The structural steelwork industry: A review

Richard Haryott

Introduction

My task is to paint a general picture of the structural steelwork industry today. I am going to concentrate on the situation concerning the role of steel in the market place, and in the main I will also be concentrating on those aspects relating to the use of structural steelwork in multi-storey frames, in one-off special structures where reinforced concrete is a successful or even dominant competitor.

Clearly the review is made through the eyes of an Arup engineer who is based mainly in London. In the time available it is obviously not possible to refer to all the many differences in circumstances worldwide, but nevertheless it seems to me that there are many areas of common experience and interest.

One of the purposes is to propose to you the following main points or themes:

1. That market forces are now such that steelwork is once again a sensible and economic alternative in many of our 'mainstream' structures in areas where concrete has perhaps been dominant for the past two or three decades.

2. That in the eyes of many specifiers, contractors and clients, there are many obstacles to the use of steel. These range from long memories regarding supply problems, lack of understanding of the scope for savings in time or material, to difficulties which exist in obtaining good advice on steel design.

3. That designing in steel can only be successfully accomplished if the task is seen as a part of the whole; that is to say the contract procedures, the fabric and finishes in relation to the structure, speed of construction, fire protection and so on must all be part of the evaluation. This may sound obvious, but in fact it seems to me that much of industry is not tackling the task as a whole.

A key must be education. There is nothing magic about designing steel, but the standard of education and training must be improved if excellence and value for money are to be attained. Arups should examine what their role should be.

Background market conditions

Steel sections have been used in construction for over 100 years and for many years the materials held a predominant position in bridgework and in major building frames. Competition from concrete and other materials gradually reduced its market share, but nonetheless the industry as a whole did not seem to run into a feeling of decline until comparatively recently. Steelmakers worldwide found themselves in a sellers' market for a considerable period of time, even if parts of the market, such as the construction industry, were losing a battle against concrete. In recent times, for a period of more than 20 years until around 1975, demand for steel products exceeded steelmaking capacity in most parts of the world, and certainly in Europe. One gets the impression that the most lucrative markets for the steelmakers were the ones that took the biggest volume of simple steels, namely ship building, the motor industry and heavy engineering generally, and as a result supply of steel in the form of sections and other structural shapes was never adequate. Delays in steel supplies were certainly seen as the rule, rather than the exception.

Sometime around 1975 the world supplies scene changed. The steelmakers, or the politicians, or both, appeared to get it wrong. In the UK for example, a grand expansion of steelmaking capacity had been made, taking capacity for production of liquid steel to something in the region of 30m. tonnes per annum, and at that with no real changes in productivity. In Europe capacity increased, and the new steelmaking plants were opened up all over the developing world. This all at a time when the post-OPEC price shocks were slowly reducing world demand and helping to bring about major recession.

Suddenly world demand for steel products fell far short of capacity. Steelmakers in the USA are working at less than 40% of their capacity, and in Europe the average output is around 60% of capacity. In the UK the BSC is currently producing around 10m. tonnes, and the private sector production brings the total to around 14m. tonnes — pretty much in line with the capacity/demand situation for the EEC as a whole.
Steelmakers have had to make a real effort to be more productive, particularly in the UK where overmanning was serious, and market forces have ensured that steel prices are now cheaper than they have been for a long time, in comparison to the main competition of concrete. Figures show the change in the last few years to be quite dramatic, no doubt even to an artificial extent because of government subsidies of one kind or another. Certainly in the USA and the main EEC countries, steel is being sold at a loss, but even if this changes, it still seems likely that overcapacity will keep prices down to make steel more competitive than in recent years, and supply problems are unlikely to be as difficult for some time to come.

So for the first time in many years, we find ourselves in a position where price and supply issues are such that steel is a material which must be considered more seriously, particularly as many clients or purchasers of construction are far more actively demanding steel, and are examining alternative design and contract arrangements to achieve better results. If we do not more actively make steel one of our design skills, we will be doing both ourselves and our clients a disservice.

Obstacles to the use of steel
The material conditions mentioned previously are clearly not the only reasons why use of structural steels declined in many of the world markets. Price and supply difficulties played their part, but many other factors also were of great significance.

Even in fields of activity where steel is a natural thing to use, things can go wrong which change the economic or technical balance. As an example we have only to think about the box girder bridge. After the Second World War a massive bridge building effort had to be undertaken in Germany to replace the hundreds of major structures destroyed or damaged, and the elegant and economical box girder steel bridge emerged as a dominant solution. Some 10 years or so ago the position was changed by the series of failures in box girders, partly brought about by a lack of understanding of both material and geometrical instability in the thin plates. The reasons were of course complex, but nevertheless one of the results was a new set of design rules (quite understandably) imposed on designers, which for a while tipped the balance of economy away from box girders to plate girders and even to concrete. For a while, in the UK and elsewhere, there was a dearth of steel bridge designs, and designers were lost to other disciplines. Lack of understanding and provision for maintenance probably also caused the pendulum to swing away from steel. In recent years, acceptance of weathering steels and the publication of simpler codes and rules have swung the pendulum back, as have problems being encountered in the life of concrete finishes.

In multi-storey buildings the pendulum had started to swing against steel much earlier. As for bridges, regulations of one kind or another were a contribution in the process, but mainly covering other problems such as fire protection. In some parts of the world, fire regulations are not very onerous, or perhaps not enforced, with the result that steel frames do not have to carry the additional cost and time penalty of encasement in concrete, splice coatings or dry linings. In some countries such as the UK it would be held that fire regulations are too severe, or perhaps unreasonably interpreted, adding unnecessarily to the cost of steel structures. We probably can all quote examples of that, and certainly many of us must have felt from time to time that more research is needed to understand better what happens to steel buildings in fires — not just isolated members making up frames. In Ove Arup & Partners, we are in a better position than many to seek and obtain good advice on this, and probably also in a better position than most to encourage new approaches, development of cheaper materials, and a wider understanding of the main issues.

At the same time that these supply, cost, legislative and other factors have helped swing the pendulum away from steel, the influence of research and graduate design aids promoted by the competing concrete industry pushed it even further. In the UK the Cement and Concrete Association has most effectively promoted the use of concrete, runs training courses for specifiers, clients, operatives and so on, and is a place to which many people in the building industry would turn for advice. They are a semi-independent organization, financed by levy on cement sales which produces them an income of around £5-6m. per annum. In contrast, the nearest equivalent in the steel industry in the UK is Constrado, which is wholly financed by the BSC, and in the minds of many is simply not able to cope in the same way on a budget of less than £400,000 per annum. The centres of research and information in the steel sector are scattered around manufacturing units, and are difficult to access. Information on such vital issues as corrosion protection, fire, special materials, welding and so on is not gathered into one centre of excellence, and in some cases the information can even be better obtained elsewhere. Apparently, for example, better advice on corrosion in steel can be obtained from the non-ferrous metals trade association than from the steel industry! In any event, familiarity in the use of steel at all levels of the industry has declined, and this is no small obstacle to its wide use. This lack of familiarity affects not only knowledge of the material, and analysis of complex details, but also the setting up of measurement, cost control, and contract procedures to take full advantage of potential savings of time and material in steel structures in those situations where it could and perhaps should be viable.

Design approach
It has been mentioned before that the market conditions for steel are such that it is now relatively cheaper and more readily available than for some time. With the pressures to construct more quickly (always there but probably now stronger as a result of high land values and the high cost of borrowing), we must be prepared to take advantage of steel frames in the right circumstances.

We have seen that the steel as a material has become more competitive against concrete as a material, but it is also worth considering what is happening to the 'finished' product. On simple structural frames, such as sections used in straightforward multi-storey frames, the fabricated and erected cost of the steel is perhaps more than double the cost of the raw material. Possibly now the labour costs are increasing faster than the material itself, but in any event they represent the area of greatest fluctuation. If fabrication is complex or difficult, then the finished product cost can increase dramatically, in some cases reaching more than seven times the cost of the raw material.

In this age of increasing accountability, and more measurement of everything, we are often under pressure to produce maximum area for minimum material, under the proposition that this will equate to minimum cost. Clearly this is not always so, and the greater use of material, if accompanied by less fabrication and greater simplicity, can easily be shown to produce a more economical result. However, even in this...
need to be careful, as recent developments in automated fabrication processes are in part changing the scene, and are making seemingly more complex fabricated shapes more economical through reduction in weight.

Again measurement methods and contract procedures must be considered as part of the design criteria. Constrado have recently (perhaps belatedly) published the result of independent assessments on several case studies which show steel-framed buildings to give very considerable savings in time over any reinforced concrete alternatives, but much would depend on the contract procedures adopted, for it can very easily be shown that in many cases to achieve the savings in time, steelwork sub-contracts have to be let before the main contract, committing the client to a process which he might not want. New forms of contract such as the JCT 80 in the UK are making nominated sub-contracts less attractive to clients, and there are vested interests which make it harder for the consulting engineer to overcome resistance to organizing contracts in some other form to overcome this. In my view this is something we must watch very carefully, particularly as there are signs that fabricators do not like quoting main contractors for steel sub-contracts, as they cannot be sure they are quoting to all the main contract tenderers, and are certain to incur far greater tendering costs, which will of course have to be passed on to the clients.

Complicated measurement is often employed, even for very small works, and inequitable conditions of contract imposed on the small man. Recent examples of this on a small steelwork sub-contract have produced tender figures perhaps three times as high as might be normal. By and large, this kind of difficulty does not apply to reinforced concrete work, and so we have to be careful to ensure that the right procedure be used if the overall aim is to be achieved.

Two new graduates from different universities in the UK were interviewed recently to see if their courses were perhaps biased towards concrete, rather than steel, and it was quite encouraging to discover that there was no bias. Both left university equally inexperienced in the two materials. However, it is plain, certainly in the UK, that from that stage forward familiarity with steel does not come easily. We have recently completed the design of a large and complex pharmaceutical research laboratory, and this was entirely a steel solution, with lightweight steel or aluminium cladding. At one stage, for planning and other reasons, the claddings to a two-storey office block were changed to brick, and the frame changed to concrete, because the architect preferred detailing brick supported by concrete rather than by steel. This illustrates that considerable effort is needed to change that confidence factor, and also the obvious point that, taken together, a decision for steel cladding and steel frames seems 'right' whereas the heavy cladding and perhaps steel frames seems 'wrong'.

Why?

The question of fire protection is fundamental. The addition of mineral fibre sprays or dry claddings, etc., increases the cost of the frame by more than 30% in the UK, although this perhaps represents only around 12% of the total cost of the structure, including floors. In the USA the increase is less because, presumably, there is far more competition in the fireproofing market. Within the fireproofing element, by far the largest element is for treating the beams (70-75%) as opposed to the columns. If we can do something to reduce the beam element, we may achieve worthwhile savings.

Conclusions

There is much evidence to suggest that steel as a material is now relatively cheaper than it has been for some time, and is far more readily available. There is a growing demand from discerning clients and specifiers to take account of this, and we should seize the opportunity to promote its use in those cases where we can gain advantage for our clients. It could be quite a potent weapon in our armoury.

There are, however, some obstacles in the way of the specifier and contractor in using steel. Many of these should be quite easy to reduce or even eliminate with the right approach. There is nothing magic about designing in steel, but rather it is a simple question of education and training. The industry has lost a considerable amount of experienced people over the years, and we must examine how we are going to fit into, or even adapt to future needs.

In designing in steel, we must not lose sight of the whole. This is true of any material, but in the present state of the art, it is particularly so for steel. We have seen how contract matters, questions of measurement estimating, and speed of construction are all major factors in the design process.

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Fig. 3
Hall 7, National Exhibition Centre
Birmingham: a successful use of exposed structural steelwork
Architects: Edward D. Mills and Partners
(Photos: Ove Arup & Partners)
Fire protection
Margaret Law

Introduction
In 1980, Ove Arup and Partners and CBLIA (Centre Belge-Luxembourgeois d'Information de l'Aacier) completed a joint study of fire protection using structural steel elements exposed to fire. The study was financed by the European Coal and Steel Community (ECSC) with the purpose of identifying priorities for research in the field of fire safety of building construction. We interpreted the broad purpose of ECSC-funded research to be to increase the use of steel in building. We therefore tried to identify not only obstacles to the use of steel but also circumstances favouring its use. This note summarizes our findings and includes some later information.

Behaviour of steel in fire
As with other structural materials, the mechanical properties (tensile strength, yield strength and Young’s modulus) decrease with temperature. Values have been codified for design. Mild steel, provided it has not exceeded a temperature of about 900°C, recovers most of its strength on cooling.

The heated steel element will fail when the yield stress decreases to the value of the working stress. (This can alter with the formation of plastic hinges.) The steel temperature at this moment is defined as the critical temperature. It depends essentially on the loading, the degree of restraint, the end conditions, and the type of steel. It is therefore a mistake to define a single value, 540°C for example, for the critical temperature of steel.

Before the element attains its critical temperature, it can deform under the effects of expansion and reduction of Young’s modulus caused by increase of temperature and/or temperature gradients in the section. In fires larger deflections than normal can be accepted, generally at least span/30.

Because of its high thermal conductivity, uniform temperatures are attained rapidly in many steel sections. For a given rate of heating, a heavy section will heat up more slowly than a light one; the massivity of the section (volume in relation to exposed surface area) has a significant effect on its performance in fire.

Steel structures and fire safety
There is no evidence to suggest that in buildings designed to modern standards of construction, the structural material itself has an effect on the life risk, i.e. there is no reason to believe that a person in a modern steel-framed building is more likely to suffer injury or death by fire than a person in a modern building of concrete, brick, wood or masonry construction.

Most major property losses in fires are those in industrial buildings – warehouses and large production and storage areas – and it is the loss or damage of the contents which makes up the main cost of the fire. The causes of fire spread include such features as combustible materials on wall and roof surfaces, delay in detection, lack of compartmentation and poor details at junctions of compartments. There is no reason to believe that the use of steel construction is more likely to lead to these major fire losses.

Fire protection of steel
Properly designed steel construction can give adequate fire resistance and in many situations subsequent repair, particularly for mild steel, is a simple operation. In certain circumstances very little fire resistance may be needed, either because the potential fire exposure can be demonstrated to be low or because the steel can be sacrificed in the event of fire without prejudice to the overall fire safety objectives.

Normally, it is necessary to design the elements so that the steel does not reach its critical temperature within the fire resistance period imposed by regulations. Regulations take into account experience of building fires and, in a somewhat arbitrary way, the nature, height and use of the building. The periods of fire resistance specified range from a half hour to two hours, or even four hours for certain locations deemed to be particularly dangerous.
In certain circumstances there are no fire resistance requirements and the steel needs no protection. When protection is needed the following measures may be possible:

- exploiting the advantages of low stress, cold working, residual cold working effects
- placing elements outside the building
- cooling elements by automatic water spray
- cooling hollow elements by water filling
- placing screens (false ceilings, partitions) between the element and the fire
- cladding the element with an insulating material. Such materials can be intumescent or can replace, that is, block the elements. They can be applied alone or mixed, and sprayed, travelled or in the form of boards.

### Steel without cladding

**Fire resistance of unprotected steel elements**

Where the performance requirement is dictated by building regulations then it is clear that the designer must provide a fire resistance of at least 30 minutes. For a given grade of steel and stress level, the fire resistance varies inversely with the 'section factor' $\frac{P}{A}$ m$^{-1}$, where $A$ is the cross-section area and $P$ the exposed perimeter. Small elements in common use have a section factor of 250m$^{-1}$ or greater and for an unprotected mid steel element with the usual design stress this gives a fire resistance of about 15 minutes when the element is subjected to the standard fire resistance test. (Massivity is the inverse of the section factor).

To achieve larger periods of fire resistance it is necessary to vary, either singly or in combination, the factors that affect the temperature rise, the stress level and the grade of steel. A column with $\frac{P}{A} = 29$ m$^{-1}$ (equivalent to a solid 140mm square) would give 30 minutes fire resistance $\frac{P}{A} = 11$ m$^{-1}$ (equivalent to a solid 350mm square) would give 60 minutes (average temperature of 540°C).

The relationship between fire resistance and stress level can be estimated fairly readily for elements with uniform temperature distribution. The results of fire tests on hollow steel sections shows that the temperature of steel beams varies both along and across the section and failure may not occur even when the temperature of the lower flange exceeds the critical value of 600°C. At the moment, since elements in common use have a section factor of 250m$^{-1}$ or greater and for an unprotected mid steel element with the usual design stress this gives a fire resistance of about 15 minutes when the element is subjected to the standard fire resistance test. (Massivity is the inverse of the section factor).

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To what extent is (or ought) design to be realized, fewer cared for the audience was he talked about the design process. Few 'luminaires' - his term for the ack(d) For further guidelines reference can be efforts. Derek Sugden's talk was as timely as it was fascinating. He led his audience, code gives guidelines aimed at limiting particularly for unpropped construction, could result from deflections. Deflections during the construction stages, the steel beam must not exceed 0.9 Fy under limit of span/250 is suggested with span/500 modular ratio of n for transient loads and 3 n deflection moment of inertia and effective section stress method is in accordance with elastic for unpropped construction the stresses in situations. Full design procedures require: selection of slab thickness and negative force can be transmitted to the concrete. The function of the shear connector is to fix metal decking to support beams. Alternatively stud shear connectors adequately fix metal and assuming no slip at the concrete steel stress method is in accordance with elastic assumption the intuitive solution with that design problems have, at least initially, seemed to have been placed there of solve the problem on its own merits. Voltaire find new solutions to novel problems. It of the slab.

Concrete stress in the composite slab.

deflection of the composite slab

Total panel stress due to slab dead load

Check on panel stress during concrete loadings

Ductility

Deflection

Concrete stress in the composite slab.

Permissible stresses and deflections are fire resistance in excess of 30 minutes either the steel sheet must be insulated by a coating or suspended ceiling or the concrete must have reinforcement. Although numerous fire resistance tests have been carried out. It is difficult to generalize the information received so that a design method can be provided. Most tests have been for statically determinate floors; better performance has been measured with continuous floors. Although in some countries the failure criterion for a floor is a deflection of span/30, these slabs can reach a deflection of span/20 or more while still retaining load-bearing capacity. A failure criterion of infinite rate of deformation is therefore more appropriate. Heat transfer calculations have not so far been very successful probably because the method needs to be more complex for concrete than for steel.

Composite steel and concrete beams

Tests have been carried out with both isostatic and hyperstatic beams for varying conditions. Some large temperature gradients were measured in the steel, between the flanges and over the supports. A calculation of critical temperature assuming a uniform temperature across the section is in reasonable agreement with the average of the temperatures measured on the upper and lower flanges. However, the calculation of the heat flow to the section could be complex and has not yet been attempted.

Single elements and frameworks

Fire resistance tests are normally carried out for single elements. The relationship between the behaviour of these elements when supported in the test furnace and when installed in a building is not well established. Work on loaded external column and beam assemblies shows greater

Figs. 3-4
8 A ‘critical’ steel temperature can be defined in all countries but Great Britain and Italy, being either a fixed value or one established by calculation. Italy recommends some fixed values for unloaded structural elements.

**Unprotected steelwork**

All countries except Sweden have buildings with unprotected external steelwork. Only one, Italy, has agreed design rules.

All countries accept unprotected steelwork for single-storey buildings (in Belgium for industrial buildings only, in France, Germany and Switzerland for road structures only sometimes in Germany and exceptionally in Belgium). Seven countries accept unprotected steelwork for car parks (in Germany, Italy and Sweden only sometimes as accepted in Belgium and France). Switzerland alone accepts unprotected steelwork for any building with low fire load (less than 60M cal/m² which is equivalent to about 14 kg/m² of wood).

**Criteria for fire resistance**

Centres in all but two countries – France and Sweden – think that the standard fire resistance test should give more information on the materials at the end of the test and all but two (France and Germany) would like more information on the adhesion of the cladding. Only one country (Australia) has a code of practice for the application of cladding to the steel. Centres in five countries are in favour of such a code.

Centres in three countries (Australia, Germany and Switzerland) have collided with their existing criteria for failure of beams and slabs. Four countries (Belgium, Great Britain, Italy, Switzerland) favour a limiting rate of deflection, one (France) an infinite rate, and the Netherlands (Sweden) favour a limiting rate for certain structures only.

**Insurance requirements**

All Centres but one (Great Britain) said that insurance requirements discourage the use of steel in some buildings.

**General comments**

The comments and observations reveal a number of problems: a lack of knowledge of fire protection on the part of the building authorities; an unjustified prejudice against the use of steel in buildings; and a lack of cooperation between the authorities and insurance companies; lack of a clearly stated performance requirement in the building regulations – what are they trying to achieve and how do the fire resistance periods relate to real fires? The cost of fire protection, particularly for 30 and 60 minute fire resistance periods, presents problems. It is considered that comprehensive tables, design manuals and more acceptance of alternative measures to meet building regulation requirements would help to promote the use of steel in buildings.

**Priorities for future activities and research**

From the investigations described in the preceding chapters we concluded that the following wider and more general problem will mainly be achieved by a reform of regulations based on:

- a more precise definition of performance requirement
- a more precise estimation of structural performance and for designers:
  - design manuals for the structural engineer
  - simple design guides for the architect and engineer which show how much protection, if any, is needed for the structural steelwork.

**References**


(9) CONSTRANDO. The behaviour of steel portal frames in boundary conditions. Constrando, 1980.
Towers and flare stacks  
John Tyrrell

Introduction
Towers are by implication tall structures that reach above the surrounding environment. They are commonly used to support equipment or some installation at a required elevation as part of an engineered system or process. The greater part of our work in this field has been associated with towers supporting microwave antennas but more recently we have designed a number of structures for supporting gas flaring systems and vent piping. These structures are sometimes designed or analyzed including lighting towers, towers supporting water storage tanks as well as guyed structures. However, this paper concentrates on microwave towers and flare towers.

Clearly there are similarities between the different types of structure but the operational or performance requirements of the system determine the functional requirements of the structure in each case. Steel is favoured largely because it can easily be fabricated as component parts and then transported and erected relatively quickly and simply in remote locations.

Types of structures
There are two main generic structural forms: self-supporting towers and guyed arrangements. Self-supporting towers are normally of the open lattice type and may be either square or triangular in plan. Guyed structures may comprise a vertical or inclined slender stem with the stiffness provided by the guys. The combinations and applications of these types of structure are too numerous to cover adequately in this paper and therefore it is intended to consider microwave and flare towers only. A brief description of the main characteristics of each system is given below. Reference is also made to transmission towers where a different design approach is adopted and different loading conditions apply.

Microwave towers are usually constructed from tubular and/or angle members bolted together. For a particular network they are commonly arranged as a family or families of towers. Each family typically consists of a number of panels arranged vertically. Panels can then be omitted either from the top or from the bottom for the required height. Each panel is braced in elevation and possibly on plan. Thin bracing members may be braced by secondary members to reduce the effective length and therefore the member size. The towers are designed to carry a wide variety of antennas such as TV radiators, yagis, open or solid dishes and horn aerials. Each type of antenna has its individual mounting and access requirements as well as the feeder cable and waveguide arrangement.

Flare towers are required to support gas flaring systems which broadly consist of ducting waste gas by pipework to a required elevation where it is ignited by a remote burner. The size of pipework or riser varies with the particular gas flows up to 1.5m in diameter. Ancillary pipework such as feed lines to the flare tip are also required to be supported. Removal of the flare tip for maintenance is an important feature of the design. These structures tend to be "one off" and constructed from welded tubular members although not exclusively so. Offshore tower structures frequently have restrictions imposed on the design such as

the base width and method of installation. They are fabricated and transported in relatively large sections to minimize erection time. Aggressive environmental conditions and infrequent ‘safe’ access periods are taken into account by specifying high quality materials and workmanship.

Transmission towers are required to support the overhead power supply lines at relatively frequent centres. The power lines are tensioned to reduce the catenary effects and are suspended from an arm arrangement as part of the tower. This in turn creates forces in the tower in addition to the wind load. Also tensions in the power lines are affected by climatic temperature variations. Because of the number of towers required for any one system the cost penalty for conservative design could be significant. The design approach therefore is to test a tower or number of towers to failure, so the strength of all members, including those normally assumed to be redundant for analysis purposes can be properly evaluated.

General tower arrangement
In cases where the tower geometry is not given or determined by factors other than strength or deflection the following empirical rules for microwave towers may be applied as a first stage towards achieving an economical design.

Slenderness:
The ratio of the overall tower base width should not exceed the values tabulated below.

<table>
<thead>
<tr>
<th>Tower height</th>
<th>Maximum deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.75°</td>
</tr>
<tr>
<td>50m</td>
<td>6</td>
</tr>
<tr>
<td>50 - 100m</td>
<td>5</td>
</tr>
<tr>
<td>100 - 150m</td>
<td>4</td>
</tr>
</tbody>
</table>

Top width:
2.25m for square towers with internal climbing ladder; 1.65m for triangular towers with internal climbing ladder: 0.45m for towers with external climbing ladder.

Leg slope:
The slope of the tower legs at the base should be such that the point of intersection of the projected lines of the legs occurs above the two thirds point.

Panel types:
The optimum height to width ratios of the typical panel bracing arrangements are:

<table>
<thead>
<tr>
<th>Height/Width</th>
<th>Panel type</th>
</tr>
</thead>
<tbody>
<tr>
<td>h/w&gt;1.5</td>
<td>Z</td>
</tr>
<tr>
<td>1.0&lt;h/w&lt;1.5</td>
<td>X1, X2</td>
</tr>
<tr>
<td>0.7&lt;h/w&lt;1.0</td>
<td>X1, X2, K1, K2</td>
</tr>
<tr>
<td>0.5&lt;h/w&lt;0.7</td>
<td>K2</td>
</tr>
</tbody>
</table>

Also, the angle θ, between the bracing and the leg member, should be maintained within the following ranges:

bolted structures 20°<θ<90°
welded structures 30°<θ<90°

Solidity ratios:
Typically solidity ratios for different widths are as follows:

<table>
<thead>
<tr>
<th>Panel width</th>
<th>Solidity ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1.5m</td>
<td>0.25</td>
</tr>
<tr>
<td>&lt; 5.0m</td>
<td>0.20</td>
</tr>
<tr>
<td>&lt; 10.0m</td>
<td>0.15</td>
</tr>
<tr>
<td>&gt; 10.0m</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Factors affecting the design
Project requirements normally provided by the client or the systems engineer include: site location and particular details that are known to provide a constraint of one sort or another on the design basic wind speed, the combination of wind and ice accretion and temperature where appropriate size, type and number of items to be supported at specified levels including associated feeder systems limits on the twist and tilt for the structure design life of the structure aircraft warning lighting painting and protection.

On occasions we may be required to carry out wind speed investigations where sufficient details are not given. Also, it is quite frequent that further interpretation of the design requirements is necessary.

In addition to the specified requirements there are other factors that are likely to influence the design such as economic and practical considerations. For example the availability of steel sections locally, transportation, lifting and erection, the maintenance of the structure and the equipment including access are all parts of the equation. Also, the lightest possible structure is not always the most economic, particularly where some standardization can be achieved and where there are difficult ground conditions and the cost of the foundations may be a significant part of the overall cost.

Loading
Wind
A basic wind speed corresponding to the 3 second gust with a return period of 50 years is usually adopted for the design, or serviceability condition of the structure. On occasions an extreme or survival wind speed may also be given for a specified limit state. Force coefficients for individual members and lattice arrangements are given in CP3 Chapter V: Part 2: 1972.

Fig. 1  
Remote microwave tower site
Wind forces on antennas can be obtained either from manufacturers' literature based on wind tunnel tests or derived from the method given in 'Calculation of wind forces and pressures on antennas', by Cohen, Vellozi and Suh.

**Snow and Ice**

Snow and ice will contribute to the gravity load on the structure but the accretion of ice on individual members or antennas will increase the surface area and hence increase the drag forces due to wind. Icing is expressed in terms of radial thickness and some guidance is given by the Department of Energy in 'Guidance of the design and construction of offshore installations'.

For the icing condition a reduced wind speed is considered and this is in the order of 90% of the design wind speed depending on the locality.

**Temperature**

Loading from climatic temperature variations is not normally significant for these types of tower structures. Temperature effects due to hot or cold gases in the risers and due to radiation from flaring may induce local stresses and possibly influence the choice of materials.

**Waves**

Offshore structures such as towers mounted on platforms may be required to withstand wave loading. The effects are usually assessed by applying an appropriate acceleration input to the base of the tower.

**Earthquakes**

Earthquake loading is rarely significant in the design of towers. In seismically active areas the effects are considered by applying a base acceleration obtained either from an appropriate ground response spectrum or from a response time history.

---

**Method of analysis**

Lattice tower structures are considered as three dimensional space frames. Fully triangulated structures with bolted connections are assumed to be pinned frameworks. There are small errors involved in this assumption such as the legs being continuous at the nodes but these effects are rarely structurally significant. Separate checks may be carried out for bending moments arising from locally applied forces or eccentricities at connections. For fully welded structures or statically indeterminate structures, framing moments must be evaluated.

There are two in-house computer programs available for carrying out a static analysis of these structures, TOWER and PAFEC. TOWER was developed specifically for pin-jointed statically determinate microwave towers and includes analysis, design and plotting facilities. The main members and the secondary members may be specified by size in which case a straightforward analysis is carried out. Otherwise the program performs an iterative analysis of each panel, selecting from library files those member sizes not specified, until a satisfactory design is achieved.

Deflections and rotations are calculated at the top of the tower, at the top of each panel and at the level of each applied load. The foundation forces are calculated at each leg of the tower and steel weights are totalled by member type and steel grade.

In most cases self-supporting lattice towers are not dynamically sensitive. As a general guide, structures having a fundamental period of oscillation less than three seconds can be designed safely on a quasi-static basis. There are exceptions to this where dynamic effects should be considered and they include: fully welded structures where fatigue may be significant structures supporting TV antennas, radiators, particularly at the top tubular members with a slenderness ratio above 180 and local cross wind effects may be significant.

There are a number of computer programs available for carrying out a dynamic analysis. In-house programs include DYMAST, GLADYS, DAFT and PAFEC but only DYMAST is suitable for a random stochastic analysis.

---

**Fig. 2**

Jebel Ali microwave tower, designed by Ove Arup & Partners
(Photos: Mike Shears)

**Fig. 3**

Microwave tower under construction in Nigeria, designed by Ove Arup & Partners
(Photos: Mike Shears)
The greatest tonnage of structural steel is used in heavy industrial works such as the actively promoted by the Australian Institute publishes journals, magazines and safe load structures.

The Dundee Theatre is a triumph of ingenuity (3) AUSTRALIA

At Pitlochry, a holiday town in the Highlands to 18m. city site. Steelwork is used in the auditorium and the Pitlochry Festival Theatre.

In the case of theatres, steelwork provides important cable-stayed road bridge with a grade 50D tubular steel.

In spite of strong competition from the reinforced and prestressed concrete but we finds applications to heavy duty structures, unless they appear as a feature in, they are concealed above lightweight thermal insulation.

disadvantages of each of these sections may not be immediately apparent and they are summarized as follows:

* Tubular members
  - Land themselves more readily to a variety of plan shapes.
  - The range of sizes is greater but they may not be readily available.
  - They require special skill in fabrication, particularly for profiled connections.
  - They are more expensive than angles, weight for weight.
  - They are structurally more efficient and attract less wind load than angles.

* Equal angles
  - Best utilized for square towers.
  - Maximum size normally available is 200mm x 200mm.
  - Easy to fabricate and readily available.
  - Cheaper than tubes weight for weight.
  - They attract more wind load than tubes.

Cruciform angles

They have the same advantages and disadvantages as equal angles.

They require complicated battening and splicing details.

Fabrication

Knowledge of the fabrication process is essential to the detailing of steelwork. It is also important when considering suitable sub-contractors for carrying out the work as the range of skills and facilities available from steelwork fabricators both in the UK and overseas is wide and varied.

Fabrication drawings are normally prepared by the fabricator based on engineering design drawings. It is recommended that the fabrication drawings are checked to minimize misinterpretation of the design requirements and inspections of the steelwork fabrication are carried out to check for quality, workmanship and performance.

It is common practice for trial erections of the tower or parts of the tower to be carried out in the fabricator's yard prior to shipment. This avoids the situation where errors found during erection are difficult and expensive to rectify. Also, base setting templates are used to ensure that the holding down bolts cast into the concrete foundations match the steelwork interface.

Conclusions

The experience we have built up over a period of years enables us to respond quickly to the demands of our clients and advise them on all aspects of the design and construction of tower structures. Our analytical tools are sharp and we keep abreast of design development in the related fields.

A large part of our work is overseas and from time to time we have relied heavily on our overseas offices for local support. We have enjoyed their understanding and close cooperation and we hope that this working relationship will continue.
The use of plated steelwork in a tension leg platform design

Nick Prescott

Introduction to the TLP

A fixed offshore oil or gas production platform structure is designed to provide a safe working platform in an exposed marine environment. On this working platform the various processes associated with oil or gas production must be carried out in all weathers and without interruption. These would include the primary separation of fluids extracted, the control of their export and the services associated with continuous production. They may also include the equipment for the drilling and testing of the wells, and for the injection of water or gas into the field.

With major isolated installations, considerations of continuous operation would require the platform to be manned round the clock, and hence accommodation units with the associated back-up services would have to be placed on the platform.

With a tension leg platform, such as the example TLP, the operational constraints are the same as those for a conventional fixed platform, since it must receive the same operational approval. The TLP concept uses a floating vessel, a semi-submersible type structure, which is connected to the sea-bed in such a manner that the vertical tension legs are kept taut by the vessel’s excess buoyancy. The vessel configuration chosen for the example TLP is a simple unbraced framework in which the members are formed from stiffened steel plate components using stressed skin design as in a box girder bridge. This concept results in close structural interaction between the square vessel base (the pontoons), the deck (which contains all the plant and facilities) and the six interacting columns (Fig. 1).

The tension leg designs were originally put forward as deep water structures for sea depths between 1,000 and 2,000ft. However, the field, at 482ft, in the East Shetland Basin area of the North Sea, is being developed using such a platform since the TLP has been recognized as being cost-effective for intermediate water depths. Had the process requirements been radically more complex, the platform payload could have become too great for a TLP type solution. In addition, the overconsolidated soils of the North Sea make the tension foundations economically viable for intermediate water depths. Details of the application of the TLP to the North Sea development can be found elsewhere.

Design approach for TLP

Tension leg platform design can make good use of the limit state approach. In the case of the example TLP this was based on the DnV2 (Det Norske Veritas) rules. Static loadings on a TLP vessel consist of permanent loadings (structure and equipment), functional loadings (consumables and live loads), buoyancy and tension leg loadings. To ensure that equilibrium is maintained, all the partial load factors (γP) on these loads have to be considered as unity and member forces factored by an appropriate stress factor. This stress factor (γM) should be chosen to ensure compliance with the ‘Ultimate limit state’ as defined in DnV Section 4.

Environmental wave and wind loadings can be treated in a similar manner with a partial stress factor (γM) chosen to be compatible with the DnV recommendations. Table 2 summarizes the partial stress factors used in the design of the operating, extreme, and fatigue limit states for the Hutton TLP.

Having carried out analyses appropriate to

---

**Fig. 1**

Schematic view of Hutton TLP and, below - the geometry

**Table 1:** Geometry (All dimensions to moulded lines)

<table>
<thead>
<tr>
<th></th>
<th>Between column centres</th>
<th>Overall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>78.00m</td>
<td>95.70m</td>
</tr>
<tr>
<td>Breadth</td>
<td>74.00m</td>
<td>91.70m</td>
</tr>
<tr>
<td>Height</td>
<td>57.70m</td>
<td>11.25m</td>
</tr>
<tr>
<td>Draught</td>
<td>Operating</td>
<td>32.00m at LAT</td>
</tr>
<tr>
<td>Freeboard</td>
<td>To underside of main deck</td>
<td>24.50m at LAT</td>
</tr>
<tr>
<td>Water plane:</td>
<td></td>
<td>1324.00m²</td>
</tr>
<tr>
<td>Columns:</td>
<td>4 Corners</td>
<td>17.70m dia.</td>
</tr>
<tr>
<td></td>
<td>2 Centre</td>
<td>14.50m dia.</td>
</tr>
<tr>
<td>Pontoon:</td>
<td>Height</td>
<td>10.80m</td>
</tr>
<tr>
<td></td>
<td>Width</td>
<td>8.00m</td>
</tr>
<tr>
<td></td>
<td>Corner radius</td>
<td>1.50m</td>
</tr>
<tr>
<td>Displacement:</td>
<td>Approx.</td>
<td>61500 tonnes</td>
</tr>
<tr>
<td>Total weight:</td>
<td>Including riser tension (Approx)</td>
<td>48500 tonnes</td>
</tr>
</tbody>
</table>

---

**Fig. 2**

Hutton TLP 1.33 model under maximum height, maximum steepness, regular waves in NMI main test tank, London

**Fig. 3**

Hutton TLP tower hull under construction in the dry dock at Nigg. Pontoon is about to be lifted in.
each set of loadings, elements can be checked for the complete design condition for compliance with DnV rules by summing member forces in accordance with the relationship for element force:

\[ F = \sum F_i \gamma_p \]

where \( F_i \) is the element force for load case \( i \), and \( \gamma_p \) is the appropriate partial stress factor. Using this approach the overall safety factor for the design can be expressed in the form:

\[ \text{Safety factor} = \gamma_m \gamma_f \gamma_p K \]

where \( \gamma_m \) is the partial factor on material strength, \( K \) is the slenderness ratio and \( \psi \) is the load redistribution factor.

In addition, the use of fracture mechanics in the design of thick plate members enables the designer to ensure that the elements prone to fatigue damage are not limited by fracture toughness resistance. This ensures compliance with the basic limit state philosophy of preventing potential catastrophic modes from dominating the design. In the case of the present project it also enabled fatigue assessments made with reference to BS 5400 Part 10 SN curves to be correlated.

**Configuration of the TLP deck**

The use of box girder technology can be seen most clearly in the form and behaviour of an integrated deck structure. The design of the hull is closer to shipbuilding, in particular to submarine hull construction practice. The TLP deck spans between two lines of three columns, each using three main and two intermediate plate girders.

The deck configuration is completed with edge girders running along the column lines, a central girder to reduce differential deflections, and three shorter girders to provide full perimeter support for the well bay area, with its riser tensioners and drill derrick support rails.

The deck is an all-welded structure formed by these main plate girders, or bulkhead girders, and the deck plating panels. The bulkhead 'flanges' are formed by compact 'I' section chords at the web plate extremities. Both deck plating and bulkhead girder webs use stiffened plate construction. The integrated deck structure is shown in Fig. 6.

---

**Table 2. Typical partial stress factors (\( \gamma_p \)).**

<table>
<thead>
<tr>
<th>Load categories</th>
<th>Operating (1/12th year)</th>
<th>Extreme (100 year)</th>
<th>Fatigue</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent (P)</td>
<td>1.3</td>
<td>1.0</td>
<td>N/A</td>
</tr>
<tr>
<td>Functional (F)</td>
<td>1.3</td>
<td>1.0</td>
<td>N/A</td>
</tr>
<tr>
<td>Static Buoyancy (B_s)</td>
<td>1.3</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Dynamic Buoyancy (B_D)</td>
<td>wave generated</td>
<td>1.3</td>
<td></td>
</tr>
<tr>
<td>Inertia</td>
<td>1.3</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Tethers – due to permanent (T_p)</td>
<td>1.3</td>
<td>1.3</td>
<td></td>
</tr>
<tr>
<td>Tethers – due to functional/live (T_f)</td>
<td>1.3</td>
<td>1.0</td>
<td>N/A</td>
</tr>
<tr>
<td>Tethers – due to environmental (T_E)</td>
<td>1.3</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Environmental (E)</td>
<td>1.3</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Deformation (D) Temperature, lack of fit, etc.</td>
<td>1.0</td>
<td>1.0</td>
<td>N/A</td>
</tr>
<tr>
<td>Accidental (A)</td>
<td>1.0</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

---

**Fig. 4**

Pontoon section being lifted into position during assembly at Hutton TLP lower hull at Nigg.

**Fig. 5**

Hutton TLP column under assembly at Nigg. Outer shell about to be lifted over inner shell to form splash zone double skin 'damage control' section.

**Fig. 6**

The deck structure of the TLP.

**Fig. 7**

Foundation template units on barge at Invergordon awaiting final shipment to Hutton TLP site, for installation.
This form of construction fits the layout of the process and services by providing a number of discrete compartments for each of the functional areas. In general terms the high risk, low manned functions occupy the areas at the well bay end of the platform, and the low risk, high manned functions the other end. In order to minimize the danger of a manifold area explosion affecting processes or services, the well bay area is provided with an open side. This is achieved by making up some sections of bulkhead girder in truss form. In order to ensure that an explosion in the process area would not threaten the service areas the partitioning bulkhead girder webs must resist lateral pressure loading more readily than the deck plating. The heart of the deck, the control room, is not only placed as far away from the high risk areas as possible, but is also given full fire protection.

**Structural behaviour of the TLP deck**

The whole TLP design has to be a weight control exercise. The connections of the deck to the hull column tops must distribute the load throughout the column wall shell. To achieve this the deck is set down on inter-connected jacks at each column top at the time of deck to hull mating. This ensures that the dead load is equally and continuously distributed around the column shell. It also ensures that the deck starts life simply supported between two lines of columns. The result is efficient use of the deck structure since the deck plating will be acting with the girders most effectively at centre span, and least effectively at the ends. The final connection is made by welding infill steel between columns and deck to give full moment and torsional continuity. Thus the design of the deck is dominated by fatigue near to the column tops, whilst static loads are more important midspan.

Of particular difficulty in design is the requirement for service and process penetrations through bulkhead girders at positions chosen by equipment layout, as opposed to structural logic. Special penetration details must be developed. Since the size of even the largest door opening causes local, rather than global, stress redistribution in the girders, penetrations detailing concentrates on the redistribution of load around the 'ineffective' penetrations.

The bulkhead webs are stiffened primarily in the vertical direction and secondarily, intercostal stiffening is added horizontally, giving rectangular unstiffened panels. Around a penetration the margin stiffening would be increased to redistribute the loads throughout all the adjacent panels.

Additionally, intercostal stiffeners would be made fully continuous rather than having snipped flanges as elsewhere. The hole itself would have stiffeners and a bolted window frame to provide the gas-tight connection for the pipe or duct sleeve or cable box.

The deck sections do not have the same problems with penetrations. These are formed by shallow plate girders, typically 1.2m deep with 4m centre spacing, with stiffened plate panels spanning between them. Inside the deck these are conceived as pallets on which equipment could be placed, and loose flange connected with full access on four sides. The pallets can then be lifted into the appropriate slot in the girder matrix and connected using infill plates.

The overall concept of the deck is based on the maximum outfitting on a pallet before placement in the deck, and then the maximum outfitting of the deck at low level on dry land. Once outfitted, the deck can be loaded out on to a barge and then set down on the hull column tops.

**Deck structural detailing**

The stiffness of the deck in relation to the stiffness of the pontoons leads to certain fatigue design problems. The hull pontoons, lacking any plan bracing, tend to warp a little under a splitting wave. Since this wave is of considerably shorter wavelength than the static peak stress waves, it falls into the top end of the fatigue damage waves. The warping imposes a torque load on the columns which are effectively built in at deck level. Thus the torque is carried out by the lower level deck plating, the main deck pallets. As a result a considerable proportion of the deck must be detailed extremely carefully, with items such as pipe supports and vessels bases positioned exactly.

In general terms many details must be adopted universally to ensure that there is a consistent standard of finish as regards on-site inspection. Examples of this include extensive use of 'softening brackets' to reduce stress concentrations, a total ban on unnecessary stiffener details involving welds to flange edges (or within 6mm), and avoidance of the use of doubler plates (which are commonly used for pipe and service supports).

Another item which strongly influences structural detailing is related to the possibility of minor wave crest impacts on the underside of the deck for certain extreme wave events. This results in the requirement for extensive tripping supports to the main deck pallet girders. It also places severe constraints on exposed service ducts running between the deck and the hull column tops.

The concept of a TLP integrated deck requires the high standard of fabrication finish normally associated with box girder plated structures. However, the bridge-type top deck fatigue sensitive finish is required throughout in order to ensure maintenance-free service. The equivalent to closing down the outer road lanes for an extended repair on a bridge would be shutting down the whole platform - not only would this result in lost production with the possibility of a financial crisis for the oil company concerned, but in certain cases it could be extremely difficult to start up again.

**References**


The Central Electricity Workshops, Johannesburg

Barrie Williams
Architects: Rhodes-Harrison Fee & Bold

General description of project
The complex is a multi-function repair and workshop facility for the City Electrical Engineer of Johannesburg on a 16 ha development located on the site of the now redundant Robinson Deep mine just to the south of Johannesburg. It comprises maintenance workshops and service facilities for the generating and distribution branches, specialized test and laboratory facilities, vehicle workshops, comprehensive stores and an administration centre.

In addition to being structural engineers on the project, Ove Arup and Partners also provided civil and mechanical engineering skills and therefore played a strong role in the site planning and co-ordination of site services. Road access was critical as a 96-wheeled, 70m long, low loader, carrying 150 tonne transformers, had to be accommodated and this vehicle also had an influence on the siting of the various buildings and their relationship to one another. A further important planning and design parameter was the requirement that the buildings should have a useful service life of up to 100 years. Apart from making substantial provision for future extension, the design also had to anticipate technological change in municipal functions and plan for a possible metropolitan servicing centre.

Details of the adopted Master Site Plan are shown on Fig. 1.

Structural form
A general interpretation of the client’s requirements suggested workshop areas with large column-free spaces serviced by full crane facilities. An important consideration was the need to offer the client a flexible working space with regard to usage and machine layout and at the same time recognize the need to adapt the buildings if a change in use or technology required it. This, together with the clearances required to house crane hooks to assemble and transport the items of equipment along the length of buildings by leap frogging over the work space rather than having clear access corridors, suggested a single-storey high bay structure. The structural form also evolved from the need to make the best use of the excellent natural light we have on the Highveld, the need to reduce maintenance and cleaning to a minimum and with due regard to developing a consistent architectural expression for the buildings on the site.

Steelwork was a natural choice for the structures of the main buildings and a number of alternative schemes were produced in which various roof profiles and framing alternatives were investigated in relation to natural lighting levels.

The clear span (45m) chosen for building C5, for example, was the result of studies which indicated that fewer columns and less perimeter cladding and a single-span building and crane were more economical and functional than a double span with duplication of cranes and supporting structure. This solution would of course apply where long line processes have to be covered and where there is a high demand for crane use but is more appropriate to the jobbing as against the production type workshop.

The interrelationship between building frame, the crane and its support structures was investigated in detail to determine the benefits of the various alternatives and resulted in the decision to design the main building columns independently of crane columns where cranes were not required immediately. However, where appropriate, all foundations were designed and constructed with provision for future cranes.

The economics of the spacing of the main frames was evaluated for each building in relation to crane picker, lift and frame costs. It was determined that a spacing of 9m seemed the most practical and suited the monitor spacing on the larger workshops. A 9m x 9m planning grid was therefore established for the whole site.

The schemewrk also investigated the economics of various types of framing alternatives such as lattice trusses, solid web and castellated portal frames in conjunction with the prime consideration to retain a roof profile that would provide a uniform and sufficient level of natural lighting.

The results of these analyses clearly showed that lattice trusses were the most economic solution suited to the monitor profile with its beneficial natural lighting characteristics.

The remaining structural components have solid webs (girders) or solid box sections (columns) as, apart from the structural requirements, the appearance and maintenance qualities are more desirable and are expressed in the architecture.

Each of the main workshop areas is served by adjacent single-storey ancillary administration, training and amenity facilities. In general, these have been constructed with reinforced concrete frames.
Aspects of the new complex

Central Electricity Workshops, Johannesburg

Fig. 3-6

Carpenters' and Black Trades' Workshop
Transformer Test and Repair Building
and Heavy Equipment Store containing two

The following are details of the six main

section columns, rectangular hollow

steel-framed buildings of interest.

Structural details

Building E:
Building D1:
Building C5:
Building C3:

Areas of concern in the design were:

We were particularly concerned about

Structural design

Building B2:

For the various loading conditions the main

columns, and the cladding given sufficient

shoulder bolts, etc., at all connections to the

designed having suitably slotted holes and

testing agency to give us the quality

documents that the steelwork contractor

We therefore initiated a

Witwatersrand University, in which the

research project, undertaken by the

architects and principal agents:

Architects and principal agents:

Services. There is no doubt that value in use

industrial building progress well into the 21st

century. It is interesting to note that the

system (UK design) appeared to have very

minimum of 8mm. Even so the effects are

any 'tin canning' due to welding distortion.

services of an experienced plumber. All the

We therefore wrote into the contract

penetration web to flange welds of the crane

tubes were concerned) and in the full

longitudinal racking action of the side

concerned about the visual acceptability of

Welding quality

We solved this more by intuition than by

initial delays in shop drawing preparation

In all the main buildings therefore, the roof

metal skin rigidly fastened together and to

We were寻求ing the system we were seeking. The system has been

any guidelines on this for general distribution

We were looking for a system that would be appropriate. On occasion we were

any 'tin canning' due to welding distortion.

perhaps some technical notes

perhaps some technical notes

structural steel have been constructed and

To date two phases involving 2100 tonnes of

Minimum plate thickness in box columns

Welding quality

With the fully welded up section complete

We therefore initiated a

Preliminary structural design

structural solutions and materials will be of

architects

The steelwork structural engineer's

Construction problems

With the fully welded up section complete

penetration web to flange welds of the crane

tubes were concerned) and in the full

longitudinal racking action of the side

concerned about the visual acceptability of

Welding quality

We solved this more by intuition than by

initial delays in shop drawing preparation

In all the main buildings therefore, the roof

metal skin rigidly fastened together and to

We were seeking the system we were seeking. The system has been

any guidelines on this for general distribution

We were looking for a system that would be appropriate. On occasion we were

any 'tin canning' due to welding distortion.

perhaps some technical notes

perhaps some technical notes
Structural details

The following are details of the six main steel-framed buildings of interest.

Building C1: a General Test (air-conditioned, dust-free) and Heavy Equipment Store containing two 10 tonne EOT cranes.

Building C3: a Transformer Test and Repair Building having one 150 tonne crane with 25 tonne auxiliary hoist.

Building C5: a Heavy Machine Shop with 30 tonne x 45m span EOT crane.

Building D1: Vehicle Maintenance Workshop
Building E: City Treasurers' Stores
Building F: Carpenters' and Black Trades' Workshop

Schedule of building data

<table>
<thead>
<tr>
<th>Building</th>
<th>Height to eaves (m)</th>
<th>Span (m)</th>
<th>Area (m²)</th>
<th>Total steel mass tonne</th>
<th>Mass kg/m²</th>
<th>EOT cranes</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>15</td>
<td>18</td>
<td>3230</td>
<td>360</td>
<td>111</td>
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<tr>
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<td>20</td>
<td>18</td>
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<td>300</td>
<td>223.9</td>
<td>1 x 150/25T</td>
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<tr>
<td>C5</td>
<td>16</td>
<td>45</td>
<td>3936</td>
<td>337</td>
<td>85.6</td>
<td>1 x 30/5T</td>
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<tr>
<td>D1</td>
<td>9.5</td>
<td>+ 36</td>
<td>3960</td>
<td>185*</td>
<td>46.7</td>
<td>—</td>
</tr>
<tr>
<td>E</td>
<td>9.5</td>
<td>+ 18</td>
<td>5783</td>
<td>730*</td>
<td>68.3</td>
<td>Hoists</td>
</tr>
<tr>
<td>F</td>
<td>9.5</td>
<td>+ 36</td>
<td>5824</td>
<td>371*</td>
<td>63.7</td>
<td>1 x 15T</td>
</tr>
</tbody>
</table>

*Includes steel framing for extensive mezzanine floors

Structural design

For the various loading conditions the main frames were analyzed using STRESS. The full range of locally available structural sections were used including built-up box section columns, rectangular hollow sections for columns and tubes for in-trusses for economy, minimum light interference and dust collection, hot rolled universal sections for internal columns and beams and cold rolled sections for roof purlins.

Areas of concern in the design were:

- **Temperature effects on stiff cladding and rigid fasteners**
  - In anticipation of a requirement for improved internal environmental standards in future years, an insulated skin with a hard durable inner lining of ribbed profile was selected.
  - In all the main buildings therefore, the roof and side cladding consisted of a double metal skin rigidly fastened together and to the support framing using the Top Speed System (self-tapping screws). A desk study suggested that longitudinal temperature effects may lead to serious distortion of the cladding and/or failure of the fasteners. We therefore initiated a research project, undertaken by the Witwatersrand University, in which the longitudinal racking action of the side panels below the crane girder was simulated. This confirmed the likelihood of cladding failure at peripheral fasteners and proved our concern was justified. To avoid the problem the panel support framing was designed having suitably slotted holes and shoulder bolts, etc., at all connections to the columns, and the cladding given sufficient freedom by using flashings around the periphery of each panel.

- **Welding and quality assurance**
  - We were particularly concerned about achieving first class welds in several areas but especially in the butt welds in tension members (particularly where hollow tubes were concerned) and in the full penetration web to flange welds of the crane girders. We therefore wrote into the contract documents that the steelwork contractor would appoint an 'approved' independent testing agency to give us the quality assurance we required. In the second phase of the project this proved to be even more of a safeguard than we anticipated, due to the initial delays in shop drawing preparation and manufacture, much of the work had to be sub-let to four or five other fabricators, some of whose experience was less than desirable. Even coding of welders was often a problem.

  - There should be no doubt that in these circumstances a quality assurance programme is absolutely essential and can be relatively inexpensive. Perhaps some technical notes or guidelines on this for general distribution should be appropriate. On occasion we were also asked to assess proposed welding procedures — another area where our inspection staff could do with assistance.

- **Side sway damage to glass gables**
  - An interesting and highly successful feature of the south gables to the main workshops is the cabling of the gable walls covering the full length of the buildings. The main frame side sway due to crane surge (40mm) caused great concern among the patien glass manufacturers, all of whom unanimously said it cannot be done. On examination one locally available system (UK design) appeared to have very flexible glazing bar fixing details using single swivel bolts and sliding shoes. Reference to their UK head office received the same reply, i.e. that their system could not accommodate the movement anticipated. However, a simple test on a mock-up section in their works soon proved that they could more than accommodate the movement we were seeking. The system has been in operation now for 2 1/2 years without sign of damage.

- **Minimum plate thickness in box columns**
  - Once the important decision had been taken to use box section columns expressed externally, the final design (stiffening and inertia wise) required relatively thin plates in many areas but was concerned about the visual acceptability of any 'tin canning' due to welding distortion. We solved this more by intuition than by design by keeping plate thicknesses to a minimum of 3mm, so the effects are visible if you care to look for them but we have had no serious adverse comments.

Construction problems

- **Welding quality**
  - As mentioned previously, welding quality was initially a major headache with initial rejection rates of 50 to 60% of key welds, i.e. truss tie butt welds and crane girder web to flange welds. The gamma radiation and ultrasonic tests of the independent QA programme avoided all the potential emotion (for us that is) and ensured that we had the quality required. Without continuous inspection and sealed labels tagging with photographs, it proved difficult to ensure that only the correctly coded welders worked on our project.

  - The need for an experienced plumber
  - The welded up box section columns each contained a 150 g galvanized steel downpipe. To get these installed and pressure tested before closing up the columns, required the services of an experienced plumber. All the workshops tried at first to do this work themselves, at great expense, and with much delay. Perhaps we should have anticipated this.

- **Minor lack of fit**
  - Occasionally the main truss shoe connection to the face of the box column was slightly skew ± 2mm gaps on one side. This problem only becomes apparent once the structure has achieved its own roof level. We could not resolve it by extra welding or trimming. The solution we adopted was to ease off the bolts of the connection and to install a suitable combination of 1mm and 0.5mm shims before retightening the bolts; a somewhat makeshift solution but undoubtedly better than leaving the joint open.

- **Sagging purlins**
  - The sagging of the cold rolled purlins caused problems even though we anticipated it both by installing a sag rod system and by insisting, in the contract documents, that the sheeting contractor was responsible for maintaining the purlins in the correct position, although he aggravated the problem by placing all the sheets for one bay in one spot and by completing one slope of the roof at a time, i.e. the forces on the sag rod system were not balanced. The space required below the underside of the roof sheeting to permit installation of the insulation and inner metal skin meant that the sag rods were not placed in the ideal position, i.e. close to the purlin top flange. In the event the sheeting contractor solved the problem by installing a supplementary sag rod system of thin strips or ties connecting both top and bottom purlin flanges. The lower of these remains visible — a not altogether satisfactory solution. For our current solution is to use sag rods having a certain amount of bending stiffness e.g. light tubes, with a double-bolted vertical end plate. This seems to work well.

Conclusion

To date two phases involving 2100 tonnes of structural steel have been constructed and the third has started on site.

We believe the project has been an all-round success that will cater for the client's needs, give him flexibility and keep pace with industrial building progress well into the 21st century. It is interesting to note that the capital cost of the buildings compares extremely favourably with the analyzed costs of similar large scale complexes constructed for the South African Transport Services. There is no doubt that value in use arising from the selection of appropriate structural solutions and materials will be of great benefit to the client.

Credits

Architects and principal agents: Rhodes-Harrison Fee and Bold

Quantity surveyors: Roos and Roos Inc.
Multi-storey steel-framed buildings in South Africa
Cliff McMillan

Introduction
Over the last 10 years our practice in South Africa has had the opportunity to be involved in the design of six multi-storey building projects using steel-framed solutions. Traditionally, building construction in South Africa has been in reinforced concrete and this remains the main medium and normally shows significant cost savings over structural steelwork, except for the obvious applications such as light roofs and industrial buildings. However, with rising rates of escalation and high financing costs during construction, there has been pressure to speed up construction and ensure earlier occupation. In certain cases structural steel has shown benefits in meeting these objectives.

This paper describes some of the projects of this kind with which we have been involved.

Penmor Tower
This was the first example and was really the first multi-storey building in South Africa which incorporated the techniques commonly used in Europe and North America for the specific purpose of building economically and quickly in steel.

![Fig. 1 Penmor Tower. Architects: Nurcombe Summerley Ringrose & Todd](image)

![Fig. 2 Penmor Tower, typical floor, steel layout and details](image)

![Fig. 3 Dennehof Park, typical floor, steel layout and details](image)

It is an 18-storey office building in Johannesburg, providing about 17,000m² of office accommodation over a three-level parking basement. The layout of the typical floor steel frame is shown in Fig. 2.

Composite action was utilized for both main and secondary beams. The composite floor was formed by a ribbed metal deck specially rolled for the project. A fire test carried out by the South Africa Bureau of Standards confirmed that, with light reinforcement in the topping, the composite section qualified for a two-hour fire rating on a span of 3.05m without treatment of the underside.

The beams were sprayed with a vermiculite/gypsum cement composite to achieve a two-hour rating for secondary beams and three hours for main beams. The columns were encased in concrete to achieve composite action, and at the same time provide the required three-hour rating.

The central service core of the building was slipformed in reinforced concrete. A concrete core proved economical and structurally necessary in order to provide adequate stability and limit horizontal sway under wind loads. The slipformed solution had many construction advantages:

(a) The slipforming was extremely rapid, taking less than three months, including the time necessary to set up the slide, for construction from foundations to the top. Sliding could commence immediately the core foundations had been completed and processed simultaneously with the basement construction. Structural steel erection could then commence off a completed ground floor slab. The alternative of providing steel columns through the basement would have added cost with no saving in time. The basement and core construction were able to proceed during the
lead time required to procure and fabricate the steel.
(b) The slid core automatically incorporated the hoisting and access requirements for later construction of the floors, including the tower crane, personnel hoist and stairs. Metal pan stairs provided the essential early access to the floors.
(c) Because many of the services such as toilets, stairs, plumbing, ducts and lifts are located in the core, work in the finishing of these could proceed immediately, unrelated to the critical path for the general floor areas.
(d) Work on and in the lift motor room could start earlier.

Once steel erection had started at ground level, the structural frame was erected in three months (1½ floors per week) and the building was completed for handover within 12 months. A floor finishing cycle time of one week per floor was achieved for each of the follow-up trades.
The steel solution showed a time saving of five months over an equivalent concrete structure. Economic comparisons indicated that the additional capital cost of the steel frame was more than off-set by reduced interest and escalation during construction and the benefits of earlier revenue.

Dennehof Park
This development, which comprises three 4-storey office blocks of approximately 1000m² per floor, was recently completed in Sandton. The decision to use structural steelwork was taken to minimize construction time, assist in pegging escalation and meet a projected demand in the letting market. Analysis of the construction programme showed a three-month saving over a concrete solution. The target programme was met on site.

The composite floor is formed by flat-soffit trough-profile decking spanning up to 2.2m with mesh reinforcement, and provides a two-hour fire rating. The beams and columns are protected by plastering with Rocklite pearlite plaster. In this case stability is provided by steel bracing in certain bays. An interesting aspect of this project was the significant saving in piled foundation cost due to the saving of 40% in mass compared with a concrete structure.

Garden Plaza
This is a 12-storey steel-framed office building recently completed in Johannesburg. The typical floor layout is shown in Fig. 3. The two-level basement was constructed in reinforced concrete. The main core and two end stair wells were slipformed in reinforced concrete. A time saving of five months over a concrete solution was projected.

Bulk excavation and piling commenced on site at the beginning of 1981 and steel erection was complete in August. The building was substantially complete in April, 1982.

A sprayed mineral fibre fire protection application developed in Canada was applied by a local sub-contractor, under guidance from the Canadian supplier. This provided a significant improvement in application rate over what had been available locally. A purpose-designed ribbed metal deck roll by G. Vincent has been used. The depth of the deck is 90mm and the overall finished floor thickness is 160mm. With supplementary reinforcement this provides a two-hour fire rating.

Carlton Hotel Annexe
This building comprises nine hotel floors of 450m² each constructed using a structural steel frame on top of a reinforced concrete transition structure which forms the ground and first floors. The annexe is linked to the Carlton Hotel by a skybridge across the Kruis Street at first floor level and a service tunnel 15m below the street.

Piling commenced on site in March, 1981, and steel erection was completed mid-August. The building was opened in June, 1982.

The site is very small and congested and this contributed to the decision to use steel.

Stand 547, Parktown
This is a four-storey office building in Parktown.

Of the group of steel structures under discussion this is the only building which has exposed steelwork as an architectural feature. To achieve this the fire authorities required the building to have sprinklers internally and the window walls to be covered internally by a fire activated water curtain.

Design considerations
A few design-related aspects are dealt with below in relation to solutions which have proved efficient and economical locally.

Figs. 5-6
Garden Plaza. Architects: Louis Karol
Table 1: Structural costs of steel-framed buildings

<table>
<thead>
<tr>
<th></th>
<th>Penmor Tower</th>
<th>Dennehof Park</th>
<th>Garden Plaza</th>
</tr>
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<tbody>
<tr>
<td>Steel beams</td>
<td>R 44.90</td>
<td>R 22.06</td>
<td>R 43.30</td>
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<tr>
<td>Shear studs</td>
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<td>3.70</td>
<td>3.40</td>
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<td>Steel columns and bracing</td>
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<td>13.30</td>
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<td>Steel decking</td>
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<td>11.72</td>
<td>16.00</td>
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<tr>
<td>Concrete topping including mesh and power float finishing</td>
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<td>10.29</td>
<td>9.96</td>
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<tr>
<td>Temporary propping of deck</td>
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<td>1.72</td>
<td>—</td>
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<tr>
<td>Fire protection (Plastered or Sprayed Vermiculite)</td>
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<td>13.16</td>
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<tr>
<td></td>
<td>R 105.53</td>
<td>R 75.95</td>
<td>R 102.68</td>
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</table>

Composite action

The economic advantage of composite action for both primary and secondary beams is well-known. Normally the beams can be unpropped during construction and only act compositely under live load. Since CP177 requires elastic working load design as well as an ultimate load check with 0.9 x yield stress in the extreme fibres of the beam, there is often no significant penalty in this approach. Sometimes, however, it is found that temporary propping under dead load is desirable.

To achieve composite action efficiently, it is desirable to weld the shear studs through the metal deck. This enables the deck to be laid in long lengths over the beams without interference and saves considerable time in laying out the deck. Fixing the studs in the shop would require holes to be located appropriately in the decking before laying, or alternatively require shorter deck lengths with loss of continuity in structural action.

Composite metal deck

There are several benefits of using a metal deck compositely with the concrete topping. The deck can be laid rapidly immediately after steel erection to provide a safe working platform and serve as a permanent shutter for the concrete topping without propping. The correct choice of deck section can minimize dead load and reinforcement while providing adequate fire rating without the underside having to be sprayed. Power floated finishes eliminate the need for a screed over the topping.

The range of available deck sections is limited. Wider rib sections have now been developed to assist with shear transmission under composite action when the ribs run across the beam, as it is normal for most secondary beams on a typical floor. With the narrower ribs generally available locally the shear capacity of the studs is significantly reduced.

High tensile steel

The cost benefits of Grade 50 steel over Grade 43 both in tension and compression are obvious.

For columns, the advantages are significant. Up to about 10 floors, all columns can be in Grade 50. However, if a concrete core is used, the differential axial shortening between core and columns can be of the order of 25mm for 10 floors. Above 10 to 15 floors it becomes necessary to limit these deflections by limiting the stress in the columns, either by using Grade 43 steel or composite columns. Deflections of 'building out' the dead load differentials can also be used.

Grade 50 steel also has cost benefit for beams, but again deflections often govern, making mild steel more desirable. This is particularly so because of the desirability of limiting overall floor depth and using locally available sections of limited depth even for the longer spans to avoid the cost of built-up sections. In these cases Grade 50 steel used at reduced stresses can still show a cost advantage.

It should be noted that the use of Grade 50 steel can increase the required lead times because of problems of availability.

Fire protection

In South Africa two to four-hour rating will generally be required for columns and primary beams and a two-hour rating for secondary beams and the floor.

This can be achieved locally by the following methods:

(a) Concrete encasement minimum of 50 mm all round. This is only practicable feasible for columns, and even then it is a tedious operation. For a high-rise building it can provide the advantage of reducing cost through composite action.

(b) Spraying or plastering with vermiculite or pearlite compositions. Thicknesses required vary from 25mm for a two hour rating to 40mm for four hours.

(c) Spraying with mineral fibre/ cementitious compositions

(d) Fixing precast fire proof panels to the member. It is difficult to provide a fixing method with the required fire rating. Prefabricated systems of fire protection are in common use overseas but only one system has been satisfactorily tested locally.

Economics of steel buildings

The following general conclusions can be drawn about the economics of steel-framed compared with concrete buildings in the local market.

(a) For low and medium rise buildings, a steel structure costs about 50% more than a conventional concrete structure.

(b) Depending on the site conditions, this penalty can be significantly reduced by the saving in foundation costs due to the lighter steel structure.

(c) The overall penalty on total capital cost for the steel structure is between 5% and 10%. When allowance is made for the effects of the more rapid construction programme on the time-dependent costs such as escalation and finance charges during the construction, this penalty is virtually eliminated.

(d) It has been demonstrated that steel buildings can be completed more quickly than conventional concrete buildings in South Africa, and savings of between three and six months have been found for low rise projects. However, the success of the project is entirely dependent on sound management and on all the parties involved meeting their commitments. The financial risk if the project is delayed is greater for a steel structure.

(e) If a steel building is chosen it is essential that the total architectural, structural and services design should take advantage of the particular properties of steel as a construction medium. A steel building requires more disciplined decision-making by the design team in the early stages if the programme advantages are to be realized.

(f) The decision as to whether to use steel or concrete is a marginal one, which will be affected by the particular circumstances prevailing at the time. Most important is the assessed market advantage of having the building completed earlier and the financial benefits of earlier revenue and possibly meeting a particular projected market demand.

(g) The effect of using steel for these buildings has been to encourage contractors to find means of building faster in concrete. A current major project involving about 2000 m^2 is being constructed using flat slabs and table forms at an average rate of four to six days per floor. This has all the time advantages of steel with significantly less cost penalty.
The use of platform design in a tension leg and for the injection of water or gas into the clock, and hence accommodation units with continuous operation would include the primary separation of various processes associated with oil or gas production. On this working platform the safe working platform in an exposed marine environment. A fixed offshore oil or gas production field.

Requirement of the Building Regulations regarding concrete encasement for corrosion protection, etc., and a reluctance to accept lightweight fire protection, until recently, have led to all the steel-framed buildings erected to date in Hong Kong having their structure totally encased in concrete, and, although faster to erect than reinforced concrete, the steel-framed structures are generally 25 to 35% more expensive than reinforced concrete.

(2) SCOTLAND

The four Arup offices of the Scottish practice: Aberdeen, Dundee, Edinburgh and Glasgow have designed a wide variety of steel structures over the years. Typical of the range are three of our projects which have received Structural Steel Design Awards: The Royal Commonwealth Pool, Edinburgh, Almondell Footbridge and Kings Buildings Boilerhouse. The superstructure of the Royal Commonwealth Swimming Pool is of 280 tonnes of structural steelwork construction with a roof of 78m x 68m in two way spanning steelwork. In addition to the perimeter columns, the planning of the interior permitted support from a single off-centre internal column.

Fig. 1 Almondell Footbridge

Fig. 2 The Royal Commonwealth Pool.
Architects: Robert Matthew, Johnson-Marshall & Partners

Almondell Footbridge is a cable-stayed girder bridge of 30m span. The simple 19m high 'A' frame and vierendeel deck produce a dramatic but pleasing structure which suits the heavily wooded surroundings. At Kings Buildings the problem of constructing a new boilerhouse and chimney round the existing plant while maintaining a continuous supply of heat was neatly solved using steelwork. The cluster of eight new steel flues is structurally independent of the existing brick chimney round which they were built.

In recent years our experience in the use of steelwork seems to have fallen roughly into three broad categories:

(1) Civil engineering
(b) Low rise buildings
(c) Roof structures.
The greatest tonnage of structural steel is used in steel construction. This organization incorporates hydraulically operated hinged steel ramps. The firm are joint engineers with Messrs. Crouch and Hogg for the Kessock Bridge, an important cable-stayed road bridge with a main span of 240m over the Beauly Firth. We assisted by checking and approving the design of the box steel pylons and deck crossbeams in accordance with the Morrison Rules.

Further north, on the stormy shoreline of Scapa Flow in the Orkney Islands, we have currently under construction three roll-on roll-off South Island Ferry Terminals incorporating. They ranged up to 58m in height with the maximum size normally available is 200mm x 200mm.

Some examples of each type are given below:

(a) Civil engineering
Most of our work in this field has been in reinforced and prestressed concrete but we have been involved in several interesting projects with structural steel elements.

In conjunction with a consortium, Caledonian Platforms Structures, we developed a design for an offshore oil production platform, Firth 150. This included a steel deck structure comprising 4,000 tonnes of grade 50D tubular steel.

(b) Low rise buildings
Within this, the largest category, are industrial, school and recreation buildings, mainly single storey, where steel comes into its own with its lightweight and speedy erection unimpeded by the need for fire protection.

In spite of strong competition from the package dealers there remains a demand for the one-off building for which an off-the-shelf solution does not readily cater.

Typical of many school and college buildings we have designed is the games hall at Anderson High School, Lerwick. It is interesting to note that at this site in the Shetland Islands, the most northerly part of the UK, the design wind pressures are very high (twice those for the south of England) resulting in significantly heavier steelwork than would be required in southern parts of the mainland.

(c) Roof structures
While the basic structure of many of our buildings is of reinforced concrete or loadbearing masonry, steelwork still plays an important role in roof structures where lightweight is often an advantage and added fire protection is not required.

In the case of theatres, steelwork provides an ideal solution where services and access can be provided within the structural depth of the roof trusses.

Two theatres which we have recently completed are the Dundee Repertory Theatre and the Pitlochry Festival Theatre. The Dundee Theatre is a triumph of ingenuity in fitting a 450-seat auditorium with all the associated facilities into an extremely tight city site. Steelwork is used in the auditorium roof, and the flytower structure which rises to 18m.

At Pitlochry, a holiday town in the Highlands of Scotland, a slightly larger theatre (540-seat) has been built in rural surroundings, again using steelwork for the roof structures.

(3) AUSTRALIA
The use of structural steel within Australia is actively promoted by the Australian Institute of Steel Construction. This organization arranges seminars, courses and lectures, publishes journals, magazines and safe load tables and it sponsors research as well as maintaining library information.

The greatest tonnage of structural steel is used in heavy industrial works such as the petrochemical industry, energy and extractive industries and power generation. The use of structural steel in building construction is very widespread but usually on a small scale.

Typically the design of structural steel within the Australian offices of Ove Arup & Partners falls into the following categories:

(a) Industrial buildings
Industrial buildings for warehouse and factory facilities are typically portal frame buildings fabricated from universal beam sections. Clear eaves heights of 5 - 6m and spans of up to 30m are common. Wind loading is resisted by portal frame action and braced panels.

Prefinished metal deck roofing has largely replaced the cheaper asbestos roof sheeting. This is supported on cold rolled Zed purins. Walls may be metal cladding on G sections or of masonry construction.

(b) Roofs to low rise offices and commercial buildings
Numerically these are probably the most common use of structural steel in each of the Australian offices. An average job would contain less than 15 tonnes of steel.

Fire regulations generally permit buildings of up to three storeys to be of metal roof construction. Flat roofing is adopted on cold formed purin sections supported on universal beams. Mineral wool provides thermal insulation.

(c) Shopping centres
Roofs to shopping centres are similar to those described above but much larger in area. Trusses may be substituted for universal beams for the larger spans. Usually they are concealed above lightweight ceilings, unless they appear as a feature in, for example, a shopping mall. Bracing to side walls or infrequent internal masonry walls is required for stability.

(d) High rise office buildings
The economic use of structural steel in the floor systems of high rise office buildings is heavily influenced by the speed of construction. It can sometimes be demonstrated that the use of structural steel framing and composite metal decking can lead to a significantly faster typical floor construction cycle in high rise office buildings. The consequent saving in holding charges can more than offset the higher construction cost of a steel structure.

(e) Mining applications
Structural steel is commonly used for framing structures in the mining industry. It finds applications to heavy duty structures, platforms, conveyor supports and de-watering bunkers. A number of commissions have been carried out for an iron ore processing plant.

(f) One off structures
The Melbourne office have designed 4 steel floodlight towers for an athletics stadium. They ranged up to 85m in height with the access ladders and platforms contained within the tapered tubular section. They were designed for dynamic effects and fatigue conditions.

(4) SOUTH AFRICA
Hangar 8 – Jan Smuts Airport
The origins of this project date back to 1973 when South Africa Airways decided that a new maintenance hangar able to cater for the new generation of wide-bodied jets would be required by the early 80's. At that time, construction of a 150 x 100m hangar
capable of accommodating two Boeing 747s had just been completed. SAA, in conjunction with a multi-disciplinary technical team from the South African Transport Services, embarked on feasibility and concept design studies. It was decided that a hangar complex comprising two major hangars with a central annexe located between them would best facilitate aircraft movement and at the same time minimize fire risk. It was decided that each hangar should, if feasible, be a clear spanning structure of 150 x 100m so as to maximize space utilization.

Our favoured solution comprised a main portal spanning 150m over the doors, with three main trusses spanning 100m from the back wall to the portal. We proposed long-span crane bridges in the 37.5m bays between trusses to handle the servicing requirements.

1. Hangar structure
Each hangar section consists of steel-framed structure supported on three sides by braced walls and on the fourth side by a pinned base portal frame of 151.2m span. The roof is divided into four equal sections of 38.8 x 100m, each of which has a suspended system of runway beams for the support of cranes and general purpose maintenance platforms. Three main trusses span 100m from the west wall to the main portal at 37.8m centres.

The roof structure supports the moving loads from the cranes and service platforms as well as the necessary lighting, fire protection services and the comfort heating system.

The 150m span portal, with a mass of 262 tonnes, was assembled in sections on the ground, welded and then jacked up 25m into position on the top of the legs. The main trusses, steel sections on to trusses, supports and welded in situ. The entire structure remained supported on the trestles until it was completed and then a system of jacking it down to release the supports was implemented.

The structure is stabilized through bracing in all three walls which interact with the portal stiffness to provide stability. In-plane bracing is provided in the roof.

Roof drainage is provided along valleys, midway between the main trusses, the ridge being over the main trusses and at the ends.

A feature of the structure is the neatness of detailing of the main members and their connections. The maximum force in the chord of the main portal is of the order of 25,000 kN. The major members are H or channel sections built up from up to 38mm plate. The result is a clean appearance on both external faces of the steel members, with minimum interruption by gussets.

Hangar doors
Each hangar has four door sections mounted on a twin track system to close them. The track system is continuous from one end of the complex to the other so that in the event of a hangar fire that needs to be opened, all four door sections are moved to the frontage of the other hangar section. Each door section comprises two leaves with a fixed link between them. Each leaf has its own drive mechanism.

(5) IRELAND
Ove Arup & Partners in Ireland, founded in 1946 in Dublin now operates also in Cork, Waterford and Limerick. Total staff numbers are about 140. The work is located in Ireland with a small amount in the UK, Middle East and the Third World. While the bulk of the projects are buildings in some form, an increase in civil works content has recently developed in the practice.

Usually steelwork is let as a nominated sub-contract. We find that this has the advantage of selected tendering as well as a later need for tender information. However, there can be difficulties in precise definition of work content when variations arise. On the other hand, hand measurement in the Bill of Quantities, in spite of the apparent precision of the information, has not always worked well and selection of tenderers is less controllable.

There has been a chronic problem in over capacity in the steelwork fabrication industry in Ireland. There are several major firms who have the resources and expertise to tackle any job which has yet been built in the country. There is also a wide selection of small and medium sized fabricators so that for smaller jobs there is an embarrassment of choice.

Technical standards can be variable but excellent results are possible provided the job documentation and site control are good, and the tenders are based on selection. Likewise, management can be uneven and tends to be skimmed when resources are stretched, either technically or financially.

The potential is certainly there to achieve very high standards if sufficient input is provided and the work well supervised.

Design is normally to BS 449 and the BCSA Handbooks are universally available. The normal source of materials is the UK. There is a degree of continental influence and during times of BSC strikes this forms the alternative source of supply. There is however a price differential which has inhibited more general use of continental sections.

There is limited capacity for economic fabrication of plate girders and castellated beams. These would normally be imported ready-made. Stock holders keep some supplies and since may jobs are modestly sized the necessity to buy there tends to increase job costs.

Certainly where anything other than Grade 43 steel is envisaged the lead in period must allow for mills ordering to be feasible.

The standard facilities available to the larger firms provide all the necessary equipment for fabrication, for almost all situations. Automated cutting and drilling has recently become available but is still unusual. Welding techniques nowadays almost universally employ semi-automatic CO₂ systems, at least in the shop. Automatic welding is rarely done, partly on account of the scale of work involved.

Most fabricators do not employ welding technologists and this service combined with NDT interpretation is provided by one or two freelance specialists. By and large this arrangement is satisfactory but difficulties can arise where the fabricator does not realize when he is out of his depth. We tend to use radiography and ultrasonics, partly as psychological tools to convey the importance of workmanship as much as the actual test. Generally this works quite well.

Building types

Advance factories
Most advance factories are sponsored by the Industrial Development Authority who share the work among many design teams. At this level concrete competes strongly with steel and is probably a bit cheaper currently. Systems of two-way trusses (built in one direction) have been used but more recently one-way members in alternating directions in adjacent bays have been found to be quite successful, almost equally effective, and cheaper.

Purpose designed factories
A large diversity of structural form has been utilized to fit specific briefs. They range from portal frames to Nodus, depending on the brief, the client’s cost consciousness and the architect’s powers of persuasion.

Office buildings
Since office buildings really got going about the mid ‘60s it has been quite unusual to find steel frames. Their use is usually confined to penthouse level.

Sports halls
These invariably use steel roofs and often steel columns, as the scale tends to be more modest than elsewhere and comparatively little scope for the spectacular has been available.

Descriptions of some individual buildings

Bailey’s Irish Cream Factory
This is an 18,000 m² building which utilizes, for the first time in Ireland, the ‘Nodus’ system on a large scale. CHS concrete-filled columns at 19.6m each way support the space frames which number 43 in all. Each was assembled at ground level and lifted into position. A ‘Mero’ truss portal spanning 19m is used to support the entrance canopy.

Getty Corporation Factory
This is a modest 2,000 m² factory notable for its unusual use of concrete block vertical structure. However the steel is also interesting, using 688 x 152 mm castellated universal beams (CUB) as main beams on a 19m, 3 span grid and lighter CUB of the same dimensions as secondary beams in altering orientation. Continuity of main beam over the circular hollow section columns, using 8.8mm bolts, permitted the use of dimensionally identical main and secondary beams which nicely unifies the design and facilities service runs.

Factory walling
We have been involved in the development of a steel factory walling system to meet a particular brief. This uses 1.2mm thick galvanized metal in interlocking trough form, 500mm wide x 100mm deep, spanning 7m
vertically between roof and floor. Horizontal ribbed metal cladding to stabilize the open sides of the trough enabled us to provide a 150mm overall thick frameless walling system. This system was designed, tested and successfully used in a pilot scheme where no commercially available system could be found.

Central Bank of Ireland head offices
This design was conceived as a major landmark in Dublin city centre and is unique in its structural design in Ireland. Measuring 45 x 30m, its seven floors are suspended from the roof and supported by two concrete service towers.

All floors are formed of structural steel trusses with a thin concrete slab. Each one is suspended by 12 pairs of Macalloy bars on the perimeter, directly to roof level. A system of roof trusses enclosing the plantroom is the primary steel structure carrying all the floor loads into the concrete cores. Floors were fully assembled at ground level and jacked into position, using the British Lift Slab heavy lifting system located on the roof.

Papal cross and canopies:
Phoenix Park, Dublin
An inception to completion period of two months was achieved for the erection of a 35m high steel memorial cross for the 1979 Papal visit to Dublin. A fabrication using six universal beam sections, each 600m deep, to form both stem and arms was devised, designed, fabricated, painted and erected with very little time to spare.

Fig. 7
Papal Cross being hoisted into position – Phoenix Park, Dublin

Fig. 8
Papal Cross and canopies as seen during the Papal visit to Dublin

Fig. 6 Central Bank of Ireland head office under construction. Architects: Stephenson, Gibney & Associates
Composite frame and metal deck construction
Ian MacKenzie

Introduction
This paper covers the use of steel decks and steel frames acting compositely with concrete slabs in buildings. This is a form of construction which in some circumstances has proved economical in high rise office buildings in Australia.

There is now a considerable accumulation of knowledge about the behaviour of composite members. The codes of practice permit composite frames to be designed as equivalent steel frame structures by the so-called 'simple design method'. This was first introduced into BS449 in 1932. A major updating in 1949 led to lighter beam sections but it also resulted in heavier columns.

Professor Baker turned his attention to the ultimate failure of steel frames and his work in the elastic plastic range helped to explain the redistribution of stresses occurring in the various components in this range. The British Standard Code of Practice, CP 117: Part 1 published in 1965, was based on simple plastic theory using idealized stress-strain relationships for both steel and concrete. For the latter an equivalent yield stress of 0.85 Fc was adopted. Using these criteria there has been good agreement with the ultimate loads for a range of beams but stresses and deformations have been less satisfactory.

Research has continued into the effect of slip at the shear connection and more accurate moment-rotation curves have been derived. A more realistic assessment of the influence of residual stresses in the steel component has also been made. Typical moment-curvature relationships are shown in Fig. 1. Beams of the type represented by Curve 1 reach a peak and then fall away rapidly leading to a sudden failure. Beams with the type of deformation represented by Curve 2 are subject to some strain hardening and fail in a gradual or ductile manner which is more acceptable to designers.

It is with a view to ensuring that this type of failure will occur that the Australian Standard 2327: Part 1 - 1980, 'Simply supported beams', contains a requirement related to ductility. This Australian code has been based on CP 117 but it is expanded to include sections on serviceability and it also covers the use of profiled steel sheeting in composite frames. It specifically excludes dynamically loaded structures and bridges.

Future issues of the code will cover continuous beams, slabs and columns.

General design provisions of AS 2327
The slab which is acting compositely with the steel frame may be either:
(a) Cast in situ on removable formwork
(b) Cast in situ on profiled steel sheeting (composite decking)
(c) Cast in situ concrete on precast formwork which is left in place
(d) Precast for its full depth.

There are design requirements to cover flexural strength, serviceability, shear strength connection and transfer of shear strength. Rules are given for effective width and effective thickness of slabs.

Stresses associated with slab flexure are not required to be added to those due to composite action except where prestressing forces are applied in the direction of the longitudinal beams.

Flexural strength
Design can be either by the load-factor (ultimate strength) method or working stress (elastic design) method.

Load factor method
The ultimate moment of resistance is calculated on the basis that:
(a) The portion of the steel beam in tension is stressed uniformly to the yield stress Fy.
(b) The portion of the steel beam in compression is also stressed uniformly to yield stress Fy.
(c) The compressive stress is assumed to be a uniform stress block of 0.85 Fc
(d) The concrete has no tensile strength.

Ultimate moment of resistance M' for neutral axis within depth of concrete

\[
M' = \frac{8 F_y A_s}{2} \left[ \frac{d \gamma + D - d_n}{2} \right]
\]

where \(d_n = \frac{F_y A_s}{0.85 F_{cc}} \leq D\)

\(\theta = 0.95\)

(Refer to Fig. 2).

Ultimate moment of resistance for compressive stress in steel beam
This occurs when \(F_y A_s > 0.85 F_{cc} b D\)

(Refer to Fig. 3)

\[H_e = F_y A_s\]

\[H_{cc} = 0.85 F_{cc} b D\]

\[H_{sc} = F_y A_c \text{ where } A_c = \text{Area of steel compression}\]

For equilibrium:

\[H_e = H_{cc} + 2 H_{sc}\]

\[F_y A_s = 0.85 F_{cc} b D + 2 F_y A_c\]

Hence \(A_c = 0.5 \left[ \frac{A_s - 0.85 F_{cc} b D}{F_y} \right]\)

\(d_n\) can then be found from properties of steel sections. The code then gives a formula for determining the ultimate moment of resistance when the axis N-N is located in the flange or in the web of the beam.

AS 2327 contains clauses which modify the assumed stress block when the depth of stress block would theoretically be located below the top of the rib of the decking.
Ductility
To ensure a ductile response of the beam there is a requirement that the depth of the neutral axis of a T beam shall be not less than 16% of the overall depth of the section.

Shear connection
The function of the shear connector is to transmit horizontal shear between the concrete slab and the steel beam and to ensure that there is no physical separation. Regardless of the method used to determine the flexural strength, shear connectors are designed by the load factor method. Characteristic strengths of both stud and channel connectors are given for various concrete strengths. Connectors are provided to resist the whole of the compressive force and are usually distributed uniformly between the two critical points of zero and maximum moments. Where beams support heavy concentrated loads, distribution should be according to the shear force diagram.

Transfer of longitudinal shear in concrete
It is necessary to check potential shear failure planes to ensure that the horizontal force can be transmitted to the concrete. The code requires minimum transverse reinforcement to cross the shear planes in the bottom of the slab.

Composite decking
An Australian code covering composite slabs is yet to be prepared. At present only two types of decking are available for use in composite deck construction. Both deckings are galvanized and available in two thicknesses. Design manuals enable selection of slab thickness and negative mesh reinforcement for simple loading situations. Full design procedures require:
(a) Check on panel stress during construction loading
(b) Check on panel deflection under construction loading
(c) Shear under construction loading
(d) Total panel stress due to slab dead load and superimposed loads
(e) Shear and bond on the composite slab
(f) Deflection of the composite slab
(g) Concrete stress in the composite slab.

Permissible stresses and deflections are based on extensive load testing and are nominated in the manuals.

Fireproofing
Based on full scale fire tests, design criteria have been determined under which, for given slab thicknesses, the provision of fire emergency reinforcement will enable fire resistance ratings of up to three hours to be achieved. This means that only the supporting steel beam requires spraying or other measures to achieve a fire rating. The provision of this reinforcement has been established for given spans and superimposed loads, and is usually a considerably more economical way of achieving the necessary fire rating than spraying the metal deck soffit.

Practical considerations
Shear studs welded through the decking are the usual method of shear connection with the concrete slab. It is important to ensure that there is an adequate power source and that the interface between the decking and the steel beam is clean, dry and free from loose rust or scale.

Consideration should be given to applying the studs in the field rather than the fabricator using them. It is preferable to use the risk of riggers erecting the steel tripping on the studs.

Stud shear connectors adequately fix metal decking to support beams. Alternatively puddle welds can be used.

Propping of the decking may be necessary to limit deflections during concreting. Alternately, proping can be avoided by locating the support beams at appropriately close spacings. If there is no propping, deflections can be of sufficient magnitude for the concrete actually placed to be significantly more than the deflection calculated from the theoretical thickness.

Soffit fixings
For soffit fixings to hang ceilings, ducts, pipes, etc., the metal decking must be drilled. Flash welding pins are not regarded as satisfactory. In the case of Bondek special nuts can be fitted into the flutes to support lightweight ceilings.

Precedent and intuition in design
John Roberts

Overture
No seminar is complete without an historical review of the subject in question. It allows us to compare past achievements with present efforts. Derek Sugden’s talk was as timely as it was appropriate: He led his audience, through the mysteries of design achievement from early China to just short of the present day. The achievements of present day ‘luminaires’ — his term for the acknowledged leaders of contemporary design — were left for another day — a pity.

Of course Derek cheated. The talk was not, billed, about the skeletal frame. Instead he talked about the design process. Few realise, however, that the audience was caught on the horns of the theme dilemma. To what extent is (or ought) design to be constrained by our own expectations — ‘precedent’; and to what extent is it (or should it be) a function of our own immediate emotion ‘intuition’?

Precedent in design arises from a number of sources — legislation, materials, manufacturing constraints, form and appearance — and our ability to resolve the dilemma. The danger in following a precedent is when the context changes so that the solution is no longer appropriate.

Extrapolation requires care. Precedent relies Society’s concern about radicalism but can lead to anonymity and irrelevance. Derek quoted Blake’s ‘Auguries of Innocence’, ‘we are led to believe in a lie when we see with, not through the eye.’

Intuition on the other hand is our ability to find new solutions to novel problems. It allows us to acquire makeshift solutions and solve the problem on its own merits.

Voltaire in the Temple de Goutt summed up his appreciation of intuitive design in this way: ‘Simple was it noble architecture. Each ornament and detail, all with its position, seemed to have been placed there of necessity.’

Throughout history intuitive solutions to design problems have, at least initially, received a hostile reception, mostly, one suspects, because they represented a break from tradition. By repeated experience, however, an intuitive solution becomes accepted and the general public begins to associate the intuitive solution with that particular problem. This happy balance is upset when some of the parameters change either in the problem or its solution. The precedents remain — an early steam engine is given a modern jacket to represent a distinctive European Order. Only later and possibly in partial recognition of this estranged situation is an intuitive solution developed — result outcry. But the general solution for chimneys continues to be satisfactory. In the case of Bondek special nuts can be fitted into the flutes to support lightweight ceilings.
Early examples of precedent and intuition

Fig. 1
temple gateway near Canton - centuries of precedent embodied in an 8th century building code

Fig. 2
An-Chi Bridge across Chiao Shui River - engineer's intuition of the same period

Bridge design

Fig. 3
Runnymede Bridge by Lutyens 1935 - Precedents
(Photo: Ove Arup & Partners)

Fig. 4
Runnymede Bridge by Arups 1979 - Precedents with a certain style
(Photo: Ove Arup & Partners)

Fig. 5
Longdon-on-Tern aqueduct by Telford 1795 - an intuitive trial of cast iron for a greater project

Fig. 6
Pont-Cysylte aqueduct by Telford 1795-1803 - Hailed by Sir Walter Scott as the greatest work of art he had ever seen
(Photo: Architectural Association)
intuition and the first iron frames:
The first iron frame buildings in England had no real architectural precedents except in the external walls and the entasis of the cast iron columns. Early examples are at Milford 1792-3, Shrewsbury 1796-7, Salford 1899-01, Leeds 1802-3 and Belper 1833-4. The use of iron spread to the external walls as in Arkwright's Masson Mill, Cranford, 1783, the cast iron Doric columns in Hartley's Liverpool Dock, 1845, and the boat store at Sheerness, 1858-60. In each case, however, elements of the iron construction were constrained by precedents developed rationa­ly in other materials.

The metamorphosis of the column:
The trabeated form of architecture (column and lintel) was derived for masonry construction by Greek intuition. The practice of shaping the columns to suit the perspective from which buildings were most frequently viewed and the decoration to the column capitals and shafts became embodied in strict 'orders' which were written down and illustrated. Thanks to Vitruvius, the rules were preserved so that by the time Europe had emerged into the Renaissance, there was an established precedent for the appearance of columns. When masonry was replaced by a new material, cast iron, designers found it hard to break with convention. Hence Jesse Hartley's cast iron columns at Liverpool dock, already mentioned, although handsome, were stone look-alikes. And in the first iron-framed buildings, the cast-iron columns remain constrained by precedent.

The reason not to break convention is unclear. Possibly it was the fact that a column is a plain affair as defined by function. The link that allowed the precedent to be followed was the technology of timber - used as patterns by both masons and iron founders. Not knowing how to deal with columnar elements seems to have persisted as a problem, for we find many otherwise functional objects having classical embellishments. Even Robert Stephenson succumbed. In 1839, one of the locomotives he designed, The Northumbrian, was put into service resplendent with a Corinthian fluted chimney.

The designer's dilemma in defining an intuitive solution for iron was overcome elegantly. By 1874-75, Jabrouste was able to define an artificial form for the cast iron column in the second of his library buildings - the National Library, Gone is the ornamental foliage, only the size of the columns merge with the domes they support and the form expresses the processes of rolling and forging involved in their manufacture. Only the shaft remains fluted as a throw-back to earlier precedent.

As we shall see later the problem of the column was more generally overcome because of two factors - firstly the need for fire protection; secondly and more important the fact that concrete and steel had become widely available by the 1950s and these did not permit the easy copying of Greek order.

Railways and the ecclesiologists hit back

The rapid development of the railway from 1825 onwards gave engineers rather than architects the chance to produce iron structures for an entirely novel set of requirements, and this they did. Lime Street, Liverpool 1836, Euston, the Trijunct at Derby, Temple Meads and York are all early examples. This new-found opportunity to derive new and appropriate solutions in new materials and for new uses did not go unchallenged. How dare classical style be ignored. Pugin in his 'Apology for the Revival of Pointed or Christian Architecture' felt that the Tudor embellishments to Brunei's Temple Meads Station, despite their theoretical relevance to the hammer beam roof of the shed, give the edifice the air of a mere caricature; 'mock castellated work, shields without bearings, ugly moldings, nonmeaning projections and all sorts of unaccountable breaks, make up a design at once costly and offensive and full of pretensions'. Clearly heresy must be stamped out. Thus did the Christian Functionalist give his verdict on what was certainly 'engineers' architecture'.

The skyscraper is born

Notwithstanding such attempted divine interventions the use of metal frames progressed, if only slowly. By the middle of the 18th century, iron had been used in commercial buildings on both sides of the Atlantic. Jamaica St., Glasgow and Oriel Chambers in Liverpool (whence the oriel window) and buildings by Bogardus and the Badger in New York (including the Otis Building containing the first public lift).

Bogardus' buildings caught on fast as they were available ex-stock in kit form and it took the concerted efforts of the New York Architects to have fire codes changed effectively to put an end to his challenge to their province.

About this time the initiative in building frame design crossed the Atlantic, germinated, then took a leap forward with the manufacture of steel sections which, unlike their iron antecedents, were of higher strength particularly in tension. Adler and Sullivan are usually credited with designing the buildings which are regarded as the first modern steel frames - the Casson Building and the Pirie and Scott Building, both in Chicago, and The Guaranty Building, Buffalo.

Engineers' architecture and more conflict with precedent:

Meanwhile engineers were finding other novel applications for which no architectural precedent existed.

Between 1795 and 1803 Telford constructed the Pont Cysyllte aqueduct carrying the Ellesmere canal over the River Dee near Llangollen. The form of the iron trough spanning the masonry columns is simplicity itself, echoing only the loads being resisted, and has no formal memories. Sir Walter Scott called it: 'The greatest work of art he had ever seen' - praise indeed. Incidentally the work of art was only produced after Telford had successfully tried out the design on much more modest scale at Longdon-on-Tern in 1795.

Then came the Forth Bridge designed by Messrs. Fowler and Baker, built between 1883 and 1889 and representing a scale change in the possibilities of steelwork construction. It too attracted influential critics. William Morris said 'There never would be an architecture of iron, every improvement in machinery being higher and higher until they reach the supremest specimen of all ugliness - the Forth Bridge'.

The skeletal frame

Fig. 7
Albert Dock buildings in Liverpool by Hartley 1848 - the Doric Columns are cast iron

Fig. 8
Sheerness Boat Store by Col. Green 1858-60 - the frame emerges

Fig. 9
Guaranty Building, Buffalo, USA 1894-95, façade

Fig. 10
Shell Mex House 1933 - Portland Stone quarries hung on steel frames as architecture is sacrificed on the altar of structural engineering

(Photos: Architectural Association)
The life cycle of the intuitive process: illustrated by the work of Labrouste

Fig. 11
St. Geneviève Library 1843 – banded barrel vaults in stone replaced by banded barrel vaults in cast iron (Photo: Architectural Association)

Fig. 12
Bibliothèque Nationale 1862-8 – the arches were removed to support the domes give ornamentation from the geometry. The fluted cast iron columns remain inhibited by the tradition of stone. (Photo: Rab Bennetts)

Misplaced precedent

Fig. 13
Northumbrian 1830 embellished with Stephenson’s Corinthian fluting to the chimney

Fig. 14
Emperor Hirohito of Japan (left) with his chief of staff inspecting sound detectors for locating enemy aircraft, Osaka 1934

Lunokhod 1, which landed on the moon in 1970 looks like a traditional samovar. The rise of the corporate client has also brought with it new problems for the designer. The principal of these is his inability to deal with the intricacy of the design problem. He now finds himself pari passu with the executive in a niche in his client’s hierarchy. The very essence of design is the need to transcend artificially created boundaries. The threat of the corporate client is the precedence of his own organization. Unless great care is taken, an intuitive design approach can cause ructions. We have not yet learned how best to relate to the large corporate organization and yet still retain our ability to innovate.

Coda 1 and 2
Derek summed up his talk in two thoughts:

1. Most architects work and most of our work is concerned with buildings which lie in scale between the house and the tower or bridge where the structure is dominant. At this in-between scale, whether structure is expressed or not is a function of the architect’s personal morality of form. What we find today is a search for originality where many of our luminaires cast light only on themselves, leaving principles in the dark. Lacking originality, there is an obedience to, but not a listening to the precedent.

Remember Blake: ‘we are led to believe in a lie when we see with, not through, the eye’. The engineer for his part with his now very powerful analytical tools – the new altars of our profession – is probably sacrificing his intuition at an alarming pace, encouraged by the academics with which he spends the most formative years of his life.

The characteristics sought today are those of performance, heat, light, sound, durability, adaptability and so on. As we seek to specify them to define our places, we remove ourselves from the reality of the materials themselves.

As we remove ourselves from the materials, we remove ourselves from the forms they take and thence from the development of any structural intuition.

2. In the construction industry new materials and their fabricated elements replace or substitute for old materials. The form of the new copies the form of the old. Arch bridges in stone are replaced by arch bridges in cast iron; cast iron baths by plastic baths in exactly the same form.

In the St. Geneviève Library, 1843, Labrouste replaced banded barrel vaults in stone by banded barrel vaults in cast iron. The details of the arches clearly indicate they were ‘old’ and the joint differentiate for us what was made off-site from that connected on-site.

At the time Labrouste designed the National Library in 1862-8, cheap puddled wrought iron was available. Here the arches which combine to support and define the domes have a completely different expression. The parts are clearly rolled or hammered and riveted together. Process and engineering are evident in the geometry of the parts and no cast foliage as in the first library. The column remains cast iron fluted and with an Ionic Capital – or one could say it remains inhibited, like all 19th century columns, by the tradition of stone.

Reprise

The theme dilemma – precedent v. intuition – remains unresolved. But we are more self-aware. One wonders whether such self- awareness is inhibiting. The next time you are detailing a concrete column to resemble the steel column the architect really wanted if only the fire regulations permitted, will you be daunted? And will you be able to convince your peers of the wisdom of precedent and the need for intuition?
Industrial building: Report from Birmingham

Robert Greenwood
Peter Handley

Introduction
Birmingham Office has been involved in a large number of single storey industrial buildings. These have ranged from small extensions of 200m² for family-owned Black Country businesses to a 20,000m² warehouse at Coventry.

Planning
Our experience has emphasized the need for good planning and realistic prices. Industrialists in the West Midlands want two basic features for their buildings - a strong floor and a good roof that won't leak.

Good planning means designing a building for its immediate use whilst ensuring that it will hold its value in the market place. Planning also means carefully balancing the minimum cost of construction with the future waste of money if the facility fails to be adaptable or flexible.

Any adaptable building will accept re-planning of storage or re-organizing of production. A flexible building will cope with expansion or alteration.

The 20,000m² warehouse we designed at Coventry Trading Estate (Job No. 6670) five years ago has already been used by Talbot as a 'knock down' production line for exporting cars to Iran. It is now used as their national distribution centre. It can, if needed in the future be subdivided into a number of smaller units.

Adequate provision must always be made for service loads and some spare capacity is always desirable - but there is rarely any need to provide for production loads. Equipment and material handling can always be supported on a separate structure. This was done at Coventry and also at Aldridge when a high level barrel-filling operation had to be included.

Layout
Structural costs increase with the span chosen. For light industry and small product storage, 15 to 30m stanchion spacing is suitable. Medium production and general warehousing usually require spans of 24 to 36m. Heavy industries and high/dense warehousing often need spans up to 50m.

Selected examples
Aspects of design and detailing can be highlighted by examples of Birmingham designed projects.

Alexander Duckham and Co. Ltd. at Aldridge (Job No. 6619) required a new warehouse and filling shop of about 6,000m² with a clear height of 10m and with a flat roof. We were appointed as prime agents with total design responsibility for all aspects except the plant. The building has a structural grid of 15m x 12m and we designed Warren braced welded angle trusses at 6m centres with Multibeams as purlins. Universal column stanchions were used internally but as a 4-hour fire rating was required at walls, the perimeter columns were of precast concrete. The supports for the barrel loading area served by conveyors were of 40 beams of 15m span and here it was most economical to use castellated universal beams. The building was fully braced with angles at roof level but the side bracing was 42mm diameter rod in Grade 50 steel with high strength friccion grip bolt connections. The entrance canopy edge beam is a 30m span box girder.

The former Armstrong Whitworth Aircraft Factory at Coventry is now Coventry Trading Estate. This is a very long building with a width of 110m, having two spans of 30m and one of 40m, and the roof is of steel angle riveted construction. As the result of a major fire near the centre of the building, we were appointed for two structural contracts. Rather than rebuild on the same area the client decided to have new gable ends to the parts which were saved from the fire (Job 6591) and then provide a new separate building (Job 6670). For the two gable ends it was necessary to repair some of the main valley girders and the best solution was provided by the site welding of new angle members. To stabilize the gable ends, props of rectangular hollow sections were used. The replacement building was 260m long x 75m wide with internal stanchions on a 20mx15m grid. Although we normally find that welded steel trusses are the most economical for this size of grid, we used plastically designed portal frames and valley beams and the bracings were circular hollow sections. The portal frames will allow easy construction of sub-division walls for any future re-organization.

At Icknield Port Road, Birmingham (Job 6669), a number of 18m span steeply pitched
trusses had to be replaced with new monopitch trusses. Parallel lattice girders were used with structural T booms to which angle bracings could readily be welded on alternate sides.

Another form of truss construction we use has universal column or beam booms with tubular bracings. The example is a 30m span with warren bracing.

This same principle has been used for the boiler house at Telford District General Hospital (Job 4623) of 16m span and 9m high. The initial design for a portal frame was abandoned when 10 tonne coal hoppers had to be carried from the roof and a full height walkway above the hoppers. The welded steel truss met these requirements with the universal columns/universal beams booms ideal for carrying local bending and the tubular bracings efficient as long struts. To allow for transportation the truss is to be site bolted with Grade 8.8 bolts.

We are finding that Grade 8.8 bolts have largely replaced HSFG on most jobs as they are easier to use on site. Particularly with overseas jobs we avoid having two bolts of the same size but different grades. Some trusses for Africa (Job 8399) were transported in containers as individual angle members and completely site bolted. The M20 and M24 bolts were grade 8.8 with the M16 bolts in grade 4.6 to avoid any possible confusion on site and we try to follow this procedure in the United Kingdom.

For the 90m x 60m factory for Adamswear at Nuneaton (Job 9195) our client instructed us to prepare a performance specification so that subcontractors could use either portal frames or trusses. The grid for the 60m width is two spans of 30m with a 6m spacing down the length of the building. The truss design proved the most economical.

Trusses were also used for a 20m span tank production shop for Joseph Ash and Sons (Job 8960) and also for an awkward redevelopment of an existing site for Samuel Heath and Sons (Job 8567) which required some operational areas to be kept in production while the new building was completed around them.

Figs. 4-5
Factory for Adamswear at Nuneaton
Fig. 6
Joseph Ash and Sons tank production shop
Architects: for both projects: Harper Fairley Partnership

Fig. 7 Samuel Heath and Sons, redevelopment of existing site.
Architects: Kinton Design Group/ Bateman Associates

Schemes on the drawing board at this moment include a further project at Coventry of a 30m span where a welded angle truss will be used and a 64m x 45m building where portal frames of 15m span have proved most suitable.

Conclusions
To summarize, our recent experience has shown:

(1) Buildings must be adaptable for future planning and flexible for future extension.
(2) Roofs must be strong enough to carry adequate services.
(3) Truss or Portal? We will usually examine both schemes but trusses more often offer the best solution.
(4) Bolted or welded trusses? Shop-welded with site-bolted joints for almost all projects.
(5) Grade 43 or Grade 50 steel? Most schemes have used Grade 43 only but we must bear in mind that the same mill delivery of four to six weeks is offered. We should therefore look for out places where Grade 50 can be used to advantage such as heavy loads on 'short' spans and for certain bracings.
(6) Grade 46 or Grades 8.8 bolts? Almost all M16 bolts are Grade 4.6 but M20 and M24 in Grade 8.8 are frequently used for major structural connections.
(7) Bracing members angle, rod or square hollow sections? All are used with the advantage taken of square hollow sections for long slender bracings.