aggressive actions. A lot of progress has been made in this area, but the models will have to be fitted into a design format (with appropriate margins or factors). However, as outlined briefly above, detailing has a strong influence on performance. This is less amenable to a numerate approach, and solutions will have to be found based on feedback ('this detail works, that detail does not and here is the evidence').

Material specifications and workmanship

These two items are considered together, since they are interrelated. Relevant material specifications should refer not only to materials but also to any components which might be classified as 'replaceable' in performance profile terms: possibly these should be derived based on the techniques suggested by Mr Jubb in his paper.

Equally important is the derivation of reliable systems to ensure that these specifications are met, preferably in terms of 'testable quantities' as suggested by Mr Beeby. While Mr Rodin's point is taken, that the design concept should not be dependent on perfect workmanship, it is clear from in-service surveillance that actual performance can be sensitive to apparently quite small deviations in workmanship. Some of this variability and uncertainty must be reduced. There is a strong emphasis on quality in all of this; the use of quality management systems, quality assurance and quality control.

Maintenance

There was a clear message from the Colloquium that maintenance had to be considered at the design stage, and a strategy developed which was both compatible with the design concept, and agreed by all the parties concerned, especially the client. Provision had to be made for inspection, maintenance and possible replacement, if this was part of the performance profile.

It was clear from those papers presented at the Colloquium, which dealt with other industries, that a more 'hands on' approach was possibly required for structures, involving more rigorous checking of levels of inspection and maintenance and even the scheduling of replacement parts.

Where are we now and what happens next?

Relating the above notes to the six key questions in the Introduction to this volume, it would appear that the Colloquium was of considerable benefit in defining the present state-of-the-art, i.e. where we are now. It also gave some pointers on what should happen next; in moving towards a more structured approach in order to do 'better'.

However, there was no time for discussion on who should take the lead or who should pay for the essential development work. The scale and nature of the task is daunting, and the application in practice would involve cooperation between parties who have not traditionally worked together for any significant period.

At the Colloquium, many speakers made the points that there was no conscious design for 'life' at present, that clients wanted only that which is perceived today, that they prefer tried and tested solutions rather than innovation, and that they have no perception of the potential benefits from lifetime planning, even if this fitted in with their investment plans. To make real progress, therefore, a major campaign would be required to 'convert' clients, collectively and individually. This requires resources, both to clearly establish the benefits and the methodology, and to do a major selling job in putting the ideas across.

It has been suggested that the results of R&D have not been translated into practice in the construction industry. However, there is now a trend for those involved in R&D to be engaged in more 'sharp end' activities, often involving collaboration with consultants, contractors or material producers. R&D would most certainly be required to develop technical life concepts in a practical way; if the collaborative trend could be harnessed to this activity, then genuine progress could be made.

The difficulty is that the construction industry is not a coherent unit—there are many sectors to it, and fragmentation within each sector. Put another way, the industry is made up of individuals and single companies of varying size, having diverse interests and attitudes. It is therefore facile to say that there should be a combined effort between, say, 'industry' and 'government', although the benefits would be of value to both. Certainly, the problem should be tackled on a national, or even international (European?), basis. Possibly, therefore, 'Government' (which does have a clear identity) should take the lead, since it is also a major client, while ensuring that the private sector and the construction industry become fully involved. Until that happens, there will only be a slow build-up of 'success stories', involving enlightened individual clients and their advisers.

It has been a feature of past Henderson Colloquia for a subsequent open meeting Symposium, organized jointly by IABSE and the Institution of Structural Engineers, to be held on the same subject but as a development from the Colloquia discussions. The Organizing Committee intend to examine these discussions and from them to formulate a Symposium programme, which will enable constructive proposals to emerge for the necessary development of the key actions the Colloquium has identified.