THE ARUP JOURNAL

JULY 1983

THE ARUP JOURNAL

Vol. 18 No. 2 July 1983
Published by
Ove Arup Partnership
13 Fitzroy Street, London W1P 6BQ

Editor: Peter Hoggett
Art Editor: Desmond Wyeth FSIAD
Assistant Editor: David Brown

Contents

All the papers in this issue of *The Arup Journal* have been taken from THE ARUP PARTNERSHIPS' Structural Steelwork Seminar which was held in London, 8-11 November 1982.

Front cover: Victorian steel – the Forth Railway Bridge (Photo: Eric de Maré)
Back cover: The Papal Cross, Phoenix Park, Dublin (Photo: Harry Sowden)

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.

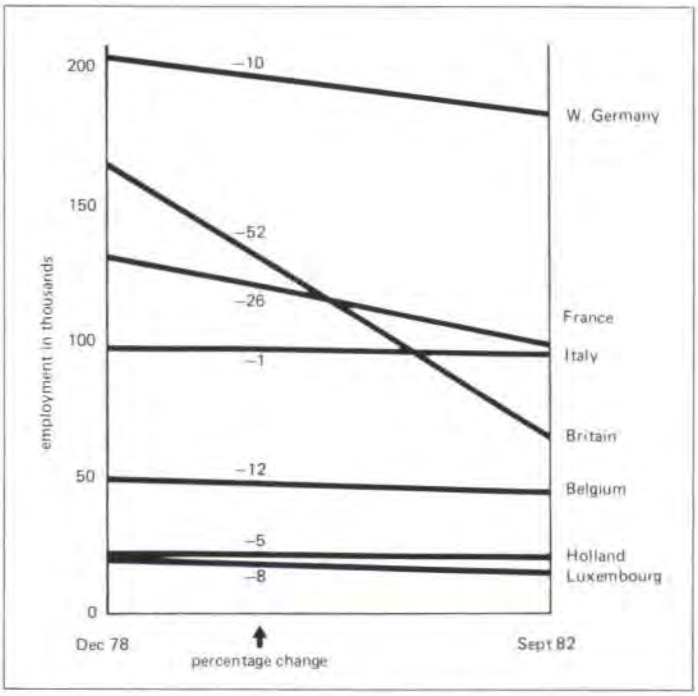
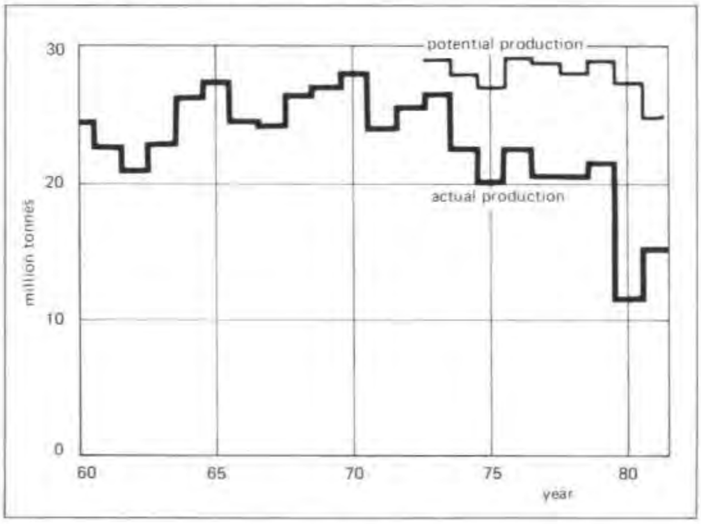


Fig. 1
Employment in the EEC steel industry

Fig. 2
Potential UK steel production capacity against actual UK steel production



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, 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 far 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, spray 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 a 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 multistorey 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 we 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 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.

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

Fire protection

Margaret Law

Introduction

In 1980, Ove Arup and Partners and CBLIA (Centre Belge-Luxembourgeois d'Information de l'Acier) completed a joint study of research into the behaviour of 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.

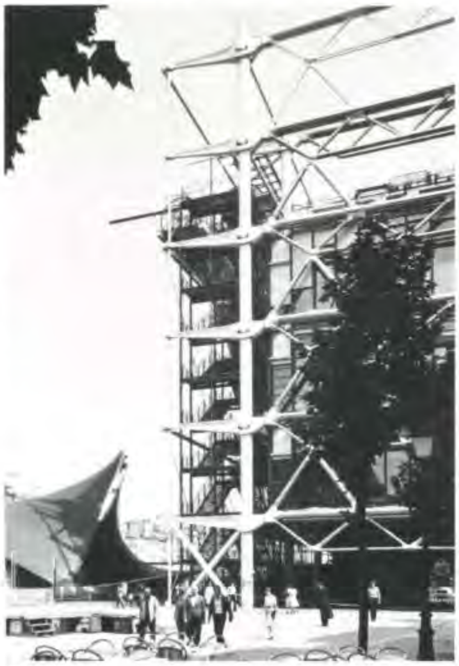
Behaviour of steel in fire

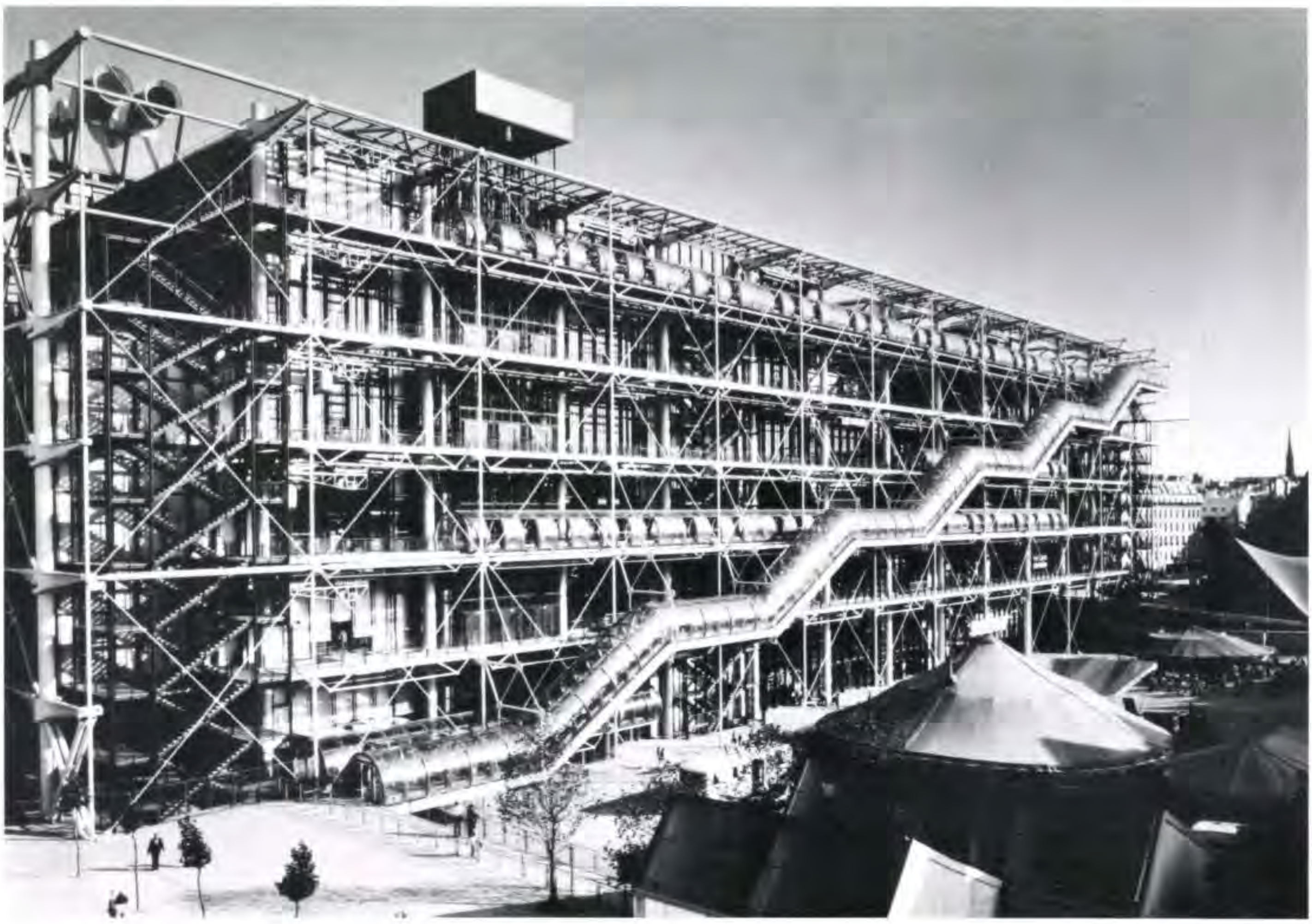
As with other structural materials, the mechanical properties (tensile strength, yield strength and Young's modulus) decrease with temperature. Values have been codified for design[2]. Mild steel, provided it has not exceeded a temperature of about 900°C, recovers most of its strength on cooling.

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.





Figs. 1-2
The Centre Pompidou, Paris.
Architects: Piano & Rogers
(Photos: Ove Arup & Partners)

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, large massivity
- 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 paint, plaster, mineral fibre, cement, concrete. They can be applied alone or mixed, and sprayed, trowelled 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 one must aim to 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' P/A m^{-1}, where A is the cross-section area and P the exposed perimeter. Most steel elements in common use have a section factor of $250m^{-1}$ or greater and for an unprotected mild 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 section factor, the stress level and the grade of steel. A column with $P/A = 29m^{-1}$ (equivalent to a solid 140mm square) would give 30 minutes fire resistance $P/A = 11m^{-1}$ (equivalent to a solid 360mm square!) would give 60 minutes (average temperature of 540°C)[3].

The relationship between fire resistance and stress level can be estimated fairly readily for elements with uniform temperature distribution[2]. However, it has been observed in fire resistance tests 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 600°C. At the moment, since calculation methods assume a uniform temperature distribution, correction factors have been introduced[2].

Water cooling

Water filling of hollow sections has become a well-accepted method of keeping steel cool without the use of cladding. The main problems are the avoidance of dry patches of steel which would overheat, and the provision of adequate water to replenish that which boiled away. A simply-filled column without replenishment can provide over 30 minutes fire resistance, but not as much as 60 minutes. Existing systems therefore usually incorporate storage tanks and also may provide extra water by inter-connection of columns. Much interest has been shown in the possibility of using simply filled non-replenished columns for two-or perhaps three-storey buildings, the water in the top storey being used to replace that boiled off should a fire occur in the lower storey. However, the likely flow pattern within the heated column is not well established; it has been suggested, for example, that the two-phase flow of steam bubbles and water could give rise to surging and large water losses. Existing data for two-phase flow have been obtained from tests with small diameter pipes (of order 25mm) and cannot be extrapolated with confidence to diameters which are an order larger. However, the effect is only likely to occur for large heat-inputs, when columns are heated over two or more storey heights.

Design methods are available for replenished water-cooled systems which deal mainly with the assessment of the amount of water storage needed to replace the water boiled off, and the associated supply systems[4]. These methods are related to exposure from the standard fire and take no account of how the fire might behave in practice.

Concrete filling

Because of expansion of the steel, there is negligible cooling effect when hollow sections are filled with concrete; thus the main structural performance in fire conditions is provided by the concrete core. A very large research programme has been completed but it has not been possible to establish an analytical design procedure. Some empirical relationships are available.

Profiled steel sheet floors

Profiled steel sheet may be used either as a non-composite shutter or as fully composite flooring for reinforced concrete slabs. When used as a composite floor it is difficult to gain acceptance for more than ½ hour of fire resistance unless cladding is applied to the exposed face of the steel sheet or extra reinforcement is placed in the concrete. There is as yet no wide agreement on the way to design such floors for fire resistance. See also the section on profiled steel sheet and concrete floors.

Low fire exposure – internal

If the fire exposure inside a building is sufficiently low then it can be established that the steel elements do not reach critical temperatures and the standard fire exposure is not relevant. The best known example is car parks where tests have demonstrated that a fire involving a motor car is small and is not likely to spread to adjacent cars. In addition, statistics indicate that the risk of fire occurring in a parked car is so low that it has not yet been quantified.

Fire exposure may be low because there is a small fire load in relation to the compartment size and ventilation. Methods of calculating the potential fire exposure already exist and a method of estimating the distribution of severities of fire exposure in rooms (total burn-outs), using results of fire load surveys, has been developed[5]. These analyses could be combined with statistical data on accidental fire behaviour to provide an estimate of the risk of a burn-out occurring. The beneficial effects of automatic sprinkler systems could also be included. The types of low fire load buildings for which such an approach could be profitable are low-rise schools, offices, hospitals, sports halls and concourses in bus stations, train stations and airports.

Low fire exposure – external

It is well established that at certain positions outside the facade of a building the fire exposure will be so low that cladding of steel elements is not needed. The external fire exposure varies not only with position but also with fire load, window area and shape, compartment area and shape.

Design manuals are available to calculate the external fire exposure and the consequent steel temperature[6, 7]. The design method has been used to calculate an average temperature across the section, by analogy with interior steel elements. Although it was known that in practice there could be large temperature gradients there were no test data available at that time which assessed the importance of this effect. Later research with external column/beam assemblies under load showed that mechanical stability could be maintained even though local flange temperatures up to 800°C were measured. Results such as these, if generalized, could greatly extend the design method. In the original analysis of external fire exposure[8] conservative assumptions were made about wind effects because of the lack of test data. More information about the effects of through draughts would be valuable.

No requirements for fire resistance

There are certain circumstances where it is generally agreed that fire resistance is not needed on the grounds of life safety: for example, single-storey buildings and roof structures are not usually required by regulations to have fire resistance if the escape routes are adequate and there is little risk of fire spread to adjacent buildings. In other circumstances, however, fire resistance requirements are made without any clear indication whether these are for the protection of the occupants of the building, or to assist the Fire Brigade to control the fire. There is a tendency to assume that providing fire resistance for every element of structure must automatically increase safety for the occupants, when in fact other measures, such as smoke control or automatic sprinklers, could be more effective. However, it is difficult to analyze the problems and find the best solutions when the purpose of the regulations is not made explicit. There is no agreed basis available which defines acceptable levels of fire safety for structures, taking into account the seriousness of the consequences of structural failure for either life or property.

It is by no means clearly established what the performance of each element of structure should be during the course of a fire. For example, is it necessary for each element of wind bracing to be protected? Is the loading during the course of the fire changing significantly – either for the better or for the worse? It has been suggested that if the wall and roof panels of an industrial building are destroyed quickly by fire, then the steel frame is very under-stressed and can survive without cladding. This would certainly explain why unprotected portal frames in practice behave better than could be expected when exposed to accidental fires. A design method for unprotected portal frames is available[9].

Steel with cladding

In the selection of a material for fire cladding, the designer takes into account the following factors:

- suitablility for use and method of installation (architectural, environmental, technical and economic aspects)
- thickness of the cladding according to the fire resistance required (whether by test or by calculation method)
- maintenance of its integrity and insulating properties.

Results of standard fire resistance tests

In principle the designer considers two aspects:

- the heat transmission through the cladding material and the consequent temperature rise of the steel
- the structural behaviour of the heated steel element and the temperature or temperature distribution at which a critical condition occurs

and for these the necessary data must be provided by tests. In practice, the only information likely to be available is the result of a standard fire resistance test which gives the failure time for a specific steel element protected by a certain thickness of a specific material. It is difficult to separate out the information needed if the results of the test are to be applied more generally to assess,

Figs. 3-4
Bush Lane House, London.
Designers: Arup Associates.
(Photos: Arup Associates)

for example, the protection needed for other steel sections, for other periods of standard fire exposure or for non-standard fire exposures (compartment fires).

The most commonly used method of exploiting the results of fire resistance tests is to assume that failure is related to the attainment of a critical steel temperature and to devise scaling methods or charts based on a simple model of heat transfer from the furnace, through the cladding, to the steel. This method avoids any definition of the structural behaviour or thermal characteristics of the cladding; it provides extrapolation of the result for the particular mode of failure experienced in the test. It can be used to predict results for standard fires, but not for compartment fires. One way of extending the use of standard test data to compartment fires is to establish an equivalence between the standard fire and compartment fires in terms of their heating effect on the structural element, the effect being attainment of a certain critical temperature.

Deficiencies of the standard test

An examination of standard test procedures and the results obtained exposes several deficiencies. First, it has been observed that some cladding materials in accidental fires may undergo physical changes not displayed in the standard test, because of higher fire temperatures in the compartment. For this reason a high temperature test, supplementary to the standard test, is sometimes suggested.

Secondly, unless the test is part of a research programme, the initial properties of the specimen are not measured. The test load is related to the characteristic strength, not the strength of the sample tested.

Thirdly, the results for test specimens which are nominally the same will vary from laboratory to laboratory. This is not surprising since, apart from any differences in heating characteristics, the real degree of restraint of the specimen is quite often unknown and may vary during the test. This is particularly a problem with slender columns.

It can well be argued that the standard fire resistance test is only a blunt instrument and in practice it serves its purpose because there have not been any serious structural failures in fires. If this is true it can also be argued that calculation methods are just as likely to give the 'correct' answer and are much cheaper. It then follows that the fire resistance test should be re-designed so as to provide information for these calculation methods.

Suspended ceilings

Fire resistance tests have demonstrated the value of suspended ceilings for the protection of steel beams and floor systems. A calculation method has been proposed which could be used to generalize test data, but the main problem is probably the difficulty of ensuring that the ceilings are installed correctly in practice. The detailing of the suspension system, lighting fittings, etc., is of great importance.

Composite steel and concrete

Structural analysis of composite steel and concrete is difficult to carry out for normal conditions. It is not surprising, therefore, if it presents problems of analysis under fire conditions. Nevertheless, it ought to be possible to carry out fire tests which would lead to a definition of the critical conditions for failure and lay the basis of a design method.

Profiled steel sheet and concrete floors

Test data so far obtained indicate that for 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 load-bearing capacity can be obtained than would be expected by considering the column in isolation. An understanding of the factors which affect the performance of assemblies might lead to the reduction or elimination of part or all cladding for the lower standards of fire exposure. The research carried out so far has not yet reached the stage at which critical conditions can be defined.

Research on columns has so far been directed towards axially loaded members. In practice there will be combined normal forces and bending moments but the effect is not well understood. It has been suggested that compression properties at high temperature may be more favourable than tension properties. However, there is a lack of data to confirm this hypothesis.

The standard fire resistance is not designed to test vertical members in tension. The critical condition must therefore be defined by calculation, since conventional critical temperatures derived from loaded column tests may not be correct, particularly when the hanger is a special steel. At present no guidance is available.

Properties of steel at high temperatures

The tension properties of mild steel at temperatures up to 600°C are well documented, for example, and there are some measurements available for higher temperatures. Creep measurements are available up to 650°C. It is not clear how important it is to separate out stress-induced strain from creep-induced strain but in view of the high steel temperatures measured before failure may occur, more information may be needed. Tests have confirmed that mild steel elements, which have not exceeded the critical temperature for failure, regain their elastic properties after cooling.

Mechanical properties of cold-formed steel C-sections for temperatures up to 650°C are also available.

Use of cold formed sections

The use of lightweight cold-formed steel sections to replace timber studs inside partitions has been studied and a calculation method devised. Tests have shown that these sections can also perform satisfactorily as columns and beams when enclosed in cladding materials but the results have not yet been generalized to give a design method.

Views of the steel industry

We considered that it would be helpful to have an indication of the views of those people in the steel industry who will eventually use the results of the research and an indication of existing regulations in different countries. We therefore prepared a short questionnaire which was circulated to the various Steel Information Centres in Western Europe. Replies were received from Centres in nine countries.

Fire exposure and fire resistance calculation

All countries favoured the acceptance in national regulations of a calculated design fire, taking into account fire load, ventilation and compartment size, as an alternative to the standard fire resistance test. It is accepted generally only in Sweden.

In only three of the countries (France, Sweden and Switzerland) is calculation of the fire protection needed generally accepted as an alternative to standard fire resistance tests. The other countries were in favour of calculation, but one (Germany) only sometimes.

Only two countries (Sweden and Switzerland) have agreed representative values of fire loads for a range of buildings; the Netherlands for office and residential buildings only.

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 Italy only sometimes) and for roof structures (only sometimes in Germany and exceptionally in Belgium). Seven countries accept unprotected steelwork for car parks (in Germany, Italy and Sweden only sometimes, not accepted in Belgium and France). Switzerland alone accepts unprotected steelwork for any building with low fire load (less than 60M cal/m^2 which is equivalent to about 14 kg/m^2 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 adhesion of the cladding. Only one country (Austria) 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 (Austria, Germany and Switzerland) are satisfied 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 three (Germany, 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 or all 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 on the part of both building 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 wider and better use of steel in building 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.

Performance requirement

A performance requirement should state the objectives of providing fire safety – whether it is for the protection of people (occupants, neighbours, fire fighters), the protection of property (contents, structure, neighbouring buildings) or a mixture of both. Ideally, the required level of safety should be stated and methods of assessing the potential hazard should be agreed. It would then be possible to determine the level of structural fire protection needed, if any, and the effect on this level of employing alternative protective measures such as automatic sprinklers. Clearly this is a wider subject than the use of steel in building construction.

Structural performance

The development of a calculation method for the fire safety of a structure, which could be integrated into the methods for structural design at normal temperatures, is hampered first of all by the present system of grading single elements in a standard test and second by lack of knowledge of the behaviour of structural assemblies and the loads acting on them in real fire conditions. The information needed would include:

- a study of the present standard test method, so that it can be adapted, altered or extended to provide more information for design purposes
- a separation and definition of its function, as a heating mechanism and a loading mechanism, would show how results could be applied when data are provided by the following:
- a study of the critical conditions for failure of loaded elements and frameworks. For calculation of these critical conditions more reliable data would be needed for the following:
- measurements of the mechanical properties of steel at temperatures above 600°C.
- assessment of the loads which should be deemed to be acting under fire conditions – wind load, live load, dead load.

Some of these studies are of a long-term basic nature, but much information which already exists could be exploited with the addition of a little more data.

References

(1) BEHETS, J.F. *and* LAW M. Study of research into the behaviour of structural steel elements exposed to fire. Report to European Coal and Steel Community, Centre Belgo-Luxembourgeois d'Information de l'Acier, Brussels. Ove Arup and Partners, November 1980.

(2) EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK. 'European recommendations for the fire safety of steel structures. Part 1. Calculation of the fire resistance of load bearing elements and structural assemblies exposed to the standard fire'. Elsevier, 1983.

(3) STANZAK, W.W. *and* LIE, T.T. Fire resistance of unprotected steel columns. *ASCE, Journal of the Structural Division.* 99 (ST 5) pp 837-852, 1973.

(4) BOND, G.V.L. Fire and steel construction. Water-cooled hollow columns. Constrado, 1975.

(5) COWARD, S.K.D. A simulation method for estimating the distribution of fire severities in office rooms. Building Research Establishment Current Paper CP 31/75, April 1975.

(6) AMERICAN IRON AND STEEL INSTITUTE. Fire-safe structural steel. A design guide. Washington, D.C., 1979.

(7) LAW, M. *and* O'BRIEN, T. 'Fire safety of bare external structural steel'. Constrado, 1981

(8) LAW, M. Fire safety of external building elements – the design approach. Engineering Journal. American Institute of Steel Construction. Second Quarter, pp 59-74, 1978.

(9) CONSTRADO. The behaviour of steel portal frames in boundary conditions. Constrado, 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 pipes. Similar structures that we have designed or analyzed include 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. Main 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 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.

Tower height	Maximum deflection		
	0.75°	1.2°	1.2°
50m	6	8	10
50 – 100m	5	7	9
100 – 150m	4	6	8

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:

Height/width	Panel type
h/w>1.5	Z
1.0<h/w<1.5	X1, X2
0.7<h/w<1.0	X1, X2, K1, K2
0.5<h/w<0.7	K2

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

bolted structures	$20° < \theta < 90°$
welded structures	$30° < \theta < 90°$

Solidity ratios:

Typically solidity ratios for different widths are as follows:

Panel width	Solidity ratio
< 1.5m	0.25
< 5.0m	0.20
<10.0m	0.15
<10.0m	0.10

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 exceeding 180 where 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
(Photo: Mike Shears)

Fig. 3
Microwave tower
under construction in Nigeria,
designed by Ove Arup & Partners
(Photo: Mike Shears)

Detailed design considerations

Design standards

Design standards may be specified in the head contract specification according to the country of origin with reference to more specific internationally recognized design codes. In recent years a number of design guides and codes of practice appropriate to tower structures have been published. The main ones are:

(1) *BS6235: 1982*
Code of practice for
fixed offshore structures.

(2) Department of Energy 1982
Offshore installations:
guidance on design and construction.

(3) *API RP2A: 1982*
(American Petroleum Institute)
Recommended practice for planning, designing and construction fixed offshore platforms.

(4) DnV (Det norske Veritas)
Rules for the design, construction and inspection of offshore structures.

(5) Draft Code of Practice 1978:
Lattice towers – loading.

These documents tend to complement each other in particular areas but they also refer to national design standards and some interpretation is often required.

Strength and stability

Where no other design standards are given, the strength and stability of towers are most commonly assessed on the basis of the design approach given in *BS 449*. The permissible stresses incorporate factors of safety against failure in the range of 1.5 to 1.7. As wind is a predominant loading feature on these structures we tend not to use the 25% overstress permitted by *BS 449*.

Foundations are designed to have a factor of safety in excess of the tower structure. The factor of safety appropriate to uplift, downthrust, sliding and overturning is a minimum value of 2.0. It is also advisable to check that an adequate factor of safety exists for an extreme or survival wind condition.

Deflection

This may be considered as a serviceability limit state for the design of microwave towers. The acceptable limits of deflection vary from aerial to aerial which in turn are related to the frequency band of the microwave system. Tall, slender structures will tend to deflect more than fatter ones for the same applied loading. However, the member sizes for slender towers may be larger than the equivalent members for wider towers to carry the applied loading. On the other hand wide towers tend to require more bracing and will therefore attract more wind loading. The objective therefore is to juggle the geometry so that the strength and deflection requirements are achieved for the minimum weight of steel.

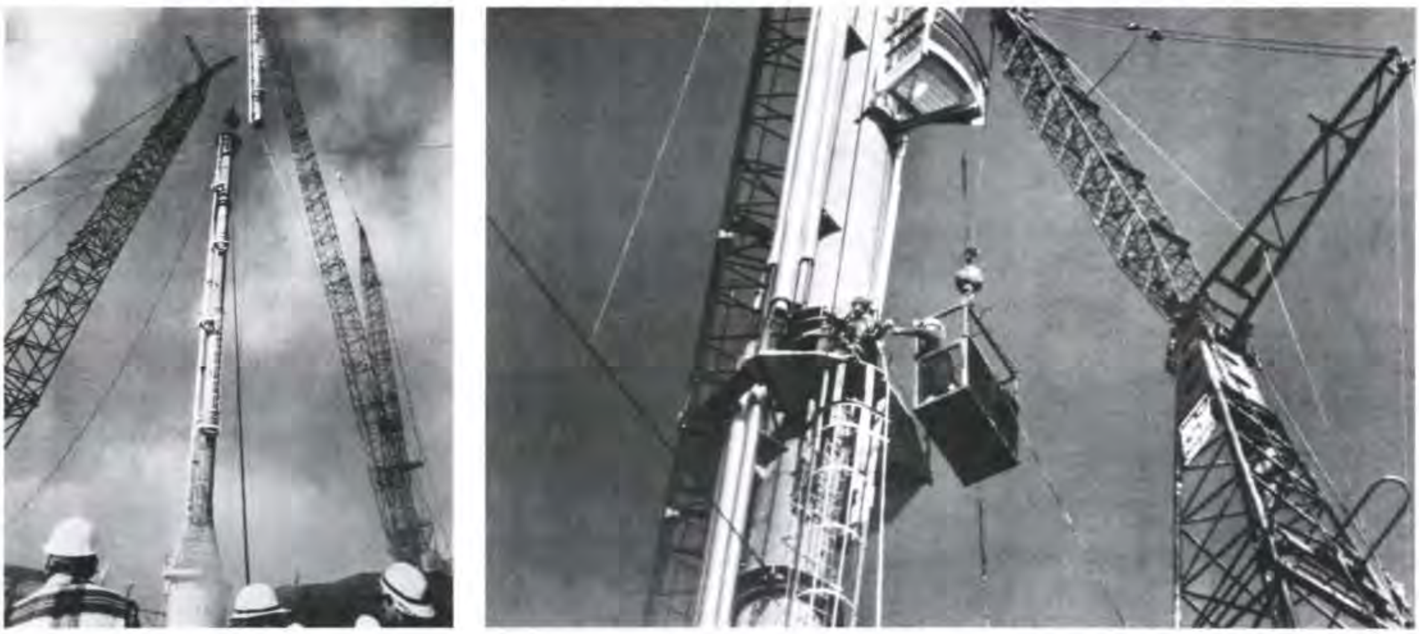
Selection of member types

There are three main types of steel sections used in tower design; tubes, equal angles and cruciform angles. The advantages and

Fig. 4
Brent C flare tower: transportation of prefabricated sections. Arups carried out dynamic fatigue and analysis checks as well as being responsible for obtaining certification. We were also responsible for the appraisal of lifting and temporary works. (Photo: courtesy of Sea and Land Pipelines)

Fig. 5
Brent C flare tower: erection of tower sections (Photo: courtesy of Sea and Land Pipelines)

Figs. 6-7
Fort McMurray flare stack, Canada: erection of guyed riser with riser fabricated in sections and erected in two parts. Designed by Ove Arup & Partners. (Photo: courtesy of Bechtel International)



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

Tubular members

Lend 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. The details of the application of the TLP to the North Sea development can be found elsewhere[1].

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 DnV[2] (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 (γ f) on these loads have to be considered as unity and member forces factored by an appropriate stress factor. This stress factor (γ p) 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 (γ p) 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 carrried out analyses appropriate to

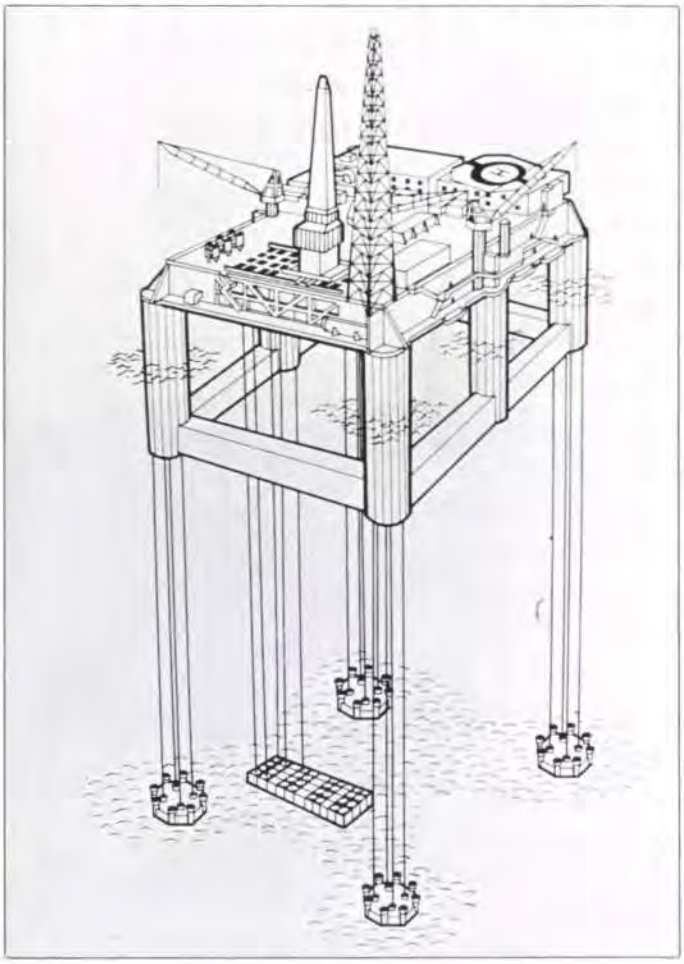


Fig. 1
Schematic view of Hutton TLP and, below – the geometry

Table 1: Geometry (All dimensions to moulded lines)

Length:	Between column centres Overall	78.00m 95.70m
Breadth:	Between column centres Overall	74.00m 91.70m
Height:	Keel to main deck Main deck to weather deck	57.70m 11.25m
Draught:	Operating	32.00m at LAT
Freeboard:	To underside of main deck	24.50m at LAT
Water plane:	Area	1324.00m^2
Columns:	4 Corners 2 Centre	17.70m dia. 14.50m dia.
Pontoons:	Height Width Corner radius	10.80m 8.00m 1.50m
Displacement:	Approx.	61500 tonnes
Total weight:	Including riser tension (Approx)	48500 tonnes

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:

$$F = \Sigma F_i \gamma_{pi}$$

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

$$\text{Safety factor} = \gamma_m . \gamma_f . \gamma_p . \frac{K}{\psi}$$

where γm is the partial factor on material strength, K is the slenderness ratio and ψ 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[3,4].

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 (γ p).

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

Fig. 4
Pontoon section being lifted into position during assembly at Hutton TLP lower hull at Nigg

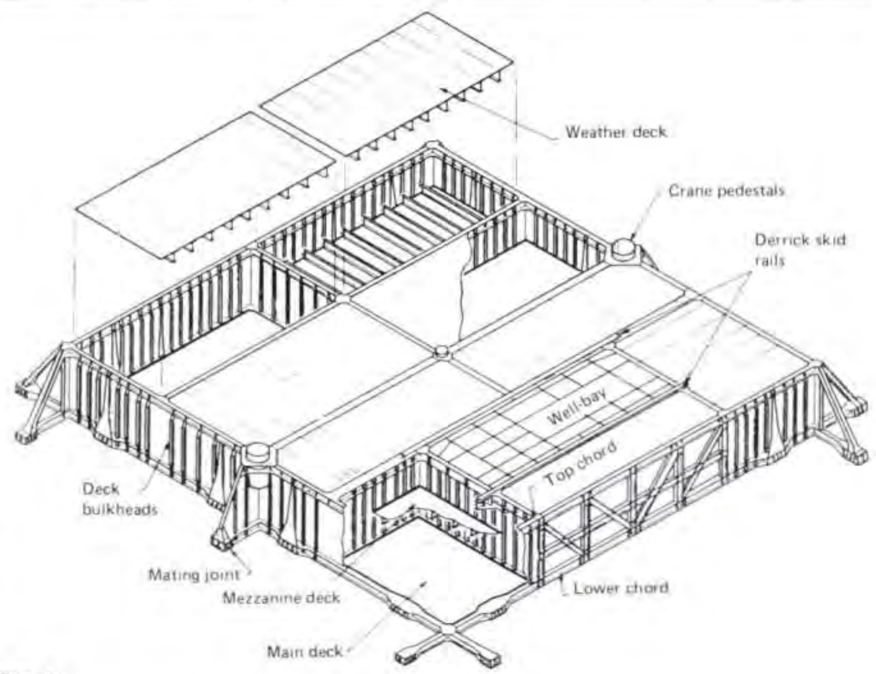


Fig. 6
The deck structure of the TLP

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

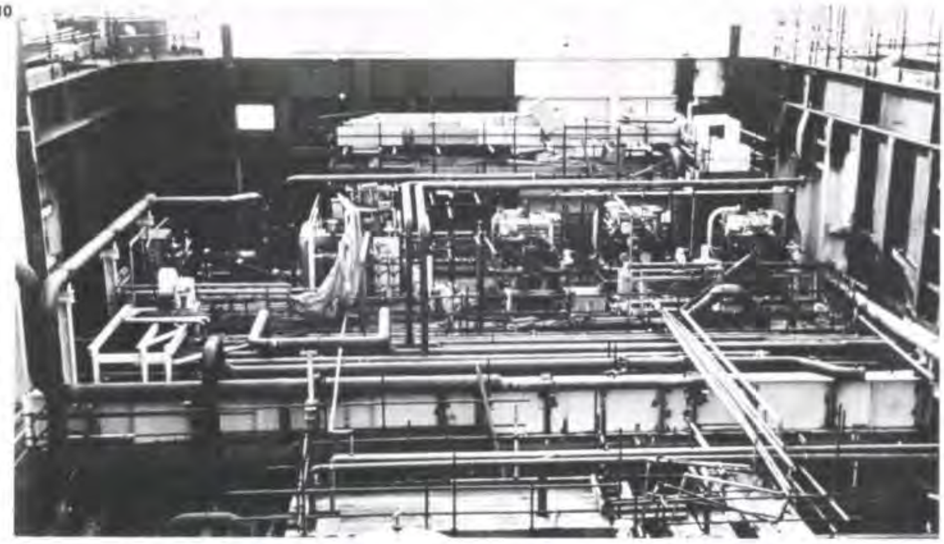
Fig. 7
Foundation template units on barge at Invergordon awaiting final shipment to Hutton TLP site, for installation

8

9

Figs. 8-10
Hutton TLP integrated deck under construction at Ardersier, Scotland. Fig. 10 shows internal area partially fitted out with bulkhead girders in background (perimeter of area shown) (Figs. 8-9 photos: S. Lewandowski, all other photos: Nick Prescott)

10



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 interconnected jacks at each column top at the time of deck to hull mating. This ensures that the dead load is equally and determinately 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 torsioned 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 panel'.

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 sniped flanges as elsewhere. The hole itself would have rounded corners 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

(1) ELLIS, N. *et al.* Hutton TLP vessel structural configuration and design features. (Paper OTC 4427) Proceedings 14th Annual Offshore Technology Conference, Houston, 1982.

(2) DET NORSKE VERITAS. Rules for design and construction of offshore structures. DnV, 1977.

(3) BRITISH STANDARDS INSTITUTION. BS 5400: Part 10 1980. Steel, concrete and composite bridges: code of practice for fatigue. BSI, 1980.

(4) SMITH, I.J. *et al.* Assessment of fracture toughness requirements for the deck structure of the Hutton tension leg platform using finite elements and the CTOD design curve. (Paper OTC 4430). Proceedings 14th Annual Offshore Technology Conference, Houston, 1982.

The Central Electricity Workshops, Johannesburg

Barrie Williams

Architects: Rhodes-Harrison Fee & Bold

Fig. 1
Master site plan

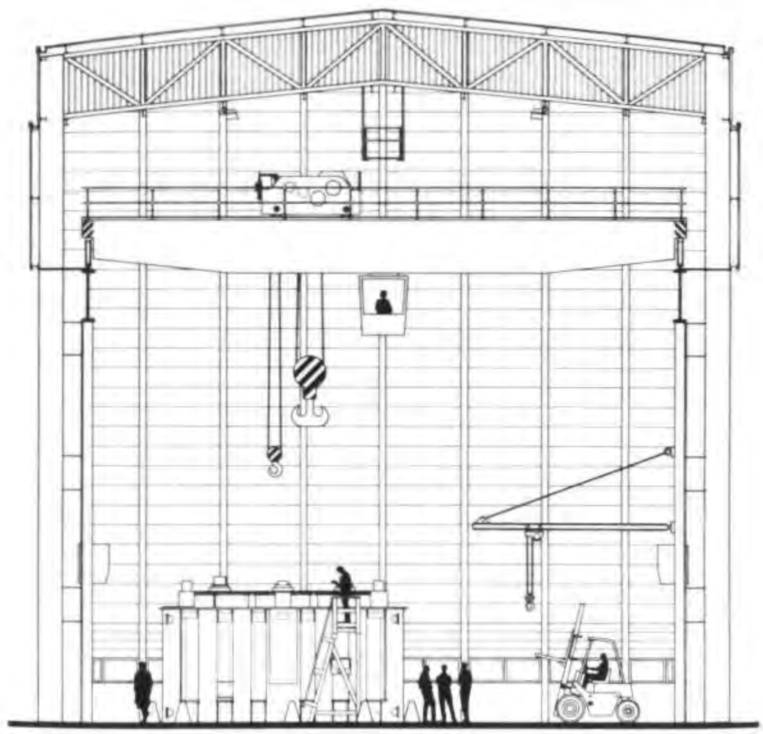


Fig. 2
Section through new workshop complex

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 below 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 not 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 girder, girt, 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 schemework 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.

Figs. 3-6
Central Electricity Workshops, Johannesburg.
Aspects of the new complex

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 Treasurer's Stores

Building F:
Carpenters' and Black Trades' Workshop

Schedule of building data

Building	Height to eaves m	Span m	Area m^2	Total steel mass tonne	Mass kg/ m^2	EOT cranes
C1	15	18	3230	360	111	2 x 10T
C3	20	18	1340	300	223.9	1 x 150/25T
C5	16	45	3936	337	85.6	1 x 30/5T
D1	7	36	3960	185*	46.7	—
E	9.5	9 + 18				3 x 2T
		+ 36	5783	730*	68.3	Hoists
F	9.5	9 + 18				
		+ 36	5824	371*	63.7	1 x 15T

*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 (fabricated) and tubes for the 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 more 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 as, due to 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 would 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 glass walls covering the full extent of the buildings. The main frame side sway due to crane surge (40mm) caused great concern among the patent glazing 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 2½ 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 (stress and inertia wise) required relatively thin plates in the webs for the column proportions chosen. With the fully welded up section complete with welded internal stiffeners, we were very 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 8mm. Even 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 big 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 name tags 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 ø 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 we have access at roof level, i.e. too late to take corrective action on the ground. 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 end the sheeting contractor solved the problem by installing a supplementary sag rod system of thin straps or ties connecting both top and bottom purlin flanges. The lower of these remains visible – a not altogether satisfactory solution. 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 work has started on a third.

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.

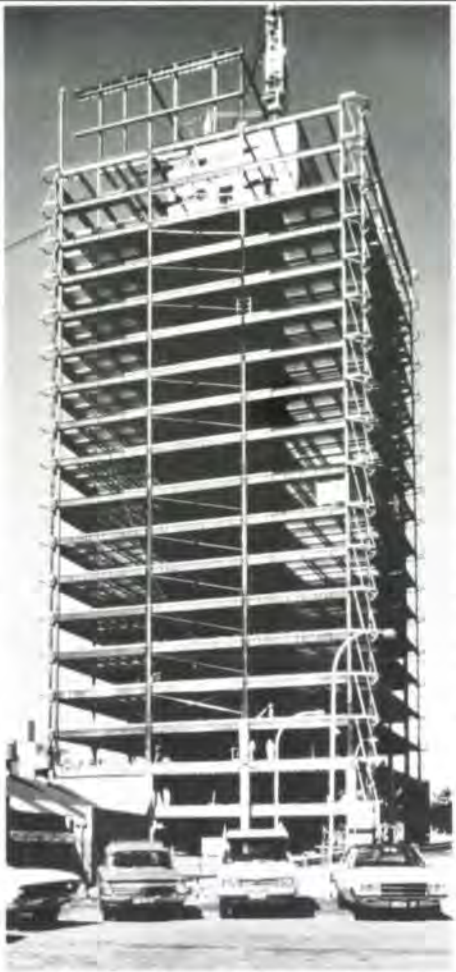


Fig. 1 Penmor Tower. Architects: Nurcombe Summerley Ringrose & Todd

It is an 18-storey office building in Johannesburg, providing about 17,000m^2 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

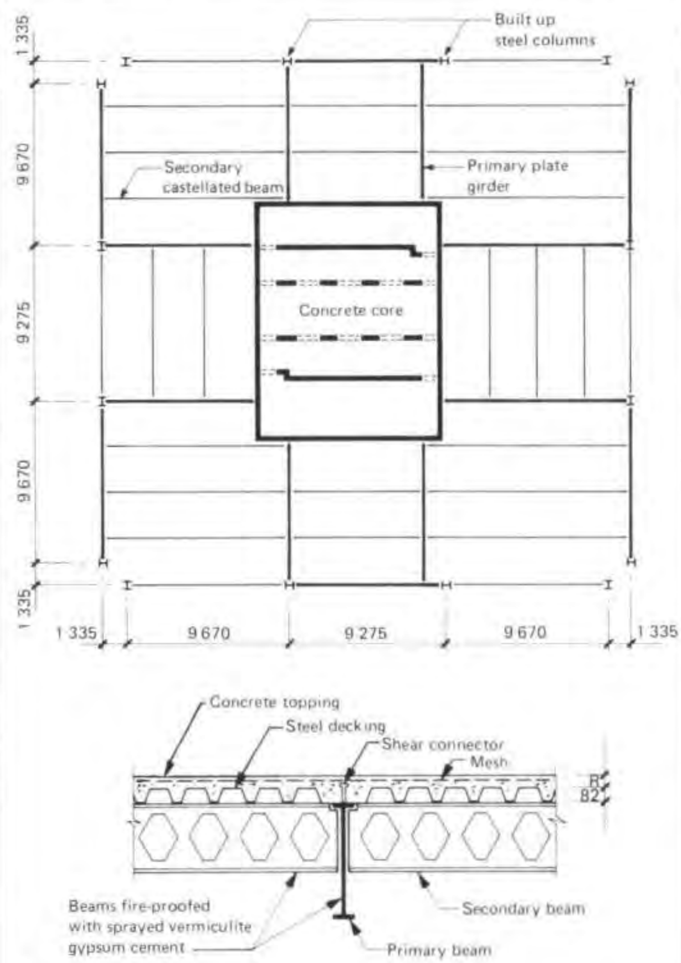


Fig. 2
Penmor Tower, typical floor, steel layout and details

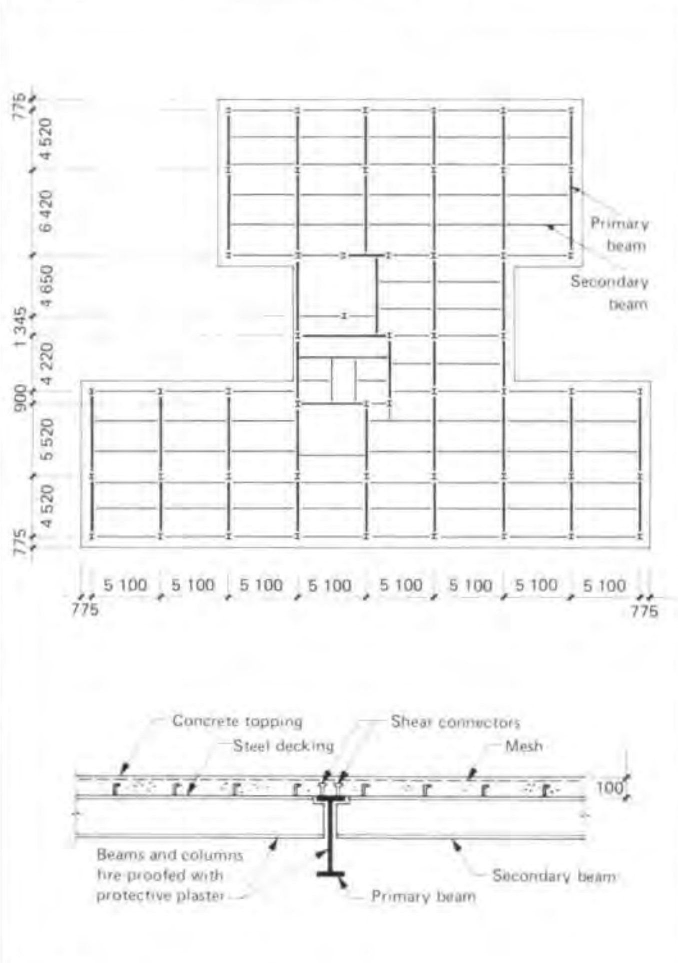


Fig. 3
Dennehof Park, typical floor, steel layout and details

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 comparisions 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 1000m^2 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 450m^2, 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

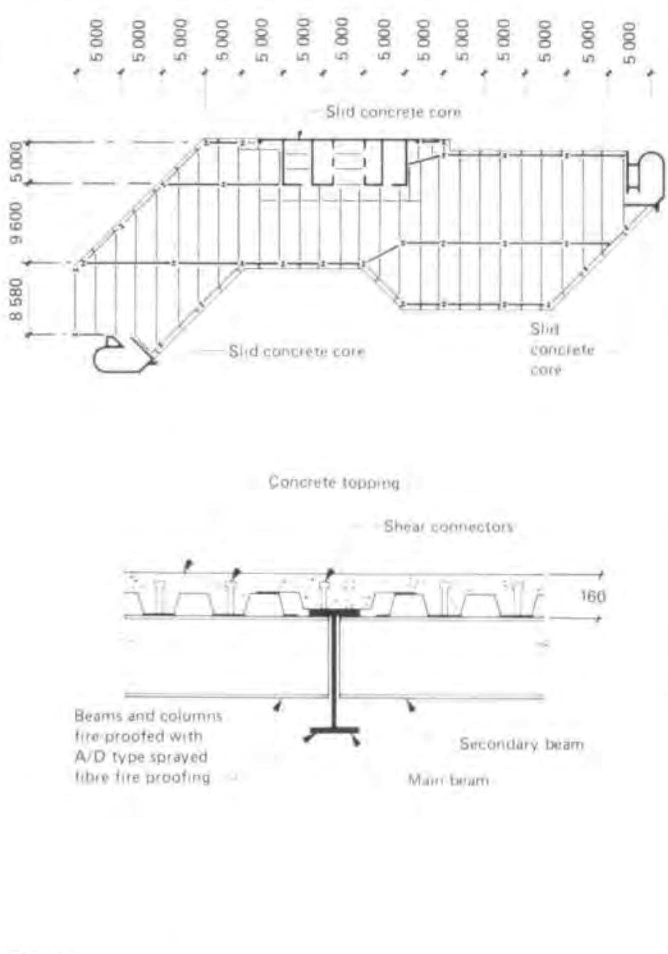


Fig. 4
Garden Plaza, typical floor, steel layout and details

Table 1: Structural costs of steel-framed buildings

	Costs at 1980 rates: Rands/m²		
	Penmor Tower	Dennehof Park	Garden Plaza
Steel beams	R 44.90	R 22.06	R 43.30
Shear studs	2.13	3.70	3.40
Steel columns and bracing	19.00	13.30	15.60
Steel decking	12.00	11.72	16.00
Concrete topping including mesh and power float finishing	11.00	10.29	9.96
Temporary propping of deck	2.50	1.72	—
Fire protection (Plastered or Sprayed Vermiculite)	14.00	13.16	14.42
	R 105.53	R 75.95	R 102.68

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 *CP117* 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 rig 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. Precautions 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

Fig. 7 11 Wellington Road, Parktown. Architects: Design Collaborative

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 practically 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.

The most commonly used methods to date have been the sprayed or plastered vermiculite or pearlite compositions. Nevertheless, these are time-consuming wet trades, and fire protection remains one of the important time and cost drawbacks to the effective use of structural steelwork for buildings, due to the limited experience in the local industry and relatively small market.

Cost comparisons

Table 1 sets out the elemental estimating components for three typical steel office structures. Changes in detail could occur – for example cement mortar screeds may be preferred in certain cases to a power floated finish. However, power floating has the advantage of speed and minimum weight. Furthermore foundation requirements would have to be analyzed to suit the particular design and site conditions and must be built into the cost estimate.

It must also be noted that the fireproofing of structures by spraying demands that the perimeter of the building be clad with temporary tarpaulins to avoid material being sprayed over the neighbourhood. The rate per m² for the spray application should include for this requirement. As a rough guide, the profile area of fireproofing to columns and beams is approximately equal to the gross floor area of the steel building.

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 to medium rise projects. However, the success of the project is entirely dependent on good 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 benefit 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 20 floors of about 2000 m², 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.

Local reports summary

Jim Hannon

The following digest is from the papers by:

(1) David Bedford – Hong Kong

(2) Derek Blackwood – Scotland

(3) Ian Mackenzie – Australia

(4) Cliff McMillan – South Africa

(5) Finbar McSweeney – Ireland

for the Arup Partnerships Structural Steelwork Seminar in 1982.

(1) HONG KONG

Arup involvement with steelwork in Hong Kong tends to be limited to specific elements of construction and the design of temporary works.

The local fabrication facilities are minimal, apart from a few local shipyards which have begun to diversify the field of structural steelwork construction. This means that, generally, structural steelwork is fabricated overseas, thus requiring a long lead-in time, and once the shipping of the fabricated sections has commenced there is little flexibility to incorporate alterations or modifications.

The 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.

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:

(a) Civil engineering

(b) Low rise buildings

(c) Roof structures.

Fig. 1 Almondell Footbridge

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

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, Forth 150. This included a steel deck structure comprising 4,000 tonnes of grade 50D tubular steel.

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 Merrison 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 hydraulically operated hinged steel ramps.

(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 unhampered 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.

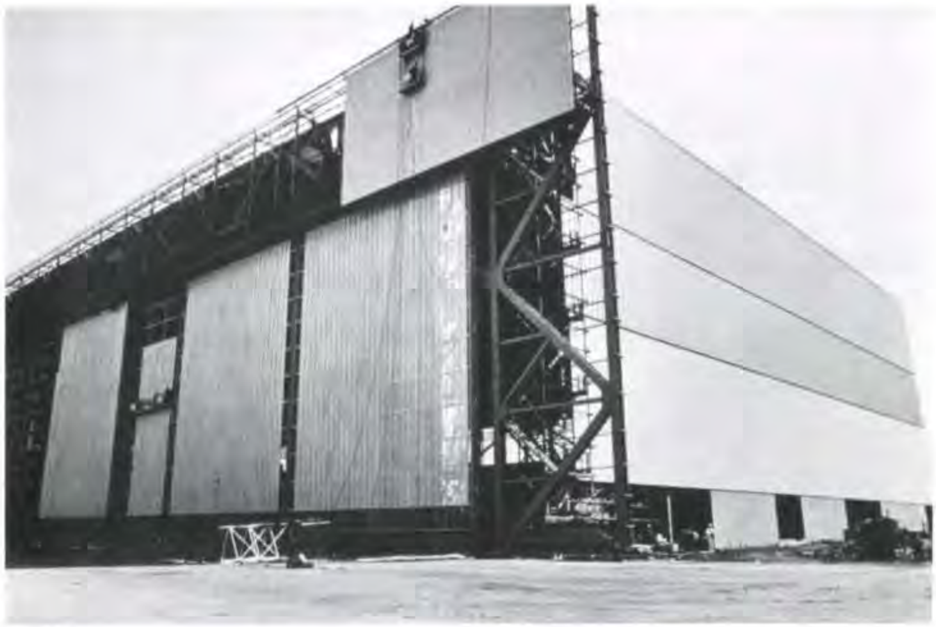


Fig. 3-4
Hangar 8, Jan Smuts Airport, South Africa. Architects: South African Railways

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 purlins. Walls may be metal cladding on C 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 purlin 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 dewatering 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 58m 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 '80s. 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 conceptual 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.

The main 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 at the top of the legs. The main trusses were lifted in sections on to trestle 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 ridges 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 a 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 one complete hangar frontage needing 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. Most of 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 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 skimped 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 most situations. Automated cutting and drilling has recently become available but is still unusual. Welding techniques nowadays almost universally employ semi-automatic CO_2 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 (bolted 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 18,000m^2 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 2,000m^2 factory notable for its unusual use of concrete block vertical structure. However the steel is also interesting, using 686 x 152mm castellated universal beams (CUB) as main beams on a 15m, 3 span grid and lighter CUB of the same dimensions as secondary beams in alternating 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

Fig. 5
Bailey's Irish Cream factory: lifting the space frames. Architects: Scott Tallon Walker

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. 6 Central Bank of Ireland head office under construction. Architects: Stephenson, Gibney & Associates

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

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 F'c 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.

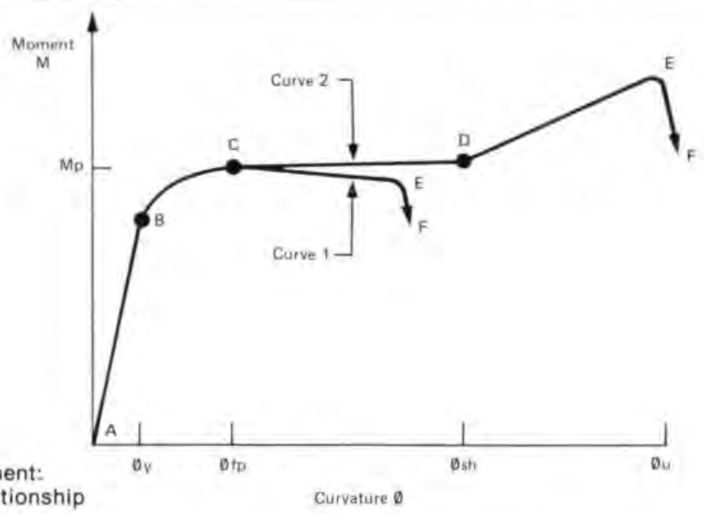


Fig. 1
'Standard' forms of moment: curvature relationship

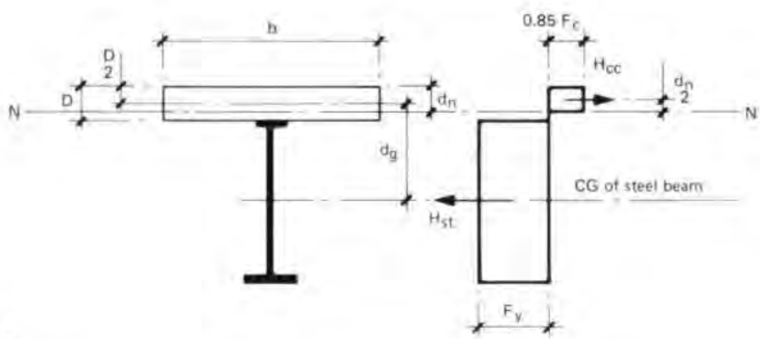


Fig. 2

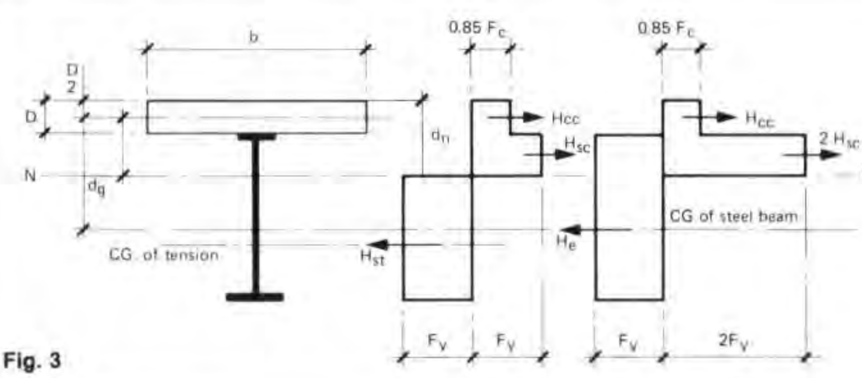


Fig. 3

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 F'c
(d) The concrete has no tensile strength.

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

$$M'r = \theta\, FyAs \left[dg + \frac{D - dn}{2} \right]$$

$$\text{where } dn = \frac{FyAs}{0.85\ F'cb} \leqslant D$$

$$\theta = 0.95$$

(Refer to Fig. 2).

Ultimate moment of resistance for compressive stress in steel beam

This occurs when FyAs>0.85 F'c bD

(Refer to Fig. 3)

He = FyAs
Hcc = 0.85 F'c bD
Hsc = FyAc where Ac = Area of steel compression

For equilibrium:
He = Hcc + 2Hsc
FyAs = 0.85 F'c bD + 2FyAc

$$\text{Hence } Ac = 0.5 \left[As - \frac{0.85\ F'c}{Fy} bD \right]$$

dn 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.

Working stress method

Design of composite sections by the working stress method is in accordance with elastic theory using a transformed area approach and assuming no slip at the concrete steel interface. The area of slab in tension is neglected in calculating the area.

Rules are given for calculating the effective moment of inertia and effective section modulus where profiled sheeting is used.

Permissible stresses are the relevant values in the steel and concrete codes.

Serviceability

Stress

For unpropped construction the stresses in the steel beam must not exceed 0.9 Fy under serviceability loads.

Deflection

Deflections can be calculated using a modular ratio of n for transient loads and 3 n for sustained loadings. A general deflection limit of span/250 is suggested with span/ 500 or an absolute value of 20mm where damage could result from deflections.

Deflections during the construction stages, particularly for unpropped construction, should be borne in mind when considering the need for cambers.

Vibration

Comparatively light structures can result from composite design and the Australian code gives guidelines aimed at limiting damage to the structure or discomfort to the occupants.

(a) Floor frequency should be greater than 5 Hz for occupancies such as residential floors and schools.

(b) Floor frequency should be greater than 10 Hz for repetitive activities such as dancing, unless there is a large amount of damping.

(c) Transient vibrations can cause discomfort or annoyance to occupants, and specialist literature should be consulted.

(d) For further guidelines reference can be made to a paper by D. L. Allen: 'Vibrational behaviour of long span floor slabs'. Canadian Structural Engineering Conference Proceedings, Toronto 1974.

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 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 loadings

(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 surface 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 fabrication shop because of 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. Alternatively, propping 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 quantity 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 fascinating. 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, as billed, about the skeletal frame. Instead he talked about the design process. Few realized, fewer cared for 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 own ability to analyze problems. The danger in following a precedent is when the context changes so that the solution is no longer appropriate.

Extrapolation requires care. Precedent relieves 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 requires that we cast aside predilections and solve the problem on its own merits. Voltaire in the 'Temple de Goût' summed up his appreciation of intuitive design in this way: 'Simple was it noble architecture. Each ornament arrested, as it were, in 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 represent a break with 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 fluted chimney representative of Greek 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 adopted and it becomes a norm. And so the life cycle of the design process repeats itself.

To illustrate the impact of precedent and intuitive thinking on the design process, Derek chose a number of key events in history when either the problem set the designer changed – in context or scope – or when new solutions presented themselves – either from developing technology or the skill of an individual.

Scherzo: Historical examples of the influence of precedent and intuition

The conflict in China:

There are early examples of the precedent/ initiative dilemma. In 8th century China, the design of timber buildings was highly regulated by the State through detailed building codes. These reflected the technology commonly used and dictated the appearance of the end product. But this same society had built a large span masonry arch bridge (c.610) and a post and lintel bridge (c.820).

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

The arch bridge bears a close resemblance to Lutyens' Runnymede bridge of 1935 (though the connection is not thought deliberate) and also with Arups' own bridge at Runnymede (which deliberately constrained itself within the earlier precedents though not without modification).

Substitution of new materials:

Unaware of the Chinese precedent, intuitive designs for bridges appeared in Europe much later – Leonardo's temporary bridge of 1490 and Rennie's Wye Bridge at Chepstow are good examples.

When it was decided to bridge the Severn at Coalbrookdale, the natural choice of material was masonry. But Coalbrookdale was the centre of iron making in England in the 18th century and the architect Thomas Pritchard sought the choice of one 'iron-mad' Wilkinson. Not surprisingly the bridge was built of cast iron, which at least illustrates how architects can be affected by passion. The bridge is a good example of how a new material – in this case cast iron – is merely substituted in a solution for which a precedent already exists. The link in this case is the pattern maker whose skill in forming the casting moulds parallels his ability to manufacture timber centring.

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-Cysyllte 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 1803-4.

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 rationally 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 capitols 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 1830, 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 in cast iron was only rarely solved elegantly. By 1874-75, Jabrouste was able to define an altruistic form for the cast iron column in the second of his library buildings – the National Library. Gone is the ornamentation and the iron foliage. Instead 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 1890s 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 Brunel'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 mouldings, no-meaning 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 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

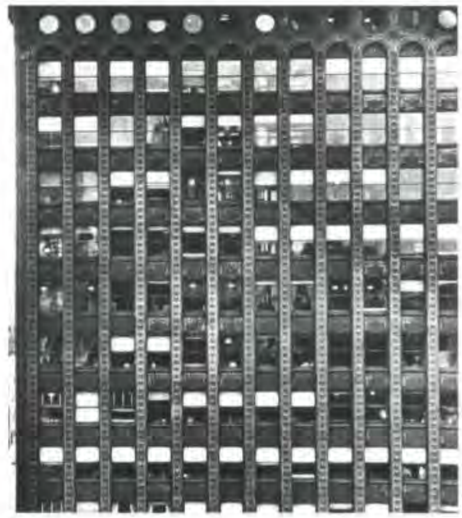


Fig. 9
Guaranty Building, Buffalo, USA 1894-95, façade

Fig. 10
Shell Mex House 1933 – Portland Stone quarries hung on steel frames as architecture is sacrificed on the altar of structural engineering (Photo: Architectural Association)

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. Incidently 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 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 which combine 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

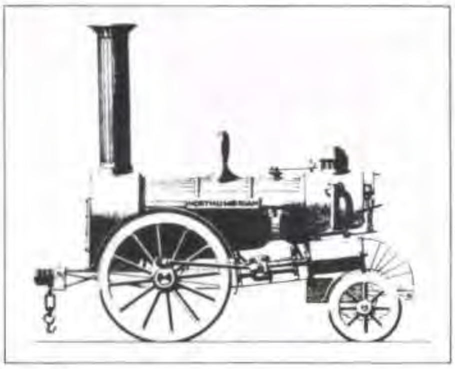


Fig. 13
Northumbrian 1830 embellished with Stephenson's Corinthian fluting to the chimney

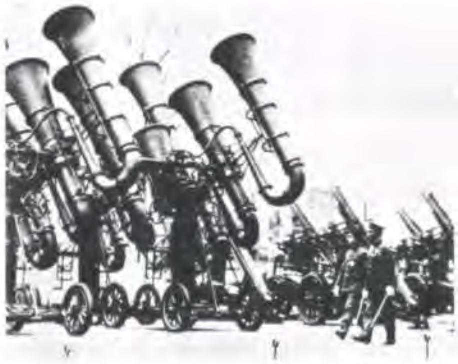


Fig. 14
Emperor Hirohito of Japan (left) with his chief of staff inspecting sound detectors for locating enemy aircraft, Osaka 1934

Good stuff. Baker replied at the Edinburgh Literary Institute. He expressed doubts as to whether Morris had the faintest knowledge of the duties which the great structure had to perform. How then could he judge the impression which it made upon those that could appreciate the direction of the lines of stress and the fitness of the several members to resist the forces? Probably, Baker went on, 'Mr. Morris would judge the beauty of a design from the same standpoint, whether it was for a bridge a mile long or for a silver chimney ornament. It is impossible for anyone to pronounce authoritatively without knowing its *functions*. The marble columns of the Parthenon are beautiful where they stand but if we used it as a funnel for an Atlantic liner, it would to my mind cease to be beautiful but of course Mr. Morris may think otherwise'.

The dangers of new technology

New technology could no longer be ignored by those supporting precedent, partly because technology had to be comprehended in order to attack it and partly because the technologists themselves were not above abusing their new found talent. Benjamin Baker was very well aware of this. His great dictum was that all theoretical calculations involved assumptions that were often convenient rather than true; that they could be valuable as a check but disastrous if accepted with blind faith. Technology, he held, was of little value unless it was accompanied by practical experience, sound judgement and bold initiative.

Other protagonists of new technology were tolerated more sympathetically. Perhaps the forms were less outrageous even if precedents were absent. Burton's greenhouse at Kew and Paxton's work at Chatsworth and the results at the Crystal Palace are testimony to this. Stephenson's Britannia Bridge was much admired. (This itself has been ruined by the intervention of an unsympathetic technological repair.)

A point had been reached where, for many structures, the architecture was derived from the engineering.

Without the development of architectural forms relevant to new technology, the pendulum could swing even further from precedent but also away from the intuition Baker recognized towards the god of analytical methods. Shell Mex House, built in 1933, was described as architecture sacrificed on the altar of structural engineering, and the new wave of buildings as Portland Stone quarries hung on steel frames.

But even the epitome of advanced technology, the exploration of space, shows us that the world of precendent remains alive and well, as comparison of Russian and American man landing vehicles demonstrates. The Northrop Corporation 1966 lunar vehicle is straight out of Disneyland. 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 totality 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 to day 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 'cast' and the joints 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 rivetted together. Process and engineering are evident with ornamentation from 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.

An 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 sub-divided 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 area 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 required 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

1

2

Fig. 1 Coventry Trading Estate from the air. Architects: W.S. Hattrell & Partners

Fig. 2 Coventry Trading Estate: structure after fire which led to provision of new building

Fig. 3 Warehouse Alexander Duckham and Co. Ltd. Architects: Harper, Fairley Partnership

3

42mm diameter rod in Grade 50 steel with High strength friction 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 is 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

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 9580) and also for an awkward re-development 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.


4

Figs. 4-5
Factory for Adamswear
at Nuneaton

Fig. 6
Joseph Ash and Sons
tank production shop

Architects: for both projects:
Harper Fairley Partnership


5


6


7

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

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 bracing. 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.

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.

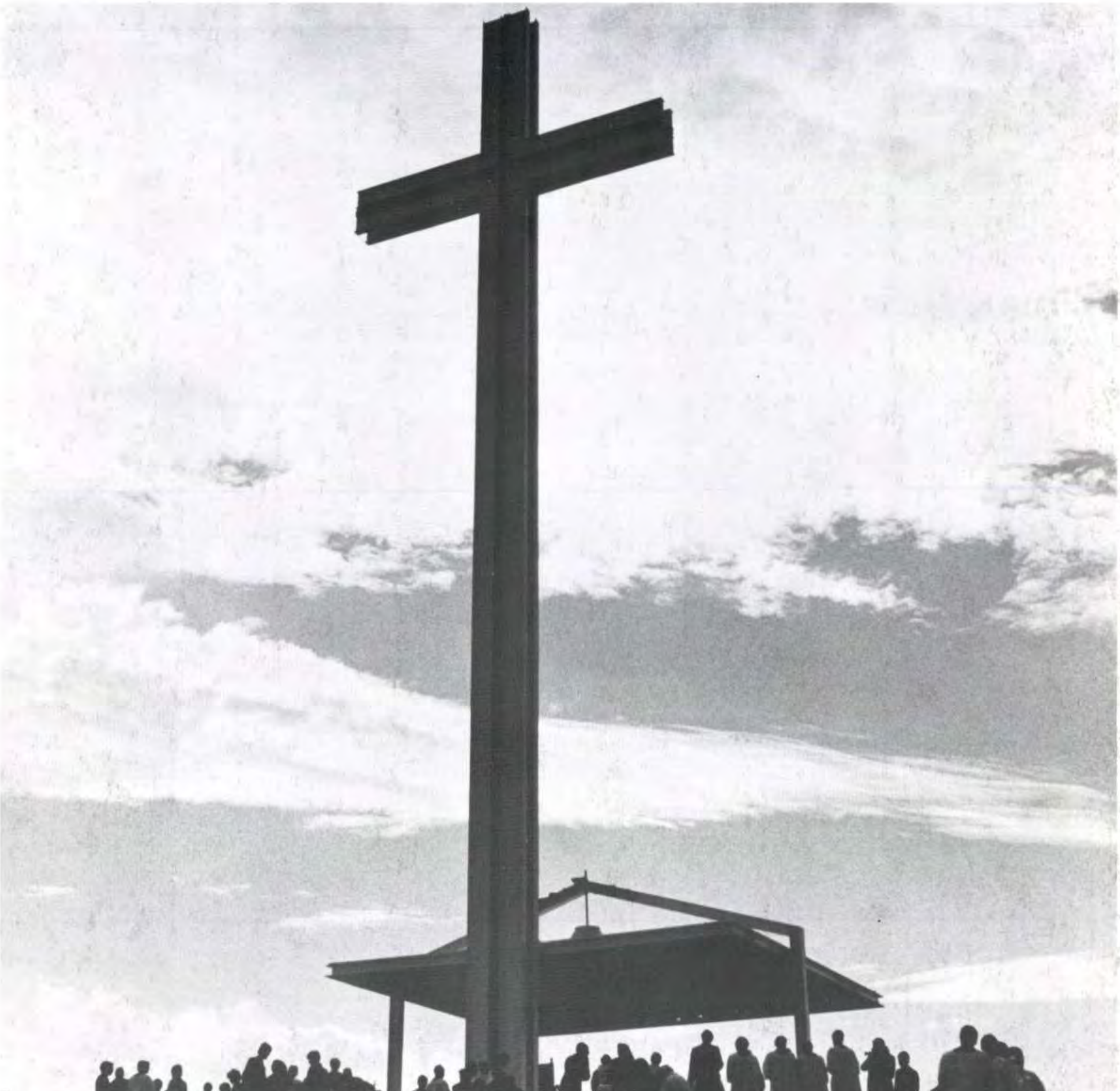
(3) Truss or Portal? We will usually examine both schemes but trusses more often offer the best solution.

(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 4.6 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.



Printed in England by Reedprint Limited, Windsor, Berkshire