PRESIDENTIAL ADDRESS
WHY SHOULD WE BOTHER?
REFLECTIONS ON AN ENGINEERING INSTITUTION PRESIDENCY

by Prof. Brian Clancy

on Tuesday 21st September 1999

Introduction

During my life to date, I have had the privilege to belong to a number of professional, charitable and political organisations. It has always interested me as to why society, particularly its male element finds these organisations so essential. Allied to this interest has been my own most prestigious appointment in one of these bodies. In 1996 I was elected President of the Institution of Structural Engineers. It was a great honour. I thought that I would try to combine the two themes by giving some thoughts on my experiences of them.

To satisfy some deep seated psychological need the human race has since its very earliest times formed associations, societies and institutions. These may have been political, military, religious, professional or whatever. Among the most enduring of these 'guilds' have those been formed by Trade organisations based on the skilled and professional 'classes'.

The old medieval guilds, originally formed as trade bodies to protect the interest of their members have metamorphosed into the Livery Companies based in the City of London. The present 'guilds', such as the Trade Unions, Chartered bodies, etc, now serve the original purpose of those old medieval guilds. They draw together people of common interest and skills, encourage training and educational standards, recognise competence by qualifying their members, encourage exchange of information between members and represent the membership to the outside world. They discipline or expel those who fall below the agreed set requirements.

In the UK, many of these Professional bodies possess Royal Charters. A Charter defines what the particular organisation can and cannot do as regards qualifying and representing its members, so as to ensure that it does not tread on the toes of other similar bodies. In other words it defines the interface between bodies that possess Royal Charters.

As most of us know, engineering was originally a military activity. Roads, bridges and most infrastructure was created for the sole purpose of getting armies from one part of the country to another in the shortest possible time, in the best condition and fully supplied. In the 1600's, alongside shipbuilding, Civil engineering started to be recognised as a specific activity, which could benefit trade and commerce. The Institution of Civil Engineers was officially founded in 1828 and the Institution of Mechanical Engineers followed in 1847. It is said that the Mechanicals were only formed because the Civils considered the obsessive interest of some of their members in mechanisms was distracting to the core civils activities of roads, bridges, canals, etc. Indeed it is a common trait of engineering that the establishment frequently fails to recognise change and progress when it is occurring before their very eyes. Hence the reason why we have so debilitating a large number of Chartered and Incorporated bodies within the engineering and construction industry.

Pursuing this tack, the Institution of Structural Engineers was originally founded as the Concrete Institute in 1908. The purpose of the original institute was to look into the development of the "new fangled" construction procedure now known as reinforced concrete. The Concrete Institute was founded with the blessing of a number of Presidents and vice presidents of the Institution of Civil Engineers. At that time civil engineers saw little in buildings to interest engineers, they preferred to be more associated with large infrastructure projects. Little did they know that their disdain of buildings would by 1922 lead to the formation of the Institution of Structural Engineers. This Institution obtained its Royal Charter in 1934. Numerically it is now the fourth largest Professional

Engineering Institution in the UK after the Electricals, Mechanicals and Civils. The other twelve follow behind.

The Structurals has its headquarters in Belgravia, close to Victoria Station, where the staff of about 35 are housed and where facilities exist for meetings, seminars, etc., as well as housing the essential well stocked library. It is a fact that most members of professional bodies rarely if ever visit their well appointed and sometimes sumptuous head quarters buildings.

Like most other Chartered Engineering Institutions, the Structurals have a branch network throughout the UK and overseas. It is governed by a voluntary Council (which includes directly elected members plus the branch chairmen) and headed by a President plus various Vices and past Presidents, all very conventional.

In 1996 I was elected President. I have often asked myself - Why? One thing for certain is that I certainly never set out to be the President.

Personal experience

After leaving University College, London with a very modest degree, I was initially employed by Oscar Faber & Partners (1962-66). Fabers had a tradition of involvement with Professional Associations and Institutions based on an obsessive interest in these types of bodies by their founder Dr Faber. He was President of two Institutions during his life and various senior members of Fabers have been Presidents of other Institutions over the last forty years.

From Fabers I moved to C S Allott and Son (1966-69) who subsequently became Allott and Lomax and more recently joined the Babtie Group. Again it was a firm which had a tradition of involvement with Professional Institutions and Associations. A small group of us regularly attended branch lectures and also the annual dinners of a number of building industry bodies; it was a pleasantly fraternal pastime.

Of my own initiative I submitted a paper to the Structurals local branch entitled "Old Cotton Mills for Modern Usage". I won the Junior Branch Prize but the topic was ignored for thirty years. I am very pleased to see that many of these fine buildings (at least of those that are left!) are now being converted for residential and other modern usage. My arguments of course applied not just to mills but to warehouses and similar turn of the century buildings.

I enjoyed my time at Allotts. I was given responsibilities far beyond my own perception of my competence, but I noted that - strangely others were actually not a great deal better. I learnt a great deal from both Fabers and Allotts.

In 1969 my boss at Allotts, Brian Moorehead, asked me to join him in business. We started our own civil and structural consultancy - Moorehead Clancy and Associates. We thrived over the next three years and built a staff of 34, but I progressively became disillusioned with the manner in which the firm was run. My somewhat idealistic views were continually overruled in the pursuit of commercial gain - something that I was to experience more than once in my subsequent career.

So I left Mooreheads in 1972 and set up again in practice as Brian Clancy Associates. I was the first, but not the last, to leave Mooreheads and set up again a successful firm.

In 1971 I stood for election to the Sale Local Council, not out of any personal ambition to emulate Gladstone or Churchill, but because I became aware that a local councillor was standing unopposed. That seemed a poor show in a democracy, so I found out how one went about becoming a candidate. I stood with the blessing of the local Liberals. What the electorate had against the other guy, I don't know but I was elected comfortably.

This drew me into the new world of local and national government and how it operates. Committees of all sort ranging from education to parks and cemeteries, from finance to housing. The 8 years I was to spend on the council (1971-78) covered the 1974 period of local government reorganisation when hundreds of small local councils were amalgamated into larger bodies. In Greater Manchester alone some 80 small bodies became the 10 we have today. It was very enlightening.

My involvement with many schools as both a parent and governor made me aware of the lamentable standards of career advice given to children. I realised that there was a distrust of science and an ignorance of engineering among most 'career advisor teachers' at the time. I wrote to three successive local branch chairmen of the Structurals about the lack of career advice on engineering.

Involvement with the Institution of Structural Engineers
It was third time lucky. Eric Taylor came to see me in my fledgling office in Altrincham in 1979 and told me how much he agreed with me and he would like me to handle the problem. That was not what I had intended, but I took it on and also joined the local branch committee. Within 6 months we had a dynamic schools careers and visits service going with 30 members of the branch involved and visits to some 50 schools a year. For my efforts I found myself as branch chairman of the Lancashire and Cheshire Branch of the Structurals in 1984 with visits to London for Council meetings as well as local lectures, committee meetings and events to organise with our excellent committee. My 'cause celebre' at this time was, with a few others, my concern about the finances of the Structurals. We were considered to be alarmist. I only wish that had been so. In the event the Institution nearly became bankrupt and in 1994 a rescue package was launched to rescue the situation, based on my simple idea of asking members for one years extra subscription and the suggestion by Prof. Mike Burdekin that it should be spread over 3 years.

However, back to the mid 1980's. By this time I had started to take a particular interest in appraisal of existing buildings and refurbishment projects; I was doing many of them. I even applied to join the RICS, but was turned down as I could not serve my professional practise under an RICS member, being a principle in my own firm - but I became a graduate member. I had developed a procedure for appraising buildings and structures. The Structurals asked to see it and that lead to my chairing a task group, which in 1991 published the 'Guide to surveys and inspections of buildings and similar structures', based mainly on my 'check list'. My other hobbyhorse being recognised and my being asked to form a task group to look into the whole matter of 'subsidence damage to buildings' soon followed this. Subsidence was something that I had been writing and thinking about for many years. It was at the time affecting some 50,000 properties each year, causing serious distress to hundreds of thousands of families and costing the insurance industry over half a billion pounds per year.

I had firm ideas as to how the problem should be tackled. The Structurals gave me a free hand and I formed a first class multi professional team to look into it. We examined the subject from basics and produced our Guide in 1994. It has become the seminal work on the subject and was even plagiarized by one of our task group for his own publication - what greater, if somewhat pathetic, flattery!

The Presidency looms
By this time I had been asked to be a vice president of the Structurals. I was flabbergasted and humbled. I thought of the great names that had been there before me. Why me?

Anyway, I addressed the role with my usual enthusiasm and gusto. I remembered David Lee a previous President saying that there is no point waiting to become president to effect change. The presidency is the big 'thank you', if you haven't done it by then - it's too late!

There were a number of challenges. The subsidence guide had to be published and publicised. The first task was achieved in early 1994 and I set out to lecture on the recommendations of the guide to as many people as possible. In six months I gave 46 lectures all over the UK to as many as would listen to me. The sales of the guide were exceptional by comparison with sales of other Structurals

guides. Over £18,000 profit was made which not only reflected the success of the guide but it was also a welcome boost to the finances of the Structurals at the nadir of its problems.

Bearing in mind the adage of David Lee, I next addressed myself to a matter that was of particular concern to me. The largest overseas group of members of the Structurals is in Hong Kong. Sovereignty of the Province was due to pass back to the Peoples' Republic of China during my presidency. An old school mate was the Governor - Chris Patten. How could we give those members our moral support at this uncertain time? Our prestigious four yearly Kerenski Conference was also due to occur during my 'year'; I decided that we would try and hold it in HK but after the hand over. The idea was warmly welcomed by our HK members, despite none of us knowing what attitude the PRC might adopt after the handover, which was at the time 3 years away. The date was fixed and we went for it.

Meanwhile I was learning the ropes of senior management at HQ. Working my way up through the various senior committees over a four year period as either chairman or a member - Professional Affairs, Membership, Engineering, Finance and Resources and Professional Practice, etc., etc., and all this from Manchester while at the same time trying to make a fair contribution to the business of my firm as its senior partner. I was very lucky to have good colleagues in the firm.

Just as I was about to enter my senior vice presidential year, the then President Prof. Patrick Dowling suggested to the incoming President Brian Simpson that I should be asked to review the whole of the CPD policy of the Structurals. Why me? But I formed a small task group and we got on with it. Again we started from basics and I believed we developed a first class system, which is based on the fundamental premise that the members are responsible professionals and not recalcitrant 4th formers. The suggestions were adopted and complemented by the accreditors of the Engineering Council as being the most imaginative that they had seen among all the Institutions in the Eng. C.

The Presidency - Inauguration and the first phase
So the date of my inauguration as President approached - 3rd October 1997. But before that there was my Presidential address to write. What on earth to write about? What had struck me about the Structurals as most exceptional - why the members of course. That is all that the Institution was - indeed any professional body.

So that was the theme of my Address - 'We the members are the Institution'.

It took ages to write but it was all my own work (edited by Mo my wife for grammar, spelling, etc.!).

It was well received and I enjoyed the evening. I was the president of the Institution of Structural Engineers. The 4th largest and one of the hardest to gain membership of. Why me?

What were to be my personal rules? Well as a young lad I had worked on the markets. I loved it at heart I was really a barrow boy - so it was easy.

Sell the members.
Sell structural engineering.
Sell the Institution.
Sell myself.

But that was to be the order of importance and I was not to forget it! I tried not to.

The day after my inauguration the hard work of visiting every branch in the British Isles began. Every local Chairman has to be inaugurated by the President. Most of these visits involve the pleasure of visiting project in the area. What a wonderful breadth and depth of skill and achievement our members possess. From North Sea oil rigs to the strengthening of the Forth Road Bridge to Sports Stadia of all types and wonderful concert halls. Renovation of timber, steel and masonry buildings. Magnificent University buildings and the pleasure of discovering an unknown tension structures expert in Norwich and another on domestic swimming pools, all convinced me that UK structural engineering was (probably like all UK engineering) brilliant but unsung with engineers being their own worst enemies in this respect.

During my year of office I tried repeatedly to get members to promise to write papers or offer to give lectures on their projects and specialist skill. My efforts were not entirely wasted but the success rate was low. Why are we so reluctant as engineers to hawk our wares?

Problems

But all was not without tragedy. A few weeks after my year commenced my senior vice president David Alsop died. He had been ill for some time but was thought to be making a good recovery. Now this was a real presidential test. Unfortunately none of the other vice presidents were in a position to 'jump the queue'. After consideration it was thought that for such an emergency we needed a nationally recognised senior member in whom we had confidence to 'do the job' with a minimum of training. Dr Sam Thorburne was the man but he was not a member of Council and there were no vacancies. So with the support of Council, I set off on a marathon to get Sam elected. With the solid support of all Council members this was achieved, but not without drama.

First a member had to be asked to resign. I went in person to Edinburgh to ask Alex Taite if he would do so for Sam a fellow Scott. Alex - a great servant of the Structurals agreed at once. But then we hit a real hurdle. A past president of many years ago decided that he did not like what was being done and reported us to the Privy Council. In fact I suspected that he did not like Sam, who was an expert in the same field and more renowned.

This required some considerable skill to deal with and my political training served me well here, but that is another story. Sam did become my successor and an excellent president he was! We also elected our first woman vice-president in my year Allyson Lawless from South Africa - another first!

Some things crawled out from under stones. They were dealt with but they are best left unrecorded here.

Representation

So the year proceeded. Judging our Institution Awards for excellence and giving them out at the Houses of Parliament. Chairing our prestige lectures - The Gold Medal given by Sir - now Lord - Norman Foster and my choice. The Maitland named after our first secretary of the Institution, that was given by Michel Virlogeux the great French bridge engineer, unfortunately on a bright sunny evening in a room with no curtains! A bonus was that Mr Maitland's daughter heard of it and attended; I was subsequently also to meet his grandson during my year.

Council meetings, committee meetings, eating for England as a guest of numerous other professional Institutions and Associations, - about 33 in all. Negotiations with 'others' with whom we had minor tiffs during the year including the Civils, the Engineering Council and the RICS. Entertaining overseas members and delegations, including from the Peoples Republic of China, who had decided that they wished to introduce a system of qualification of Structural engineers modelled on our Part 3 final exam rather than any other system in the world - much to the annoyance of the Civils who tried to scupper our chances and those of the UK through the stupidity of some of their staff.

All Branch visits are enjoyable, but it gave me particular pleasure to come to the Hot-pot supper and annual dinner of my own Branch. I had received such great support from the members over the years. I am told that the Branch Annual Dinner was one of the most entertaining for years, with the 'stars' being Roy Partington and myself! Amazing!

The official highlight of the year is the Institution Annual Dinner held traditionally at Guildhall in the City of London in May. Choosing menus requires attending 'tastings' and 'tipplings'; it is not just

banged down in front of you on the night. Principal guests have to be chosen and invited. A President's speech has to be prepared. Then there is the concern as to whether anyone will attend!

In the event it went off famously, with Jonathon Porritt, the environment champion, and Sir Derek Roberts, Provost of University College London - my old college, propping me up as principal speakers. Awards were made to deserving members and all in front of some 400 guests, including my family, my partners and numerous of my friends - not 400!

Overseas - the First visit

Immediately after the Annual Dinner in May 1997, Mo and I set off for Singapore and Malaysia to attend an international conference where I was to deliver the 'keynote' lecture and one other. We visited projects and government departments, met local members in both countries and we were able to contrast the markedly different approach of the two governments to professional organisations; probably best summed up by contrasting their economies over the last 25 years.

Then back to the UK for the last weeks of the professional year of lectures, committees, dinners and visits before setting off on a marathon second overseas tour in August.

The second overseas visit - the Marathon

I had decided at the beginning of my year that I would try to meet as many members as possible. They are entitled to see their President. I enquired as to the sum allocated for the overseas visit(s) and decided that, if I travelled second or tourist, it would go a long way and so would I.

It would also help if I earned my passage by giving lectures and seminars from which the local branches could raise some funds and from which my fares and accommodation could also be fielded.

So Mo and I set off. First to South Africa - 4 Cities, 4 receptions, 4 lectures and 6 seminars. Plus golf at Sun City and a visit to a National Park - lots of big and dangerous animals! A visit to the heart of Soweto - even more dangerous! Most of all, lots of quality contact with quality people of all races.

Then on to India - 5 Cities across North India, 6 lectures, visits to universities, government departments, sister professional bodies, modern engineering projects and equally impressive ones of centuries ago. Again the opportunity to meet and understand people who are trying to improve the lot of their fellow citizens against tremendous odds - and winning.

Next was to be Hong Kong, now part of the PRC. The idea of three years before had worked. The Kerenski Conference was a great success and a credit to the local members who organised it. Visits to local project, universities and the ever-enjoyable receptions and presentations all followed.

International recognition for the UK

Then came the most uniquely important event of my year of office, the signing of the international agreement with the PRC for the mutual recognition and examining of Structural Engineers. It took place after the conference, but in Shenzhen across the border in China 'proper'. This was a great honour for me to be the signatory and the culmination of some three years dedicated work by many people in the UK and the PRC. It was the first agreement to be signed by the PRC with any outside professional body from any country. So important was it that the Senior Vice Minister for Construction, Mr Y Rutang, came down from Beijing to attend the signing. This was despite the fact that the five yearly convention of the Communist Party of the PRC was occurring the same week and every senior politician should be there. That is how important the occasion was considered by the PRC. A banquet and an incredible outdoor spectacular followed the event. This was not just for us, but we did have the best seats!

Sorting out more problems

Mo and I then thought that we would go up to Beijing for a few days to stay at Tsinghua University where we had Chinese friends on the staff. However the Ministry of Higher Education of the PRC heard of my visit and summonsed me to a meeting to clarify to senior officials exactly what was the relationship between the UK Institutions and our Joint Board of Moderators. Some earlier visitors from another UK Institution had confused them. I sorted it out!

The closing stages

Then followed a few days of sightseeing, visits to projects, lecturing at the University and eating exotic foods - I never knew there were so many different ways of preparing snake!

And so back to Britain after 6 weeks away and fascinating experiences. My year of office was nearly over, yet I was full of ideas of how I would help our overseas engineer colleagues, but I was nearly 'yesterday's man'. The few last functions and then to hand over to Sam. How glad I was that he had accepted the office of President, I could not have managed a second year - and the Institution certainly did not want me for another one!

What had I learned - so much.

- The wonder of engineering and its internationalism.
- Its crucial place in the modern world
- The commitment of so many people everywhere and the apparent complacency of the majority
- The excellent work they do
- How bad they are at selling themselves
- The internationality and commonality of engineering and the respect in which engineers are held outside the U.K. - particularly in the third world
- The importance of high quality democratic professional bodies - supported by government - not controlled by it
- The appalling living conditions of so many of our fellow men
- The selfish neurosis of most possessors of power
- The dangers of the population explosion
- The devastation being caused to the world by man
- The increased likelihood of nuclear conflict

FINAL THOUGHTS:

There are only some 20 past presidents still alive in the Structurals so probably only some 300 among all the engineering institutions. Unfortunately many of these are now quite old and infirm, so there are only a few sufficiently active and able to pass on their experience. I know that I have been exceptionally privileged even among these.

If I could do one thing it would be to somehow show every young structural engineer - indeed every engineer - what I have seen and experienced during my year, to make them realise how important and international engineering is.

I could not have done it without the support of many people already mentioned, but especially my wife Mo, my partners at BCP, so many members of the institution and of its staff.

I am glad I bothered to be involved, because it has given me the rewarding opportunity to meet so many of that relatively small number that also bother and upon whom the world depends. I enjoyed seeing something of what they do professionally, technically and for their communities.

I always knew they were valuable but I know it even more now. It is so reassuring to know they are everywhere and in all countries. That is good for the world and all its inhabitants.

The address was illustrated with some 80 slides and overheads.

SIZE MATTERS FOR STRUCTURAL ENGINEERS

by Prof. F.M. Burdekin

on Tuesday 26th October 1999

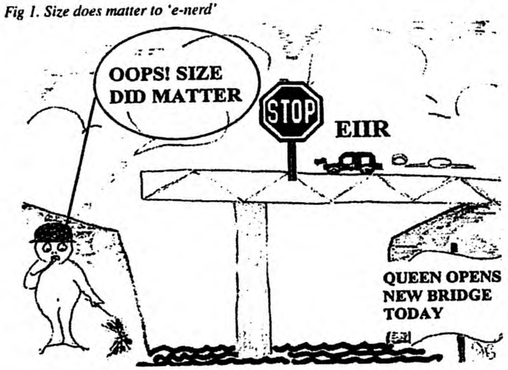
originally given before a meeting of the Institution of Structural Engineers on 29 September 1999

Introduction

The award of the Gold Medal of the Institution of Structural Engineers is a great honour and an immense privilege. One has only to look at the honours board with the list of previous recipients to be truly humbled to be placed in such company. The citation for this year's award referred to contributions to applications of fracture mechanics to fracture and fatigue of welded structures, to teaching and research concerning design of welded structures, and to consultancy: it is therefore appropriate to include aspects of each of these topics in the Gold Medal Address. At the same time, it is hoped that some examples may help others to solve potential problems and to foresee difficulties that might occur with large structures.

The particular field in which I have spent most of my career is that of the behaviour of welded structures. This has involved buildings, bridges, off-shore structures, nuclear and other pressure vessels, pipelines, cranes and ships, fairground rides: in fact, virtually all types of welded construction. My initial involvement in this field was almost entirely a matter of chance, being concerned with the atmosphere of potential workplaces when I left college and having a rebellious attitude to conventional graduate training-schemes under the auspices of the Institutions. I have since become a professional member of five Institutions but through my own choice and interest later in life! I also have a background in materials behaviour generally, in non-destructive testing, and in risk/reliability analysis.

Fig 1. Size does matter to 'e-nerd'



During my career, I have been involved in many failure investigations and research projects. One of the principles I have learnt has been that the effect of scaling-up the size of structural components has hidden pitfalls, and it is on this topic that I propose to concentrate primarily in this address. It really is the case that, in many aspects of structural engineering, size does matter (see Fig 1 where the ubiquitous character 'e-nerd' (*electronic-never ever recognises disasters*) has got his metric conversions wrong).

Development of large structures

Architects and structural engineers are to be congratulated in helping to create many superb examples of wonderful structures that can be found in the world today. They have been entranced not only by creating beautiful structures but also with creating the best and, if possible, the largest in the world. In short, they have been fascinated by size! There have been many excellent books giving examples of such structures, and usually the design and construction has been reported in a series of technical papers. A summary of the development of heights of buildings is given in Table 1.

Currently, the tallest inhabited building in the world is the Chongqing Tower in China at 457m, closely followed by the twin towers of the Petronas Building in Kuala Lumpur. The tallest structures overall are masts, for which the record was held by the Warsaw Radio Mast at 646m. This collapsed during renovation in 1991, leaving the North Dakota TV Mast, at 629m, as the current record-holder. Strictly, the Washington Monument is the highest masonry structure, but it has an iron frame, while the massive cathedrals, for which the record for the highest is held by Cologne, are examples of bulk masonry alone.

Table 1 - Development of heights of buildings, excluding antennae [1-5]

Building	Height (m)	Material	Completion year
Cologne Cathedral	156	Masonry	1880
Washington	169	Masonry/iron	1884
Canary Wharf	245	Steel	1991
Eiffel Tower	300	Iron	1889
Jin Mao Shanghai	421	Mixed steel/concrete	1998
World Trade Centre	417	Steel	1972
Empire State	381	Steel	1931
Sears Tower	442	Steel	1974
HK Bank of China	369	Mixed steel/concrete	1989
Petronias Tower Kl	452	Mixed steel/concrete	1997
Chongquing Tower	457	Mixed steel/concrete	1998

One cannot but be impressed that the tallest structures primarily involve construction in steel. Prior to the 1960s the steel frames of these buildings would have been riveted, but virtually all the later buildings to claim the record for highest structure are welded spaceframe-type structures.

Bridges are often regarded as amongst the greatest achievements of structural engineers. Table 2 shows the development of main spans of bridges over the years. The Forth Rail Bridge is remarkable for the span achieved at the time, and the Golden Gate Suspension Bridge, built in 1937, held the record for the longest span for some 40 years until the Humber Bridge was completed in 1978. It has taken a further 20 years for the record to be taken again by the Akashi-Kaikyo Bridge in Japan. Most of the major-span bridges built after the 1960s have been welded steelwork. It is interesting to note that a powerline in Norway, at 4888m, holds the record for the greatest span of a structure.

Table 2 - Development of bridge spans [1-5]

Bridge	Main span (m)	Material	Completion year
Runcorn	330	Riveted steel arch	1961
Sydney Harbour	503	Riveted steel arch	1932
New River Gorge US	518	Arch	1976
Forth Rail	521	Iron cantilever	1889
Pont de Normandie	856	Cable/stayed	1995
Sydney Gladesville	305	Concrete arch	1964
Tacoma Narrows	853	Steel/suspension	1940-1950
Severn	988	Steel/suspension	1966
Golden Gate	1280	Steel/suspension	1937
Humber	1410	Steel/suspension	1978
Tsing Ma	1377	Steel/suspension	1997
Storebaelt	1624	Steel/suspension	1998
Akashi-Kaikyo	1990	Steel/suspension	1998

Bridge construction over the years has had its share of catastrophes as engineers pushed forwards with daring designs. The film of the Tacoma Narrows disaster is compulsory viewing for all structural engineering students as an example of aerodynamic instability. All suspension bridges are now assessed for such problems by computational and/or experimental model studies. The failures of the Hasselt bridges in Belgium and the Duplessis Bridge in Canada by brittle fracture were precursors of the Liberty Ship failures in World War 2. It was really the failure of King's Bridge in Melbourne by brittle fracture that brought about changes in Codes of Practice to try to avoid this type of failure.

The collapse of Milford Haven, Koblenz and Yarra box girder bridges by plate-buckling problems led to the Merrison Inquiry and substantial tightening-up of the relationship between design and permissible fabrication imperfections for compression members now incorporated in Codes and standards.

The demands for offshore structures in the deeper waters of the North Sea produced enormous challenges for structural engineers. Table 3 shows the developments in height and weight of these structures since the construction of the West Sole Field Platform in 1966. The largest of these structures in the North Sea, the Magnus Platform, has a height greater than the tallest onshore UK building structure (Canary Wharf), excluding mast structures such as the Winter Hill and Emley Moor TV masts. Both the tallest steel off-shore structure to be built so far, the Baldpate Platform, and the tallest concrete platform, Troll, are higher than the Chongqing Tower, which holds the record onshore.

Table 3 - Development of offshore structures

Platform	Height (m)	Weight (t)	Material	Year
West Sole	60	599	Steel	1966
Forties A	193	31,802	Steel	1974
Ninian South	232	51,000	Steel	1977
Magnus	312	77,000	Steel	1982
Britannia	268	38,500	Steel	1997
Baldpate (GOM)	535	44,800	Steel	1998
Troll	472	1,050,000	Concrete	1995
Hibernia (GBanks)	335	587,000	Concrete	1997

Even in the field of leisure, man strives to push the limits of size further and further. Fairground roller coasters have been around for many years, starting with wooden structures, although the major ones are now almost always manufactured from welded steelwork. A summary of recent developments in speed and height of roller coasters is given in Table 4. The record for the UK is held by the Pepsi Big One at Blackpool, with a height of 65m and a maximum speed of 75mile/h. Even since this was completed in 1994, competition in the USA has reached 100mile/h maximum speed and 126.5m height, with the Superman Coaster. The development of the London Eye or Millennium Wheel on the banks of the River Thames in London has presented a great challenge in size, and aspects of this will be returned to later. Sports stadia have become larger and more aesthetically pleasing but have had to face the challenge of controlling the dynamic response of large, open span structures. Recent examples of such stadia include stands at Manchester United, the McAlpine Stadium at Huddersfield, the Reebok Stadium at Bolton, and the Cardiff Stadium.

Table 4 - Development of fairground roller coasters (source: Internet)

Coaster	Height (m)	Max. speed (mile/h)	Year
Pepsi Max Big One	65	75	1994
Desperado	63.7	80	1994
Steel Phantom	49	80	1991
Oblivion	55	85	1997
Superman	126.5	100	1997

The design of these major structures has been accomplished almost invariably through the use of finite element analysis (FEA), and their construction has been made possible through developments in welded steelwork. Essentially, FEA models are mathematical models that can be readily scaled up and down in size by changing input data, without the constraints of reality! As with any aspect of structural engineering, the experience of competent professional engineers is essential to produce safe and satisfactory designs. The availability of sophisticated FE packages has enabled such engineers to push the frontiers of the possible further forward. Similarly, the development of weldable high strength steels has been an essential factor in the advance of major structures. The construction of the big offshore platforms would not have been possible without both advanced FE analysis and improved weldability of steels.

Increasing the size of structures brings with it additional problems that are not always appreciated by structural engineers; these will be explored in the next section.

The effects of size on structural behaviour

There are six main ways in which size affects structural behaviour:

(1) The overall weight of the structure increases.

(2) The area exposed to environmental forces increases.

(3) The dynamic response of the structure changes.

(4) Material properties tend to deteriorate with heavier section thicknesses.

(5) More and larger flaws are likely to occur in thicker sections (Weibull analysis).

(6) Stress gradients tend to be less steep in thick sections.

The first three of these are matters that should be taken into account by normal design procedures, although problems sometimes arise with dynamic response.

Conventional design starts with the requirements of the client for function, form and size of any structure. The designer then determines loadings, support/reaction positions, and decides the form of structural action to be employed before making initial estimates of member sizes. The preliminary design is then analysed, often using FE methods, stress levels are estimated, and an assessment of overall structural integrity is made.

There can be many interpretations of structural integrity but, essentially, it is the production of a structure for which the risks of failure are acceptably low under all anticipated loadings. Inevitably, occasional failures do occur that lead to expensive legal cases (e.g. 'e-nerd' in front of the dock in Fig 2), but, in general, design to Codes and standards gives satisfactory results, unless mistakes are made or the designer pushes beyond the boundaries of experience intended by the Code.

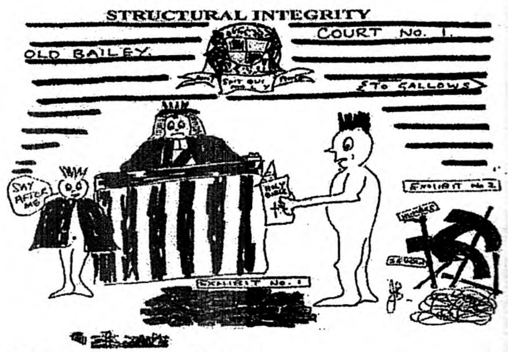


Fig 2. The Court of Structural Integrity

For conventional design, limit state design principles are applied, with the structure designed to reach a series of limit states under loadings multiplied by partial load factors and with resistance variables divided by their partial factors. The appropriate partial factors are chosen to give an overall target reliability, taking into account the variability of the input data and the consequences of failure. The design establishes limiting stresses for various types of loading condition as follows:

Tension/compression: force divided by area of cross-section of member

Bending: bending moment divided by section modulus

Buckling: curves of limiting critical stress against slenderness ratio l/r_y for different strengths of steel

Shear: force divided by effective shear area

It is interesting to consider the effect of scaling-up the dimensions of members of a structure in proportion to loadbearing capacity. If all dimensions are scaled-up in proportion and the same limits on stress applied, one would expect from dimensional analysis that the increase in loadbearing capacity would be as follows:

Tension/compression force: area: square of proportionality factor

Bending moment: section modulus: cube of proportionality factor

Buckling: slenderness ratio: permissible effective length increase linear with proportionality factor

Shear force: shear area: square of proportionality factor

As noted in considering the form of large-scale building and offshore structures, spaceframes or 3-dimensional truss structures are often the most effective. An example of a truss of span L under a central point load is shown in Fig 3.

The maximum bending stress is given by:

$$\sigma = W/A.L/4D$$

where A is the cross-sectional area of each of the chords and D is the distance between chord centrelines. Thus if all that is required is to carry the same load over a greater span, this can be achieved by increasing the depth of the truss to keep the span-to-depth ratio L/D constant. If it is required to increase the load to be carried, the area of each chord A should be increased to keep the ratio W/A constant. The inevitable implication of this is that, as the size of structures increases, the cross-sectional area of the members must also increase to carry the increased loads involved. It is this increase in cross sectional area that give rise to the less well-perceived size-effect problems.

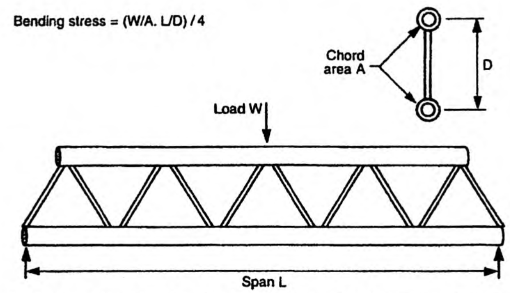
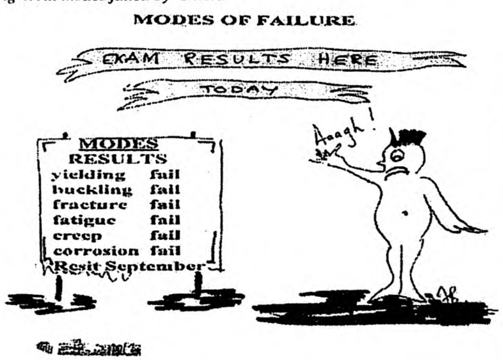


Fig 3. Effect of span/depth ratio and chord area on bending stresses in trusses

It is necessary to consider all modes of failure, as shown by 'e-nerd' in Fig 4. Usually, conventional design is concerned primarily with yielding/plastic collapse, bending, shear, buckling, and overall stability. These modes of failure are concerned with the size of the overall cross-section of members in relation to the applied loads. There is, however, a second group of modes of failure where failure can occur locally and then spread progressively across the cross-section. These modes of failure are fracture, fatigue, and stress corrosion. The latter problem requires a combination of tensile stress, aggressive

Fig 4. All modes failed by 'e-nerd'



environment, and susceptible material. It does not normally occur with conventional structural steels in atmospheric environments but can become a problem with very high strength steels or hardened heat-affected zones in some corrosive environments. It is necessary for structural engineers to take steps to design against fracture and fatigue.

Stress concentrations and cracks: brittle fracture and fracture mechanics

Regions of stress concentration will occur inevitably in any structure at discontinuities or changes in cross-section. Fig 5 shows two classic examples, the cases of plates with circular or elliptical holes. In the case of the circular hole, the stress level at the edge of the hole, at the ends of the diameter perpendicular to the direction of the applied stress, is three times the applied stress; this is referred to as a stress concentration factor (SCF) of three. This is independent of the size of the hole, provided the hole is small compared with the overall width of the plate. There is also, of course, the effect of reduction in cross-sectional area and increase in net section stress. Holes are, of course, present in many structural elements, particularly in bolted connections, but properly designed connections perform perfectly satisfactorily. With a stress concentration factor of three and applied stresses up to two-thirds of the yield strength, it is evident that local yielding will occur, but, provided the material has sufficient ductility to accommodate the plastic strains involved, the structure can perform perfectly satisfactorily. There is an effect on performance under fatigue loading which will be considered later.

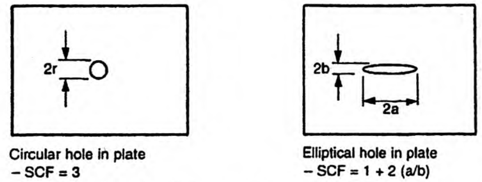


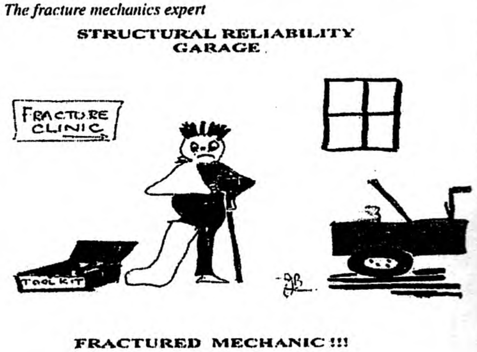
Fig 5. Stress concentration effects at holes

For an elliptical hole with its major axis perpendicular to the direction of applied stress, the stress concentration depends on the aspect ratio of a/b and is given by:

$$SCF = 1 + 2(a/b)$$

As the minor axis decreases, so the SCF increases and in the limit, if the elliptical hole degenerates to a crack with b = 0, infinite stresses are predicted locally. These results are derived from analyses that assume the material properties to be linear elastic. Clearly, in real materials, infinite stresses cannot occur and, in practice, either the material will yield or it will break. If it fractures, the crack tip moves forward and there is the possibility that the same situation will prevail, resulting in an unstable fracture propagation to break the complete member. The analysis of stress fields around cracks and the implications of this on material performance and structural integrity are part of the subject of fracture mechanics originated by A.A. Griffith[6] and developed extensively by G.R. Irwin[7] in the first instance. Of course, 'e-nerd' is an expert on fracture mechanics, as shown in Fig 6.

Fig 6. The fracture mechanics expert



Analysis of cracks subject to remote tensile stress in an elastic medium do indeed predict infinite stresses at the crack tip, and the stresses perpendicular to the crack plane ahead of the crack have the general form shown in fig 7:

$$\sigma = K/\sqrt{(2\pi r)}$$

where K describes the gradient of stress away from the crack tip singularity and is a function of the applied stress, crack size, and geometry, as follows:

$$K = Y\sigma \sqrt{(\pi a)}$$

where Y is a non-dimensional factor to account for the crack shape and geometry

σ is the remote stress

a is a measure of the crack size.

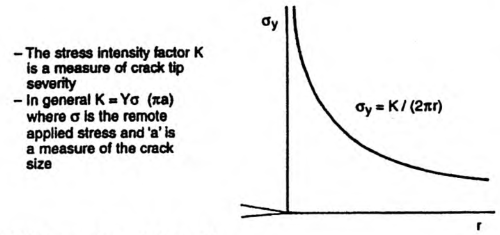


Fig 7. The stress intensity factor K

The original proposal of linear elastic fracture mechanics was that fracture would occur at a critical value of the stress intensity factor which was a material-dependent property known as 'fatigue toughness'. While linear elastic fracture mechanics is valid for ultra high-strength brittle materials, the situation is considerably more complex in materials that yield, such as structural steels.

The general effect of yielding and plasticity is to increase the crack tip severity conditions compared with the linear elastic case because yielding allows the crack faces to separate more. This can be represented by the concepts of crack opening displacement or by the J contour integral.

A convenient method for practical assessments of the significance of defects in elastic plastic materials is the use of the R6/PD 6493/BS 7910 failure assessment diagram[8] shown in Fig 8. This diagram allows the interaction between failure by fracture and failure by plastic collapse to be assessed in a convenient manner. The ordinate axis is the fracture axis on which is plotted the ratio of applied crack tip severity to fracture toughness

$(K_r = K_I/K_{mat} = \sqrt{(\delta_I/\delta_{mat}} = \sqrt{(J_I/J_{mat})}$.

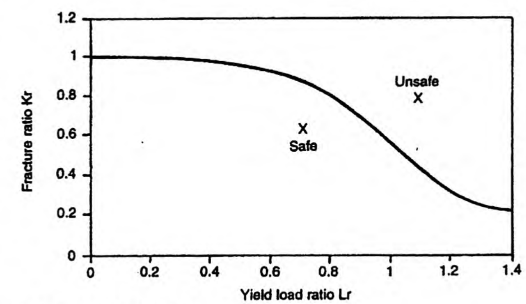


Fig 8. The R6/BS 7910 failure assessment diagram

The abscissa axis is the plastic collapse/yielding axis on which is plotted the ratio of applied load to yield/collapse load or net section stress/yield strength ($Lr = \sigma_{net}/\sigma_y$). To assess the acceptability of a particular defect, the values of K_r and L_r are calculated for defect size, geometry and loading conditions concerned, and a point is plotted on the assessment diagram. If the point lies inside the assessment curve, the defect is deemed acceptable, but if it lies outside the curve, the defect is not acceptable. It should be noted that only the elastic value of the applied stress intensity factor is needed for the ordinate axis, as the shape of the curve represents the effect of plasticity in increasing the crack tip severity and reducing the permissible elastic part of the total crack tip driving force. Indeed, structure and material-specific assessment diagrams can be calculated using elastic and elastic-plastic FE analyses to determine the J integral values at the same load and plotting the ratio (J_e/J_{ep}) against σ_{net}/σ_y. The standard assessment diagram shown in Fig 8 represents a lower bound to common material and defect geometry combinations.

For the important case of cracks at the toe of a weld, there is a magnification of the stress intensity factor due to the stress concentration effect of the weld. This can be expressed by the non-dimensional factor M_K which is a function of the ratio of the depth of crack-to-plate thickness a/T, the ratio of length of attachment to thickness, the weld angle, and the weld toe radius. An equally important case is that of partial penetration butt welds or fillet welds where the unpenetrated region between the welds can be considered equivalent to a crack-like defect, as shown in Fig 9. This figure also shows the results of FE analysis calculations at UMIST by Motarjemi et al.[9] expressed in terms of M_K against 2a/W, so that the elastic stress intensity factor can be calculated from:

$$K = M_K \sigma \sqrt{(\pi.a.\sec(\pi a/W))}$$

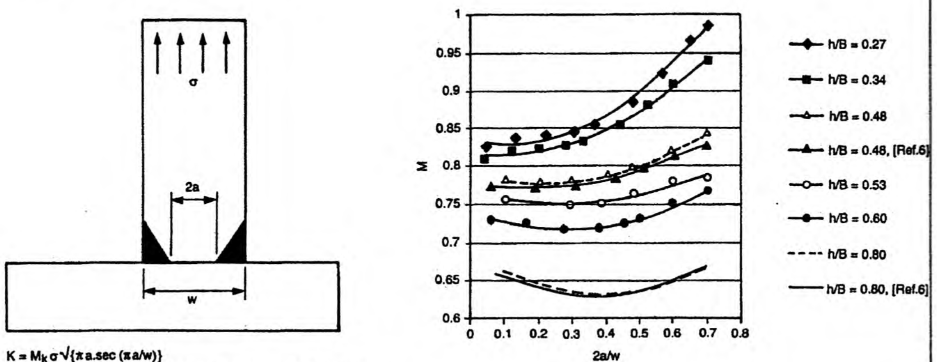


Fig 9. Stress intensity factors for partial penetration butt welds

The important effects of size on fracture are that, if a structure is scaled-up in size, any inherent flaws will also be scaled-up and there is an absolute effect of defect size on crack tip severity. Furthermore, the fracture toughness properties of thick sections will usually be worse than thin sections owing to the effects of the slower cooling rate during manufacture. Similarly, large weld beads will usually have poorer toughness than smaller beads, so that it may not be possible to use very high heat input, high production techniques if high toughness welds are required.

The significance of these results will be demonstrated using an example for acceptability of partial penetration butt welds later in the address.

Brittle fracture

Most practising engineers do not wish to get involved in detailed fracture mechanics analyses, but should be aware that they must take precautions against brittle fracture failure. For normal design purposes the route to follow is the selection of adequately tough materials by the use of the Charpy V-notch impact test. The Charpy test does not actually reproduce the service phenomenon of fractures occurring at low applied stress, but it is a quality control test on the susceptibility of a steel to brittle cleavage fracture at different temperatures. In the period from the 1940s to the 1960s, it was generally found that the steels involved in brittle fracture failures had low values of Charpy energy absorption at the service temperature involved. Fortunately, the quality of weldable steels has improved enormously over the past 30 years or so.

The provisions in the various codes and standards to avoid brittle fracture are based on a mixture of experience, large-scale tests, and fracture mechanics analyses. They give either maximum thickness of different grades of steel for different minimum service temperatures or a requirement for minimum Charpy energy absorption at the minimum temperature as a function of thickness and yield strength. It is generally observed that the risk of fracture is higher in thick-welded constructions. This is because of the general aspects of size effects, i.e. the tendency for lower fracture toughness in thicker sections owing to both metallurgical and constraint effects, the greater probability of more and larger flaws in thicker weldments, and effects of residual stresses and stress concentrations. The fracture mechanics analyses used to underpin Code requirements require various assumptions about input

data, and these have to be tuned to give results consistent with practical experience. The differences between different Code requirements usually arise from the different underlying assumptions made. The requirements of the Codes for bridges (BS 5400) and for buildings (BS 5950) have been reviewed recently in a joint research programme between TWI and UMIST, and some recommendations for modifications have been made to take account of different stress concentration effects and improved correlations between Charpy impact test results and fracture mechanics toughness tests. At the same time, complex recommendations have been put forward by others for Eurocode 3, based to a large extent on fracture mechanics calculations. One of the key assumptions is the size of initial defect that might be present, and the various approaches all assume initial defects that are a function of the thickness, but different functions have been used by different parties.

The important point for practising engineers is to ensure that the minimum requirements of existing Codes are met with respect both to material selection for Charpy test properties and to quality of workmanship and freedom from harmful defects in welds. Expert advice should be taken for cases where the Codes do not appear to be appropriate.

Fatigue

The basis of conventional design against fatigue is the use of series of S-N diagrams based on experimental test results for different types of welded detail. These are included in slightly different ways in Codes and standards such as BS 5400 for bridges, BS 7608 for fatigue of welded structures, and Eurocode 3 for steel structures. In the UK standards, the results of a series of tests on a particular welded geometry to determine life to failure (N) at different stress ranges (S) are analysed to find the mean and standard deviation. The design curves for each category of weld geometry are based on the mean minus two deviations results, and each category is allocated a letter such as C, D, E, F, F2, G, S, and W. The curves take the form of straight lines with slope -1/3 (except C) on logarithmic axes, down to the fatigue limit at stress ranges below which an infinite life is obtained. In the Eurocode 3 approach, based on the recommendations from the International Institute of Welding, the S-N diagram with logarithmic axes is divided into a series of equally spaced parallel lines (slope -1/3), and the same experimental results are then allocated to appropriate spaces between the lines. The nearest line under the results (also mean minus two standard deviations) is used to define the design category. In some approaches, a change of slope to -1/5 is incorporated in the S-N diagrams between 5 x 10^6 and 10^8 cycles where variable amplitude loading is involved, with the horizontal fatigue limit extending beyond 10^8 cycles.

All of the experimental tests on which the design guidance was originally based were carried out on specimens of a size convenient for testing machines available. The majority of tests on welded joints were on specimens made up from plates of 12.5mm (0.5 in) thickness. In the early days of this research, the major effort was to understand the effects of geometry, of welding residual stresses, and of variable amplitude loading. It came as a something of a surprise when some larger-scale joints were tested, and it was found that they failed at shorter lives than the standard specimens for the same stress ranges. Work on tubular joints of the major offshore structures, carried out in the 1970s and 1980s, showed that it was necessary to take account of both the stress concentrations due to bending of the walls of the tubular members and the deterioration in performance with increasing thickness. This led to modifications to standard fatigue design

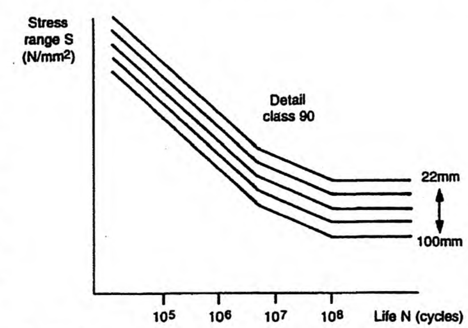


Fig 10. Effect of thickness on S–N design curves for welded joints (schematic)

curves, as shown in Fig 10 where, for a given weld category, a series of lines are given parallel to the basic curve but displaced to lower stresses:

$$S_T / S_{22} = (T / 22)^{0.25}$$

Considerable additional insight into size effects in fatigue can be obtained from the application of the fracture mechanics Paris Law for fatigue crack propagation to this topic:

$$da/dN = C(\Delta K)^m$$

where ΔK is the range of stress intensity factor

C and m are material constraints

da/dN is the crack growth rate/cycle.

Typical values for C and m for steels are 2×10^{-13} (N, mm units) and 3 (dimensionless), respectively.

Fig 11 shows a fillet welded connection at two different scales, with the same fatigue stressing applied to the two main plate thicknesses, T_1 and T_2. The stress concentration factor at the weld toe depends on the ratio of the weld toe radius ρ to plate thickness T. Since the weld tow radius is a feature of the welding process, procedure and consumables, typically having a value of 1 - 2mm, increasing the plate thickness reduces the ratio ρ/T and increases the local stress concentration factor. For the same remote stress in the plate, this means that fatigue crack initiation from the weld toe will occur earlier with the thicker plate than the thin. Once the crack has initiated, there is a short period known as 'short crack growth' before the fracture mechanics crack propagation relationships apply for cracks of the order of 0.5mm length. The second effect of scaling-up the thickness is that the size of the stress concentration zone increases, as shown in Fig 11. Thus at any given crack size the local stress level is higher, or at any local stress level the crack size is bigger, in the thicker joint. Since the rate of crack propagation depends on the range of stress intensity factor raised to the power 3, the crack in the thicker section grows faster at all stages. Although the crack may have further to grow to final failure in the thicker section (depending on net cross-section and fracture toughness), the earlier initiation and faster crack growth mean that the thicker section fails sooner. This is the explanation for the experimentally observed thickness effect. It should be noted, however, that it applies to welded joints with weld toe-type stress concentrations and does not apply to welded joints without stress concentrations, unless there are more inherent defects present. Thus the fatigue size-effect should be applied to weld details with surface attachments and will be much less severe for in-line butt welds, particularly if ground flush. Furthermore, the implication is that the thickness effect can be counteracted by grinding the weld toe to a specified radius to increase the effective ρ/T.

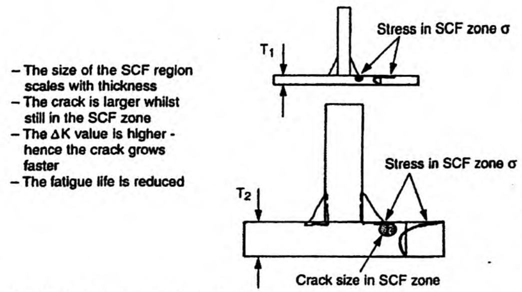


Fig 11. Explanation of fatigue size-effects

Application examples

Welded joints: the transfer girder

As an example of the effects of scaling-up plate girders, a series of major transfer girders was designed to support a very large multistorey building over a large, clear open space at the ground level. The transfer girders were designed as continuous plate girders of length 75m, supported on three columns and carrying columns at much closer spacing for the main building above. The spans

and loads were so great that the plate girders were designed as 5m depth with webs and flanges of up to 200mm thickness over the central column. The steelmaker had confirmed that excellent properties could be supplied in thickness up to 200mm, and the mill certificates showed that Charpy energy absorptions well in excess of 100J could be obtained at 0°C. In fact, the design at the central column involved the girder web being split to allow the column flanges to extend up to the underside of the top flange, as shown in Fig 12. Because of the thickness involved and to save costs, the designer decided to specify partial penetration welds, not only at all the web-to-flange joints but also at the in-line flange butt welds. The welds were designed originally on the basis of required throat area. No consideration had been given to the effect of the absolute dimension of the unpenetrated region between the two welds from either side of the joint or to any fracture toughness requirements for the welds. Furthermore, the web-to-flange welds for the central column had been designed for shear requirements from bending of the column, and no thought had been given to the behaviour as part of the main transfer girder close to the top flange.

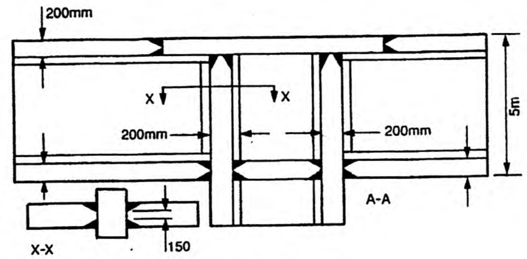


Fig 12. Transfer girder over central column

The situation as designed (and manufactured!) originally is shown in Fig 13(a) with a depth penetration of only 25mm, leaving an unpenetrated dimension of 150mm. The applied stresses close to the top flange were calculated as 180N/mm². From the dimensions of the unpenetrated region and the use of results shown in Fig 10, a BS 7910 fracture mechanics assessment was carried out which resulted in an estimate of required fracture toughness of about 11000N-mm$^{-3/2}$.

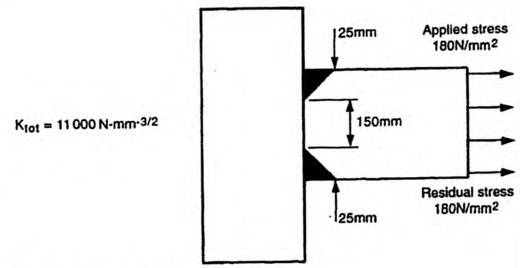


Fig 13(a). Web/flange as designed

This was clearly far higher than could be expected for the submerged arc welds used. The designers were persuaded to have their own checks carried out, and although different fracture mechanics procedures from a different country were used, it was accepted that remedial action was necessary. Over the top half of the column web-to-flange welds within the transfer girder height, the welds were gouged out to give an increased depth of penetration of 80mm, leaving a reduced lack of penetration of 40mm, as shown in Fig 13(b). The fracture toughness requirement was now reduced to 2500N-mm$^{-3/2}$ and, on the basis of Charpy test results on the weld material used for the repairs, this was considered acceptable. The same type of analysis was carried out on all the partial penetration welds in the original design, and a significant number required repair because the absolute effect of size of lack of penetration had not been appreciated.

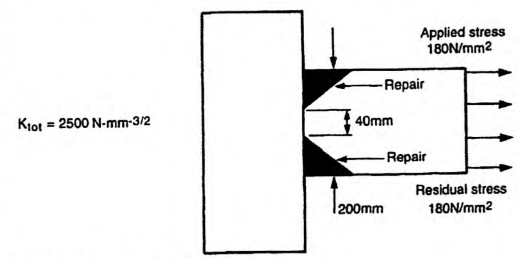


Fig 13(b). Modified web/flange detail

On the other hand, when the stresses are predominantly shear, it is usually sufficient to design welds on the basis of net cross-sectional area. This is the position with the web butt welds at the teeth of castellated beams where, for the web thicknesses involved, full penetration welds are not essential and the welds can be designed on the basis of throat area.

Effects of proof loading: the London Eye

One way to improve both the fracture and fatigue resistance of welded structures is by the use of proof loading. This requires loading of the structure in the same direction as the service loads to stress all parts to a level 10 - 20% above the maximum stresses that will be experienced in service at a temperature where the fracture toughness is sufficient to avoid failure during the proof test. Such a loading is, of course, applied routinely to pressure vessels before they go into service.

The effects of proof loading are twofold. First, in any locations where high tensile welding residual stresses are present, the proof loading causes local yielding and equalisation of the stress distribution. On unloading, the effect is that the original high residual stresses will have been relaxed by the effects of this mechanical stress relief. Secondly, there is the effect of warm prestressing. The initial proof loading causes local yielding and a local plastic zone at the tips of any defects present. On removal of the load, the surrounding elastic material forces the crack tip region to form a local compressive plastic zone, and this effectively reduces the stress intensity factor or crack tip severity on subsequent loadings. Alternatively, warm prestressing can be thought of as increasing the effective fracture toughness at lower temperatures, compared with the results if initial loading was applied at the lower temperature. The net result is that successful completion of the proof loading under ductile conditions will usually protect the structure on subsequent loadings up to normal design conditions at lower temperatures.

This principle has been employed on numerous occasions in the past where the materials used for a structure have been suspected of having fracture toughness levels below those desirable. More recently, proof loading was used as part of the manufacturing process for the spindle of the Millennium Wheel or London Eye which, at 135m diameter, will be the fourth highest structure in London. The spindle consists of a series of cylindrical castings of diameter about 2m and thicknesses up to 200mm - 300mm welded together with circumferential butt welds to form the complete length of over 22m. The spindle is supported from one end only and is a safety-critical element for the whole assembly. Castings of this size inevitably contain imperfections of various types, and the slow cooling rates of the mass and thickness involved make it difficult to obtain high fracture toughness values. Tests carried out on samples from the castings showed that, while in general reasonable toughness values were present, there were occasional low results. Fracture mechanics analyses using the methods of BS 7910 showed that critical flaw sizes were generally acceptable and well above the acceptance levels by NDT, but that, for the occasional low toughness values, the limiting flaw sizes were relatively small compared with the size of the spindle. It had always been intended that the spindle would be given a proof load test, and the successful completion of this has ensured that there need be no concern if any minor residual defects coincided with low toughness.

Conclusions

The many beautiful and very large structures created by structural engineers have often become possible through widespread use of FE analysis and developments in welding of structural steels. Increasing the size and scale of structures usually involves heavier section thicknesses which tend to have poorer properties than thinner sections.

Modes of failure should be divided into those that cause failure of a complete cross-section and those that cause progressive failure from stress concentration regions or from defects. Heavier sections in plates, castings, welds, etc., tend to contain more and larger defects. This makes them more susceptible to failure by fracture and fatigue. There is an absolute effect of defect size that needs to

be taken into account in scaling-up structural components, and practical examples of this with structures have been described.

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VISIT TO THE SCOTTISH COURAGE BREWERY, MOSS SIDE

Alison Jackson, Scottish Courage

7 December 1999.

A group of members and visitors visited the Scottish Courage Brewery during the evening of 7 December 1999. The Brewery provided a short talk on the brewing process and the history of the site and then took us round all stages of the operation in small groups. The groups were small to enable us to hear our tour guide over the noise of the various processes.

The tour started in the area where the hops are prepared as a mash, boiled and filtered. We then passed on to the area where yeast is added to the mash. The brewery makes several Scottish Courage beers, and brews Fosters under licence. The different beers and lagers each have their own yeast strain, so the area where the yeasts are kept included several varieties. Each strain of yeast can be used a few times before being replaced.

The tour then moved on to the canning and kegging area. The canning machines are hypnotic, as the cans twist and slide from upside-down (through the washing process) to upright (for filling) and back again - the cans have to be upside-down for pasteurising. They are then packed in parcels of 24 with a cardboard base and plastic wrap before being palletised in an amazing robotic device which swaddled the whole pallet-full in plastic. The kegs were also tipped upside-down for washing and then turned the right way up for filling.

The tour was finished with an opportunity for questions and then we were offered a delicious buffet and an open bar!

A REMARKABLE VARIETY OF ANALYTICAL APPLICATIONS AND ENGINEERING SOLUTIONS - MASS SPECTROMETRY

Martin Elliott, formerly Technical Director, VG Gas Analysis Systems Ltd, Winsford, Cheshire.

11th January 2000

1. INTRODUCTION

The lecture attempted the difficult task of giving an understanding of a rather complex type of scientific instrument, while at the same time covering its many and varied applications. So, before describing three out of many applications, it was necessary to answer the basic questions -

1. What IS a Mass Spectrometer? 2. How Does a Mass Spectrometer Work?

An application of a relatively simple type of mass spectrometer to Process Gas Analysis was then described in detail. Armed with this knowledge it was then possible to describe other more complex types of mass spectrometer and some applications to, for example, such areas as geology and the pharmaceutical industry.

Limited space makes it impossible to include all the lecture material in this report.

1.1 Mass Spectrometry - The Basics

A mass spectrometer can be defined as "a device which performs an analysis of the properties of some physical material based on the different masses, or weights, of the atoms or molecules of which it is composed".

Such atoms may, for example, be those of nitrogen (N, atomic weight 14), oxygen (O, atomic weight 16) or uranium (U, atomic weight 235). Sometimes isotopes of atoms (nitrogen has a small number of atoms of mass 15 as well as the majority of mass 14) are the focus of attention.

Molecules may range from the simple constituents of air such as nitrogen (N_2, of mass 28) and oxygen (O_2, of mass 32) to simple organic compounds (for example butane gas, C_4H_{10} of mass 58 or decane, $C_{10}H_{22}$, a liquid of mass 142), to more complex organic compounds like cholesterol of mass 386 or much larger biologically-important protein molecules which can weigh tens or even hundreds of thousands of atomic mass units. Nowadays even such huge molecules are accessible to study by mass spectrometry.

For any material containing species of various masses, there is a good chance that a mass spectrometer will be able to separate and measure these lighter and heavier components and learn a great deal about the nature of the material.

1.2 Mass Spectrometry - Applications

Mass spectrometry is in fact a technique with an amazing variety of applications, including:

Analysis of Gas Mixtures	Leak Detection
Geological Dating of Rocks	Pesticide Analysis
Identification of Proteins	Palaeontology
Pharmaceutical Applications Monitoring of Vacuum	Forensic Science
Monitoring Industrial Atmospheric Contamination	Detection of Drug Abuse in Sport

Detection and Measurement of Impurities in Liquids, Gases and Solids

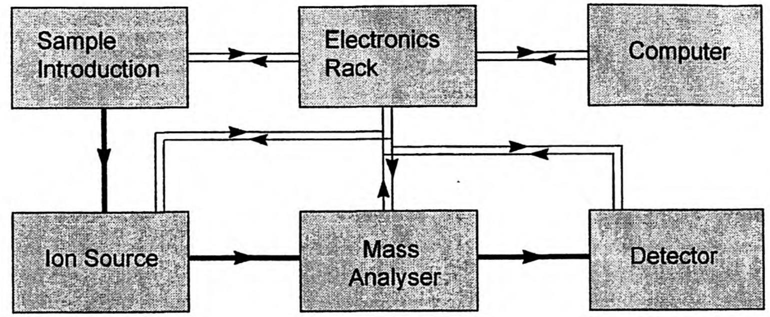
of which there was time to describe only the first three in any detail.

1.3 Mass Spectrometers in Practice

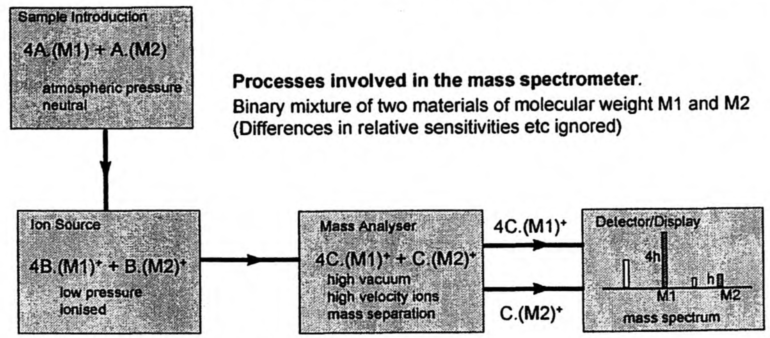
In physical terms a mass spectrometer can vary between something the size of a grapefruit, plus a small box of electronics, costing as little as £4000, to an instrument which would fill a large living room and might cost £300,000 or more.

1.4 Mass Spectrometry - Basic Operation

The diagram below shows the basic components of a mass spectrometer



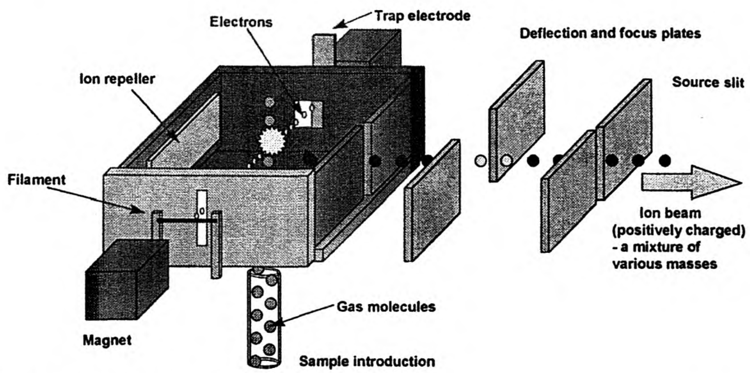
The next diagram shows (apart from the electronics and the computer) the functions which the component parts perform in the case of analysis of a binary mixture of two materials of molecular weight M1 and M2; normally the ratio will be unknown, but we use a ratio 4:1 as an example.



The processes are somewhat simplified, but the diagram shows how the relative numbers o molecules (4A and A) in the sample are maintained (i) through the process of sample introductio and ionization (4B and B positive ions are produced), (ii) mass analysis (4C and C ions traverse th mass analyser and are separated), and (iii) detection, ensuring that the mass spectrum finall produced shows peaks of height 4h and h. So from this mass spectrum one can deduce that th original mixture contained the two materials in a ratio of four to one, i.e. 80% of species M1 and 20% of species M2. Deviations from this exact proportionality do in fact occur, but they can be allowe for by calibration with a known mixture.

2. A MAGNETIC SECTOR MASS SPECTROMETER FOR GAS ANALYSIS

To complete the picture of how one type of mass spectrometer works, the operation of the ion sourc and the mass analyser and the detector of a relatively simple type of instrument is described. Th following diagrams show these two aspects.

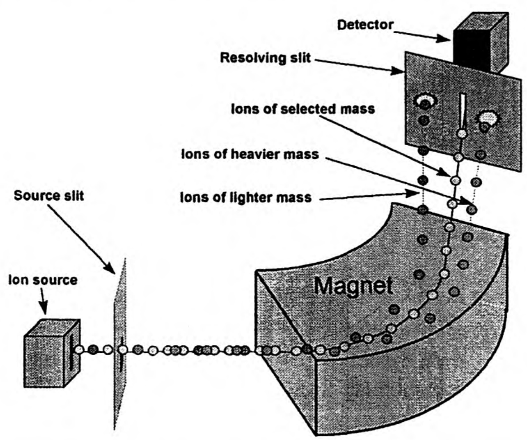


2.1 Ionization / The Ion Source

The diagram shows an ion source where a gas sample is converted into a form suitable for mass analysis. As it is a gas already, we do not have to vaporise the sample, but we must (a) reduce the pressure drastically, because the mass spectrometer has to operate at a very high degree of vacuum, and (b) change the atoms or molecules from electrically neutral particles into electrically charged ones. A common way to do this is to use a beam of energetic electrons to bombard the molecules. The electrons are produced from a heated filament and formed into a beam with the aid of a magnet.

Paradoxically, electrons are removed in the collision and the molecules become positively charged ions, but still of essentially the same mass. Because the collision is relatively violent, the positive ions are in what is known as an excited state, and some may fall apart. However, one of the fragments will always retain the positive charge. Hence some molecular ions of the same mass as the original particles, and also some fragment ions of lower mass, will usually be produced.

The ions are extracted from the source by the electric field between the ionization chamber and the deflection and focus slits, and are passed into the mass analyser as a narrow high energy ion beam. The beam now contains ions of various masses characteristic of the original material.



2.2 Mass Analysis.

In the magnetic sector mass spectrometer, the ion beam enters a magnetic field where ions of different masses travel on circular paths of different radii according to their mass. In the diagram only

the ions of one selected mass are able to reach the detector. Usually the magnetic field strength will be varied from low to high values so that ion beams of successively higher masses reach the detector in turn, resulting in a mass spectrum as shown in the earlier schematic diagram.

2.3 Detection and Display.

In many cases the detector will be a simple metal plate at which the ions give up their charge resulting in an electrical signal. The successive ion beams create a series of signals of different intensities, which are displayed as the mass spectrum.

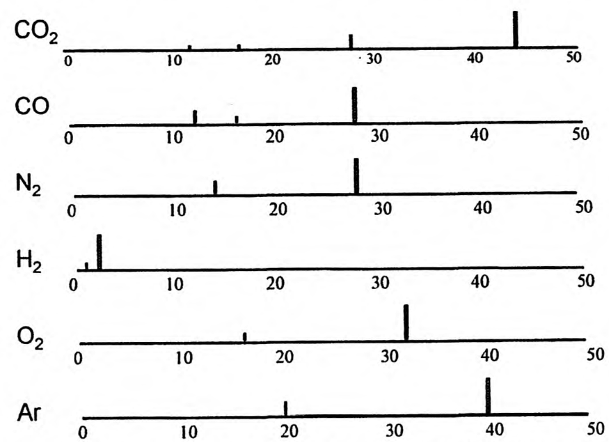
3. Applications to Process Control of Gas Mixtures

Even within this small sector there are many applications, such as monitoring of fermentation processes, ethylene production, coal gasification and blast furnace gases. The last application will be described in more detail.

In process control there is always plenty of sample; one litre per hour taken from the process stream is neither here nor there. For blast furnace gases, the gas mixtures are not too complicated - sometimes a dozen components but more usually four to eight. What the customers need for this application is accuracy of composition and reliability of operation. They want to set up the mass spectrometer in a heavy industrial environment and have it monitor (unattended) the composition of several gas streams around the plant night and day, week in week out, so that they can control the flow of fuel gases or feedstocks to optimise the process. This can save them huge amounts of money.

3.1 Blast Furnace Gas Analysis

A process mass spectrometer would typically be installed on a blast furnace for measuring the following components in the "top gas": CO, CO_2, N_2, H_2, O_2 and Ar. The mass spectra are shown below; note that all the spectra have a large peak at the molecular mass, but they all have smaller peaks at lower masses. These are usually "fragments" as described earlier in 2.1, or sometimes due to multiply charged ions; these lower mass peaks can sometimes be very useful.



We can use the unique molecular mass 2 peak for hydrogen and similarly we can use 32 for oxygen, 40 for argon and 44 for CO_2. But for CO and N_2 we cannot use mass 28 because both components have their largest peaks at that mass. Fortunately the mass 14 peak is unique to N_2 and can be used to find the nitrogen content, and although mass 12 occurs in the spectrum of CO_2 as well as in CO, some simple arithmetic is sufficient to extract the content of CO. A combination of unique peaks (making the analysis of the data very easy) plus a few overlapping ones is typical in mass spectrometry.

Analysis of furnace top gas is continuous, day and night over months of operation, and the analysis time is 6 seconds. The performance of the mass spectrometer is typically assessed on a similar gas mixture, as shown below:

Component	Units	Known	Measured			
			Mean	Std Dev	Minimum	Maximum
CO_2	%	26.00	26.028	0.015	25.995	26.073
CO	%	24.00	23.946	0.0209	23.891	23.995
H_2	%	4.01	3.986	0.004	3.979	3.995
N_2	%	45.99	45.999	0.0174	45.959	46.048

We can see the sort of results obtained - this is a very precise analysis. The errors in analysis (taken as twice the standard deviation) are around 0.04% of the total, or expressed in other terms, about 0.2% of the amount present.

4. FURTHER APPLICATIONS - AND DIFFERENT TYPES OF INSTRUMENT

Two more applications in very different fields are described. The first, in geology, uses a relatively simple mass spectrometer of the same type as previously described. The other uses a complex system of a very different kind. So it is timely to describe briefly the many other different types of ion sources, mass analysers and detectors which can be used in different fields

4.1 Types of Ion Source / Ionization Method

Not all materials can be analysed by mass spectrometry, but a good fraction can be, whether they normally exist in gas, liquid or solid form. Some materials like inorganic solids have to treated very drastically in order to vaporise them into a form suitable for ionization; others like delicate biological molecules have to be treated with extreme care and "soft" ionization methods found so that they are not destroyed in the vaporisation / ionization process. Some of this variety is shown below:

Electron Bombardment (EI)	}	
Chemical Ionization (CI)	}	sample is a gas
Field Ionization (FI)	}	
Photoionization	}	
Electrospray (ES)	}	
Thermaspray	}	sample is usually a liquid
Inductively Coupled Plasma (ICP)	}	
Laser Desorption (LD)	}	
Field Desorption (FD)	}	
Fast Atom Bombardment (FAB)	}	sample is usually a solid
Nuclear Recoil Ionization	}	
Spark Source	}	
Glow Discharge	}	

4.2 Types of Mass Analyser

There is again a large variety of different types of analyser. Most of them involve forming the ions into a beam of some kind, but there the similarities end. In some, there are no magnetic fields; in the quadrupole there are just rapidly varying electric fields. In the Time-Of-Flight analyser, ions of different mass are separated according to their time of arrival after drifting along a tube.

Magnetic Sector
Double Focusing (Electric and Magnetic Sectors)

Quadrupole (Q)
Monopole
Ion Trap

Time of Flight (various kinds) (TOF)

Fourier Transform Ion Cyclotron Resonance (FT-ICR)
Omegatron

Some of the above can be combined in tandem systems e.g. Q-TOF.

4.3 Types of Detector

Faraday Bucket

Electron Multiplier, or variations such as Microchannel Plate or Scintillation Detector
Photographic Plate

Charge-Coupled Solid State Devices

So we have 13 types of ion source, 8 types of analyser and 6 types of detector, and most of the 624 combinations could in principle be used! In practice there are dozens of combinations in use, for the very good reason that specific ones are essential for certain applications, and in many other applications one type of system will give vastly better performance than the others. Mass spectrometry is versatile!

4.4 Application of Mass Spectrometry to Age Determination of Rocks

Used by geologists, this is a very different type of application, but uses a mass spectrometer quite similar to the blast furnace application. The object is to determine the age of rocks, which may be anywhere in the range from thousands of years to over 4 billion years.

All the mass spectrometry methods of rock dating are based on radioactive decay. When a molten rock solidifies, its composition is then basically fixed for all time. Many rocks, including volcanic rocks such as feldspar, contain potassium; the stable isotopes of potassium of mass 39 and 41 remain unchanged in the rock but the radioactive isotope of mass 40, present in very small proportions, i slowly converted into argon gas which is trapped in the rock. The quantity of argon-40, relative to the stable potassium, is therefore a measure of the age of the rock. In principle, the geologist has only to melt the rock and measure the argon-40 released and to measure the potassium content by othe means.

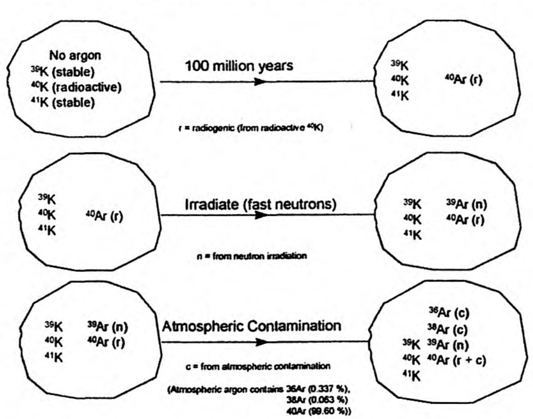
In practice, some of the stable potassium-39 is converted into argon-39 by neutron irradiation in nuclear reactor. The argon released in the mass spectrometer inlet system, typically by vaporising small area with a pulsed UV laser, now contains argon-40 from the radioactive decay and argon-3 representative of the original concentration of potassium in the rock. It then becomes a relatively simpl task for the mass spectrometer to measure the ratio of the two argon isotopes at mass 39 and 40.

A further sophistication in the technique allows measurement of the argon 38 level to make correction for any atmospheric contamination. The only problem arises because of the extremel small quantities of gas available; in the process gas application, several millilitres of gas wer available. There is typically one billionth of that amount available for the rock analysis, requirin very special attention to the vacuum system.

Because the ratio of Ar-39 to Ar-40 can be measured to 0.1%, the age of a 100 million year old roc can be determined to better than 1 million years, and so on. An example was shown of an ag determination by the Open University of a meteorite from South Africa which was shown to have h the earth 2018 + 10 million years ago.

The radioactive decay / irradiation scheme is shown in the diagram overleaf.

Age Determination of Rocks by the Potassium - Argon (K - Ar) Method



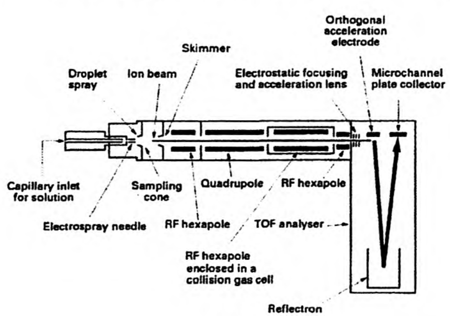
4.5 Application of Mass Spectrometry to Biological Molecules - Human Haemoglobin

There are many applications of mass spectrometry to organic molecules, starting with hydrocarbons in the 1950's through synthetic chemicals and drugs in the following decades to more and more complex, large and delicate molecules in the 70's, 80's and 90's. There was a continuous search for methods of studying biologically interesting molecules, many of them very large. The search occupied two decades or more and required a number of major new developments in ion sources, mass analysers and detectors for a successful outcome.

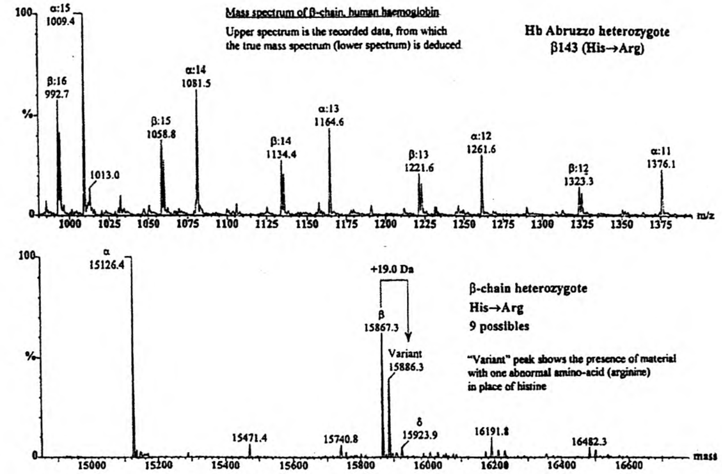
Simple molecules can often be vaporised in a reservoir and the vapour allowed to flow into the ion source. More delicate molecules fall apart under this treatment, so direct vaporisation inside the ion source was the next technique to be used, but it was still too violent for many biologically interesting molecules such as haemoglobin, whose principal components alone have molecular weights in the region of 16000. The most successful ion source for this type of molecule has turned out to be the electrospray; a spray of droplets is introduced into the ion source region with the molecules dissolved in the liquid. Under the right conditions, the solvent evaporates leaving positively charged molecules to enter the ion source proper.

Large magnetic sector instruments, usually thought to have the best high mass performance, were the natural instruments to try out this new source, but success was very limited. After a number of twists and turns in the story, the quadrupole analyser was found to be more suitable despite its apparently much more restricted high mass performance. One of the favourite combinations currently used combines a quadrupole analyser in tandem with a time of flight analyser, the so-called Q-TOF.

A diagram of this system, manufactured by Micromass UK Limited, is shown in the diagram below.



Below is the spectrum of the ß-chain of an abnormal human haemoglobin obtained on such a system. The anomaly is in the peak shown as "Variant", 19.0 Da (molecular mass units) higher than normal, due to one of the normal amino-acids in the protein, Histine, being replaced by Arginine in some of the ß-chains. This genetic variation is similar (in that there is just one amino-acid change) to the one which is responsible for sickle cell anaemia.



The technique, and similar instrumentation, is now widely used for many other less demanding, but nevertheless important, applications in the pharmaceutical industry

5. SUMMARY

Three of the many applications of mass spectrometry have been described, together with one or two of the many ways in which different forms of hardware can be put together to tackle different applications. These are examples of the achievements of thousands of workers in the field who have found it an extremely interesting and fruitful field in which to work. As specialised as it may appear, it is far from a fringe activity, with a large international market in many industrial and research fields amounting to around £500 million per annum.

There was no time to compare the importance of mass spectrometry to other analytical techniques, but it certainly turns out to be the method of choice for a whole variety of applications. One of the reasons is the very specific nature of the information produced, in that mass 40 is almost certainly argon and mass 2 is certainly hydrogen. There may be uncertainties at times in the intensities of the signals measured, but by and large you know precisely what you are looking at.

It is interesting to note that many of the advances since 1950 have been made in the UK, especially in the Manchester area, and a substantial fraction of the world output of mass spectrometers is currently produced in Manchester and North Cheshire.

ACKNOWLEDGMENTS

I am indebted to dozens of colleagues for the knowledge gained by working with them in the field of mass spectrometry over many years, which formed the basis of this lecture. I should like to thank several people for specific help in preparing the talk - Bob Bateman of Micromass UK Limited, Chris Walker of VG Gas Analysis Systems Limited and Mike Lynch of Mass Analyser Products for general information, Simon Kelley of The Open University for the geochronological application data and Brian Green of Micromass UK Limited for the haemoglobin application data.

HEALTH AND SAFETY IN PAPERMAKING

Bob Hudson

1st February 2000

Papermaking has become one of the most dangerous industries in the United Kingdom, overtaking construction, agriculture, etc. In the last nine years, there have been six deaths in the North West alone. However, over the last couple of years, the papermill at Ramsbottom has been changed from what was really a night-mare into a centre of excellence in Europe.

Concern about safety in working environments can be traced back to the opening years of the nineteenth century which led to legislation. In 1802, Sir Robert Peel introduced the Health and Morals of Apprentices Act. This was the start of the principle that legal intervention was possible on behalf of a group of employees and of the principle that rudimentary health standards could be laid down and that a system of enforcement could be introduced. But, with the developments of new industries, legislation had to be constantly brought up to date. Of course such legislation did not cover those working at home.

In 1970, Lord Robins chaired a committee of enquiry which reported back in 1972 and found that the existing legislative framework did not prevent the unacceptable number of injuries and deaths that occurred every year. Much of the existing law was obscure, difficult to understand, and frequently out of date. There were many enforcement authorities with overlapping jurisdiction which caused confusion. The result from his report was the Health and Safety at Work Act in 1974, which in turn created the Health and Safety Commission with its enforcement arm, the Health and Safety Executive. The Act placed responsibility for safety on the employers, employees, self-employed and others including the public who may enter the premises. In 1987, the Treaty of Rome was amended and since then the European Commission has produced an accelerated output of Health and Safety Directives. This has come to a point where U.K. regulation is almost totally directed by E.C. directives. Recent legislation has moved from prescriptive controls to management oriented risk assessment based risk approaches. This recognised that each workplace is different and so flexibility is required, whilst maintaining the protection of workers. The employer is a person who should know what activities are being carried out in the workplace, and that also Health and Safety is a management responsibility which recognises the current belief that twenty per cent of all accidents are attributable to poor management.

Papermaking uses large capital equipment, and, for that equipment to pay back, it has to run twenty-four hours a day, and three hundred and sixty-five days a year. We run Christmas Day and Boxing Day, and effectively, apart from very short maintenance shut-downs, which can be as short as four hours a month, we do not close. That gives tremendous pressure to keep everything running and so to take short cuts. Some practices of the past would not be countenanced today. Engineers have a leading role in the management team and that gives them a special responsibility to take the lead in current developments and especially in Health and Safety.

While everybody on site has a responsibility for Health and Safety, engineers particularly carry a heavy responsibility because, at the end of the day, we can normally trace it back to the equipment and maintenance. You can assess risk, you can do a safe system of work, you can instruct people, you can train them, but that won't be enough. If something happens, and you are in court, that will not stand up. You will need written records. If it is not written down, legally, it was not done. While the safety adviser can advise, at the end of the day, it is your responsibility. You take decisions and you are going to have to stand by them. Rules must always be observed, and it is necessary to make sure that they are observed and confirmed with written records. It is necessary to

audit these records to ascertain that the actions have actually been carried out. Auditing these audits is written into the management job description.

We asked the question:- How do you get people to do the right thing safely? The answer we came up with is culture. We had to come up with a systematic set of actions and initiatives that would change the culture and that doing the right thing safely would be the natural thing for people to do. We started to turn around our site in 1996. Our goal was to improve the real life physical safety of all people on site. We were out to make people safer. This has to involve all people coming onto our site, including visitors, as you are responsible for their health and safety.

We had to change the mental pressure that people have from "How can I do that?" to "How can I do that safely?" We wanted that to be the automatic question, even at three o'clock in the morning when no supervisor was around. A safety committee listed the various issues that could be looked at, and were actioned to a point where we could be legally defensible. All the mills in the paper industry submit statistics on accidents to the Federation. However, on starting our new initiatives, we sank way down the list because we began to report near misses, treating every near miss as an accident. Even the guy who choked on his apple core in the canteen and had to be slapped on the back to cough it up was reported to the Federation.

In risk assessments, shop floor supervision and management were involved because, with engineering maintenance, there is almost an infinite variety of tasks which can be done. The audits formed a crucial element, being reviewed and reassessed by management at monthly risk assessment meetings to put things right. We picked specific items, or things to look at, which weren't safety oriented but we used them as a vehicle to communicate with people, to involve people, to educate them, to change their attitudes and to try and inherently make them more safe. Through our safety committee, we said we would survey attitudes, use sub-groups to spread involvement, enhance reporting of near-misses, put up safety notice boards, started workshop discussions, etc.

Contractors on site deserve the same respect for their health and safety. Now, every visitor to our site is inducted for health and safety before proceeding into the mill. Now this involves seeing a video. Safety passports are issued after a two day course for workers and a three day course for supervisors. Everybody working at the Stubbins site has a one day off site basic health and safety training each year. Now our engineers are sent away for NEBOSH training which is one day a week for twelve weeks. Involving people in other initiatives gave them a link with health and safety.

The HSE has long recognised the paper industry's poor record on health and safety. Now there is a triple initiative between the HSE, the management and the unions. Safety action plans have been drawn up. Every mill in the country now has to have a safety action plan to identify, in conjunction with the workforce, the areas of risk, to prioritise them, to define actions, to assign responsibility, and to assign resources. The action plan is up-dated every twelve weeks and sent to the HSE. The HSE treats every action plan as a contract, and will want to see how it is carried out, who is responsible, the job description, the resources are in place, and have concrete evidence that what is produced in the plan is actually real. If its contents are not being met and we fail to live up to our safety plan, that is a reason for prosecution, which is a different kind of attitude than what we had seen before. It is the move of the HSE to try and force health and safety in management in every paper mill and reduce what is even now still an appalling death toll and accidents.

DECOMMISSIONING WORK AT SELLAFIELD

Barry Kavanagh

14 March 2000

General

B241 comprises ten concrete tanks situated within the Sellafield Separation Area. Eight of these tanks date back to the early 1950s and are prestressed concrete structures approximately 15m diameter and 10m high. Each tank is surrounded by a reinforced concrete bio-shield wall. The remaining two tanks (PS7 and ST3) are conventional reinforced concrete tanks of similar size. The inventory of the six sludge tanks PS1 - 6 comprises a total of about $7500m^3$ of active sludge and supernate which BNFL will remove for treatment and disposal. The supernate contains up to 25 wt% ammonium nitrate. Organics in the form of odourless kerosene and tributyl phosphate and its degradation products are bound to the floc.

Refurbishment Project

The scope of the refurbishment Project included:-

a) Removal of redundant plant and equipment and general tank improvements.

b) Construction work to facilitate the later Overbuilding construction.

c) The construction of the Overbuilding foundations around the tanks, the provision of a seismically qualified concrete bund wall 2.4m high to provide secondary liquid containment and water retaining base within the bunded area.

The Bund Wall and Overbuilding foundations were designed as a continuous structure with no permanent movement joints. This was a unique approach for a 110 x 50m 'picture frame' foundation and wall on plan. This approach gave continuity to the foundation and wall and facilitated the Overbuilding which was erected remotely and jacked/slid over the tanks.

Tank Integrity Project

Tank strengthening measures were required to give additional assurance that the tanks would safely withstand the demands placed on them during Floc Retrieval. Tank strengthening work consisted of independent new steel tanks and foundations surrounding each of the 8 prestressed tanks. Each new tank was designed to resist the full hydrostatic head or liquor resuspension loadings. A leak management system is incorporated at the base of the steel tank to collect seepage. The new steel tank will provide an alternative hoop tension capacity to the primary wall and is not dependent upon the structural integrity of the existing prestressed structure. Foamed concrete was used to fill the two cavities between primary wall/shield wall and shield wall/steel tank. The steel tanks and the existing concrete structure modified by the grout infill has been assessed for all 50 year and 10-4 extreme environmental conditions including high and low temperatures. The tanks are expected to retain their serviceability post 0.125g DBE.

Floc Retrieval Project

The objective of this project is to ensure long term safe storage of the flocs. The elements of this being the Overbuilding containment and resuspension and retrieval of around 95% of the volumetric and active inventory in the sludge tanks; treatment and encapsulation via other BNFL plants and interim storage prior to final disposal in the NIREX repository. The waste in the tanks has settled out over many years and now consists of a thick sludge below an aqueous layer. This material will need to be thoroughly mixed before it can be pumped out, which will be achieved using a submersible pump, operating at a flow rate 100 litres/second. The pump itself will be lowered into the tanks from

a containment flask, about half the size of a double decker bus, which is moved from tank to tank on an air-film transporter on the Overbuilding operating floor.

Service Building

A Services Building, housing the main services for the Tank resuspension and retrieval process, has been built to the East of the Overbuilding and is now linked to it. This building is of structural steelwork with concrete and steel flooring. The Services Building is seismically qualified to protect the B241 Tank Complex and contains:- Personnel access facilities, Sub Changerooms, Control Room, Ventilation Fan/Filter Rooms, Electrical Equipment Rooms, Inactive Services, Offices/Stores/Workshops, Stack Sampling Room, Lift personnel/goods, Sentencing Cell/Tank and Sampling Facilities, Effluent Tanks. Construction of the Service Building on this sensitive plot required that an existing in-ground duct and service trench required bridging. The building was constructed over live services and an active duct, resulting in complex raft foundation details.

Overbuilding

Floc Retrieval Contract 5 includes the construction of an Overbuilding to enclose the tank and provide an operating floor to support the floc retrieval equipment. The structural steel framed Overbuilding which is seismically qualified to 0.25g has a 46m clear span over the tank complex and has the operating floor suspended from the roof. Due to safety considerations and the limited construction space available, the building was progressively erected in modules and slid over the tanks on a purpose designed slide track through sliding gates in the bund wall. The Overbuilding was extended three frames at a time with each slide moving the building 15 metres until the final slide of 23 frames when the Overbuilding weighed 2,800 tonnes. Now complete, the building is 110m long comprising a total of 23 portal frames. The building was temporarily lifted by 15mm to transfer the column loads onto PTFE lined sledges. Horizontal jacking was by 50 Te propulsion rams to each side of the building. This was followed by the erection of Stairtowers, Hoist Well and Link Building, which connect the Overbuilding to the Service Building, erected in a previous contract. The Contract also includes the installation of all the M, E & I facilities in the Overbuilding, Service Building, Link Building, Stairtowers and Hoist well.

The B241 complex is within the separation area, the steelwork erection zone lay outside and this work proceeded mostly without the normal controlled area constraints. Additionally, a temporary construction platform was built at tank top level to allow installation of pipework and cabling prior to the building being moved into the separation area. Only the final connections to the tanks had to be carried out under controlled conditions to minimise the requirement for extensive scaffolding within the Separation Area tank complex.

THE MILLENIUM WHEEL

Dr A P Mann, Babtie Allott & Lomax

11th April 2000

Introduction.

No recent structure has captured the public's imagination more successfully than the British Airways London Eye; for its purpose is clear, to delight and provide spectacular views. At 135 m high, the Eye is the fourth tallest structure in London. Acclaim for its design has been universal and the demand for a 'flight' overwhelming. An integral part of the design vision was to create an experience like no other and this has been achieved by advanced engineering to provide a flight of unprecedented smoothness. The Wheel integrates architectural, structural and mechanical skills with innovative methods of construction and alongside this, close teamwork delivered the project in just 16 months.

Design

Components

All the main features of the Wheel can be seen in Figure 1; these being the capsules, rim, columns and general support system making heavy use of cables. Significantly, the capsules are on the outside of the rim. The rim is supported via cable spokes onto a large spindle cantilevering out over the head of an A frame, and this frame is inclined and stabilised back to foundation level by a set of four cables. There are two Restraint Towers guiding the rim and maintaining its alignment at passenger boarding level, guidance is achieved via banks of vertical rollers positioned on each side of each tower. Each tower also supports mechanical equipment used for Wheel operation; the Southern Tower supports the electrical supply and the Northern Tower supports the drive system.

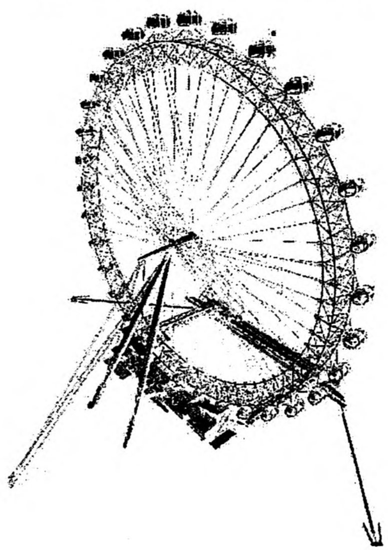


Figure 1

Under normal operating conditions, the rim runs freely between towers, just lightly restrained in line by sets of vertical rollers. This arrangement is permissible for light winds. When high winds threaten, the rim is rotated to a predetermined parking position where locking pins are then inserted through large lugs and lateral 'plungers' are hydraulically activated to press against the rim side. By this means, firm lateral and radial support is offered to the structure in storms.

Applicable forces

Live load and its variability are important for capsule design and for design of local capsule attachments to the rim. But for the Wheel as a whole, self-weight is the dominant force since passenger weight amounts to only a few per cent of the total. Internal forces are very important since the spoke cables connecting the rim to the hub have to be prestressed, and their reactions must be balanced by rim ring compression and this self generated axial load turned out to be a dominant factor governing rim strength.

Wind loading is obviously important but its assessment is complicated by the relative flexibility of the structure, which suggests the need to allow for dynamic enhancement.

The Wheel rotates slowly so forces related to motion are negligible. Nevertheless, the rotation does introduce some issues. The rim could not be fabricated plane, nor circular, for there has to be a tolerance. But lack of planarity implies development of a lateral force at rim level as the rim is driven through the restraints and the induced lateral force is cyclical and acts to introduce fatigue in the bracing connections.

No structure should be designed without considering its strength and stability during erection and this was particularly so for the Wheel. The method of assembly and method of lifting the completed rim from the horizontal (Figure 2) lead to loading states dominating parts of the structural sizing e.g. the critical loading state for the rim and foundations existed for a short time during the main lift.

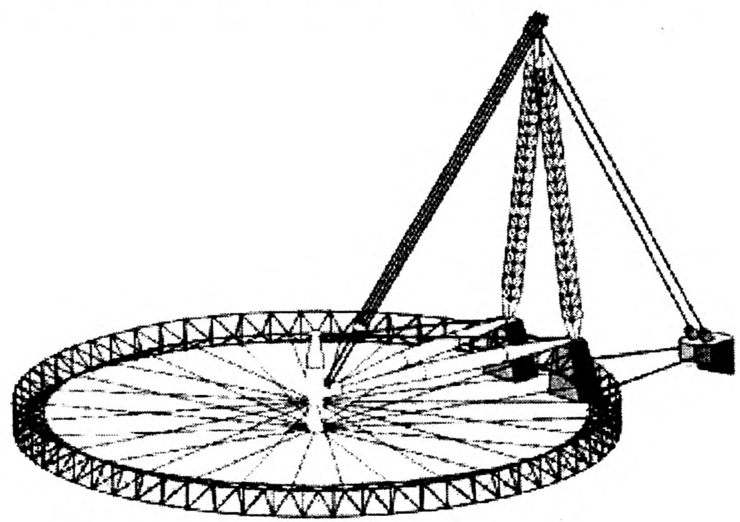


Figure 2

Rim Design
Under very high axial load the mode of static rim failure would be by ring buckling into four quadrants in line with classical elastic solutions and buckling strength is controlled by both lateral and torsional stiffness and can be assessed by computer analysis.

Cables
Cables are provided to stabilise the whole structure by attaching the A frame head to the rear tension base, these cables are 110mm in diameter. More cables are used to act as the rim spokes and to ensure hub rotation (70mm diameter). Since the Wheel is driven from the rim, the hub must be made to either follow in driving or stop in braking and this achieved by slightly inclined cables known as rotation cables (60mm diameter). All the cables were of the locked coil type, a form primarily chosen for corrosion resistance and for best linearity characteristics. The cable ends are secured in conical metal sockets in such a way that end socket strength always exceeds cable strength.

The spoke cables are arranged in sets on opposite faces of the Wheel and slightly inclined up from the hub to their rim attachment points. This inclination allows the cables to transmit horizontal wind loading. To prevent slackening all the cables are prestressed.

The spoke cables sag under their own weight creating a bending moment at their ends (the ends are only nominally pinned). Then as the Wheel rotates, the direction of sag reverses such that the end moment also reverses creating a fatigue environment at the anchorage points. To ease the degree of cable bend and thereby reduce cable stresses and prolong life, a rubber lined sleeve known as a bend limiter is attached.

There are a number of ways in which cables may be exited by wind motion, mainly by vortex shedding. Vibration resistance is linked to the interaction between a cable's natural frequency, its inherent damping and the frequency of vortex shedding All the cables have a natural frequency linked to their prestress tension. To counteract vibration at high frequency, all rim cables were fitted with specially developed dampers that are like a dumb bells. When the cable tries to vibrate, the weights respond in an opposite sense to suppress the motion.

Separate vibration problems exist for the rear stay cables since the stress in these is much lower than for the spokes, this makes their natural frequency quite different and makes them potentially vulnerable to a motion known as 'galloping'. Vibrations are suppressed in three ways: firstly the cables are interconnected at intervals; secondly they are fitted with dumb bell dampers to suppress higher frequency modes and thirdly they are fitted with hydraulic dampers to suppress the lower modes. The hydraulic dampers are fixed just above the tension base.

The Hub
The hub anchors all spoke and rotation cables and transmits their net vertical reaction (the 'rim' weight) to the spindle via bearings fitted below. Since all cables are in tension, the hub itself is in constant ring tension. The hub was made as a single casting then machined internally to make a tight fit to the bearings. The maximum hub diameter had to be limited for road transportation.

Bearings
The bearings between the hub and the spindle are large proprietary roller bearings. In use on the Eye, they are under very low duty i.e. low loading and very low rotational speed.

The Spindle
The spindle is a massive structural element. Its characterising dimensions are a length of 25m, a diameter of 2.1m and wall thicknesses up to 300mm. The cantilever part, some 11m long, carries the entire weight of rim, capsules and passengers.

Following successful experience on an earlier project, the decision was made to fabricate the spindle using steel castings. This concept fitted in ideally with the required circular shape and offered the ability to taper wall thickness and to cast on the various eyes required for cable attachments. To facilitate the casting operation and subsequent transport, the spindle was made in seven sections (one is rolled, six are cast). Most welds joining the thick sections together were machine welds made in steel up to 200mm thick by the sub arc process.

The spindle is simply a cantilever at the front with a long rear section to which cables are anchored to tail it down. However, there are two distinct modes of spindle failure, one of bending and one of fracture. The latter was most important since the fundamental requirements for fracture initiation are thick steel, welding and low temperature, all of which are present. Given a wall thickness that satisfied stress margins, the metallurgical challenge was then to provide adequate assurance against fracture. This was achieved by a package of measures: the cast quality was targeted at achieving high ductility and high Charpy values, and all the as-welded castings were heat treated to reduce residual stresses and enhance fracture toughness. NDE coverage was thorough with 100% volumetric examination in the tensile areas.

For total protection it was then decided to proof test the whole spindle before installation. Proof stressing assures strength, but just as importantly it increases a specimen's fracture toughness. Because of the spindle's size, proof testing was a major task. The spindle was assembled on its side in the Rotterdam fabrication shops, bracings were added and then load was applied via strand jacks to the bearing end. This imposed a bending pattern similar to that acting in service. Throughout the test, the spindle behaved linearly with measured stresses being very close to theoretical predictions.

A frame
The tapered tubular shape of the A frame was an architectural requirement. The two legs were fabricated as large tubes made by rolling and seam welding 40mm plate. The head of the A frame is attached to the spindle through a pin made from high strength stainless steel. The decision to erect the rim from the horizontal had a significant effect on the A frame detailing. It was first of all necessary to include strong attachment points in the complex fabrication piece that joins the two legs together just below the spindle. Secondly it was necessary to include a hinge at the base of each leg to allow rotation during the lift.

Damping
Reference to Figure 3 will show that the whole structure can be simplified to a mass simply restrained by the rear cables acting as springs. The frequency of this mass / spring system in a sway mode is around 0.25 Hz. This low natural frequency suggests possible dynamic amplification of the wind forces through resonance with wind buffeting. If excitation persists, the amplified deflections and corresponding forces could become critical since the structure has very little natural damping.

Recognising the problem, it was decided that the best engineering approach was to add damping. For global response, this has been achieved by fixing 64 tuned mass dampers equi-spaced around the rim. These consist of hollow tubes, each containing a movable mass controlled by a spring attached to one end.

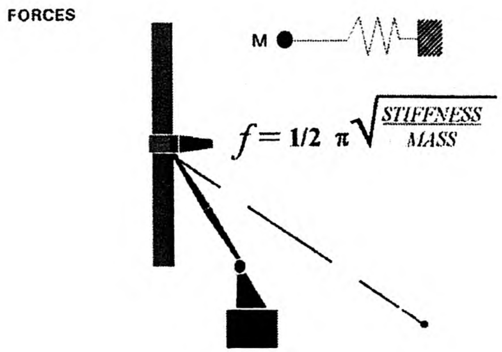


Figure 3

Erection

The method of erection is shown in outline in Figure 2. It consists of constructing the rim flat over the river on temporary supports and then hauling it upright utilising large shear legs (borrowed from a crane) and a head strand jacking system. This method was possible because the Thames happens to be about twice the diameter of the rim at the site and because the shipping lane is on the opposite bank. As an erection proposal, the method was immediately attractive, not least because there was so little working space on the land site itself. Moreover the method offered the spectacular opportunity of lifting the fourth tallest structure in London in just one day and offered many other safety advantages.

Foundations

The wheel cantilevers over the River Thames producing enhanced vertical compression loads in the front foundation below the columns and associated vertical tie down loads in the rear foundations. The raking nature of the columns and rear stay cables provide an equal and opposite horizontal force component between the separate compression and tension foundations. Rather than allow this force to be transferred into and out of the ground, two simple ground beams were used interlinking the two bases.

Plinths

The agreed method of erection from the horizontal resulted in the need for an uplift erection rotation point located some 11 metres above Queen's Walk. Two high plinths, were therefore built to support heavy steel castings into which the A frame legs were hinged at their base. These plinths were inclined to match the final slope of the A frame legs.

Original ideas of casting the plinths in white concrete were dismissed once the practicality of constructing them was considered. It was considered too difficult to achieve a good quality concrete finish. Moreover the raking nature and density of steel reinforcement required for the temporary erection loads forced the design team to consider alternatives. The final solution comprises a permanent steel outer mould filled with reinforced concrete: this gave both a good finish, a rapid build and a strong enough support..

Compression Base

The compression foundation supports the base of the raking A frame columns and transmits their vertical loads into the London Clay via bored piles each 32 metres long. Vertical load was transferred onto the piles via the large pile cap that required just under 1000 cubic metres of concrete.

The plinths are located on top and at the ends of the cap and induce large bending moments into it. Up to four layers of reinforcement were required to handle these bending effects.

Tension Base

The tension base forms the anchorage to the rear stay cables. It consists of another large concrete beam anchored down at each end by 6 long piles. All the cable anchors are secured onto the beam's lower face. With the cable angle imposed and the need for anchorage inspection, a large foundation was required with the base of its access and cable stressing chamber located approximately 9 metres below ground level. The tension base design was principally driven by the temporary erection loading case because maximum pile uplift occurred during the initial lift of the wheel.

Capsules

The passenger carrying capsules are a key part of the London Eye's image. They are designed to give passengers freedom to walk around and enjoy the view. To maximise that view, much of the capsule exterior is glass and all the capsules are mounted on the outside of the rim. The capsules must provide a comfortable and pleasant environment and they must safely contain the passengers during their 30 minute 'flight'. For safety, the capsules had to be fire resistant and lightning proof.

Capsules are circular in cross section with a floor at low level and the void below houses equipment. This equipment is needed for heating and cooling, for communications and for the motorised control. The capsules are contained within circular mounting rings and rotate within them on bearings fixed to the inside of the rings. To keep the floor horizontal, capsules are driven around in synchronisation with the rim. This is achieved by using a motor inside each capsule to turn it 360° in one direction whilst the rim rotates a full revolution in the opposite direction. The motors receive instructions on how far to turn through a system which includes sensors measuring tilt, computers calculating a response and a radio system which transmits the instruction.

In an emergency, there are provisions to extricate passengers via the doors using rope attachments.

Glass

One of the ride safety principles is that passengers must be safely contained within the capsules once they leave the boarding platform and this includes protecting them from falling objects and ensuring the capsule walls have adequate strength against lateral pressure: this has implications for the quality of the glass: altogether this had to be

- capable of being curved in two planes
- of high optical quality for observation purposes
- impact resistant
- have enough strength for containment

These demands led to the use of laminated glass with an interlayer down the centre. This interlayer sheet is invisible but its strong bond ensures that should one pane fracture, the unit as a whole stays together and does not shatter.

Testing

Because the stability system was so novel it was tested out in France in a mock up rig before final capsule production was started. The same rig was used to load test the capsule frame and assure its strength.

Pier And Impact Protection System

Access to the Wheel is catered for by public transport systems such as the tube, but in line with the overall plan for London, river access is also provided via the new Westminster Pier. This takes the form of a 100m long floating pontoon connected to Queen's Walk by two bridges each consisting of a short fixed length linked into an articulated brow. The Thames, is still tidal by the site with significant current flows up and down river of about 2 m/sec. The tidal range is very high at about 7m. These factors determine the drag forces acting on the Pier plus the articulation required within its components.

The Eye is aligned over the Thames, which is a busy highway for all types of commercial shipping and this makes it potentially vulnerable to an impact. In certain tides it is physically possible to position some of the known larger river vessels in a manner that would clash with the lowest capsule. To counteract this threat, the pontoon and brows, plus their set of mooring cables, double up as an impact protection system.

Heavy vessel impact was considered a plausible but only a slight threat since the through route for heavy traffic is along a channel on the opposite bank to the Eye. Lighter passenger craft criss cross the river and some of these dock onto the pontoon, so the impact risk from these craft is clearly greater though involving less energy. The design need was to assign an impact energy linked to vessel mass and speed. To determine this energy rationally, a numerical probabilistic approach was adopted.

The likeliest impact comes from an errant vessel drifting with the tide off its intended route. If this route were aligned towards the Wheel, the vessel would have to cross the protective boom before it could cause damage. The arresting medium of this boom is a strong cable terminating in a proprietary unit capable of absorbing significant energy. For the boom to be effective, the cable must be kept afloat on the water surface. This is achieved by the attachment of empty cylinders interlinked on either side stretching for the full length of the cable.

LASER EYE SURGERY - AN ACCEPTABLE ALTERNATIVE TO GLASSES AND CONTACT LENSES ?

Dr. Stephen J Doyle

Manchester Royal Eye Hospital and Optimax Laser Eye Clinic.

9th May 2000

Introduction

Recent high profile people having laser eye surgery rather than wear contact lenses or glasses, notably Richard Branson and Tiger Woods, have made people realise that perhaps this is a technology that is coming of age. Many tens of thousands of people in the UK have had PRK (Photorefractive Keratectomy), which is the most popular modality in the UK and now LASIK (Laser Assisted Keratomileusis), a mixture of laser and microsurgery, offers surgery for the more severe cases. The ophthalmologist who first thought of this surgery, Prof. Steven Trokel in New York, did his first degree in engineering !

How It Works

This technology utilises an ultra-violet "excimer" laser of 193 nm to alter the shape of the cornea. This is an argon-fluoride laser with a very high photon energy of 6.5 eV. This cuts off about 0.25 microns per pulse. In the case of myopia (short sight), the cornea is flattened and in hyperopia (long-sight) it is steepened. The process might be thought of as "carving a contact lens" onto the eye. The Cornea is about 500 microns (0.5mm) thick and, in an average myope of 3 Dioptres (D), the laser

takes off about 40 microns. The cornea is not weakened physically and the operation is not visible to the naked eye (and indeed it is often not possible to detect even using an operating microscope). The laser hardly raises the temperature of the cornea and hence does not cause any scarring due to collagen thermal damage.

Accuracy

No-one yet knows how to measure what the laser is doing in "real time" i.e. during the operation. People are not as identical as the inert materials of circuit boards, for which this type of laser has been used for many years in industry, and the laser may cut off a bit more or less from the cornea than predicted. People's healing characteristics also vary. Hence, the bigger the prescription, the bigger the spread of results. As a rough guide, most low myopes (less than -6D) achieve within 1/2 D of aim and most higher myopes (-6 to -10D) achieve within 1D. Although it is less accurate for the higher myopes, the patients are often even more pleased, as they are effectively blind without glasses or contact lenses. (Try putting on a couple of +3.5 reading glasses, one on top of the other, to see what a -7D myopes is like!). About 25% of the adult caucasian population are myopic and 90% of these are -6D or less. PRK is suitable for up to about 6D of myopia, 4D of astigmatism and 2D of hyperopia. Treatment for hyperopia is at an earlier stage of development than for myopia, as there have been more problems in working out the best shape to cut onto the eye.

Risks

No operation has zero risk, including PRK, despite the "street cred" of lasers. PRK is elective surgery on a healthy eye, so the criteria are more strict than operating on a diseased organ. No one, to my best knowledge, has been blinded by PRK and at least 50,000 have now been treated in the UK alone.

The commonest risk is loss of sharpness of vision that is not correctable with glasses. About 5% will lose 1 line of vision on the Snellen chart (the commonest eye chart) and about 1 in 1000 will loose 2 lines. This is mostly due to micro irregularities of the surface that cannot be corrected optically by the regular surface of glasses. Most patients will not notice the loss of one line of vision but will notice 2 lines of loss. Patients can also have a "touch-up" for residual refractive errors and most clinics do not charge extra for this. About 5% of the higher myopes will have such an "enhancement".

Patients have to understand that they could have some ghastly problem such as a corneal abscess and might end up needing a corneal graft. Such situations may be exceedingly rare, but if it happens to you then statistics are no consolation! The risk of such a disaster is probably about the same as when wearing a soft lens, which many patients will have used for some years.

How about LASIK?

LASIK (Laser in situ keratomileusis) is the other major alternative in laser eye surgery. This combines PRK with an older surgical procedure known as keratomileusis. In this, a powered microkeratome (a fancy sort of bacon slicer!) is attached to the cornea with a suction ring and a partial flap of about 160 microns is created with a "hinge" at one side. The excimer laser is then fired in the same way as in surface PRK and the flap is replaced. It has hence been called the "flap and zap" operation! The flap re-attaches initially by osmotic pressure and no sutures are needed. LASIK is a better surgical experience for the patient than PRK because, as the corneal epithelium is left almost intact, there is little pain and a faster visual rehabilitation. A LASIK patient at day one post-op will see what a PRK patient will see at 1 - 2 weeks. This has been called the "wow" factor of the surgery.

What are the risks?
LASIK is clearly a "proper" surgical procedure and has more to go wrong than PRK. Perhaps the most important piece of advice for the patient contemplating LASIK is to choose the surgeon well! LASIK costs more than PRK, being about £800 - 1500 per eye as opposed to £500 - 600.

How does a person decide whether to opt for PRK or LASIK?
The end results of PRK and LASIK are the same. As they both use the same laser, the accuracy levels and numbers of people who lose sharpness of vision is also about the same. LASIK "gets there" faster and with less pain than PRK whereas PRK is safer. Which procedure to chose depends on each patient's attitude to risk versus convenience. Neither procedure should be used for myopia greater than -12Dioptres as the optical zones carved on the cornea are too small for low light vision. One eventually just "runs out of cornea"! PRK is the procedure that most low myopes chose and LASIK is better for the high myopes because of the speed of visual recovery. There is an "overlap" area of those between -5 to -7 Dioptres where the pros and cons are about even. (90% of myopes are -6D or less).

Summary
PRK and LASIK are now beyond the experimental stage and into the developmental stage. Having treated one of my GP partners (-2.25D) and my brother (-6.5D), who are both delighted, I think that it is a reasonable alternative to glasses or contact lenses, particularly for the lower myope.

The Royal College of Ophthalmologists (Tel: 0171-935-0702, Fax: 0171-935-9398) publishes an excellent leaflet for prospective patients which costs £5.

OBITUARY:

Stanley Barnes

Stanley Barnes moved from Brighton to Manchester in the mid nineteen thirties and made his home here, settling in Gatley. Eventually he became responsible for all the business of Allen West in the Manchester area and remained with them until his retirement in the early nineteen seventies. He was a great letter writer and kept in contact with his former colleagues nationwide. Until well into his eighties, he made himself available for any electrical or other emergencies.

In Gatley, he joined the Congregational Church and remained a faithful member for some seventy years, soon becoming a deacon. He was interested in music, particularly choral and church music. He was also a competent organist and pianist. He was one of the main producers for the local Dramatic Society in the years following the Second World War. Sadly a simple accident on an icy surface led to several years of immobility and suffering, borne nevertheless with the courage and fortitude of his faith. He lived to the good age of ninety one.

OBITUARY:

Cyril John Knott

1910 - 2000
President of the MAE 1967 - 68

Cyril John Knott was born 1st October 1910 in Ashton under Lyne. His father, from an old Ashton family, was a church organist and choir master and also a piano teacher. His mother, of Cornish origin, was a primary school teacher. At eleven he went to Manchester Grammar School and as it was in Long Millgate in those days he got to know the city well. He was always proud to be an Old Mancunian.

At the age of sixteen he left school and started work as a trainee for the Chloride Electrical Storage Company by whom he was employed for all his working life. He started as an office boy at Clifton Junction, near Manchester, and spent time in the drawing office and many other departments, studying at night to obtain qualifications as a production engineer. From 1931 to 1940 he was a member of the Works Manager's Plant Office staff and was involved in the design of special machines and maintenance supervision of general works plant. In 1936 he was sent to Dublin to re-establish the Chloride Depot which had suffered a fire.

During the early part of the war, he was Night Manager at Clifton Junction and in 1943 he was sent to Calcutta to select a suitable site (assessing availability of power, water, fuel and labour) for a factory to supply batteries to the British Army in the Far East. He returned to England in 1944 to present his report, place orders for the plant and plan the layout of the factory. In 1945 he returned to India to supervise the installation, engage labour and commission the factory, acting as Works Manager until mid 1947. He then returned to Manchester and resumed his work at Clifton Junction holding over the ensuing 18 years the posts of Assistant Works Manager, Battery Production Engineer and Works Engineer.

He became a member of the Manchester Association of Engineers in 1944 and regularly attended meetings. In 1967 - 68 he was President: at his inaugural lecture he spoke on the history and manufacture of lead acid batteries, a subject on which, as a result of his work, he was extremely well informed.

He was also an enthusiastic member of the Newcomen Society for the Study of the History of Technology and Engineering in the days before industrial archaeology was fashionable. Days out in the Peak District usually involved a visit to old mine workings, railways, canals or mills and holidays in Cornwall always included photographing surviving beam engines. He read widely and was very knowledgeable on many aspects of industrial history. Being a keen photographer he amassed a large collection of pictures of interesting industrial installations.

In 1969 he retired from the Chloride Electrical Storage Company: his last post with the company was Lead Smelting Department Manager. His engineering knowledge was utilised in car and home maintenance and much time was spent travelling with his wife Marie in the UK and Europe. In 1974 he and Marie moved from Altrincham to Natland near Kendal from where they explored the Lake District and Yorkshire Dales.

He was a very private person who led a quiet life devoted to his wife and home. Though always happy to attend a lecture on a topic of interest he was never keen to attend social gatherings. In his youth he had been a keen walker and always enjoyed being out in the countryside especially if excursions could, in some way, involve a little industrial archaeology.