THE ARUP JOURNAL

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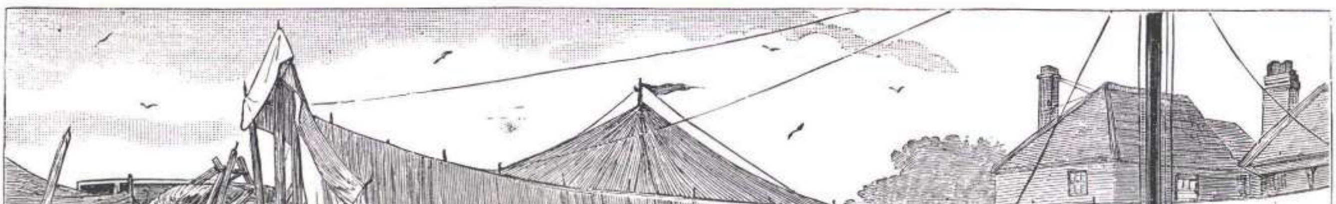

RAISING THE TENT

THE ARUP JOURNAL

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Contents

Front cover: Print of a circus tent being erected, circa 1880 (Reproduced by courtesy of Mary Evans Picture Library).
Back cover: Prestressed special cable nets from Frei Otto's book *Tensile Structures* Vol. 2

Ecology and the role of the engineer in the protection of the environment

Don Montague

I am starting this piece in the first person, to show quite clearly that what follows is a personal view of our situation and of what we should be doing about it. Many of my colleagues share some of the views I shall express, and some of them are too busy doing the sort of things I am writing about to have time to sit down and write about them themselves. I claim no great originality for what I have to say, but I believe it needs thinking out, writing down and sharing.

At an Arup Technical Staff Meeting in October 1974 I spoke on this topic, having prepared some lengthy notes on ecology and what it's all about. They are difficult to cut, but I will try.

Ecology I define as the study of organisms in relation to the environment of which they themselves are a part. This study may be from the point of view of one organism, man, an earthworm, an ape or whatever, or it may look at the whole, and seek to explain the inter-relationships between all the identifiable organisms in the ecosystem, without emphasis on one particular organism.

An ecosystem is simply a bounded space, defined and limited for the purposes of study or description. It might be an estaury, a whole river basin, the human skin, the arctic tundra, a forest compartment, Regent's Park or the site of a new brewery. It could be a wholly natural system—an uninhabited island, or a man-made one—a building such as an opera house.

Within an ecosystem, ecologists have defined what they call *trophic levels*, at which various resources exist and at which various *processes* may take place, in which *agents* act to transfer or transform resources into *products*. For example, at the first level the *resources* of air, water and parent rocks are acted upon by weather, plants and animals as *agents* in the *process* of pedogenesis or the production of soil.

In classical ecology the second level is vegetable and the third is animal, but some more sophisticated ecologists distinguish up to six levels.

An ecological study may start with identification of the resources and agents and then look at the processes going on within the ecosystem. These are then studied in terms of their inputs and

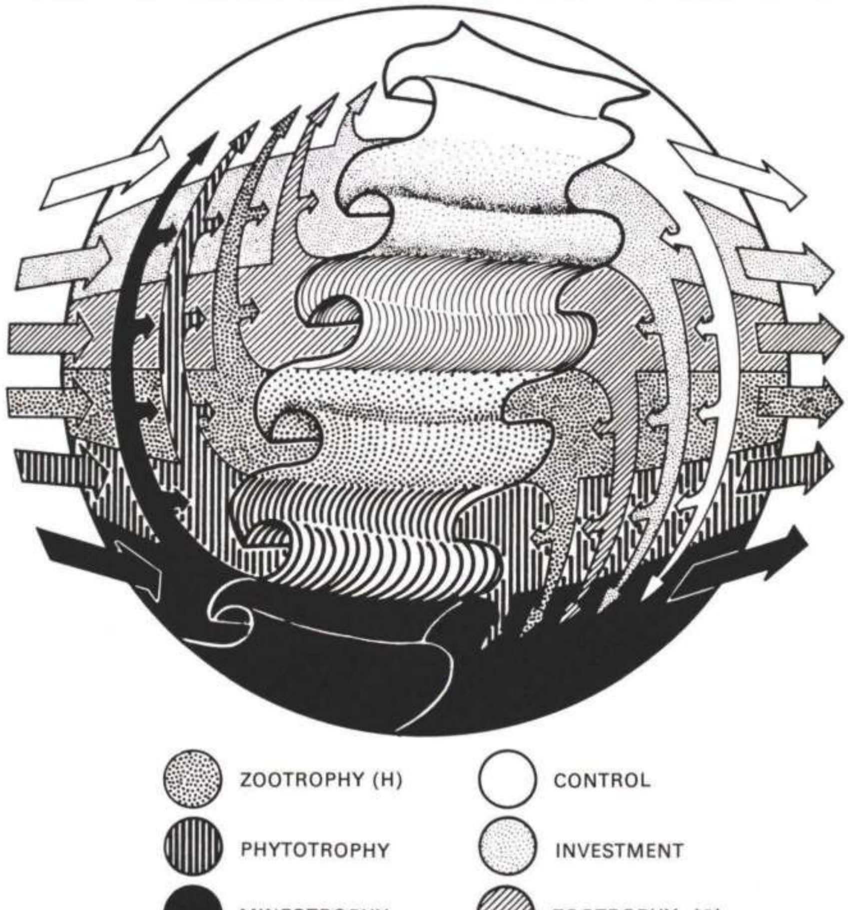


Fig. 1
A scheme showing a projection on six trophic levels (and their characteristic process) of the main-stream flow of energy (central part), the inner supply of resources (left part), and the reinvestments (right part), as well as the import (left margin) and export (right margin)

Trophic Level	Regime	Resource	Agent	Process	Product	Consumption or Utilization	
						Agent	Quantity
VI	Control	auxiliation information	plants, animals	migration, transportation, cleaning, engineering, processing	silt, organic matter, migrants	plants, animals	restricted
V	Investment	minerals, air, water, plant and animal organs	soil, plants, animals	flooding, propagation, reduction	population, fertilization, shelter, dams, etc.	soil, plants, animals	small
IV	Zootrophy	air, water, soil, animal tissues	carnivores	support, absorption, respiration, carnivory, digestion, excretion	flesh, chitin, muscle, bone, skin, etc.	parasites, animals	very large
III		air, water, soil plant tissues	herbivores	support, absorption, respiration, phytophagy, digestion, excretion			
II	Phytotrophy	soil; mineral and organic nutrients; substratum	plants	rooting, absorption, transpiration, nutrition	starch, sugar, fibre, tissue, fruit, bark, wood, etc.	parasites, animals	very large
		water		absorption, circulation, transpiration, nutrition, excretion			
		air: light heat oxygen carb. diox.		growth reproduction respiration			
		energy	green plants	photosynthesis			
		nitrogen	bacteria, algae	fixation			
I	Minerotrophy	air, water parent-rock	weather, plants, animals	pedogenesis	soil	plants	very large

Fig. 2
Resources, agents, processes, and products at six trophic levels in a wild landscape. All resources and agents are primaeval

outputs, scale and rates, time or cycles of occurrence, stability and efficiency.

For even the simplest system these basic studies can present tough problems of identification and measurement and require scientists of many disciplines and great ingenuity. In situ measurement of processes in real time will often be impossible, beyond simple factors like rainfall or silt deposition, or gross factors like increase in plant height. This is one of the great frustrations of ecologists, and one of the factors which make it extremely difficult for them to develop predictive models of ecosystems.

So, members of the ecological team may be able to tell one *why* certain things happen, why excessive use of nitrate fertilizers leads to lodging of wheat and excess nitrite levels in rivers for example, but they will not necessarily be able to help you if you want to know *what* will happen if you apply a given set of changes or stresses to the system. As an example, the effects of pollutants on sea creatures are extremely complex, and vary as the water temperature, the levels and combinations of pollutants and other factors change. The relationships between the sea creatures themselves will be rather complex, and if a combination of pollutants kills more than a certain proportion of the population of one creature, the population may crash, with chain reaction effects right through a food system. It's easy to say, but it takes more than a few months or years of study even to describe biological systems which have survived and developed over a few million years, let alone to understand them.

Another major problem of ecology is defining the quality of an ecosystem. Is high quality a function of stability, or productivity, or self-sufficiency, of diversity, or what? From the point of view of one organism quality is easier to define, even if one does not know how to achieve it. For example, from a human point of view an arable ecosystem might be an optimum one if it gave the maximum sustained yield of protein for the minimum fertilizer and energy inputs. On the other hand if you or I look at a largely untouched landscape a judgement of quality may be aesthetic.

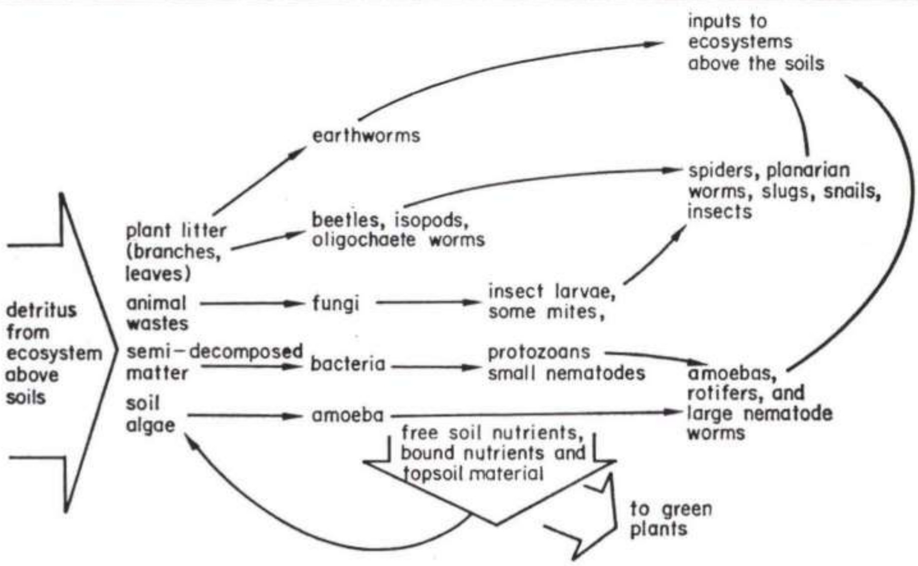


Fig. 3
A simplified soil ecosystem

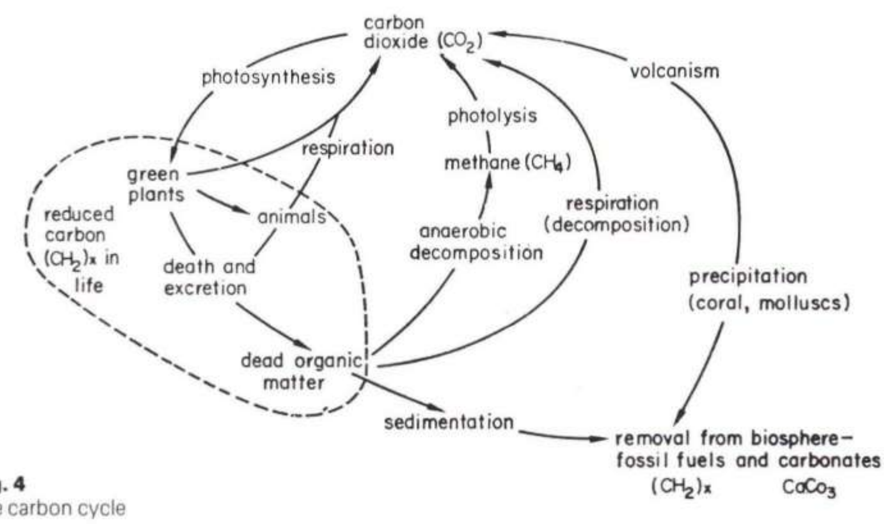


Fig. 4
The carbon cycle

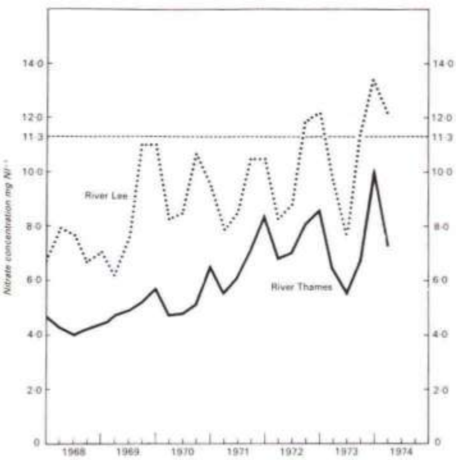


Fig. 5
Nitrate concentrations in River Lee
and River Thames: quarterly averages

I am not trying to blind you with science, or to convince you that ecology is bunk, as Henry Ford might have said if he had heard of the word. Ecosystems with biological elements are almost unimaginably complex, and we can only move towards understanding them with long-term, multi-disciplinary ecological studies. It is no use thinking that all one needs to do to ensure minimum environmental damage or maximum benefit, is to hire 'an ecologist' and put him to work.

There may be a place for an ecologist or for practitioners of some or many of the sciences which make up ecology in the project team for a major development. The Swansea Valley project is a classic example, where teams from universities and Government departments studied the grossly degraded and polluted land and water of the Lower Valley, and developed various means of reclaiming the land for a variety of uses. Within Arups an ecological approach has been used in jobs such as Projet Suroit and others, but I will leave it to my ecologically better qualified colleagues to write about these.

For our present purposes I want to list and illustrate three 'principles of ecology' which may be useful to us in our work as engineers. They are by no means original, one of them is merely the First Law of Thermodynamics in disguise, but I think they provide a useful link between the engineering and the ecological approach to problems. They are stolen from Barry Commoner's 'The closing circle', with salutations and apologies.

(a) Everything is connected to everything else: if one tinkers with one part of an ecosystem the effects will spread like ripples in a pond, ultimately affecting every other part of the system to a greater or lesser degree. There may be redundancies in the system, but don't bank on it; look for connections and beware of side, secondary and tertiary effects. The social, medical, climatic and other effects of the big dams in Africa have vividly illustrated the need for ecological understanding of connections and interdependencies.

(b) Everything has to go somewhere: the products or wastes of any process in an ecosystem must go somewhere, into store, to another process, or out of the system as exports. The nitrates poured onto fields as fertilizers and the sulphur in power-station coal may turn up again as nitrites in the Thames and sulphuric acid falling on the acid soils of southern Scandinavia.

(c) There is no such thing as a free lunch: or you can't get something for nothing. If one diverts any resource or organism from an ecosystem this will affect the system, possibly adversely, and one must be prepared to accept the bill for the effects. If you don't, somebody else may have to. If you take too much water out of a borehole the dry weather flow in a nearby stream may drop to a level where pollutants are insufficiently diluted. Local residents may have to pay through the nose.

Now it's about time I got down to the role of the engineer in the protection of the environment. Let's look back a little bit first and ask what 'the engineer' does. In ecological terms he is an organism in an ecosystem. He uses its resources, processes and transforms these, and changes the ecosystem to produce a result which will benefit his fellow organisms. While doing his thing he may use resources, processes and knowledge from within and outside the ecosystem. The changes he produces may be classifiable in one or more of the following ways:

(1) Extraction of a resource and export, e.g. mining

(2) Use of resources and processes to create new processes, e.g. building railways or factories

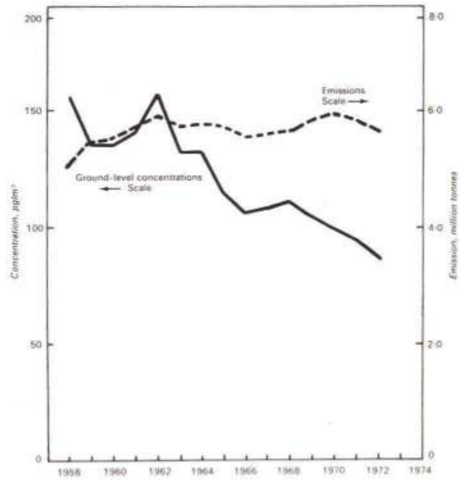


Fig. 6b
Sulphur dioxide emissions
and urban concentrations
in the United Kingdom

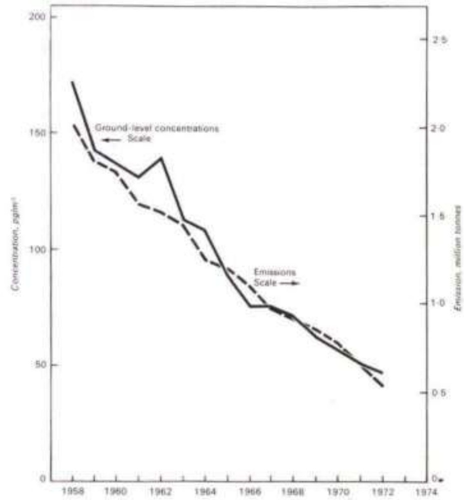
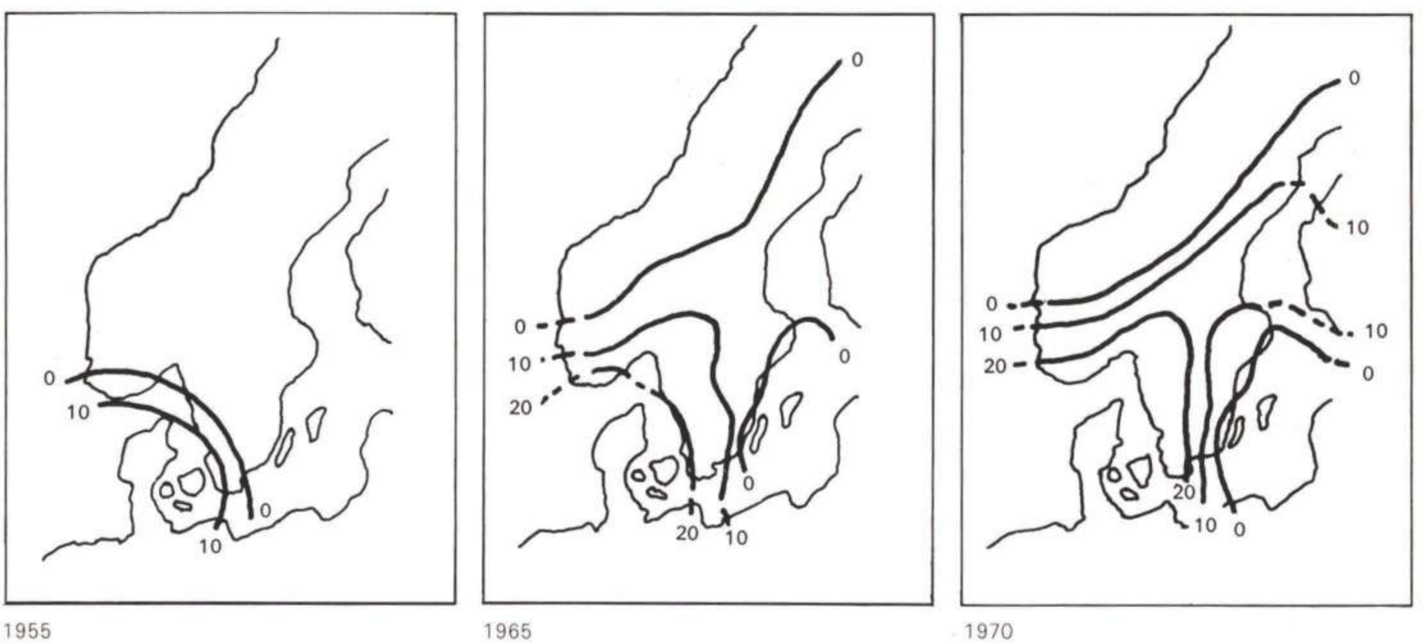


Fig. 6c
Smoke emissions
and urban concentrations
in the United Kingdom



1955

1965

1970

Fig. 6a
Deposition of excess acid through precipatation during one year, expressed in mg hydrogen ions/m²

(3) Transforming resources to produce others, e.g. electricity generation
(4) Control of a process to increase its efficiency, e.g. damming and irrigation
(5) Use of resources and processes to improve or change conditions for the functioning of organisms, e.g. office or house building
And so on.

Basically, we act on behalf of man, and using any resources we can tap, we alter ecosystems to satisfy man's demands. Man has demanded water and then more land for growing food, so we developed irrigation systems several thousands of years ago, and later made machines to clear land, to build dams to hold back the sea and to drain the enclosed land. Then engineers developed powered tools to till land and harvest crops, and equipment to process, store and distribute food.

Coming out of his cave and moving away from the equator, man needed shelter and engineers built it. He needed warmth and engineers developed means of providing it in many forms. He gathered his sisters and his cousins and his aunts and others together in settlements, and eventually the public health engineer was invented to remove his wastes. The canal, railway and highways and bridges and automobile engineers invented themselves, to collect food for people living in social environments called cities, and to make it possible for the goods produced in the cities to be moved back to the countryside or to other cities.

Recently, having satisfied the basic needs of a larger and larger proportion of Western men, engineers have been asked to undertake other tasks, most of them with limited 'survival value' such as making machines for carrying out very simple thought processes very fast, for peering at the surface of planets, and for moving David Frost to and fro between London and New York several times a week in less time than it takes the dawn to traverse the Atlantic. Clever chaps, we engineers.

In Arups we are hardly out on the lunatic fringe of engineering, though some of our best structures are unrelated to any basic human biological need. In fact, some of the things we have designed can fairly be said to serve some of the highest and most noble needs of man. But, like other engineers, we generally do what we are told, we play the tune. Like other engineers, we take what materials are available, and use them as cleverly as we can within the limits of our clients' purses. We don't worry too much about where the materials come from or what our clients may do with the buildings, except in as much as they affect the construction costs or the design brief. We rarely, if ever, write the client an operating manual about how to get the best out of the building, and the only times we go and look at it after it's finished are if we are pleased with it, want to show it off, or if it has fallen down unexpectedly.

Every single building we design or development we work on has some environmental effects. From one point of view they almost invariably increase the quality of at least part of the ecosystem for man. Our buildings may improve or destroy parts of the environment for other organisms and affect balances and cycles within the local ecosystem. They also usually draw resources from other ecosystems, and discharge wastes into them. Where these secondary effects have been shown by experience to have possible harmful effects on man, we find that man has set up regulations, from the Clean Air Acts of 17th century London through the Town and Country Planning and Public Health Acts to the Building Regulations 1972, to control location, design and use of buildings.

As Professor Darlington said recently, '. . . every invention of man has made his environment more favourable for his short term multiplication'. As engineers we have played our part and will continue to do so. Darlington went on, however, to say that these inventions 'have

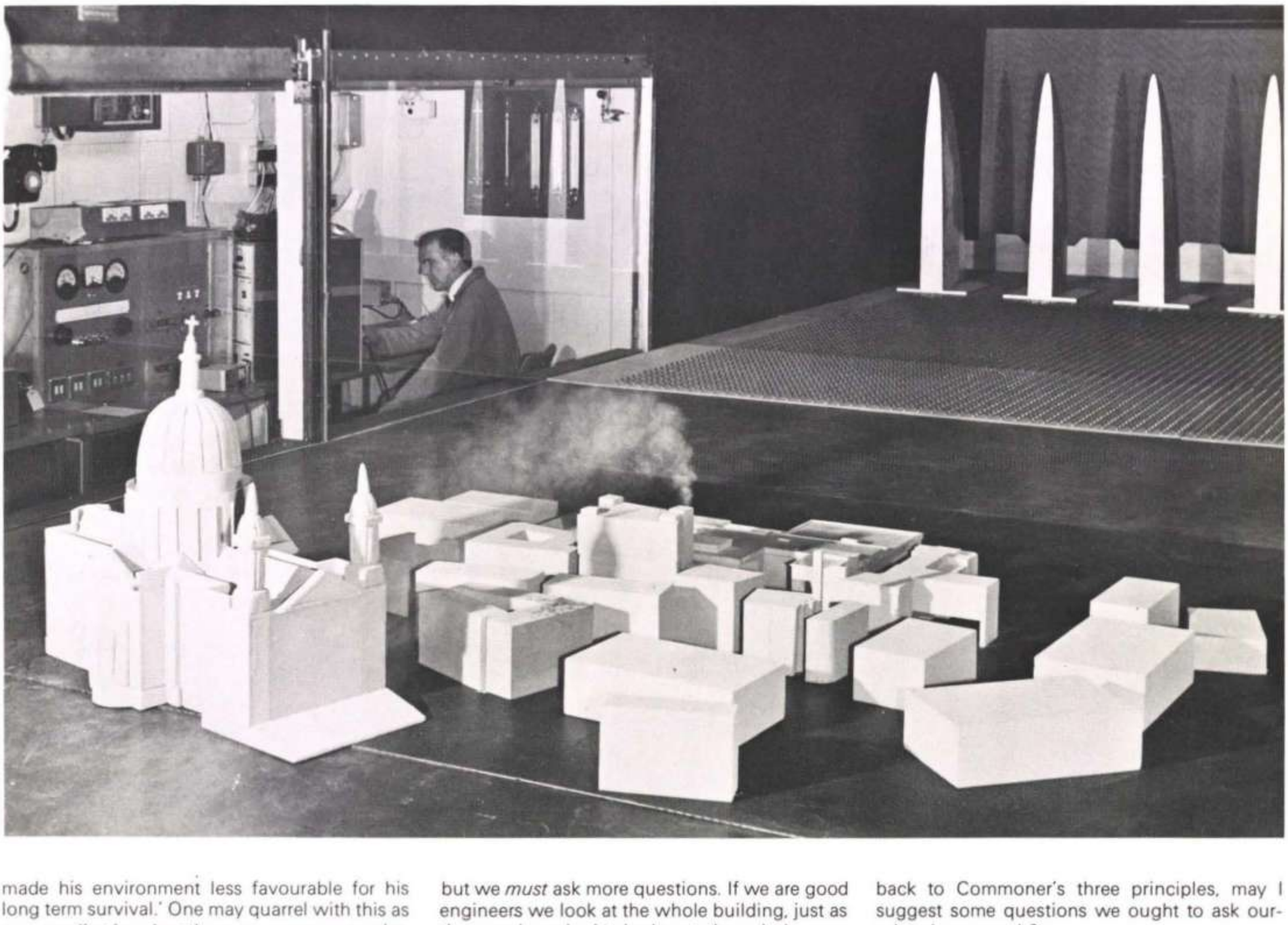
Type of pollution	Govt. dept.	Division	Responsibilities
Air pollution	DOE	HM Alkali and Clean Air Inspectorate	Air pollution from registered works.† Advice to local authorities
	SDD	HM Industrial Pollution Inspectorate for Scotland	For air pollution has the same responsibilities as the Akali Inspectorate, but also has some additional responsibilities as described below
	DOE	Directorate of Vehicle Engineering and Inspection	Air pollution (smoke) from road vehicles
	DI	Warren Spring Laboratory (Air Pollution Division)	Co-ordinating centre for the National Survey of Air Pollution.
Freshwater pollution	DOE	Directorate General Water Engineering (DGWE)	Overseeing of sewage disposal schemes. Technical advice to local and water authorities
	SDD	Engineering Division	
	SDD	HM Industrial Pollution Inspectorate for Scotland	Advice on the control of water pollution
Pollution from agricultural chemicals	MAFF, DAFS	Safety, Pesticides and Infestation Divisions	Control of pesticides and advice on disposal of farm wastes
Marine pollution	MAFF, DAFS	Fisheries Division	Discharges at sea outside territorial waters
	DOT	Marine Division	Oil pollution at sea
	D of Energy	Petroleum Production Inspectorate	Pollution from drilling operations
Toxic waste disposal	DOE	DGWE Toxic Wastes Division	Advice on methods of disposal of toxic wastes, monitoring and collation
	SDD	HM Industrial Pollution Inspectorate for Scotland	Advice on the disposal of toxic wastes
Refuse disposal	DOE	DGWE Public Cleansing Division	Advice on methods of refuse disposal, refuse collection and street cleaning
	SDD	Engineering Division	
Noise	DOE	Directorate of Vehicle Engineering and Inspection	Noise from road vehicles
	DOT	Civil Aviation Division	Aircraft noise

DAFS: Department of Agriculture and Fisheries for Scotland
DI: Department of Industry
DOE: Department of the Environment
DOT: Department of Trade
MAFF: Ministry of Agriculture, Fisheries and Food
SDD: Scottish Development Department

Fig. 7
Pollution control responsibilities of Government departments

Fig. 8
A flash or pond caused by mining subsidence in the Midland countryside. (Photo: Nan Fairbrother)

Fig. 9
Test on dispersion of pollutants in a built-up area being conducted in a wind-tunnel at the Warren Spring Laboratory (Crown copyright)



made his environment less favourable for his long term survival.' One may quarrel with this as a generalization, but there are many examples, starting with the irrigation works of the Mesopotamians which produced rich harvests and gradually rising water tables, until in the end salinity and lack of maintenance led to the collapse of the whole system. There could be a parallel with the nuclear power station building programme, unless we develop infalliable methods of dealing with redundant reactors and radioactive wastes.

As civil and building engineers we are consuming resources and may be modifying environments on a large scale. I think we must ask a lot more questions about the effects of what we are doing, not necessarily publicly or to our clients, but we *must* ask more questions. If we are good engineers we look at the whole building, just as the good ecologist looks at the whole ecosystem. As engineers we try to design something which is technically, economically and aesthetically whole and satisfying, which enhances rather than degrades the environment. We're not ecologists, and we shouldn't pretend to be, but I think we can do a better job if we use where we can some of the knowledge and principles which ecologists and their helpers in other disciplines are gradually piecing together.

I don't think we do always ask all the questions we ought, not simply because we don't know the answers or what to do with them, but more often because our background and training simply doesn't include such things. Coming back to Commoner's three principles, may I suggest some questions we ought to ask ourselves in our work?

Everything is connected to everything else

If we work in this country possible harmful connections or interactions between our buildings and their environment are largely regulated by statute. In many countries overseas they are not, or not as comprehensively. Have we drawn up and used our own check lists for the connections not covered by statute in this country? Have we any plans to do so for projects overseas? Or to work out Codes of Practice for general cases or specific jobs? Are we using the services of local people overseas to assess the possible effects of our work? When commissioned to work overseas do we ever recommend to our client the ecological study of the area to be developed?

Some of the answers are 'yes', some of the time, but I suspect we could do more. On another occasion I'd like to put to you some of the pros and cons of environmental impact statements of the sort required by the EPA (Environmental Protection Agency) in the USA for all Federally aided projects.

Everything has to go somewhere

What goes into a building comes out sooner or later. We know where the sewage goes, and the surface water. We generally assume the solid waste will be collected, and in Britain the Clean Air Acts regulate to a degree particulate smoke emissions, but where does the sewage with all its chemical and biological potential actually go? What happens to the surface water, could it be re-used? What happens to solid waste? Should it be segregated into different categories for recycling, recovery, burning or tipping? What happens to the gaseous emissions from the building? Where does the waste heat go? Should we advise clients how to manage emissions, from rain-water to people going home, to minimize adverse effects on the local systems from drains to roads? Or do we just give them a building, like a car with no handbook or chauffeur, but more expensive?

There is no such thing as a free lunch

Everything used in constructing, running or using a building comes from somewhere, and costs somebody or some ecosystem something sooner or later. We *do* ask the Water Authority if they can supply our developments, but do we ask them at what cost, to whom, in economic let alone ecological terms? Limestone aggregate for road base: do lovers of England's National Parks among you know that limestone aggregate production increased from 11 to 62 m. tons between 1955 and 1972. Most of that comes from the hills in those very Parks. Gravel extraction in the Thames valley may give opportunities for water sports and nature reserves, but at the rate we're digging we may soon have a surplus of these. How often do we seriously try to economize, beyond the exigencies of the cost plan? Use two-stage flushing or smaller lavatory cisterns or single spray taps? With water rates in Britain based on rentable values of buildings there is little financial incentive to economize.

Coming back to energy we should, I think, beware of both North Sea Euphoria and Doomwatcher's Deafness. An age of plentiful energy is not about to dawn, not now or in 1980 or 1984. Nor are we going to run out. But energy will be relatively dearer and scarcer in some forms in the future. The energy trade is international, and other people's demands or expectations or aspirations to attain our profligate levels of energy use will keep prices high. The more energy from non-renewable sources which are used, the deeper the open cast miners will burrow, the more man-made landscape and undisturbed ecosystems will be invaded by the energy miners and producers and the more radio-active ruins we will leave for our descendants to look after. There is no such thing as a free lunch.

Ask yourself, have you been able to get into any project early enough and well-armed enough to really influence its basic shape, form, materials, construction, lighting or internal climates in the direction of energy economy? Have you ever tried to persuade your client right at the beginning that extra fees for analyzing his annual energy costs by computer may be money very well spent?

Have you ever, in a Structures only job, sought to develop or apply knowledge of thermal inertia and other properties of structural materials affecting heat losses of fuel consumption? Again you Structures men, if you have done this, have you developed relationships with your Mechanical and Electrical consultants to try to exert a concerted influence on the thermal properties of the buildings at an early stage?

Estimates of reserves (in 1,000 million barrels)

	Published Proved	'Most likely' (Birks)	Possible (Birks)	Speculative (Odell)
	Whole North Sea			
Commercial	10.0	25.0	39.0	
Sub-commercial	2.0	4.0	8.5	
Total	12.0	29.0	47.5	100
	UK Sector			
Commercial	6.5	16.5	26.0	
Sub-commercial	1.0	3.0	5.5	
Total	7.5	19.5	31.5	66.0

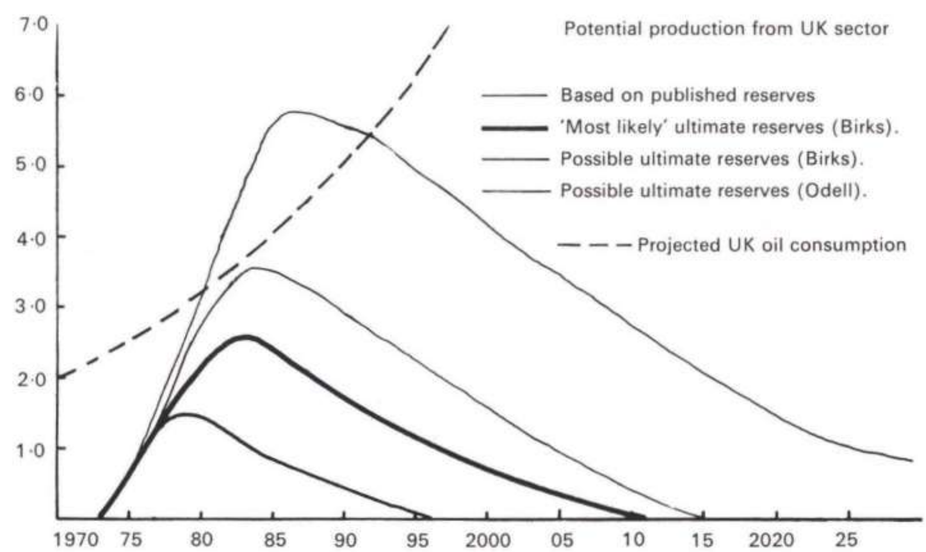


Fig. 10
UK oil production

How many Heating and Ventilating engineers can say that they have thoroughly investigated any unconventional heat sources on any recent job? Some can, I know, but how many schemes based on low grade heat from power stations or using heat pumps have we looked at? How many buildings have you handed over with a Heat Economy Manual?

Under the title I started with and in the space allowed it has not been possible to discuss the political and sociological aspects of our role in changing the environment. We cannot ignore these, and each of us should try to understand and weigh their implications, to sort out and refine a personal attitude. Among my colleagues I know several loquacious enough and well qualified, and I hope they will take up these topics in a future *Arup Journal*.

As I said at the beginning of this article, this is a personal view. If you have read as far as this, perhaps I have written something useful. Bear with me while I try to summarize for those who have skipped through to this point.

We can learn from the methods of ecologists, but we should not pretend to be ecologists ourselves. We need a personal commitment to asking more questions about the effects of our work. Working overseas we must be more sensitive to enviromental effects than regulations demand. Working in the developed countries we must design for tomorrow, not just for today. Above all, we design for a finite world with limited resources, of which everyone wants a share.

Illustration acknowledgements

Figs 1 & 2: Reproduced from *Human ecology in the Commonwealth*, the report of the First Commonwealth Conference on Development and Human Ecology, Malta, 1970, pp. 6 and 7.

Figs 3 & 4: Reproduced from *The oxygen revolution*, by Geoffrey Mains, David & Charles, 1972, pp. 51 and 128.

Figs 5, 6b, 6c, 8 & 9: Reproduced from *Royal commission on environmental pollution, Fourth report: pollution control progress and problems*, HMSO, 1974, pp. 12, 13, 37, 59-62 & Plate 3a, by permission of The Controller.

Fig 6a: Reproduced from *The Ecologist*, October 1973, p. 378.

Fig. 7: Reproduced from *New lives, new landscapes* by Nan Fairbrother, Architectural Press, 1970, p. 98.

Fig 10: Reproduced from the *New Civil Engineer*, special feature on North Sea oil, 1973, p. 4.

Keeping it quiet: The Wellington Hospital

Tony Stevens

In early 1969 we received an enquiry from the estate surveyor's office of Cayzer Irvine Ltd., the shipping firm. They had found a plot in St. John's Wood, London, and required a feasibility study for the development of the site. The owners were British Rail and the site was a partially disused railway cutting about 10 m deep on the line from Marylebone immediately to the north of Lords cricket ground. Two main lines of railway had to remain but British Rail proposed to sell most of the land in the cutting and the rights of development, under certain conditions, above the running tracks.

Our report was submitted in January 1969. It said, among other things, that in addition to the two tracks on the site there were, immediately to the east, four more—two Bakerloo and two Metropolitan lines in tunnels under the road (Fig. 1). Moreover the traffic in Wellington Road, which the new building would face, was heavy and noisy. We had not at that time been told the purpose of the project, but to be on the safe side we warned that the building would be noisy and subject to some vibration unless precautions were taken. We suggested that, to minimize costs, buildings should be located on the western side of the site as far as possible from the railways.

Importance of noise insulation

Our report was accepted and the client decided to proceed. We were involved in assisting him to negotiate the purchase of the site. We learned that the development was to be a private hospital but at that point our involvement ceased. It was not until late 1969 that we learnt that Yorke Rosenberg Mardall had been appointed architects for the project, Steensen, Varming, Mulcahy were to be the mechanical and electrical engineers, and that we had ourselves been commissioned for the structural work.

It was then that we found that our new client had taken our warning about noise and vibration very much to heart. The standards in the hospital were to be of the best and those for noise and vibration no exception.

At the inaugural briefing, Sir Nicholas Cayzer, looking us straight in the eyes, laid particular emphasis on the importance to the success of the project of effective insulation against noise and vibration in the private wards and on what he considered to be our responsibility in that field. In fact, the responsibility for achieving this high standard of protection against noise within the hospital was split three ways. Yorke Rosenberg Mardall were responsible for the attenuation of air-borne noise likely to enter through the building fabric, and Steensen, Varming, Mulcahy for limiting the noise generated by the services installation. We were to be responsible for eliminating vibrations which might otherwise come from the ground, pass through the structure and disturb patients.

We decided we needed some good advice. We suggested to the client that Grootenhuis Allaway Associates should be appointed and this was done. Professor Peter Grootenhuis holds the chair of Mechanical Engineering Science at Imperial College, London and specializes in vibration. Peter Allaway, his associate, is an expert in the complementary field of noise attenuation.

Their first task was a survey of noise and vibration on the site, which they carried out in early 1970. Lack of mains power restricted readings of vibration to those which could be taken by a battery-powered accelerometer. Peak accelerations due to British Rail, London Transport Executive (LTE) and road traffic were measured and found to be between 0.001 g and 0.006 g (g=gravitational acceleration). Close to the east wall the LTE trains produced vibrations almost as great as the BR diesel car sets. Noise due to the BR rail traffic was measured at the equivalent of 95dBA for one train. Two trains passing in a reverberant tunnel gave a reading at about 110dBA.

Acceleration measured on the ground would be more useful if associated with given bands of frequency of vibration. Professor Grootenhuis was able to supplement the site measurements by readings taken over the tunnel in the US School, a building under construction further to the north. Mains power was available there and it was possible to measure acceleration at known frequencies. The noise generated in the tunnel was confirmed in the ventilation shaft at the school to be about 110dBA.

Meanwhile the architects had discovered that it was not possible to meet the brief without building over the whole site, including the railway. Because of the light angles the new hospital had taken on its now familiar wedge shape (Fig. 2). But before the design of the building could proceed very far, we had to decide what noise and vibration measurements would be needed.

We received a report on the survey in April 1970 and in close consultation with Grootenhuis Allaway Associates, we set about deciding what our proposals ought to be. First we had to consider the standards for which we should aim.

The noise standard we employed was that likely eventually to be adopted by the International Standards Organization. It is a dB scale weighted across the frequency range to allow for the non-linear sensitivity of the human ear to sound pressure and was that developed by Kosten and van Os. The scale is given in noise rating numbers NR. NR zero is the threshold of audibility. Kosten and van Os suggested other standards which they considered appropriate:

	Maximum NR
Concert hall	20
Television studio, classrooms and bedrooms	25
Hospital, church, courtroom, library	30
Private offices	40

We decided that NR25 would probably be adequate but recognized that at that level a keen ear would perceive a very slight rumble from the trains when all else was extremely quiet. Since it was specifically required that the occupants of the hospital should not be conscious of the railway, we decided to aim for NR20, though we knew that such a standard would be extremely difficult to achieve in the services system and against traffic noise.

The standard of vibration required in the building was clear—the occupants must not feel tremors from the railway. We used the Dieckmann scale, which classifies vibrations according to certain ratings which it calls K (see *BRS Digest No. 117*). We aimed for ratings, K, below the lower level of perception, i.e. K=0.1. By means of this scale we were able to determine the maximum amplitude of vibration which could be allowed to occur in the structure.

Conditions

Next, we examined the conditions which would prevail if we took no precautions for attenuation other than to confine the railway in a heavy reinforced concrete tunnel to limit the air-borne noise. We had to assume that at least some of the structure would be resonant with the source of vibration in the foundations and that consequently a K value of about 2 might reasonably be expected in the superstructure. Noise levels in chambers due to vibrations passing through surrounding structure are notoriously difficult to predict but we considered that without attenuation, the level in the hospital would be about NR40. Thus we concluded that if no insulation were provided to break the path of vibration through the structure, the required standards would not be achieved.

After that, there was not very much choice. We decided that the insulating break should take the form of resilient bearings on which the whole of the building containing the working space for the hospital would be mounted. These bearings would be located in a void which in the following is referred to as the pad void. The chosen location of the insulating layer (Fig. 1) had several advantages: it was fairly simple to construct; it isolated the services plantroom with the railway from the remainder of the building and the double structure pad void assisted in the air-borne noise attenuation.

We had difficulties with the stair and lift shafts. Two shafts at the north and south are not mounted on resilient bearings, but it was possible to separate them completely from the protected structure of the hospital. The central lift shafts could not be discontinuous at the lower

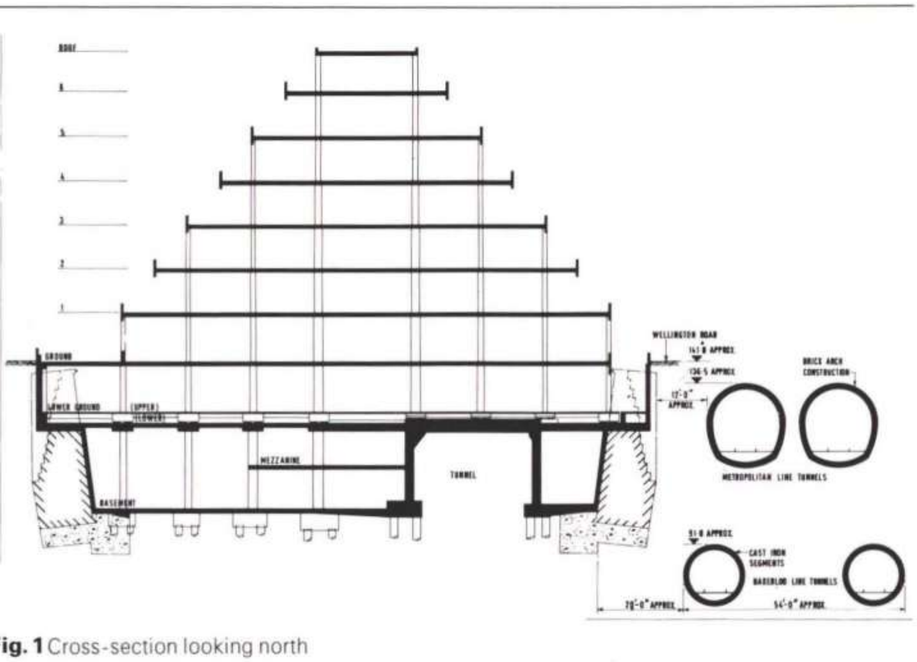


Fig. 1 Cross-section looking north

Fig. 2
The completed Wellington Hospital (Photo: John Laing & Son Ltd)

ground level and could not be easily separated from the protected structure. We decided to locate the bearings to these shafts at the lowest basement level and to separate them carefully from the surrounding basement structure.

Thus our porposals combined the air-borne noise protection of the pad-void and the heavy concrete slabs and walls below lower ground level with resilient bearings to limit the structure-borne vibration.

Now we had to decide on the properties required in the resilient bearings. Here you might say we worked backwards. At that time the market in vibration insulation was not so well developed as it is today. We were particularly worried about the stability and creep characteristics of the bearings that we were to choose. Grootenhuis Allaway Associates advised that, of the bearings available, those likely to be both suitable and which had been proved reliable in service were rubber bearings designed to support bridges. We could guess that their natural frequency of vibration at the loads they would carry could not be much less than about 10Hz (10 cycles per second). The question was, would that be good enough?

The rubber bearing pads could not be expected to provide all the energy attenuation needed to reduce NR115 to NR20. The structure in between the source and the protected chambers would have to contribute. Peter Grootenhuis felt intuitively that bearings providing a natural frequency of vibration of about 10Hz would be adequate but we needed argument in support. The first of these depended on our knowing the vibration in critical surfaces at the Royal Festival Hall corresponding to NR25 in the hall when trains were passing in the tunnels below. Professor Grootenhuis was able to correct the readings at the Royal Festival Hall to account for the difference between NR25 and NR20 and for the fact that the dominant frequency at Wellington Road was 40Hz and thereby to derive a maximum surface vibration allowable in the superstructure. We could thus find the

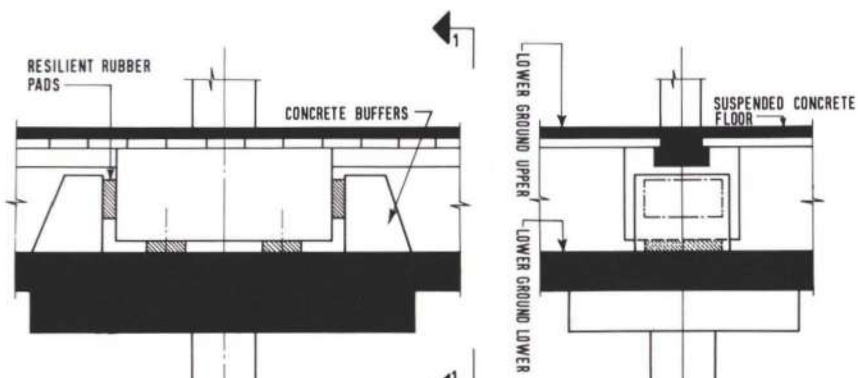


Fig. 3
Typical detail of resilient pads

vibration transmission reduction that the hospital bearings would have to provide in order to achieve the required noise level; 10Hz bearings just qualified.

The second argument employed measurements taken at the US School in a chamber which happened to be resonant to the noise coming through the ground from the railway tunnel. The level had been read at NR40. We argued that the resonant chamber gave the worst situation possible and could be assumed analagous to chambers at the hospital. Thus the bearings needed to reduce the energy of transmission by known amounts over a range of frequencies to reduce NR40 to NR20. On this account 10Hz bearings were more than satisfactory.

Finally we questioned whether the resilient bearing could achieve the vibration standards. We wanted to take account of possible dynamic magnification of the source vibration by the structure in the protected building, especially the floors. We estimated the level of transmission reduction the bearings would have to provide to allow for such dynamic magnification and yet maintain the standard in the hospital at less than K=0.1. Once again, bearings at 10Hz just met the requirements.

Thus in October 1970 we reported to the architect and client that the standards for noise and vibration required would be achieved, with some qualifications, at a cost of about £30,000 by the implementation of our proposals. We recommended that the bearings should be supplied by the Andre Rubber Company Ltd. An important qualification was that if the spectrum of noise and vibration were to change, for example, due to a change in rolling stock or speed of operation, then there would be no guarantee that the particular system of protection we proposed would work so well or at all.

We also advised that the dynamic characteristics of the bearings under load should be established in a programme of testing.

Our report was accepted. We were authorized to proceed with the design and with the testing. Our original proposals for each column foundation was two bearings inclined downwards at the middle in order, so we thought, to limit horizontal movement. But the bridge bearings had very low lateral stiffness since of course they were designed for use at expansion joints. We soon abandoned our first attempts in favour of two separate sets of resilient bearings, one horizontal to support the building, and one vertical to prevent lateral movement. Our design for each column footing is shown diagrammatically in Fig. 3.

We had therefore to design two dynamic systems. The bearings laid horizontally were to have a natural frequency of vibration of 10Hz. The system of vertical bearings had to have a frequency below the range which would be affected by railway vibrations and above that which might easily respond to wind and other disturbing cyclic forces. We chose 2.5Hz.

There were three further considerations. Firstly, fire resistance for the bearings was provided by the sealed envelope to the pad void. Secondly at the invitation of the District Surveyor, we devised a system by which the columns could safely settle onto a concrete block, should a bearing fail. And, thirdly, we designed a sequence of operations using flat jacks by which a failed bearing might be replaced.

Specification

This was the basis for a Material and Performance Specification for the rubber bearing pads which we wrote in March 1971. The required frequency of vibration of the bearings under a given range of load was stated and the supplier was asked to make proposals which he believed would meet these requirements. There was an extensive specification for the testing of materials including the sampling of rubber in the completed bearings. Each bearing was to be statically load-tested and had to meet given standards of performance within stated tolerances. In short, every reasonably inexpensive test we could devise for the material and the mechanical properties was called for so that the chances of our including a defective bearing in the works was reduced to the minimum.

The specification was directed exclusively at the products of the Andre Rubber Company, of course, and constituted a very effective basis for the contract. They adhered closely to the testing programme which we observed. Only one bearing was rejected because of mechanical damage and that was replaced at works. We specified a number of spare bearings for use in an emergency but they were not required during construction and I hope they never will be.

We knew that we could not expect assurances of the life of each bearing. Our report had noted that although their life expectancy was probably more than 25 years and might be considerably more, no-one would guarantee it. In the specification we stated that the building had an expected life of not less than 75 years over which period the bearings would be required to maintain their resilience. We understand that the Board of the Andre Rubber Company decided that this requirement called for no qualification to their offer.

The enquiry to Andre Rubber had referred in the performance specification to the tests to be carried out on a typical bearing at the CEBTP (Centre Expérimental de Recherche et d'Etudes du Bâtiment et des Travaux Publics) Establishment in Paris. This is one of the few places that Professor Grootenhuis had discovered which had equipment to apply small amplitude vibrations at low frequencies to test pieces under heavy load. Andre Rubber supplied the bearings—not, unfortunately, one of the 610 mm×406 mm, 150 tonne capacity bearings we were going to use, they were too big—but a small one about 300×300 mm. The client paid for the testing and expenses.

The results were very interesting. Up to that time Andre had designed their bearings for the required dynamic performance using the static stiffness in compression modified by a factor, about 1.2, to convert to the supposed dynamic stiffness. It transpired that this figure had been established in tests on bearings in which fairly high amplitudes were induced. The Paris tests showed that at very small amplitudes such as would occur under the hospital, the dynamic stiffness was nearer twice that of the static.

The results came in time for Andre Rubber to modify their designs. Bearings which had been 77 mm thick before now became 108 mm thick. Happily, the differences in cost, and programme of manufacture and the implications for the structural design, were negligible. For our part we were satisfied that our advice on the need for tests had proved worthwhile.

The manufacture of the bearings began soon afterwards in mid-1971, just in time to supply the first bearings to site for the foundations to the lift shafts at low levels. All the bearings were on site by the end of 1971 and the lower ground floor was complete, including bearings, by early 1972. The bearings were bedded in epoxy cement mortar with close tolerances of level (Fig. 4). Precast column foundations were jacked down on top (Fig. 5), again bedded on epoxy cement mortar, all under the eye of the resident engineer, Elizabeth Mount as she then was.

Measurement of compression

We made arrangements to measure the compression of some of the pads as the construction of the building proceeded. For this reason and others, Elizabeth spent quite a bit of her time on site crawling about in the pad voids. The time she spent making sure that the pads were well bedded and that all the insulation gaps were clear, contributed significantly to a successful outcome. When the building was complete and most of the load applied, the compression of the bearings was more or less as we expected, i.e. about 8mm. This figure allows for some creep deformation.

There was not much point in measuring the noise until the building was nearly finished with all the sound insulation installed. But after the bearings were fully loaded, we could check the vibrations. There was an isolation gap in the lower ground floor, flanked by protected and unprotected structure. In December 1973 we held our breath and Peter Grootenhuis put the accelerometer first on one side and then on the other whilst trains passed in the tunnel close by.

Fig. 4
Bedding in the bearings
(Photo: Mark Gerson)

Fig. 5
Precast column foundations being jacked down on top of bearings.
(Photo: Mark Gerson)

Unprotected reading; 0.01 g; protected reading: slight flicker on the needle. Happy smiles all round.

Then last year in February just before the hospital was due to be opened, Grootenhuis Allaway Associates completed a formal survey in the hospital of noise and vibration due to the railways. In some ways it was not as perfect as I had hoped. In the Operating Theatres on the lower ground floor the noise reading was NR25 peaking to NR30 over very short periods. Vibration was 0.15 on the Dieckmann scale but still imperceptible. Elsewhere in the wards and other rooms above the ground floor, the noise and vibration due to the railway could not be detected above the background. It was certainly better than the standards we had aimed for. We tried to discover if the high values in the lower ground floor could, for example, be explained by a local short circuit across the pad void in the services system, but soon found that a detailed investigation was out of the question on the grounds of time and money. All we could really say was that it worked.

But was it worth it? I had my suspicions that no-one would have noticed much if we had left the bearings out. But I heard the nurses comparing conditions in the hospital with those in their hostel further north along the Wellington Road and over the railway tunnel. In the hostel apparently, there were rumbles and tremors from the railway; in the hospital—nothing. Now I assume that if the nurses appreciate the difference, then the patients will, although of course, they may never know it.

The Lightweight Structures Laboratory

Michael Dickson

October 1, 1974, marked the first birthday of the Lightweight Structures Laboratory of Ove Arup & Partners.

Two years ago the partners of the structural divisions decided that this small group should be set up to expand and develop the firm's activities in a field which offers alternative forms of architectural expression, is economic in covering of large roof spans and can show substantial savings in the consumption of scarce materials. The laboratory was to concern itself especially with cable structures, air supported and air inflated structures, and stressed membrane tents and to develop the firm's basic core of knowledge in this field, provide design advice to the structural divisions and their clients and to carry out its own design commisions.

However, this is a field where, as was found when designing the hanging cable roofs for the Conference Centre at Mecca, it is not easy to develop a method of design that enables the portrayal and understanding of the complex hanging or inflated shapes either in architectural terms or in specifically technological terms. The experience of Structures 3's association with Professor Frei Otto and Büro Gutbrod during the design of the Mecca roofs (see Figs. 1 and 2) was enormously fruitful to us in establishing a number of specific aims and providing us with a basis on which to found the laboratory's development. Other design projects in collaboration with Professor Otto followed; work on the competition for a new Olympic Stadium in Berlin (Fig. 3), the projects for a City in the Arctic (Fig. 4), Shadow in the Desert (Fig. 5) and more recently the exacting design work of the Mannheim timber grid shells in association with Büro Mutschler.

From these early beginnings Professor Otto became the firm's consultant in lightweight structures and he and Ted Happold are now leaders of the Lightweight Structures Laboratory.

The laboratory is on the second floor of Ove Arup & Partners' Soho Square offices. It has a small model making facility and a specialised

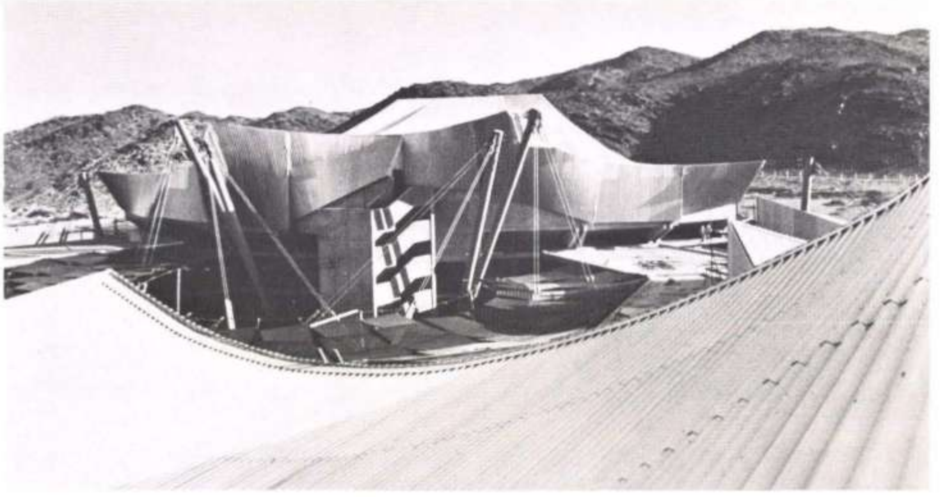


Fig. 1
The main auditorium at Mecca
Architect: Büro Gutbrod with Atelier Warmbronn
(Photo: Thinet)

Fig. 2
Inside the Mecca auditorium

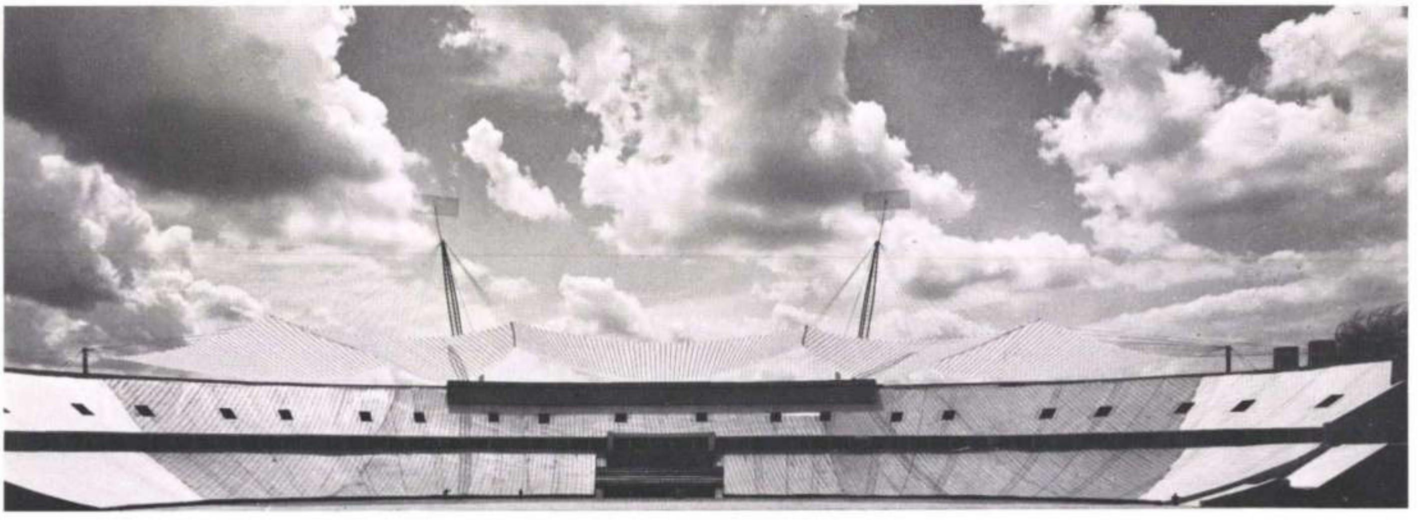


Fig. 3
Design for the Olympic Stadium roof, Berlin

Fig. 4
Arctic City project

Fig. 5
Project for Shadow in the Desert
(Photo: Fritz Dressler)

architectural and technical library, which is expanding all the time as the result of our development work.

Perhaps at this stage, it is useful to try and explain the role of the laboratory in the design of a lightweight structure in a world where increased computer capability is now such as to enable the creation of amazingly complex shapes with the barest boundary definition by the designer and their subsequent presentation on the electronic screen.

Whereas in conventional structures, a designer can fairly quickly assess the nature of his project, the problem for the designer of a lightweight structure is precisely this definition of boundary points which he cannot make without understanding the whole shape and the processes he has undergone to attain the best general solution. The electronic screen and light pencil do enable the presentation of his decision as perspectives in two dimensions (or may be with stereo pairs as a three dimensional representation). Essentially the problem is one of the slowness of the designer to comprehend the nature of the design he is creating – in all its artistic, environmental and structural aspects – and computers are notoriously expensive when man is involved in their operation.

So still the problem remains that two-dimensional drawings of three-dimensional non-regular structures are at the best unsatisfactory and at the worst misleading. Hence the evolvement of design methods by Professor Otto over 20 years or more of modelling techniques that give accurate enough shapes to enable basic decisions and contributions to be made by all parties to the design at an early enough stage and so permit communication and modification. With experience, fairly crude models made from stretched tulle or stocking material enable designers to judge the geometry, and hence assess the basic building performance and so permit comparison with more conventional methods of construction. These sketch models can be made in the laboratory. From them, with the aid of some approximate analysis, and the

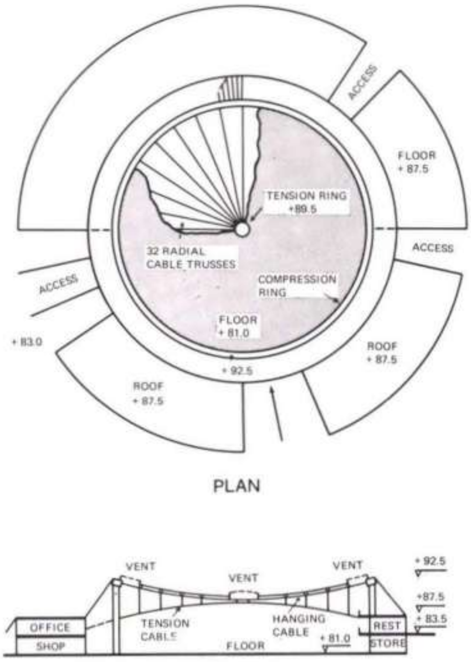


Fig. 6
Wolverton Agora
Architect: Milton Keynes
Development Corporation

technical library of material types, performance, range of hardware, etc., the sketch design can be assembled. Only then can the sizing of members, the assessment of foundation loads and the production of sketch drawings, outline material specifications and quantities proceed so that outline costs can be sought from contractors. Normally, this is the 'report' stage of the job by when all the major design decisions of all parties should have been taken. It is particularly important for lightweight structures where all the various factors that affect the design are so inter-related that this stage in the design is observed. It can be accompanied by a model illustrating the final evolution of the design if the client so wishes.

The next stage requires the more accurate assessment of the design shape, member forces and outline details from which working drawings and tender documents can be prepared. Traditionally, for complex shapes at least, this involves the use of accurate form models and the subsequent progression into structural models and structural model testing. The design of the German pavilion at Expo '67 and the main Munich stadium of the 1972 Olympics were both achieved this way. However, Munich also enabled the development of an impressive range of computer software by the Institut für Statik und Dynamik (Stuttgart) to undertake large system non-linear analysis. From these earlier techniques, the mathematical methods described above that enable the production of full accurate shapes from the barest boundary information are now emerging. The necessary combination of the slow, labour intensive hand driven techniques of design with these faster modern computerized methods is perhaps where the laboratory can offer advice. To put it another way, the laboratory is already some way along its learning curve and so perhaps able to give advice as to possible design methods so that the designer more quickly reaches a sufficiently comparative comprehension of the entirety of his design to play effectively and efficiently on the computer.

As elsewhere in the construction industry, the promise of interesting and useful work in the design office was rather curtailed by the effects of the three day week. At that time, we were undertaking a sketch design for a roof over Wolverton market place with Structures 3 which was to be a 40 m ø radial cable roof truss (Fig. 6). Complete sketch drawings for this bicycle wheel roof were prepared and an outline price for the cable roof sought from British Ropes. The initial outline costs for this scheme were £25/m² inclusive and Table 1 shows the unit resources required.

Another interesting outcome of this particular project came from the mixed usage requirements of the central public space and peripheral shops, cafes, offices, etc. when considered in conjunction with the performance of the unusual (in this country at any rate) cable roof in the event of fire. It was necessary to prepare a report evaluating the performance in a fire. The bones of this report are given more fully in a paper given at the Tension Structures Conference in London in April 1974 'A study in the structural performance and protection in fire of a cable roof' In brief the report concluded that the radial cables and tension ring did not require fire protection, but that because of its closeness to the surrounding shops a $1\frac{1}{2}$ hour fire resistance was required of the compression ring. Additionally the cladding material should have at least Class I flame spread resistance but need not necessarily be of incombustible material if there were a sprinkler system.

Another interesting scheme for which sketch drawings and preliminary material specifications were prepared was a roof scheme employing an undulating cable roof over the proposed Snow Hill Sports Centre, Birmingham (Figs. 7 and 8), with BED. Table 1 illustrates the economy possible with scarce materials for such a tension structure over an 85 m span arena.

Encouraged by the figures of Table 1, efforts were made to predict the theoretical costs of such structures and compare them on a resource basis with more conventional structures. With the help of Kendrick White of Widnell & Trollope, Quantity Surveyors and Vic Gill of British Ropes Ltd. the laboratory produced a discussion paper 'The theoretical costs of cable structures' for the Tension Structures Conference in London. Figs. 9, 10 and 11 are derived from these studies and agree in general with the outline costs received for the two cable structures mentioned above. They show too that for regular buildings with spans as little as 30 m, cable beam roofs may well be economic propositions, especially where transport costs are large and speed of erection is significant.

Table 1: *Outline feasible quantities*

	Birmingham Snow Hill	*Wolverton Agora*
Use	*Indoor Arena*	*Market Place*
Maximum dimensions	85m × 140m	40m
Plan area	10920m²	1200m²
Cables	12.5kg/m²	4.0kg/m²
Steel mast	24kg/m²	−kg/m²
Concrete ring beam	—	0.06 m³/m²

One or two opportunities for nice looking stressed tents have come our way over the year. These employ PVC covered polyester fabric membranes stretched between cable boundaries into a shape adequate to allow good double curvature. Figs. 12–15 show the scheme models for Scarborough Open Air Theatre (with Structures 2), a covering tent for an antique carousel owned and operated by the Smithsonian Museum, Washington DC, and a scheme for a shade net over a hotel tennis court for tropical countries. These schemes indicate the variety that is possible with polyester membranes. The Scarborough membrane was to be an open weave fabric, soot covered to give better protection against ultraviolet radiation and coated in translucent PVC to have 30–50% translucency. The carousel was to have a plain white PVC coated polyester fabric of a closer weave and the shade nets were to be porous with approximately 30% free area. Blacks, reds, blues, greens are also possible and all are flame spread resistant.

However, most of our current efforts are being directed at basic development – on the costs of lightweight structures, on their fire performance, on the basic technology of steel cables and the necessary connection hardware, on suitable methods of approximate analysis and applicable wind loadings.

In particular, much has been done to define the criteria for the design of air-supported structures in order to contribute effectively to the draft code for air-supported structures currently being prepared for the BSI by Frank Newby and Cedric Price. Other study projects currently in the pipe-line are textiles for building (both as membranes and ropes), methods of dynamic analysis and a survey of anchorage systems for use with tension structures.

One cannot pretend that a tension structure is the solution for all large building roofs or that every temporary cover to an outside activity is solved by the elegance of a stressed membrane tent. However, in other countries, cable structures have achieved a far greater usage than in this country and should, if judged by the results of the laboratory's efforts, receive much more serious attention here, particularly now that resources are scarce and knowledge much improved.

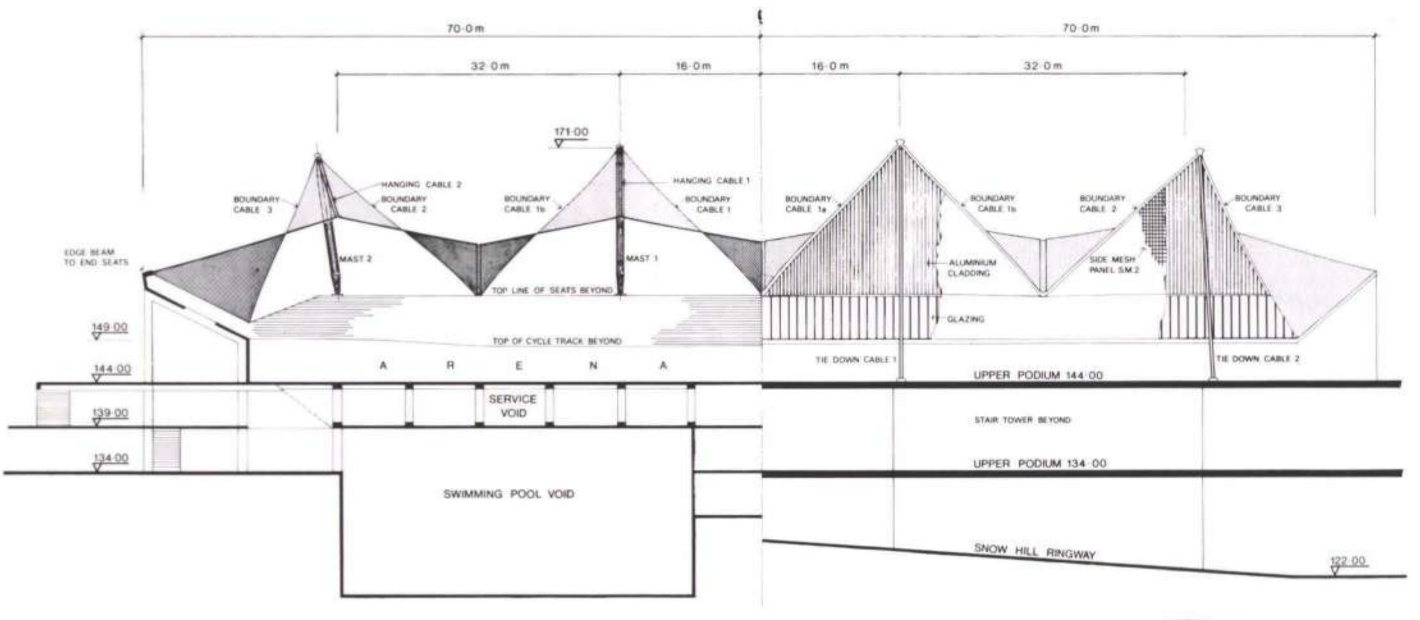


Fig. 7
Snow Hill—long elevation and section

Fig. 8
Birmingham Snow Hill, indoor arena. Architect: Birmingham Corporation Architects

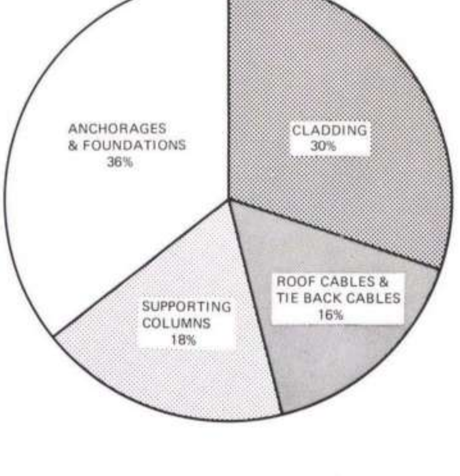
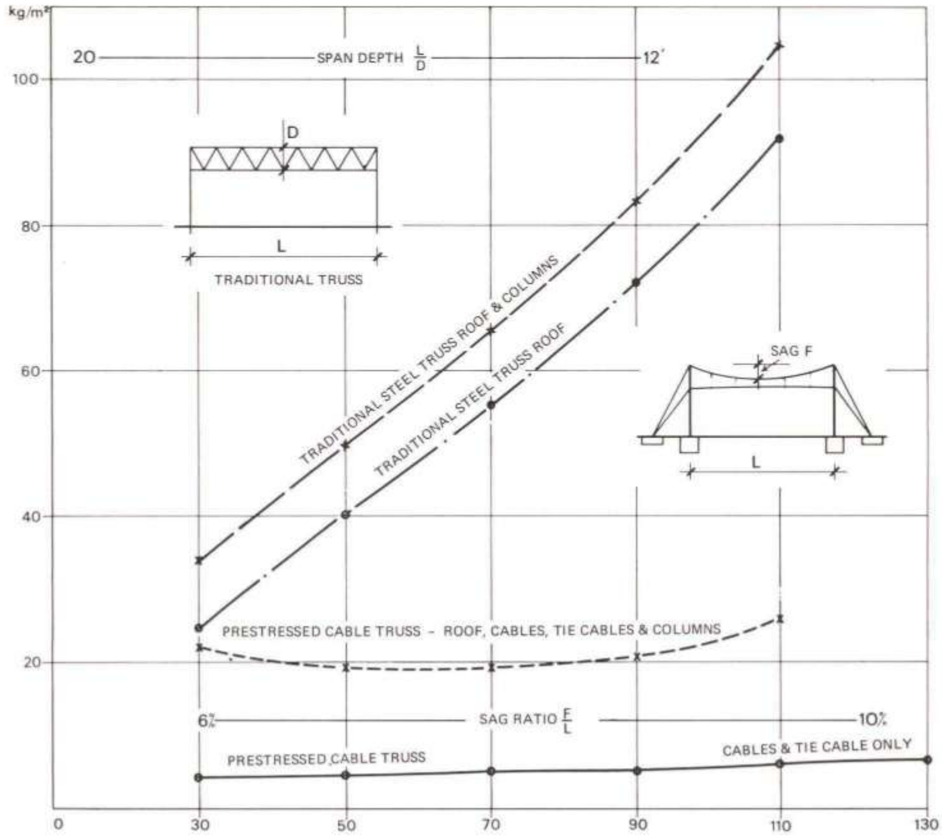


Fig. 9
Proportional expenditure for a 50 m prestressed cable truss roof

Fig. 10
Comparison of steel quantities
for traditional trussed roof
and prestressed cable truss



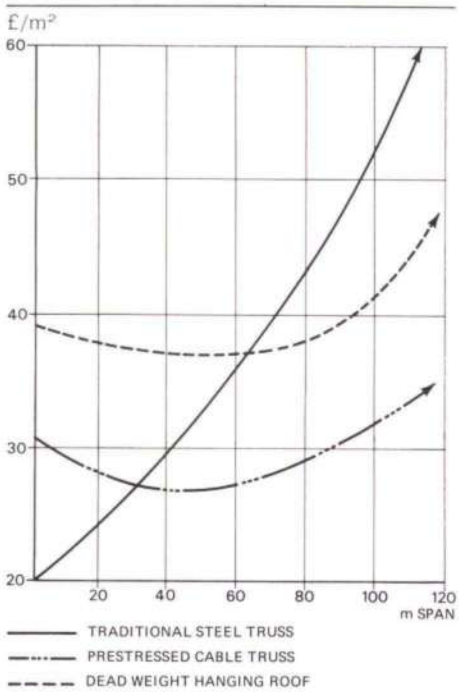


Fig. 11
Theoretical cost comparison
for the finished roofs

Similarly, stressed membrane tents and air-supported structures offer specific alternatives to the more conventional forms and have not been much exploited in this country. Industrial concerns have perhaps realized this in their use of semi-cylindrical air supported structures for office or storage expansion where long term requirements are uncertain and considerable economies can be derived by accepting this fact in the design. It is sufficient to add that the convertibility possible with stressed tent structures has been exploited many times in the past 20 years at exhibitions, outside events on the continent of Europe, and in America, often with startling architectural effect. Perhaps it is time to look to such alternatives of construction for some projects in this country.

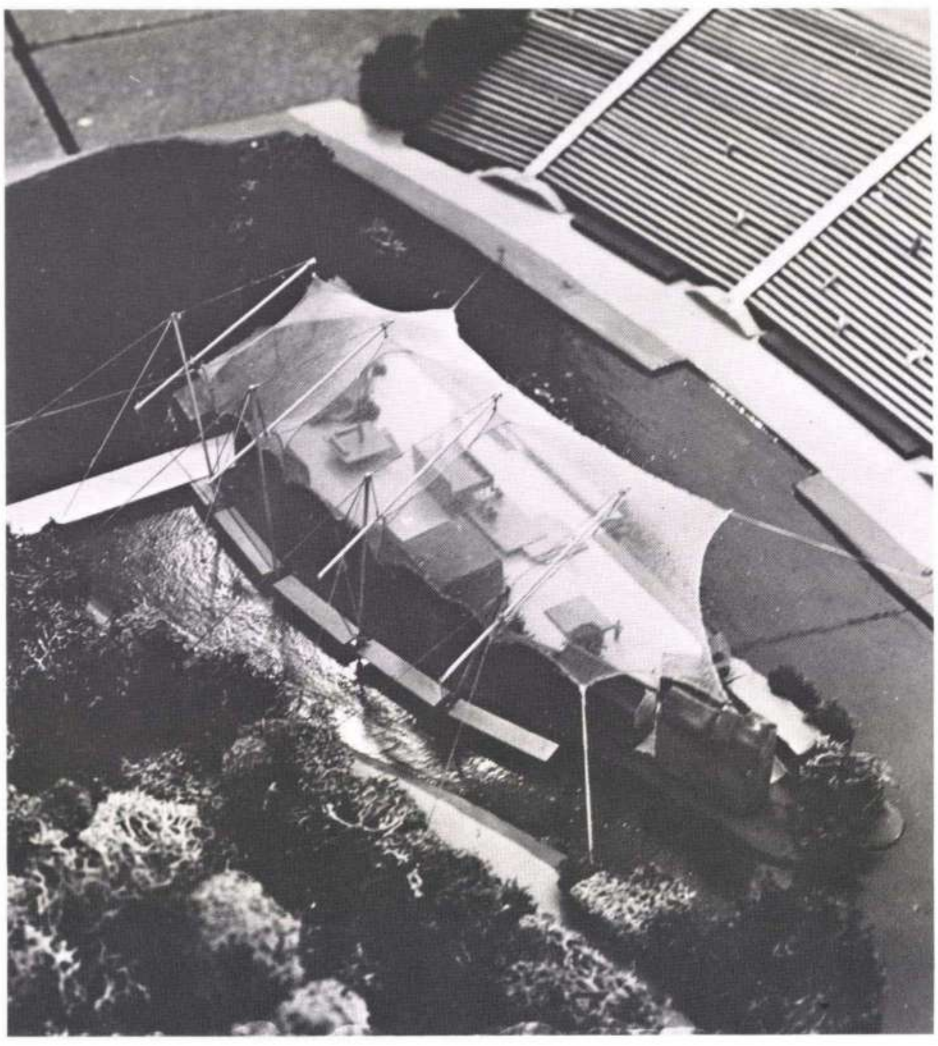


Fig. 12
Scarborough open air theatre:
Tent covering over the stage
Architect: George Trew Dunn Beckles Willson Bows

Fig. 13
Scarborough open air theatre:
view of stage from seats

Fig. 14
Theatre over carousel for
Smithsonian Museum, Washington DC
Architect: Atelier Warmbronn

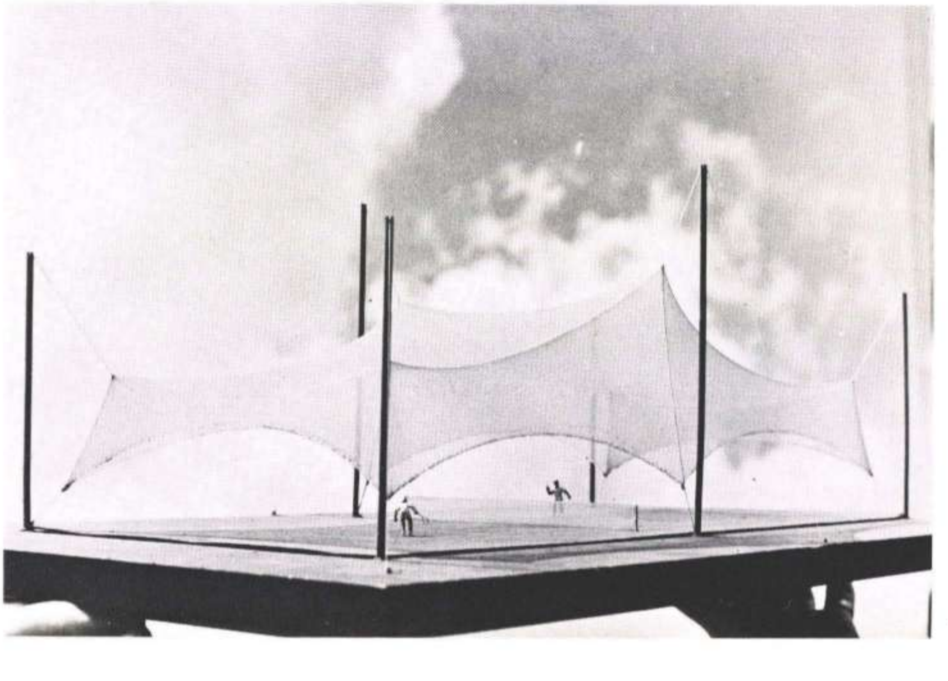


Fig. 15
Shade covering over a tennis
court for a tropical country
Architect: Atelier Warmbronn

Kings Reach Development: The Office Blocks

John Crouch
Stein Ingolsfrud

The office complex within the Kings Reach Development consists of two main blocks, a 31-storey tower and a five-storey low-rise block, which are linked together to provide a total of about 35,000 m^2 of office accommodation. The boiler house is at the top of the tower block.

Work commenced on the site in April 1971, as part of the main contract that had been let to John Laing Construction Ltd., and it is anticipated that the offices will be completed for handover during the summer of 1975.

The tower block is to be known as Milroy Tower (after Milroy Wharf which used to occupy part of the site) and the entire office complex will be occupied by the International Publishing Corporation Ltd., one of the members of the development company. The total cost of this part of the works is expected to be about £5m.

The site

The site is situated in Central London, on the south bank of the River Thames just upstream from Blackfriars Bridge, and was previously an area of warehousing and commercial properties which had remained virtually unchanged since the middle of the 19th century. The office complex, however, is located at the western end and is bounded by Stamford Street on the south, Hatfields on the west and Upper Ground at the north.

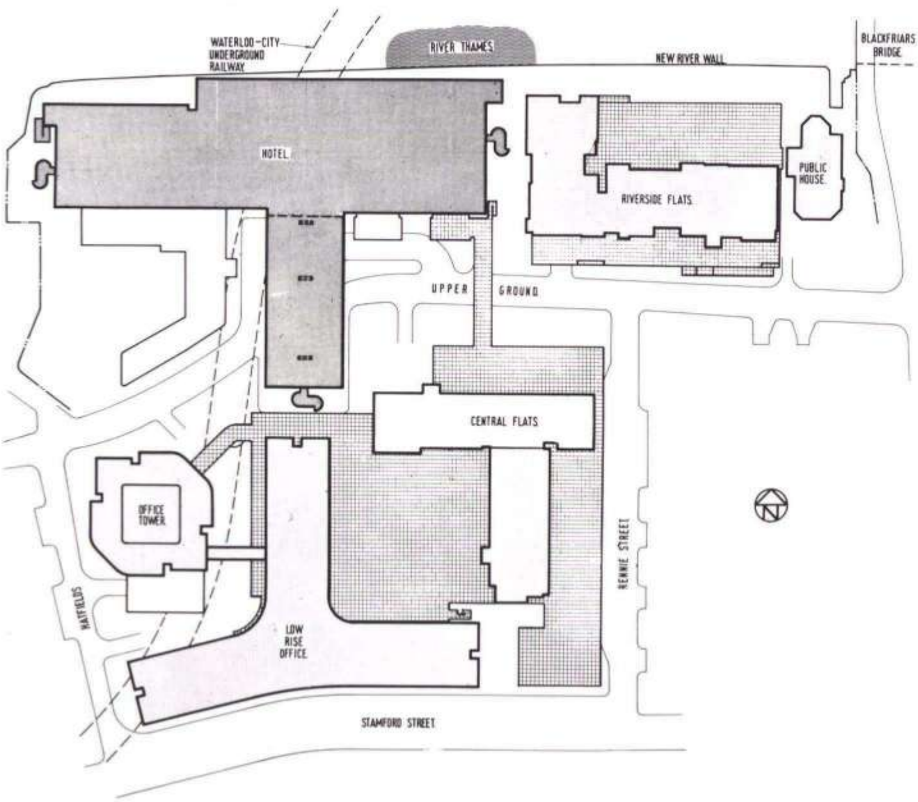


Fig.1
Kings Reach site plan

Fig. 2
General view of site from north west. August 1974
(Photo: Handford Photography)

The twin tunnels of the Waterloo and City Railway ('The Drain') traverse the site in a north-south direction and pass under both of the office blocks.

Ground conditions

The average ground level of the site is about +4 m AOD (Above Ordnance Datum).

The typical Thames-side Succession consists of about 2 m of fill overlying the old marsh deposits of soft silty clay and peat with a thickness of about 4 m. Below this, a gravel terrace about 4 m thick covers the London clay which extends from about −6 m OD down to −36 m OD, where it overlies the Woolwich and Reading Beds.

The ground water in the gravel is under a modest artesian head, the natural water table level being about +2 m AOD.

The office tower

The 110 m high tower consists of 31 floors with a central concrete core containing all the usual services. The office floors are generally 380 mm thick and of coffered slab construction, supported by 42 precast columns (or mullions) around the perimeter.

Foundations

The 38,000 tonne structure is supported by 79 large diameter bored piles each of 1.22 m diameter and bored 26.5 m into the London clay with 2.5 m diameter underreams. The pile cap is generally 3 m thick.

Due to planning requirements, however, the block had to be positioned such that the foundations were restricted on the north and west sides by public roads and by the Waterloo and City Railways passing underneath to the east.

This meant that the tower could not be supported by the piles on the west side of the tunnels only, and therefore the foundation slab had to be made to bridge the tunnels, with 73 piles being put down on the west side and six piles on the east side. The piles were set out on a triangular grid, generally at the minimal centres of 3.5 m with the settlement being the design criterion.

Average working loads were 4800 kN per pile with an estimated settlement of 35-40 mm. The group factor used was 0.65, thus achieving an ultimate capacity of 14,900 kN with a factor of safety of 3.1.

The substructure was also checked to ensure that the building would settle evenly without tilting.

The effect of the tower on the twin railway tunnels passing underneath is equivalent to a net increase in pressure of 7% of the overburden. Special precise surveys are being carried out inside the tunnels to monitor the effects of constructing the new buildings over it.

Because of the high water table and the position of the public roads, a temporary sheet pile coffer dam 190 m long and mainly stabilized by ground anchors had to be installed to enable the ground works to be carried out. In addition a chemical grout curtain (the Joosten two shot process) had to be used in the south-west corner because it was not possible to get complete 'cut off' of the sheet piles into the clay due to the presence of the tunnels.

Superstructure

The core, which is 15 m square in plan, consists of four segments, connected together by beams at each floor level. It has been designed to accommodate the vertical loads and the horizontal forces from the wind and the thrust due to the step in the column line. The internal walls are all 230 mm thick, but the outer ones vary in thickness from 400 mm at the base to 230 mm for the upper floors.

The external concrete columns are all of precast concrete using Balidon Limestone aggregate with tinted white cement and with a specified crushing strength of 45 N/mm². The columns are generally double storey-height and alternate joints are staggered. All exposed faces are ground.

The tops of the columns have two 25 mm diameter dowel bars which protrude into the holes provided in the base of the unit over. The unit above is lowered on to an asbestos shim to give the correct joint thickness of 20 mm, the neoprene gasket only acting as a grout seal.

Embeco grout was used for all the column joints and produced a strength of 70-80 N/mm² at 28 days. The maximum stress through the joints due to vertical load reaches 20 N/mm², increasing to 30 N/mm² due to bending.

The column line on three sides of the building cranks or steps out 1.34 m at three different levels, i.e. between 1st and 3rd, 3rd and 5th and 7th and 9th. The moments caused by this eccentricity are taken out by a push/pull action over two floor heights. The push/pull forces, 1,000 kN per column at low level, are taken out by a compression in the lower floor and by *Macalloy* pre-stressed bars on the upper—this slab also being tied into the core by other pre-stressed bars.

To minimize the bending effect due to the elastic deformation in the two slabs, movement joints have been provided between the column and slab at the centre of the 'step' units and above and below the floors taking the push/pull forces.

The pre-stressed floor, compression floor and the floors above and below the movement joints are 380 mm thick and of solid construction and have been designed against explosion to prevent progressive collapse.

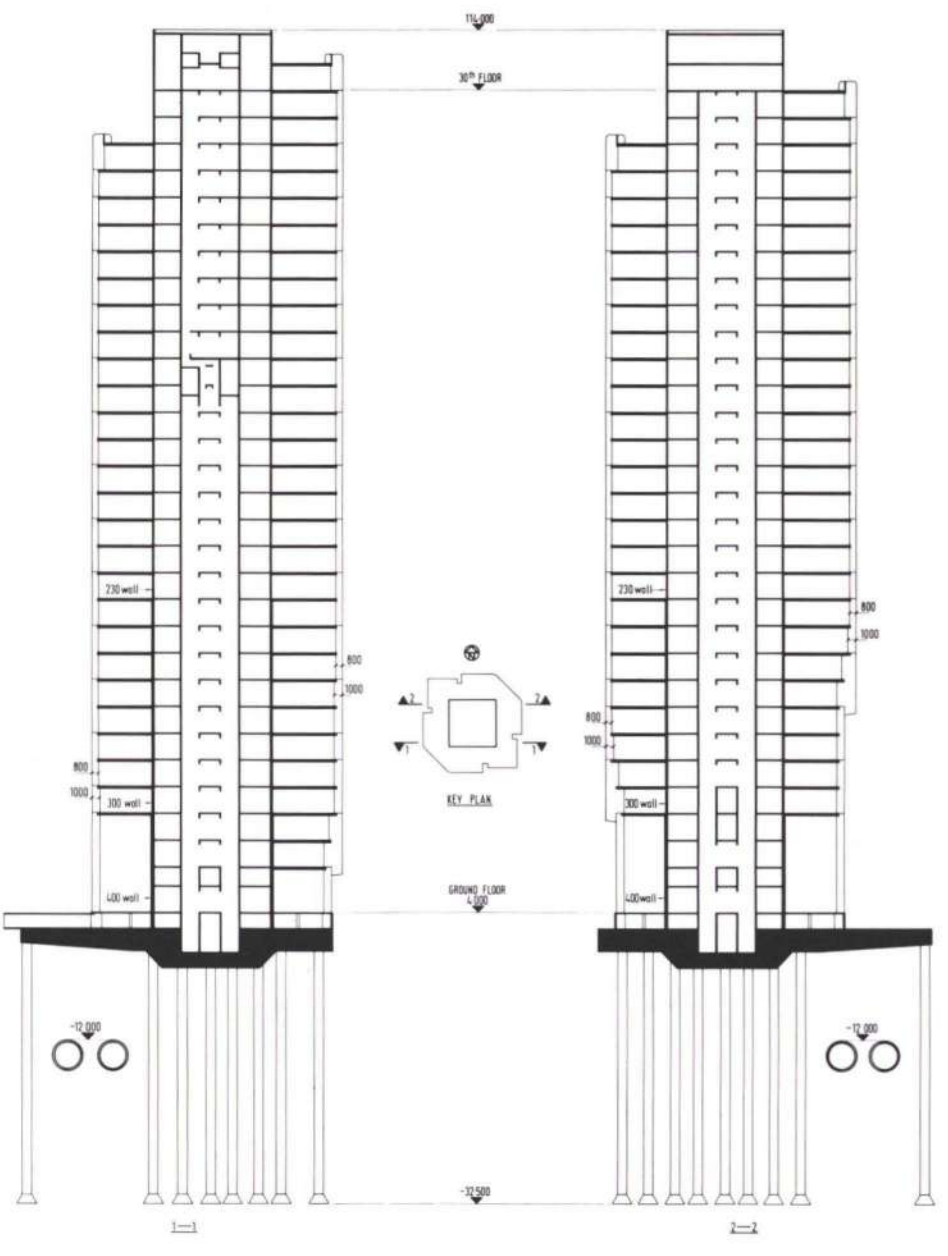


Fig. 3
Kings Reach office tower: sections

Construction

The method of constructing the superstructure was inherently complex and so much thought and preliminary work had to be given to the methods of erecting the various types of precast units. As a result, however, their installation was quite easily accomplished and there were few snags.

As 29 of the 42 precast columns up to third floor level were 13 m long and weighed about 14 tonnes (a few specials were up to 19.25 tonnes) a mobile crane—a Coles *Centurion* of 105 tonnes capacity—had to be used for their erection. Installation was carried out after the 3rd floor slab, except for an edge strip, had been cast and left on its scaffolding to facilitate their temporary support and also to ensure that the mullions achieved better vertical alignment.

The 34 'step out' mullions, of double storey height and weighing about 14 tonnes, were also erected by mobile crane—the Coles *Centurion* at 1st to 3rd, and a Lorain Moto 110 ton (with 150 foot tower) for the 3rd to 5th and 7th to 9th levels.

Special temporary frames were designed and manufactured to support these heavy 'out of balance' units from the partially cast slabs at their mid-height until they were permanently tied into the floor slabs. All the other mullions—typical ones weighed about 5 and 6.4 tonnes—were erected by the tower crane as part of the normal erection process. Prior to the manufacture of any of the precast units a full-scale model was made up of a typical joint detail between two mullions, to test both its efficiency as a joint and its ease of being grouted on site, and as a result the internal faces of the upper unit had to be 'streamlined' to ensure that a joint full of grout was obtained. It was also found to be necessary to use a small frame around the gaskets during the grouting operation to prevent them from 'blowing'.

Low rise office

This 25 m high block, T-shaped in plan, is adjacent to and linked up with the office tower. It is generally of six floors, but there is also a half-basement car park under the north wing and a full basement which accommodates many of the services, below the western end of the south block.

Substructure

The building is supported by a combination of 432 mm and 483 mm diameter *Frankipiles* (700 kN and 900 kN respectively) of quite short lengths bearing into the gravel layer but, in order to minimize the effects of pile heave or bounce, it was necessary for them to be installed using the multitube system.

However, where the tunnel passed under the

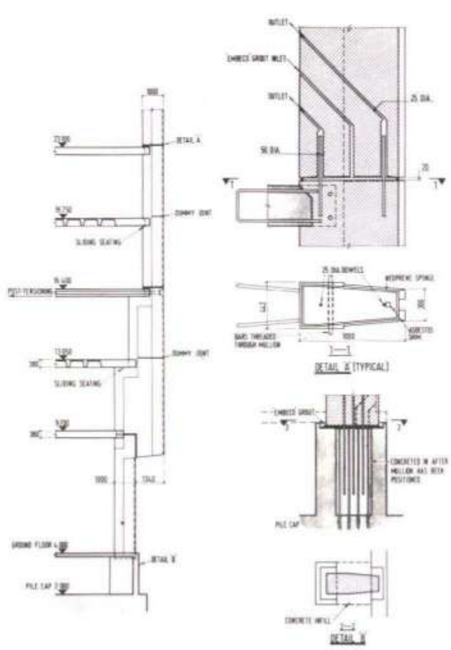
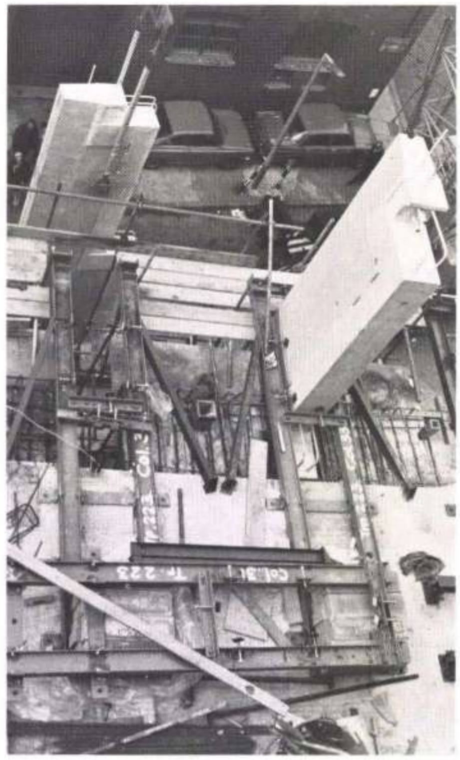


Fig. 4
Office tower—typical mullion details

Fig. 6
Full scale 'mock up' of typical joint between precast columns
(Photo: Ove Arup & Partners)

Fig. 7
Temporary steel bracing frames to support 'step out' columns
(Photo: Ove Arup & Partners)



western end of the south block, piling was not practicable and therefore a raft 1.3 m thick had to be used, also bearing onto the gravel. This need to excavate down 6 m to a suitable bearing stratum was exploited by the construction of a useful basement to house many of the services.

Superstructure

This is a reinforced concrete frame of column and coffered slab construction.

Around the perimeter of the building the vertical structure consists of precast concrete mullions at 2.68 m centres, made with Balidon limestone aggregate and similar in appearance and specification to those on the office tower. Generally they are of double-storey height and the joints are staggered.

The typical mullions were erected by the tower crane but the 'heavies'—caused by the building stepping outwards about 1 m at 2nd floor level—had to be lifted by a mobile crane.

A central line of in situ columns provides the internal support for the slabs, which are generally 380 mm thick coffered flat slabs. The blocks are generàlly 15 m wide.

As this block may be classified as a typical six storey, low-rise concrete-framed office block (approximately 10,000 m^2 gross) the manner in which the steel reinforcement was actually used may be of interest and is therefore indicated below—in percentages:

Pile caps and basement slab	10%
Ground floor slab and beams	13%
Floor slabs and roofs	56%
Columns—precast and in situ	12%
Walls—including basement walls	7%
Staircases	2%

The total weight of steel used was approximately 1000 tonnes.

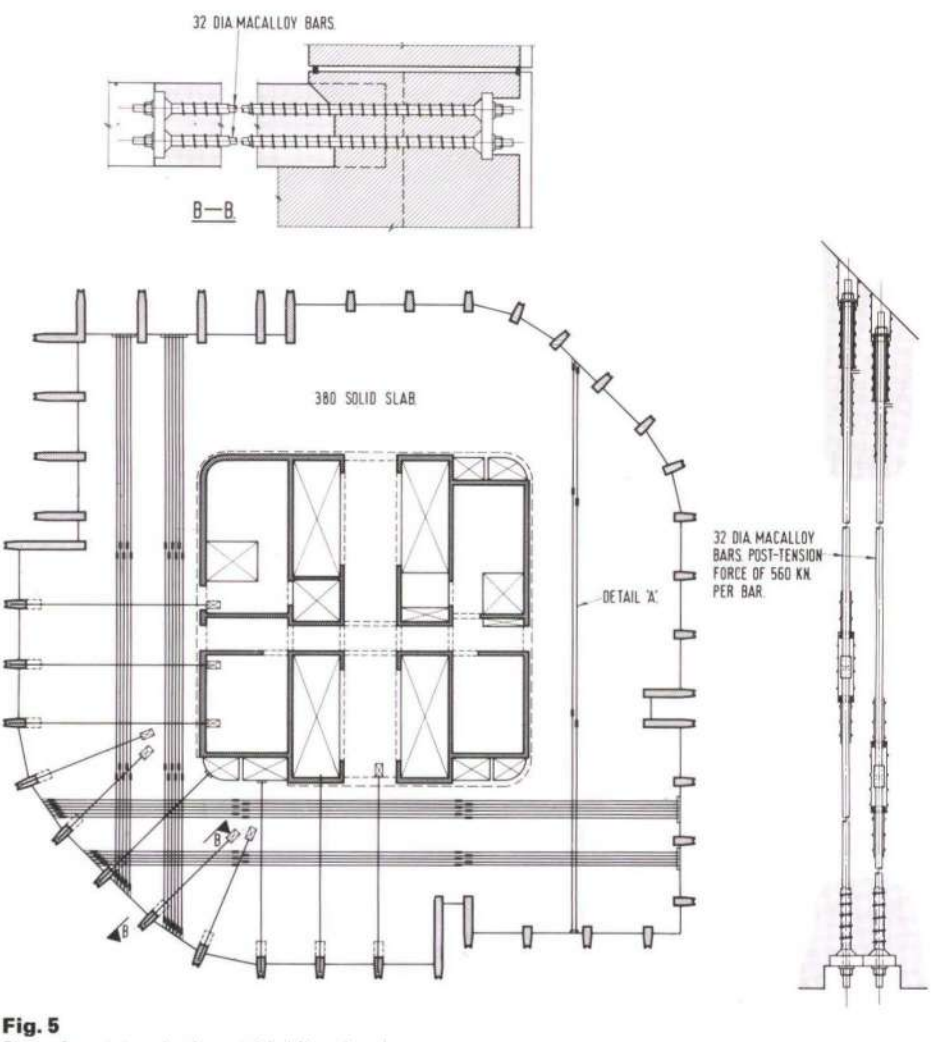


Fig. 5
Plan of post-tensioning at third floor level

Credits

Architects:
R. Seifert & Partners

Quantity surveyors:
Langdon & Every

Mechanical and electrical engineering consultants:
F. C. Foreman & Partners

Main contractors:
John Laing Construction Ltd.

Sub-contractor for supply and installation of the precast mullions:
Empire Stone Company Ltd.

Halford's Headquarters at Redditch

Ernie Irwin
Martin Trend

Introduction

Halfords Ltd. have over 350 shops throughout the United Kingdom in which they sell motor accessories, camping equipment, cycle parts and a multitude of other items. Although the firm originated at Halford Street, Leicester, their headquarters has been in Birmingham for the last 70 years. Expansion and the problems of parking and transportation for warehousing facilities near the centre of Birmingham made them decide to move to a new town and leave a headquarters which had been built for their needs only 10 years previously. They chose Redditch New Town, about 18 miles south of Birmingham.

The new headquarters consists of warehousing to receive over 5,000 stock items from manufacturers, and the storage and redistribution of these items, often in small quantities, to their various shops throughout the country. In addition offices for management, central accounting and computer, as well as training centre and restaurant facilities, are provided.

Redditch Development Corporation made available to them a 5.5 ha site in the developing Washford Industrial Estate, and Halfords appointed Harper Fairley Partnership as their architects. This firm had designed the client's previous two headquarters and were thus familiar with Halfords' special needs. One year was allowed from commencement of design to start on site and the client wished to start stocking the new warehousing exactly one year after commencement of construction on site. As the floor area of phase 1 warehouse was 17,000 m² this formed a very tight programme, even though a further six months was allotted for completion of the office block.

The site was a green fields site with a drop of about 4.5 m diagonally across the 5.5 ha area which was to comprise the immediate development. About 450 mm of top soil overlaid keuper marl over most of the site. However, towards the main road near where the office block was to be located, the marl fell away under a layer of clay as the ground dropped towards the River Arrow. The river is parallel to and on the far side of the public road, which incidentally is the old Roman Road, Icknield Street.

A site investigation of trial pits showed that the

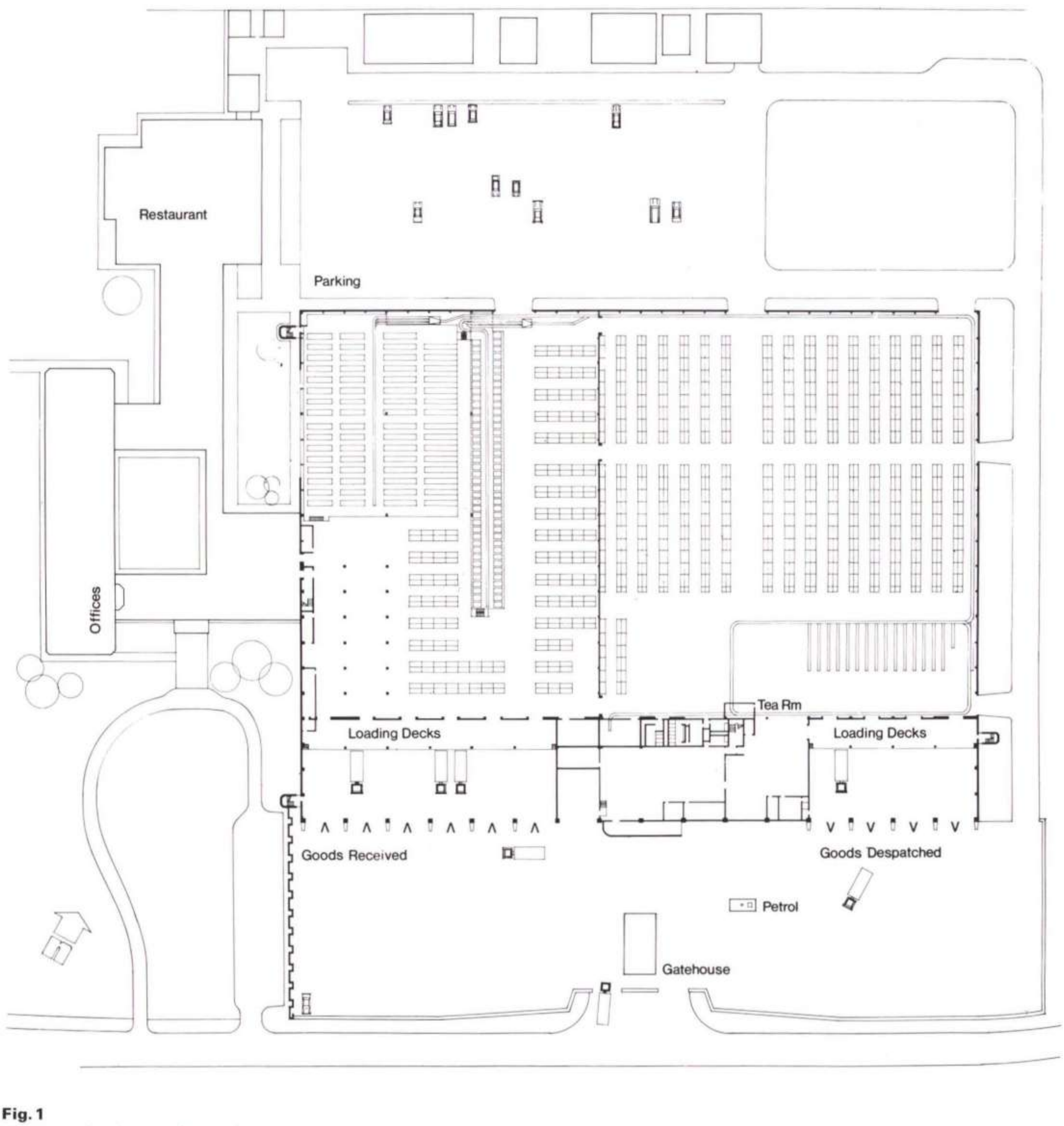


Fig. 1
Site layout showing warehouse plan

marl varied from firm to stiff to hard with depth and we estimated that satisfactory foundations at 400 kN m^2 could be founded in the firm stiff marl. In addition the marl was deemed satisfactory for use as compacted fill on the vast earthworks operation that was needed to level the 4.5 m diagonal cross fall.

Warehouse planning and materials handling

Success of the project depended on making the right decisions for the warehouse design and then implementing them within one year. All the basic criteria such as column grid, clear height, floor loading, roofing system and natural lighting requirements were intensively examined and compared with alternatives before firm decisions on any one item were reached.

The client has appointed his warehouse manager, Mr. Mark Rushbrook to liaise directly with the design team and he participated in the major decisions. Together with his materials handling consultant, he chose a fork lift truck system of distributing the large goods, called *Bulkflow*, to and from the main racking on ground level within the warehouse. The fork lift trucks can lift materials to 8.2 m above the floor. Higher fork lift equipment would demand the use of a floor to a level of precision which would have been extremely expensive structurally. Alternative stacker crane systems did not offer the speed and flexibility of use required.

In addition to the main 8.2 m high racking for bulk storage, the Speedflow* system was used for the selection and packing of small boxes of goods for distribution to the hundreds of shop outlets. A three-storey system of steel racking was chosen for this purpose and the concrete floor slab was designed to receive the supports to the plant in any position. Individual foundations were unacceptable as they would have prevented any future replanning of the racking and would have led to its classification as a structure, thus requiring fire resistance. Female operatives work in all three levels selecting goods in optimum sequences worked out by computer. Using the Speedflow system the completed orders in boxes are then taken by roller conveyor around the perimeter wall of the building to the despatch area where they separate automatically by means of electronic control and enter the particular loading channel where orders for the chosen destinations are being collected.

Loading bay areas were provided within the warehouse area and as the headroom requirement was less than 5.1 m, it was possible to provide a mezzanine for the full length of the building. The mezzanine has various uses such as storage of special items and slow throughput stock. A loading of 7.5 N/m^2 was chosen with a headroom of 3 m.

An approximately square grid of structure was required to provide virtually unimpeded planning on both axes within the building. Consequently, various space frame type structures, including proprietary brands, were examined, together with lattice truss systems of rolled hollow section and rolled steel angle construction. These alternative structures were considered for grids varying from approximately

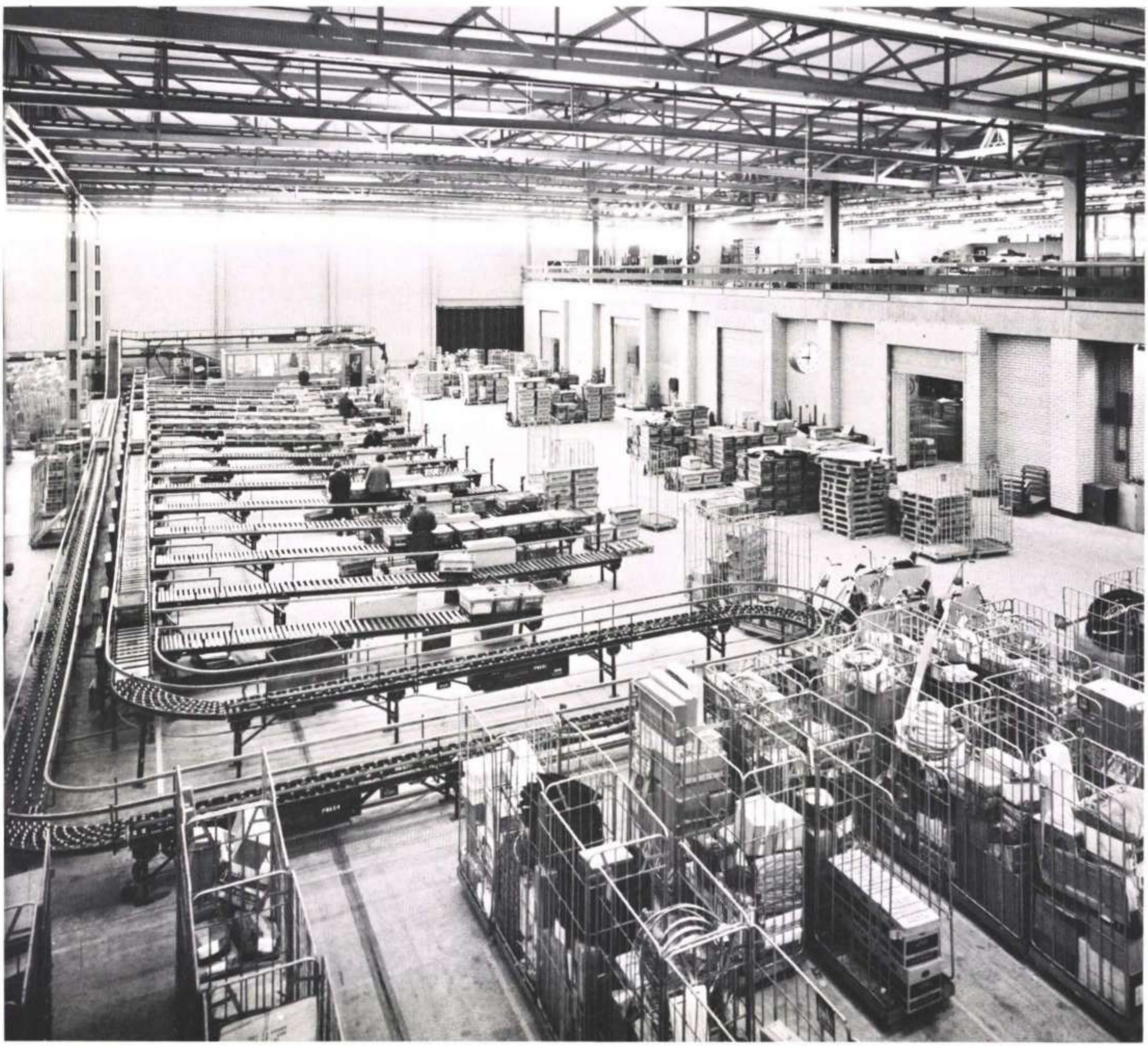
*'Speedflow' is a term used by Halfords to describe this characteristic passage of small goods through the warehouse. It is not a proprietary system.

Fig. 2
Front view of office block and warehouse

Fig. 3
Aerial view showing warehouse phases 1 and 2 completed. The office block is in the right foreground, the restaurant in the left foreground

Fig. 4
Interior view of warehouse showing lattice truss roof. Three tier steel racking for 'Speedflow' system is in centre, while bulk storage racking is shown to either side

Fig. 5
Despatch area and view of mezzanine over the loading bays. Some 'Speedflow' boxes can be seen arriving from the pick-up area for despatch



15 × 15 m up to 24 × 24 m. The operation of the warehouse was such that as few columns as possible were desirable, but clearly above 24 × 24 m the roof structure began to become more expensive. While the 15 × 15 m grid gave the cheapest structure, a grid of 19 × 22.5 m was finally adopted as reasonably economic and likely to accommodate future replanning while also fitting the aisles spacing and storage rack module.

Warehouse foundations and floor slab

Due to the good soil-bearing capacity providea by the keuper marl it was feasible to build a floor to support loadings up to 8.2 m high, even in fill areas provided that the earthworks were carefully compacted in the right conditions as described later.

Column and wall foundations were simple pad or strip footings at least 0.9 m into original ground. From the load requirements of 8.2 m high racking for general hardware and weights of Lansing Bagnall fork lift trucks, alternative reinforced concrete slabs were designed on hardcore and on lean mix concrete. The lean mix concrete combination of 200 mm thick doubly reinforced slab over 150 mm lean mix concrete was chosen for construction reasons as described later. Floor loads were determined by measuring a variety of goods in the client's existing warehouse.

The slab was designed to be placed on the long strip method, using a monolithic grano topping, with strips up to 4.5 m wide. Construction joints were saw-cut directly over timber fillets which had been placed in the underside as crack inducers. The concrete saw cuts were made at selected times within 24 hours of casting to avoid random shrinkage cracks.

Warehouse wall claddings

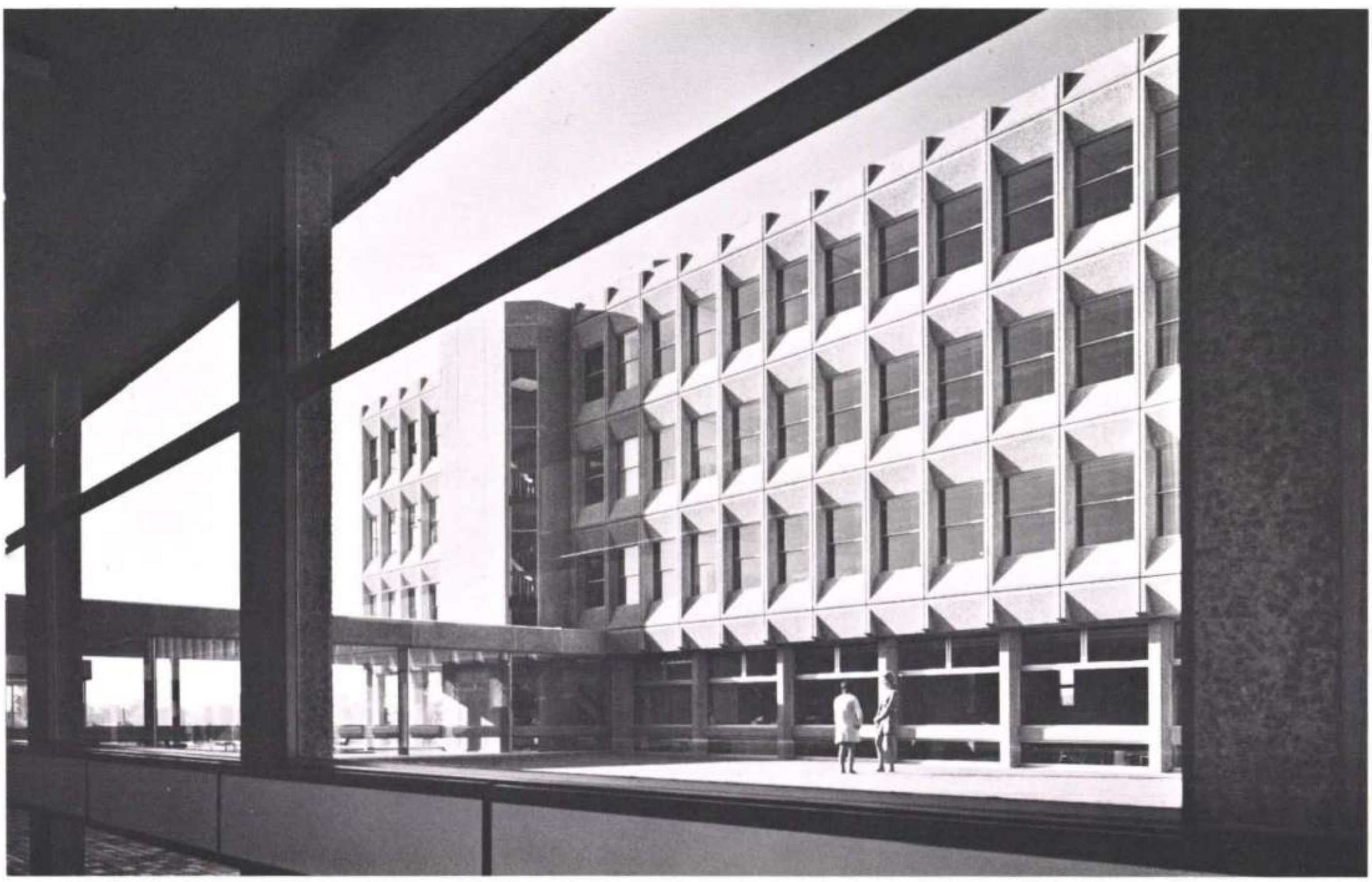
For aesthetic reasons, the architect chose to clad the building with brickwork. This was made 330 mm thick using brown facing bricks on the outside and sand lime bricks internally. The walls, which were 7.5 m high surmounted by clerestory windows, were supported on vertical steel members at 4.75 m intervals. Vertical movement joints were provided every 9.5 m and this proved very satisfactory as virtually no cracking has occurred.

Warehouse superstructure and roofing

To provide 17,000 m² of floor, which included some mezzanine, a building of dimensions approximately 152 × 112 m was required. Portal frame systems, though often used for warehouses, were clearly unsuitable for this size of building because the close spacing of columns transverse to the span greatly limited its flexibility. In addition the roof pitch for a large span would enclose a large volume which was not suitable for storage, nor was a sloping structure suitable for the support of the various services required such as sprinklers, heating pipes or lighting. At the same time studies of heating requirements and insulation led to the decision that natural lighting was not required in the roof although perimeter clerestory lighting was desirable so that staff could maintain a sight of the outside environment. As a result of these various studies it was decided to adopt a flat roof system with a structure that would provide frequent support for services

Fig. 6
Loading bays showing precast prestressed double T-beams supporting mezzanine above. The main beams are encased steelwork

Fig. 7
Office block as seen from link buildings. The precast cladding is loadbearing



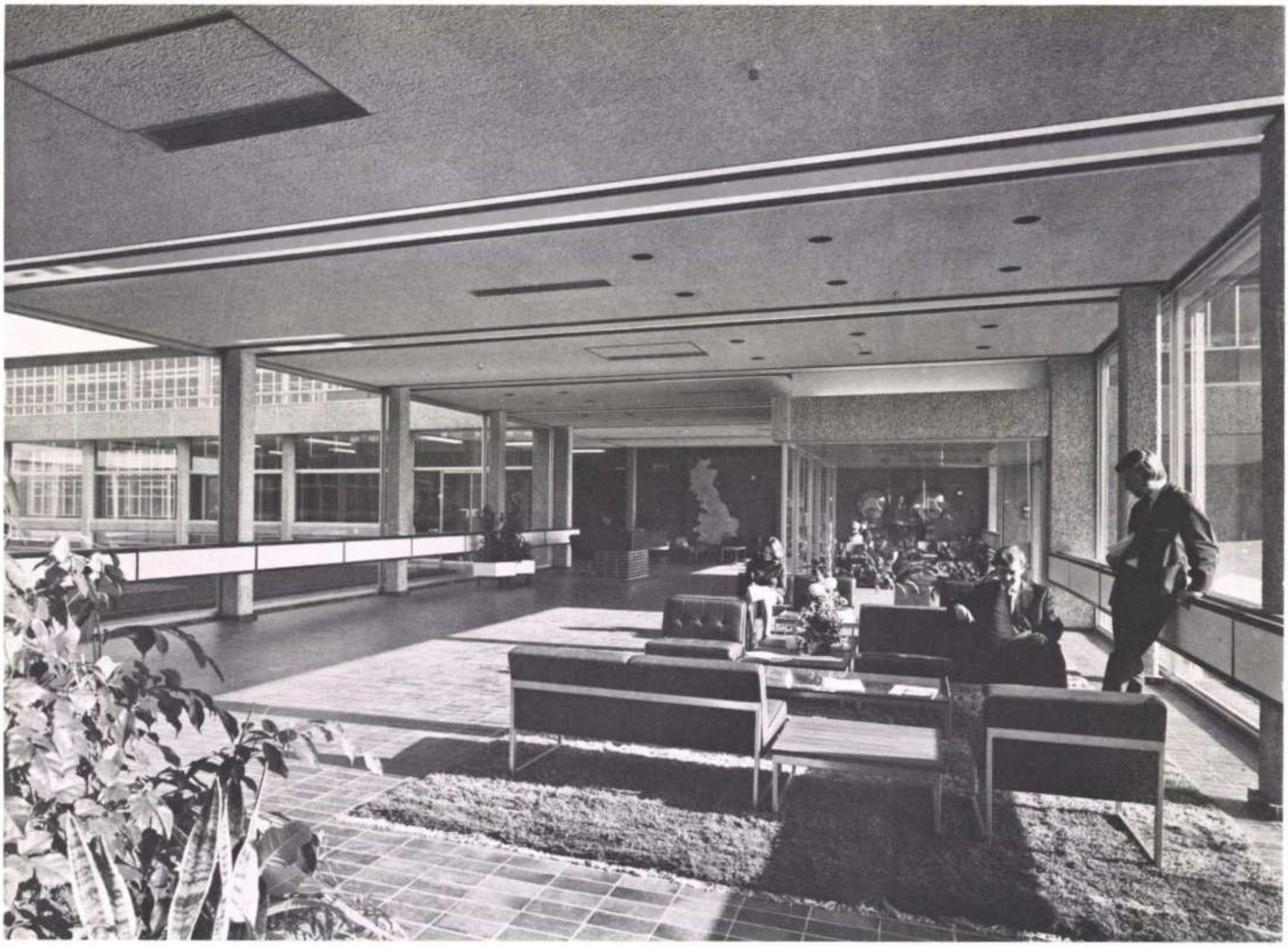


Fig. 8
Entrance to office block and warehouse

Photos: John Whybrow Ltd
Photography, Birmingham

and also allow the services to pass through the structure. In the paragraph on warehouse planning it was explained how a 19×22.5 m column grid was chosen, to support the roof. From a variety of exercises it was found that lattice trusses constructed from angle were cheapest and could provide an elegant structure if detailed with care. The client's insurance company offered better rates if an incombustible roofing material were used, and this led to the selection of lightweight concrete precast slabbing known as *Siporex* (now taken over by Durox) over cheaper and lighter forms of decking. The architects chose an asphalt membrane on this material and to give suitable falls after load deflections, the trusses were cambered upward by 215 mm. Drainage was achieved by downpipes at each column position, and to protect the downpipes from damage, they were housed between pairs of channels braced together to form each column.

The roof structure consisted of secondary lattice trusses 1.5 m deep at 4.75 m centres, 22.5 m long spanning onto main trusses 19 m long between columns. Fig. 4 shows this construction and every effort was made to obtain good details while also fulfilling the conditions of simplicity of fabrication and lowest cost.

The protective system chosen for the steelwork was blast cleaning preparation with one coat of *Erozin* zinc rich epoxy primer at works.

Mezzanine

Spans 9.5 m long taking a superimposed load of 7.5 kN/m² were required for the mezzanine over the loading bays. Consideration of these spans and a total warehouse construction time of 12 months led to the choice of precast prestressed double T beams with an in situ topping. As there was no requirement for fork lift truck loading at this level precast members with an in situ topping were adequate. Holes at regular intervals for services were planned and Fig. 6 shows a view of the underside of the floor.

Office block and ancillary buildings

The office block is a four-storey building with part basement and is designed to permit an additional storey. Various exercises showed that a hollow pot in situ floor construction in two spans was the most economical for the 15 m wide building. A high quality external finish was required with individual windows, and the architect chose precast panels with a Derbyshire spar exposed aggregate finish. The large area of concrete in the facade suggested using the precast panels as load-carrying members in conjunction with the in situ floors. This relieved the interior of projecting perimeter columns and resulted in a very economical building structure.

Single-storey link buildings were constructed of precast loàdbearing columns to match the office building with castellated steel beams supporting the flat roof. The single-storey restaurant was constructed of brick walling with a steel roof.

The boiler house and gas storage buildings were constructed in steel frames and the boiler chimney in precast concrete units.

Construction

The construction period allotted for the warehouse was 12 months commencing in December 1969. The diagonal crossfall of 3.5 m in the warehouse building alone required a large cut and fill operation with carefully controlled compaction of the keuper marl, which is particularly difficult in wet conditions. To solve this problem it was decided to let a preliminary earthworks contract during the design period which would last three months and could take place in summer with time to spare if weather conditions were poor. To protect the earthworks after this construction and to provide a working platform for the steelwork erector, lean mix concrete was chosen instead of hardcore and included in the preliminary contract.

The earthworks were constructed to a higher level than required with a shallow slope so as to remain dry until the lean mix was placed. The high water table in parts of the site was relieved by construction of a french drain around the entire perimeter of the buildings.

The main contractor commenced in December 1969 with a dry and level site permitting him to get steelwork started on schedule, although the need to construct column foundations and surface water sewers in poor winter conditions made things difficult. Nevertheless building work progressed to allow occupation on time at the end of 1970. A second phase extension of 5,200m² was added 18 months later.

Acknowledgements

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Main contractor:
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Preliminary siteworks:
R. M. Douglas Ltd.

Precast concrete:
Trent Concrete Ltd.

A simplified method for analysis of flexural members to resist low velocity head-on vehicle collision

Ari Danay

Introduction

Vehicle impact on rigid barriers has been intensively investigated both experimentally[5 6 7] and analytically[4] with the emphasis on improving the vehicle design with respect to the passenger's safety. However, there is insufficient direct information or comprehensive recommendations for the design of flexible barriers, such as car park parapets and columns. This article, which is an extract of a technical report under preparation in the R&D section of Ove Arup & Partners, is meant to fill that gap.

In Ref. 2 various codes and recommendations are reviewed. It appears that some codes do not have any specific recommendation for horizontal forces, or require only a nominal value. The German Code, for instance, refers to 2 tonnes applied at a height of 1.2 m, and allows double permissible stresses; Ref. 2 also mentions a 'rule of thumb' criterion—the horizontal force is the weight of the vehicle factored by the impact velocity in Km/hr.—and this value can be reduced to 30% if the structure is protected by a 50 mm ruber shock absorber. Though, due to deflection limitations, many flexural elements might behave comparably to a rigid barrier, a characteristic imposed horizontal force, representing a vehicle collision, should include also parameters pertinent to the dynamic nature of the load.

An apparent attempt in that spirit is reflected in the British Standard Code of Practice, *CP 3, Chapter 5, Amendment Slip No. 3*[1]. It requires that barriers in a car park should be designed to withstand a horizontal force at a height of 0.375 m, calculated from:

$$P\,(\text{ton}) = \frac{\dfrac{\frac{1}{2} W_C V_O^2}{g}}{U_C + U_S} \qquad (1)$$

where: W_C = car weight;
g = gravity acceleration (m/sec.^{2})
V_O = impact velocity (m/sec.)
U_C = car deformation (m)
and U_S = structure deformation (m)

For the design of car parks for vehicles not exceeding 2.5 tonnes, the following values should be substituted in Eq. 1.

W_C = 1.5 tonnes
V_O = 4.47 m/sec.
U_C = 0.1 m, *unless better evidence is available,*

while for heavier vehicles the appropriate weight should be substituted.

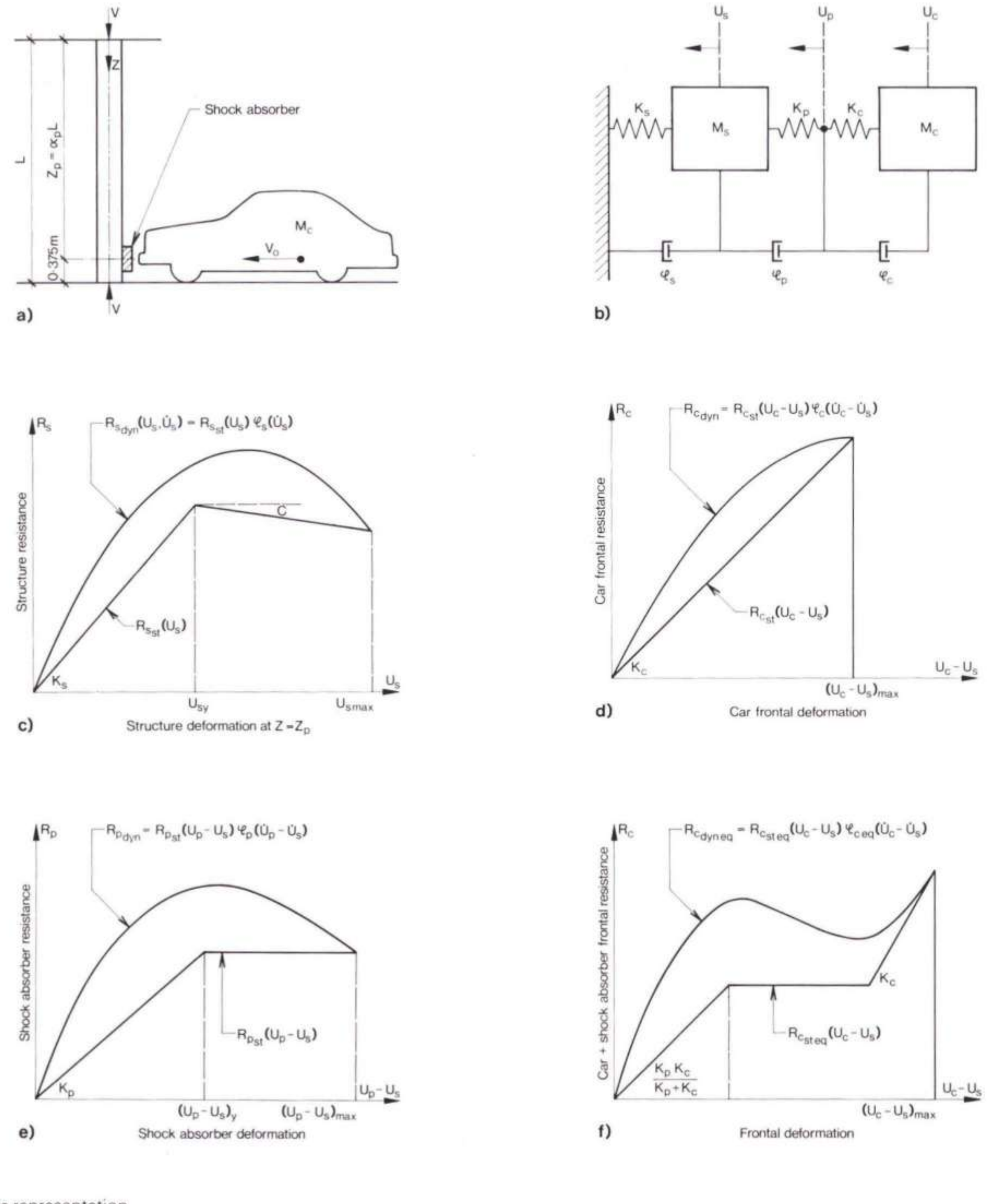

Fig. 1
Car/structure representation

For a rigid barrier and 1.5 tonne vehicle, the required horizontal force is 15 tonnes. It is also mentioned that the design may be also beyond serviceability limits of the materials.

The purpose of the paper is to establish an alternative method, which, though not lacking in simplicity, will be more comprehensive and include:

1 dynamic properties of the structure, like mass, stiffness and inclusion of high speed straining
2 further evidence of car deformation
3 inclusion of shock absorbers
4 load factors pertinent to structural behaviour to vehicle impact

In order to investigate these points, a simplified non-linear two degrees of freedom mass-spring system was used, backed both by quantitative and qualitative information supplied by vehicle collision experimental reports.

The analytical model

Most modern passenger cars have the following common features:

1 A shell type light gauge metal skeleton with high axial but little flexural resistance
2 Lack of steel frame chassis
3 A compact steel block power unit, including engine and gearbox, attached to the car skeleton by low resistance joints (such as the engine mountings and the propeller shaft)

Though this represents a highly difficult structural system to analyze, Ref. 4 reports good analytical results with a model of five lumped masses and ten springs, for both high and low velocity collisions against rigid barriers.

By using a numerical interpretation of collision test results, published in Refs. 5, 6 and 7, a simple car model of one lumped mass and spring was established on the following experimental evidence:

1 During low velocity impact, most of the kinetic energy of the total mass is dissipated by large plastic deformation of the car frontal section.
2 The static frontal resistance may be approximated by a linear spring with no recoverable deformation, as a result of a progressively expanding local buckling failure.
3 The static equivalent frontal stiffness is nearly proportional to the weight of the car, for most types.
4 The corresponding dynamic frontal stiffness is a predictable function of the rate of strain.

The shock absorber, assumed to have a negligible mass, may be included as a modification of the car frontal resistance function.

The simplified structural member representation, based also on the conventional mass-spring analogy is modified to account for experimental evidence of rate of strain factors for dynamic loading of reinforced concrete and steel members (see Ref. 2).

The two degrees of freedom, non linear model of the vehicle-structure impact (Fig. 1(a)) is shown in Fig. 1(b).

The parameters of the structure model are: U_S = the horizontal displacement at the location of the collision with the vehicle; K_S = the equivalent spring stiffness; φ_S ($\dot U_S$) = the rate of strain function; $\dot U_S$ = velocity; and M_S = the equivalent mass.

The vehicle model is represented by: U_C = the horizontal displacement; K_C = the frontal spring stiffness; $\varphi_C(\dot U_C-\dot U_p)$ = the rate of strain function; $\dot U_C-\dot U_p$ = the relative vehicle-shock absorber velocity; and M_C = the total mass.

The shock absorber, interposed between the vehicle front and the structure, is included by means of the parameters: U_p = the displacement of the face in contact with the vehicle front; K_p = the spring stiffness; $\varphi_p(\dot U_p-\dot U_S)$ = the rate of strain function.

The structural model (Fig. 1(c)) includes also the effect of the vertical force V on plastic deformation, by means of a 'negative' stiffness C, defined by

$$C=\frac{V}{\alpha(1-\alpha)L} \tag{2}$$

for a constant section column, where: $\alpha = Zp/L$; Zp = bumper height; and L = length of column.

The static and dynamic resistance function of the structure, car front, shock absorber and modified car front are shown in Fig. 1 (c, d, e, f), respectively.

The equilibrium equations of motion may be written as

$$R_{Csteq}(U_C-U_S)\varphi_{Ceq}(\dot U_C-\dot U_S)+M_C\ddot U_C=0 \tag{3}$$

$$R_{S_{st}}(U_S)\,\varphi_S(\dot U_S)+M_S\ddot U_S+M_C\ddot U_C=0 \tag{4}$$

and the boundary conditions

$$U_S(t=o)= \quad \dot U_S(t=o)=o \tag{5}$$

$$U_C(t=o)=o;; \quad \dot U_C(t=o)=V_o \tag{6}$$

The linearized set of Equations 3 and 4 was solved by means of a step by step analysis, using a third degree polinomial for the finite differences approximation of the first and second time derivatives, of the type: (see Ref. 8).

$$\begin{aligned}\dot f_j&=\frac{1}{\Delta t}\left(\nabla+\tfrac{1}{2}\nabla^2+\tfrac{1}{3}\nabla^3\right)f_j\\&=\frac{1}{6\Delta t}(11f_j-18f_{j-1}+9f_{j-2}-2f_{j-3})\end{aligned} \tag{7}$$

$$\begin{aligned}\ddot f_j&=\frac{1}{\Delta t^2}\left(\nabla^2+\nabla^3\right)f_j\\&=\frac{1}{\Delta t^2}(2f_j-5f_{j-1}+4f_{j-2}-f_{j-3})\end{aligned} \tag{8}$$

where: ∇ is the backwards finite differences operator; $t=j\Delta t$; and f is any function of time.

The stability of this numerical method was proved in Ref. 9 for any time interval Δt.

Numerical examples, solved by means of an IBM 1130 computer program, covered a wide range of parameters, like vehicle weight, frontal stiffness and impact velocity together with the flexural member mass, stiffness, axial load and maximum elastic and post-yield deflection.

A set of results for the maximum elastic response of flexural members under the impact of a 1.5 tonne car at 4.47 m/sec. initial velocity is shown in Fig. 2. The maximum horizontal resistance developed in the flexural member (P_{eq}), referred to as an equivalent static load, is plotted on the basis of the corresponding elastic deformation of the structure (U_S), for equivalent member weights ranging between 0.5 tonnes to 5.5 tonnes.

These results show:

1 A pronounced dependence of the equivalent static load upon the weight and stiffnes of the structure ($K_S=P_{eq}/U_S$)
2 Higher equivalent static loads than those prescribed by Ref. 1. If the results are plotted against the parameter $\Omega_S^2/1000g$, where

$$\Omega_S^2=\frac{K_S}{M_S} \tag{a}$$

Fig. 3 is obtained. This is the basis of a simple curve fitting formula for the horizontal load to be used in the design of the structural member, and shown in the following section.

Design recommendations

The following proposed limit state design recommendations for flexural members in car parks, likely to be subjected to low velocity vehicle impact, were worked out on the following principles:

1 The characteristic horizontal load, based on the elastic deformation of the structure, is the product of three parameters:
   (a) The maximum dynamic load against a rigid barrier
   (b) The dynamic amplification factor
   (c) The correction factor for the shock absorber.
2 The partial load safety factors take into account up to 50% of the plastic potential of the structure to absorb energy.

When reinforced concrete columns are designed for high axial loads, there is a tendency to use a small concrete cross section with a high percentage of longitudinal reinforcement.

More often than not, in such cases the design results in columns with no significant plastic ductility and potential danger of brittle failure; though the following recommendations penalize that design by a higher load safety factor, some additional restrictions should be imposed, similar to the requirements for flexural members designed to resist earthquake loads.

The proposed design recommendations are meant to be used as a supplement to Ref. 1, and the recommended design loads should not in any case be less than required in that reference.

1 The vertical structural members in a car park, located in the areas designed for parking or access to parking purposes and referred to in the following as barriers, should be designed in accordance with the following clauses.

The barrier shall be designed with a horizontal characteristic imposed load F, distributed over a contact area of 0.4 m height and any length

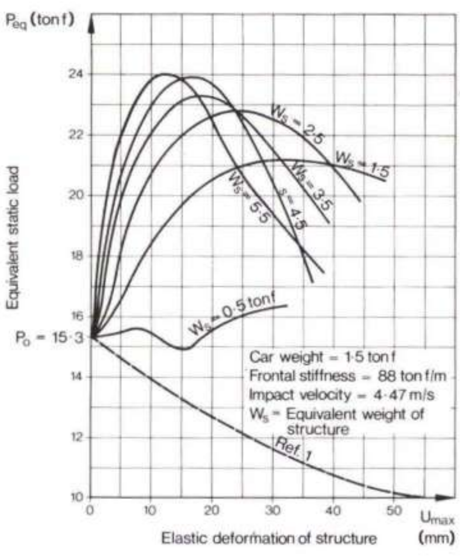

Fig. 2
Equivalent load/deformation diagram

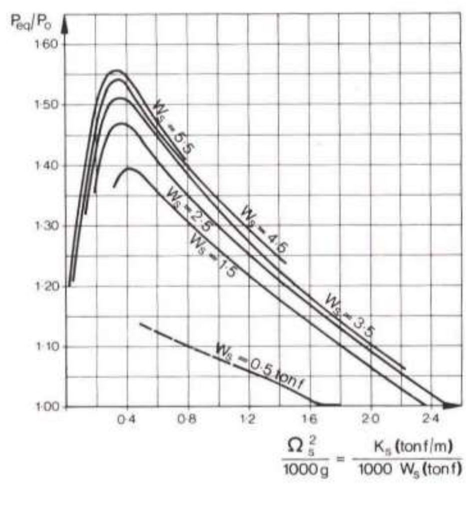

Fig. 3
Transformed relative equivalent load diagram

between 0.4 m and 1.5 m (but not more than the width of the barrier), where

$F = F_o D S$ tonne

F_o = maximum dynamic load on a rigid barrier
D = dynamic amplification factor

S = shock absorber correction factor

2 The maximum dynamic load on a rigid barrier shall be calculated from

$F_o = 2.28 W_c V$ tonne
W_c = Weight of the vehicle, in tonnes
V = Velocity, in m/s

3 The dynamic amplification factor shall be calculated from

$$D = 1 + 0.42 W_{se}(2 - 0.25 W_{se}) \frac{K_s}{1000 W_{se}}$$

if $\frac{K_s}{1000 W_{se}} \leq 0.33$, or

$$D = 1 + 0.14 W_{se}(2 - 0.25 W_{se})$$
$$(1.15 - 0.45 \frac{K_s}{1000 W_{se}}$$

if $2.55 \geq \frac{K_s}{1000 W_{se}} \geq 0.33$, or

$D = 1$

if $\frac{K_s}{1000 W_{se}} \geq 2.55$

K_s = the stiffness of the barrier, in tonnes/m
$W_{se} = K_m W_s$, in tonnes
W_s = total weight of the deflected part of the barrier, not exceeding 4 tonnes
K_m = geometrical weight correction factor, calculated on the basis of spring mass analogy (a table for the basic cases should be added)

4 If the barrier is protected by a shock absorber, covering uniformly the area from the finished floor level up to twice the bumper height, the shock absorber correction factor will be calculated from

$$S = 1/\sqrt{1 + U_p/U_c}$$

U_p = the deflection of the shock absorber under the force F_o, distributed over the contact area

U_c = the deflection of the vehicle, calculated from $U_c = 0.141V$, in metres

If the force F_oS is greater than the yield resistance of the contact area of the shock absorber, the correction factor should be calculated from

$$S = \sqrt{1 - \frac{F_y}{F_o}\left(\frac{2U_{pl} + U_{py}}{U_c}\right)}$$

U_{py} F_y = the initial yield deformation and yield resistance of the shock absorber, in metres and tonnes, respectively.

U_{pl} = the post yield crushing deformation of the shock absorber, in metres.

The shock absorber protective layer should be designed such that it will attain its maximum deformation without losing stability and sustain the load F_oS on half the contact area without local crushing.

5 Where the car park has been designed on the basis that vehicles using it will not exceed 2.5 tonnes, the following values shall be used to determine the force F

W_c = 1.5 tonnes
V = 4.47 m/sec.

6 Where the car park has been designed for vehicles exceeding 2.5 tonnes

W_c = the actual weight of the vehicle
V = 4.47 m/sec

7 The horizontal force shall be considered to act at bumper height. In the case of car parks intended for motor cars not exceeding 2.5 tonnes this shall be taken 0.375 m above the finished floor level.

8 The partial load safety factor γ_f to be used with the characteristic imposed load F shall be determined from the following considerations

$$\gamma_f = 1.5 \left[\sqrt{\left(\frac{U_{sm}}{3U_s}\right)^2 + 1} - \frac{U_{sm}}{3U_s}\right]$$

U_s = the deflection of the barrier under the load F

U_{sm} = the post-yield plastic deformation of the barrier at which structural failure will occur.

If the failure of the barrier is not likely to cause extensive damage to the serviceability of the remaining structure, γ_f should not be less than 0.5

If the failure of the barrier is likely to cause extensive damage to the serviceability of the remaining structure, but not collapse, γ_f should not be less than 1.0.

If the failure of the barrier is likely to cause collapse of the remaining structure, or part of it, γ_f should not be less than 1.5.

9 The materials safety features are 1.5 for concrete and 1.15 for steel members or reinforcement.

10 The load F may not be included in the usual serviceability check of the barrier.

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