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STEEL STRUCTURES IN THE NEW MILLENNIUM

P.J. Dowling, B.A. Burgan

Since steel was introduced over 100 years ago, it has made significant strides to the forefront of the construction arena. This paper reviews the implications of recent developments in its use; it looks at progress made in the formation of steel sections, structural systems and the use of steel in onshore and offshore applications. Developments in fire engineering, protection systems and environmental strategies are also discussed. The paper demonstrates the commitment of the steel construction sector to a path of continued improvement in a market where advances in process and product technology will become increasingly led by the clients' need for higher performance materials and systems. By looking at the past achievements and the present trends and developments, the paper provides a glimpse of steel structures for the next millennium.

1. Introduction

Steel is synonymous with modern construction. Since the end of the last century it has provided opportunity and inspiration for generations of designers. Today, in an era of architectural pluralism, and of engineering innovation, its use is being taken to new levels of expression and technical sophistication. This is in part attributable to the strides that have been made in the metallurgy and structural understanding of the material, and in production engineering; but perhaps more fundamentally it is testament to the continuing commitment and fascination of architects and engineers with the outstanding design opportunities offered by steel.

Steel is also a dynamic material; 70% of steels available today did not exist 10 years ago. Specifications are constantly improving in accordance with developments in manufacturing technology and the needs of the market place. The steel industry invests continually in new technology, research and development, anticipating demands for greater lightness in weight, and improvements in such properties as toughness, yield stress and weldability.

The steel industry operates within a structured system of standards which embraces thousands of specifications. The development of new applications leads to the creation of new steels which, in turn, are translated into new specifications for general and more utilitarian applications. The alloy steels which gave Britain's jet aircraft engine industry a place in the world's aerospace markets are now finding new uses in more basic engineering applications and stainless steel, once the preserve of the few and the fashionable, is now to be found in ever-increasing construction products.

Steel is easier and cheaper to recycle than any other manufactured material; around 45% of the steel manufactured in the 1960's and 70's is already being reclaimed. In just two years in the early 1990's the UK steel industry spent an estimated £50m on environmental improvements. Similar sums have been invested throughout the past 20 years to ensure that steel making meets modern environmental standards.

The end of the last century gave us steel, the present century is notable as a time when architects and engineers earnestly began to discover the potential of its usage. This paper includes a broad overview of current technology and indicates the practical possibilities of the material as currently understood. Perhaps the greatest delight is that each decade has brought new applications and demonstrated new possibilities, thus pointing to an exciting future.

2. New structural forms

2.1. Parallel flange sections

A change from tapered to parallel flanges for all steel sections which commenced in the 1950's is now virtually complete. It includes beams, columns, T-sections and channels. The new parallel flange sections offer a variety of advantages which include:

- Improved structural efficiency by an average of 4% on the major and 15% on the minor axes of the section.

- Average section weight reduction of 1.3%.
- Cheaper sections.
- Improved dimensional control and surface finish.

Easier detailing, stiffening, connections and elimination of tapered washers.

2.2. Asymmetric beams

The asymmetric beam is a rolled section with a narrower top than bottom flange (Fig 1). Such a component is ideal for use in composite floor construction (see Section 2.3).

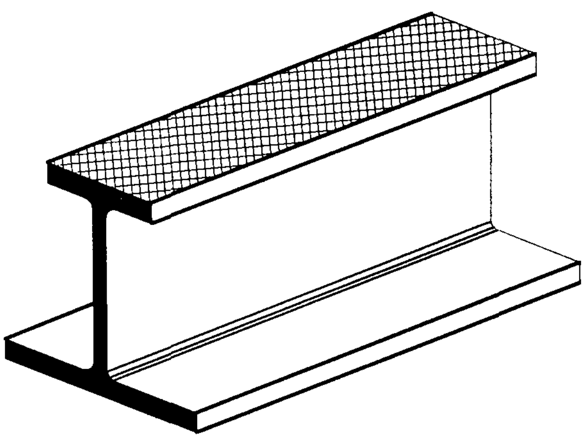

Fig 1. The asymmetric rolled beam

I-sections are rolled on their side and, in the past, it was not possible to prevent curling of the member during rolling if the cross-section was not balanced with equal flanges. The eventual solution to this problem came from a £90m investment in 1991 by British Steel which led to significant advances in rolling technology. A computer controlled close coupled universal beam mill, the only mill in the world that can produce this type of section, was developed. During rolling, large unequal horizontal forces are applied as the hot metal is squeezed through the mill, thus preventing the section from curling. The first experimental asymmetric sections were rolled in May 1995.

The section starts as a 25 t steel billet heated to 1,150°C. This is formed into a rough I-section on a cogging mill before transfer to the close coupled mill where it is subjected to 37 passes. The asymmetric shape comes from the last three passes and, in the final pass, a checker plate rib pattern is rolled into the top (narrow) flange. This enhances shear bond when the section is used in composite floor construction. Another feature of the asymmetric beam is that the web is thicker than the flanges. This enhances fire resistance when the section is used in composite floors.

Asymmetric beams are produced in a small range of sizes, specifically chosen with multi-storey building floors in mind. Beam depths of 280 mm and 300 mm are rolled in a range of weights from 100 to 153 kg/m. All sections use S355 steel to BS EN 10025 (former grade 50).

2.2. Developments in slim floor construction

Slim floor is a form of construction in which the steel section lies within the depth of a concrete or composite floor slab. Early developments of this system saw the use of Universal Column (UC) sections with a plate welded to the bottom flange acting as the slim floor beam. The plate supports the floor slab directly and is the only part of the section that is exposed (Fig 2).

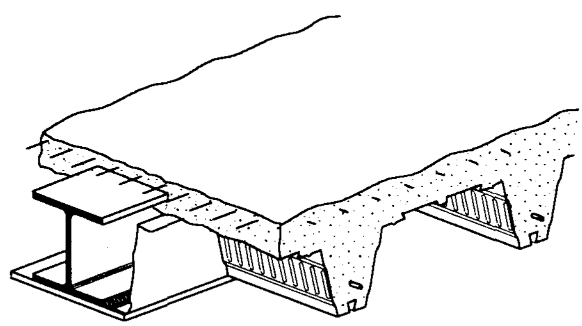

Fig 2. Conventional slim floor construction using UC beam and deep decking

Two forms of slab construction are used: precast concrete slabs with or without *in situ* concrete; and deep decking acting compositely with an *in situ* concrete slab, as illustrated in Fig 2.

This second form of construction is similar to composite construction using shallow deck profiles, in which the decking sits on the top flange of the beam. In both cases, the decking supports the loads during concreting, and resists the imposed loads subsequently as a composite slab. With deep decking, this composite action is enhanced by the transverse embossments in the deck profile (Fig 3), and by reinforcing bars located in the ribs of the slab.

Slim floor construction is marketed by British Steel under the trademark *'Slimflor'*, and has achieved considerable success in the last 4 years.

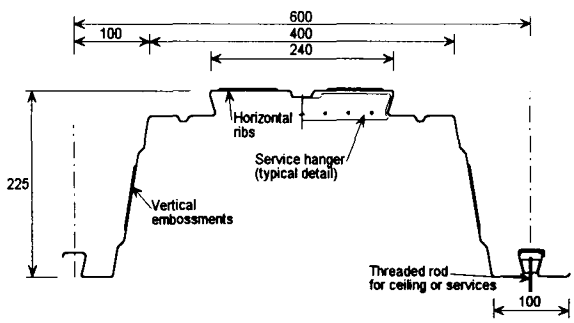

Fig 3. Cross-section through deep deck profile

Since its inception in 1992, *Slimflors* have been built in 74 new projects in the UK. The principal advantages of *Slimflor* construction are:

- Shallow floor depth.
- Flexibility of service layout, including space between the deck ribs.
- Inherent fire resistance (60 minutes without fire protection).
- Light floor construction in comparison to concrete alternatives.

Design guidance on *Slimflor* construction is available in two References [13, 15].

The development of the asymmetric beam led to improving the economy of *Slimflor* construction. The Asymmetric *Slimflor* Beam (ASB) does not require welding of an additional plate and achieves optimum properties for design at the ultimate, serviceability and fire limit states. It is complemented by an improved form of deep decking, and the system is marketed under the registered trademark of *Slimdek*.

This form of construction is illustrated in Fig 4.

The additional benefits of the ASB are:

- Reduced steel weight (and hence cost) in comparison with conventional *Slimflor* sections.
- Savings in fabrication costs.
- Readily available section with defined properties.
- Less distortion due to welding.
- Good composite action (as obtained from tests).

ASB sections are designed for use *only* with deep deck composite slabs. The *in situ* concrete used to form the slab achieves composite action with the ASB, which is used in evaluating load-span tables for these sections. Composite action is enhanced by the raised rib pattern rolled into the top flange. The steel decking sits directly on the bottom flange and the system is suitable for floor spans typically up to 7.5 m with a similar spacing between beams. Typical depth of the floor system is 350 - 400 mm.

Economic assessments of the use of the ASB sections have shown that the potential weight saving relative to conventional *Slimflor* beams is of the order of 15 to 25%, and the additional saving in fabrication cost is significant because it is not necessary to weld a bottom plate to the section. With these economies, ASB construction may be shown to be cheaper than conventional composite beam and slab, and reinforced concrete flat slab construction in the same medium span range. Design principles, design methods, and practical detailing measures for *Slimdek* construction using ASB sections is now available and is illustrated with design examples [12]. The first use of *Slimdek* (80 t) is Berlin's Innovation Centre. A 90 minute floor fire resistance is provided on this project by welding additional studs to the beam webs for greater bond to the concrete, which is further enhanced by reinforcing bars running across the studs.

At the edges of the floor, a traditional design approach has been to rest the decking on the top flange of an I-beam such that the vertical load acts through the shear centre of the beam. This is because the use of a conventional *Slimflor* beam for edge beams may give rise to torsion problems due to the eccentric loads from the slab.

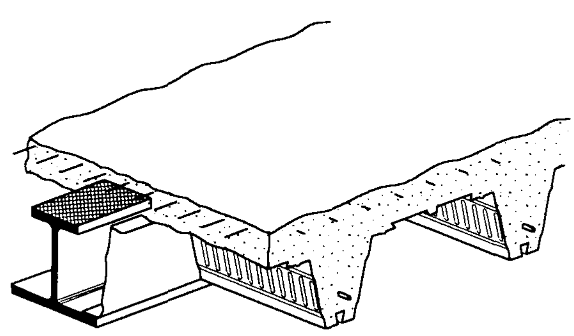

Fig 4.'*Slimdek*' construction using ASB sections

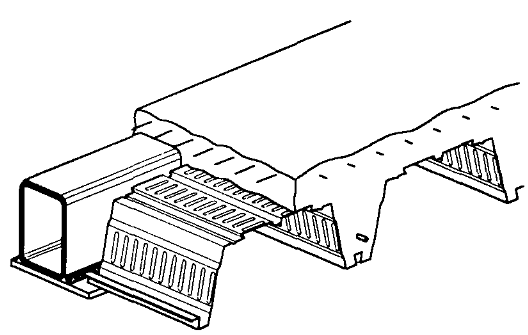

Fig 5. RHS *Slimflor* Edge Beam using deep decking

Relatively heavy sections would be required to avoid excessive twist. To provide a more efficient solution for edge beams in this form of construction, the RHS *Slimflor* beam has been developed, as shown in Fig 5 [16]. The good torsional properties of the RHS section ensure that stresses and movements due to eccentric loads are minimised.

2.4. Double skin composite construction

Double skin composite (DSC) construction consists of two steel plates either side of a concrete infill. Composite action between the facing plates and the infill is achieved by stud connectors welded to each steel plate (Fig 6). In addition, the connectors serve as transverse shear reinforcement for the concrete and they inhibit plate buckling by restraining the plate at intervals which are governed by the plate thickness.

DSC offers the following advantages:

- Clean internal and external finishes.
- The steel plates act as permanent formwork.
- The concrete is protected by an impermeable membrane.
- The steel plates are located for maximum lever arm, leading to more efficient and lighter structures than reinforced concrete.
- The system lends itself to prefabrication leading to shorter construction times.

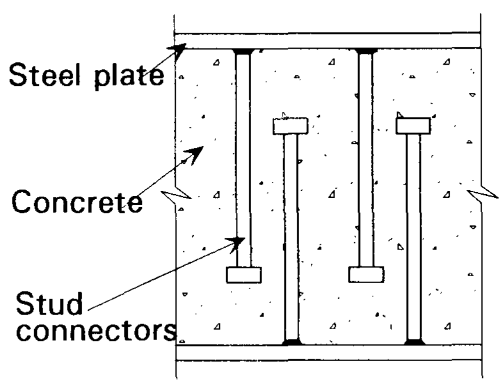

Fig 6. Section of a DSC element

The flexibility and inherent advantages of this form of construction, coupled with high ductility and impact resistance, mean that DSC can be used for many civil, offshore and marine structures.

DSC construction was proposed as an alternative to reinforced concrete during the initial design stages of the Conwy river crossing in North Wales in 1988. Although the system was not adopted due to lack of sufficient design information, it demonstrated such promise that extensive further research and development was initiated.

The first comprehensive series of tests was carried out at the University of Wales College [21]. In all, 53 one-third model scale specimens with a cross-section of 150 mm square and lengths 1.5 m and 2.3 m were tested as beams, columns and beam-columns and led to the definition of failure modes for each load type.

A further series of larger scale tests included beams, columns, corners, T-junctions and one model tunnel cross-sections. These furthered the understanding of possible failure modes and led to the development of design rules for DSC construction [18] and rules for the design of DSC immersed tube tunnels [17].

In addition to the above, current work on DSC construction includes fire studies and tests to study the fatigue behaviour of DSC beams. This will lead to the publication of further design guidance in 1998.

In spite of the structural efficiency of DSC construction, its use in practice has remained very limited. This is largely attributed to construction issues: (a) prior to concrete setting, the facing plates act independently and lack rigidity leading to handling problems, (b) during concreting, the plates need to be propped to resist hydrostatic pressure from the concrete and (c) difficulties relating to butt and T-joints between panels had not been addressed. All these problems have been resolved with a new product developed jointly by The Steel Construction Institute and British Steel. It is marketed by British Steel under the trade name of *Bi-Steel*.

In *Bi-Steel*, the facing plates are permanently connected by a series of bars (Fig 7) thus creating a modular concept. Both ends of the bars are simultaneously friction welded to the facing plates in a unique manufacturing facility developed specifically for this product. The resulting panels are more easy to handle prior to concreting than the separate plates in conventional DSC and the friction welded bars eliminate the need for any propping during concreting. Furthermore they can be manufactured either as flat panels or panels curved in one direction

The problem of joining was also addressed in *Bi-Steel*. A system of bent strips acts as a guide during panel assembly and as backing during welding (Fig 8).

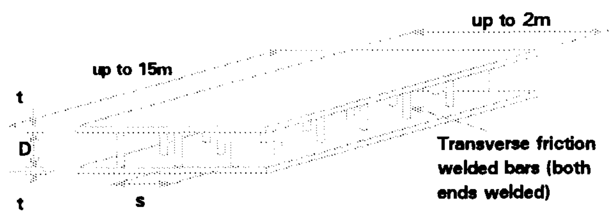

Fig 7. Typical Bi-Steel panel with friction welded bars

At T-joints, the use of rolled sections is proposed (Fig 9), leading to robust connections. The rounded corners of rolled sections offer further advantages with respect to fatigue where cyclic loading is involved.

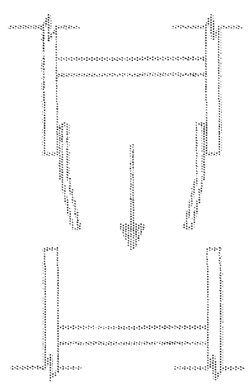

Fig 8. Bent bars act as guides and as backing strips for welding

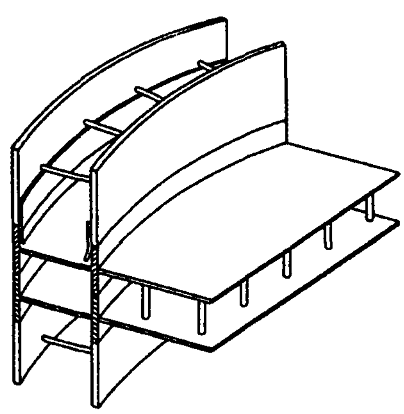

Fig 9. Rolled sections are used in the construction of T-joints

3. Single storey buildings

The most important factor which kept steel ahead of concrete as the material to be used for single-storey buildings in the UK was the rapid adoption of plastic design methods for portal frames in the 1960's. In recent years an innovation originating in the USA but adapted to European needs has been the automated design and fabrication of portal-framed buildings using components welded from thin plate and tapered to give maximum efficiency and economy. Basically, it takes advantage of modern automated fabrication methods to reduce the material content required compared with the more traditional plastically designed portal frames fabricated from hot-rolled sections and requiring haunches to the eaves and apex.

Today, more than 90% of single-storey building in the UK are steel-framed and about half of these are portal frames.

4. Multi-storey buildings

The share of steel in the UK multi-storey building /**structure** frame market has increased from less than 30% 15 years ago to nearly 60% at present. Most of these frames are designed using principles of composite construction for both beams and floor slabs. However, given that the cost of the structure is only in the region of 10-20% of the total building cost, structural choice is more heavily influenced by adaptability of the building to changing requirements. The importance of this flexibility becomes obvious when we compare the average life of the structure of 50 years with the average tenancy of seven years.

4.1. Service/structure integration

A key ingredient of the desired flexibility is longer column-free spans (12 - 18 m). However, as spans increase, so does the depth of the floor structure. This increases the floor to floor zone, with higher cladding costs or, where building height is restricted, a smaller number of floors. The structural solutions which emerged to meet this challenge have sought to integrate the structure and services into one common layer thereby minimising the overall depth of the floor zone. A number of systems are available which offer service/structure integration and these include:

Composite beams and slabs: Shallower structural depths are achieved by composite action. Ducts run below the beams whereas terminal units and other equipment are located within the depth of the beams.

Composite beams with web openings: Openings of up to 70% of the beam depth can be provided in the webs to allow the passage of large service ducts. Horizontal stiffening may be required in high shear zones.

Cellular beams: Regular circular openings up to 70% of the final beam depth can be provided by cutting and re-welding rolled sections (Fig 10). By welding together tees from two rolled sections of different mass, weight distribution between the compressive and tensile flanges is optimised to maximise the efficiency of the composite action.

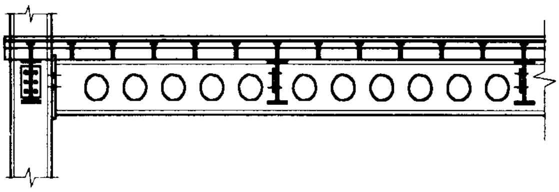

Fig 10. Celllar beam used in composite floor

Haunched beams: Haunching can be used to transfer moment to the column, thus reducing beam depth by about 30%, allowing ducts to pass below the beams.

Tapered sections: Fabricated sections can be tapered to provide a greater zone for services adjacent to columns. The taper angle is typically 6° and the beam depth at the columns can be about half its depth at mid-span.

Composite trusses: Warren trusses offer the greatest zones for passage of ducts. The trusses are fabricated from angles, tee sections or hollow sections.

Stub girders: These comprise a vierendeel arrangement in which a heavy steel bottom chord is connected to a composite floor slab by separate lengths of steel stubs. Service ducts pass between the stubs.

Parallel beams: This comprises an orthogonal system of beams, with continuity provided in both directions, thereby reducing beam depth. Service distribution can be achieved in two directions.

4.2. Over-cladding

In addition to the above developments which are primarily aimed at new building structures, the steel industry has been addressing ways in which more steel products can be used in the renovation of existing buildings. The cost of renovation of existing buildings as a proportion of expenditure on building construction has increased to around 40%. Throughout the EC, over 10,000 tower blocks built since the Second World War may need attention over the next 20 years. Problems with such buildings include decay of the external fabric, high heating bills, condensation and poor appearance. The installation of a new facade over the existing cladding of the building, known as 'over-cladding', can save energy, eliminate water penetration, reduce condensation, reduce deterioration, improve appearance and thus extend the life of the building. It can also be carried out whilst the building is in use.

Over-cladding involves three distinct components: the cladding panels, the sub-frame and the attachment system. For the sub-frame, either horizontal members supporting vertically spanning panels or vertical members supporting horizontally spanning panels can be used. The members are made from cold formed galvanized steel. The cladding takes one of three forms:

- Profiled steel sheeting with insulation attached to the existing wall.
- Composite panels of flat or lightly profiled steel sheets with polyurethane or other filler material.
- Cassette systems of flat sheets with bent edges.

Although to date the use of light steel in over-cladding is relatively limited in the UK, considerable experience exists in Scandinavian countries such as Denmark and Finland. This is partly because of the larger number of 1950's and 60's concrete buildings in these countries as well as their harsher environment.

5. Offshore structures

The first fixed structure in the UK sector of the North Sea for the production of hydrocarbons was Amoco's Leman 'A' platform, installed in 1966. By the end of 1994 some 205 installations (see Table 1 and refs [1] and [2]) had been installed in the UK sector, a remarkable achievement in a 30-year period. They are the end product of progressive evolution, although a number of discrete phases can be identified, arising from varying needs, of which cost, design Codes, materials, research and client schedules are but a few.

By the end of the century a total of 250 platforms in UK waters is a reasonable assumption, and if all North Sea platforms - British, Danish, Dutch and Norwegian - are included, one could arrive at potentially 500 platforms of fixed structure in the North Sea, by the year 2000 [11].

Up to 1994, the North Sea had provided a market for steel of 300,000 t for piles, 700,000 t for jacket structures and 555,000 t for topsides. Over 95% of the platforms installed had steel supporting structures.

Over the same period of time, fabrication yards have improved and now offer a superb range of facilities for constructing jackets up to 25,000 t and large integrated decks up to 10,000 t. These yards are probably now at their best in terms of worldwide capability and facilities.

5.1. Jacket structures

Over the last 30 years there has been a continuous development of North Sea jackets, and this will no doubt continue with the search for lighter structures and cost savings, whilst maintaining required levels of reliability.

The 'first-and-second-generation' North Sea jackets were barge launched and usually required additional buoyancy tanks to keep the jacket afloat until it was upended and piled to the seabed. Other features of these early jackets were a large number of small diameter piles dictated by the relatively small pile-driving hammers available by today's standards (e.g. 48in diameter in 1980 as against today's 96 in diameter). Battered piles were the norm because surface-driven steam hammers required external pile guides which are not only costly to fabricate, but attract additional wave loading if they are not removed after installation.

Third-generation jackets demonstrate how better understanding of design loading can lead to significant cost savings. For example, better appreciation of vortex-shedding behaviour led to reductions in conductor diameter and to increases in bay height. Further benefits realised in the mid to late 1980's were the introduction and reliability of underwater pile-driving hammers with direct benefits in reduced fabrication of pile guides and indirect savings from reduction in wave loading. The above primary changes, together with secondary benefits in efficient bracing and framing patterns, all led to lighter structures and cost savings for late 1980's jackets.

The next development was lift-installed jackets, made possible by the increased lift capacity of offshore cranes which led 'fourth-generation' jackets. The introduction of these large heavy lift crane vessels has provided significant savings, with the possibility of lift-installing jackets in up to 200 m water depth.

Future plans are for tripod designs in up to 90 m of water, supporting lightweight topsides and braced mono towers for marginal field developments.

Design of early North Sea steel jackets were based on American design Codes and even today these dominate design of steel space-frame structures, not just in the North Sea but throughout the world. The main reasons for the use of API and AISC Codes are:

- British design Codes in the late 1960s (e.g. BS 449) when North Sea designs commenced dealt primarily with onshore building design. It did not cover offshore environmental loads and gave no appropriate guidance on the design of tubular members and joints.
- Following the issue of the first edition of the API (working stress) Code in 1969, it has remained the dominant design Code for offshore structures and is now in its 20th edition.

In future, there is likely to be an industry shift towards the new harmonised ISO standard for the design of offshore structures. A draft of this document was issued in 1996. The new ISO standard for design of steel structures is based upon the new American Standard (API RP 2A LRFD 1993), which adopts limit state design principles.

5.2. Topside structures

The American Codes which dominated jacket design have been used in topside design, albeit to a slightly lesser extent. Many of the large box and plate girders required to support modules (ca 1000-2000t) were designed using Interim, Design and Workmanship Rules (IDWR) the Merrison bridge rules, which led to BS 5400.

The trend for British Codes for topsides continues with, more recently, the use of BS 5950. This Code still presents gaps, as it is an onshore building Code and therefore does not give guidance on transient stage load factors for offshore installed structures such a load-out, transportation and life, nor does it offer guidance on accidental load factors for, say, dropped objects, ship impact and blast. However, there is evidence that the use of BS 5950 can lead to weight saving of around 5% compared to American Codes.

The future ISO standard should rationalise these Codes and provide one all-embracing design Code for jacket and topside designs.

5.3. Steels for offshore structures

Early North Sea structures used typically 'mild-steel' with yields in the range 275-300N/mm^2. In time the use of higher strength steel grade 355EM and EMZ became the norm, once consistent manufacturing and welding abilities were established. 'Mild steel' became limited to secondary structures such as walkways, ladders, deck plates, etc.

More recently, high strength (quench and tempered) steel with yield strength of 450N/mm^2 have been used successfully for the fabrication of large fabricated plate girders and rolled tubulars. It has also been used successfully in the fabrication of jackets. This has led to the greatest weight and cost savings compared with, say, design Code developments. Perhaps the future will bring even stronger steels for specialised parts of the structure.

The application of castings is now well proven in the offshore industry and data have been built up on their performance in fatigue sensitive areas and adequacy of NDT inspection techniques. However, overmatching weld strength is still a big issue to ensure robustness and avoid brittle fractures.

Fatigue behaviour of tubular joints has been researched extensively since the start of North Sea developments and today a lot more is known on the theoretical and actual behaviour. Sophisticated computer programs are now readily available, and offshore inspection techniques using ultrasonic testing make it possible to inspect complex joint geometries underwater. This vast knowledge on fatigue behaviour has given the industry confidence in its existing structures and the ability to predict behaviour of much larger structures in deeper water.

6. Stainless steel

The first stainless steels were developed in the early part of the twentieth century by adding chromium, an d later nickel, to carbon steel. Since then many different types and grades have been developed, each with their own performance characteristics and consequent areas of application. Unlike carbon steel, stainless steel has a natural corrosion resistance. In the presence of air, an oxide layer forms on the surface that inhibits corrosion. The layer is thin and reinforces the natural colour without compromising the characteristic metallic lustre. Significantly, this means that the surface of the material can be exposed without any applied coatings.

Stainless steel appeals to architects and engineers for its excellent mechanical properties, a wide variety of surface finishes and its durability. The first major architectural application of stainless steel was probably the cladding on the top of the Chrysler building in New York in 1929 (Fig 11).

Fig 11. Stainless steel cladding dating from 1929; Chrysler Building, New York

Today this building has become an affirmation of the longevity of the material. This longevity has gained stainless steel numerous engineering applications, sometimes seen and sometimes unseen. They range from simple and unassuming brickwork wall ties to heavy civil engineering structures, wherever the imperative to perform reliably over long periods with little maintenance dominates material selection.

The increase in the popularity of this material in construction has been reinforced by the development of improved grades offering guaranteed higher strengths, better corrosion resistance and an ever-increasing range of surface finishes. This has been coupled with the provision of effective design guidance for architects [2] and engineers [7].

Since the late 1980's, there has been a considerable amount of research and development work in Europe studying the behaviour of stainless steel members and connections. The culmination of much of this work has been in the publication of a European Standard dealing with structural design of stainless steel (EC3: Part 1.4 1996).

6.1. Common stainless steels and their composition

Stainless steels are alloys of iron containing a minimum of 10.5% chromium and usually at least 50% iron. The controlled addition of alloying elements results in a wide range of materials, each offering specific attributes in respect of strength, ability to resist certain atmospheric and chemical environments and to operate at elevated temperatures. Examples from within the major families of stainless steels and their compositions are shown in Table 1.

Table 1. Typical content of main alloying elements in the principal grades of stainless steels.

EN 10088 designation	Popular name	Weight (%)				
		Cr	Ni	Mo	N	Others
1.4301	304	18	9	-	-	-
1.4307	304L	18	9	-	-	Low C
1.4401	316	17	12	2	-	-
1.4404	316L	17	12	2	-	Low C
1.4541	321	18	10	-	-	Ti
1.4362	2304	23	4	-	0.1	-
1.4462	2205	22	5	3	0.15	-

Austenitic stainless steels are the most widely used and are based on 17-18% chromium and 8-11% nickel additions. This combination of alloying elements results in a different crystal structure of iron from that of ordinary structural carbon steels. Austenitic stainless steels have excellent resistance to general or uniform corrosion, different yielding and forming characteristics and significantly better toughness over a wide range of temperatures. Their corrosion performance can be further enhanced by additions of molybdenum. Austenitic stainless steels are also readily weldable. Grades 1.4301 (304) and 1.4401 (316) are readily available in a variety of forms, e.g. sheet, tube, fasteners, fixings etc. 1.4401 (316) has better pitting corrosion resistance than 1.4301 (304). Low carbon (L) grades should be used where extensive welding of heavy sections is required.

Duplex stainless steels have a mixed austenitic/ferritic microstructure and are based on 22-23% chromium and 4-5% nickel additions. Grade 1.4462 (2205) has generally better corrosion resistance than the standard austenitic stainless steels due to the higher content of chromium and presence of molybdenum and nitrogen. Duplex stainless steels are stronger than austenitic steels. They are also readily weldable.

6.2. Properties of stainless steel

The shape of the stress-strain curve for stainless steel differs from that of carbon steels. Whereas carbon steel typically exhibits linear elastic behaviour up to the yield stress and a plateau before strain hardening, stainless steel has a more rounded response with no well-defined yield stress (Fig 12).

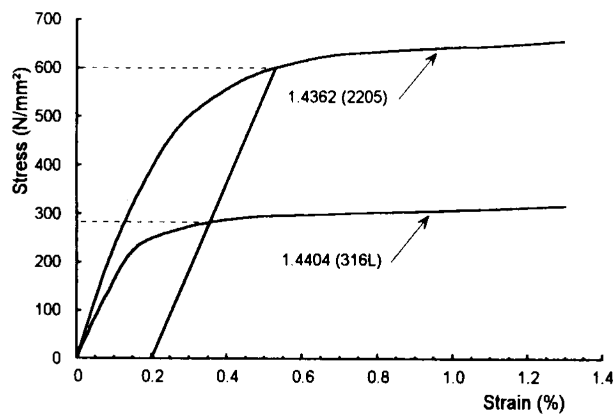

Fig 12. Stress-strain curves for stailess steel

Stainless steel 'yield' strengths are generally quoted in terms of a proof strength defined for a particular offset permanent strain, conventionally the 0.2% strain. The mechanical properties of hot rolled plate in grades 1.4404 (316L), 1.4362 (2304) and 1.4462 (2205) are given in Table 2. By way of comparison, the table also shows properties of some other widely used austenitic grades. All these materials are also available as cold and hot rolled strip with slightly different material properties. The values given in the table relate to material in the annealed condition. In practice, these values will be exceeded if the material is cold worked. There is provision within the new material standard (EN 10088: Part 2 1995) for supply of certain steels (including austenitic steels 1.4401 (316) and 1.4404 (316L)) as cold rolled strip with 0.2% proof strengths up to four times greater than those of the annealed material.

Table 2. Minimum specified mechanical properties to EN 10088-2 for hot rolled plate

Steel grade	0.2% proof stress N/mm^2	1.0% proof stress N/mm^2	Ultimate tensile strength N/mm^2	% Elongation
Austenitic				
1.4307	200	240	500-650	45
1.4401	220	260	520-670	45
1.4404	220	260	520-670	
Duplex				
1.4362	400		630-800	25
1.4462	460		640-840	25

6.3. Structural design of stainless steel components

The non-linear material properties of stainless steel give rise to differences in structural behaviour when compared with carbon steel. Consequently, different design rules should be used [7] which take into account the effect of material characteristics as well as appropriate levels of imperfections and residual stresses for cold formed and welded elements.

The effect of material non-linearity can be illustrated by reference to Fig 13. This shows theoretical Euler column buckling curves for stainless steel, derived by using a Ramberg-Osgood representation of the material stress-strain curve:

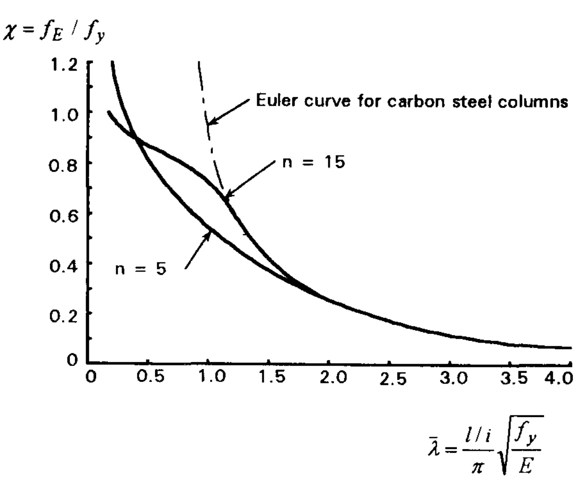

Fig 13. Effect of material non-linearity on buckling behaviour

$$\varepsilon = \frac{\sigma}{E} + 0.002\left(\frac{\sigma}{\sigma_{0.2}}\right)^n$$

where $\sigma_{0.2}$ is the 0.2% proof stress of the material and n is a constant which provides a measure of material non-linearity.

In studying the buckling behaviour of stainless steel members it is helpful to consider the effect of the non-linear stress-strain curve of stainless steel by reference to similar carbon steel members. Three distinct regions of slenderness can be distinguished:

a) At high slendernesses, i.e. when the axial strength corresponds to the linear part of the stress- strain curve, there is little difference between the strength of stainless and carbon steel members, assuming similar levels of geometric and residual stress imper fections. The limiting slenderness beyond which similar behaviour can be expected depends on the proportional limit and hence the n factor in the Ramberg-Osgood representation of the stress-strain curve. This can be seen in Fig 13.

b) At low slenderness, i.e. when members attain or exceed their full plastic capacity, the benefits of strain hardening become apparent. For very low slenderness, materials with higher hardening rates (low n factor) will have superior buckling strengths than those with high n factors (e.g. carbon steel). This effect too can be seen in Fig 13.

c) At intermediate slenderness, i.e. when the average stress in the member lies between the limit of proportionality and the 0.2% proof strength, stainless steel is "softer" than carbon steel. This leads to reduced stainless steel buckling strengths compared to similar carbon steel members.

7. Advances in fire engineering

Improving the cost-effective resistance to fire in steel structures remains a high priority for this industry. Substantial progress has been made with passive fire protection, modern materials have halved costs and minimised the impact on construction programmes and structural steel systems have been developed that do not require fire protection. In addition to these developments, one of the greatest achievements of the past decade has been the adoption of goal-based fire safety concepts and the application of structural fire engineering as a means of satisfying fire resistance requirements. These lead to a reduction in losses due to fire and to lower financial investments in fire safety measures.

7.1. Behaviour of steels at elevated temperature

At the core of developing fire engineering methodologies for steel structures is the understanding of the behaviour of the material with increasing temperature. There are two test procedures for the evaluation of the mechanical properties of steel at elevated temperature; these are known as the isothermal and anisothermal tests.

Isothermal or steady-state tests have been used for many years in mechanical engineering applications and the test procedure has been codified (BS EN 10002: Part 5). Tensile specimens are heated to the desired elevated temperature and strain applied at a steady rate. In this way the stress-strain curve at that temperature is generated. Fig 14 shows the stress-strain curves for a grade S275 steel at a range of temperatures.

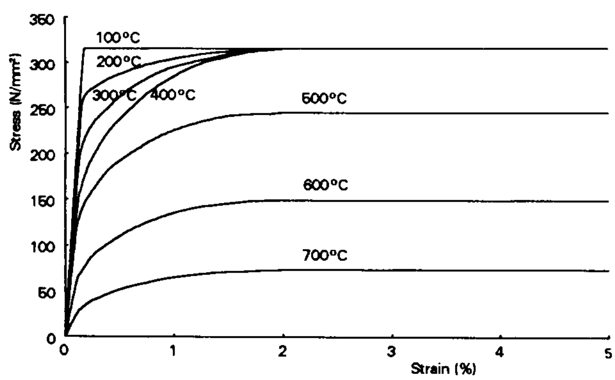

Fig 14. Isothermal stress-strain curves (S275 steel)

Anisothermal or transient tests where developed more recently for generating material data for fire engineering [10]. In these tests, the specimen is subject to constant load and the rate of heating is set at a pre-determined level (typically 10°C/minute). The resulting strains are measured and the effect of thermal strains is deducted using 'dummy' unloaded-specimens subject to the same temperature conditions. An example of the resulting strain-temperature curves is shown in Fig 15 for different values of initial applied stress. Stress-strain curves at a particular temperature are obtained by interpolation from a family of strain-temperature curves at different stresses. Anisothermal tests result in slightly lower strengths than isothermal tests for carbon steel. The difference diminishes with increasing strain levels (> 1%).

The value of strain at which the strength of the steel is measured is also of importance. At elevated

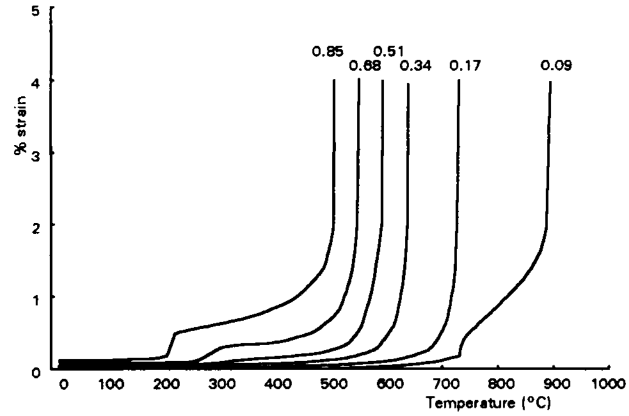

Fig 15. Anisothermal strain-temperature curves (S275 steel)

temperatures there is a gradual increase of strength with increasing strain. The selection of the appropriate strain limit (which reflects the behaviour of structural members in fire) is therefore important.

Values of elevated temperature 'yield stress' are defined in terms of a strength reduction factor. This gives the strength of steel at a particular temperature and mechanical strain as a ratio of the room temperature yield strength. The strength reduction factors at different values of strain for grade S275 steel are shown in Fig 16.

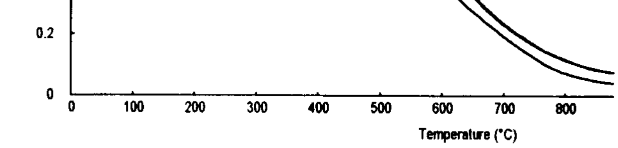

Fig 16. Strength reduction factors for S275 steel as unction of material temperature

7.2. Codified design methods

Numerous fire engineering tools and safety assessment methods have been developed. Most notable is that structural fire engineering has been codified (BS 5950: Part 8 1990, EC3: Part 1.2, 1993) and is now widely recognised by the regulatory authorities in many countries. Using these methods, designers are encouraged to treat fire as one of the basic limit states. The methods incorporate the level of load at the fire limit state, the type of structural member and the actual characteristics of the stress-strain behaviour of steel at elevated temperature.

By relating the critical failure temperature of members to the member load ratios, significant economies in protection requirements can be achieved. The load ratio describes the combined effect of all the force actions on the structural member in fire conditions. Mathematically, it is defined as:

$$\textit{Load ratio} = \frac{\textit{Forces or moments at the fire limit state}}{\textit{Member resistance to these forces or moments}}$$

For members in bending, the load ratio is simply the applied moment at the fire limit state divided by the moment capacity of the member. Clearly the higher the load ratio, the higher the required retained strength of the member in fire, and consequently, the lower the critical temperature. This is known as the limiting temperature and is tabulated as a function of member type (column, beam *etc.*) and load ratio. The limiting temperature varies with mode of loading because loaded fire tests indicate that the strain at failure is strongly dependent on the loading mode. Figure 17 shows the limiting temperature curve for columns together with existing test data. Figure 18 shows the equivalent curve for beams supporting a concrete slab [12].

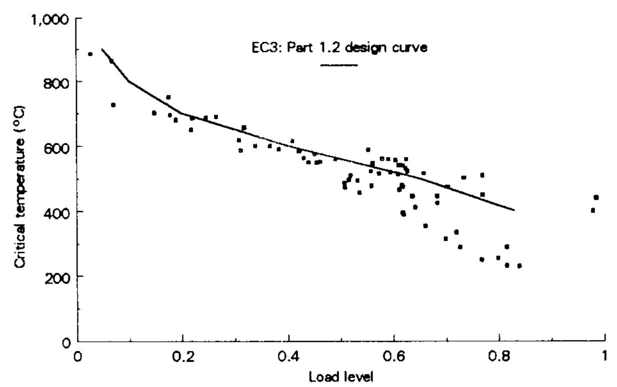

Fig 17. Critical temperature of columns as a function of load ratio

7.3. Use of advanced analysis in fire engineering

Analytical capabilities have developed enormously in recent years, from the examination of isolated beams and columns to the possibility of modelling complete building structures in fire. The next phase of development is likely to refine the treatment

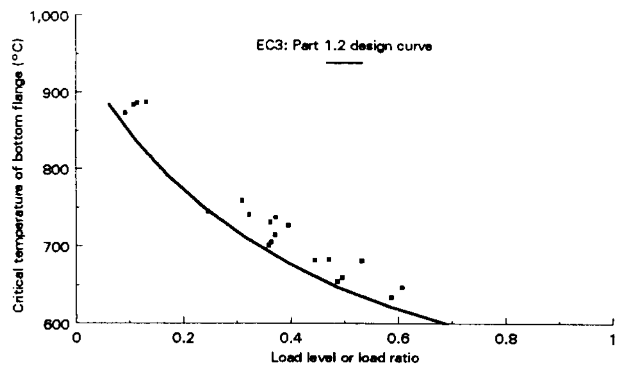

Fig 18. Critical temperature of beams as a function of load ratio

of steel-concrete interaction and to permit more realistic investigation of what really constitutes collapse. Recent tests [14] and analytical studies [3] have shown that failure temperatures are significantly increased if secondary actions, such as the ability of the slab to span over severely weakened beams, are accounted for. Unprotected steel beams are becoming a more realistic option, although it is anticipated that columns will generally still need to be protected.

The combination of full-scale tests and more powerful and validated analytical techniques will lead to continued improvements, more rational design methods and savings in passive fire protection requirements.

8. Environmental issues

8.1. Environmental investment and recyclability

With today's increased awareness of the vital importance of the environment, most people recognise that there is a counterbalance in terms of pollution and depletion of the world's resources for every industrial and social development. Yet the times when a pall of smog was the price a local community accepted for a thriving steelworks are long gone. Not only is today's steel industry motivated to match production processes to the planet's regenerative capacity but the requisite knowledge and technologies to achieve that result are catching up with that motivation.

Steel's unique environmental characteristic relates to the ease with which the material can be recycled. Steel is the most recyclable material known to man, and more than half the steel around us today has already been recycled from scrap: this is notwithstanding the fact that steel in bridges and buildings, for example, may stay in place for 100 years. Statisticians have estimated that up to 45% of the steel manufactured in the 1960's and 1970's is already being reclaimed with specific grades of steel scrap often fed back to the producer of the original material.

Scrap is an essential part of steel making. It provides a furnace feed for the electric furnace and is used as a top up material, to help cooling and slag formation, in many of the melts made in the basic oxygen furnace. All 'new' steel has a percentage of iron and steel scrap in it: some steels, such as the durable engineering steels used to manufacture gear boxes and transmissions may be manufactured almost entirely from scrap.

Recycling is of value too within the industry itself because it enables steelworks to consume its own surplus and rejected items - the tops and tails of billets, for instance, or the ends of ingots - for new steel making.

Energy consumption is a matter of concern for everyone. Technological developments have reduced the energy requirements for steel making by over 30% compared with 30 years ago. Moreover, the energy needed to make steel from recycled scrap is only 35-40% that required to make it from iron ore.

8.2. Building structures

In the construction sector, environmental burdens in the form of non-renewable energy and the production of CO_2 arise from all the life stages of construction materials, components, and complete buildings and structures. However, the 'operational phase' or 'building in use' phase accounts for the majority of the impacts. This has been widely reported by many experts [20] and has more recently been verified by the use of full environmental Life Cycle Assessment (LCA) studies. Researchers in Australia, Sweden, Finland and the United Kingdom [1] have carried out LCA's on a variety of buildings and have shown, in general, the domination of operational energy/CO_2 over embodied energy and embodied CO_2.

In order to reduce energy consumption and CO_2 many details can be considered at the design stage. Technical considerations of the interaction between the fabric of the structure and the air in the rooms, user-controlled natural ventilation, automatic night-time cooling, double skin walls, ventilation towers, atria, permeable suspended ceilings and other devices can all be effective.

The passive thermal performance of equivalent steel and concrete framed buildings has been another area of environmental research. A folklore has developed that the greater the physical mass of a building, the greater its effective thermal capacity for internal temperature control. However, new research shows that a massive form of structure is not needed to provide effective thermal mass.

The critical factor is admittance - the ability for heat exchange to take place from air into material. Only a fabric thermal storage capacity equivalent to around 70mm of concrete can be capitalised during daily cyclic temperature variations and, even allowing for effects of ramping up of temperature over sustained hot weather, greater depths of concrete tend to become irrelevant.

Conclusions

The demands of clients in the next millennium point towards higher performance construction materials and systems. These demands can be met by achieving the following objectives:

- Greater flexibility for maintenance, change of use and replacement.
- Better quality and aesthetics.
- Superior strength, toughness and ductility.
- Improved durability and resistance to corrosion, fatigue and fire.
- Lower life cycle costs.
- Improved safety in construction and operation.

Whatever construction application, development of successful solutions will require a fundamental reappraisal of material grades, design and construction processes to achieve these long term objectives.

For steel, advances in process and product technology, fabrication methods, joining techniques and protective systems are underpinned by a determination throughout the industry that the present competitive edge should be maintained and developed further.

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PLIENO KONSTRUKCIJOS NAUJAJAME TŪSTANTMETYJE

P.J. Dowling, B.A. Burgan

Santrauka

Nuo to laiko, kai daugiau nei prieš 100 metų buvo įdiegtas plienas, jis tapo viena iš svarbiausių statybos medžiagų. Šis straipsnis apžvelgia jo naudojimo raidos išdavas; apžvelgia pažangą, kuri padaryta kuriant plieno skerspjūvius ir konstrukcijų sistemas bei plieno naudojimą sausumoje ir jūroje. Taip pat aptariama gaisrosaugos, aplinkosaugos ir apsaugos nuo korozijos sistemų strategijų raida. Straipsnyje atskleidžiamas plieno konstrukcijų sektoriaus finansinis įsipareigojimas nuolat tobulinti rinkos kelią, kur procesų ir produkcijos technologijų pažanga visuomet bus skatinama klientų reikmių turėti kokybiškesnes medžiagas ir sistemas. Be praeities pasiekimų ir dabarties krypčių ir raidos apžvalgos, straipsnis nukreipia žvilgsnį į būsimojo tūkstantmečio plieno konstrukcijas.

Plienas yra moderniosios statybos sinonimas. Nuo praėjusio šimtmečio galo jis teikė galimybes ir įkvėpimą daugeliui projektuotojų kartų. Šiandien, architektūros pliuralizmo ir inžinerinių naujovių eroje, jis naudojamas naujais išraiškos ir techninio moderno lygiais. Tai tikriausiai iš dalies susiję su metalurgijos pasiekimais ir laimėjimais išsiaiškinant medžiagų konstrukcines savybes bei tobulinant gamybos technologiją; bet svarbiausia tai yra testamentas tolesniam architektų ir inžinierių įsipareigojimui kurti naudojantis išskirtinėmis projektavimo galimybėmis, kurias siūlo plienas.

Plienas taip pat yra dinamiška medžiaga; 70% plieno, kuris naudojamas dabar, nebuvo prieš 10 metų. Vartojimo instrukcijos nuolatos tobulinamos, nes kinta gamybos technologijos raida ir rinkos poreikiai. Plieno pramonė nuolatos investuoja dideles lėšas į naujas technologijas, tyrimus ir naujovių kūrimą, keldama reikalavimus mažinti masę ir gerinti tokias savybes kaip tvirtumą, takumą ir virinamumą.

Plieno pramonė veikia struktūruotoje sistemoje standartų, kurie apima tūkstančius vartojimo instrukcijų. Naujaip taikyti plieną tampa įmanoma tik sukūrus naujas jo rūšis, kurioms savo ruožtu rašomos naujos vartojimo instrukcijos, skirtos pagrindiniam ir daug utilitaresniam taikymui. Plieno lydiniai, kurie suteikė galimybę Britanijos lėktuvų variklių pramonei užimti deramą vietą pasaulio oro erdvės rinkose, dabar randa naujo taikymo galimybes įvairiose kitose technikos srityse, o nerūdijantis plienas tapo sparčiai plintančiu statybos gaminiu.

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Professor Dowling was President of the Institution of Structural Engineers from 1994 to 1995. He will be awarded an Honorary Fellowship of Imperial College (FIC**) on 23 October 1997.

He is the author or co-author of over 200 papers and six books and is the recipient of a number of awards from the Institution of Civil Engineers, the Institution of Structural Engineers and the Royal Institution of Naval Architects.

He has also received international recognition for his work by being presented with the Gustave Trasenster Medal from the University of Liege, by his election as an Honorary Fellow of the Singapore Structural Steel Society, and Foreign Associate of the National Academy of Engineering of Korea, by receiving the President's award of the Association of Consulting Engineers of Ireland and Honorary Doctorates, both from the National University of Ireland and, more recently, from the Gediminas Technical University of Vilnius, Lithuania.

Widely involved in the design of major steel structures, he was consultant to the design of the Thames Barrier and the Hutton Field Tension Leg Platform, both of which received the Queen's Award. He also led the development of European Design Codes in the field of steel structures as well as serving on many major international scientific and engineering committees.

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Since 1991 he has worked at the Steel Construction Institute. He is currently responsible for all the research activities relating to offshore structures, hazards, advanced analysis, materials and civil engineering. His work covers research, development of design guidance and design studies. In the field of fire engineering, he has been responsible for the development of design methodologies and the use of advanced techniques to optimise protection systems. In addition to offshore steel structures, his work covers other materials and structural forms such as stainless steel and composite steel-concrete structures, and the performance of these materials in fire. He maintains a "hands-on" interest in all the Division's technical work and regularly lectures on courses (including Continuing Professional Development cources) on a number of hazard and structural engineering topics.