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The Open University

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Front cover: Symbol of the Open University (Reproduced by courtesy of the Open University)

Back cover: Paradise Circus reference library – The circular cast iron staircase from the old library building erected in the local studies library. Four coffers were made solid where the staircase penetrates the floor. (Photo: Logan Photography)

The Open University

Frank Coffin

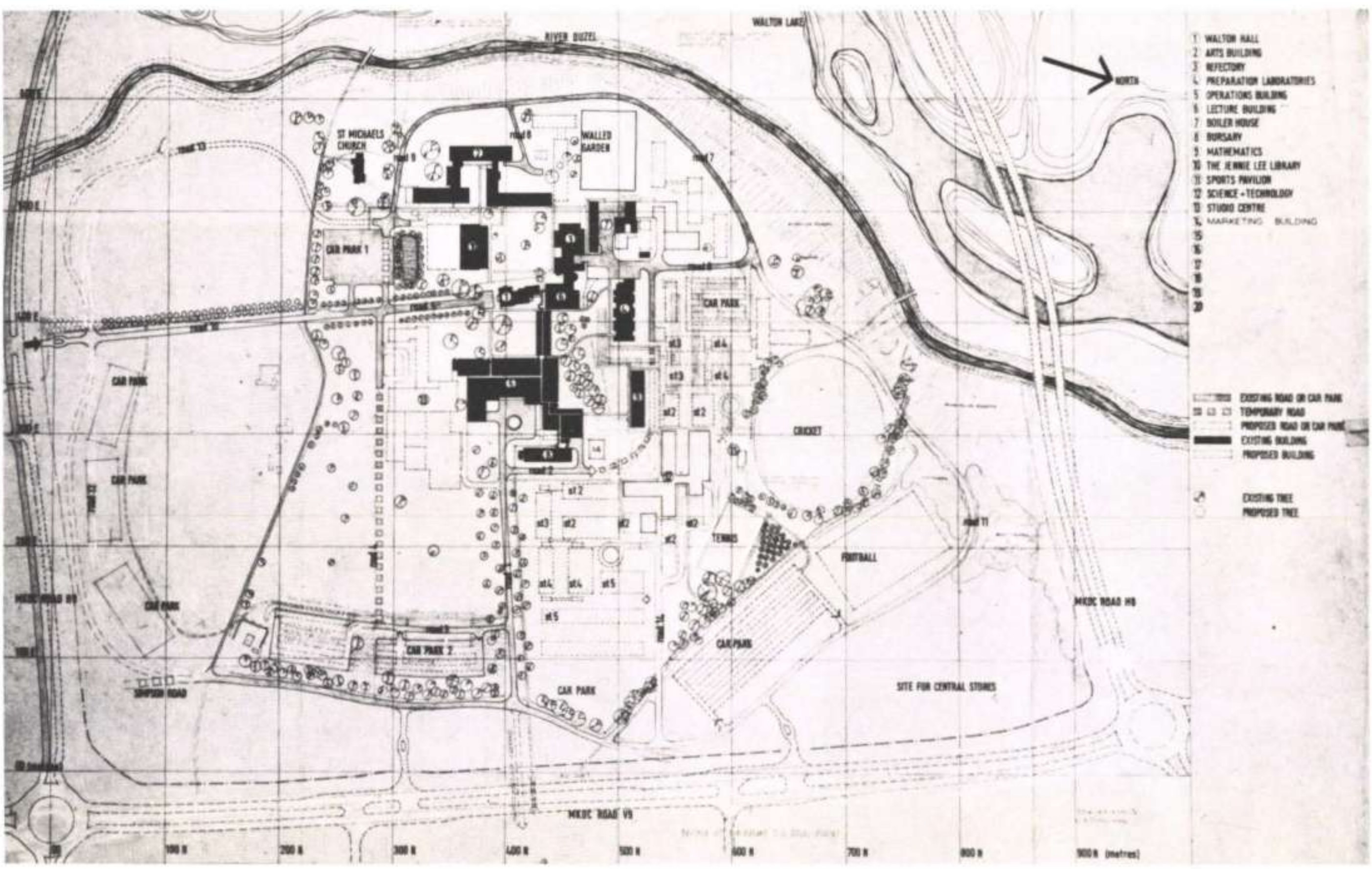
The beginning

The idea was first suggested by Harold Wilson in a speech in Glasgow in 1963. When he became Prime Minister in 1964, he asked Jennie Lee, now Baroness Lee of Asheridge, to become the first Minister of the Arts. He also asked her to investigate that idea for making higher education more widely available by use of technological developments. This was for an open university teaching through television and radio. She accepted the challenge, and initiated an examination of the potentialities of the scheme which resulted in a White Paper in 1966, *The University of the Air*, the title by which the project became known at the beginning.

A planning committee set up under Sir Peter Venables, now Pro-Chancellor and Chairman of Council of the University, studied the early suggestions. Their report, in early 1969, proposed that broadcasting would be only one of the media by which the university would perform its teaching role. This report gave birth to the Open University as we know it today.

In March 1969, Ove Arup & Partners received a letter from the Secretary, then Secretary Designate, A. Christodoulou, informing us that Fry, Drew & Partners had been commissioned to act as site development architects, and offering us the appointment as consulting engineers for the structural design of the new buildings on the chosen site. It is typical of the speed with which things were happening in those early days that we had already started digging trial holes and concerning ourselves with minor repairs and extensions to an existing building so that it could be made ready as the main reception area and offices for the Vice-Chancellor, the Secretary and the vanguard of the people necessary to develop and manage this new project.

Fig. 1
Architects' Strategic Development Plan



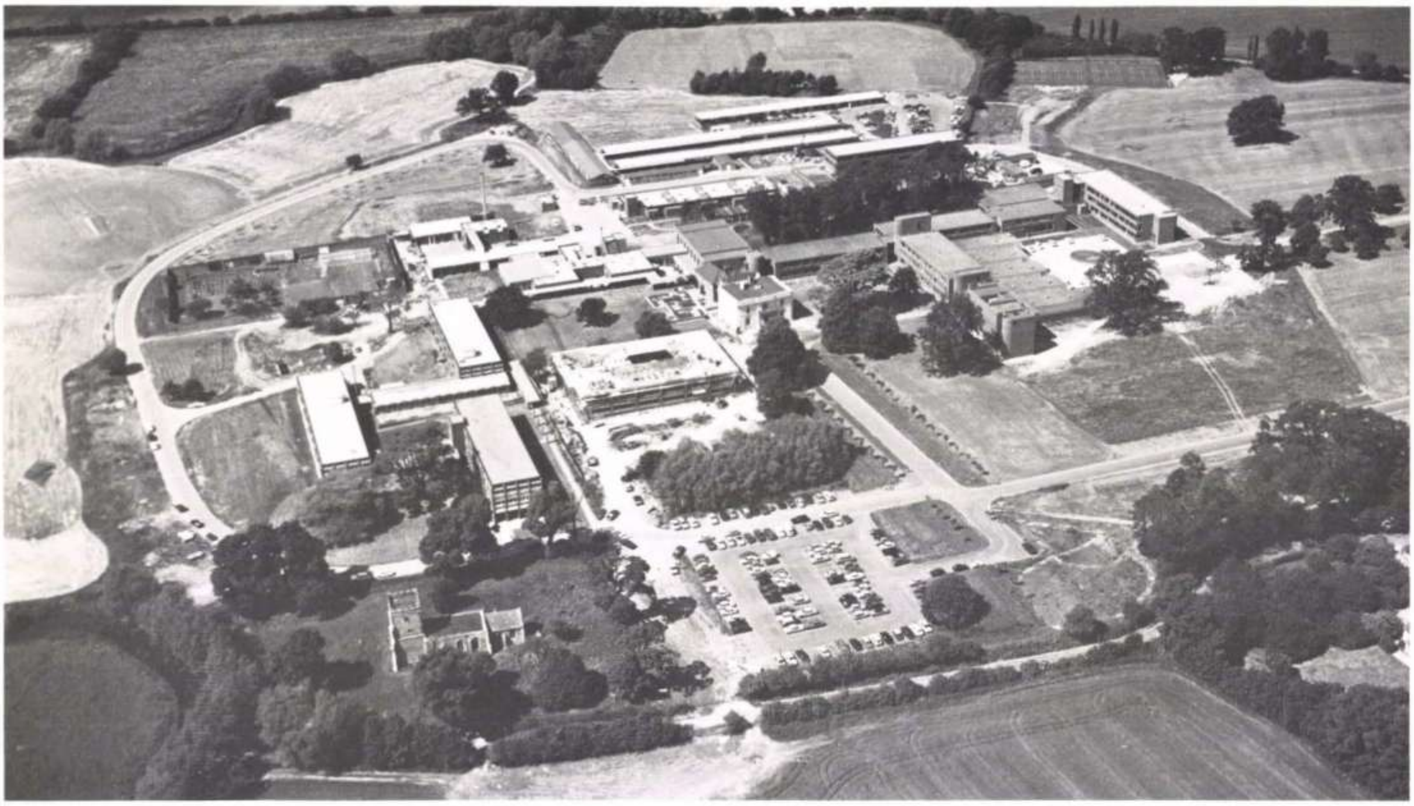


Fig. 2
The Open University site
(Photo: the Open University)

Fig. 3
Walton Hall (Photo: the Open University)

The site

The site chosen was in open countryside, four miles north-east of Bletchley in Buckinghamshire, 50 miles north of London, and in the area designated for the new city of Milton Keynes. Readily accessible from the exit from the M1 just south of the Newport Pagnell service station, it is in an area of gently sloping open fields interspersed with the occasional clump of trees, bushes and hedges. The east boundary is a minor road leading southwards from the original village of Milton Keynes through the small village of Walton. To the west is the small meandering River Ousel, little more than a stream, running through a low marshy area in normal conditions, but liable to flood over part of the western area of the site under adverse conditions.

In the centre of this area, on the crest of a rise a few feet above the surrounding area, was the original building, Walton Hall. Not large by any stately home standards, it was nevertheless a pleasant and quite imposing building set amongst some magnificent trees, forming a reminder of the pattern of life in this area in earlier days. The only other building nearby is St Michael's Church, set amongst trees on the south-west corner at the end of Rectory Lane, which leads from the Rectory on the main eastern approach road and forms the southern boundary to the university site.

On one of our early site visits we noticed some familiar temporary huts and tripod rigs about, and wondered whether Geotechnics were even quicker off the mark than usual in anticipating our needs. Further investigation disclosed that Wimpeys had been commissioned by the Milton Keynes Development Corporation to investigate the subsoil over the line of a proposed deep main trunk sewer to run in a tunnel under the site. By a little hard talking we were able to persuade the Corporation and Wimpeys that a few more boreholes and samples were neither here nor there, and got ourselves a cheap site investigation. We found that generally over the site we had a fair depth of topsoil, up to 0.6 m in places, overlying 2–3 m of soft boulder clay, and then the Oxford clay, which was stiffer but still softish in the upper layers.

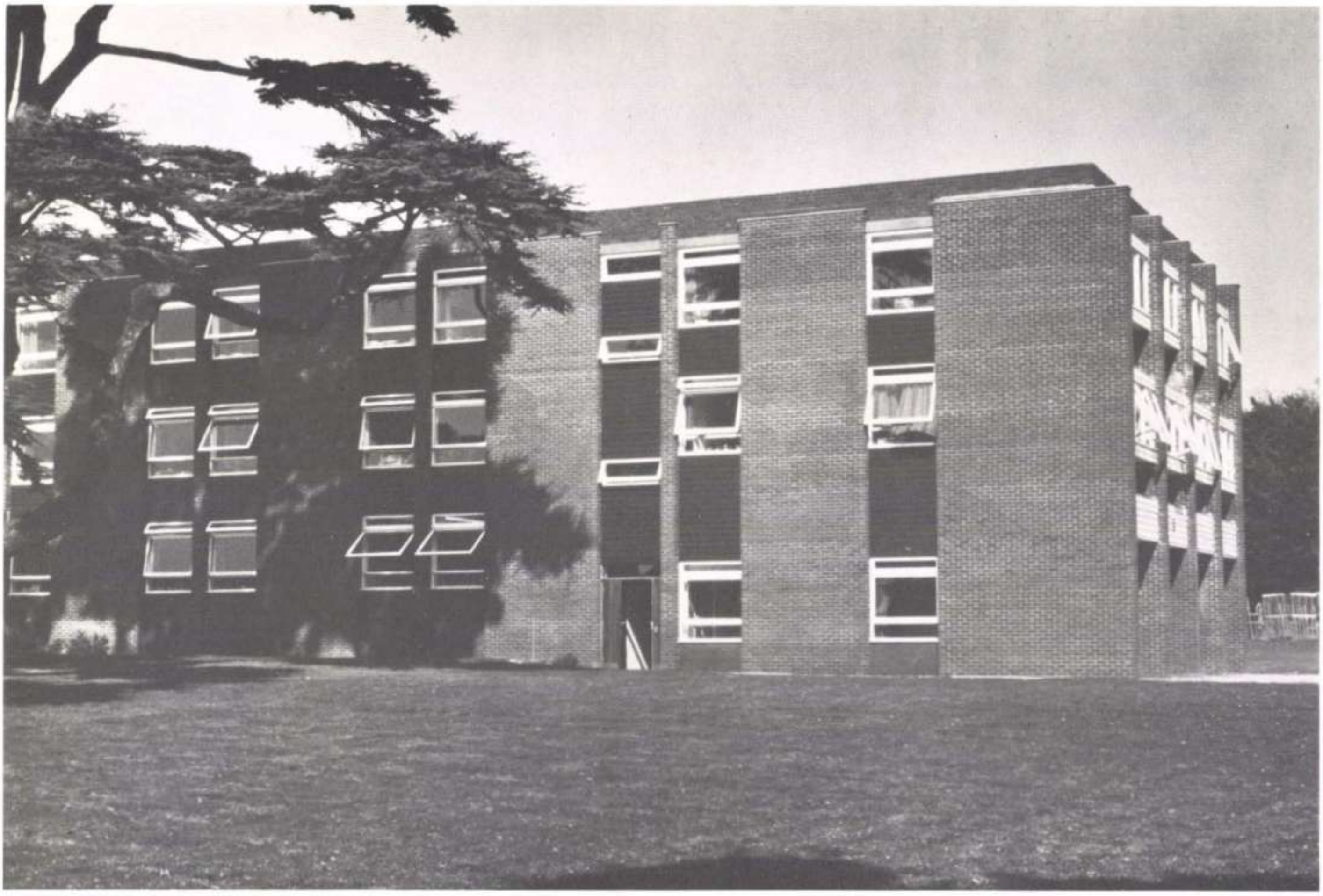
These ground conditions produced no serious problems in the foundation designs for the future buildings. Normal spread foundation were found to be appropriate for all the early buildings, but the depth of the topsoil, and of the soft boulder clay, meant that those foundations tended to be costly because of the depth of dig and the lower ground pressures to which they were designed. Piling is being considered for one of the buildings now under design because of the heavier loadings involved, but the site cannot be considered as difficult in any way from aspects of foundation design.

The first buildings

Two things became apparent to us when we first started working with the architects. The first was that, having decided to embark on such an unusual adventure, everyone from

Fig. 4
The Arts Faculty (Photo : the Open University)

Fig. 5
The Operations building
(Photo : the Open University)



Jennie Lee downwards was desperately anxious to get it off the ground, or on the ground so far as the buildings were concerned, as quickly as possible. The second was that, since no-one else had started an open university before, there was no previous experience to draw on and no-one really knew precisely what facilities were required for such a venture.

It was therefore decided, just before we arrived on the scene, to split the early buildings into two phases. Phase 1 was to consist of the simplest possible buildings, to be constructed of load-bearing brick and proprietary precast concrete floors, based on a straight central corridor plan and relatively shallow offices either side. Together with the first part of the refectory, the boiler house and a small building to act as science preparation laboratories, these were to be built on a design and construct basis to the requirements of Fry, Drew & Partners, by the appointed contractors, Y. J. Lovell Ltd., in the shortest possible time. The second and larger phase was to include the many other facilities which had to be worked out as the design progressed, and which were essential to the early functioning of the new university.

The main Phase 1 buildings, later to be extended to a similar plan and elevational form but an amended structural design prepared by us, became the faculty buildings for arts, social sciences, educational studies, and for the Institute of Educational Technology. In one important respect they set the pattern for the remaining buildings which followed. They are basically low buildings, three storeys maximum, with an elevational treatment relying heavily on brickwork. The Phase 1 buildings were started with a rather harsh red local brick, but on the later buildings a warmer reddish-brown facing brick was used which, in my view, blends well into the environment into which the University is set.

Phase 2

Whilst the first buildings on the site were simple in form, and based on the requirement for relatively straightforward office accommodation, it was in the development of the next and larger phase that the real early needs of this special form of university had to be determined and met.

Inevitably there was a need for more office accommodation, but with more flexibility than the rather rigid structural form the early faculty buildings allowed. It was also apparent that one of the urgent and vital needs of the new University would be the means to process and handle information flowing in and out on paper. It was also evident that there would be lots of it. Finally, there was a need for a place of assembly. The University would not have the need for a number of lecture rooms, for obvious reasons, but it was important to them that they had some accommodation on site which would enable the academic staff and administrative staff to get together for discussion. An assembly hall large enough to accommodate approximately 350 was included in the second phase, but designed as a multi-purpose space to provide recreational facilities for the university staff also.

The Phase 2 buildings occupied a different part of the site, on the east side of Walton Hall.

The visual connection between the two is therefore broken by the previous buildings on the site and some magnificent trees which are being preserved with loving care around the hall itself. This meant, for the designers, that within the overall framework of low rise buildings with a basically brick elevation, they could consider the second phase without being restricted by the rather specialized details of the first faculty buildings.

The administrative and operations buildings were therefore developed as 12.2 m wide blocks, two and three storeys high, interrupted by separate staircase units set at appropriate points along their length. To give maximum flexibility in their internal planning, the structures of the individual blocks are 250 mm deep flat slabs on columns at 6.1 m centres in both directions. The top surfaces of the slabs are floated off to receive a carpet finish and avoid the cost of screeds. The external walls run proud of the outer row of columns and are formed of continuous strip windows with cavity brick below. The external skin of brickwork is carried on a continuous steel angle over the windows. These angles were galvanized and painted with pitch epoxy paint strictly following Turlogh O'Brien's recommendations at that time and turning a deaf ear to the rumblings in other sections of the design team about the cost of these most essential precautions.

Fig. 6
Arts Faculty block
(Photo: the Open University)

These buildings are now basically planned and used on a central corridor theme, with the main services running horizontally and exposed below the slab. The cast-iron down pipes run vertically by centre columns, also exposed. The runs of services were well planned, and despite the lack of false ceilings and enclosing duct-work, the interior gives an impression of order and good design out of austerity. The central corridors now form a main communication link between various different parts of the University, but in some of the areas off this communication route, advantage is taken of the flexible planning arrangement by enlarging the corridor out into open spaces to meet the needs of the particular users.

An essential requirement for the University is the facility to process and handle information pouring backwards and forwards from the students. Inevitably therefore one of the requirements within Phase 2 was a building to house the computer. It was found possible to incorporate this within a wider block, but of similar form to the remainder of the administration buildings. But information transmission and processing also means masses of paper. This means that the Open University is to become one of the largest printing organizations within the United Kingdom, and needs a

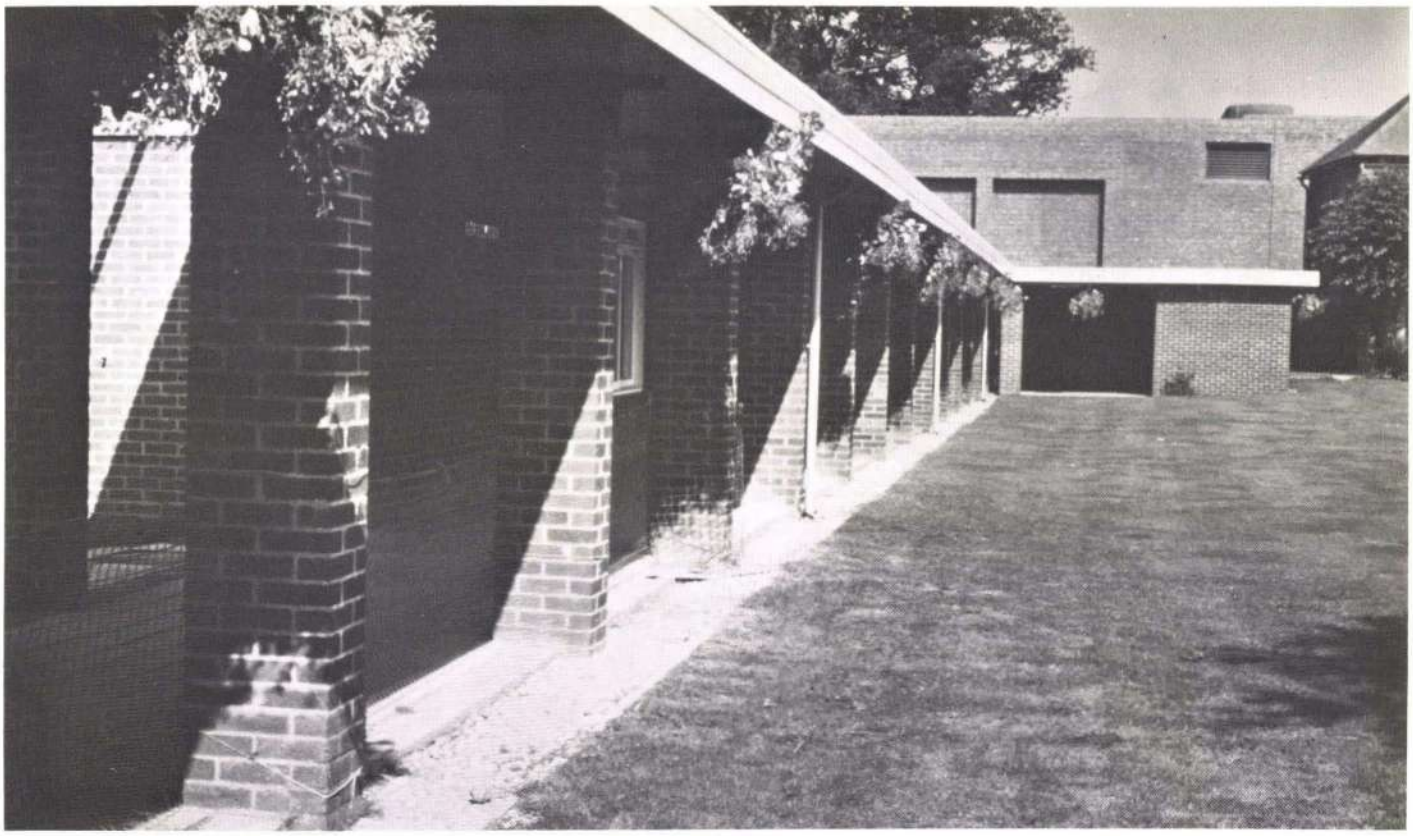


Fig. 7
The Catering block
(Photo: the Open University)

correspondence service and printing department of the appropriate size.

Phase 2 therefore included a series of single-storey buildings alongside one of the main operations blocks to handle incoming and outgoing paper and house the necessary printing equipment. It is these buildings in particular, and the roads and service yard required to feed them, which begin to illustrate the difference between the Open University and the more conventional university scene. The architects were left with the problem of incorporating what are basically industrial buildings within an academic setting. In arriving at a solution, they rejected the standard north light form of roof, and the adopted solution was a series of six 12.2 m bays of single-storey steel framed construction, with brickwork up to gutter level, and equal sloping north and south light glazing between each bay. The resultant buildings blend in well with the remainder of the University site, but inevitably some problems are experienced in the middle of summer by the occupants.

The next phase

By the time Phase 2 was nearing completion, plans were well afoot for extending the buildings already built, or nearly so. Another faculty block was required on the western side of the site, to extend the original Phase 1 buildings. This was built to the same basic design as before, but with a slightly amended structure. Also the existing refectory was extended to cope with the larger numbers now involved in running the University.

Like all good computers, the Open University model, an ICL 1902A, started demanding more data to feed it. At the same time the mathematicians decided that their new building must have more affinity with the computing centre than with the arts and social sciences people on the other side of Walton Hall. Two more blocks of similar construction to the Phase 2 offices were therefore designed and built for data processing and mathematics. The first of these was a near replica of what had gone before, and was tied up to the communicating corridor network and the computer building itself by one of the standard staircase towers slightly amended. Mathematics stands on its own, however, with an internal staircase, which allowed the architect to introduce a variation in the theme at one end at least.

The most important new project in this phase of the work is the library, or at least the first stage of the library. It is perhaps significant, perhaps not that a site was found for it between the arts faculties and the rest, but nearer and connected to the former. It was intended that the library should be built basically in two more or less equal halves separated by an expansion joint. The first half is nearing completion and handover now. It is a simple building, two storeys, approximately 27.4 m by 40 m with columns at 6.7 m centres and a 300 mm solid flat slab. The elevational treatment at ground and first floor follows the same pattern again as the Phase 2 buildings, with cill height brickwork and continuous strip glazing. The edge detail at roof level, however, introduces a new variant in the design in the use of a deep fair-faced concrete fascia beam. It was originally intended that this should be precast, but Y. J. Lovell Ltd. elected to cast it in situ, and have produced a good standard of finish throughout its length.

The temporary end of the first part is probably the least satisfactory elevation of the building, but if the library follows the previous pattern of development as the rest of the site, it will not remain visible for too long. On the same basis, it is also unlikely that the second stage of the building will be the same as was envisaged in the original design, such is the rate of change of requirements of the University as it continues to develop.

One other problem started to arise on site during Phase 2, and during this phase. Whereas the Open University is unlikely to experience the same problems of students driving their cars around the University plain, as occurs elsewhere, they do have other traffic problems, which include the arrival and departure of heavy lorries carrying tons of paper and other supplies. Ove Arup & Partners were therefore asked to extend their brief to include the design of the service area, roads and car parks.

Part of this work had been started to designs prepared by others. Our first task therefore was to link up these to a new road round the western side of the site to produce a circular ring road. Two problems arose here. Because of the proximity of the western part of the River Ousel, the route took it inevitably through the area liable to flooding. At the same time, and arising partly out of its location, this was an area with a fair depth of topsoil overlying a very soft clay. Frank Fowler and Ray Ham have been concerned with the design and supervision of the construction of this road and it is performing well. The University are aware that at its lowest point it is liable to flooding, but have accepted this as a calculated risk.

Some trouble was experienced with the service yard adjacent to the single-storey correspondence and printing department. It was apparent that the traffic in this area was both heavier and greater than had been anticipated originally. Again Frank Fowler produced a design for the reconstruction of this area, which is one of the focal points for heavy traffic on the site. This has been completed to form a first class service yard in this area.

The immediate future

At present the facilities on site tend to be weighted towards the faculties of arts, social sciences, educational studies and mathematics. We are at present engaged on the design of the first stage of the science and technology building, to be located on the north-east corner, beyond the computer area and mathematics building. Not only will this building house the staff of this faculty, but it will also provide a research laboratory for those staff and for their research students.

The additional servicing requirements arising from laboratory use mean that the structure of this building will partly break away from the pattern developed on Phase 2 with a closer spacing of columns along the length of the blocks to allow greater penetrability of the horizontal structure. This will undoubtedly be reflected in the external treatment, but the basic principles of low-rise construction with a basically brick treatment will be followed.

The first stage of the science and technology building is planned to be followed by two further stages, which in terms of the present planning will make it one of the largest of the faculty buildings on site.

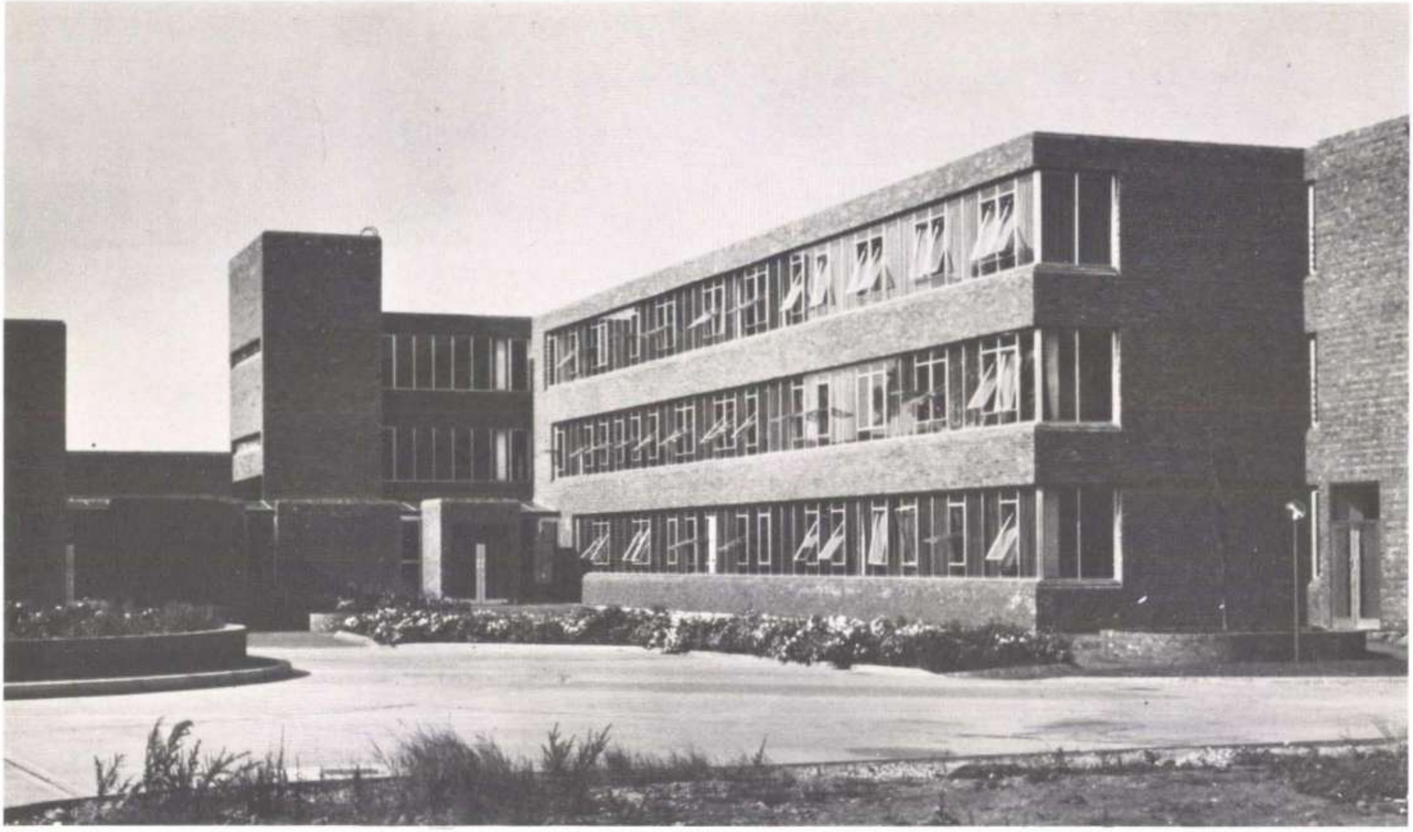


Fig. 8
Administration building
(Photo : the Open University)

Although it was initially conceived as 'the university of the air', radio and television in fact occupy only a small amount of the student's time, and the most important element in the system is a correspondence package which most of the students receive weekly. The television and radio programmes forming part of the course are recorded at the BBC studios at Alexandra Palace. The lease for these studios runs out in 1977, and the University has to find other accommodation in which to produce its programmes. It was decided therefore to build new studios, with the co-operation of the BBC, on the University site. The architects appointed for these studios are the Ware MacGregor Partnership, who have considerable experience in television and broadcasting work. The preliminary designs have been prepared, and it is hoped to start construction in March 1975. The scheme includes an extension of the Phase 2 office blocks in similar construction, to provide the inevitable requirement for more office accommodation. This leads into a large one- and two-storey studio complex more than 200 feet square on plan, housing the two studios and an additional small drama and music studio, the control rooms, rehearsal rooms, property and scenery stores, and all the various conference and technical accommodation required to process and package sound and television programmes so that they can be delivered to transmitting stations and then on to the students over the air.

The technical requirements for the individual parts of the studios are so specific that they tend to dominate the architectural and structural design. The height of the buildings and their elevational treatment will be compatible in form with the existing buildings on the site. By its very size, however, the studio complex will undoubtedly form a dominant feature of the site.

As structural engineers for the studios, our aim from the beginning of the design process was to produce a structure flexible in use, but keeping as much out of the way of the inevitable services, air handling ducts and electrical and electronic hardware as possible. The required column spacing produces a more rectangular grid. To maintain a flush soffit, give easy fixings for cables and allow the maximum penetrability of the floors, we are developing further on the ideas which Nigel Thompson started on York Hospital of a flat slab with *Lignacite* blocks inserted where the heavier concrete is not working hard. This form of structure will be used over the technical areas wherever it is appropriate. Over the property and scenery stores the roof will be in lightweight construction supported by steel. The studios themselves require special treatment for acoustic reasons, and here a double skin construction is being considered with *Siporex* autoclaved lightweight concrete supported by structural steelwork forming the roof over.

The future

One of the exciting things about working on the Open University site is that the present is always quite different from the earlier forecasts. When we first started it appeared to us that we were building something not very different from a quiet university campus around Walton Hall. As the Phase 2 buildings, and subsequent extensions, came into the picture, the scene appeared to change subtly, as the size of the operations required to process and distribute the packages of learning material to the students became apparent.

Undoubtedly, the completion of the studio complex will change the scene again. But already the need is becoming apparent for a large warehouse on the site, which may well be under way whilst the studio is being built, so the future pattern of the whole site continues to change to the extent that it remains as uncertain as before.

I should perhaps emphasize finally that this article has been written solely from the point of view of one involved only in the buildings at Milton Keynes, from the beginning. Through its widespread regional offices and study centres, the total University is much wider than those buildings suggest. Based solely on student numbers, with 38,000 at present, the Open University is now the largest university in the country. This is obviously an invidious comparison, because its research and other activities cannot begin to compare with the others. But when one considers that the first members of staff were appointed in January 1969, the royal charter was presented in July 1969 and undergraduate teaching started in January 1971, then the progress to date of the Open University can be seen in rather startling perspective.

Fig. 9
Catering block (Photo : the Open University)

Paradise Circus

Ernie Irwin

Introduction

The Paradise Circus complex forms the heart of Birmingham's Civic Centre. It is situated between Birmingham's Town Hall and Art Gallery on one side and the City Council's Administration Centre, Baskerville House, and the new Repertory Theatre (Job No. 1976) on the other side. The complex consists of a major new reference and lending library, a school of music, lecture theatres, offices, shops, public houses and public areas. These are arranged over a bus interchange and underground car parking. The bus interchange in turn is built over the city's new Queensway Ring Road which passes through the site in an underpass (Figs. 1–3). All of these buildings are being constructed within Phase A of the project and the last of this stage will be completed when the old Birmingham Library on the site has been demolished. The new libraries and school of music have now been opened (Figs. 4–6).

Design of the project commenced in 1964; preliminary construction started in 1968, with the main contract for the buildings commencing in 1969. Detail design continued until 1971 and in all some 3,000 structural drawings were produced for the project. It is an extremely diverse project to describe and this article is necessarily lengthy. Consequently, the paper has been divided into sections as listed below for easy reference for those with interest in special aspects of the work.

(1) Introduction
(2) Structural materials
(3) Foundations
(4) Underpass and gyratory road
(5) Concourse
(6) Reference library block
(7) Lending library block
(8) Link blocks
(9) School of music
(10) Contract arrangements
(11) Setting out
(12) Acknowledgements.

The north west end of the site, known as Phase B, will consist of further underground car parks and other civic buildings, but no programme is yet established for that stage.

Planning for comprehensive development of the site for civic use commenced in 1964, although the general traffic plan to run the A38 through the underpass and form a gyratory road around the site was already determined.

The main library was located in order to avoid it sitting directly over the underpass, although many of the ancillary buildings are necessarily on the underpass route and there are in general three levels of different functions across the main part of the site, i.e. bus interchange, concourse and library.

The various buildings have different uses and accommodation needs and accordingly a particular construction system was chosen for each separate building type. Different structural systems were economically viable due to the large size of each individual building, but in the service areas the structural grids were determined both by the building above that area and by the bus interchange layout.

The total building works were valued in excess of £8m. in 1968 and the Phase A building project was let at a value of £5m. and the roadworks at £1.5m. in that year.

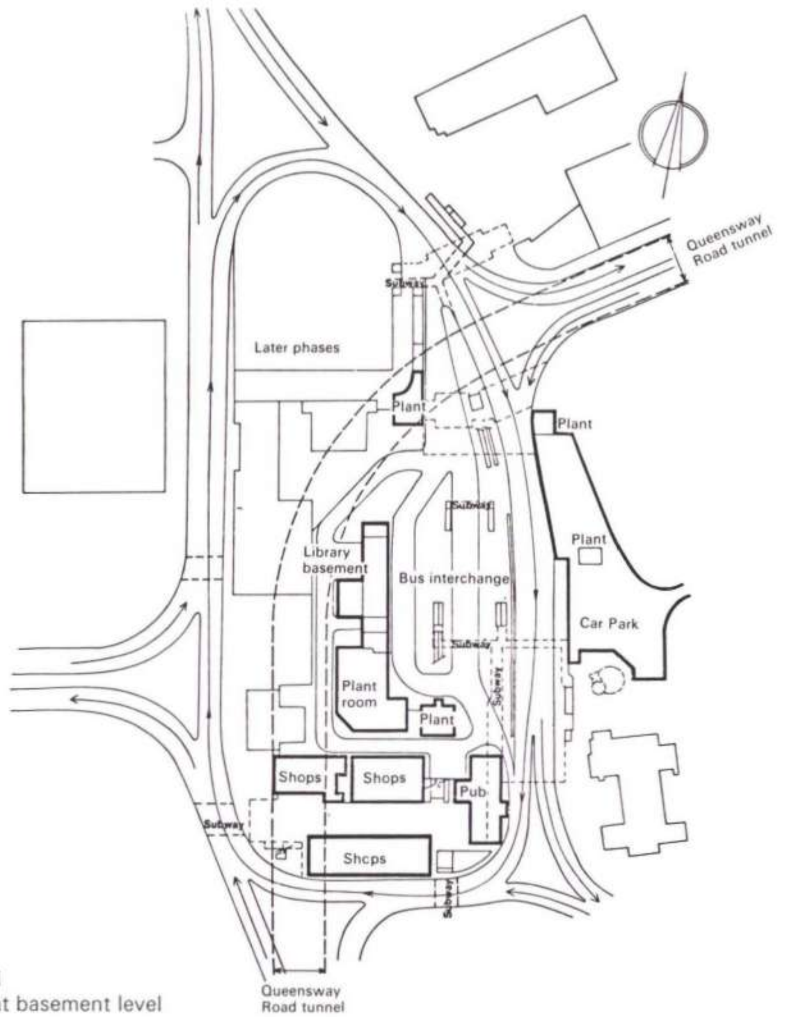


Fig. 1
Plan at basement level

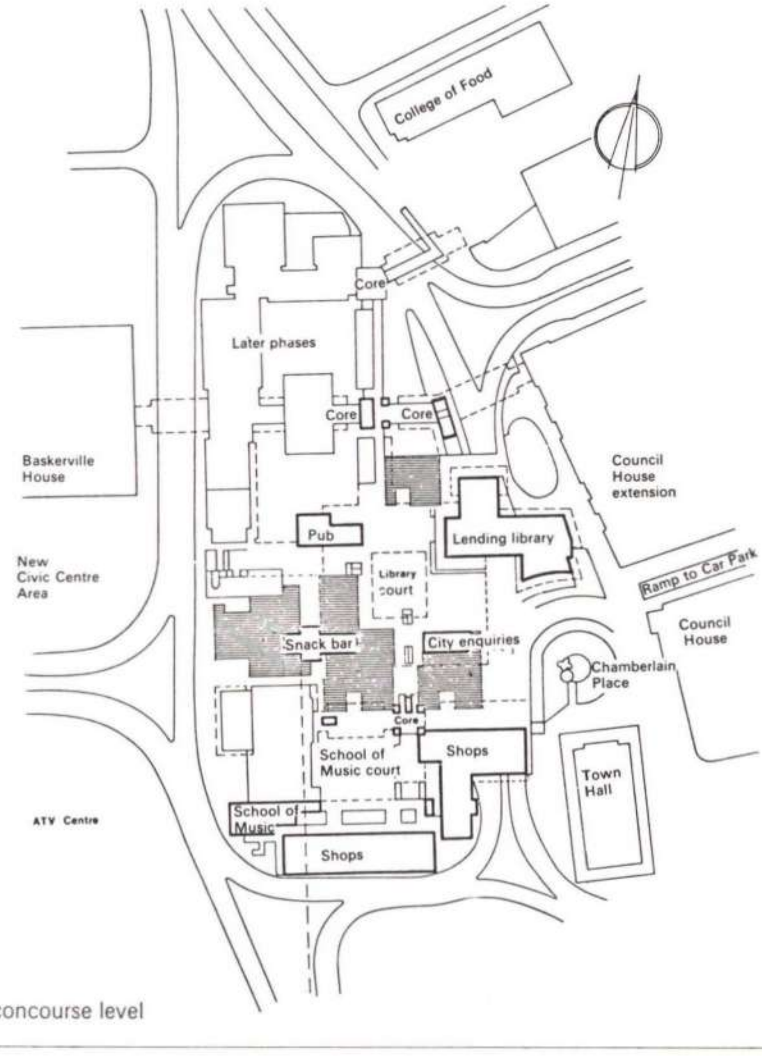


Fig. 2
Plan at concourse level

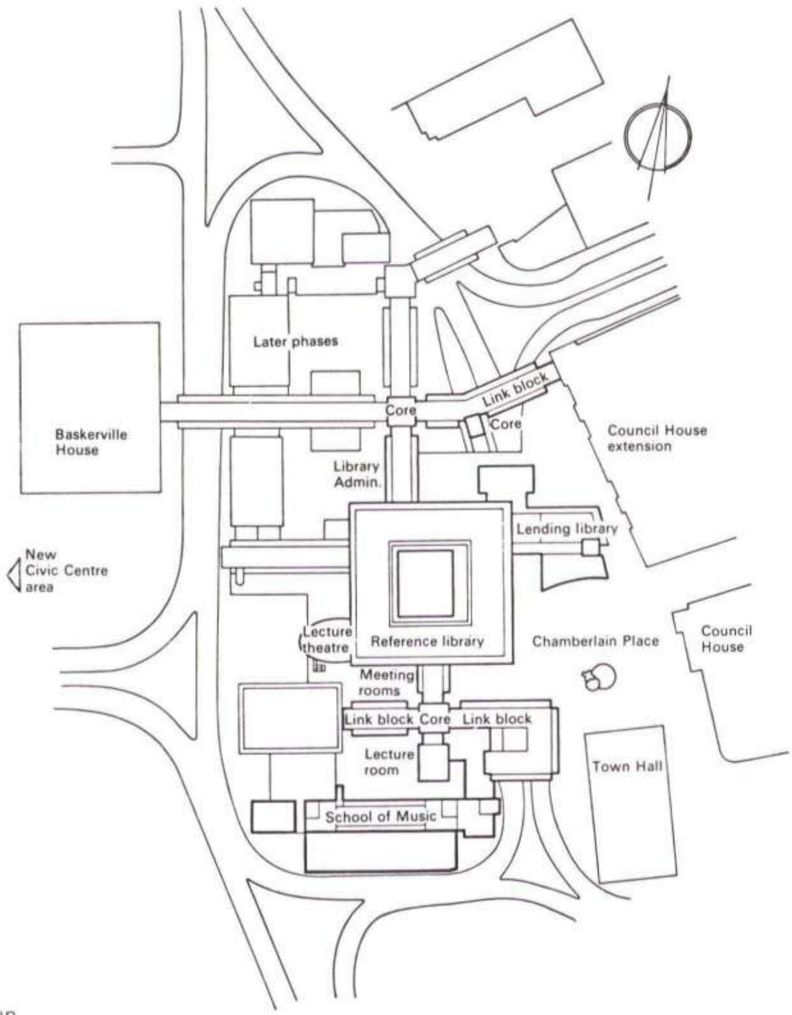


Fig. 3
Roof plan

Structural materials

On a project so large and diverse most structural materials were used. However, notwithstanding the fact that several hundred tons of structural steel were incorporated, the buildings are primarily reinforced in situ concrete with precast concrete cladding. Almost everywhere within the buildings the concrete is fair faced and exposed to view.

All in situ concrete for the building is 30 N/mm² compressive strength at 28 days, and was made from a Trent Valley 20 mm natural aggregate. The concrete was site mixed to ensure consistency of strength and colour, as the colour was critical due to columns and most walls being grit-blasted. In addition stocks of aggregate were stored at the pit to allow supplies to continue when variations occurred in the material.

Four types of concrete finish were employed:

(1) Plain concrete from plywood forms used in service areas, below ground and on hidden surfaces

(2) Grit-blasted concrete to approximately 3 mm depth on most vertical surfaces throughout the libraries and link blocks

(3) Grit-blasted ribbed concrete on service core walls

(4) Smooth fair faced concrete from glass reinforced coffer moulds on reference library floors.

Very consistent concrete was obtained in terms of strength and colour, but dark blemishes occurred on a number of grit-blasted columns due to knot hole repairs in the plywood, or the vibrator touching the formwork.

Fig. 4
View of libraries looking west from the archway over Edmund Street. The main entrance is through the lending library in the foreground

In all cases joints in the plywood show, and a better arrangement of plywood joints would have improved the appearance in many cases.

Precast cladding and roof tiles were manufactured using a Hopton Wood limestone mixed with Derbyshire Spar with white cement and these were lightly grit-blasted so as to simulate the Portland Stone used on the existing civic buildings.

Reinforcement was generally high tensile steel with mild steel links, although building bye laws limited stresses in foundation reinforcement to those of mild steel. Structural steelwork included plate girders approximately 2 m deep.

Foundations

The site is founded on Birmingham 'rock sand', which is a non-cemented sandstone capable nevertheless of high bearing pressure. The rock sand had been proven by the Local Authority's boreholes taken for the Inner Ring Road.

It was decided to found all the buildings on pad footings at a bearing pressure of 750 kN/m². However, preliminary excavation of the underpass below the site revealed the presence of thin inclined strata of marl rising from north to south across the site. In some cases these layers occurred immediately below foundation level. As a result the foundations were lowered where feasible and in other cases the foundation pressure was reduced to 430 kN/m².

When the marl layers had been discovered further site investigation was quickly undertaken with one of our engineers continually present, and by careful observation it was possible to detect the layers of marl which would and did escape a normal investigation.

Fig. 5
View of reference library with Baskerville House and Hall of Memory in foreground

Fig. 6
School of music with shopping centre beneath. Note recital room adjacent to busy road with underpass beneath

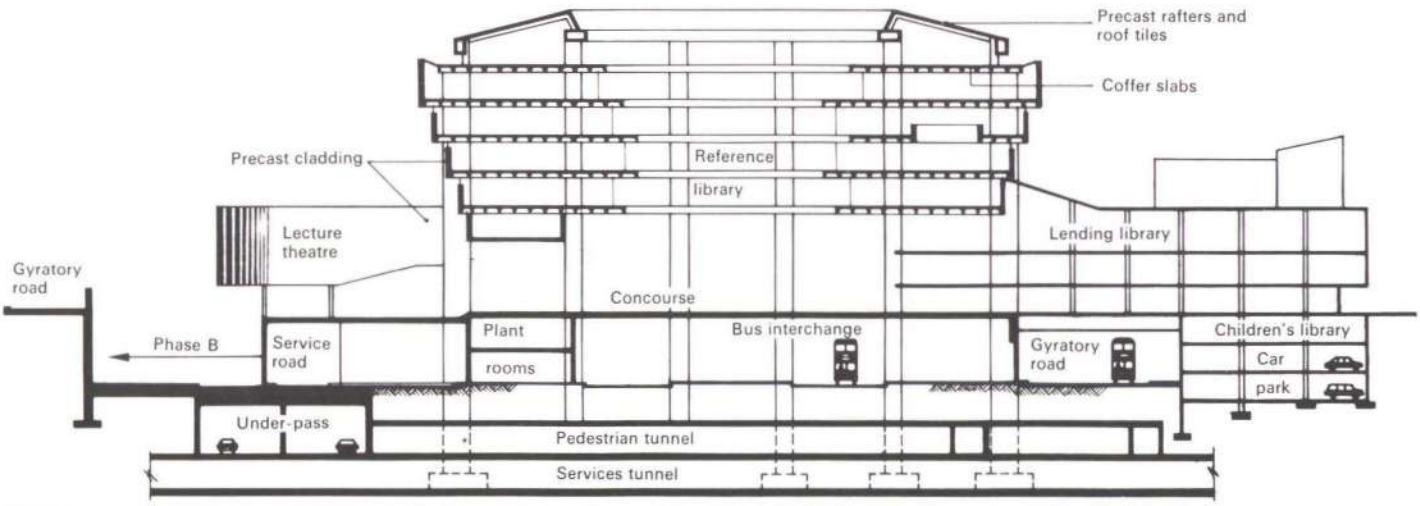


Fig. 7
Paradise Circus – Phase A. Structural section

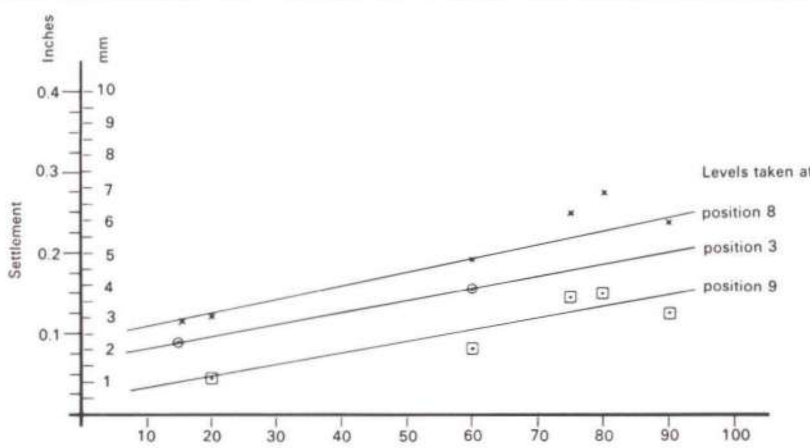


Fig. 8
Reference library load/settlement graph

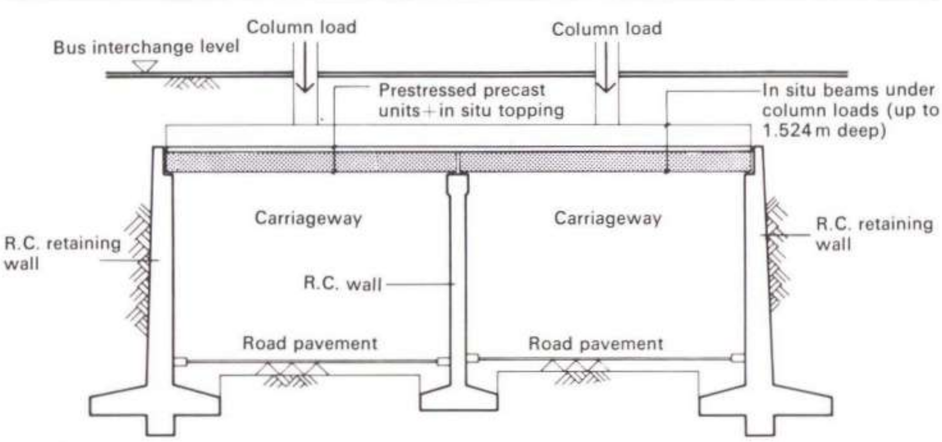


Fig. 9
Underpass cross-section

From the information found the strata were plotted across the site and all further foundations were adjusted accordingly.

Brass settlement plugs were placed in some of the columns at bus interchange and concourse level as the project commenced and accurate settlement measurements have been observed throughout construction.

The average settlement on completion is approximately 4 mm and a load settlement graph is shown on Fig. 8.

Rock sand, though uncemented, was found to remain stable in deep excavations without shoring, even when exposed for several months. However, we insisted on protection of the banks of the 14m deep excavation around the existing library which was to remain open for several years. Grout-covered hessian was used to prevent erosion by run-off of surface water.

Underpass and gyratory road

The new Birmingham Inner Ring Road was planned through the site as an underpass below the bus interchange level. It was designed as cantilever retaining walls built in open cut with standard prestressed precast inverted tee beams and an in situ topping forming the roof (Figs. 9 and 10).

While the main library was sited so as to avoid the underpass, many of the ancillary buildings were necessarily supported on it and in these locations large in situ reinforced concrete beams were used to carry the column loads to the main retaining walls of the underpass. The maximum load carried is 9000 kN under one of the lift cores which supports two link blocks. With experience of this construction, it would likely have been better had the underpass been designed as a double box culvert in reinforced concrete over the length of the site. This was not possible as we were given authority to make only local alterations to the standard Public Works Department prestressed beam design.

Provision has been made on the underpass roof, and adjacent to it, for loads from Phase B of the project.

The other major road affecting the site is the new gyratory road which forms the north, south and west boundaries, but on the east side dips through the complex at bus interchange level and beneath the lending library and concourse. Fig. 7 illustrates the relationship of underpass, gyratory road and main buildings.

Fig. 10
Underpass during construction

Walls and columns in reinforced concrete were constructed on either side of the gyratory road to provide support for the concourse areas and lending library above it.

Prestressed precast concrete beams with in situ concrete topping span over the gyratory road below the concourse area. Where it passes under the lending library, 1.2 m deep steel beams were used in groups of three to support the building which varies in height from three to five storeys.

Construction depth at this point was limited to 1.2 m because the levels of the gyratory road could not be adjusted without seriously affecting the road layout, nor could the levels of the lending library alter without affecting the total building development.

The problem arose of how to carry a reinforced concrete building with core walls rising to five storeys over 15.2 m span with an available depth of 1.2 m without excessive deflection. An additional complication was the short time available for constructing this deck as it was sandwiched between the road being made available by the Public Works Department and the requirement for it to be opened to traffic a number of weeks later.

The solution found was to use *Autofab* steel beams in groups of three which were united by steel diaphragms at 1.37 m centres and flange cover plates (Fig. 11).

Braithewaite & Co. Ltd. fabricated the triple beams completely at their works and brought them to site as 30 tonne units.

The last portion of the gyratory road is presently on a temporary alignment and will be completed as soon as the old library is demolished.

The road construction and the underpass structure were designed by the Public Works Department but the amendments to the underpass structure to enable it to carry the building loads and the structures around the gyratory road were designed by ourselves.

Concourse

The entire development is served by an extensive multi-level public concourse at general ground level affording access across the site as well as to individual elements of the project. It is intended that in addition to general use by the public, the concourse should be used for mounting of exhibitions including those of sculpture. A further requirement is that this level should be capable of supporting fire appliances.

The grid of columns supporting the concourse is basically the 10.87 m square grid derived for the reference library and extended outwards. The pattern becomes irregular, however, where the column positions are dictated by other buildings above or by plant room and service areas below. These parameters, together with the incorporation of ornamental pools, called for the adoption of substantial design live loads. A solid 450 mm deep reinforced concrete flat slab was chosen as the typical structure for this area. However, over the gyratory road the concourse is constructed of prestressed bridge beams and with structural steel as described in the section dealing with the gyratory road.

Two of the main cores providing lifts and stairs through the library commence at concourse level because of the bus route immediately below. These cores attract a very substantial loading through the library structure and consequently major supporting beams are required at concourse level. Due to the presence of internal walls in the main core, a grid of beams was necessary to collect the loads and these beams were designed using a grid analysis taking a torsional beam stiffness of 50 per cent of the theoretical. Even with this factor extremely high torsional moments were induced. The resulting beam system is 2.4 m deep and Fig. 12 shows some of the reinforcement within these beams.

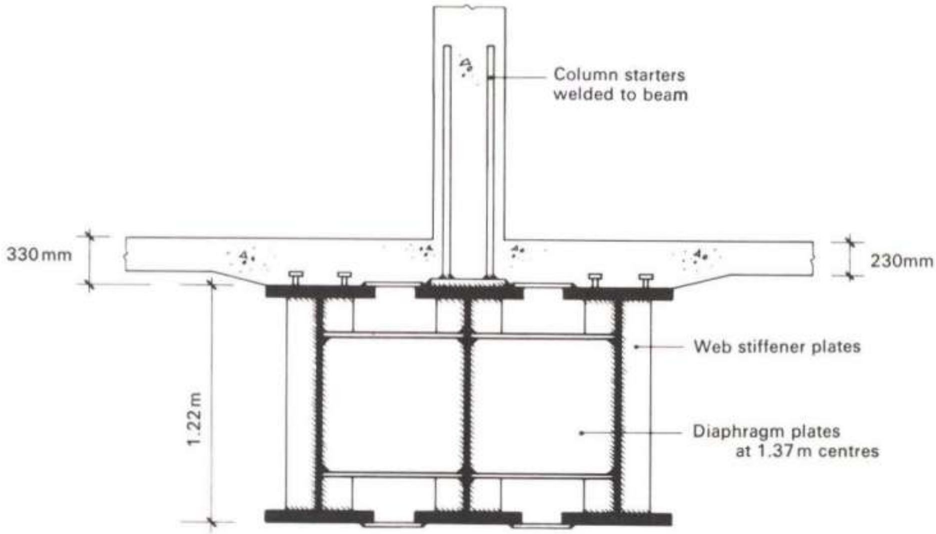


Fig. 11
Section through triple girders spanning New Congreve Street

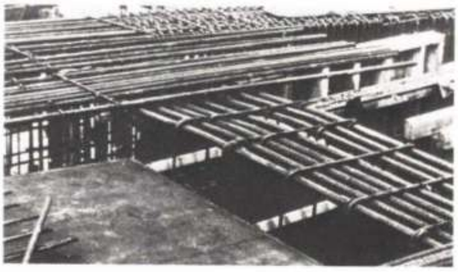


Fig. 12
Concourse area – View of reinforcement in 2.67 m deep beam grid which carries one of the library service cores

Fig. 13
South east corner of reference library with Big Brum, the Council House clock tower, in the background

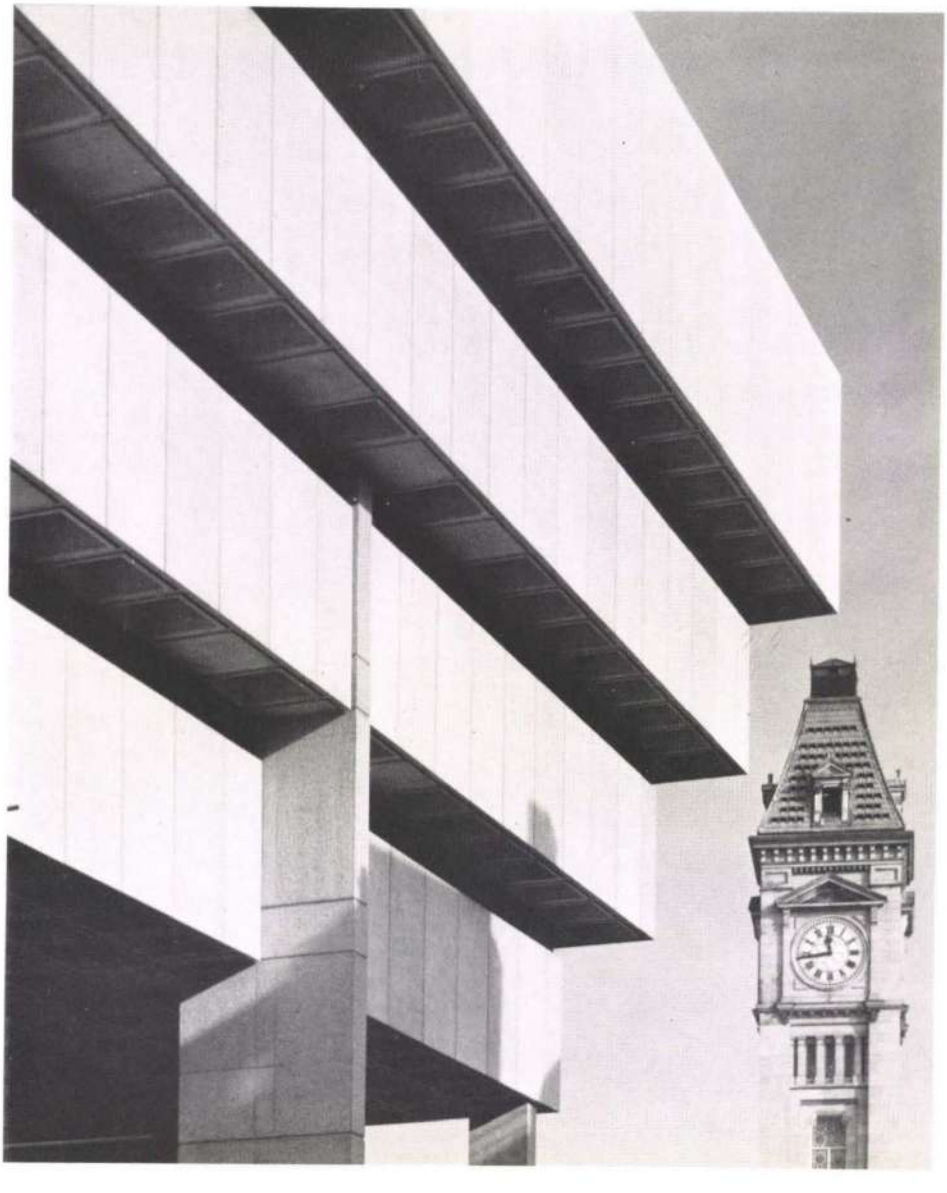


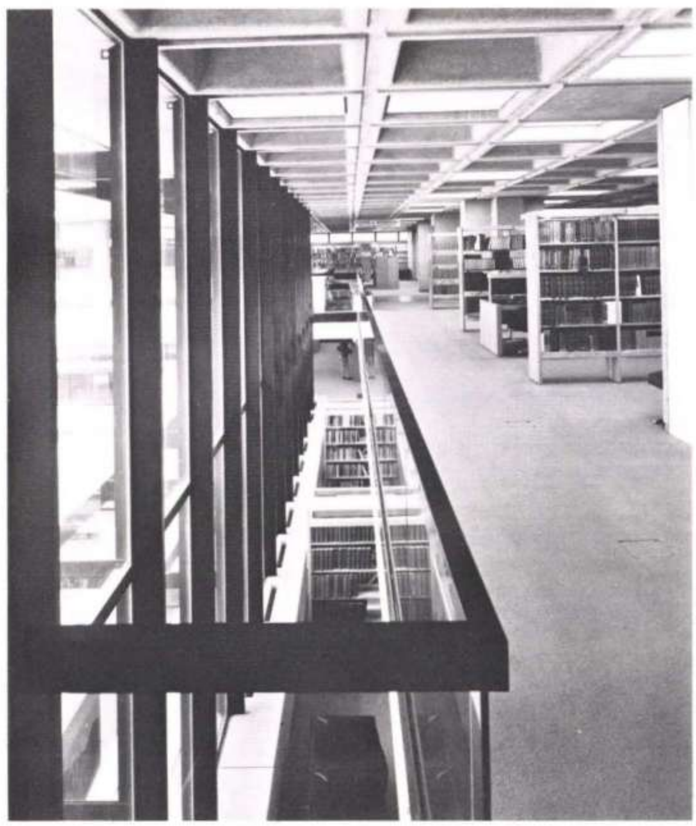
Fig. 14
Reference library – Social science reading area looking down through floor void from fine arts

The position of the central void in the library varies from floor to floor (Fig. 16), so that cantilevers of various lengths occur on the inner perimeter of the building in addition to cantilevers on the outer perimeter at the upper storeys. These cantilevers are subject to heavy and concentrated loads, particularly in book stack areas and at external corners, and required a rigid floor structure with good load spreading characteristics.

A coffered floor slab system satisfied all these various constraints and in addition suited the basic square grid while providing a fair faced soffit. Consequently, a 2 ft. deep coffered slab of 4 ft. 6 in. module was chosen for all the main library floors (Fig. 17).

The individual coffers, each 4 ft. 6 in. square, are a non-standard size but, due to the repetition involved, it was an economical proposition to have glass reinforced plastic moulds purpose-made for the project.

Fig. 15
Reference library – Early printed book and fine printing area view through to the local studies library. Air conditioning duct within grit blasted column, right foreground



The entire concourse when completed will be some 230 m long and as most of it is exposed directly to the elements, considerable thermal movement was anticipated. Due to the complexity of the concourse area with pools, steps and emergency access roads, it was difficult to introduce movement joints in a regular pattern. In general, movement joints have been located at approximately 40 m centres in each direction while the main library, which is 60 m square at concourse level, has been isolated by a movement joint around its entire perimeter.

Reference library block

The structure of the library block was determined by many factors, but the major influences were the inverted pyramidal shape of the building (Fig. 13), the central void through each floor and heavy floor loadings, particularly in book stack areas.

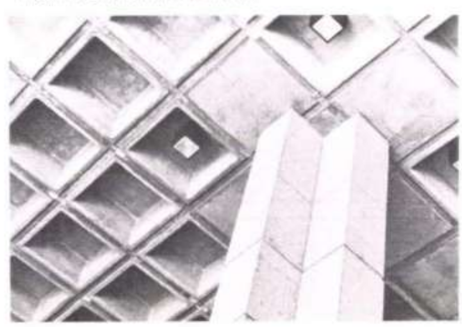
The library is designed throughout with a common module of 1 ft. 6 in. with column spacings of 18 ft.* in the lending library and 36 ft. in the reference library. Book presses are spaced at 4 ft. 6 in. centres in the closed access areas. The shelving and consequently the internal panelling is at 3 ft. centres for maximum economy of shelf design. The building elements, wall panels, windows and coffered floors are on a 4 ft 6 in. grid which is a satisfactory size for the constructional design of these elements and is suited to the aesthetic scale of the building. This dimensional co-ordination produces a strong architectural unity between the building structure and the interior furnishings and fittings (Figs. 14 and 15).

A basic column grid of 36 ft. in each direction was adopted as it suited both the library planning as well as the bus interchange area beneath, without being excessive structurally.

* This project was designed in Imperial units. The common module size was 1 ft. 6 in. and other relevant dimensions given are multiples of this.

Fig. 16
Reference library – Science and technology library looking down to the history library. The double storey height glazing faces on to the central void

Fig. 17
Reference library – View of the underside of the coffer slab from the public concourse area showing the junction with one of the main cruciform columns



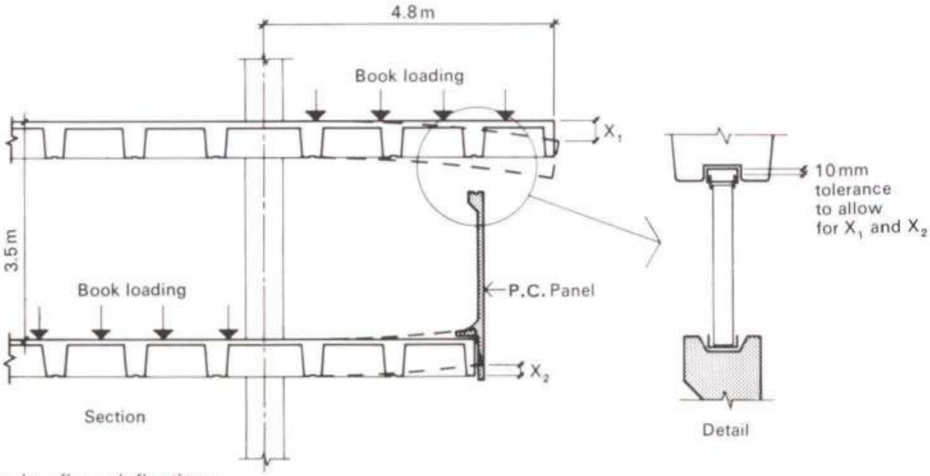


Fig. 18
Reference library section showing floor deflections

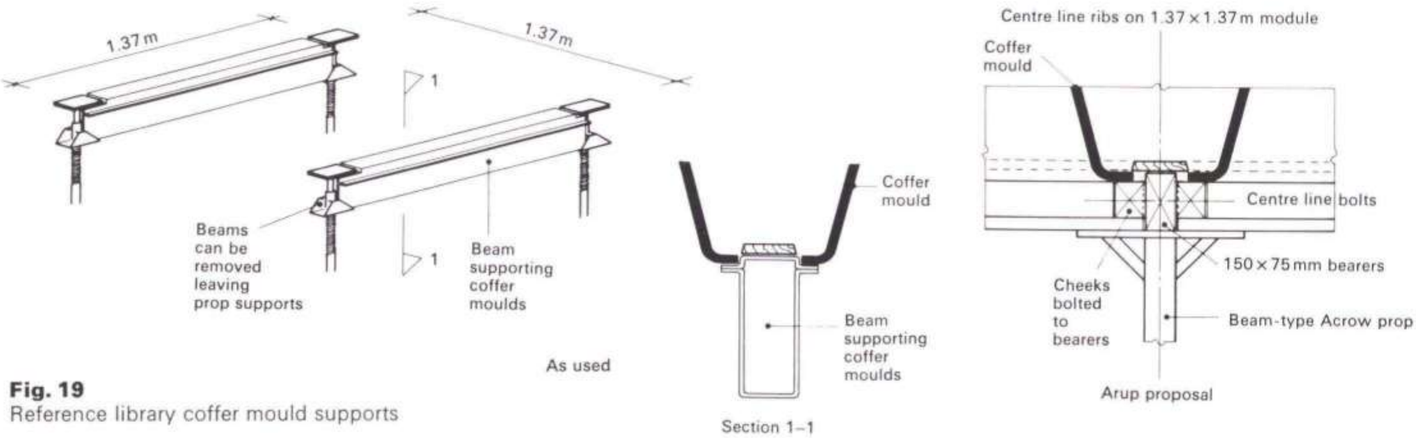


Fig. 19
Reference library coffer mould supports

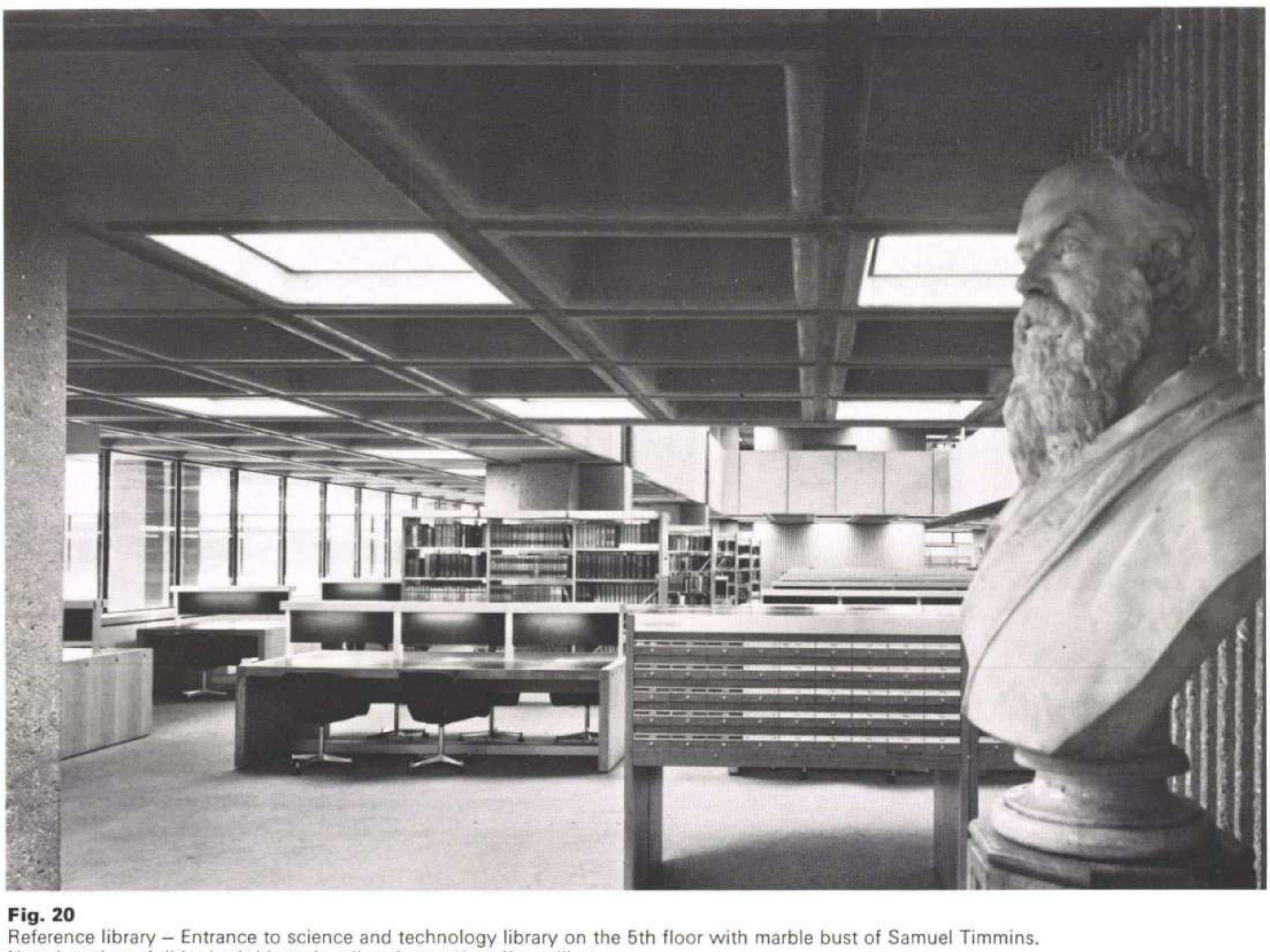


Fig. 20
Reference library – Entrance to science and technology library on the 5th floor with marble bust of Samuel Timmins. Note junction of ribbed grit blasted wall and smooth coffer ceiling

The coffer floors, while giving an impression of uniformity, embrace structural arrangements that are extremely varied as no two floors are alike, due to the varying cantilevers on both inner and outer perimeters. The floors were analyzed for bending shear and deflections on a computer, using a grid analysis, and the unit area taken was the half width of the building by two column bays long. To incorporate this area each pair of ribs was taken as a single grid line and further analyses on a smaller grid were carried out around the column heads where shear stresses are high.

The coffer voids are used to house the electric lights and the electric service feeds down through the coffer topping.

The cantilevers around the outer perimeter reach a distance of 5 m at floor six which, on the four corners of the building, is a cantilever of 6.7 m. As a consequence substantial cambers were needed on these cantilevers and at the corners this reached 50 mm.

In addition, conditions of live loading could cause the floors to open or close relative to each other and it was necessary to detail for this movement, both in the partitioning and in the clerestory windows (Fig. 18).

Due to the special construction and cost of the moulds it was appreciated that they would be required for re-use before the concrete could be depropped. We devised a method of release, as in Fig. 19, in advance of the contract. In the event the contractor used a system which permitted the entire propping beam to be removed by leaving a prop at the junction of the ribs. This method, however, allowed grout leakage at the junction of the fibreglass mould and timber insert which we were unable to eliminate throughout the job, and, unfortunately, the very smooth coffer surfaces had to be touched up at their junction with the recess in the rib. The specification laid down strict requirements for rigidity within the moulds and a sample mould was carefully examined. The very regular coffer ceilings which were achieved are to a large extent due to the quality of the mould that was thus obtained.

The recess within the rib of the coffers serves several purposes and it is carried throughout the library coffer floors, even in locations where the coffers were made solid due to high shear or other reasons. The primary purpose of the recess is to house partition heads and allow differential movements. In addition it masks the junction between coffer moulds. Around the perimeter of the building the windows are also housed in the rib recess (Fig. 16).

The typical superimposed load was 4.8 kN/m², but this increases locally to 11.2 kN/m² in areas of rolling book stack loading.

Even though this relatively high loading was required on 36 ft. spans, the storey height, and thus the structural depth, had to be minimized for planning reasons. Consequently, the choice of a 2 ft. deep coffer slab was a particularly happy one in that it eliminated the need for any structural down stands within the relatively short floor to floor height of 3.5 m (Fig. 20).

The cantilever precast cladding to the outer perimeter is shown in Fig. 21. Each unit was fixed to the floor slab with two stainless steel bolts with levelling nuts, and erection using diagonal props went very smoothly. Additional fixings to give continuity between precast units were fitted through the lifting holes that had been cast in the side flanges. The inner perimeter precast cladding is shown in Fig. 21a.

The libraries are air-conditioned using a high velocity induction system. The air for the building feeds vertically from the roof through ducts formed within the cruciform columns. Two legs of each cruciform are hollow and it was intended that the hollow shells should be precast. However, in order to obtain matching concrete the contractor elected to cast the projecting legs in situ 100 mm thick and this was successfully achieved.

At each floor level air is fed horizontally to the centre of the rooms by ducts passing through the coffer ribs (Fig. 22). Although it is not generally desirable to pass service ducts in a structural zone, the holes formed a simple direct route, identical at each column location. It was carried out without difficulty apart from the structural analysis and made a considerable saving in height in the building.

The plant rooms, together with archives and staff areas, are located within a raked hipped roof. The roof is constructed of precast concrete rafters spanning between in situ concrete ring beams around the outer perimeter and inner void. The rafters are spaced at 1.37 m centres and are clad with white concrete tiles

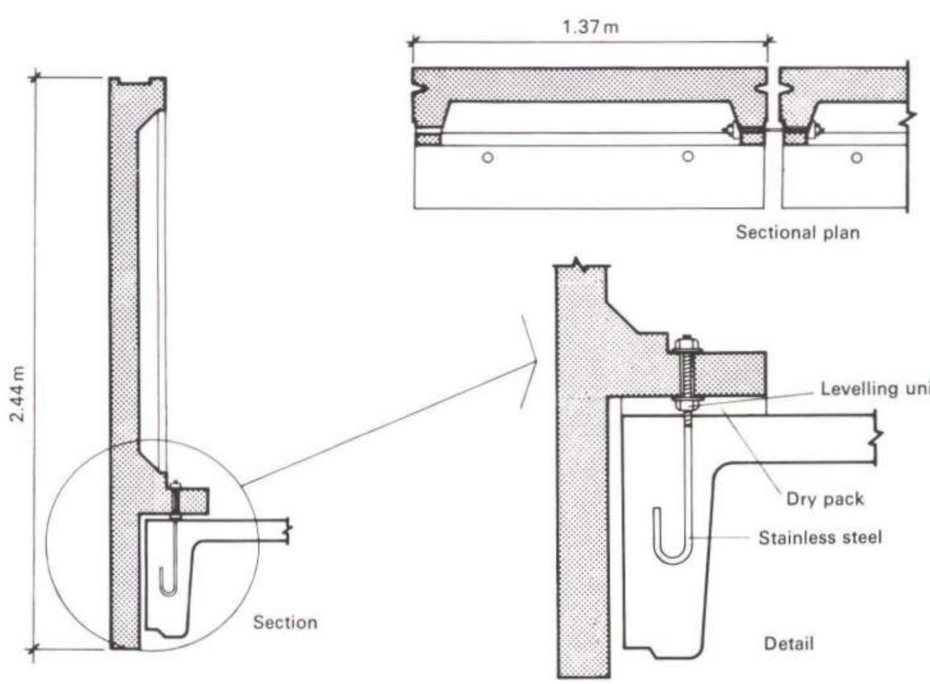


Fig. 21
Cantilever precast panels on reference library

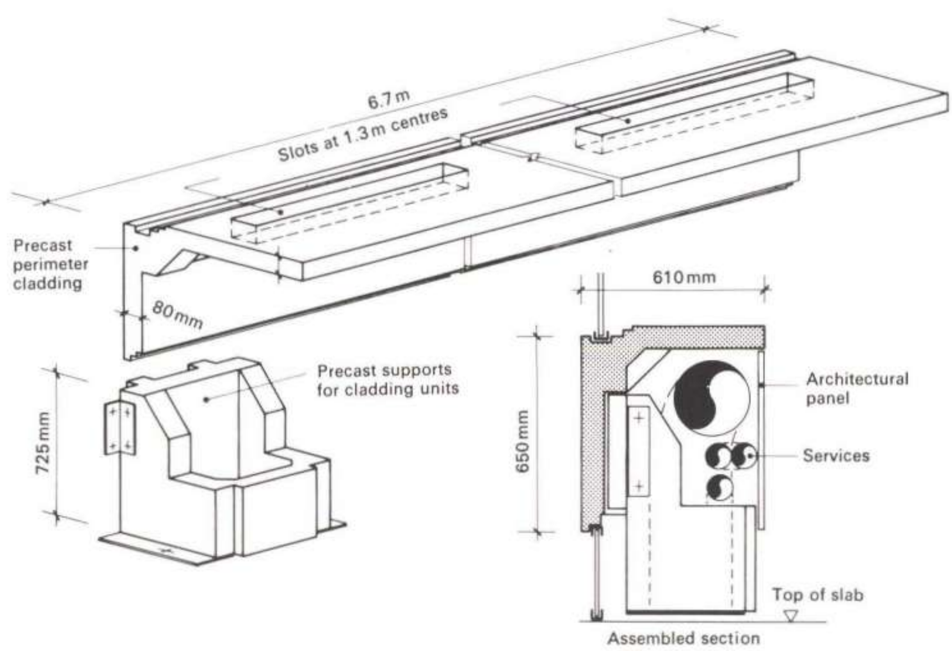
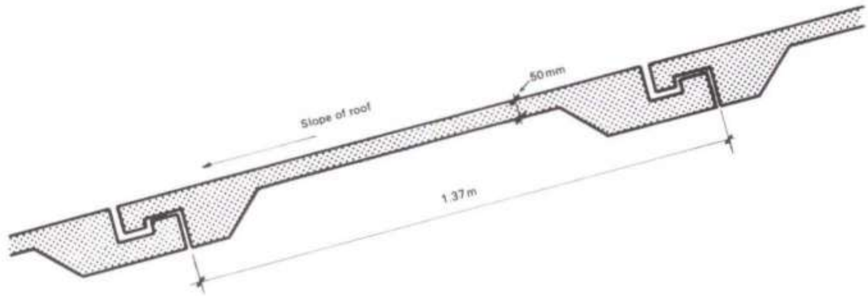


Fig. 21a
Reference library inner perimeter cladding units

Fig. 22
Reference library – View of coffer ceiling showing coverings to air conditioning duct within coffers leading from a cruciform column



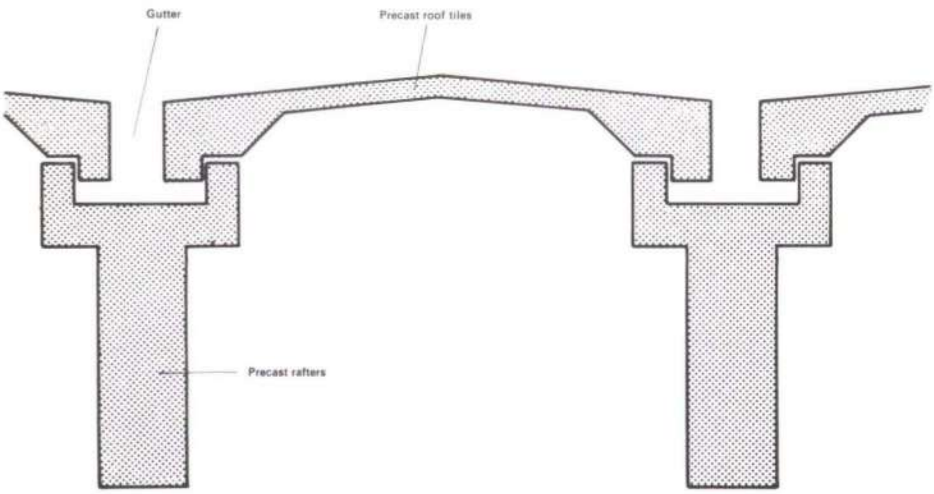


Fig. 23
Reference library precast roof assembly

(Fig. 23). Each rafter is Y-shaped in section and the valley of the rafter acts as a gutter.

The white concrete tiles are 1.37 m square and 50 mm thick with reinforced ribs. They are a development from those used on Dunelm House (Job No. 1534).

On advice from the Cement and Concrete Association only the perimeter ribs of the tiles were reinforced and the 50 mm thick units were unreinforced. They are prevented from sliding by means of lugs projecting into the precast rafter and their dead weight prevents uplift.

Between the concourse and the lowest reference floor are located two levels of offices and other library facilities. These floors are generally about 10 m wide, inserted between the main columns, and associated with them are a number of elevated walkways, ramps, stairs and escalators. In order to achieve a clear articulation of the various elements of the design, the floors, ramps and walkways were designed as separate structures supported on corbels cantilevered out from the columns.

Lending library block

The lending library, although connected to the main library, yielded a different structural solution due to its position over a roadway and the fact that smaller floor spans were acceptable. The lending library varies between two and five storeys and is situated over the gyratory road and underground car park. A structural column grid of 5.5 m × 5.5 m was acceptable to the planning requirements, and the floors were designed as 200 mm flat slabs (Figs. 24 and 25). This relatively light struc-

Fig. 24
Lending library – Internal view from 3rd floor near main entrance

ture was designed to reduce the overall loading on the beams spanning the gyratory road and basement car park. Live loads are $4.8\,kN/m^2$ generally with $11.2\,kN/m^2$ in rolling book stack areas. The children's library is at the rear of the lending library (Fig. 26).

Link blocks

The link blocks are two- and three-storey buildings of offices which connect principal buildings and span over roads and concourse areas. The spans in Phase A of the project range from 10 m to 28 m and the object of the design was to produce a uniform structure which could be built over various trafficked areas. The design will also fit the spans and site conditions in Phase B of the project.

The floor levels in the link buildings had to match the levels both in the existing and new buildings to which they connect and this resulted in a maximum of 600 mm structural depth between storeys within the links.

To achieve the spans required, much greater structural depth was needed, and this was obtained by suspending all the loads from roof beams which could project upwards and had no depth limitation.

Each building is carried by two longitudinal steel plate girders, encased in concrete from which the rest of the building is hung

Fig. 25
The readers' lounge in the lending library. Note the grit blasted columns and ribbed grit blasted walls

Fig. 26
Children's library – This is housed beneath the lending library

(Fig. 27). The hangers from the roof beams are Macalloy steel rods at 5.3 m centres at low stress which are fire protected by precast concrete casings. Precast concrete cross beams span between the steel hangers supporting standard precast prestressed hollow concrete floor planks with an in situ topping.

The upper floor of each link is clad in white precast concrete storey height wall panels. Cumulative deflections were calculated for each link, and the tolerances in the precast construction were arranged to suit the deflections.

The standard link design is for spans up to 22 m. Several variations from the standard link have been necessary to fit special conditions, such as a third storey or very long spans.

The art gallery link had to span the gyratory road close to the junction with Great Charles Street, and consequently, the span reached 28 m. This span, if carried in the normal way, would have involved roof girders some 3½ m deep which posed difficult fabrication and erection problems. Fortunately, at this location the level of the road was such that ample headroom was available below the lowest floor and so a second pair of main girders was introduced to carry this floor. Normal 2 m deep girders at roof level were thus sufficient to carry the top storey over this 28 m span (Figs. 28 to 30).

The continuation link from that serving the art gallery carries a pedestrian walkway over the bus interchange below, which effectively made the building three storeys. For visual reasons it was appropriate to continue bottom girders as in the art gallery link, and these girders were used to carry the suspended walkways as well as the lower storey of the link (Fig. 31).

The town hall link, which will be constructed after the old library is demolished, will again be a special case as it was decided not to make a physical connection with the town hall but to return the link to form a U-shaped plan. The link structure was not conceived to serve this plan shape and special adaptation of the structural system has been necessary.

The hangers encased in precast concrete posed an interesting erection problem and the assembly is shown in Fig. 34.

The hanger rods were first hung and then the casing was slid on and held up by a nut on the main rod. A steel location plate was then added before the precast floor beam was fitted. Macalloy threaded couplings were used to add the lower hanger. A group of spring washers in the assembly held the casing firmly after the rods extended under load.

At the bottom of the whole assembly a plate washer and nut, together with locking bar, was added.

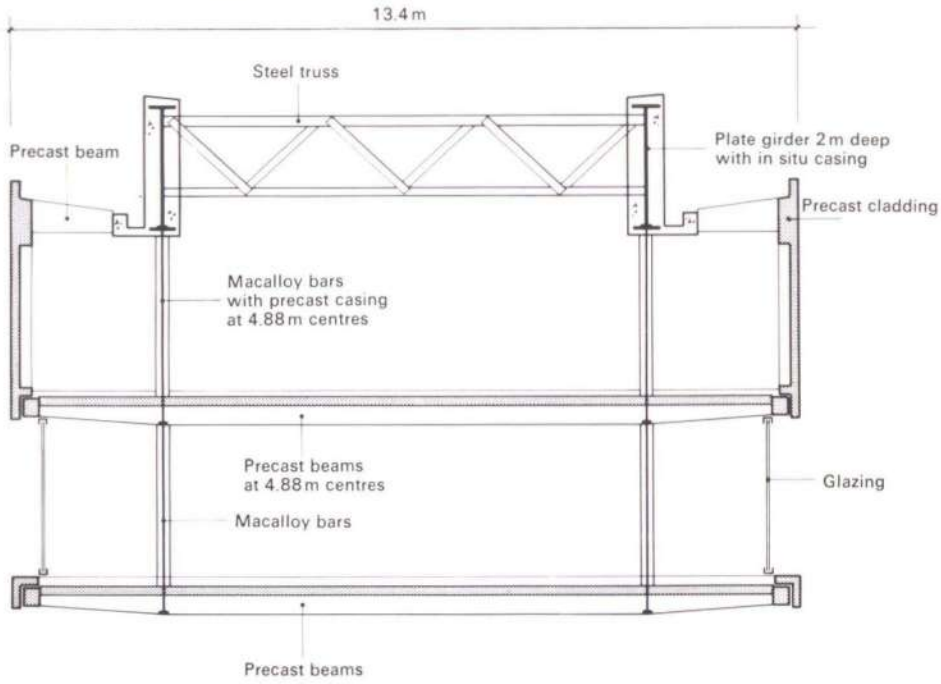


Fig. 27
Typical link block showing structure suspended from roof girders, span up to 22 m

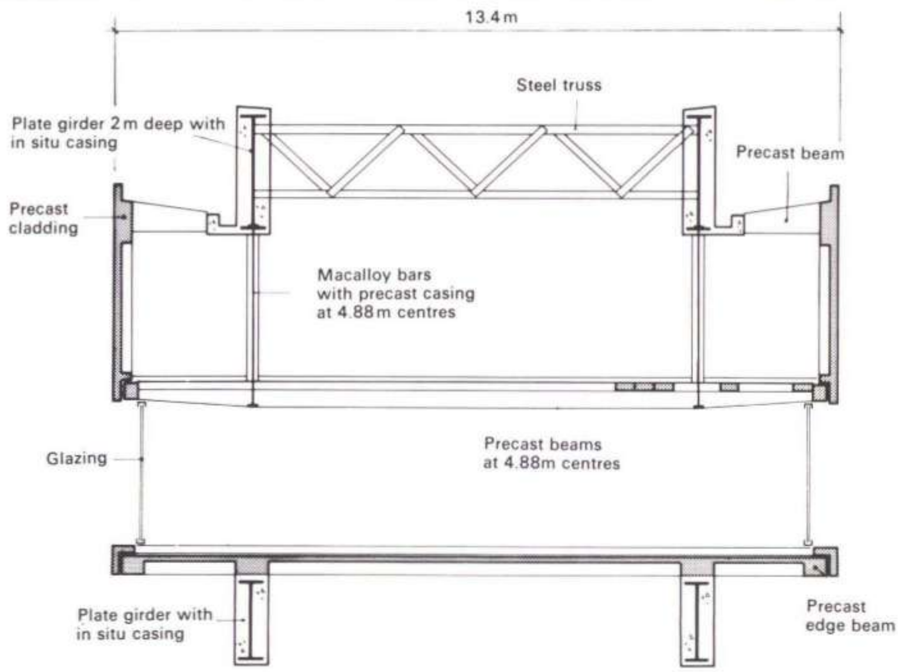


Fig. 28
Art gallery link block 28 m span, with additional girders to support lower storey

Fig. 29
The main girders to the art gallery link during erection over the roadway. The stiffeners indicate the hanger positions for the lower floors

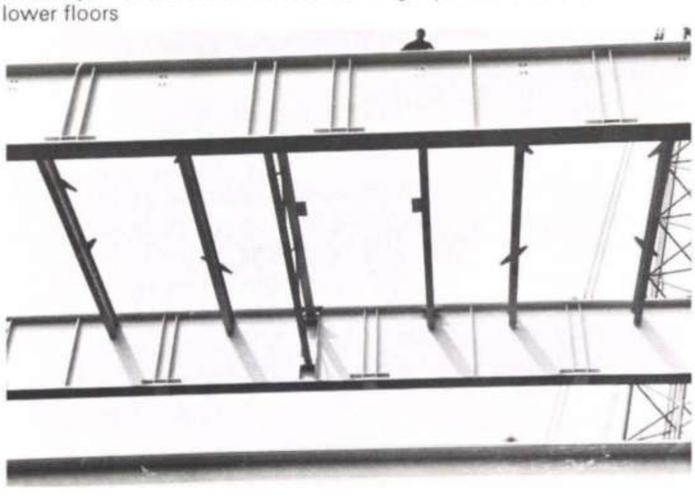
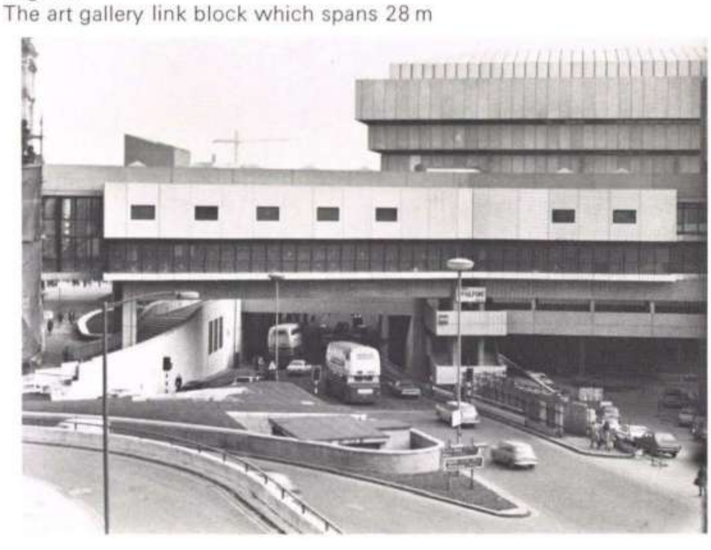


Fig. 30
The art gallery link block which spans 28 m



School of music

The school of music building is at the south end of the site bordering on Paradise Street (Fig. 6). The concourse level and the basement comprise a shopping centre and four upper storeys form the school of music. The main block of the school of music is 12 m wide and 74 m long.

The lower three storeys are constructed entirely in reinforced concrete consisting of a simple two-way spanning slab on beams and columns. Concrete surfaces in the public areas have exposed aggregate surfaces. Bush hammering was chosen due to the public nuisance caused by the dust from grit blasting and this has worked out successfully as the transition from bush hammered areas to grit blasting on the rest of the project is not noticeable.

The upper storeys of the school of music are largely divided into practice cells.

The need for sound insulation led to a loadbearing brickwork structure over these three upper storeys. Each cell is divided by a loadbearing wall but is further separated by an inner skin brick wall, mounted on rubber pads and with its own ceiling (Fig. 33).

The recital room is mounted on stilts and is situated on top of the underpass at the extreme south east corner of the site with continual traffic noise from two sides. Consequently considerable mass was required for acoustic reasons and the building was designed as a concrete box of 225 mm reinforced concrete walls with precast cladding panels. The plan area is 12 m by 18 m.

The roof of the recital room is a coffered slab using standard moulds and the soffit is painted. Despite the proximity to very heavy traffic the recital room is successfully isolated from extraneous sounds (Fig. 34).

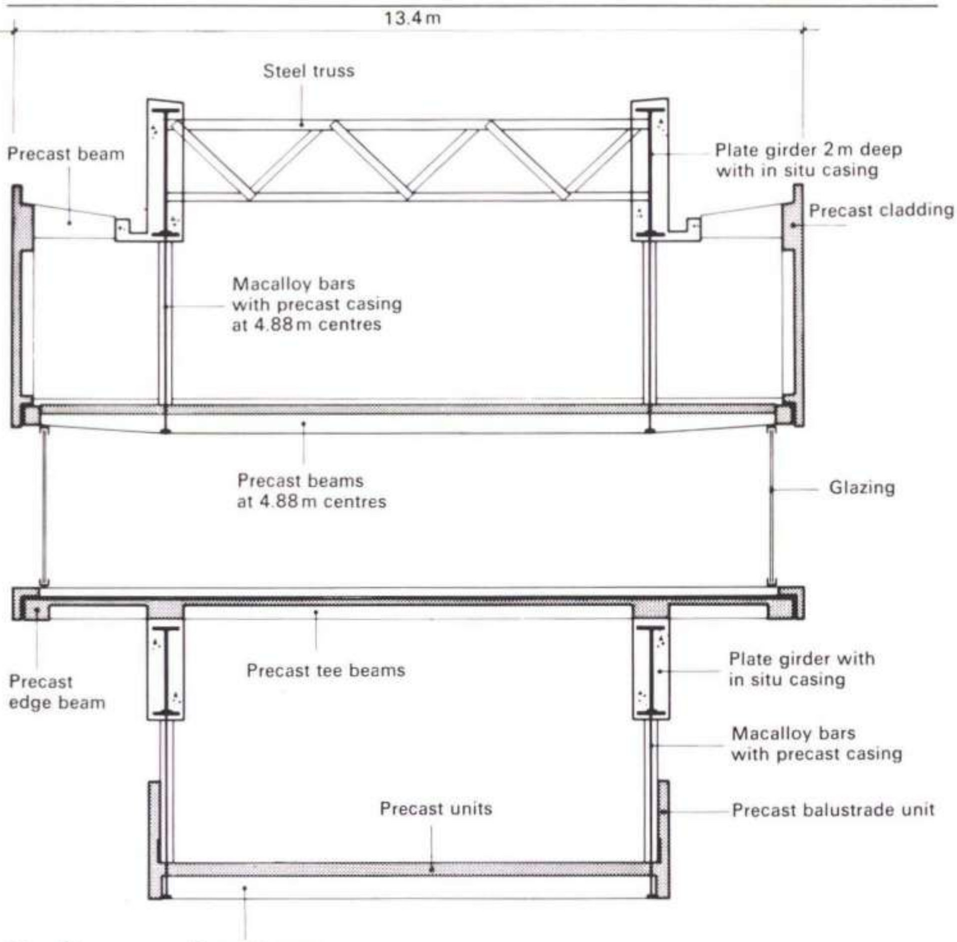


Fig. 31
Link block 7, top storey suspended from roof girders, walkway suspended from lower girders

Fig. 32
School of music – recital room. The structure is of 225 mm concrete walls with roof slab formed from standard coffer moulds

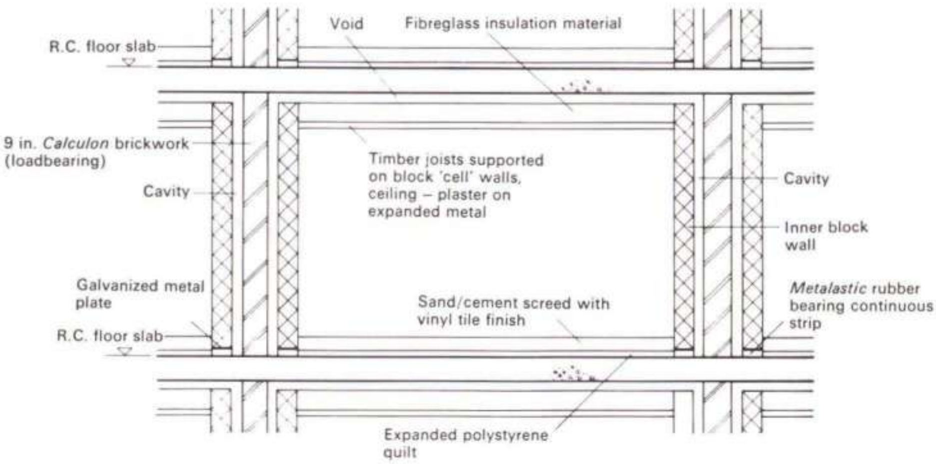


Fig. 33
Section through typical music practice room

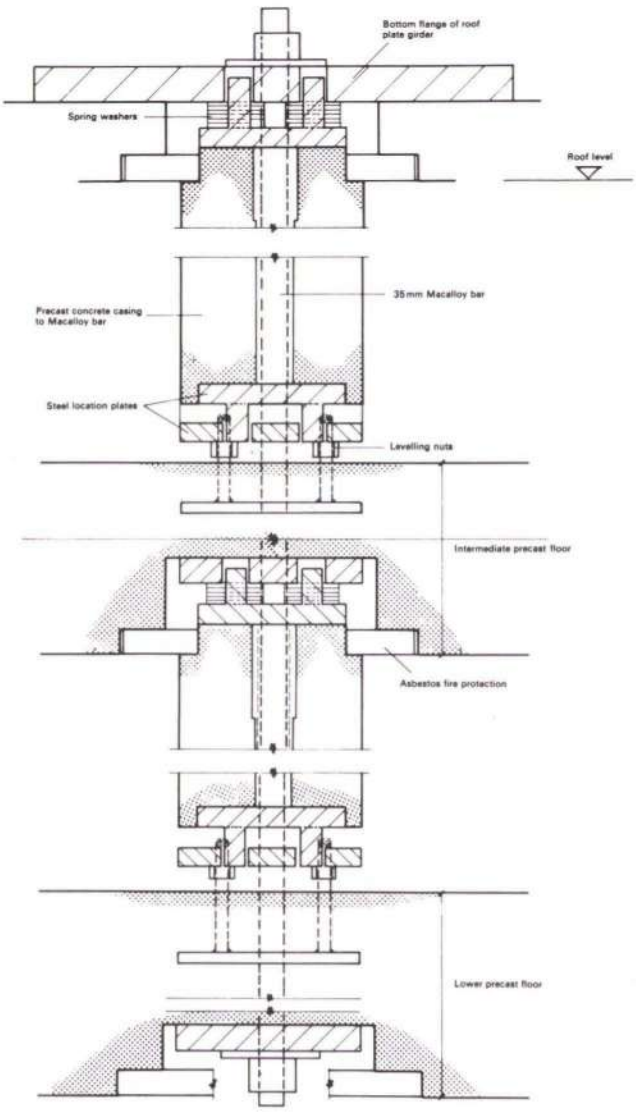


Fig. 34
Link blocks typical hanger assembly

Photos in this article:
Figs. 12 & 29 by Ernie Irwin. 17 & 22 by Malcolm Jordan. 4, 5, 6, 10, 14, 15, 16, 20, 24, 25, 26 & 32 by Logan Photography. 13 & 20 by John Whybrow Ltd.

Contract arrangements

The underpass through the site and the gyratory road for Birmingham Public Works Department carried a Department of the Environment grant and thus had to be let as a separate civils contract. On the other hand the buildings contract under the authority of the Birmingham City Architect drew other Government grants and was let separately by competitive tender using the special Birmingham building contract.

A detailed phasing arrangement had been worked out between the two contracts as they had common boundaries and overlapped in some places. As a result of various factors and largely due to late diversion of services, the phasing did not go according to plan and this greatly complicated administration of the contracts.

At the time of tender in 1968, competition was very great and the building contract was awarded to the lowest bidder. Throughout the contract a great deal of attention has had to be devoted to claims and possible claims which have been a major distraction from the main job of creating the complex.

Setting-out

Two main contractors on site at the same time is never an easy problem, but when they are both constructing foundations for the same building with one commencing 18 m below ground at the side of an underpass and the other operating at ground level, several hundred feet away, the problem of checking setting-out becomes formidable. We decided that normal ground-mounted setting-out points would soon be uprooted by either contractor and become unreliable and a novel solution was found. The site was divided into an approximately 60 m grid in two directions at right angles and a survey firm was appointed in advance of the contracts to set out these grids between the roofs of the surrounding buildings. Each contractor was then instructed to set-out his portion of the building from these accurately positioned high level grid lines. This method proved very successful, and only one column was found out of position.

The building works had been planned as parallel to the nearby Baskerville House. On the other hand the underpass, planned by the City Engineer, was related to the national grid. As many of the building columns were on the roof of the underpass it was necessary to relate the two setting-out grids. Relative bearings were obtained and a computer programme used to predict building grid references at intervals along the three walls of the underpass.

Acknowledgements

The structural design was initiated in 1964 by Frank Coffin and John Martin. The Birmingham office was set up in 1968 to continue the design of the project and Ken Anthony and Ernie Irwin jointly led the design teams from London and Birmingham respectively.

Architect:
The John Madin Design Group in association with Birmingham City Architect

Mechanical and electrical engineer:
R. W. Gregory & Partners

Quantity surveyor:
L. C. Wakeman & Partners

Resident engineers:
Matt MacKay, Bob Astley, Alan Tricklebank

Main contractor:
Sir R. McAlpine & Sons Ltd.

Contractor for underpass:
C. Bryant & Son Ltd.

Underpass and road planning:
Birmingham City Engineer and Surveyor

Precast cladding supplier:
W. Sindall & Sons Ltd.

Structural steel sub-contractor:
Braithwaite & Co. Ltd,

Design of continuous composite beams for buildings

John Morrison

Introduction

Early last year Ove Arup & Partners were commissioned by the Post Office to prepare a scheme design for a £6m. Parcel Concentration Office in Liverpool. The operational requirements of the building were such that a fairly large column-free space and floor to ceiling height were required, and in our case a 12 × 12 m grid with floor to floor heights of 7 m for three storeys was adopted.

Composite design of the frame was considered as one of the possible solutions as it was known that structural steelwork would be quick to build and that the quantity of material required would be reduced to a minimum.

For design guidance, the information available consisted of *CP117: Part 1: 1965*, which gave design rules for simply supported beams. This Code would have been satisfactory if the structure had been braced by cross frames or stair towers. In our case the overall stability could have been provided by concrete stair towers but these would have been expensive and time-consuming to construct. It was therefore decided that a study should be made into the possibilities of using sway frames with moment-resistant joints. This decision to investigate fully continuous frame action meant that we were faced with the problem of finding an acceptable design procedure.

There was one other Code available for guidance, *CP117: Part 2: 1967*, 'Beams for bridges'. This Code gave some information for continuous beam design using elastic methods. However, it was not really suitable and it was therefore decided to make a study of all available technical papers to see if it would be possible to draw up our own set of design rules.

The design guide which follows is the result of this study and it has been amended from time to time as a result of our continued interest in this subject. We feel that its development has now reached a stage where we are able to give a complete design method together with some advantages and disadvantages of this form of construction. A Hewlett–Packard program has also been written to aid the design process.

Methods of analysis

The method of frame analysis has an important bearing on the system to be used for the analysis of the sections. For example, we would suggest that plastic design of the frame and its sections should only be used for continuous beams where side sway is prevented, due to the difficulty of achieving a fully rigid beam to column connection and hence adequate lateral stiffness. This problem will be discussed in more detail in a later section of this article. However, when a plastic design for continuous beams is adopted it is important to carry out an elastic analysis as well, as this will be required for checking stresses at working loads.

Where side sway can occur,[1] an elastic analysis of the frame is preferable; the results should then be factored by 1.75 as specified by *CP117: Part 1*, to give the ultimate moments and forces. These factored values should then be used for the determination of the section sizes. This design procedure of applying a load factor to the elastic analysis is the method most commonly adopted.

For both braced and unbraced frames the bending moment diagrams can be determined by ignoring the inertia contribution of the concrete flange in the hogging moment zones which is assumed to be cracked. This has the advantage of reducing the bending moments in the columns and can easily be achieved by substituting the equivalent stiffness of the composite beam in the framework analysis. The equivalent stiffness can be expressed as follows:[2]

$$S = \frac{\lambda I[1+\alpha(\lambda-1)(2\alpha^2-3\alpha+3)]}{L[1+2\alpha(\lambda-1)][1+2\alpha(\lambda-1)(4\alpha^2-6\alpha+3)]}$$

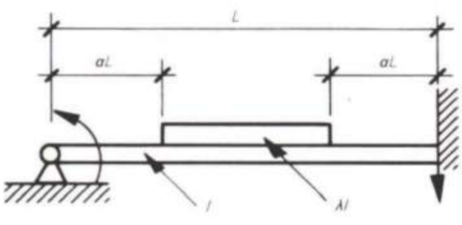


Fig. 1

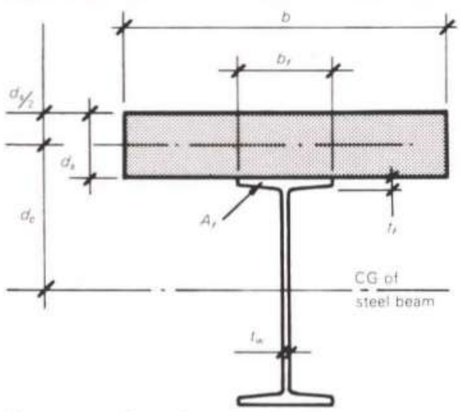
For most cases a value of $1.2\,\frac{I}{L}$ will be sufficiently accurate for use in the overall frame analysis.

Moment of resistance

The moment of resistance of the section M_r should be found making the following assumptions. (The formulae assume the use of standard rolled sections with equal flanges.)

(a) *Midspan sections*

Use methods given in *CP117: Part I: 1965*, where the breadth of concrete flange should be taken as one-fifth of beam length between points of contraflexure, or one-tenth of distance for edge beams or the actual flange width, whichever is the lesser.



A_s = area of steel section
A_f = area of flange
Y_s = yield stress of steel beam
U_w = cube strength

$$\alpha = \frac{9}{4}\frac{Y_s}{U_w}$$

Fig. 2

(1) If $\alpha A_s < bd_s$ then the plastic neutral axis d_n is within slab and

$$M_r = A_sY_s\left[d_c + \frac{(d_s-d_n)}{2}\right]$$

(2) If $bd_s < \alpha A_s < (bd_s + 2\alpha A_f)$ then the plastic neutral axis is within the top flange of the steel beams and

$$M_r = Y_s[A_sd_c - b_fd_n(d_n-d_s)]$$

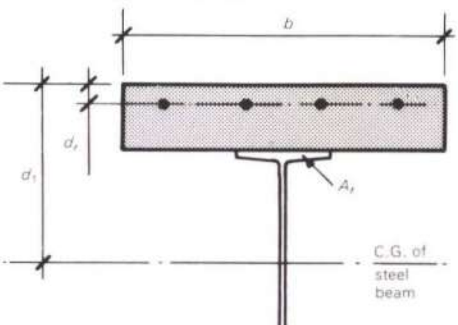
(3) If $\alpha(A_s-2A_f) > bd_s$ then the plastic neutral axis is within the web of the steel beam and

$$M_r = Y_s[A_sd_c - A_f(d_s+t_f) - t_w(d_n+t_f)(d_n-d_s-t_f)]$$

(b) *Sections at continuous support*

The moment of resistance of the section at the support has been developed using the same procedure as that for the midspan.

The effective breadth of slab at the support should be taken as 0.6 × the effective breadth at the midspan. The effective breadth in fact varies according to the type of loading but the value given above is a reasonable mean. As the concrete is always assumed to be cracked, the only importance of this effective breadth is for calculating the area of reinforcement within this zone.



$$\beta = \frac{Y_s}{f_{yt}}$$

f_{yt} = yield stress of reinforcement
A_r = area of reinforcement

Fig. 3

(1) If $A_r > \beta A_s$ then the plastic neutral axis is within the reinforcement and

$$M_r = A_sY_s(d_1 - d_r)$$

(2) If $\beta A_s < (A_r + 2\beta A_f)$ then the plastic neutral axis is within the top flange of the steel beam

$$M_r = Y_s[A_s(d_1-d_r) - b_f(d_n-d_s)(d_n+d_s-2d_r)]$$

Conditions (1) and (2) cannot occur unless special stiffening is provided to stabilize the web, otherwise the amount of reinforcement required would mean that the beam would fail the 'compact section' rules (see later).

(3) If $A_r < \beta(A_s - 2A_f)$ then the plastic neutral axis is within the web of the beam, and

$$M_r = Y_s[A_s(d_1-d_r) - (A_f + t_w(d_n - t_f - d_s))(d_n+d_s-2d_r)]$$

There are some special conditions which can occur in sway frames.[3] A typical sway frame bending moment diagram in the lower storeys could be as follows: (Fig. 4).

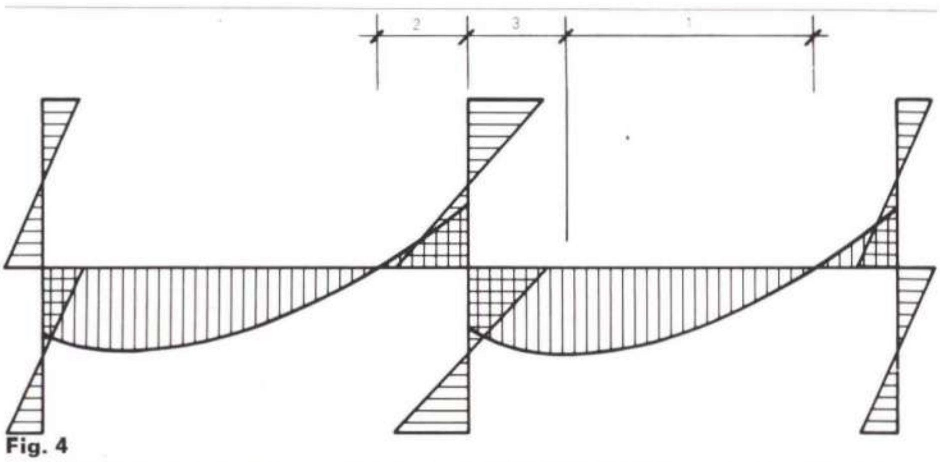


Fig. 4

For Zone 1 (Fig. 4)
Moment capacity should be calculated as for section (a) above.
For Zone 2
Moment capacity should be calculated as for section (b) above.
For Zone 3
The moment capacity of the composite beam adjacent to a column face and subject to a sagging bending moment should be based on the steel section acting together with a concrete flange with a width equal to that of the column face. Condition (3) rarely occurs, usually only in the lower storeys of multi-storey frames.
The effective compression zone can be increased as shown below.

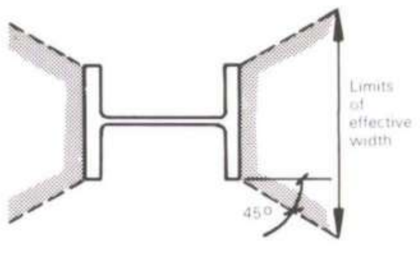


Fig. 5

If necessary the concrete could be contained to allow higher stresses or a spreader could be incorporated to provide greater width.

Compact sections

An important restriction has to be placed on the section sizes that can be used for continuous beams to ensure that local buckling of the beam does not occur in hogging moment zones. This buckling is caused by the compressive stresses in the bottom flange and the web where they are away from the stabilizing influence of the concrete flange.
Sections which are suitable for the formation of plastic hinges are described as compact. Research has shown[6] that for structural steels to Grades 43 and 50 a section can be considered compact if it satisfies the following:
For Grade 43 steel:

Flange slenderness: $\frac{b_f}{t_f} \leqslant 17$

Web slenderness: $\frac{d}{t_w} \leqslant 70\ (1-1.4\Phi)$ with a lower limit of 43

For Grade 50 steel

Flange slenderness: $\frac{b_f}{t_f} \leqslant 14$

Web slenderness: $\frac{d}{t_w} \leqslant 58\ (1-1.4\Phi)$ with a lower limit of 36

where Φ, the force ratio $= \frac{A_r f_{yt}}{A_s Y_s}$

The rules given above are more severe than those controlling the plasticity of universal beams for conventional steelwork design. It will be found that the range of universal beams available is severely restricted and in many cases it will be necessary to resort to the use of universal column sections.

The rotational performance of slender sections can be improved by the addition of a horizontal web stiffener at mid-depth extending 1.5d from the support. At the end of this stiffener the support moment is likely to have dropped to 70 per cent. Horizontal stiffeners should always be provided for the column webs in line with the bottom flange of the beam to prevent local buckling of the column web.
Without additional stiffening the rules given above effectively limit the value of force ratio to 0.28, because at the limit of $d/t_w < 36$ there are only a few rolled sections that can comply with this criteria.

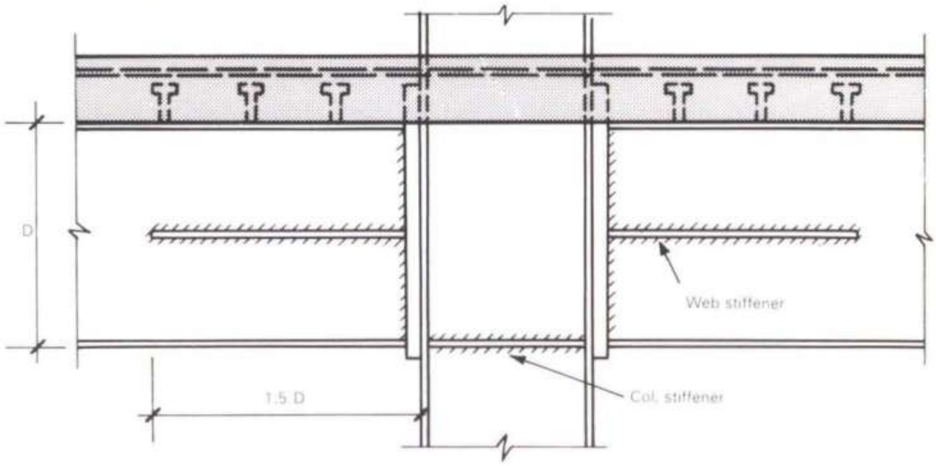


Fig. 6
Typical web stiffening

Where a horizontal stiffener is provided, the upper effective limit for the force ratio becomes 0.4. For force ratios greater than 0.4 the plastic axis remains approximately constant within the top flange of the beam, thus reducing the efficiency of the extra reinforcement.
It should be noted that only web stiffening is required, for although initially there is a small amount of flange buckling, it is the web buckling that produces a rapid loss of strength. For further details of this buckling refer to the paper by Climenhaga and Johnson.[6]
The rules given above are satisfactory where it is intended that the full ultimate moment of resistance of the support is to be developed. However in many instances the design will not require that the full support moment be developed. In this case, applying the compact section rules would result in uneconomic sections.
If the applied moment is less than 70 per cent of the ultimate moment of resistance at the support, and providing that the appropriate elastic checks are carried out, then it would appear reasonable to relax the rules given above.

Elastic analysis check

An elastic design check should be made for both the midspan and support sections at working loads.
In both cases the steel beam should be checked to ensure that the elastic stress does not exceed 0.9 yield stress. At the midspan the concrete stress should be checked to ensure that it does not exceed one-third of the specified cube stress, although this condition is unlikely to control the design. At the support the tensile strength of the reinforcement should be checked to ensure that this does not exceed 0.6 yield stress of the reinforcement.
In virtually all continuous composite beam design the 0.6 yield stress in the reinforcement or the 0.9 yield stress in the steel beam will be the controlling factors in the design.
To enable the elastic properties of the section to be calculated, a knowledge of the modular ratio m will be required. Part 1 of *CP117* recommends $m=15$ while Part 2 recommends that $m=83\sqrt{u_w}$.
For a midspan section it can be shown that there is little change in stress in the critical bottom flange for a range of modular ratio between 8 and 16. The stress change is less than 5 per cent and it is therefore recommended that $m=15$ should be adopted.
For the support condition the modular ratio is not included in the calculations as the concrete is assumed to be fully cracked.

Shear

For simply supported beams the shear force should be supported entirely by the web of the steel beam.
For continuous beams the situation is slightly more complex in that the shear and moment capacities of the section are reduced by the inter-action of these forces.
The ultimate shear capacity of an I beam, V_{ult}, can be expressed as follows:

$$V_{ult} = V_o \sqrt{1-\left(\frac{M-M_f}{M_w}\right)^2}$$

Where $V_o = \frac{Y_s t_w d}{2}$ Tresca Criterion

M = applied moment

$M_f = t_f(b_f - t_w)\ (d - t_f) Y_s$

$M_w = \frac{t_w d^2 Y_s}{4}$

and $M_r = M_w + M_f$

Research by Johnson and Willminton[7,8] has shown that for composite beams there is an additional shear capacity available in the hogging moment zones. This additional capacity is provided by the reinforcement in the negative moment regions which acts together with the steel section so that shear forces exceeding the plastic capacity of the web can be achieved. For the commonly adopted force ratio values of 0.28 the increase in shear capacity is likely to be in the order of 10 per cent.

The steel section must be 'compact' to ensure that sufficient rotation and plasticity can be achieved before the maximum moment is reached. If this rotation cannot occur, then the section is likely to fail by buckling.
Usually this additional shear capacity, due to the dowel action of the reinforcement, will not be required and a simple check using the Tresca Criterion will be sufficient.

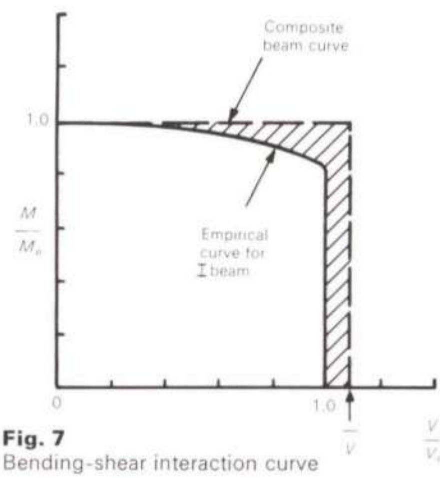


Fig. 7
Bending-shear interaction curve

Design of shear connectors
Number of connectors

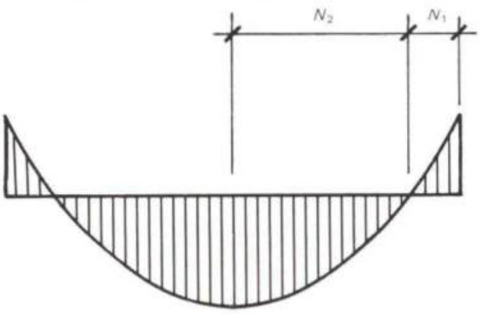
For simply supported beams the design load per stud *Pc* should be the value given in *CP117* which is 80 per cent of its ultimate

capacity. It has been shown[14] that if this value of Pc is adopted then the amount of slip will be limited to a value which will not impair the ultimate moment of resistance.
For uniformly distributed loads the number of connectors required can be determined as follows:
When plastic NA is within depth of steel section the number required$=\frac{Fcc}{Pc}$, where Fcc is compressive force in slab at ultimate load.
Or when plastic NA is within depth of slab the number required$=\frac{Fcc}{Pc}$ or $\frac{Fst}{Pc}$, where Fst is tensile force in steel beam at ultimate load.
The number of connectors given above represent the total required between the points of zero and maximum shear. Thus for a simply supported beam with a uniformly distributed load, twice the value given above will be required over the full length of the beam.
The connectors may be spaced evenly along the length of the beam.
Where point loads predominate, it is necessary to divide the shear force diagram into sections between the points of application of the loads and to calculate the total shear force for each zone. The number of connectors required can then be calculated and these should be evenly distributed along the section considered.
For continuous beams the connectors in the hogging moment zones are likely to be in an area of cracked concrete. For this reason it has been found that an acceptable design load for the studs to control slip is 80 per cent of the value given in *CP117*, i.e. 64 per cent of the ultimate capacity.[9]
The number of connectors required should be found by calculating the horizontal shear force, which is equal to the tensile force generated in the slab reinforcement over the support.

Thus the number required $N=\frac{A_r f_{yt}}{0.8\,Pc}$

Spacing

Although the method of determining the horizontal shear force is well known, there appears to be far less agreement as to the manner in which shear connectors should be distributed along a composite beam.
Johnson, Greenwood and Van Dalen[9] recommend that the number of connectors in the hogging moment zone, as found above, should be equally spaced between the points of maximum hogging moment and contraflexure.

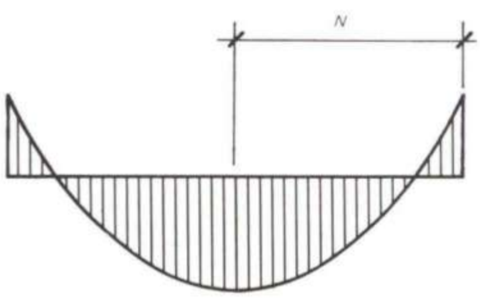


$N_1=\frac{A_r f_{yt}}{0.8\,P_c}$ $\quad N_2=\frac{F_{cc}}{P_c}$

Fig. 8

For the sagging moment zone the number of connectors is $\frac{Fcc}{Pc}$ which should be equally distributed between midspan and the point of contraflexure.

On the other hand, Yam and Chapman[10] recommend that the number of connectors should be determined using the sum of the horizontal shear forces between the points of maximum and minimum moment and that these connectors should be equally spaced along the beam.



$N=\frac{(A_r f_{yt}+F_{cc})}{P_c}$

Fig. 9

There is little difference in the final result between the two methods. The latter system is slightly more convenient in practice and is therefore recommended.
In all cases it is recommended that the spacing of the connectors should not exceed three times the slab thickness or 600 mm, whichever is the lesser.
If a single line of shear connectors is all that is required, these should be staggered on either side of the centre line to reduce the possibility of longitudinal splitting of the slab.

Height of shear connectors

In sagging moment zones the height of the studs should extend 25 mm into the compression zone and never be less than 50 mm high to ensure adequate resistance against uplift.
For hogging zones uplift is no longer a problem. The problem is now one of loss of efficiency, due to slip, as the connectors are embedded in a zone of cracked concrete. In this zone it is recommended that the shear connectors should project at least 25 mm above the lowest level of reinforcement.
However for practical reasons the height of connectors should be the same throughout the length of the beam and therefore the height of connectors determined for the sagging zones will usually control. (See Table 1).

Design of transverse reinforcement in slab

Using the method set out in *CP117: Part 1*, the following checks should be made:
The shear force Q in N/mm²

$$Q=\frac{Nc \times \text{load in N on one shear connector at ultimate load}}{\text{Longitudinal spacing of connectors in mm}}$$

This force should not exceed the lower value of either:
(1) the shear resistance per mm run of beam $=0.23\,Ls\sqrt{u_w}+A_t f_{yr} n$
or
(2) $0.62\,Ls\sqrt{u_w}$
Where $Ls=$ shear perimeter of the connectors, but $\not> 2\times$ slab thickness of T beams $\not>$ slab thickness for L beams
$n=$ number of times the lower level of reinforcement intersects the shear perimeter Ls.
For T beams $n=2$, for L beams $n=1$.
The amount of transverse steel A_t in the bottom of the slab should not be less than $\frac{Q}{4f y_t}$ mm² per mm run of beam.
Reinforcement provided to comply with this requirement can also be assumed to resist transverse slab moments provided that the minimum quantity in the bottom of the slab is maintained.
This rule is especially important to prevent splitting of the slab when high shear forces occur.
An alternative method which results in a reduction in the quantity of transverse reinforcement has been proposed by Johnson.[11]

Deflection

Although composite design of a beam has many advantages, such as a reduction in the depth of steel beams, the use of these shallower beam depths means that deflection has to be carefully considered.
For continuous beams the deflection is unlikely to exceed 20 per cent of the deflection for a simply supported beam with the same moment of inertia. Even if the support moment were reduced to 50 per cent of the midspan value the deflection would still only be 30 per cent of the simply supported condition. A further benefit of continuity is that shrinkage and creep of the concrete flange are unlikely to give rise to a serious deflection problem and can therefore be neglected.
The most serious deflection occurs with simply supported beams and for these special care should be taken. Not only are dead and live load deflections important, but also the creep and shrinkage deflection, which in some circumstances can be as much as 100 per cent of the dead load deflection[12].

Table 1

Design values of shear connectors for different concrete strengths

Type of connector		*Connector material*	*Design values of connectors for concrete strengths U_w N/mm²*		
Headed studs			20	30	40
Diameter mm	Height mm		Load per stud kN		
25	102	Minimum yield	117	135	153
22	102	stress 386 N/mm²	97	112	126
19	102		77	88	100
19	76	Minimum tensile	66	76	87
16	76	stress 494 N/mm²	56	65	74
13	64		35	42	47
Bars with hoops		*BS 4360* Grade 43			
50×38×200			380	580	780
Channels					
127×64×4.5 Kg×152		*BS 4360*	223	260	297
102×52×3.18 Kg×152		Grade 43	206	242	280
76×38×2.04 Kg×152			194	230	266

For the calculation of the deflection it is recommended that the effective modulus method should be used. This is a simple and well-proven method and is used in both the CEB and *CP110* recommendations. It has been shown by Knowles[15] that the results do not vary significantly from the more 'exact' methods.

The modular ratio adopted should be modified to allow for the effects of shrinkage and creep, either by using the CEB graphs or by adoption of the simpler values given in Appendix A of the explanatory handbook to *CP110*. The final calculation can then be carried out using the curvature method together with the K factors given in Table 1 of the explanatory handbook.

The construction sequence also has an important bearing on the final deflections. If the beams are not propped during the casting of the slab, deflection due to this self-weight of wet concrete acting on the steel beam alone should be allowed for, possibly by pre-cambering the beams. If the beams are propped, usually at one-third points, the concrete is allowed to harden, before the props are removed after 14 days. The total load is now resisted by the composite beam and the deflection is therefore reduced. A further advantage of this second method is that it reduces the initial stressing of beams due to self-weight of the concrete and in certain instances beneficial effects can be achieved by an initial jacking up of the beams.

However, jacking up of the beams on site is difficult to control in practice. It may also be found that in many instances even propping is an unnecessary inconvenience. The effects of all of these factors will need to be taken into account for determination of the beam size.

Beam to colum connections

The design of the beam to column connections could be looked upon in many respects as the starting point of the design. For until a decision has been made whether to adopt simply supported, full fixity or some intermediate partial fixity, it is impossible to analyze the frame or determine the beam moments.

In an earlier section of this report the effect of analysis and section properties was discussed. It is now intended to take this a stage further and to discuss the implication of various assumptions about the beam to column connections.

Simple support connections

These connections should only be used for single bay structures. Simple supports can be provided in multi-bay structures, provided that a joint is formed in the concrete slab at all lines of support, but this is not recommended.

If reinforcement is provided in the slab over the support then the section is no longer simply supported and in fact some degree of partial fixity has been introduced. Thus depending on the loading, span and depth of beam, the mid-span is likely to be oversized while the support may be overstressed.

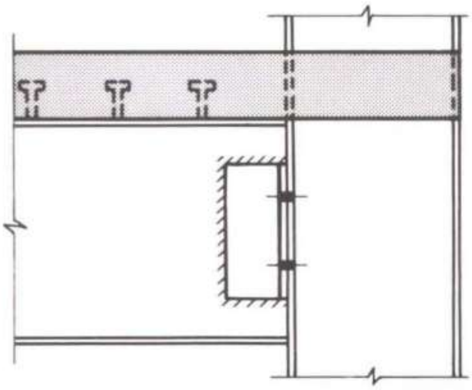


Fig. 10
Simply supported connection using black bolts

Rigid connections

If a rigid connection is provided either by welding or the use of high strength friction grip bolts, and provided that the section is compact, then the greatest economy can be achieved by taking advantage of fully continuous design.

However, welded joints are likely to be expensive. Bolted joints also have their problems due to practical difficulties of providing sufficient bolts to effectively develop the full plastic moment. There is also the problem of serviceability at working loads, for in an ideal condition midspan and support moments should be of the same order. However, at working load the elastic stress in the steel must be limited to $0.9\,Y_s$ and the stress in the reinforcement to $0.6\,Y_s$. These two conditions are always likely to be the controlling factors for determining the moment of resistance at the support.

Assuming that the elastic stresses at the support are within acceptable limits, it may still be difficult to provide a bolted connection with sufficient strength to take the balance of the forces after making allowance for the reinforcement. (See Fig. 11).

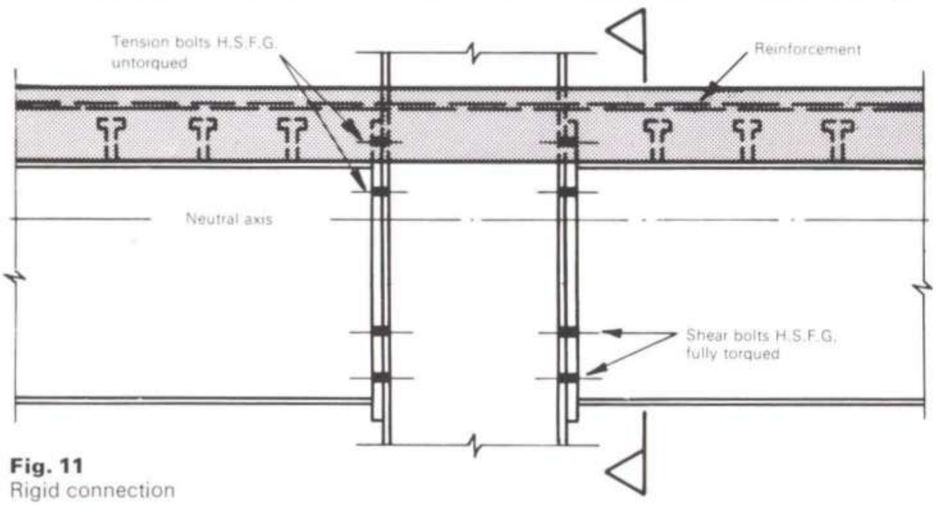
$$M_{ult}\ (\text{Bolts}) = M_{ult} - M_{reinf}$$

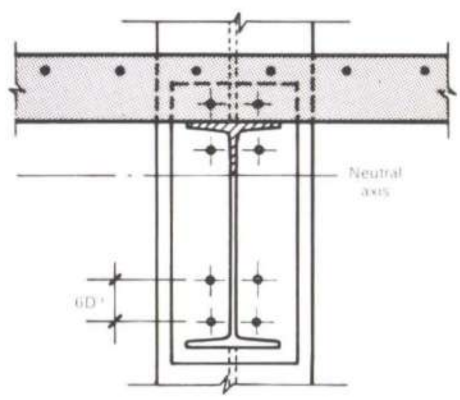
Partial fixity

It has been shown above that the simply supported design can lead to uneconomic section sizes, while rigid design gives support moments which for all but a very few instances are difficult to achieve. Yet as we have seen earlier there are several distinct advantages to be gained from continuity, i.e.:

(a) The section sizes can be reduced.
(b) The deflections can be minimized.
(c) The reduction in deflection makes the use of HYS more advantageous.

The desirable joint is therefore one which leads to a fairly simple and conventional bolting arrangement while providing a reasonable support moment acting in conjunction with the reinforcement.





*N.B.—6D bolt spacing adopted for program

Fig. 11
Rigid connection

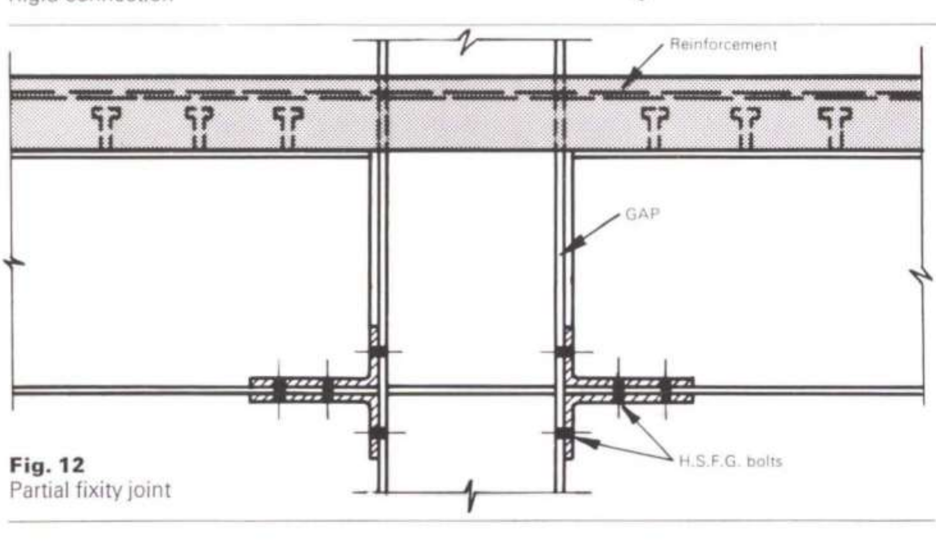


Fig. 12
Partial fixity joint

For the purpose of providing a partial fixity joint, either of the details shown in Figs. 11 and 12 can be used.

The details shown in Fig. 12 have been developed by Johnson and Hope-Gill,[13] although somewhat clumsily. It is simple to fabricate and the small gap between the beam and column ensures adequate rotational capacity.

Partial fixity joints should not be used in sway frames as these joints can only be used where stability is provided by some alternative system.

The stresses set up by the selected support moment must be checked to ensure that at working loads the stresses in the beam and reinforcement are not exceeded. In the case of the detail shown in Fig. 12, it is also important to check that slip of the bottom flange bolts does not occur at working loads.

Design procedure

(1) Derive ultimate and elástic bending moment and shear force diagrams.
(2) Determine slab depth, using span/depth ratio for continuous slab.
(3) Calculate effective flange width at both midspan and support.
(4) Select a suitable steel beam, approximately $Z = \frac{M}{1.4\, f_{bc}}$.
(5) For the sagging moments calculate:
 (a) ultimate moment of resistance
 (b) the elastic properties of the section.
(6) For the hogging moments, repeat (a) and (b).
(7) Check that the section is compact.
(8) Check elastic stresses in steel beam and concrete, midspan section, at working loads.
(9) Check elastic stresses in steel beam and reinforcement, support section at working loads.
(10) If stresses at support are a major problem, consider the possibility of simply supported design or some form of partial fixity.
(11) Check shear at supports for ultimate load condition.
(12) Design moment-resistant joint between beam and column.
(13) Calculate longitudinal shear force at ultimate load, for calculation of shear connectors.
(14) Select type of shear connector, calculate number and spacing.
(15) Check longitudinal shear force in concrete.
(16) Design transverse reinforcement in slab.
(17) Calculate deflection taking account of creep, shrinkage, construction sequence and method of propping.
(18) Check stresses in steel beam under construction loads.

Hewlett-Packard programs

One of the most daunting aspects of composite beam design is the laborious nature of the calculations and for continuous composite beams the amount of work is more than doubled. To carry out a complete analysis of a composite beam, including elastic and deflection checks, could easily take one day. Therefore this design guide has been embodied into a computer program, enabling the engineer to explore all possible variations before making a final decision on the section sizes.

A program in three main parts, based on the use of structural rolled steel sections, has been prepared.

Part 1

(a) Determines the ultimate moment of resistance at the midspan and support
(b) Checks that the section is compact if the beam is continuous
(c) Checks the elastic properties of the section at both midspan and supports giving:
 neutral axis depth
 (equivalent steel) moment of inertia
 Z concrete top
 Z steel top
 Z concrete bottom
(d) Checks at midspan that the concrete stress $< 0.33\ u_w$ and that the steel stress $< 0.9\ Y_s$
(e) Checks at support that the stress in the reinforcement $< 0.6 f_{yt}$ and that the stress in the beam $< 0.9\ Y_s$
(f) Checks the shear capacity of the web.

Part 2

(a) If required, checks the additional shear capacity by the Johnson, Willminton method[8].
(b) Gives the number and spacing of shear connectors
(c) Gives the area of transverse reinforcement in the slab
(d) Given bolt size, designs the joint for either simply supported, partially fixed or fully fixed conditions and gives the thickness of the end plate.

Part 3

(a) Checks deflection using *CP110* method
(b) Checks additional deflection due to creep and shrinkage using the effective modulus method
(c) Checks deflection due to various sequences of construction, i.e. free span, single midspan prop, or props at one-third points.

Conclusions

It is fairly well-established that the composite design can result in a 30 per cent saving in structural steelwork as compared with a non-composite design. When continuous composite beams are adopted there can be a further 10 per cent saving. Together with the reduction of beam depth and deflection, and by virtue of this reduction in deflection, a more advantageous use of high yield steel can be made.

Full fixity of the beam column is possible for beams with a depth of up to 550 mm. For beams deeper than 550 mm, some form of partial fixity is desirable.

Without special stiffening of the web and beam flanges, it is important to limit the force ratio to 0.28, otherwise the 'compact section' rules will be failed.

For preliminary designs the steel section size can be found by determining the elastic modulus using the midspan bending moment and the allowable bending stress which should be factored by 1.4.

It has not been possible to deal with the many other possible variations of composite beam construction, but as food for thought the following comments may be interesting.

Haunched beams

The formation of a concrete haunch in the slab greatly improves the efficiency of simply supported beams due to the increase in lever arm. However, for normal building frames, the advantages are likely to be eliminated by the expense of the shuttering.

Included in this category are the steel sections with unsymmetrical flanges, although these again are likely to be expensive to fabricate and are only suitable for simply supported conditions.

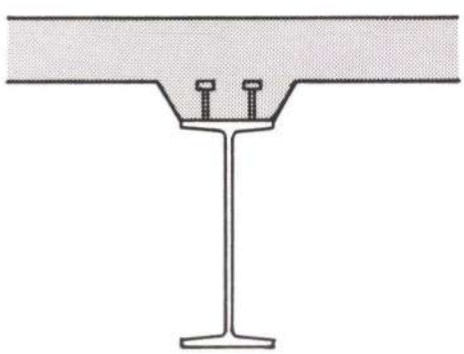


Fig. 13
Haunched beam

Precast concrete decks

These are again ideal for simply supported beams, but care must be taken to ensure that there is sufficient bearing on the flange for the slabs and at the same time sufficient width of concrete to enclose the shear connectors. Care should also be given to providing sufficient transverse reinforcement to prevent splitting of the slab.

Precasting could be an effective way of achieving a haunched beam.

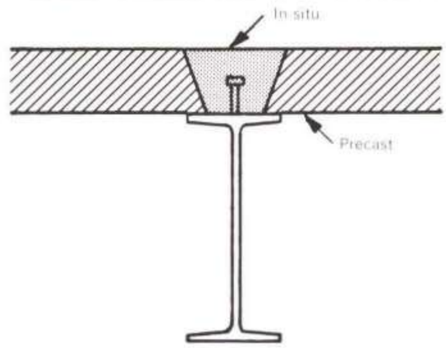


Fig. 14
Precast slab

Formed metal decks

This type of decking is ideal for both simply supported and continuous designs. The shear studs should be welded to the beam through the decking to ensure the beam stiffness and to minimize slip. For rib heights of less than 40 mm there is no significant loss of strength. However, with rib heights greater than 40 mm there is a possibility of shear occurring across the ribs.[17] (See Fig. 15).

Prestressing

This can be achieved in many ways; by conventional stressing methods, the use of *Preflex* beams; or systems which involve jacking. The principle advantage is an increase of the elastic range of the steel beam. Preflexing and prestressing are usually confined to use on simply supported beams, although the jacking system can be useful with continuous beams.

Castellated beams and lattice trusses

Considerable increases in rigidity can be achieved. Due to the problems of web buckling these are not suitable for continuous beam design. The most efficient design will result where the shear forces are low.

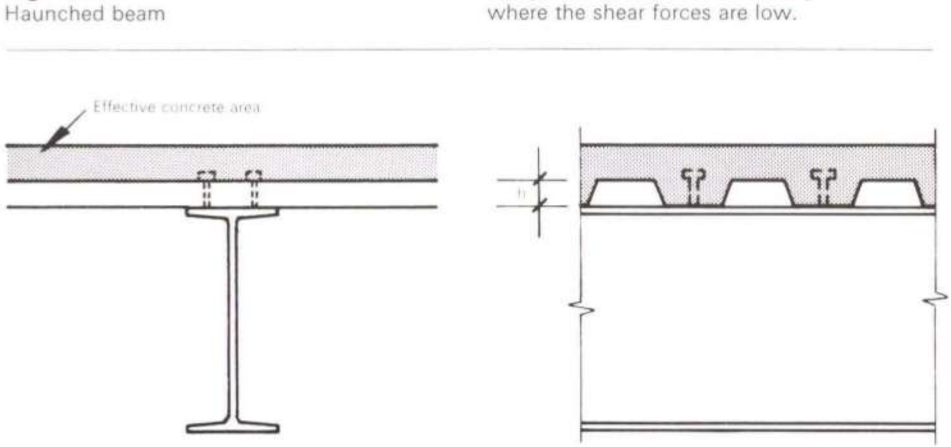


Fig. 15
Formed metal deck

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