THE ARUP JOURNAL

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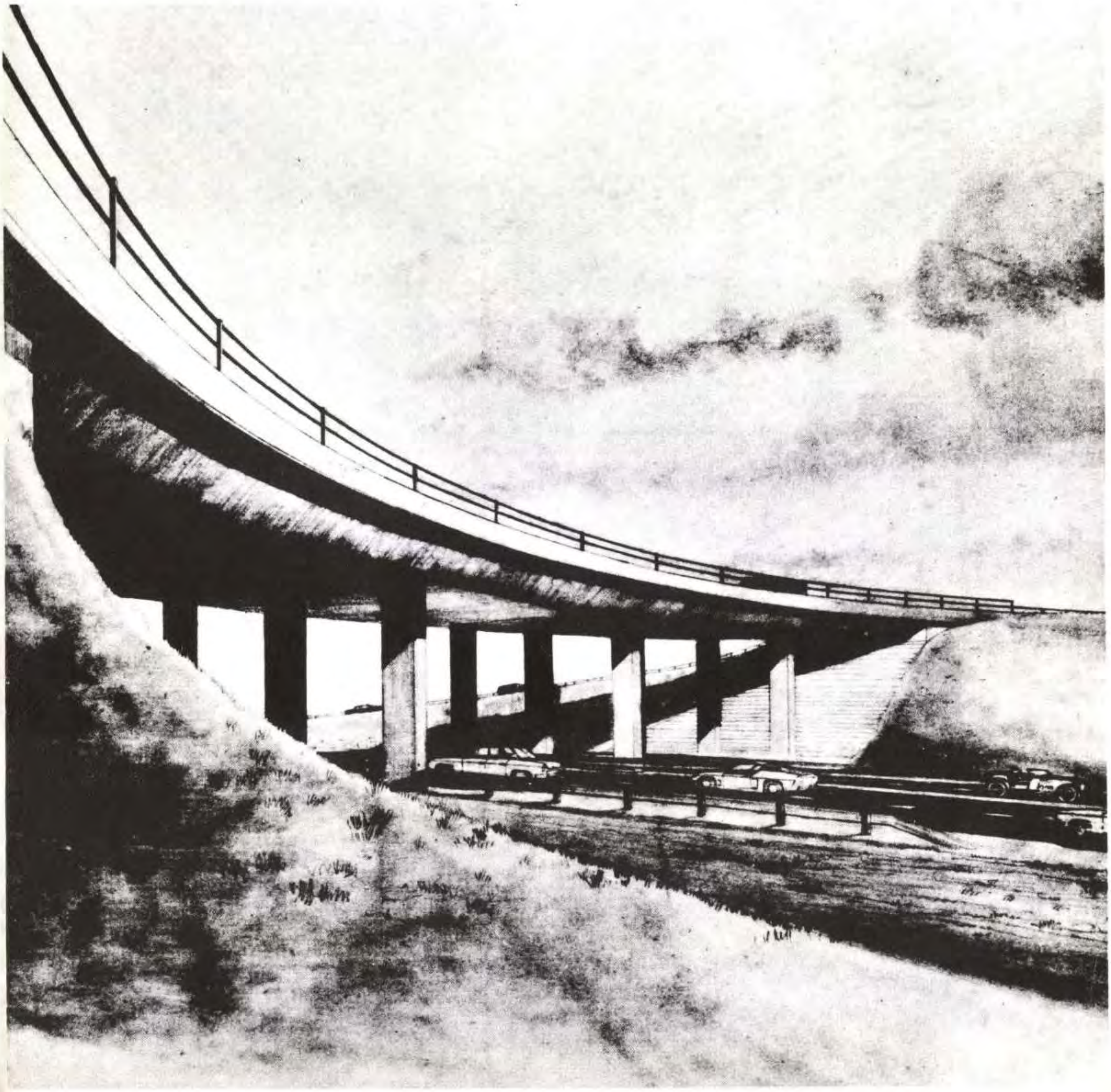

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Bridges on the Gateshead Western Bypass

Keith Ranawake
Duncan Calkin
Ian McCulloch
Chris Slack

Introduction

The Gateshead Western Bypass is the first large road and bridge project which we, as a firm, have designed in this country. This article is concerned with the bridges which form about 27% of the total contract value of £5.5 m. It is intended that a further article concerning the roadworks and the soil mechanics aspects of the project will follow.

The bypass runs from the southern end of the Team Valley Trading Estate to the new Scotswood Bridge, carries traffic from the A1 and also acts as part of the local road network in the Gateshead area. The road runs through both rural and urban surroundings and there are five grade separated interchanges along the total length of 5½ miles. The topography of the site and the rather difficult ground conditions, which include areas of old mine workings, have combined to create difficulties at each bridge location. Care has been taken to give all the bridges a common and unobtrusive character and we have worked closely with Humphrey Wood, of Renton Howard Wood Associates, to achieve this.

Medium and short span bridges

Design

These bridges were designed with the following standards and objectives in mind:

1 The layout of the bridge decks to be determined as far as possible by the most efficient road layout.

2 Economy.

3 Minimum visual obstruction from abutments and piers.

4 Consistency of form and detail.

The first condition resulted in a large range of deck layouts. Horizontal alignments range from straight to those with considerable curvature and skew. Vertical alignments take the form of constant grades, circular curves or sinusoidal curves.

Economic considerations necessitated the inclusion of supports in the central reserve of the bypass. This reduced the maximum span to approximately 15.2 m (50 ft.) except for the A694 bridge which has a central span of 22.8 m (75 ft.)

All the bridges have been designed with open abutments because they give minimum visual obstruction and, for this project, proved to be marginally cheaper than closed abutments.

Various types of deck construction were examined and costed. It was found that the unit deck costs for in situ concrete slabs, composite steel and concrete construction, and prestressed precast concrete beams with in situ topping, are comparable for use in straight bridges. However, the considerable variation in the horizontal geometry of the decks made it clear that continuous in situ reinforced concrete slabs of constant radial cross-section provided the most practical and economic solution for spans up to 18.2 m (60 ft.). A span to depth ratio of about 1:20 limits deflections to acceptable levels. Static to transient loading ratios are generally in the region of 2:1.

The deck slabs are supported at discrete points by columns usually arranged in parallel rows which are also parallel to abutment lines (see Figs. 1, 2 & 3). The columns have been designed using one rectangular cross-section only. It would have been possible to arrange the columns and abutments in radial lines but this would have made the decks longer and would not have suited the topography so well. Each bridge deck is anchored at one point to one abutment and free to move in one direction only at the other abutment (Fig. 6). Three types of mechanical bearings are used at the abutments; fixed point, guided uni-directional and free multi-directional. All bearings permit rotations in three directions. At the columns the deck is supported on pairs of laminated rubber bearings to the same specification. These permit horizontal movement up to 38 mm (1½ in.) and have the same shear stiffness in any direction.

Abutment beams are supported on spread footings, buttresses or piers. Where piling is necessary, bored cast in situ piles up to 610 mm (2 ft.) diameter are used, some of which are raked at anchor abutments. Columns are cantilevered off the bases which bear on spread footings or piles.

In accordance with Ministry of Transport requirements, the bridges have been designed for full HA loading and checked for 45 units of HB loading.

Analysis

The bridge decks and all other structural members have been analyzed elastically. The preliminary deck analyses were rather crude but we were able to make reasonable estimates of upper bounds for individual column loadings and for the total longitudinal bending moments at particular sections. Estimates of transverse bending moments and torsions were less satisfactory but, in any case, they are considerably less than the peak values of longitudinal bending moments.

The final analysis of individual decks has been carried out using grillage programs, or P. Lim's finite element plate bending program at Imperial College (Figs. 7 & 8). The equivalent grillage in the curved decks is formed using straight radial and curved circumferential members. At the time these analyses were being carried out, neither the curved grillage program nor the finite element program had been fully tested. We therefore

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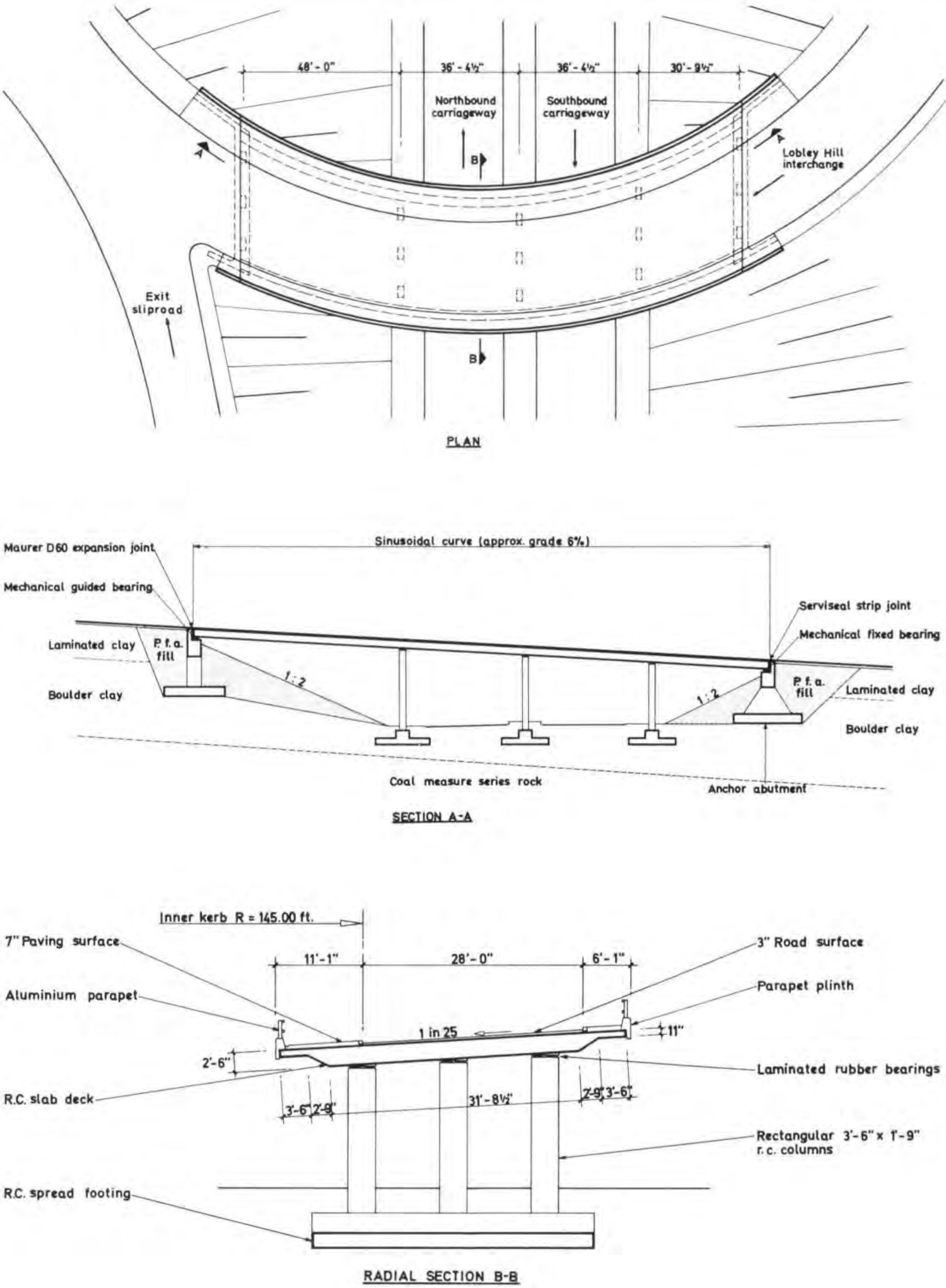


Fig. 1
Lobley Hill South Overbridge

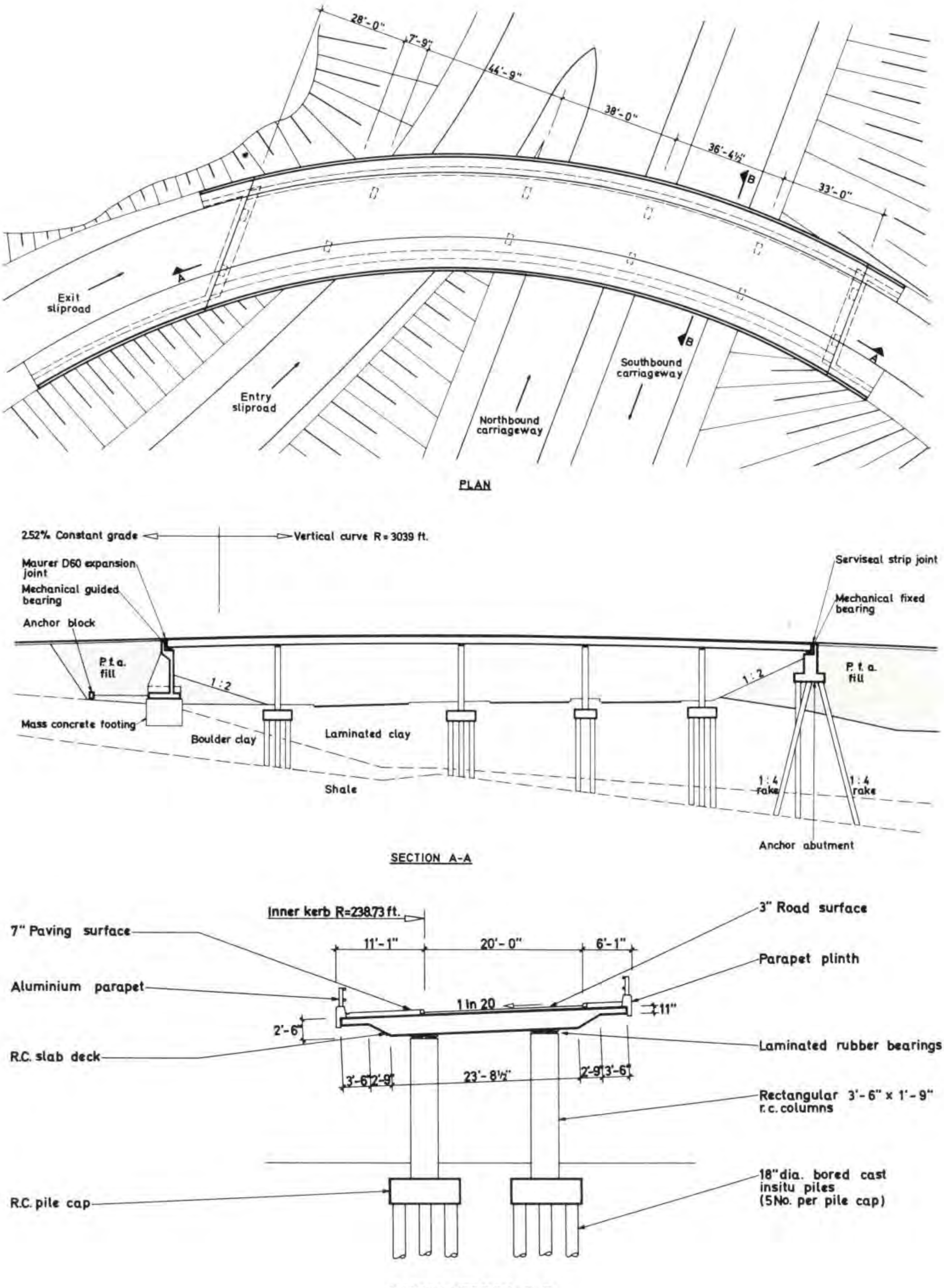


Fig. 2
Consett North Overbridge

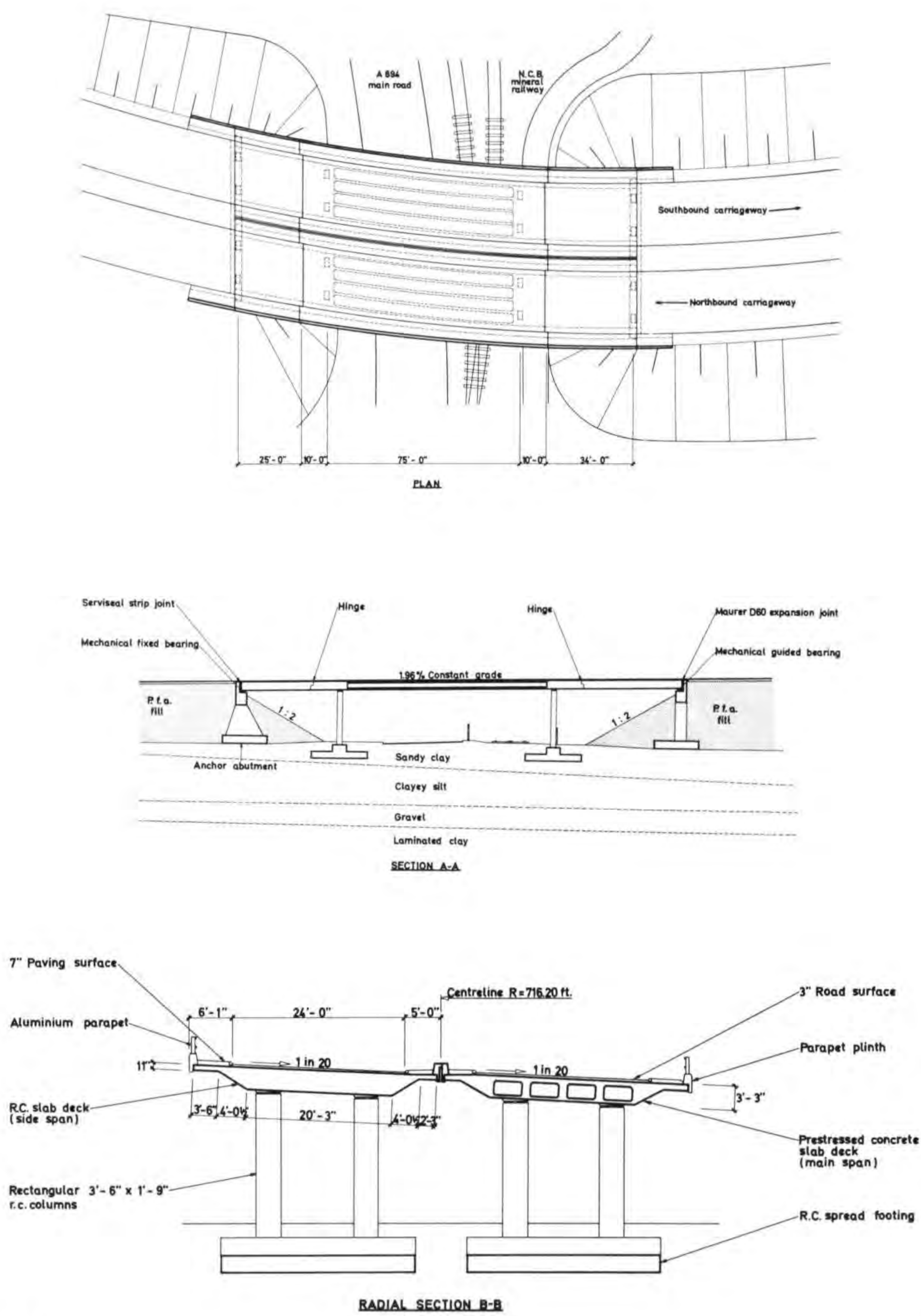


Fig. 3
A694 Underbridge

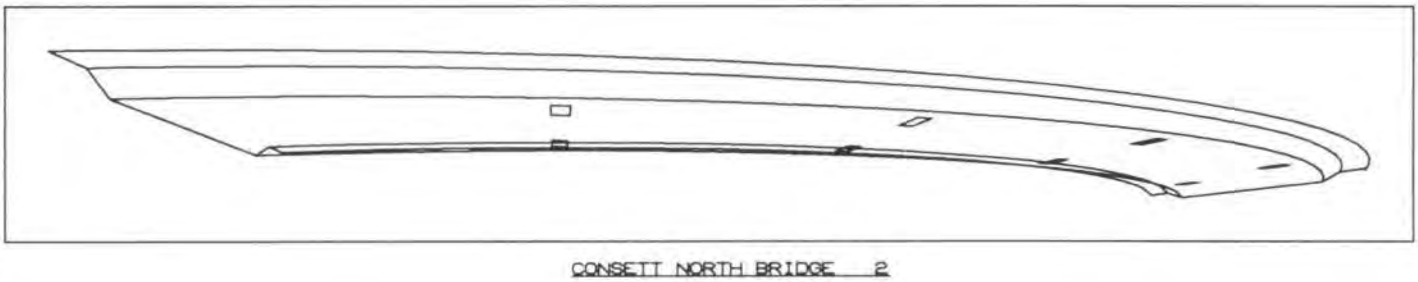


Fig. 4 above
Perspective plot of Consett North Overbridge (Program OA 160)

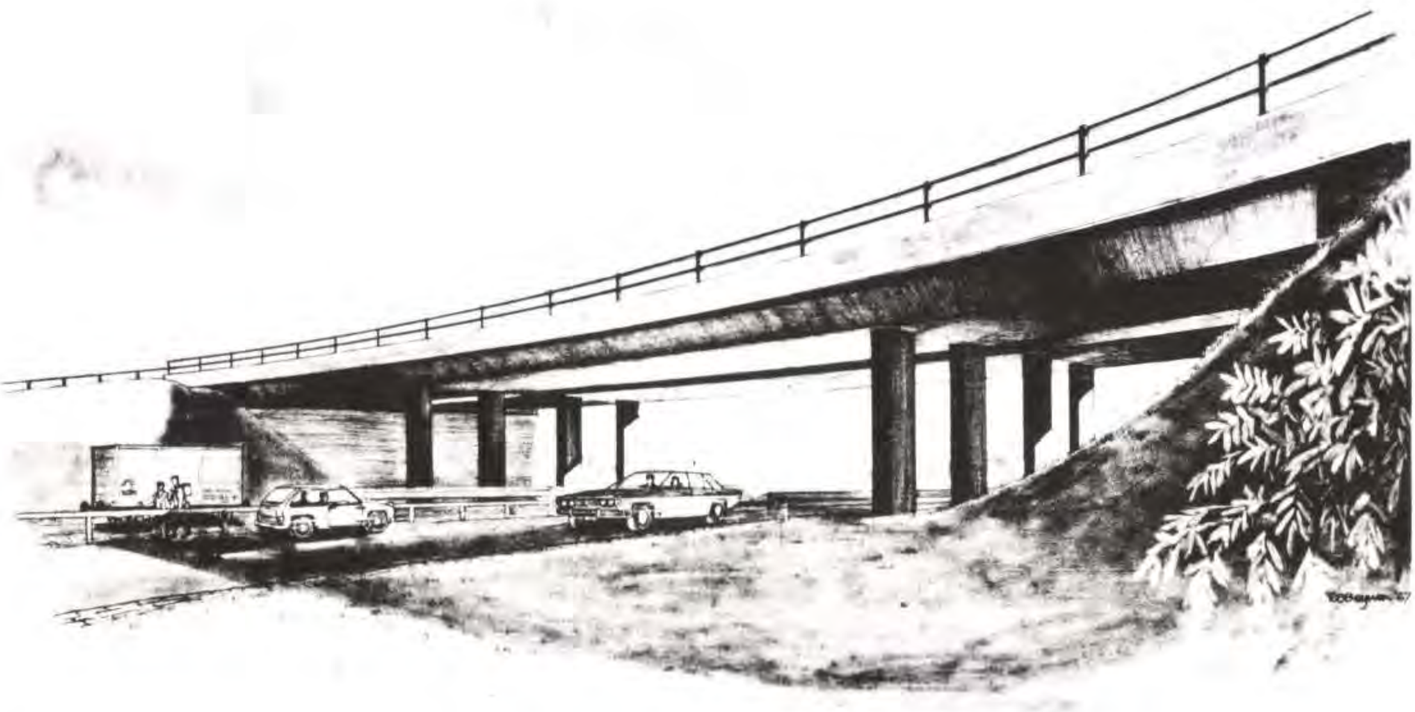


Fig. 5 above
A694 Underbridge (Drawing by courtesy of Renton Howard Wood Associates)

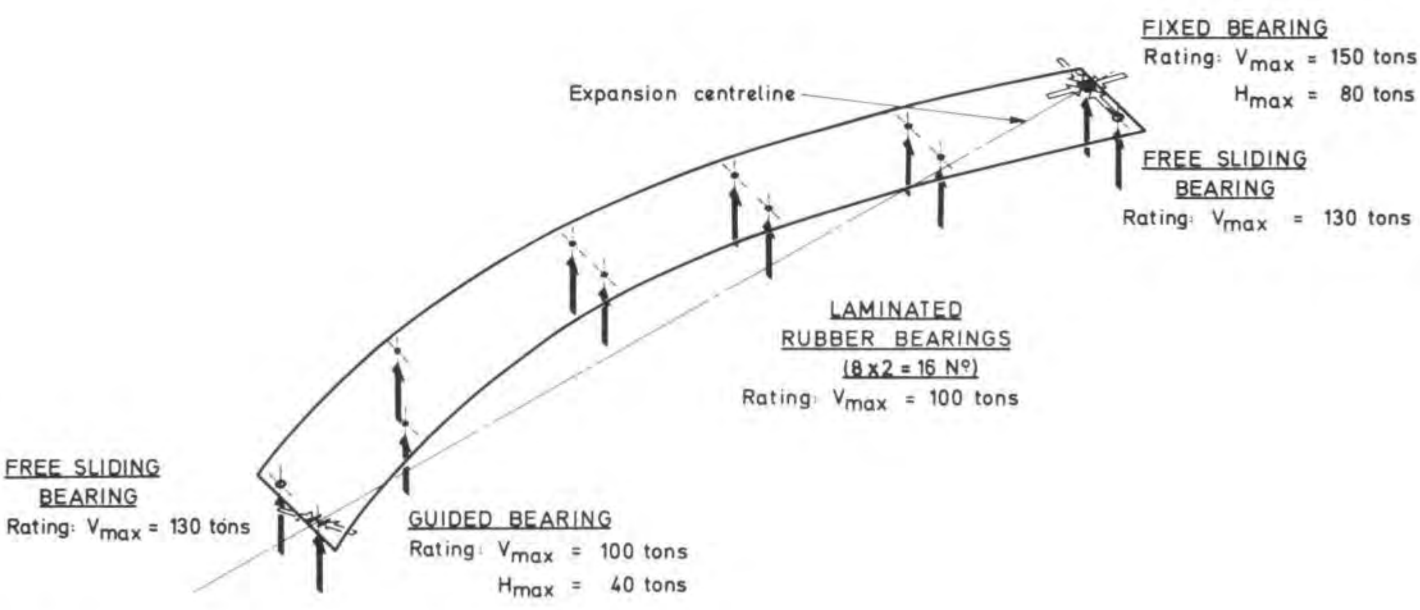
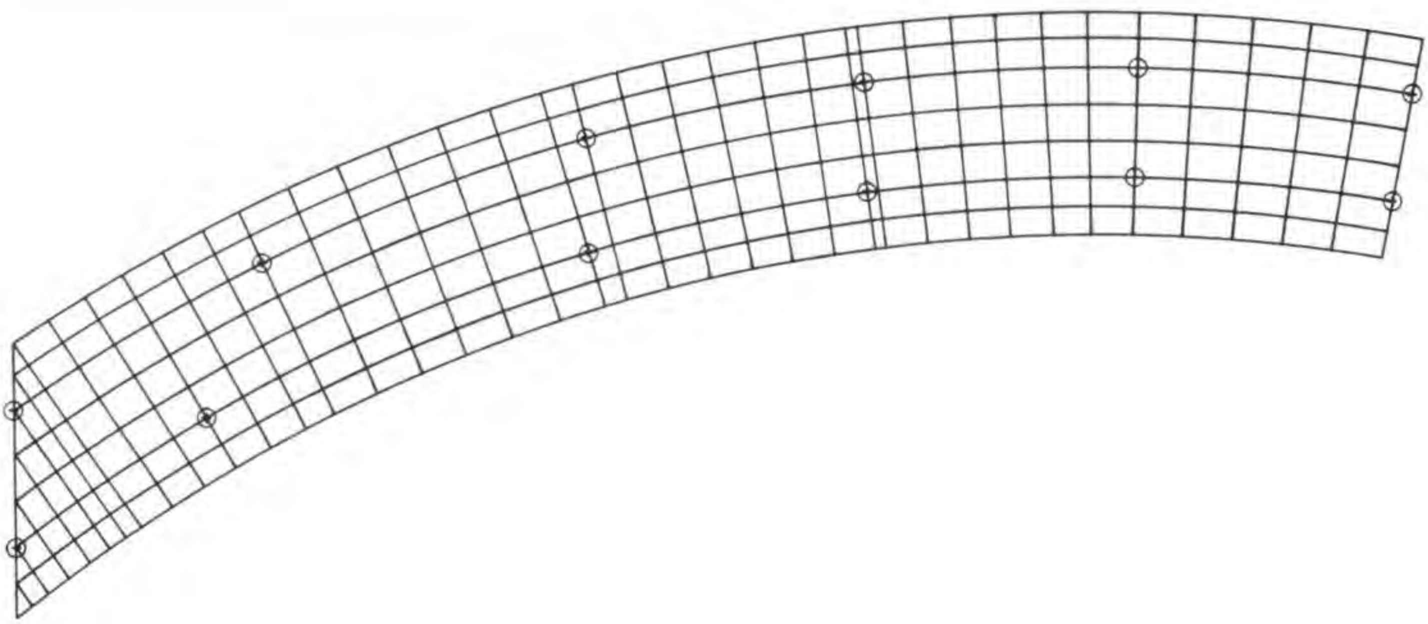


Fig. 6
System of external forces, acting at bearings, for a typical medium span bridge



approached the Ministry of Transport about this problem and were authorized to carry out two model tests at Imperial College under the direction of Dr. J. C. Chapman. The models were made to 1:30 scale out of a homogeneous unreinforced *Araldite* sand mix, the strength of which was designed to ensure elastic behaviour through the design stress range in tension and compression. They were very accurately made and tested. The combined compressive stiffness of columns plus bearings was also simulated as these had a significant effect on the distribution of moments. The effects of differential settlement between rows of supports were also investigated. The results obtained from the tests compared very well with the computer program results (Figs. 9 & 10), under both uniformly distributed and point loading cases. A report is being prepared at Imperial College for the Ministry and may be referred to by anyone interested. Once the validity of the programs had been established the deck analyses reduced to a routine which, as usual, involved the processing of large quantities of data.

In addition to vertical loading, each bridge deck is subject to horizontal forces arising from vehicle braking and the combined build up of forces at the bearings due to deck movement. The specified braking force of 45 tons in the case of the more sharply curved bridges, induces horizontal bending moments in the deck, which do not substantially increase the deck stresses but have to be resisted by fairly large horizontal reactions at the fixed and guided abutment bearings. The resultant horizontal reaction at a fixed bearing can be approximately 100 tons.

The analysis and design of the abutments and columns follows standard procedures but considerable emphasis is placed on limiting both short and long term movements. Crack widths are always limited to a maximum of 0.25 mm (0.01 in.).

In the case of the A694 bridge (Fig. 3), the structural system differs from the others in that the 22.8 m (75 ft.) main span is a prestressed concrete voided slab because the practicable limit for a reinforced slab is exceeded. In addition, hinges have been introduced in the two reinforced concrete side spans to allow for differential settlements of up to 50 mm (2 in.) between the columns and

Fig. 7 above
Consett North Overbridge: equivalent curved grillage (Program OA 108)

Fig. 8 below
Consett North Overbridge: arrangement of triangular 2D elements for finite element analysis (Program OA 115)

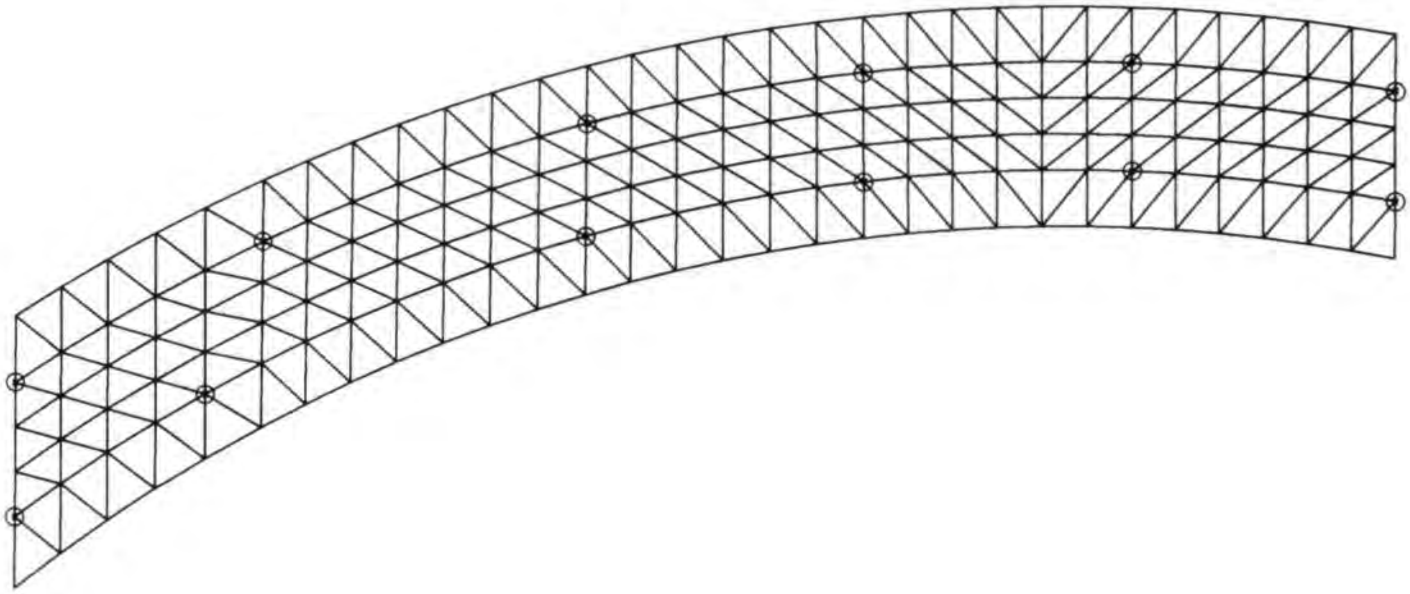
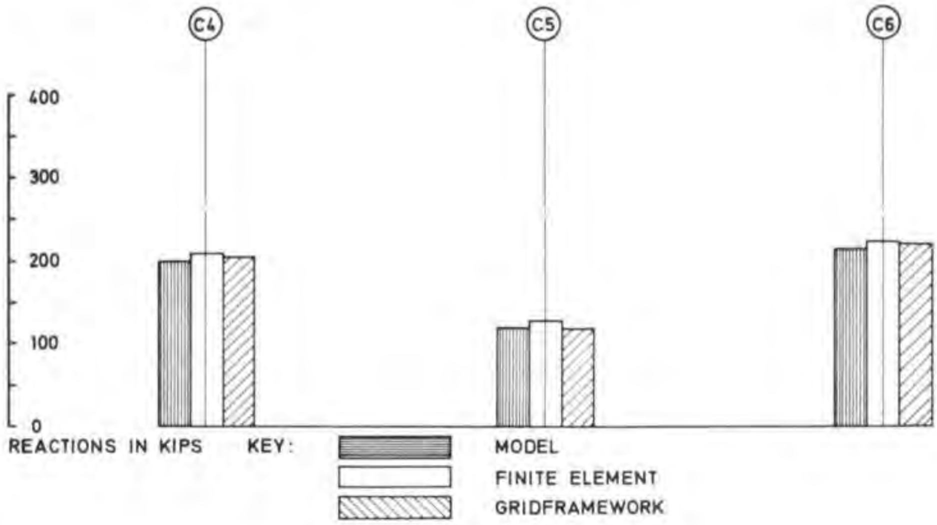


Fig. 9
Lobley Hill South Overbridge: comparison of reactions obtained from analysis and model test for one column line



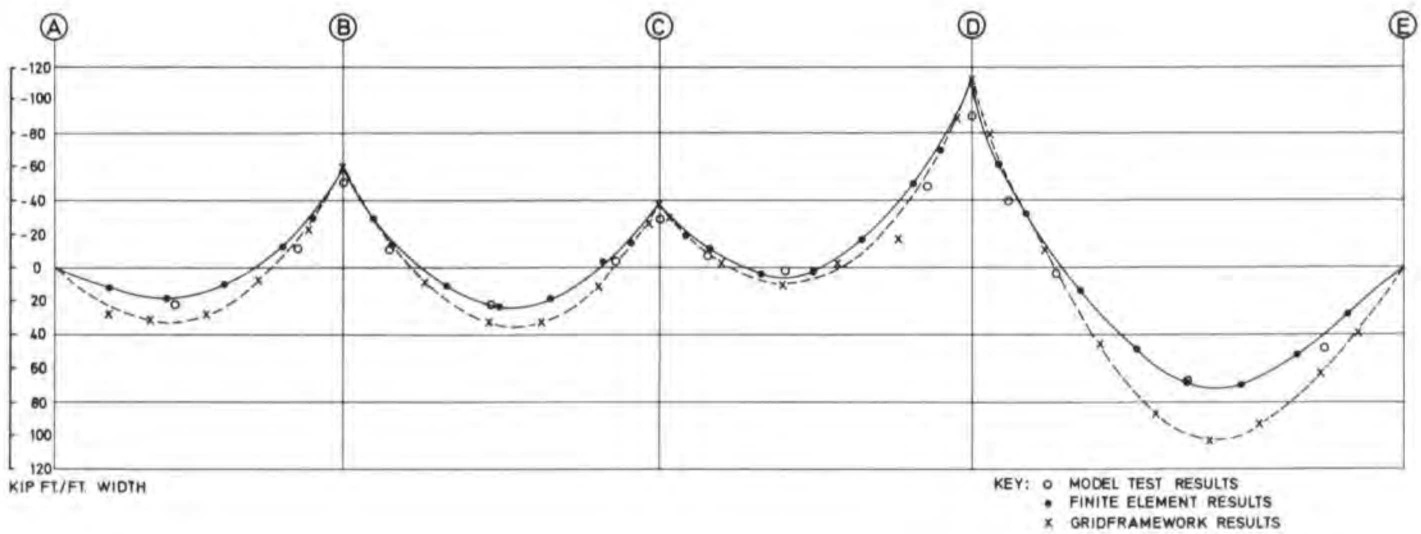


Fig. 10
Lobley Hill South Overbridge: comparison of bending moments obtained from analyses and model test

the abutments. The hinges provide a longitudinal moment release only. Columns and abutments are supported on spread footings at 80.5 kN/m² (¾ ton/ft.²). Piles could not be used economically in this location and ground pre-consolidation by the *Vibroflotation* process is specified to reduce settlements.

Derwent Bridge

Design

This bridge, which has a 50.3 m (165 ft.) main span, crosses over the Derwent River on a skew of 32°. Various schemes were investigated which divided into two main groups; three span arrangements and those with a series of short spans ranging between 15.2 m (50 ft.) and 21.3 m (70 ft.). The latter group proved to be more expensive due to the high cost of foundations in this location. Under the first group, designs in composite steel and concrete were investigated as well as in prestressed concrete. The prestressed concrete scheme finally chosen consists of two structurally independent decks staggered in plan (Figs. 11 & 12). The side spans are shallow voided slabs conforming in shape to the cross section of the short bridges. This section transforms to a two-cell box for the main span and is prestressed with a total of twenty eight 220-ton BBRV cables of which 18 extend the full length of the deck. Problems associated with the skew are thus confined to the flexible side spans. The edge treatment of each deck, the columns and the abutment details, are similar in character to those of the other bridges. The abutment beams and the columns are supported on large diameter bored piles varying in length from 21.3 m to 30.4 m (70 ft. to 100 ft.). These piles are sleeved at the abutments to prevent load shedding onto them from the PFA (pulverized fuel ash) embankment as settlement occurs. For the same reason the abutment piles will be bored through the completed embankments. Horizontal forces are resisted by the raked corner piles under the columns (see Fig. 14). River training works have also been designed to keep the length of the bridge to the minimum and to protect the banks and the main column foundations. These works will also succeed in reclaiming some rather extensive and unsightly mud flats.

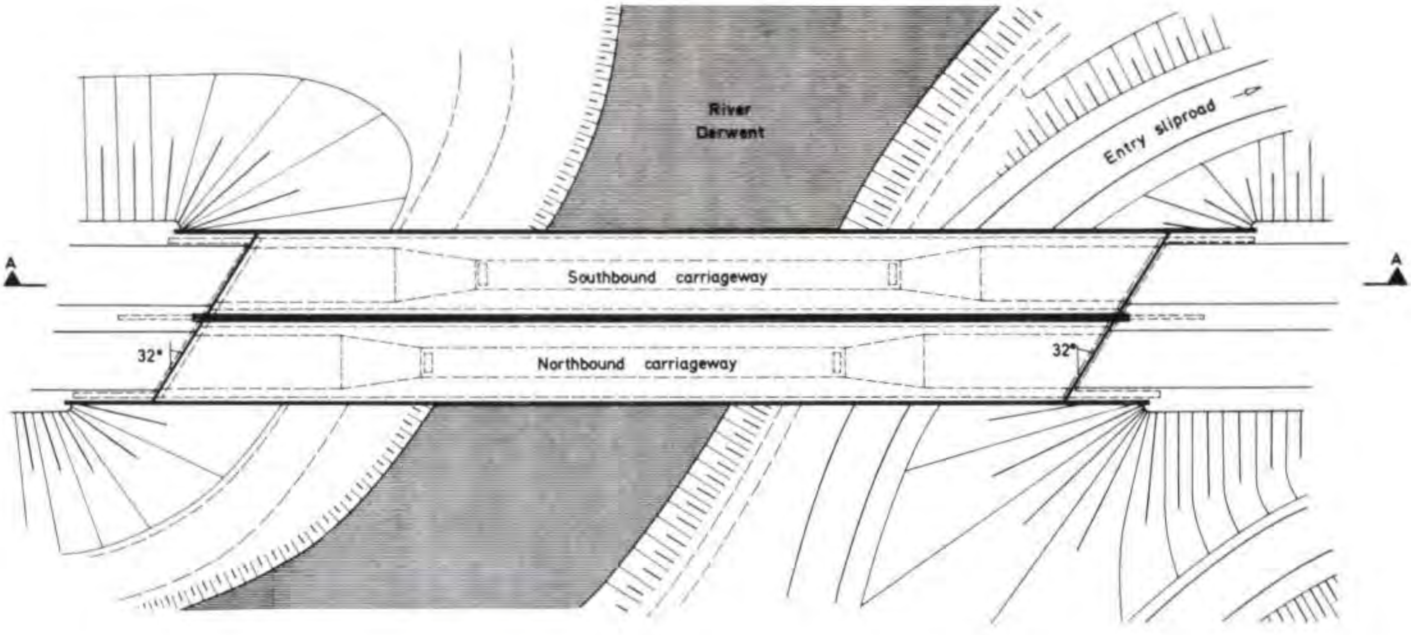
Deck Analysis

Fig. 15 shows the relative properties of the structural elements of one deck. The main span is both flexurally and torsionally much stiffer than the side spans and consequently the deck is considerably less sensitive to the effects of differential settlements than one of constant depth. The columns, being stiffer than the side spans, absorb most of the deck torsion moments except those due to heavy vehicles close to the abutments.

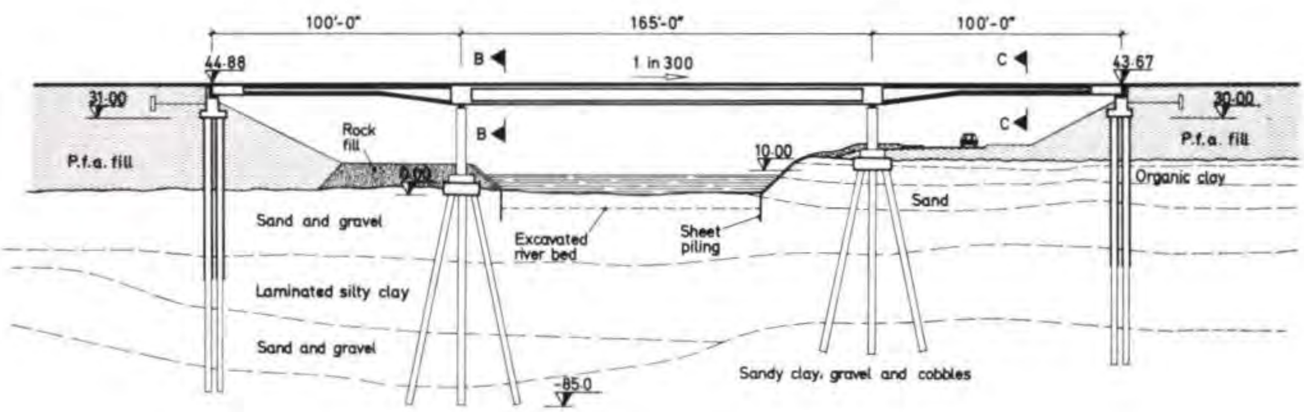
There were several possible approaches to the deck analysis including the use of a space frame program or a three dimensional finite element plate bending program. For a satisfactory analysis of the deck by these methods, a larger computer than the Elliot 4120 would have been required and we were deterred both by the cost and by the prospect of processing large quantities of data. The final decision was to treat the deck as a beam of varying section and to make adjustments and checks for the effects of transverse load distribution, warping stresses and skew.

A simplified chart of the design process is shown in Fig. 16. Difficulties which resulted in repetition of some processes are omitted for clarity. For instance, we found at a rather late stage that the most critical section for longitudinal bending stress is at the end of the transition and not at the support or mid span sections. This, and problems with the cables, involved adjustment of the internal void geometry during the final prestressing analysis.

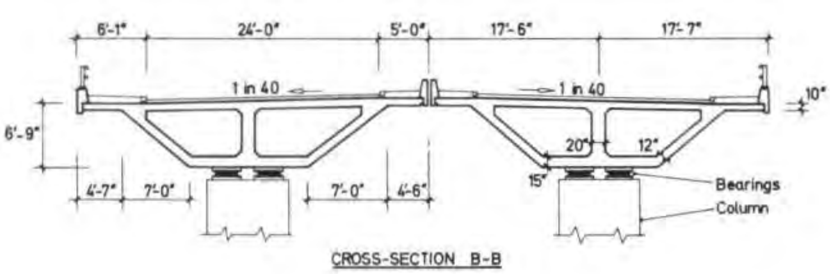
Bending and shear influence lines were drawn after treating the deck as a beam using a plane frame program (PROG 2) and torsion influence lines were drawn from a hand analysis of the deck and column system. These are essential when applying the rather complicated loading conditions specified by the



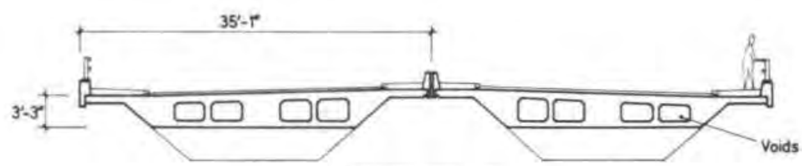
PLAN



LONGITUDINAL SECTION A-A



CROSS-SECTION B-B



CROSS-SECTION C-C

Fig. 11
Derwent Bridge

Fig. 12
Model of Derwent Bridge
(Photo: Henk Snoek)

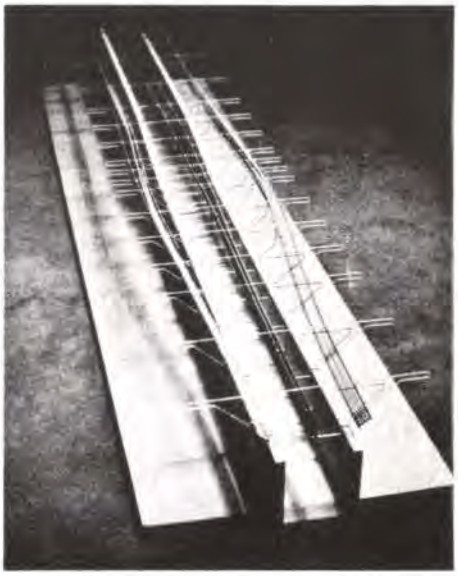


Fig. 13
Model showing prestressing cables for Derwent Bridge (Photo: David Osborne)

Ministry of Transport. The live load bending moments were modified by factors which took account of non-uniform bending stresses across the section due to the load distribution properties of the structure and of warping stresses associated with the distortion of the cross-section (1). These factors were established for the main span both by hand calculations (Vlasov's method) and by using P. Lim's finite element program at Imperial College (PROG 3). In each case a two-cell box with simple symmetrical and anti-symmetrical load cases was analyzed. The side spans were examined using the OAP finite element plate bending program (PROG4).

A major part of the total design effort was expended in prestressing calculations and in the production of cable profiles. The problem was twofold. Firstly, to decide how many cables of which size and at what eccentricity were needed to satisfy the allowable stress limits and, secondly, how to evolve cable geometries which gave the required centroid profile. At the same time the cables had to flow smoothly from the box section to the side spans to give reasonable friction losses. These processes had to be carried out by hand. The profile geometry was solved graphically by drawing plans and elevations because the individual cable profiles could not, for practical purposes, be defined mathematically (Fig. 13).

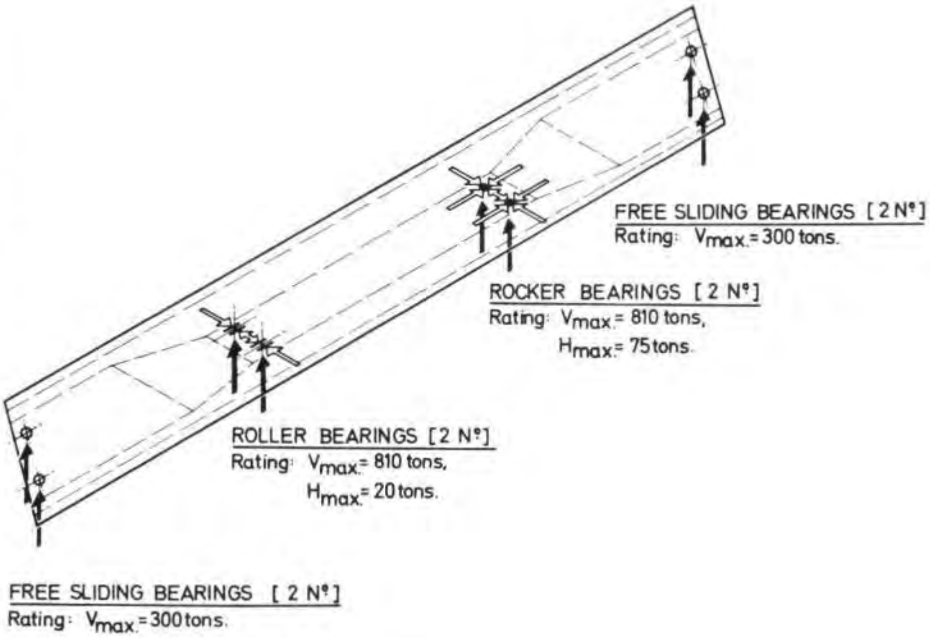


Fig. 14
Derwent Bridge: system of external forces acting at bearings

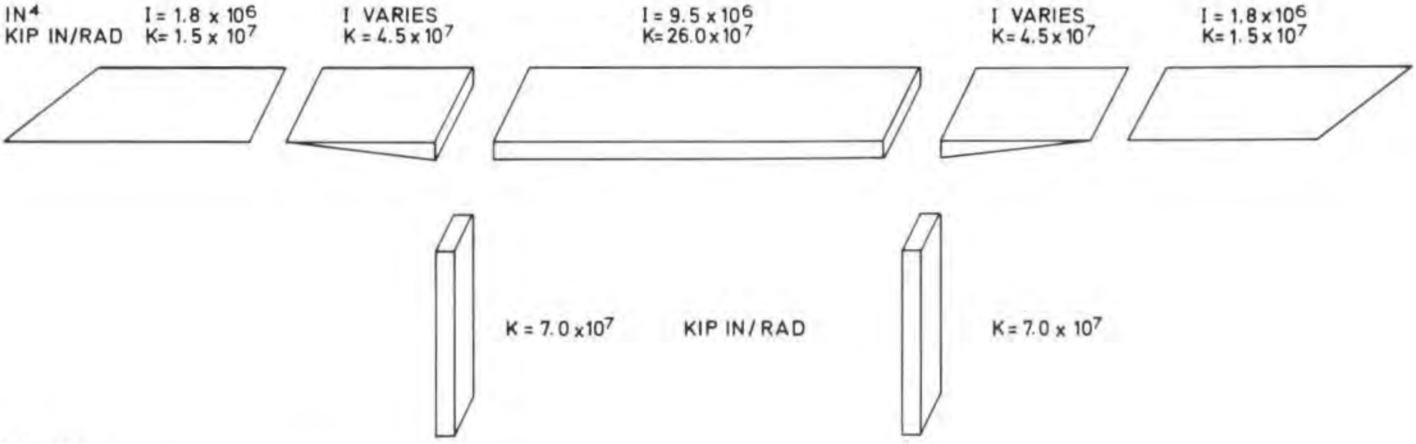


Fig. 15
Derwent Bridge: exploded view of one deck (diagrammatic)

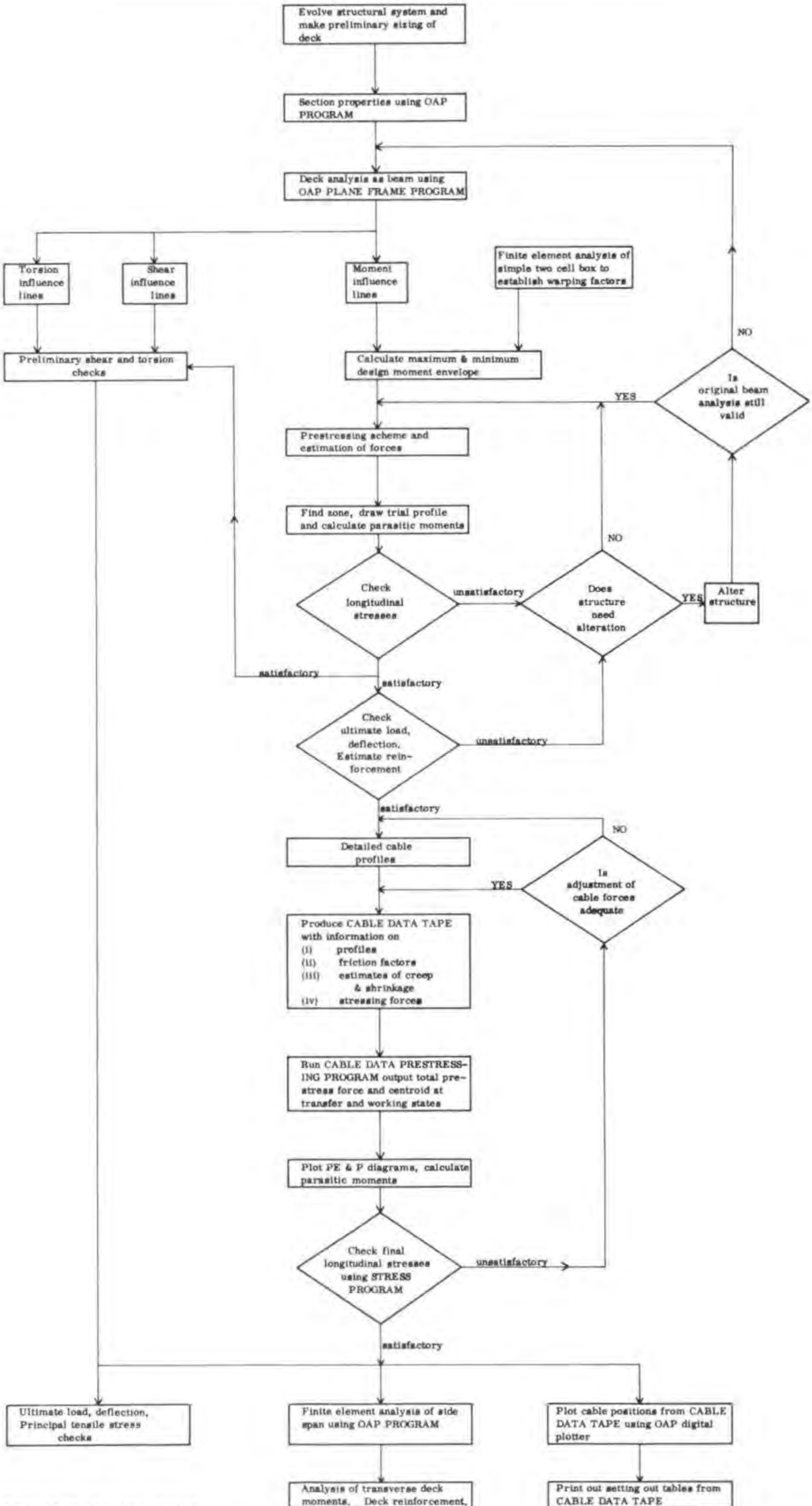


Fig. 16
Derwent Bridge: flow chart showing design process

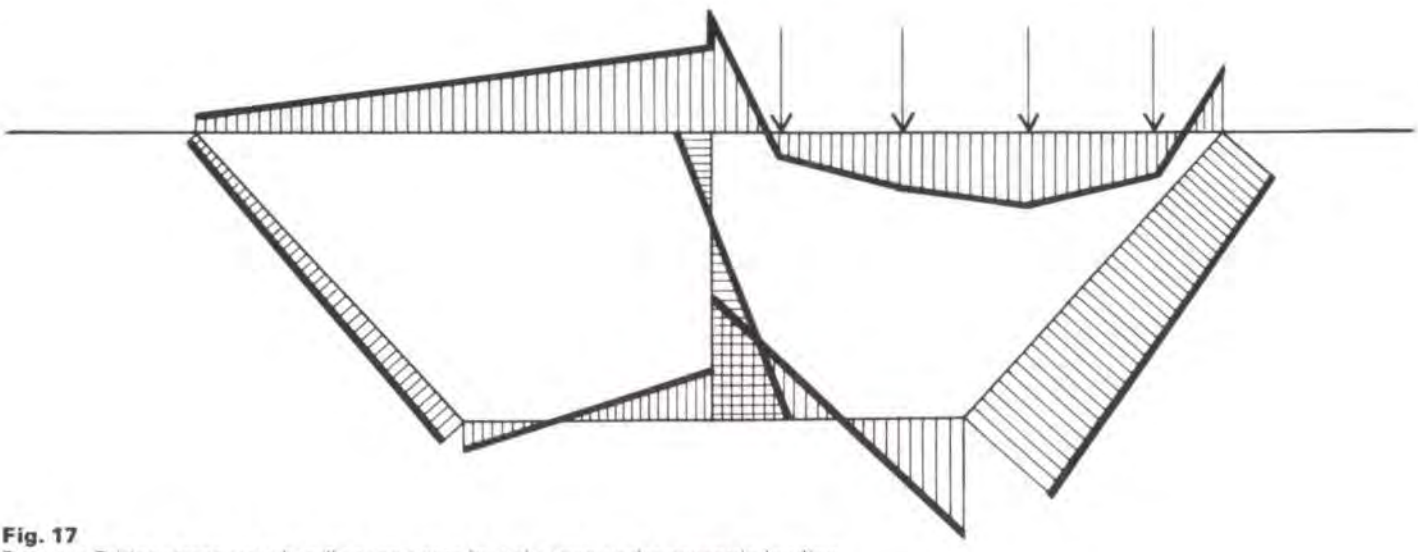


Fig. 17
Derwent Bridge: transverse bending moments in main span under eccentric loading

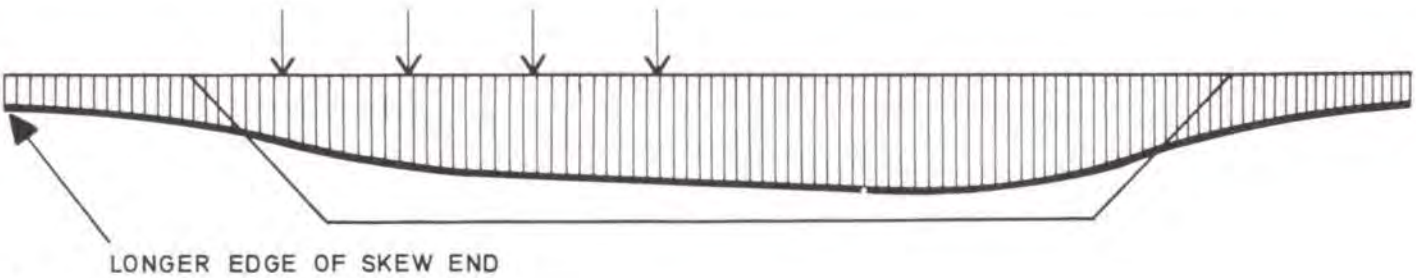


Fig. 18
Derwent Bridge: distribution of longitudinal bending moment across mid span due to eccentric loading

After producing a set of cable profiles and tabulating each one relative to the cartesian co-ordinate system for the deck, we then had to check that the proposed scheme was satisfactory. At this stage, computer programs were written to cope with much of the tedious calculation. The same data tape was used as the input to each of the programs. This contained the co-ordinates of all the cables, stressing forces, friction coefficients, estimates of creep and shrinkage effects and a table of section centroid heights. The tape was first run with a checking program (PROG 5) which printed out difference tables from the cable co-ordinates and enabled data errors and kinks in the profiles to be corrected. A general friction losses program was written to deal with cables curving in both plan and elevation. This was then built into a larger program (PROG 6) which, when fed with the main data tape, gave, at each design section, the total force and eccentricity at both transfer and working conditions. Parasitic moments, which were small due to the relatively flexible side spans, were calculated by hand using the influence coefficient method. The final bending stresses were worked out with the aid of a short stresses program (PROG 7). When a satisfactory prestressing scheme had been achieved the main data tape was input to the computer and the digital plotter drew cable positions on deck cross sections at a suitable scale for the preparation of detailed drawings, (PROG 8). Another program (PROG 9) interpolated the profiles at 760 mm (2 ft. 6 in.) intervals and printed setting-out tables.

For the analysis of the side spans, and to test the assumption that the skew of the abutment support did not disturb the beam action of these spans, the finite element plate bending program (PROG 4) was used. The prestressing was simulated by lines of point loads and a program (PROG 10) was written to calculate the data. Various load cases were run and the results confirmed that there would be no uplift at the abutment bearings and that the distribution of longitudinal bending stresses across the section was within the limits allowed for by the beam analysis factors (Fig. 18). This work also determined the transverse bending moments and torsions for the reinforcement design. The main span was reinforced for the transverse action on the basis of hand calculations on the distortion of a slice of the deck treated as a frame (Fig. 17).

Extensive use was made of the computer to reduce the amount of manual computation, some of which arose directly from the shape of the deck. Seemingly small details, such as the soffit not being parallel to the deck surface, complicated the design, particularly when calculating section properties and working out cable profiles. When using structural analysis programs, and then trying to understand apparently inexplicable computer results, we were often forced to look at the action of the structure in new or simplified ways in order to enable quick hand checks to be made. Generally these agreed well with information won from the computer after considerable processing of data.

Computer programs used in deck analysis

Program

1 Section properties (OA 63)
2 Plane frame (OA 100)
3 Finite element 3D plate bending (Imperial College)
4 Finite element 2D plate bending (OA 115)
5 Data check using finite differences*
6 Prestressing program*
7 Section stresses*
8 Cable plotting on cross-sections*
9 Setting out tables*
10 Equivalent loading from prestress*

*These programs were written in the course of the design.

Kingsway South Viaduct

Design

This viaduct crosses an old glacial valley now filled with soft alluvial deposits to a depth in excess of 46 m (150 ft.). Piled foundation costs are, therefore, high and comparative estimates were made to determine the most economical arrangement of spans. The approach embankments are expected to settle from 760 mm (2 ft. 6 in.) at the centre to 305 mm (1 ft.) where the fill terminates.

The deck consists of two continuous structurally independent halves with the end spans hinged to allow for differential settlement (Fig. 19). The profile of each half is similar to the medium and short span bridges and the same details are used. The continuous spans are voided slabs, prestressed with eighteen 220-ton BBRV cables. The floating end spans are constructed in reinforced concrete.

The columns and abutments are supported on 1.2 m (4 ft.) diameter bored cast in situ concrete piles extending approximately 30.5 m (100 ft.) below existing ground level and under-reamed at the base. The down drag on the abutment piles is severe and cannot reasonably be prevented. This problem also occurs on the nearest column pile groups but has been minimized by locating these columns as far as possible from the embankments. The differential settlement between the column pile groups is not expected to exceed 13 mm (½ in.).

Analysis

The final analysis of the deck is being done at present. The longitudinal actions are calculated, treating the prestressed section of the deck as a continuous beam. The actual distribution of longitudinal moments across the section, as well as the transverse bending moments and torsions, is being determined for a limited length of the deck, one span plus, using the grillage program OA 107 which allows for shear deformations of the equivalent transverse members. This program was

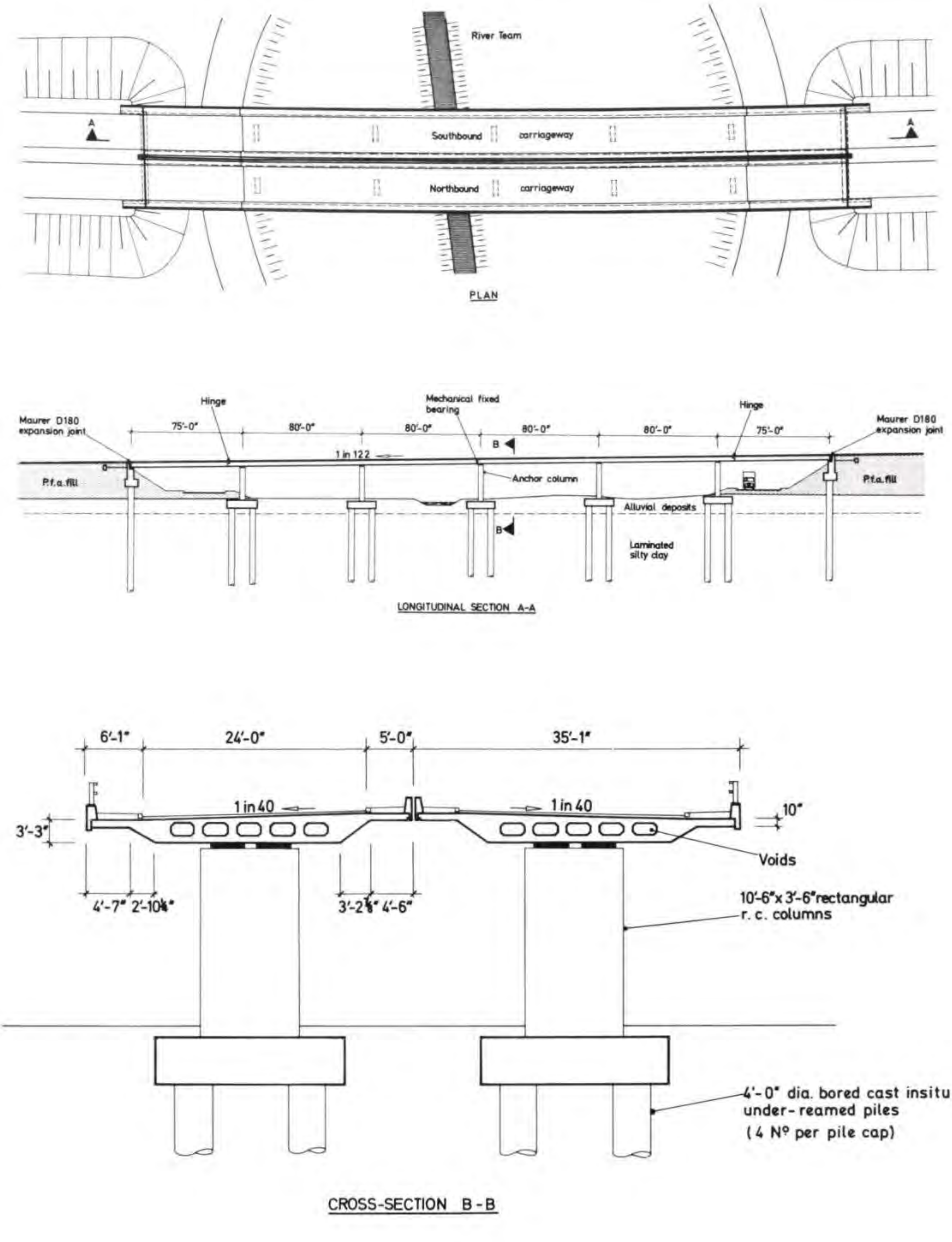


Fig. 19
Kingsway South Viaduct

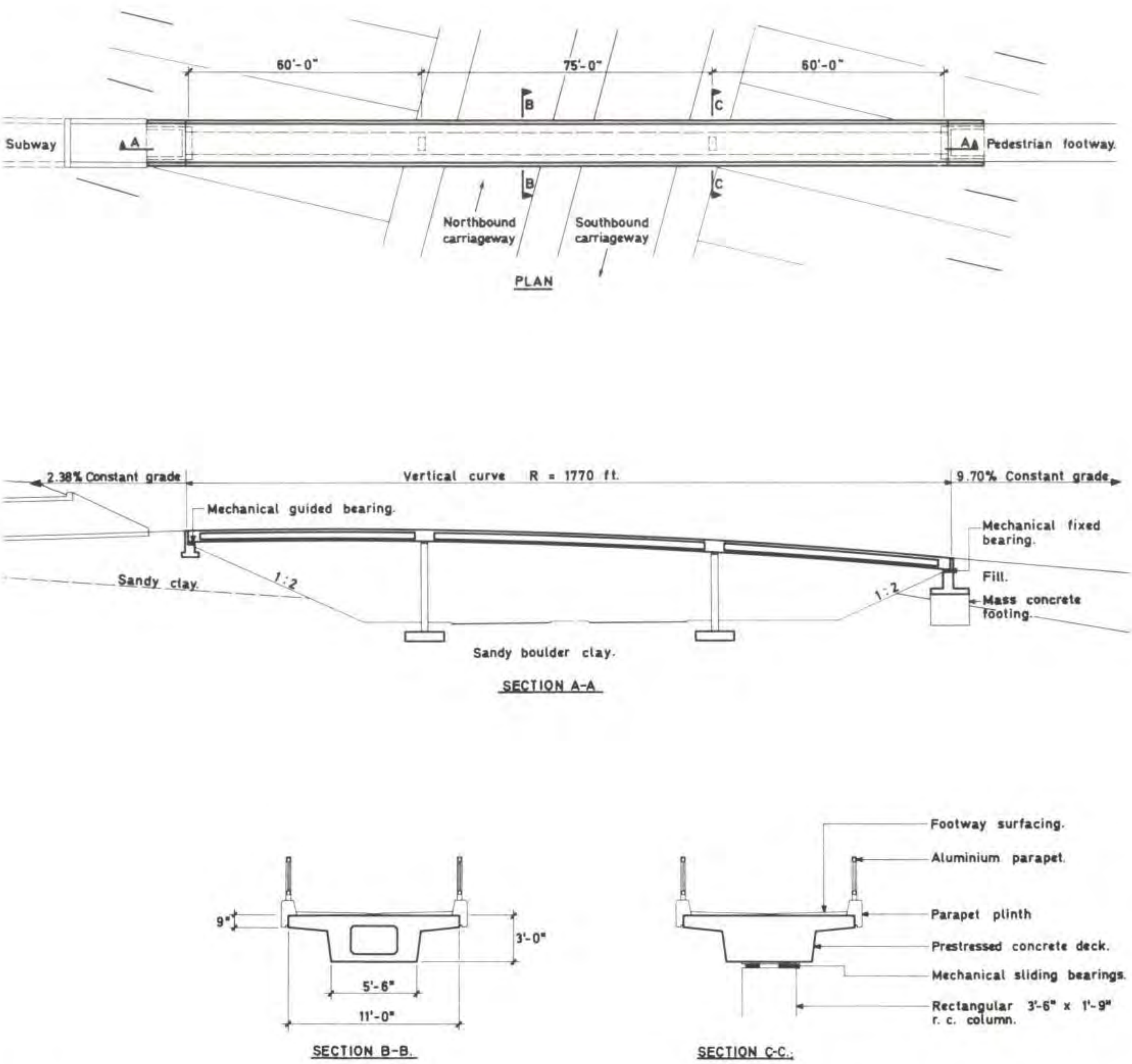


Fig. 20
Lobley Hill Footbridge

developed for the A1 Viaduct, Gateshead. Out-of-balance torsional moments in individual spans are taken in bending at the columns because the torsional stiffness of the column support system is higher than that of the deck. The floating end spans are analyzed as simply supported rectangular slabs using the same grillage program mentioned previously.

Footbridges

The largest spans on the footbridges (Figs. 20 & 21) range between 15.2 m (50 ft.) and 30.5 m (100 ft.). A deck cross-section has been designed which can span up to 18.3 m (60 ft.) as a solid reinforced concrete T-beam, or up to 30.5 m (100 ft.) as a hollow prestressed section with the same overall profile. Ramps are provided where necessary. In each footbridge the spans are continuous over a single row of column supports. The columns have the same cross-section as those for the medium and short span bridges. The edge treatment of the decks and the design of the abutments are also similar in form to the road bridges. The decks are restrained horizontally at the abutments.

Gateshead Western Bypass estimated cost of structures

Road Bridges	£
Kingsway South Viaduct	310,000
Lobley Hill South	60,000
Lobley Hill North	62,000
Consett South	53,000
Consett North	62,000
Ellison Road	45,000
Dunston Road	65,000
Derwent Bridge	269,000
A694 Bridge	71,000
Footbridges	
Lobley Hill	12,000
Chiltern Gardens	11,000
West Way	27,000
Ancillary structures	
Pedestrian Subways Accommodation Subways Retaining Walls Underpasses RC Culverts	408,000
Total cost of structures	£1,455,000

Conclusion

At the time of writing, the preparation of draft contract documents is nearing completion and we hope to go out to tender later this year. The anticipated contract period is two years but this may be extended.

CLIENTS

Our joint clients for this project are J. L. Hurrell Esq., Borough Planner and Engineer, County Borough of Gateshead, and W. H. B. Cotton Esq., County Engineer and Surveyor, Durham County Council.

ACKNOWLEDGEMENTS

The drawings for this article were prepared by Susan Pickard, Roy Butler and Ian Fowler.
We would like to thank Renton Howard Wood Associates for giving us permission to use their perspectives.

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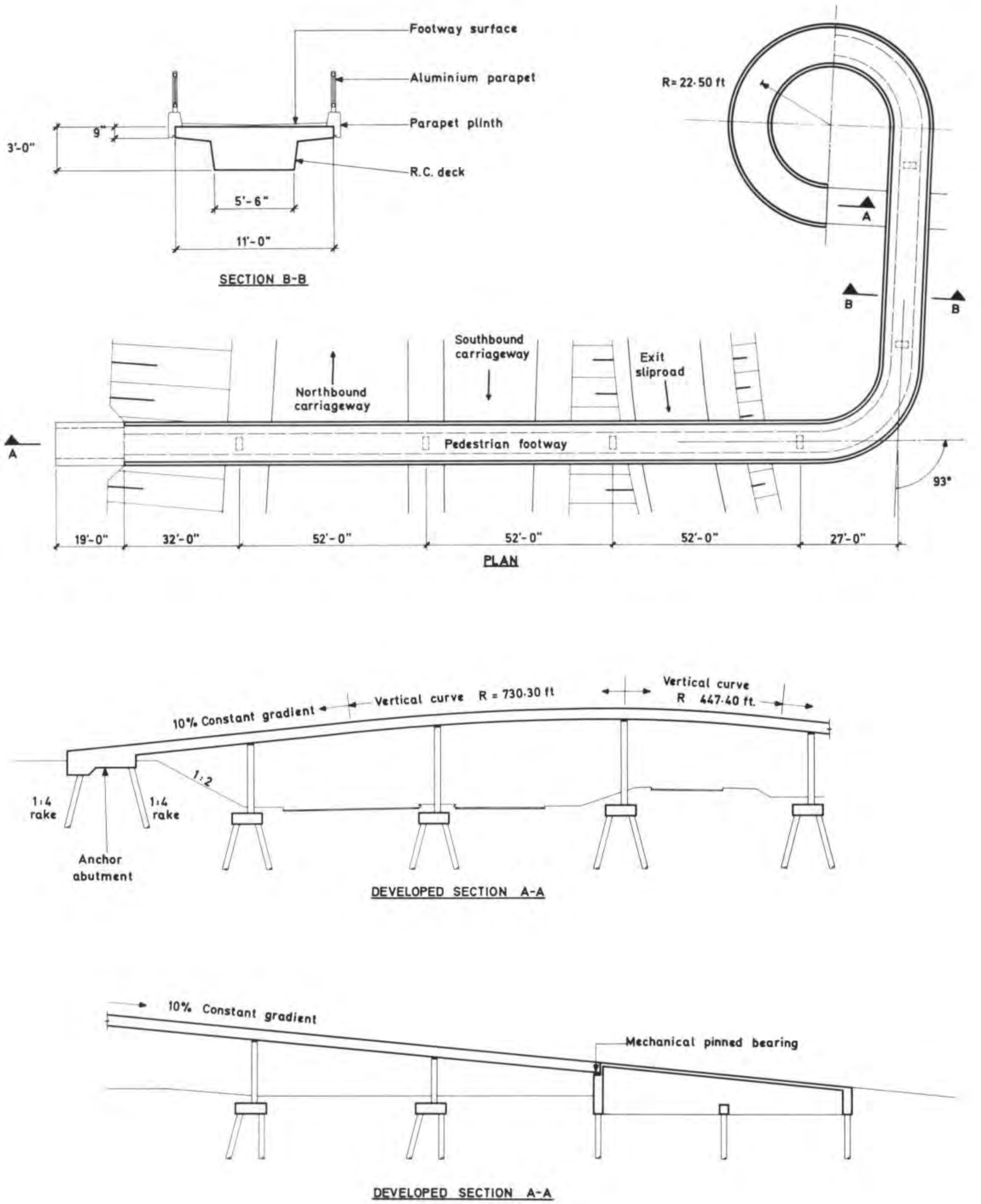


Fig. 21
West Way Footbridge

Sunderland Civic Centre

Sam Price

Working method

When work started on the design of the new Sunderland Civic Centre in March 1965, Mark Kitchen and I moved into Sir Basil Spence, Bonnington and Collins' office in Fitzroy Square and thus completed a professional circle. Spence's had already established their own section of services engineers, and Reynolds and Young, their quantity surveyors, had installed a team of surveyors in the same building. There was, therefore, within one office, a complete multi-professional group working on the new project. For the other designers and ourselves the Sunderland experiment was a most interesting and enjoyable experience, and there was a general feeling that the building had profited from this method of working. It was a pity that, during detail design, the number in the team grew to about fourteen, and communications suffered. As a result of this some of the later work was designed more than once. But perhaps the only way of avoiding this is to stick to small jobs.

The design

For many years the original imposing Victorian buildings have been unable to contain the expanding departments of the Sunderland Corporation, and the Council Offices are, at present, housed in a variety of buildings scattered all over the town. Although some still enjoy the standards of the good old days—the Borough Engineer has a fine view from his bay window onto the main shopping street—others are not so fortunate, and some of the departments occupy very old and inadequate buildings. So in 1964 the Corporation asked Sir Basil Spence to design them new accommodation. The site for the new building was West Park, a fine open space overlooking the centre of the town from the south. It used to be a quarry about a hundred years ago, but during this century, at least, has been public open space. Not all the ratepayers were in agreement with the Corporation that this was the ideal place for new council offices—one man, in fact, stopped on his daily walk with the dog to express his views very clearly to us as we stood in West Park looking at the early plans and trying to make out the extent of our proposed building.

An Ordnance Survey map of the site is shown in Fig. 2. Most of West Park is flat and lies between a large red-brick Victorian school to the west and Burdon Road, in the cutting, to the east. At the northern end the ground drops steeply to the top of the railway cutting, which is a nearly vertical rock face about 7.5 m (25 ft.) high. The railway, which carries coal from, and sometimes to, Newcastle, is still working. At the top of the railway embankment runs a 384 mm (1 ft. 3 in.) diameter pipe carrying sea water at high pressure to Turner and Newall's chemical works at Washington. Further to the north is a triangle of rough land entirely enclosed by railway lines. This was not originally included in the site.

The brief asked for an office block of about 18,200 m² (200,000 sq. ft.), a civic suite, containing a council chamber and committee rooms, and car parking for staff and visitors to the new building. In early discussions about the project the Council pointed out that they had plans for building a multi-storey car park on the triangle of scruffy land to the north of the railway line. The architects suggested that if this were included in their brief, they could carry the car park over the railway and integrate it with the new building. The parking space could be partly for the civic centre and partly for the public using the town's shopping centre. The Corporation agreed to this idea and the brief was extended accordingly. Reconstruction and widening of the bridges on Burdon Road and Park Lane were also included in the brief and the architect's proposal to link the new Civic Centre to Mowbray Park with a footbridge spanning Burdon Road was also accepted.

The brief stressed the need for easy access at ground level to most of the departments, preferably with separate entrances, so that, for example, people coming to a wedding in the registry office didn't get mixed up with road sweepers collecting their pay. The car parking, both for visiting public and for the council staff, was required as close as possible to the offices.

From this brief Jack Bonnington developed

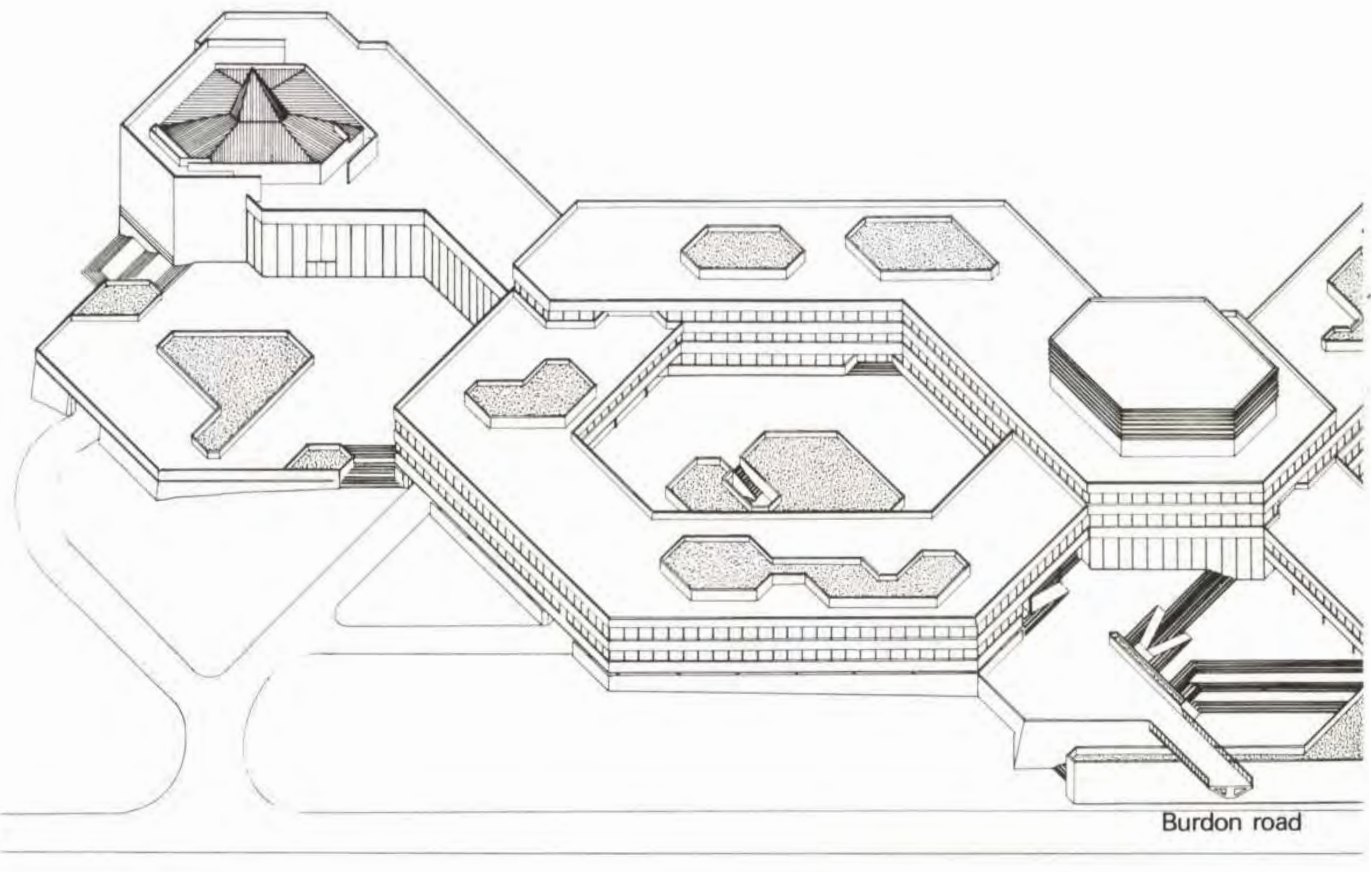


Fig. 2 right
Site plan

rough outlines for the building. These early sketches showed a building almost identical in massing and arrangement to the final design (Fig. 1). Two, three, or four storeys surround two large hexagonal courtyards linked together at one point to form a figure-of-eight. The civic suite joins onto the southern end of the office block and the south courtyard completely covers car parking at ground level. There are wide underpasses through the building into the courtyards, and several entrances from each courtyard into the building. In this way the easy access to the various departments is provided.

This was the point at which we moved up to Fitzroy Square. There were no floor plans, only the vaguest idea of elevations, and a few basic principles:

1. The office building would be in some parts about 12 m (40 ft.) wide and in some parts about 18 m (60 ft.) depending on whether the section was office/corridor/office or office/corridor/storage space/corridor office (Fig. 3a).
2. There would be car parking under quite a lot of the offices.
3. At the pedestrian underpasses through the building into the courtyards columns were acceptable at reasonable spacings. The edge columns should preferably be a little back from the face of the buildings above.

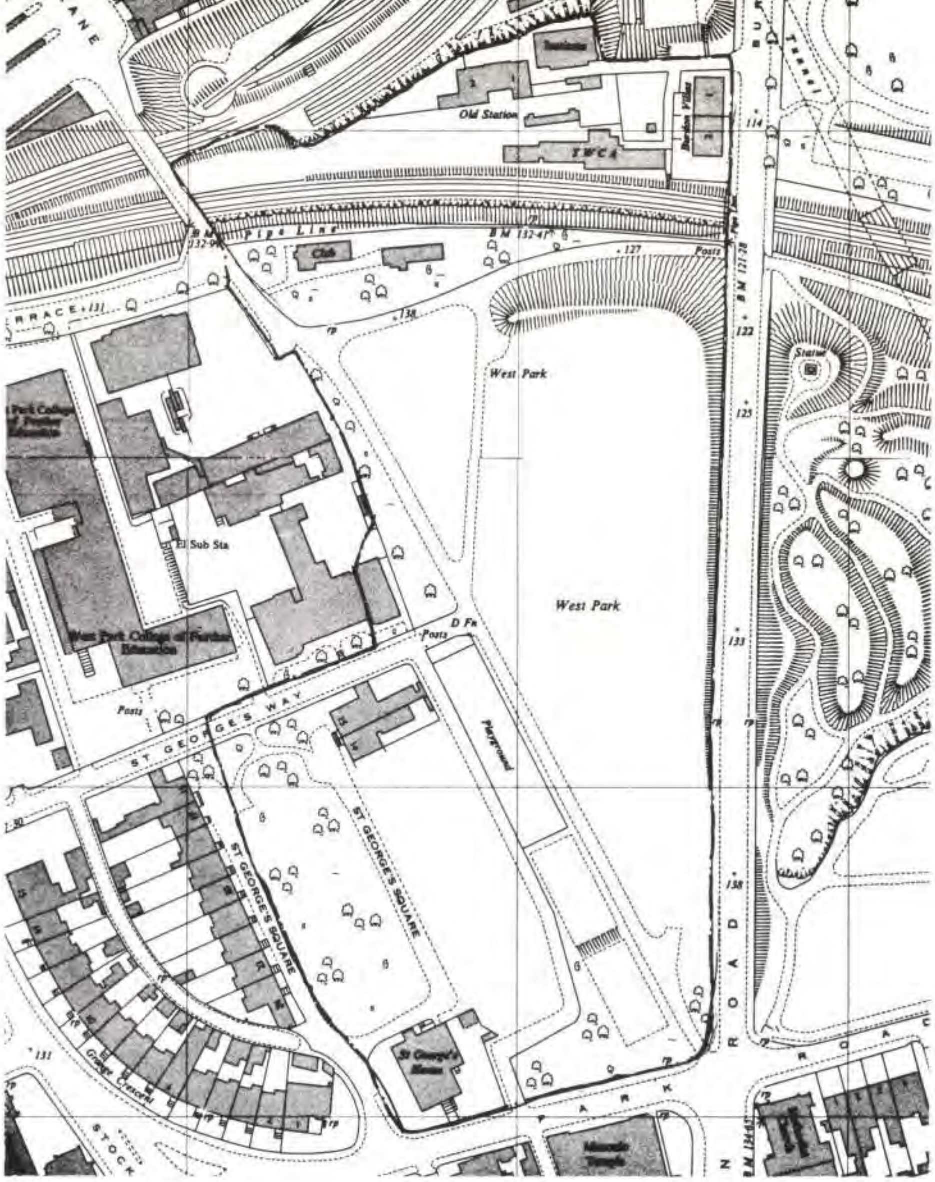
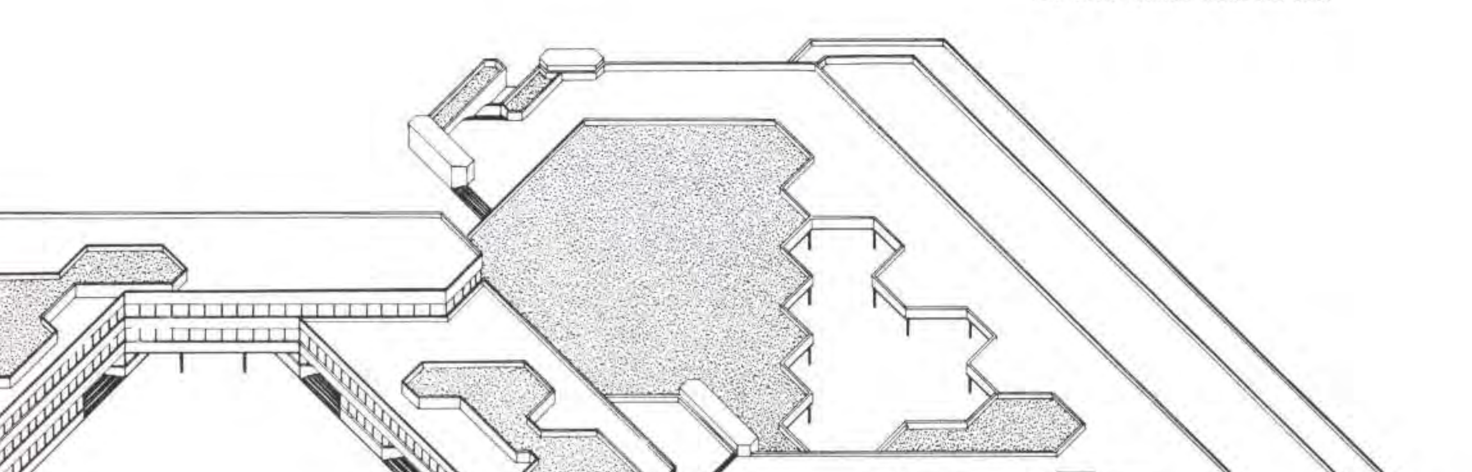


Fig. 1 left
The Civic Centre from the east



4 A convenient module for the planning of offices was thought to be about 1.5 m (5 ft.), and the edge of the building should preferably be supported on concrete mullions at about 1.5 m (5 ft.) centres. This is a feature of the recent Spence, Bonnington and Collins' buildings at Southampton University.

5 The foundations were unlikely to affect the design as there was good rock very near the surface.

The chief problems facing both the architects and ourselves were how to turn 60° and 120° corners, both internally and externally, and sometimes both at once at different levels, and how to plan car parking under the offices so as to avoid awkward and expensive beams transferring loads at high level in the car park.

The solution developed is shown in Fig. 3b. A major grid of 6.1 m (20 ft.) equilateral triangles covers the whole site of the building. All columns stand at the apex of a major grid triangle. The major grid is broken down into 1.52 m (5 ft.) equilateral triangles and the mullions supporting the edge of the floors stand on the apexes of minor grid triangles and one small triangle beyond the major gridline. The corners are turned internally or externally around a column point. The system is much easier to draw than explain (Figs. 3c, 3d).

Block widths are two big triangles and two little triangles on the 13.2 m (43 ft. 4 in.) wide blocks, and three big triangles and two little triangles on the 18.5 m (60 ft. 8 in.) wide blocks. In the car park underneath the building, the aisle width is 5.28 m (17 ft. 4 in.) (the perpendicular height of a 6.1 m (20 ft.) triangle) and two cars park at 60° between the columns. This is, in fact, an economical arrangement. The floors in the office block are 230 mm (9 in.) concrete flat slabs. Generally the edges of the floor slabs are supported on 128 mm x 385 mm (5 in. x 1 ft. 3 in.) concrete mullions at 1.52 m (5 ft.) centres. Precast concrete cladding panels faced with cast-in brick tiles rest on the edge of the slab and are bolted to the mullions. Above slab level the panels, which are E-shaped, house the services, and below the slab light fittings and venetian blind boxes for the floor beneath. At the underpasses the edge load from, at the most, three floors comes down 1.32 m (4 ft. 4 in.) (the perpendicular height of a 1.52 m (5 ft.) triangle) beyond the columns underneath, and the floors become 513 mm (2 ft.) deep triangular coffers, which are deep enough to transfer the mullion loads back to the columns and also line through with the soffit of the cladding panels.

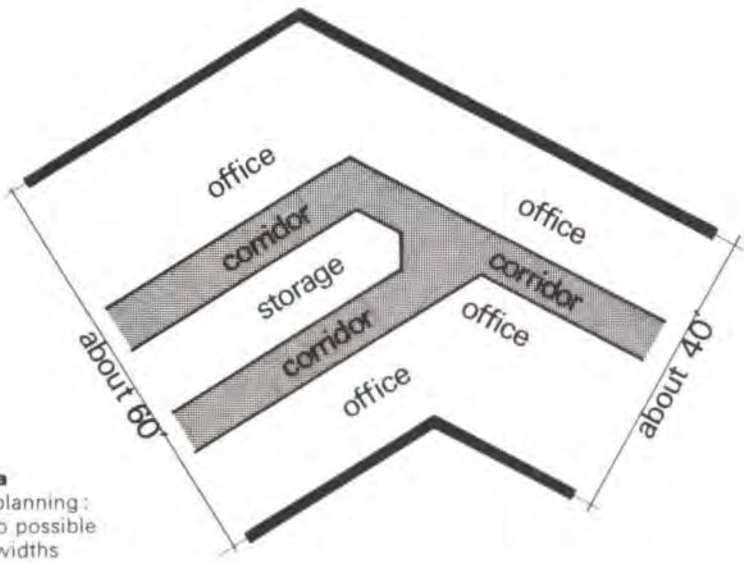


Fig. 3a
Basic planning: the two possible block widths

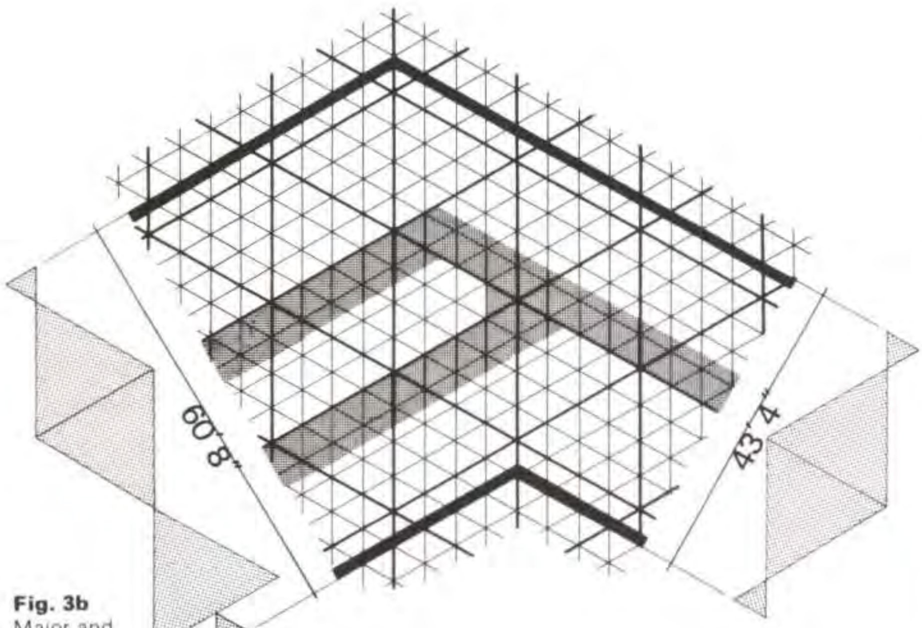


Fig. 3b
Major and minor grids

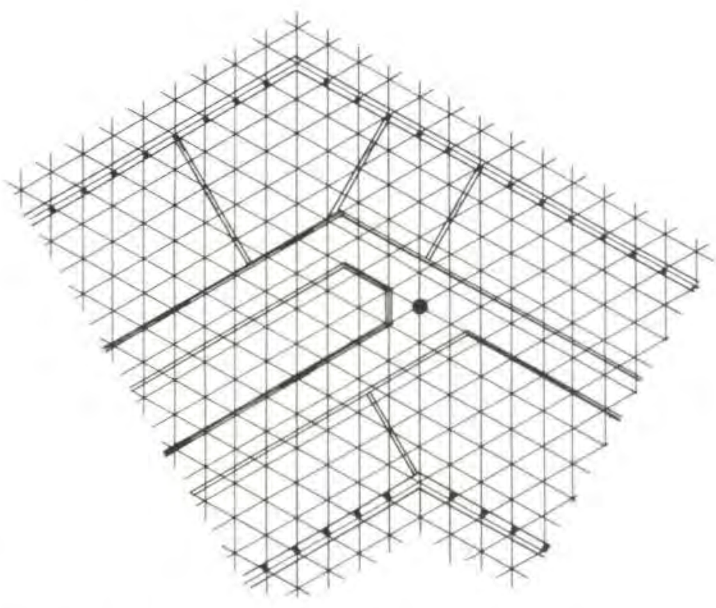


Fig. 3c
The plan developed. Mullions support the edge of the slab. Flat columns absorb into the corridor walls

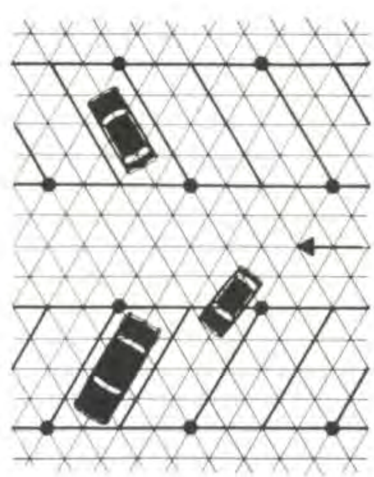


Fig. 3d
Car parking at 60°

Inside the building there are three types of column: round; slim-jim; or lance-corporal (Fig. 4.) The last two names were, or course, invented on site. Jack Bonnington prefers to be unaware of structure in the offices and corridors and hence the use of the slim-jim, which is absorbed as a panel into the corridor wall. It quite often happens that a slim-jim on one floor points along a different gridline to the slim-jim below, and in these cases the columns are reinforced against splitting forces like the end-blocks of prestressed beams.

Originally the change of direction of corridor walls always occurred at least 1.52 m (5 ft.) from the column grid point, but during the development of the detail design this principle was sacrificed and the lance-corporal was born. The lance-corporal can also occur above or below a slim-jim, pointing in any of the three directions, although this junction is not so easy to analyse as the centres of gravity of the two columns are not quite in line.

Services

It was assumed from the start that the main services distribution would run in a void above the false ceiling in the corridors. Heating is by finned tube convectors in the spandrel walls below the windows. As there are no false ceilings in the offices there is the usual problem of joining the main distribution circuit to the convectors, and this has been dealt with in the usual way by leaving chases in the screed and slab. As the slabs span two-ways the positions at which this is possible are limited and the arrangement is probably not ideal.

The lavatories are always planned to be vertically stacked within the central area of the wide blocks, so that the plumbing services drop in straight ducts to ground level.

The electrical main runs above the corridors false ceilings to distribution boards, and from these the conduits are cast in the slab.

Expansion joints

It was decided at an early stage in the design that there would be no expansion joints in the office block floors. They would have been difficult to accommodate without making breaks in the structural grid system, and these would have altered the whole concept. The problems of shrinkage and temperature movement could not, of course, be ignored in a building more than 168 m (550 ft.) long, and the slab reinforcement was carefully detailed so that there were no weak points. This appears, so far, to have been successful.

Cost

The cost plan cost of the office block superstructure, including proportions of preliminaries, etc., is 15/- per sq. ft.

Second thoughts

The structure as designed is in one respect unsuited to the building form. It is not really sensible to design for continuous support to the slabs at their edges when, at the lower floors, there are several entrances to the building which necessitate the omission of mullions. The mullions on either side of the opening are not strong enough to carry the concentration of the load. At these entrances the undercroft structure of round columns on the major grid was used, but for a number of reasons it was necessary to take these columns through the upper floors as well and this does perhaps spoil some of the rooms affected.

The difficulty was appreciated during the scheme design and two possible solutions were found. The purest was to support the floors only on columns on the major grid, and completely free the elevations of load-bearing mullions. This meant accepting the columns in the large rooms and planning the small rooms so that the walls passed through column grid points. It also meant a slight change of minor grid size so that the window mullions

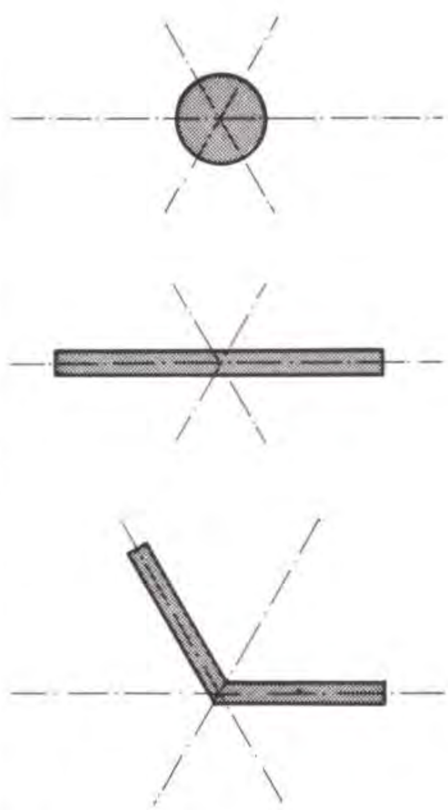


Fig. 4
The three column types: round, slim-jim, lance-corporal

would be two bays beyond the column grid and thus in line with a column grid point. However, it was felt that the design was too far advanced and time too short to try again.

Car park

As the triangular grid system for the office block had been developed partly to suit the car parking it seemed logical to extend the same grid over the area of the multi-storey car park. The two northern sides of the triangle of land between the railways are, in fact, exactly parallel to the grid lines, but otherwise the site is a very awkward shape, restricted by the coal railway line and the seawater pipeline, and an efficient parking layout at every level was not easy to find.

In order to make the most of the site the seawater main had to be moved about 4.6 m (15 ft.), and put inside the car park against the back wall at one level. Turner and Newall were naturally very concerned at the prospect of this operation, as the main runs day and night and they only have enough salt water stored at the works for one day's production. However, they did agree that it could be done subject to colossal damages if the reconnection to the new pipe took more than 24 hours. It didn't.

The construction over the railway line could have been designed in two ways; either the upper floor columns could occur at normal grid positions and stand on big beams over the track, or they could follow the curve of the railway at every level. The arrangement of car parking was difficult enough without having to cope with the curve of the railway on all the upper floors, and the first solution was, therefore, adopted. A 6.1 m (20 ft.) high retaining wall was necessary on the south side of the track and so the big beams are, in fact, portal frames. They are identical in all respects except the points at which the column loads from above are applied. The computer was a great help in solving this repetitive problem of analysis.

Fig. 5
Moving the seawater main. The old main is on the left, the new main being laid in parallel on new stools close to the back wall of the new car park (Photo: Sam Price)

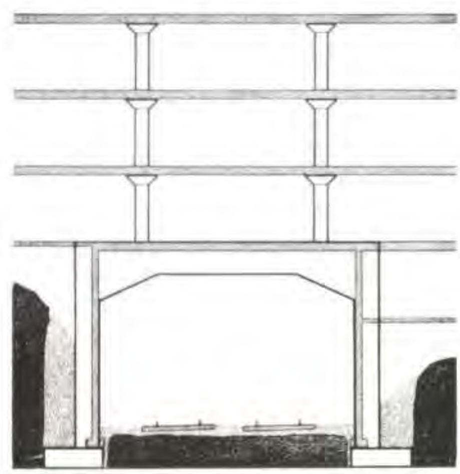


Fig. 6
Portal frames over the railway

The railway is in daily use and no propping was possible between the tracks. The contractor was given the option of either precast or in situ construction, and chose to precast the beams on site. The placing of the beams required a mobile crane capable of lifting about 28,400 kg (28 tons) at a radius of about 10.7 m (35 ft.)—the largest mobile crane in England. An in situ connection was formed at the back of the beams and the top of the columns after the beams had been levelled in.

The ramps were really no problem as there was virtually only one place they could go. The whole conception of the civic centre is horizontal and sloping ramps on the elevation were quite unacceptable, so they were put near the middle of the lowest level, and because of the geometry they are curved through 120° in plan. The position of the ramps necessitated a parking layout with carriageways at right angles to the north-south gridline and this meant leaving out the

Fig. 7
Precasting portal beams on site. The three columns in the foreground are ready to receive beams (Photo: Sam Price)

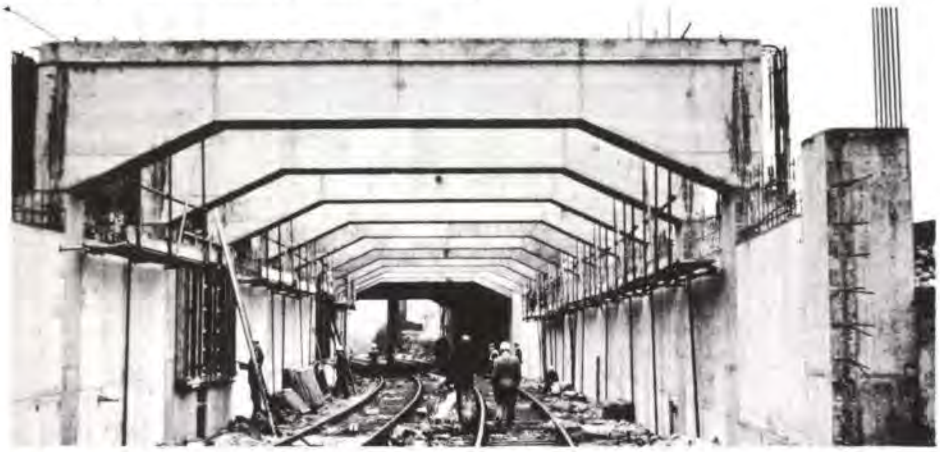


Fig. 8 above
Portal beams erected. In situ connections still to be concreted (Photo: Sam Price)

Fig. 9 below
Car park ramps (Photo: John A. Herring, Sunderland)

columns at certain places (Fig. 10), so that the slabs in these areas are supported only at the corners of regular hexagons, 12.2 m (40 ft.) across the points. These slabs are 282 mm (11 in.) thick. The yield-line analysis of these areas was simplified by the discovery in Wood's 'Plastic and Elastic design of slabs and plates', of Bill Smyth's solution to this very case.

Cost

The cost plan cost of the car-park superstructure, excluding the cost of bridging the railway, but including proportions of the preliminaries, was about 13/6 per sq. ft.

The three bridges

Burdon Road Bridge

The reconstruction of Burdon Road Bridge would have been necessary even if there had been no civic centre. The old bridge was in bad shape, and the vibration caused by passing traffic was quite alarming. Although the choice of structure for the new bridge was quite easy, the planning of the sequence of demolition and construction was really difficult, even though the Borough Engineer was prepared to close the road for a month. The difficulty was due to the variety of services running in and beside Burdon Road (Fig. 11).

To the west of the bridge there was a 564 mm (1 ft. 10 in.) diameter pipe carrying main water supply to the southern end of the town. Within the bridge deck of cast iron girders and brick jack arches ran the main gas supply in a pipe specially transformed from 487 mm (1 ft. 7 in.) diameter to a flat elliptical section so that it could fit and in various places under the pavements were an 11 kV cable, some small water pipes and the street lighting. Eventually all the services were either rerouted into a new suspended pipe duct beside the bridge or cut off temporarily or permanently. The old bridge deck was demolished and replaced with a 460 mm (1 ft. 6 in.) slab formed of C & CA precast, prestressed, inverted T-beams with an in situ concrete topping.

Park Lane Bridge

Park Lane Bridge was a completely different problem. The existing four-span arch bridge, built of local limestone, was as sound as a bell and contained practically no services. However, the carriageway was only about 6.1 m (20 ft.) wide and there was only one narrow pavement which would certainly be inadequate to cope with the traffic generated by the new civic centre. The aesthetic problems of building immediately beside an old bridge could have been avoided by removing the arches or even the whole structure. But there were no good structural reasons for doing this; cost was positively against it, and so was the Borough Engineer, who was understandably fond of the old bridge.

After much scheming and some model-making it was decided to widen the road by building a new bridge with a similar elevation beside the old one (Fig. 12). At a late stage this design was almost vetoed by the Ministry of Transport who pointed out that the arched deck was almost certainly more expensive than a deck of C & C A beams, and that their own publication 'Appearance of Bridges' expressly advised against any attempted imitation of an adjacent bridge. Fortunately architecture was a hobby of the Ministry's engineer and he was persuaded by the eminence of our architect.

The bridge deck is formed of arched precast reinforced concrete beams, 230 mm (9 in.) wide, at 924 mm (3 ft.) centres, with permanent concrete shutters and in situ concrete topping forming a 154 mm (6 in.) slab on top. The analysis of this structure produced one point of interest. BS 153, part 3a: Loads, states that 'where longitudinal members are spaced at less than half the

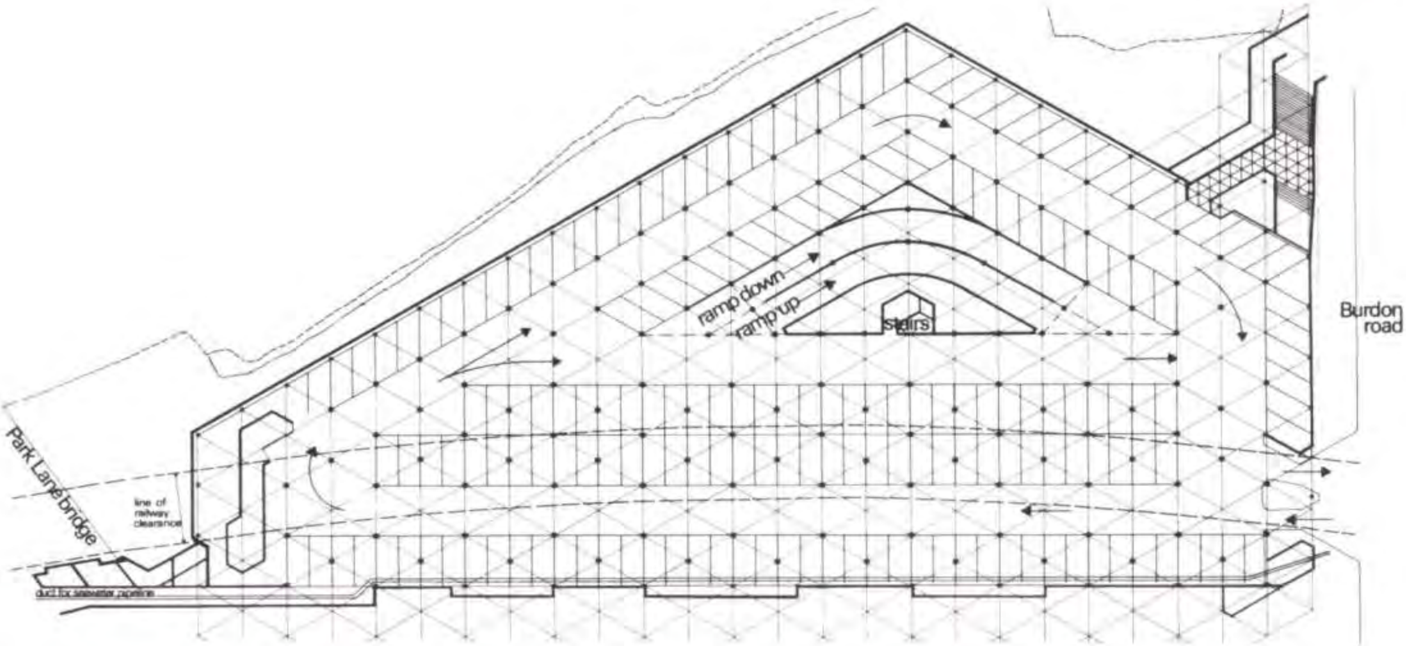


Fig. 10 above
The car park. The level over the railway

Fig. 11 above
Burdon Road Bridge services. From the left: new water main, new gas main, old water main (Photo: George Curry of John Laing & Sons Ltd.)

Fig. 12 above
Park Lane Bridge (Photo: John Laing & Sons Ltd.)

width of the lane the loading to be taken on these members shall be that appropriate to a half lane width.' The lane width was 3.66 m (12 ft.) and the beams, therefore, had to 'take' the loading from twice the width of road slab that they were actually supporting. However, the slab was clearly capable of some distribution of load and the deck was, therefore, analyzed by the computer for various combinations of loading of alternate beams. The results showed that, for the spans and stiffnesses of this bridge, the slab distributed the load very well, and the loaded beams carried only a little more than their neighbours.

Both abutments are of in situ concrete clad in brickwork. The north abutment being only about 4.6 m (15 ft.) deep and resting partly on clay is backfilled inside with PFA, but the south abutment, 10.5 m (35 ft.) high is an empty box of concrete walls cut a few feet into the rock, and closed on top with a suspended road slab of precast prestressed planks with in situ topping.

Fig. 13 left
Park Lane Bridge: cladding panels in lightly knapped, ribbed concrete; aluminium flashing; edge beam in fine-ribbed concrete (Photo: Sam Price)

The parapet of Park Lane Bridge, like the parapet of Burdon Road Bridge, is made of panels of precast concrete, with the external face in knapped, ribbed, 'elephant house' concrete, and infilled on the pavement side with facing bricks. The Park Lane Bridge edgebeams have a fine-ribbed surface produced by casting against rubber floor matting.

Burdon Road Footbridge

With the footbridge there were, of course, no services or other bridges to complicate matters but it was, nevertheless, quite as difficult to design as the others. A multitude of different schemes adorned the office walls including one splendid version, by Mark Kitchen, of the Coalbrookdale cast iron bridge in concrete. Eventually the die fell on the side of safety and it was decided to make our footbridge as shallow as possible. This was, at least, a definable end and ensured that the concrete would be post-tensioned which satisfied the structural appetite. The span is 26 m (85 ft.), and the section is an 839 mm (2 ft. 9 in.) deep aerofoil, hollow inside, with a central spine containing the cables. The footway is tiled and the soffit is in the fine-ribbed concrete.

Construction

The successful tenderer was John Laing Construction Co. Ltd., who elected to build the whole civic centre in 27 months for a sum of £3,382,120 on a fluctuations contract. They started on site in January 1968 and have so far achieved a very satisfactory standard of workmanship.

There have been few serious problems during construction. Under the south courtyard an area of very poor rock was encountered which necessitated a slight alteration in foundation sizes, and, under the civic suite, the rock was much deeper than anticipated because of quarrying, which the bore holes had missed. At Laing's suggestion the pad footings were changed to piles to save a lot of expensive planking-and-strutting.

Laing's took a lot of trouble with the setting-out as they realized that the tolerances in our specification would have to be achieved if the windows and precast cladding panels were to fit. The precast mullions were set up in jigs, about six at a time, and checked by an engineer before grouting, with a resulting high degree of accuracy.

Reinforcement

When the grid was evolved it was assumed that, although the columns were laid out in triangles, the slab reinforcement would be orthogonal. However, when we came to work out the steel, we found that the corners, which occupy a large part of the floor area, were extremely difficult and wasteful to detail, so a triangular steel arrangement was checked and found to be actually more economical than the orthogonal system. It was very satisfying that, despite our initial obtuseness, the triangular design triumphed in the end. The bars are placed on all three directions and at constant centres, so that a uniform mesh of steel covers the slab, and the bar diameters are altered to suit the bending moments. Laing's steel fixers had no complaints with the system. The slabs are so far remarkably free from cracking, even at the construction joints, and we suspect that the three-way arrangement is actually superior to orthogonal steel.

Structural steelwork

The council chamber, which stands at the southern end of the building, is a large irregular hexagon in plan and, so that it should be easily identifiable as a separate entity, has a pointed roof like a witch's cap. The roof has a span of about 24.4 m (80 ft.) each way and contains large ventilation ducts. The structure is a sort of three-dimensional tied, warped, arched vierendeel truss system in steel. For a better description

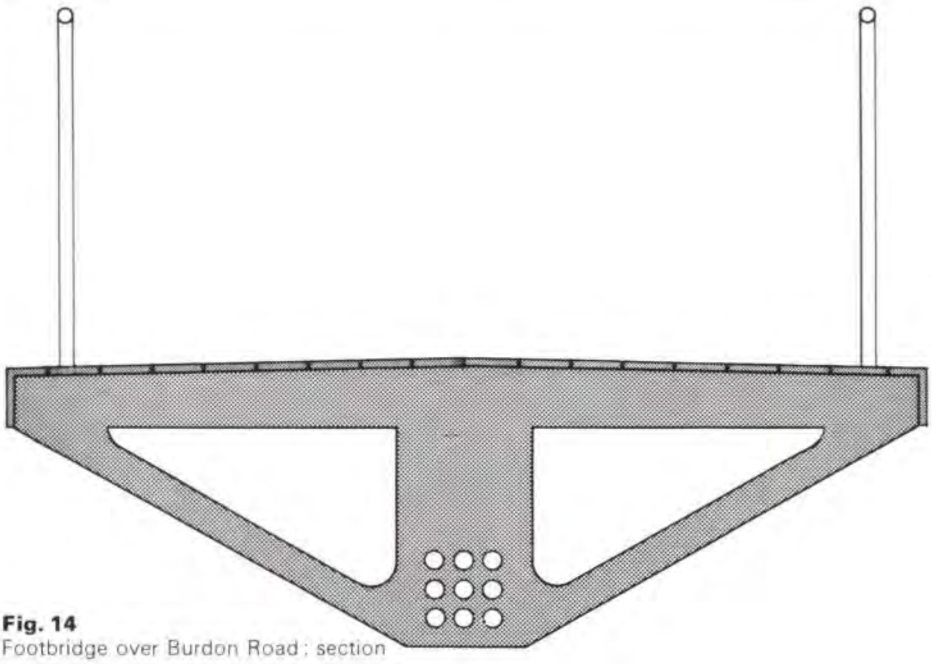
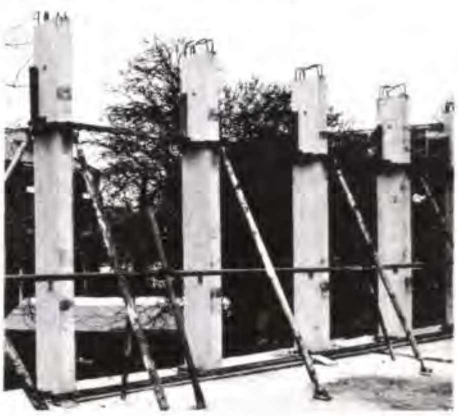


Fig. 14
Footbridge over Burdon Road: section

Fig. 15
Footbridge over Burdon Road (Photo: Sam Price)

Fig. 16 left
Shutter box for a lance-corporal column. All slim-jims and lance-corporals are in fairfaced concrete, and shutter bolts through the columns are not acceptable, hence the strong-backs (Photo: Sam Price)

Fig. 17 below
Precast mullions in an erection jig (Photo: John A. Herring, Sunderland)



the computer should be consulted as it is the only one who really understands it. Unfortunately it will soon be entirely enclosed, with copper sheeting on top and a ribbed timber ceiling underneath.

It will come as no surprise to a follower of Jack Bonnington's work that the staircases at Sunderland are all in steelwork. There is one, however, which is in the usual idiom but has a different shape. This is a helical staircase with curved flat steel stringers and folded steel treads, rather similar to the staircases in Russell & Bromley's London shoe shops, except that the one at Sunderland is only supported at the top and bottom. The amount of bounce is negligible.

Fig. 19 above
Council chamber roof steelwork (Photo: Sam Price)

Fig. 18 below
Three-way steel. The mats over columns, being separate, are reinforced orthogonally. Projecting steel from precast mullions can be seen in front of the edge shutter. In the distance, moulds for triangular coffered slab (Photo: George Curry of John Laing & Sons Ltd.)

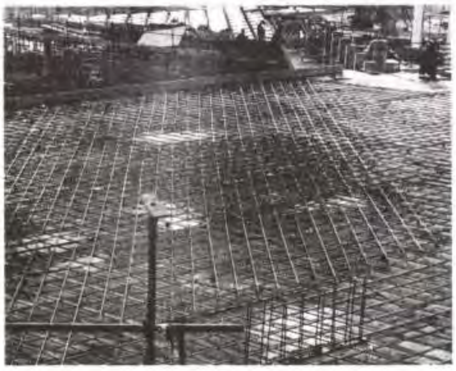


Fig. 20 below
Helical steel staircase (Photo: Sam Price)

Conclusion

With the landscaping hardly begun there is not yet enough to attract the photographers. We shall have to wait at least six months for their usual glamorous deceptions. Meanwhile the aerial photograph shows the extent of the work in September 1969.

Architects	Sir Basil Spence, Bonnington & Collins
Quantity surveyors	Reynolds & Young
Main contractor	John Laing Construction Co. Ltd.
Value	£3,382,120

Fig. 21 below
Aerial photograph of Civic Centre (Photo: *Sunderland Echo*)

Carbon fibres: potential applications in structures

Turlogh O'Brien

This paper is based upon reports that were prepared for discussion with the National Research Development Corporation (NRDC) during the latter's recent survey of potential applications in various industries. It is divided into two parts; the first describing the nature and properties of the material, its price compared to other materials, and the ways in which it might be used in buildings; the second, evaluating the economics of its use in long span structures.

PART A: THE MATERIAL AND ITS USES

The present situation

Carbon fibres were first patented by the Japanese, but the breakthrough in methods of production was achieved at the Royal Aircraft Establishment, Farnborough. Development work has been carried out at the Atomic Energy Research Establishment, Harwell, and the NRDC controls development and licences for the government. At present the 'carbon fibre club' in the private sector consists of Rolls-Royce, Courtaulds and Morgan Crucible. Rolls only manufacture enough for their own uses; Courtaulds produce the raw material, a special variety of their synthetic fibre *Courtelle;* Morgan Crucible manufacture carbon fibres through their subsidiary Morganite Research and Development, and, unlike Rolls, sell it to anyone prepared to pay the current price of around £110 per kg. (£50 per lb.)

There has been quite a lot of publicity recently concerning ICI's investigation into whether it should also enter the field. A select committee on carbon fibres urged in February 1969 that a plant be built to produce 450 metric tons of fibre per year, with an investment of £5m. ICI have decided that this proposal is not viable, as the cost of the product will still be higher (£11 per kg. : £5 per lb.) than is estimated to be required for large scale use. ICI have, therefore, withdrawn from the field, at least temporarily.

The only other company involved in a large way is Fothergill & Harvey. It is a small company, well established in glassfibre reinforced plastics, which has invested considerable effort in solving the problems of making carbon fibre reinforced plastics. Its know-how is reckoned to be as advanced as Rolls-Royce, and this will put them in a very strong position as the use of the material develops.

The present situation is one of small demand (mostly from the aerospace industry) and very high prices. Heavy investment in large scale production facilities will bring the price down, but not quite far enough. There are worries that, unless this country takes the gamble and invests the required capital in the hope that uses will be found, the Americans will take the plunge and Britain will end up having to import the material from across the Atlantic at considerable expense in foreign exchange.

Whether or not these fears are justified is not within the scope of this paper to consider. It has been suggested that the construction industry should be a potential large-scale user of carbon-fibre reinforced plastics (CFRP), and should lead the way by using it now, thus paving the way for the low price/large volume production that everyone wants. This paper seeks to show whether this is realistic, or whether it is merely the idle dream of people bemused by the large quantities of materials that disappear into structures.

Carbon fibres and CFRP

Carbon fibres are produced from polyacrylonitrile (*Courtelle*) by a secret process which involves heating to very high temperatures (exceeding 2000°C; 3630°F) in a special atmosphere. Fibres up to 300 m (980 ft.) in length may be produced. To make CFRP these must then be increased in a resin matrix. Matrices of thermoplastics (e.g. nylon), metals or ceramics may be used but, for the purposes of structures, either polyester or epoxy resins are likely to be the principal ones.

The properties that create all the excitement are the density, the high Young's Modulus, and the high strengths. The differences between carbon fibres and other materials are best illustrated by the specific tensile strength (strength divided by specific gravity) and specific modulus (modulus divided by specific gravity). The only materials to show higher values than those for carbon fibres are graphite, alumina and silicon whiskers, which are not likely to be useable as structural materials for many many years. Table 1 summarizes the properties.

Table 1: Mechanical properties *

Material	Specific Youngs modulus kN/mm²	Specific U.T.S. kN/mm²	Youngs modulus kN/mm²	Ultimate Tensile Strength kN/mm²	Specific Gravity
CFRP (type I) †	110	0.34	179	0.55	1.6
CFRP (type II) †	67	0.59	96	0.86	1.45
High tensile steel	26.2	0.13	207	1.00	7.8
Aluminium alloy	25.9	0.17	72	0.46	2.8
Titanium	24.1	0.21	110	0.93	4.5
Glassfibre reinforced plastics †	20.7	0.52	41	1.03	2.0
Carbon fibre—type I	186–228	0.69–1.03	380–450	1.38–2.07	2.0
Carbon fibre—type II	145–180	1.38–1.73	240–310	2.41–3.10	1.7
Boron fibre	165	1.10	414	2.76	2.5
Steel wire	26.2	0.35–0.53	207	2.76–4.14	7.8
S—glassfibre	33.8	1.83	86	4.48	2.5
E—glassfibre	24.1	1.38	62	3.45	2.5

† Unidirectional fibres, fibre content 60% by weight.

* This table is reprinted from
FOTHERGILL & HARVEY LTD. *Publication no. 23.* Resin impregnated high modulus carbon fibre. 2nd edition. 1969.

Carbon fibres are made in two basic grades; a high tensile and a high modulus grade. The differences between them are shown in Table 1. The specific strength of CFRP is no better than that of a good GRP (glassfibre reinforced plastic), but its specific modulus (for both type I and type II carbon fibres) is higher than for any other structural material.

As with GRP, the properties of CFRP vary with fibre content. GRP laminates used in building have glass contents in the range 30-40% weight. Current work with CFRP in the aircraft industry also uses about 60% of carbon fibre by weight. If a laminate of CFRP is made with this amount of type 1 fibre (high modulus) a product is obtained with tensile strength and modulus properties similar to high-yield steels, but with only $^1/_5$ of the density.

Of course, being fibrous, carbon fibres have directional properties. In a laminate the strength obtained in any direction will depend upon fibre orientation. Most work to date was either long unidirectional fibres, or short (say 1 mm) fibres randomly dispersed. The latter method is used for gear wheels and the like, where the very good wearing qualities are advantageous. For structural purposes cross-ply laminates would normally be required. However, this produces a difficulty. The coefficient of thermal expansion is not the same along the length of the fibre as it is across its section. In fact, the co-efficient is negative (—5.14 x 10/°C) longitudinally, and positive (50 x 10/°C) laterally. This can give rise to fabrication difficulties, particularly when hot setting resins are being used.

It should also be noted that CFRP structural units will be worse in fibres than steel. The resins used normally soften at about 200°C (390°F) and the fibres oxidize in air at 600° C (1110° F). Fatigue properties, however, are said to be outstandingly good, although data is not available on the tests that have been done. The bond between resin and fibre is also good, and is thought to be durable, although there is a lack of long-term performance data.

The stress-strain diagram for CFRP is essentially linear over its whole length. Laminates exhibit brittle behaviour.

Fabrication of CFRP components

Whereas with GRP the main fabrication technique used for building work is hand lay-up, very little of this has been done with CFRP. At present it is a sophisticated material being employed in industries used to mechanized production processes. The view has been expressed that carbon fibres will always be too sophisticated for crude production methods, as these will not be capable of realizing the full properties, and hence of justifying the costs. If this turns out to be the case, it has implications for building components, as very rigorous standardization would be needed to keep fabrication costs to reasonable levels.

Two main processes are used for making CFRP components.

When long fibres are used, matched metal moulding is employed. In this the fibres are usually pre-impregnated with resins (Fothergill & Harvey sell these 'pre-pregs') that only cure at high temperatures (approx. 150° C 300°F). The fibres are placed in the mould and, after closing, heat is applied.

With short fibres, injection or compression moulding may be used. Good reinforcement is obtained with short fibres if the aspect ratio (length:diameter) exceeds 100. However, the fibres are easily broken during moulding due to their brittleness.

As work has not yet been carried out on fabrication of CFRP building components, it is not possible to identify all the problems. Sufficient is known from other industries to suggest that it will not be all that easy. For one thing, the stiffness of the fibres does not allow them to take up the mould shape easily. Current GRP methods may not be extensively used for CFRP. Of course, it all depends on what sort of components are required.

Price comparisons

Any discussion of uses for carbon fibres or CFRP cannot long avoid the question of price. It has already been stated that the current price is about £100 per kg. (£45 per lb.) and that this would drop to about £11 per kg. (£5 per lb.) if a plant was built to produce about 450 metric tons per year. These prices are compared with those for some other materials in Table 2.

It may be seen from the tables that carbon fibres are in the 'precious' metals price bracket, if the comparison is made by weight. However, it has been suggested that this is an unfair way of making comparisons, and that a volume basis should be used. This has, therefore, been included as well, and it is clear that carbon fibres are not in the 'jewellery' class when looked at in this way.

Obviously the comparisons of price of the basic materials are not all that useful. It is better to compare the unit price of CFRP. This is, of course, difficult to estimate at this stage, due to the unknown fabrication costs. If these costs are excluded, a basic materials cost of £7 per kg. (£3 per lb.) or £10,000 per m³ (£283 per ft.³) is obtained, using the low price of carbon fibres.

From this it is apparent that the price ratio for CFRP/mild steel is of the order of 126 (by weight) and 23 (by volume). The equivalent ratios for CFRP/GRP are 11 and 8. Other ratios may be calculated. In all considerations of potential uses it is necessary to bear these prices in mind, as CFRP is only likely to achieve large scale use if these extra costs can in some way be shown to be justified, or if the overall design of the components can bring the total cost down to the level of the components being replaced.

Some general considerations

For convenience, the problem of the potential applications for CFRP in load-bearing components may be divided into two parts:

1 substitution for other materials in existing structural solutions

2 new structural solutions

The purist might argue that a new structural material requires new structural solutions to achieve maximum economy and effectiveness. This approach rarely leads to the early development of applications for a new product. Faster results are obtained by investigating the possibilities of substitution. Quicker feedback is obtained, and structural solutions may then be progressively modified to improve the fit between properties available and properties used.

Despite this, it is probably worth taking a look at the work being done on new structural solutions for plastics materials, as CFRP materials differ from GRP only in certain important respects. The technology of use may not be too dissimilar (but see the section on fabrication above). It may be noted anyway that new structural solutions for plastics have, to some extent, evolved out of the substitution/development process mentioned above, and CFRP should be able to benefit from this. To follow on from there is merely to treat CFRP as just another reinforced plastics material, rather than a completely new product.

Table 2: Price comparisons

Material	£/kg	£/m³ (000's)
Mild steel	0.055	0.43
Lead	0.11	1.25
Aluminium	0.25	0.78
Polypropylene fibre	0.35	—
Glassfibre	0.38	0.95
Stainless steel [1]	0.50	3.90
Copper	0.60	5.35
Tin	1.41	8.45
Titanium	2.20	9.90
Carbon fibre [2]	11.00	18.70 [4]
Silver	23.40	246.00
Carbon fibre [3]	110.00	187.00 [4]
Gold	543.00	10,500.00
Platinum	1590.00	34,000.00

Notes. [1] Price before the recent shortage
[2] Price if a 450 metric ton plant is installed
[3] Price in 1969-70
[4] For type II carbon fibre

New structural solutions

With CFRP a material is available with the strength and stiffness of a high yield steel. The material behaves elastically almost to failure and so the elongation at break is small. There is no yielding and negligible plastic deformation. The forming and fabrication processes will be very different from those of steel and so the structural shapes evolved may be quite different, particularly as it is anisotropic.

With GRP structural members the problem has been to achieve adequate stiffness. This has been solved by providing rigidity through shape. Complex forms have often resulted which could be made by the hand lay-up technique used for large units. Standardization and repetition of units have been desirable things to ask for, but they are not absolutely vital in GRP work. With CFRP, hand lay-up has not so far been used, but neither has it been proved to be useless. Difficulties would be expected as a result of the fibre stiffness, but these would not be insuperable. However, if the analogy with GRP is valid, the properties that would be achieved with hand lay-up would be far short of those listed in Table 1. This technique is just not geared to producing a high performance engineering material. With new structural solutions, work should concentrate on utilizing forms that can be produced as standards in matched metal moulds, using hot setting resin systems.

The directional properties in the finished laminate offer some hope for attempting to match the fibre orientation closely to the lines of stress imposed on the member. Economy could be sought by matching the properties very closely to the imposed stresses and limiting the redundancies so often built into structures. Attempts to do this would, of course, run counter to the standardization required for economical fabrication.

Without being clairvoyant and without having carried out the lengthy programme of research and prototype testing that would be necessary, it is difficult to predict the outcome of the search for the optimum structural form for CFRP members. It is likely that the components produced will have forms defined by the shaping of sheets, rather than by the rolling or extending of sections.

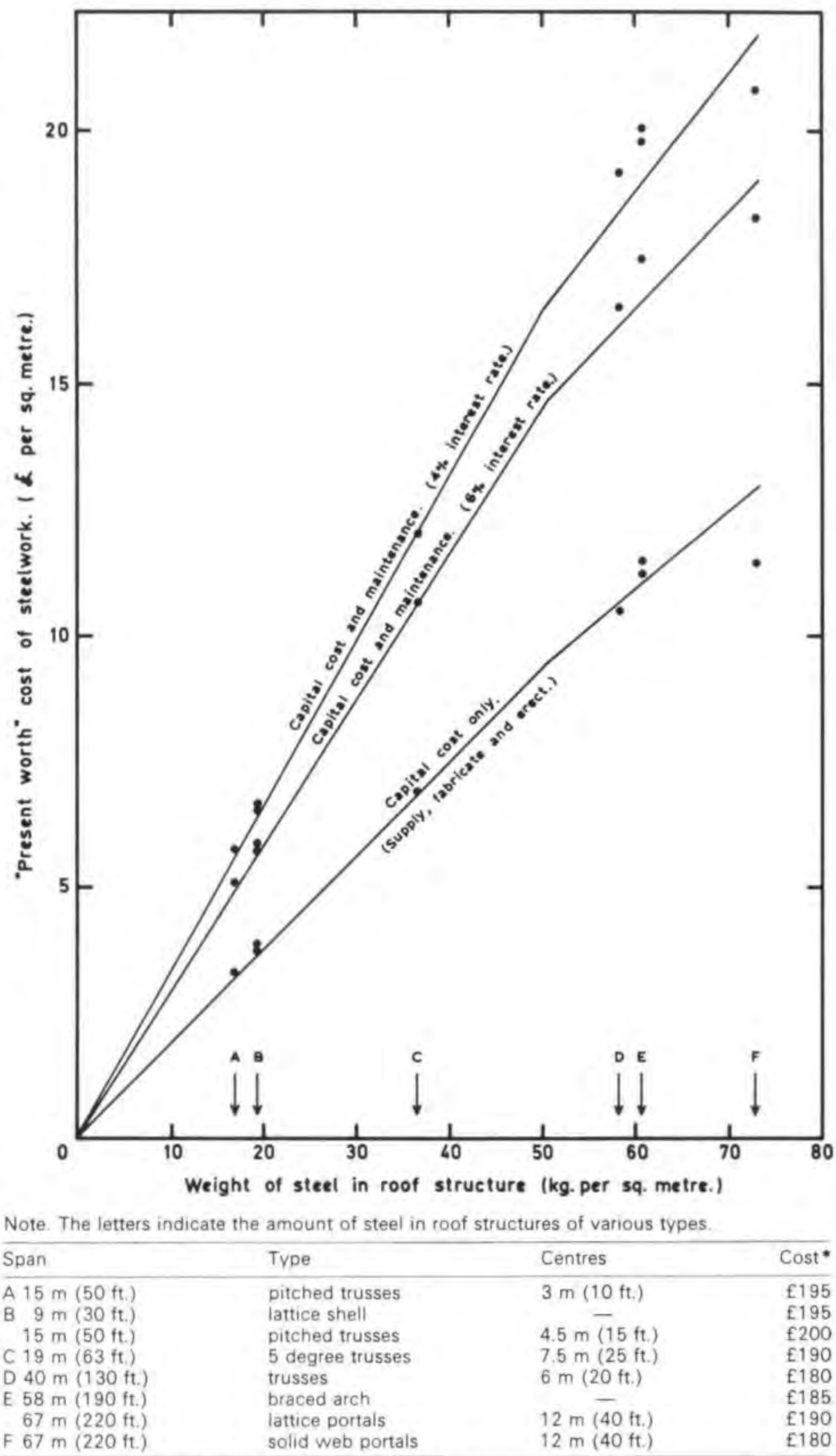
Substitution for other materials

When CFRP is compared to steel, its prime advantages are low weight and lack of susceptibility to corrosion. If it is to be used as a substitute for steel, either one or both of these advantages must be utilized. In building, it is rare for low weight to be considered an advantage for which extra should be paid. This is in contrast to transport industries, where great savings in operational costs can be realized by low structural weight. The cases of long span roofs and bridges are considered in part B.

The orientation dependence of CFRP properties suggests that it might find uses for reinforcing bars or prestressing cables in concrete. However, reinforcement for concrete is not required to be of light weight. Corrosion resistance is an advantage, but this can be obtained by using galvanized steel or even stainless steel. The price ratio for galvanized steel/mild steel (about 1.5) is increasingly being considered to be acceptable for corrosion protection in certain projects. The price ratio for stainless steel/mild steel (about 9) is far too high for any large scale use of the material for concrete reinforcement. With a price ratio of about 23 (by volume excluding CFRP fabrication costs) for CFRP/mild steel, the scope for its use as reinforcement is very limited, even before the problem of achieving adequate bond strength to concrete is considered.

When reviewing the possible use of CFRP for cables, the problems of price ratios are compounded by technical difficulties. In this application, corrosion resistance is an advantage, while high stiffness may or may not be required. With GRP cables, difficulties have been experienced with anchorage devices. It seems likely that the same problems will arise with CFRP. Although the strength properties are good, they do not compete with the best steel wire strengths.

Another possible application that could be investigated is in the field of loadbearing fixing devices. With these, high strength and good corrosion resistance are an advantage. High stiffness may sometimes be useful, but low weight is very rarely important. Currently, stainless steel and phosphor bronze are used. The price ratio of 2.6 (by volume) is the lowest yet considered. (It is only 2.0 at the temporary inflated stainless steel price.) However, this does not include fabrication costs which could be a significant proportion of the price of a fixing device. Once again, technical difficulties could limit the scope for substitution in this field. Fibrous materials are not necessarily the best type for making connections, as the stresses in them can be very complex and it is difficult to ensure that the fibre distribution is optimized. Similarly, it is difficult to make connections between CFRP



Note. The letters indicate the amount of steel in roof structures of various types.

Span	Type	Centres	Cost*
A 15 m (50 ft.)	pitched trusses	3 m (10 ft.)	£195
B 9 m (30 ft.)	lattice shell	—	£195
15 m (50 ft.)	pitched trusses	4.5 m (15 ft.)	£200
C 19 m (63 ft.)	5 degree trusses	7.5 m (25 ft.)	£190
D 40 m (130 ft.)	trusses	6 m (20 ft.)	£180
E 58 m (190 ft.)	braced arch	—	£185
67 m (220 ft.)	lattice portals	12 m (40 ft.)	£190
F 67 m (220 ft.)	solid web portals	12 m (40 ft.)	£180

*Assumed costs of steelwork per metric ton supplied and erected.

Fig. 1 Cost of structural steel in roofs and either itself or other structural members so that the connections are as strong as the basic member.

Clearly the possibilities for the substitution of CFRP for loadbearing members in construction are limited. In the aircraft industry, an industry which must use lightweight structural materials, CFRP is being examined with interest. However, the extent of the development work is fairly limited. For the BAC 3-11 programme, it is being evaluated for reinforcing aluminium airframe components. The addition of CFRP to building structural members, however, seems to offer less scope. There is little point in stiffening steel with it, unless a complete skin of CFRP was built around the steel to give corrosion protection. Surface reinforcement of concrete members may be a possibility, when fire resistance is not required. Semi-structural members (curtain wall mullions and transoms) could be made very slender but stiff by incorporation of CFRP into aluminium sections. Cladding panels of GRP or other composites (fibre reinforced cements or gypsum plaster) could be stiffened by adding carbon fibres instead of ribs. All potential applications of this type would have to be evaluated in prototype tests.

PART B: THE ECONOMICS OF CFRP IN LONG SPAN STRUCTURES

(*This part has been compiled by John Blanchard and Raymond Payne and takes the form of a commentary on the graphs*)

Fig. 1 shows the cost of structural steelwork in various roofs, plotted against the weight of steelwork per m.[2] The higher weights represent longer spans, up to 67 m (220 ft.). Maintenance has been allowed for, assuming a painting cycle of five years and a building life of 40 years. The salvage value has been assumed at £10 per metric ton.

The costs shown are the present worth costs, i.e. the sum of money which would, if invested now, produce the necessary money at the required time. For materials which require no maintenance the present worth equals the capital cost.

Fig. 2 shows the cost to which carbon fibre reinforced plastics should be reduced in order to make it competitive with structural steel. This is plotted against the weight of steelwork per m^2 in the competing roof. Roofs carrying two types of loading have been considered:

1 'Light load' i.e. 0.48 kN/m^2 (10 $lbf./ft.^2$) show load, plus 0.05 kN/m^2 (1 $lbf./ft.^2$) cladding

2 the so called 'medium load' (though it is in fact rather light) i.e. 0.72 kN/m^2 (15 $lbf./ft.^2$) show load, plus 0.14 kN/m^2 (3 $lbf./ft.^2$) cladding.

CFRP was assumed to have a density $^1/_5$ of that of steel and allowable stresses under working load of 130% of that of ordinary mild steel (i.e. 1.3 x 160 = 208 N/mm^2 or 13.6 $tons/m^2$). Its modulus of elasticity was presumed to be 154,000 N/mm^2 (10,000 $tons/m^2$), or 77% of that of steel, but this was not used in the calculation.

The calculation was made on the simple basis that the volume of structure required is proportional to the total load carried and inversely proportional to the allowable stresses. For example, when maintenance is not considered and the weight of structure is negligible compared with the superimposed loading, the competitive cost of CFRP approaches

$$\frac{1.3}{0.2} \times 195 = £1,270 \text{ per ton}$$

A number of assumptions is involved in this as detailed below.

1 Those tending to underestimate the competitive price of CFRP:

i) The savings in foundation costs due to reduction in weight of the roof are negligible.

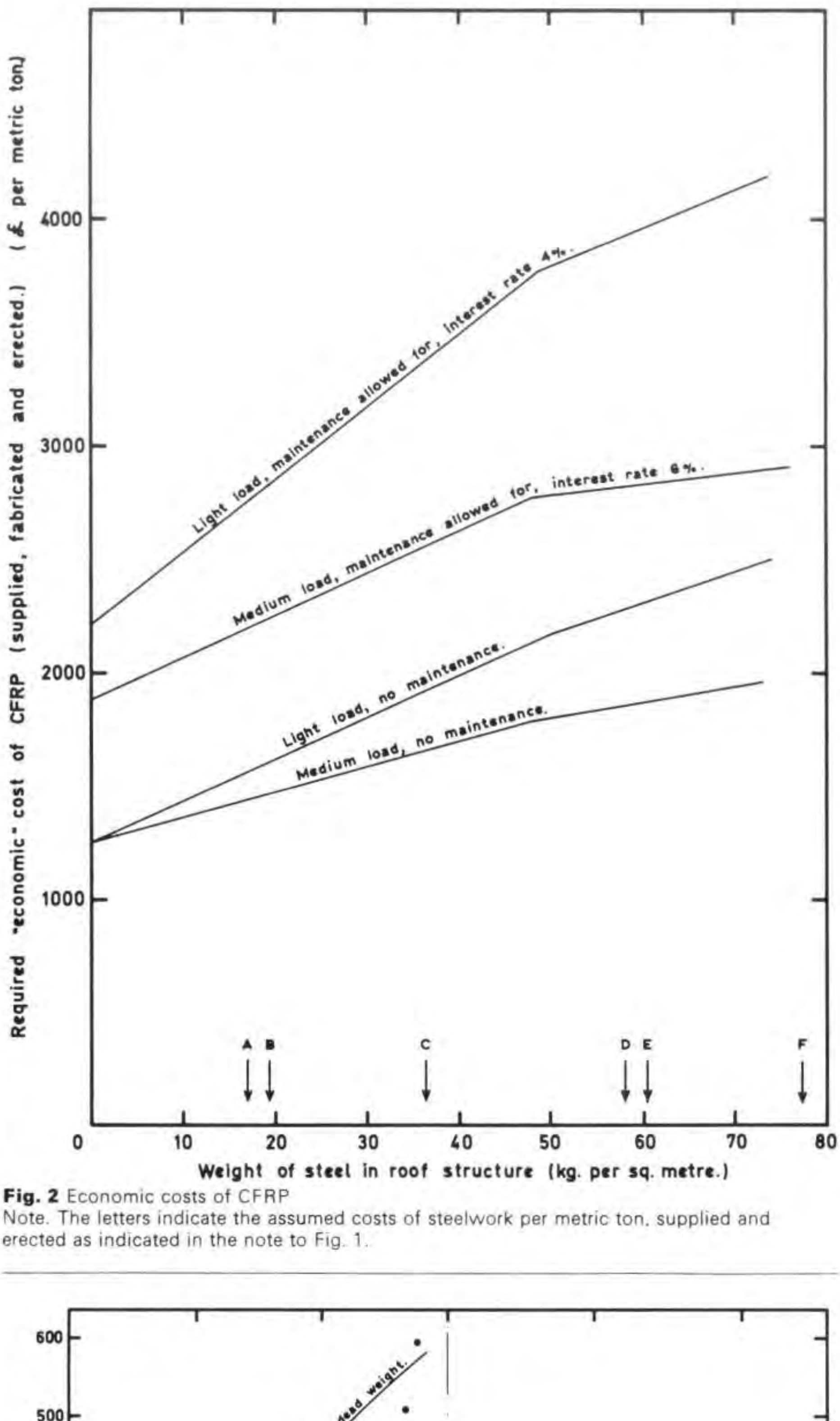


Fig. 2 Economic costs of CFRP
Note. The letters indicate the assumed costs of steelwork per metric ton, supplied and erected as indicated in the note to Fig. 1.

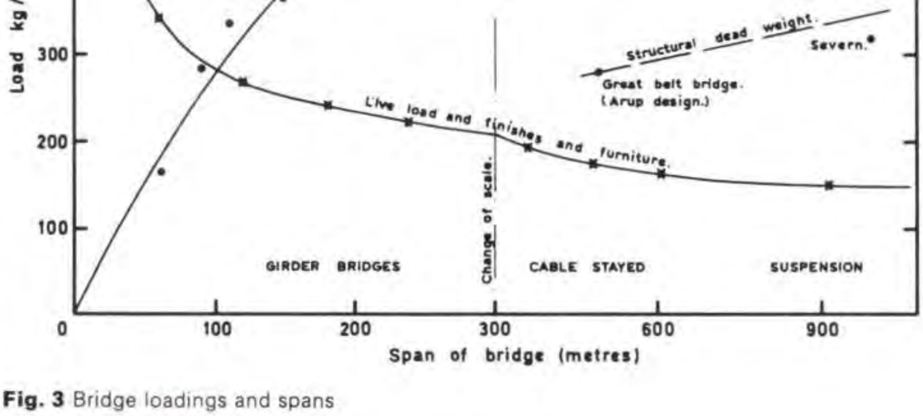


Fig. 3 Bridge loadings and spans

ii) The cost-saving due to the more efficient cross-sections that might be obtained with the adaptable manufacturing techniques of CFRP is negligible.
iii) Those cost savings are negligible which would arise from the possibility of varying the cross-section along the length of a member (probably by varying the thickness of material).

The possible saving of volume in a member is zero for tension members, a theoretical maximum of 15% for compression members, and a theoretical maximum of 33% for bending members though in practice this saving is very difficult to achieve.

2 Those tending to overestimate the competitive price of CFRP:
i) The reduction in the modulus of elasticity would not increase the sizes of CFRP members in order to limit the deflections or the possibility of buckling.
ii) The dimensions of the CFRP members (e.g. wall thicknesses) would not be so small as to be impracticable.
iii) The extra cost of connections in CFRP members (e.g. reinforcement round holes) is negligible.

The net effect of all these assumptions is probably, in general, that the competitive price of CFRP has been overestimated.

It will be observed that in the most favourable circumstances the competitive cost of CFRP is £4,250 per metric ton supplied and fixed. If we allow the same cost of erection per unit volume as that of steel i.e. 5 x 130 = £650 per metric ton, then the competitive price for the supply of CFRP would be not greater than £3.6 per kg. (£1.6 per lb.)

Fig. 3 shows the structural dead load and the live load carried for bridges of various spans. Fig. 4. shows the ratio of structural dead weight to total load. This ratio approaches 75% for girder bridges (with steel decks) at a span of 300 m (1,000 ft.). For higher spans, cable-stayed and suspension bridges are more economic and they tend to have lower proportions of dead load. Suspension bridges are unsuitable for substitution of CFRP because about 70% of their structural weight comes from the stiffening girders. Now these girders are designed to carry only live loading and wind so that any saving of weight due to substitution of CFRP would only affect the cables (i.e. 30% of the bridge structure) though there would be some saving on the towers. However, the cost of towers is only about 16% of the total cost and the saving would in any case be significantly less than the proportionate saving in weight. The bridges, therefore, that are most likely to profit from the substitution of CFRP are the girder bridges of between 180 m and 300 m (590 ft. and 1,000 ft.) span for which the ratio of structural weight to total load is about 70%.

Fig. 5, is a general curve showing the relative weights of CFRP and steel required for various proportions of dead load. This is a general curve for any structure where the working stresses of the two materials are equal and is appropriate for long span bridges which use a high-strength steel at a working stress of 205 N/mm² (13½ ton/in.²). Using this curve the weight of CFRP would be 0.07 of the weight of steel for a dead load ratio of 0.7. The capital cost of bridge steelwork is about £170 per metric ton erected. The capitalized maintenance cost is about £28 per metric ton for an exposed site and £22 per metric ton for an inland site assuming a 60-year life and a 4% interest rate. For CFRP to be economic in long span bridges its cost including erection would, therefore, need to be less than £198/0.07 = £2,800 per metric ton or £2.8 per kg. (£1.3 per lb.) It should be remarked that a steel girder bridge is not necessarily the most economic solution for any particular site. On the other hand the use of CFRP would increase the critical span above which other solutions become more economic.

Thus long span bridges appear less suitable than roofs as an application of CFRP although the dead load ratio is more favourable. This is because the initial costs and, more significantly, the maintenance costs of the small members used in steel roofs are much higher.

The assumptions implicit in this cost analysis are similar to those made for roofs except that 1 (iii) and 2 (ii) are likely to be unimportant. For 1 (iii) this is because in large bridge members it is practicable to vary the steel thickness.

For bridges, the behaviour of CFRP near ultimate load and how this affects the required factors of safety and fatigue strength would be more important. Consideration would also have to be given to the suitability for carrying traffic of thin sheets of CFRP topped with asphalt.

A special type of structure which approaches a zero value for the dead load ratio is the movable dish ratio, telescope or satellite tracker when it is enclosed in a so-called radome to protect it from the wind as is sometimes done in the US. Virtually, the only weight then carried is the structural weight. But lightness carries two other bonuses:
1 the weight has to be carried by expensive movable bearings and
2 when the dish is rotated in azimuth the deflections change causing unevenness resulting either in loss of signal or requiring expensive compensating electronics. The saving of maintenance could well be significant also. A design enclosed in a radome is essential for this application because otherwise the wind effects swamp the dead load effects.

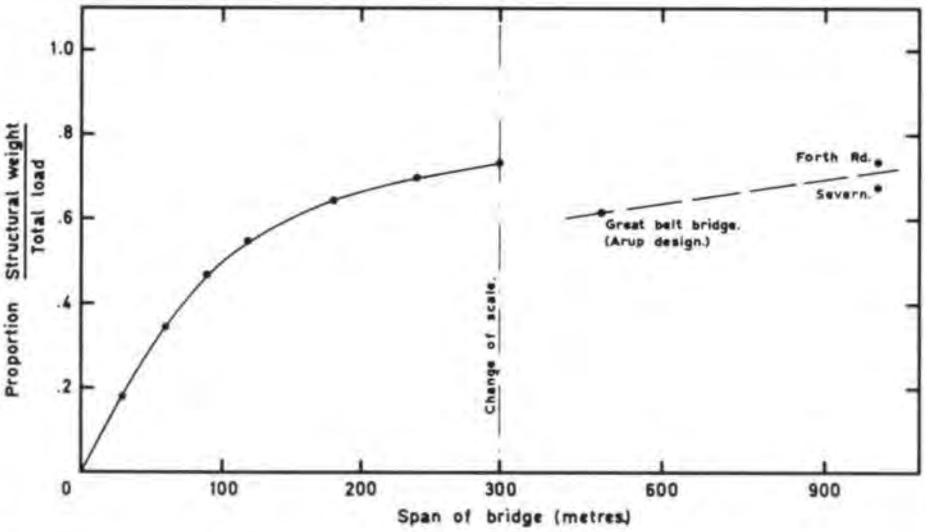


Fig. 4 Proportions of structural weight to total load for various spans

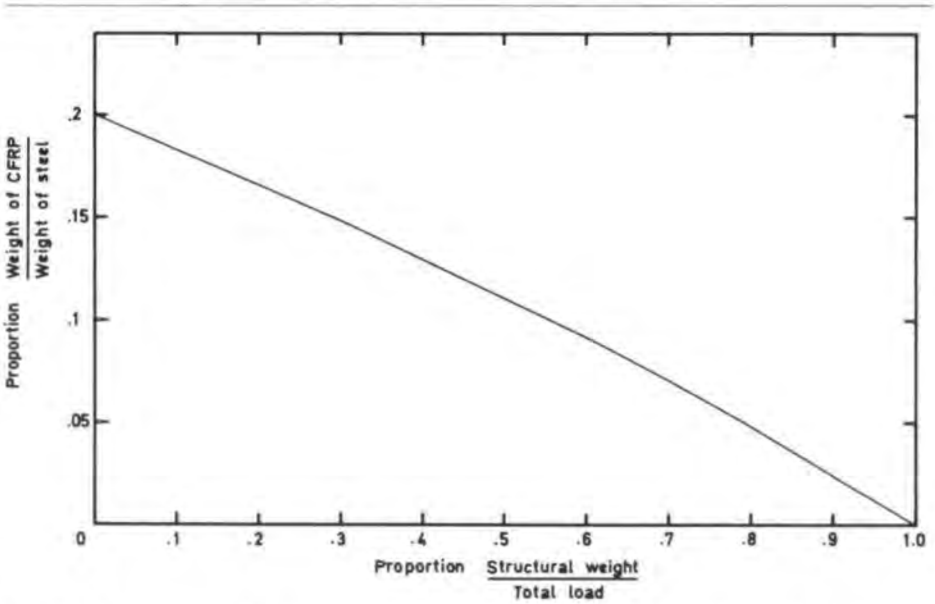


Fig. 5 Reduction in weight achieved by using CFRP for various dead load ratios (equal working stresses taken for steel and CFRP)

Conclusions

It is always regrettable when a new development as interesting as carbon fibres is shown to be so acutely short of potential applications. This paper about its possible uses in structures has been almost exclusively negative, in that the price of the material cannot be expected to come close enough to the maximum acceptable figure in the foreseeable future (say ten years). However, these conclusions must be faced, and they may best be summed up in the following statements:

1 Carbon fibres will still be too expensive, (even when made in large quantities) to justify their large scale use in CFRP for structures.
2 A number of technical problems must be resolved before it is possible to identify all the applications where small amounts will be used to advantage.
3 The most important technical problem is that of making connections between CFRP and either itself or other materials.
4 Even when these problems are resolved, it is unlikely that the construction industry will be able to make full use of the prime characteristic of CFRP—its very high specific stiffness.

The geometry of Shahyad Ariamehr

Peter Ayres

Introduction

The monument of Shahyad Ariamehr is being built near Teheran to celebrate the 25th centenary of the foundation of the Iranian Empire, and of the Declaration of Human Rights by Cyrus the Great. As is fitting for such an occasion, it is a monument to the past—its inspiration clearly coming from traditional design. But it has another purpose concerned very much with today. Iran is not advanced in the modern techniques of building and the monument is seen as an opportunity of introducing to that country some of the sophisticated methods of construction available today—a stepping stone to the future, perhaps.

Outer surface geometry

The monument is essentially an external visual experience and the external surface geometry is thus of the greatest importance.

Although the monument has the qualities of a piece of sculpture, considerable rationalization of the details of the geometry has occurred during its development, with no loss of free form effect.

The final geometry is controlled by a 3 m (10 ft.) square module in plan and elevation, with overall proportions governed by a 21 m (69 ft.) square grid.

The external surface is defined by four separate surface geometries and the areas to which these different geometries apply are shown in Fig. 1.

Surfaces governed by the Defining Curve

Most of the external surface is defined by a family of curves related to the Defining Curve which is the projection in the y-z plane of the curve shown in Fig. 1. This family of curves is formed from the simple relationship given in Fig. 2. All the members of this family lie in vertical planes. A surface is formed as soon as plan sections at level 0 m and 45 m (148 ft.) are specified. The plan at level 45 m (148 ft.) is formed entirely of horizontal straight lines, and that at level 0 m generally so, with the exception of the curved portion AB which is represented by a hyperbola. (See Fig. 3.)

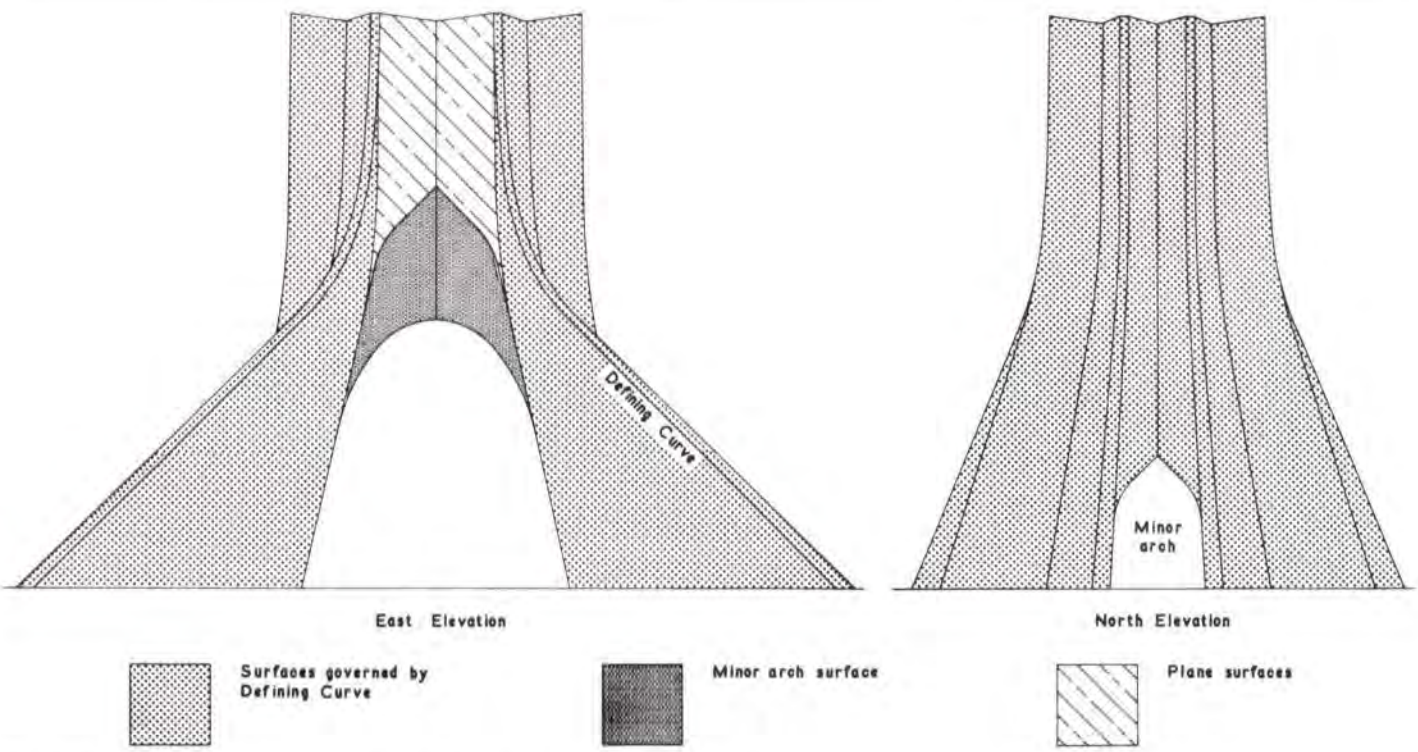
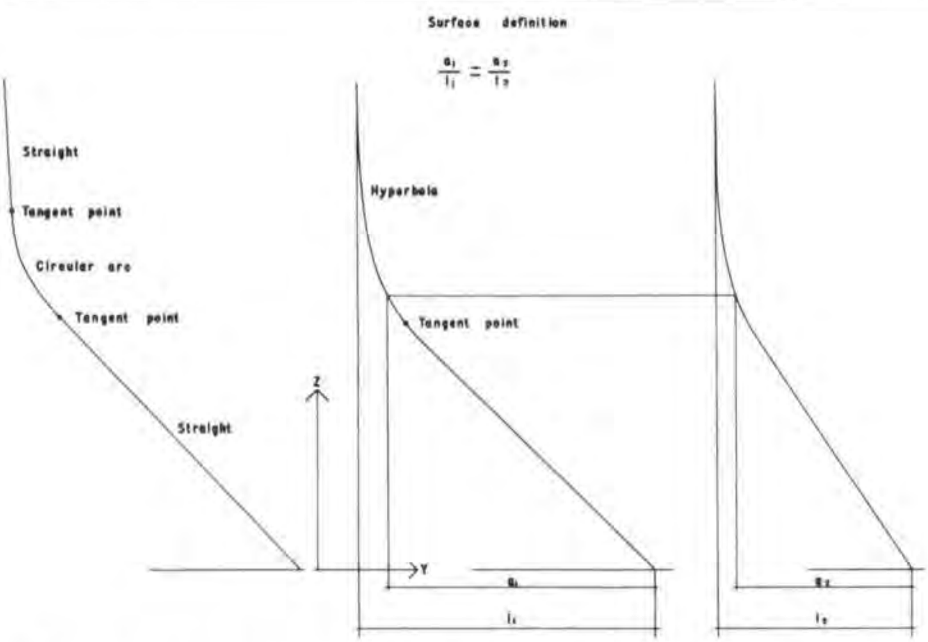


Fig. 1 Elevations of the monument showing breakdown of surface definition



a The original form of the Defining Curve

b The final form of the Defining Curve

c A typical member of the family of defining curves

Fig. 2 The Defining Curve

This figure also shows the simple relationship which governs the position of a defining curve of characteristic length l_2. An outcome of this definition of geometry is that at any horizontal section, the wall profile will consist of straight lines, with the exception of the length A'B'.

The surfaces are formed by slabs of marble as external cladding/permanent shuttering and the family of defining curves is expressed in the form of grooves running from top to bottom. The depth and width of these grooves vary in such a way that the ratios of depth and width to the horizontal extent of the surface at any level are constant. Typical shapes used for the marble slabs are shown in Fig. 4. The vertical projections of the heights of these slabs decrease in an arithmetic progression as the level increases.

The Defining Curve can be considered as being made up of two parts.

The lower straight portion below level 21 m (69 ft.) is the diagonal of a basic 21 m (69 ft.) setting out square, while the upper portion is curved and lies within a 6 m (20 ft.) wide by 24 m (79 ft.) high rectangle.

The Defining Curve was originally envisaged by the architect as being made up of two straight lines joined by a circular arc which was tangential to both. (See Fig. 2a.) This was

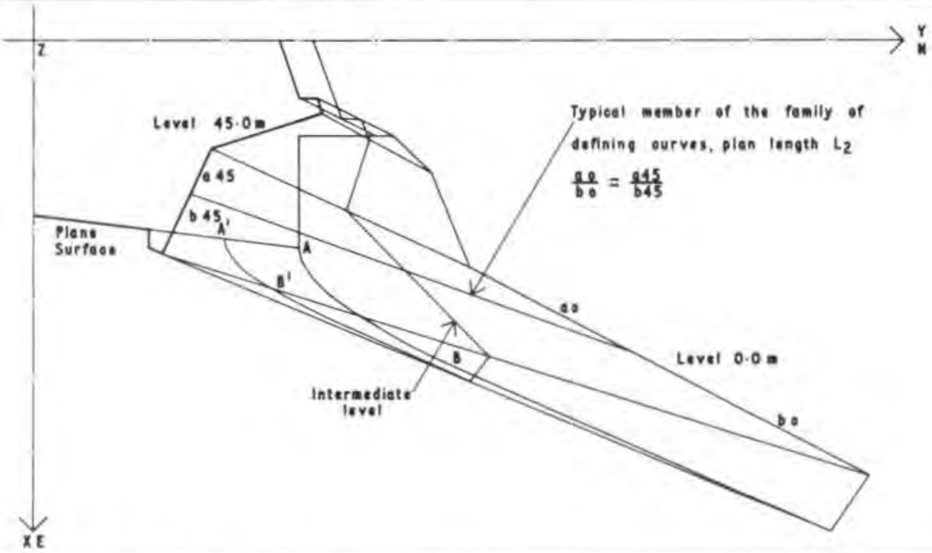


Fig. 3 Surface definition by plan sections

not a positive requirement, but instead represented a statement of the general profile needed in terms of simple geometrical forms. It also had a number of disadvantages. Although easy to draw, it was not easy to represent in a continuous form algebraically, and the lack of transition in curvature between straights and the circular arc could read badly on the large scale of the finished monument. The circular arc only applied in the case of the Defining Curve itself, the other members of the family being made up from straights and a second order curve. Because of these disadvantages, it was decided to replace this compound definition with a smooth curve. A second order curve was obviously required to eliminate the possibility of reversals of curvature occurring between control points, which might occur with a higher order curve. The general second order curve:

$$Ax^2 + By^2 + Cxy + Dx + Ey + 1 = 0$$

has five degrees of freedom to be controlled. To represent the complete Defining Curve in this way proved impossible as the architect required a near perfect straight line from level 0 m to level 21 m (69 ft.). A full height curve could have been used if a maximum chord offset between levels 0 m and 21 m (69 ft.) of about 100 mm (4 in.) was acceptable, but the architect was not satisfied with this. When this offset was reduced by adjusting the position of a controlling point, a sudden change in the form of the hyperbola resulted and two separate branches with a gap in the middle were obtained.

It was then decided to keep the straight line definition to level 21 m (69 ft.) and introduce a second order curve from level 21 m (69 ft.) to the top. Four controlling points were selected from the original Defining Curve, together with the condition that it should be tangential to the straight portion at level 21 m (69 ft.). A close approximation to the original was obtained. The architect realized the infinite possibilities of this procedure, and a number of further modifications was made to achieve the final profile.

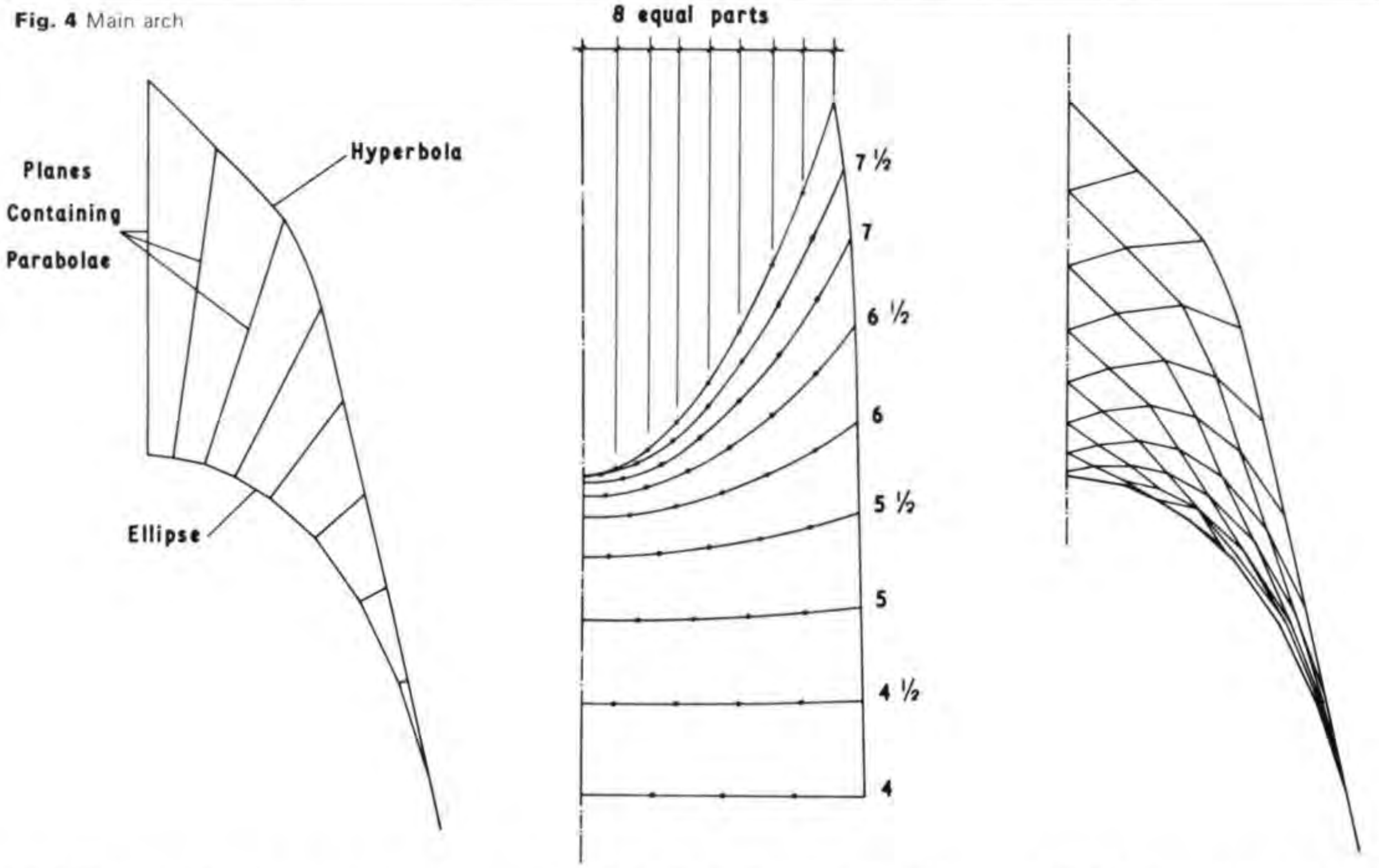
Considerable attempts were made to establish a general algebraic expression for the surface, and, for the portion below level 21 m (69 ft.), it proved to be of the form:

$$Az^2 + Bzx + Czy + Dx + Ey + Fz + 1 = 0$$

However the surfaces above level 21 m (69 ft.) proved too difficult, as did the obtaining of general expressions for normals and parallel surfaces and, as these were the main reasons for wanting the surface equations, no further effort was put into this approach.

At the same time, a parallel line of attack produced a numerical solution to the problem of surface definition by developing computer programs based on the fundamental synthesis of the geometry. This did not, of course, eliminate the problems of normals and parallel surfaces (which still remain unsolved), but enabled the global co-ordinates of any surface to be obtained at any level, or at a number of levels at equal intervals or at

Fig. 4 Main arch



a Half of east elevation of main arch showing typical members of the family of defining parabolae

b Half of east-west section of main arch showing the division of parabolae

c Half of east elevation of main arch showing basis of stone rib pattern intervals equal to the characteristic stone heights.

Plane surfaces

The plane surfaces are vertical and inclined slightly to one another as shown in Fig. 3. They rise from the boundary of the main arch to the top of the monument.

Surface of the main arch

The main arch surface takes the form of a saddle. It is bounded below by inclined walls at level 12 m (39 ft.) and above by the vertical plane walls. The lower boundary is a horizontal straight line. The arch base curve is elliptical, tangential to the walls at level 12 m (39 ft.) with its crown at level 21 m (69 ft.).

The form of the upper boundary is very much governed by the form of the Defining Curve. As with the latter, it is defined in its projection in the y-z plane. It was originally drawn by the architect in three parts:

1 a straight portion from level 12 m (39 ft.) to level 23 m (75 ft.), which was coincident with the inclined springing walls below level 12 m (39 ft.).

2 an upper straight portion which coincided with the continuation of the lower straight portion of the Defining Curve and

3 a circular arc tangential to both of these straights. In order to remove the visual effects of abrupt changes in curvature which would result from this compound curve, the original definition was replaced with a single second order curve of the same general form as that used for the Defining Curve. This straightforward algebraic form would also help to simplify the overall surface definition. Four points were used as controlling conditions for determining the equation, together with the condition that it should be tangential to the inclined walls at level 12 m (39 ft.).

The surface is formed by a family of parabolae. These lie in planes which are normal to the ellipse where they meet it, and intersect the boundary hyperbola in such a way as to divide its projection on the y-z plane into equal arc lengths. This is shown in Fig. 4a. The parabolae are in turn divided up as shown in Fig. 4b. The nodes so formed are then joined to form the marble rib pattern which is required (Fig. 4c). The final form of the ribs which are curved is shown in Fig. 6. The surface defined above corresponds to the outer surface of the ribs.

Much effort was expended in an attempt to define the surface algebraically but the rigid constraints in form and boundary prevented this, or perhaps experience of a similar unavailing task performed for the Defining Curve surfaces made surrender too easy. The geometry was, therefore, again evaluated by numerical techniques. Computer programs were used to determine the actual boundary intercepts: the co-ordinates of the curved ribs at increments along their length and horizontal section co-ordinates of the outer surface.

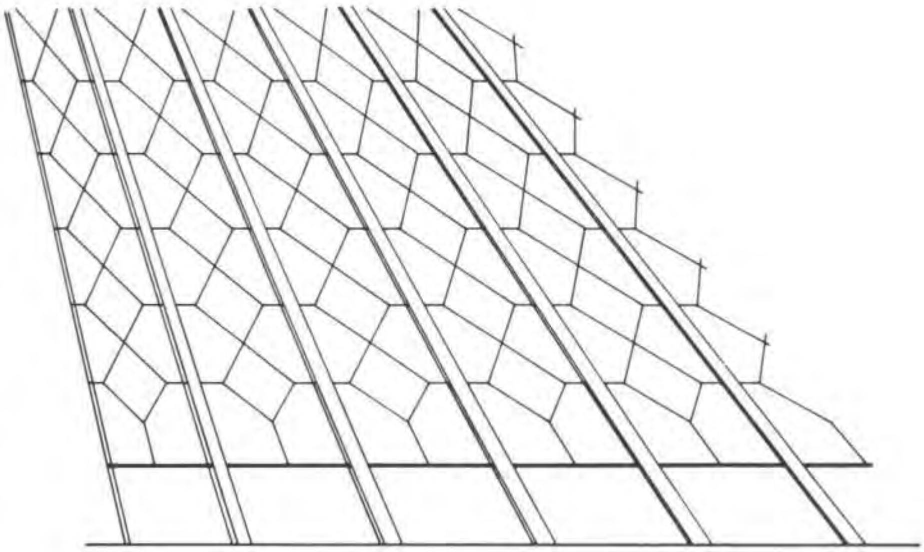
Surface of the minor arch

This arch forms a side entrance to the main arch area. It has a constant section of a form similar to the upper boundary of the main arch.

Internal structures

Although the external surface is of the greatest importance there is a large volume of space inside the monument, and the architect has chosen to fill this with a number of interesting reinforced concrete structures.

Fig. 5 Typical pattern of marble cladding



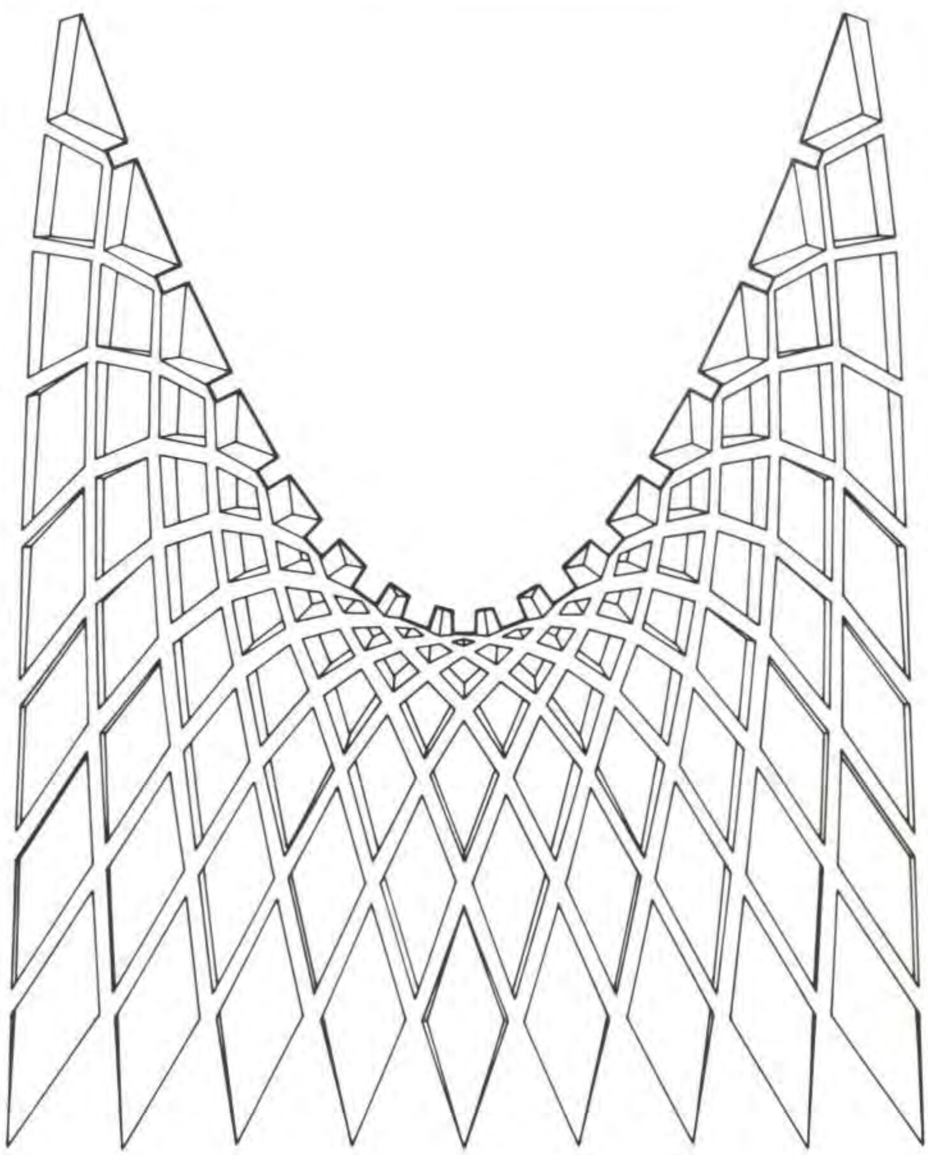


Fig. 6
East-west section through main arch showing stone rib pattern

Dome

Situated near the top of the monument is a dome about 10 m (33 ft.) high. The geometry of the dome is based very much on traditional Iranian architecture, the dome being the standard method in Iran of enclosing large spaces. They are commonly made of mud bricks and lined internally with coloured ceramics.

The dome in the monument is a multi-facet surface made up of triangular planes, of which there is considerable repetition. The layout of these elements and a section are shown in Fig. 7.

As can be seen from the plan, the geometry of the dome is governed very much by the external wall profile of the monument at the level at which it occurs.

The triangular elements will be of precast white bush-hammered concrete with ceramics and coloured glass lights as decoration.

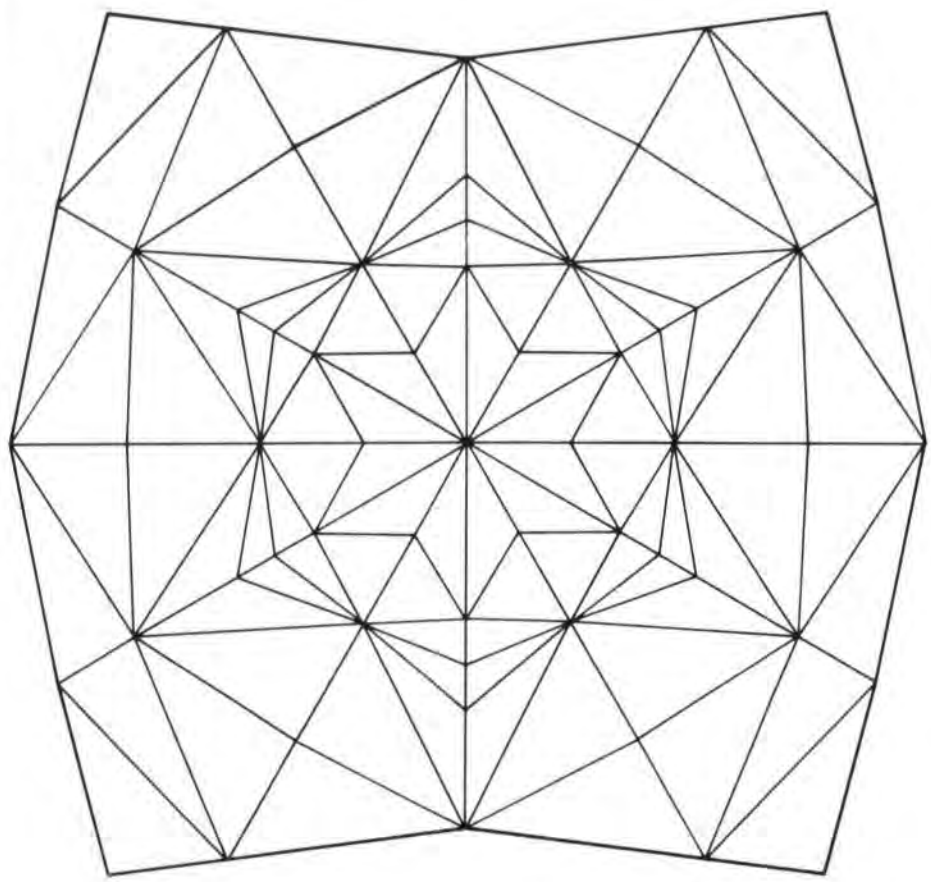
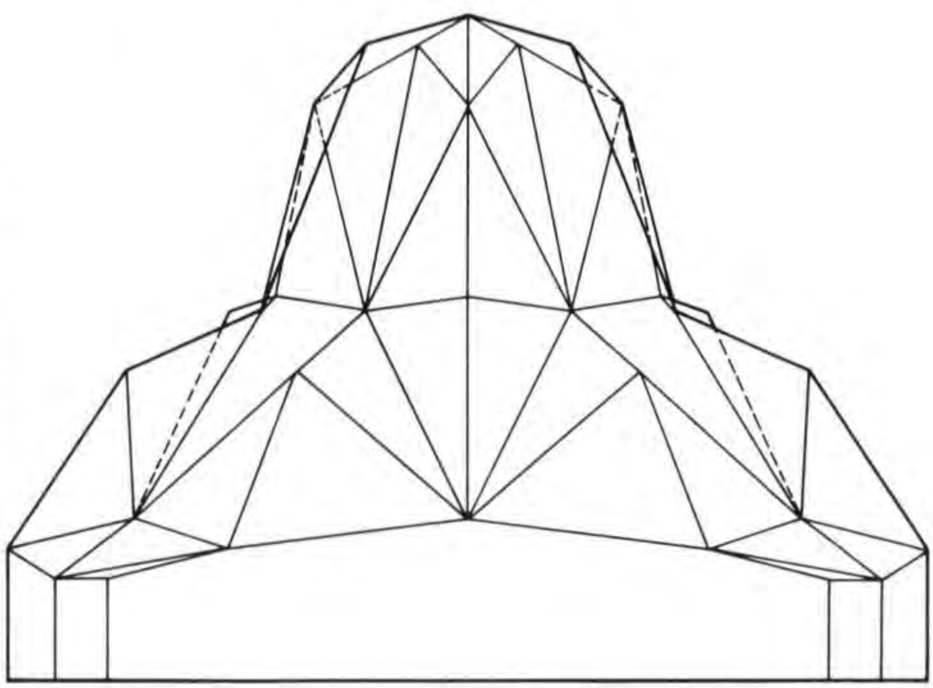


Fig. 7a above right
Plan of the dome

Fig. 7b below right
Section through dome



Floors

The geometry of the major floors which occur above level 21 m (69 ft.) is a function of the internal profile of the monument at their respective levels.

Floor at 23.4 m (77 ft.)

A section through the monument at this level includes a profile of the main arch, presenting an internal space in the shape of an egg timer (Fig. 8). The floor at this level spans by means of ribs from wall to wall, with a support on the crown of the arch. The ribs have profiles similar to that of the main arch section at this level. The internal profile at level 23.4 m (77 ft.) was obtained from the external one, and a best fit second order curve found to represent this. The rib profiles were then obtained at increments along their length to give the profiles shown in Fig. 8.

Floor at 33 m (108 ft.)

The main arch surface ends below this level and the section at level 33 m (108 ft.) is typical of the upper levels of the monument. The geometry of the ribs of this floor is based on the boundary wall profile at soffit level. The rib profiles are set out in a floral pattern from circular arcs as shown in Fig. 9a. The ribs corresponding to this setting-out are shown in Fig. 9b.

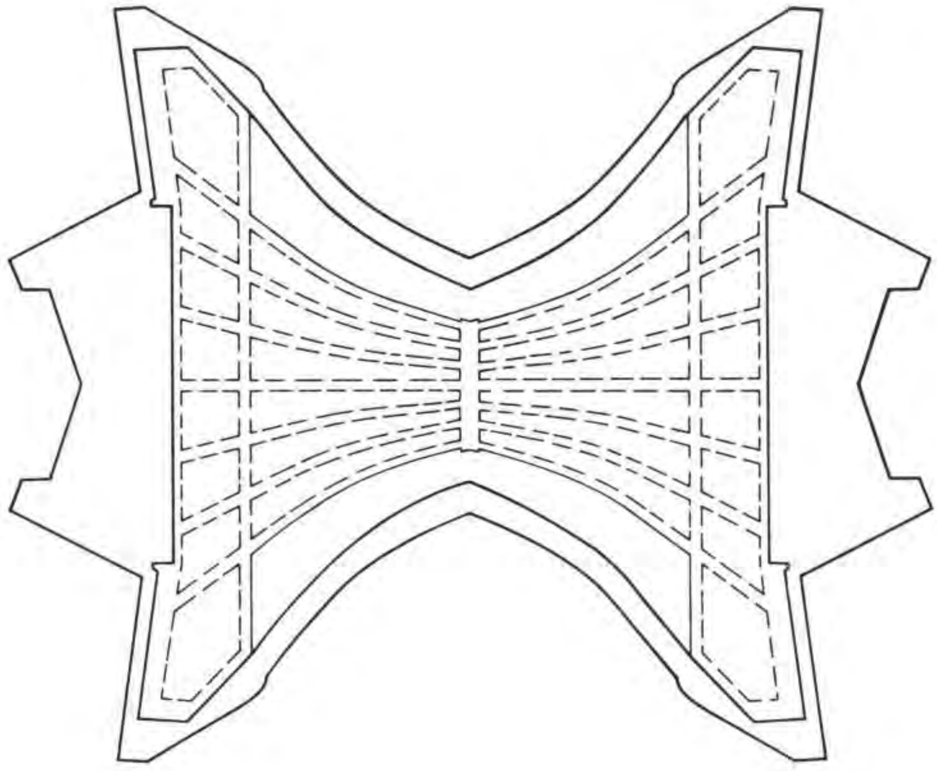


Fig. 8
Plan of the floor at 23.4 m

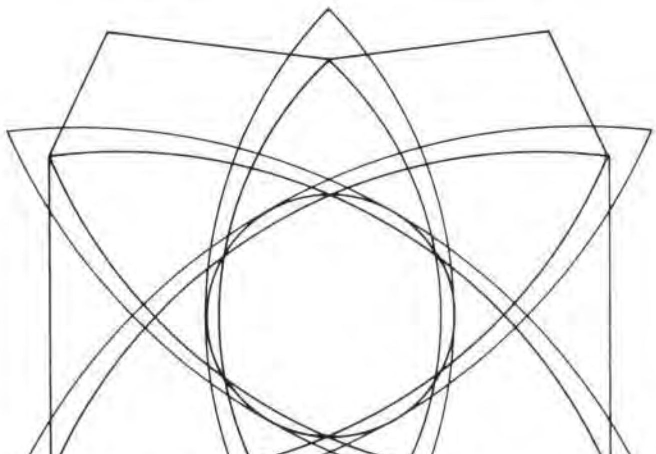


Fig. 9a above
Plan showing the geometry of the rib pattern of the floor at 33 m

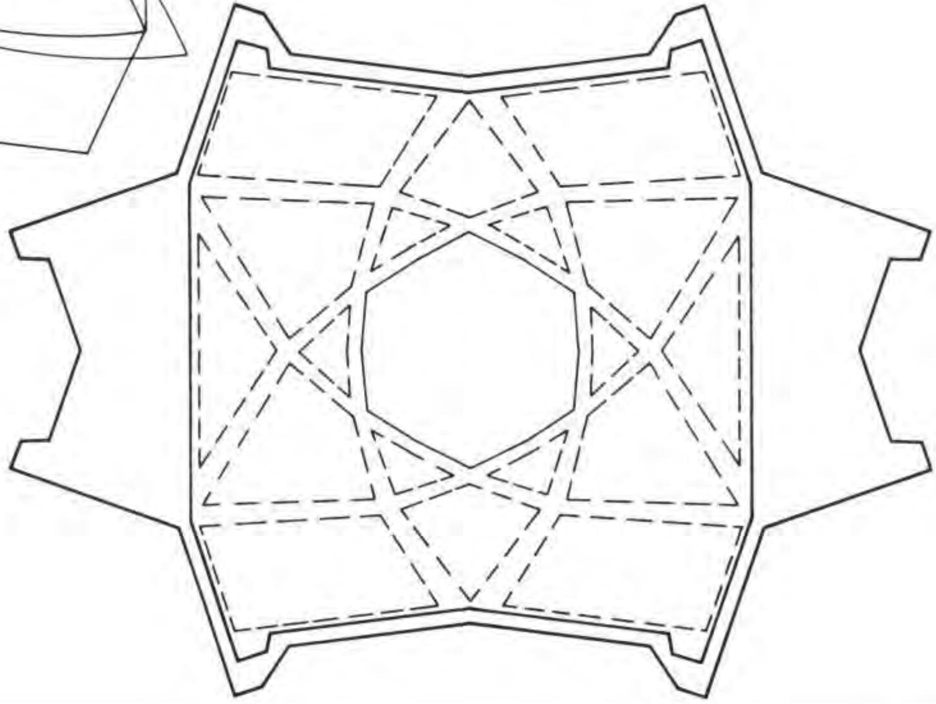


Fig. 9b right
Plan of the floor at 33 m

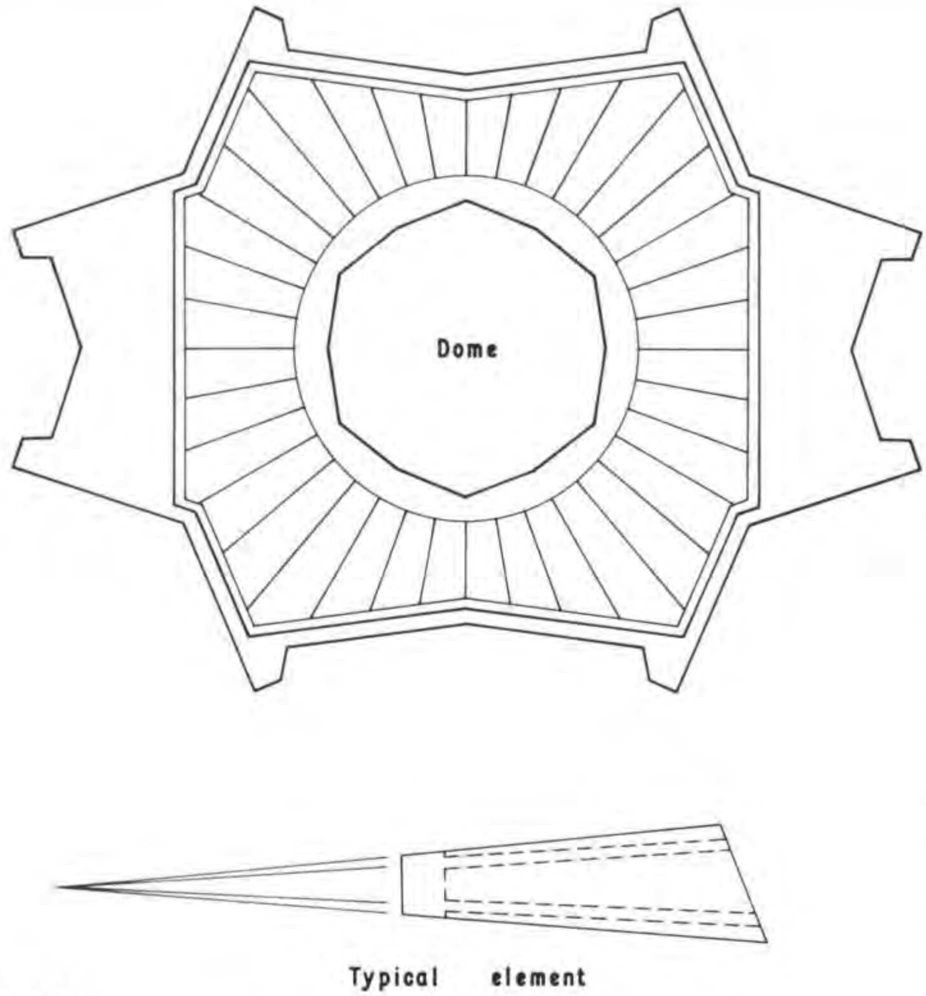


Fig. 10
Plan of the floor at 39.5 m

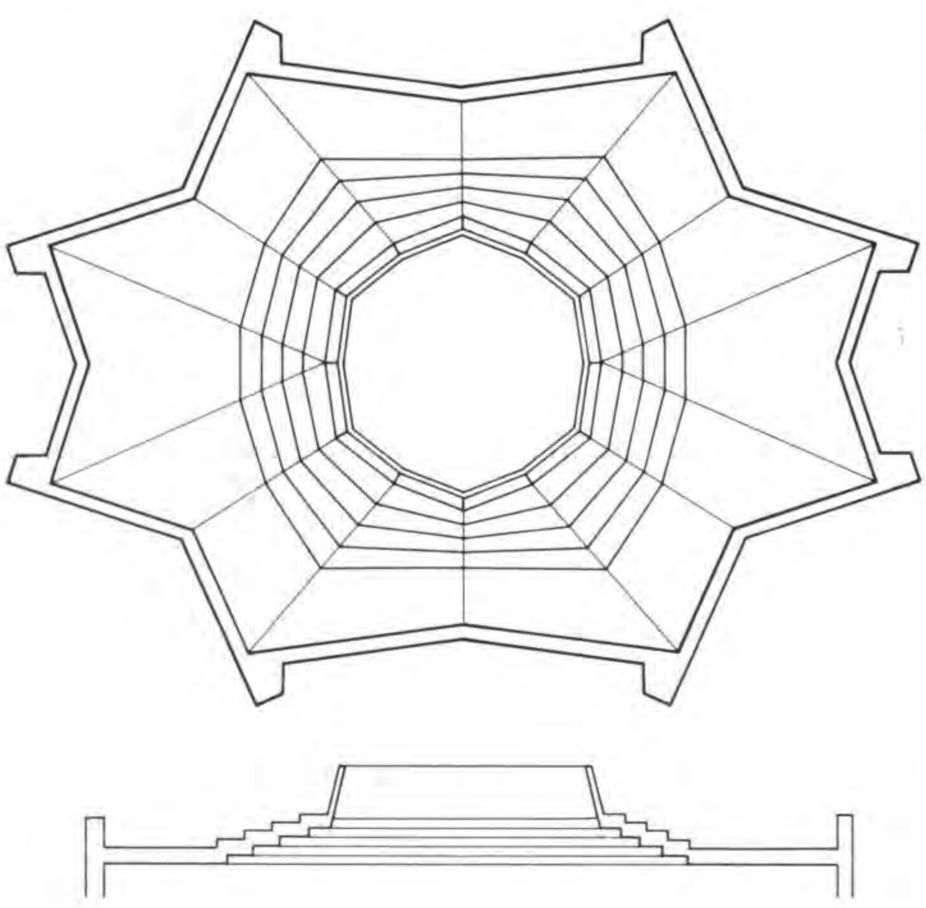


Fig. 11
Plan and section of the floor at 43.5 m

Floor at 39.5 m (130 ft.)

This floor is bounded on the outside by a typical external wall profile and on the inside by the dome, and spans between the two. The soffit of the floor is not visible and a sophisticated expression of the rib geometry was not required for architectural reasons. The geometry chosen, for purely structural reasons, gives a floor structure of radial ribs, enabling economical precasting of 36 elements to be achieved while taking account of the dome boundary and the varying perimeter line, Fig. 10.

Floor at 43.5 m (143 ft.)

This floor comprises two conical surfaces, resting on a flat boundary slab (Fig. 11). The inner cone forms a parapet to the edge of the floor, leaving a gap between itself and the dome to allow light to penetrate to lower floors. The geometry of this cone, which consists of twelve planes, is related to the geometry of the dome at this level. The base of the lower cone is set out from the external wall profile at this level as shown in Fig 11. In order not to introduce warped surfaces, both the top and the soffit of this part of the floor are stepped.

Structures below ground level

In order to remove the need for ground level door openings which would interfere with the surface of the monument, the internal space is entered by means of an entrance tunnel and a basement. Both of these are of reinforced concrete.

Basement

The basement occupies a volume about 21 m (69 ft.) x 16 m (52 ft.) x 5 m (16 ft.) high, occurring between the buttress legs as shown in Fig. 12 and will be used for display purposes. The roof has to be able to support loads caused by heavy vehicles which may be required to pass under the arch on festival occasions, and so the structural form used has to be efficient. To combine this requirement with the need to provide an interesting space architecturally, a two pinned portal structure of varying cross section spanning in the shorter direction, is used. This produces the internal folded effect shown in Fig. 13.

Entrance tunnel

The basement is entered by means of an underground entrance tunnel which begins about 30 m (98 ft.) from the basement with steps down from ground level to underground cloakrooms and a ticket office. The tunnel itself also has a folded form but this has no structural significance. The form is shown in plan and section in Fig. 14.

Conclusion

This article has concentrated on the development of the geometrical forms used in the monument. The further development of the material and structural forms is another story.

Ove Arup & Partners' contribution to the geometrical form has been described in some detail, but it must be pointed out that the inspiration for all the forms used came from the architect, Hossein Amanat of Teheran. His design was selected from a national competition for a monument for this important celebration. It was because of the architect's tenacity in keeping, and improving where possible, all his original ideas that the problems described here had to be faced and solved.

The contract has been let to an Iranian firm, the MAAP Construction Company and the work up to ground level is now complete. The marble is being cut, and the task of construction of the superstructure has just begun. Robert Afia is our resident engineer for the project.

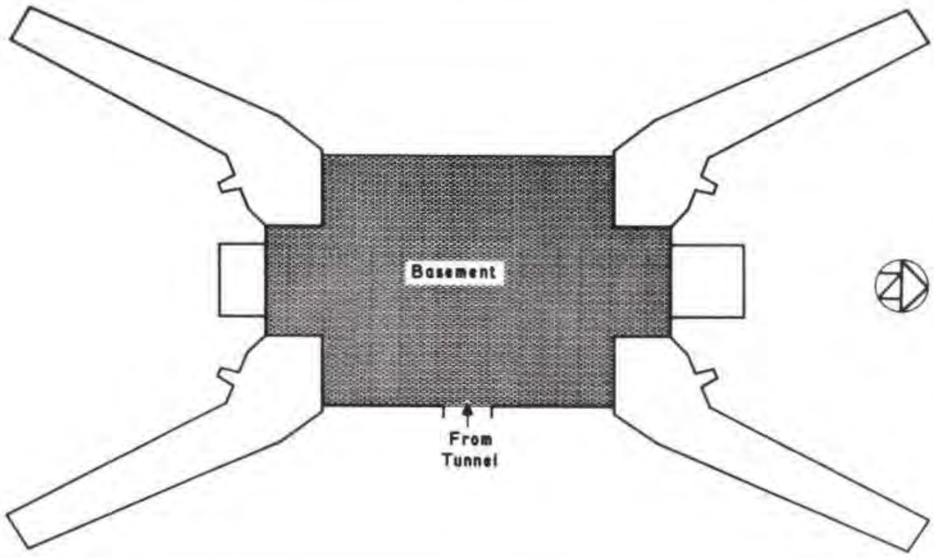
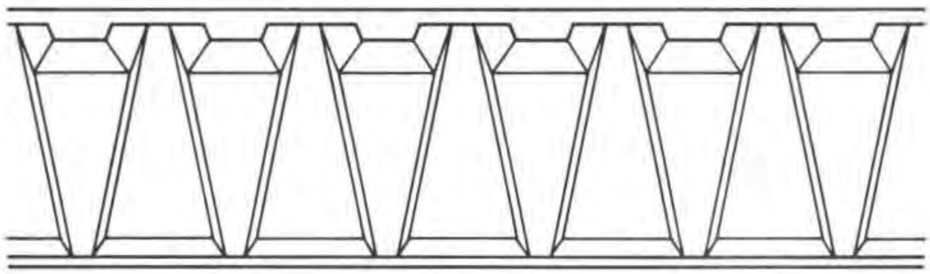
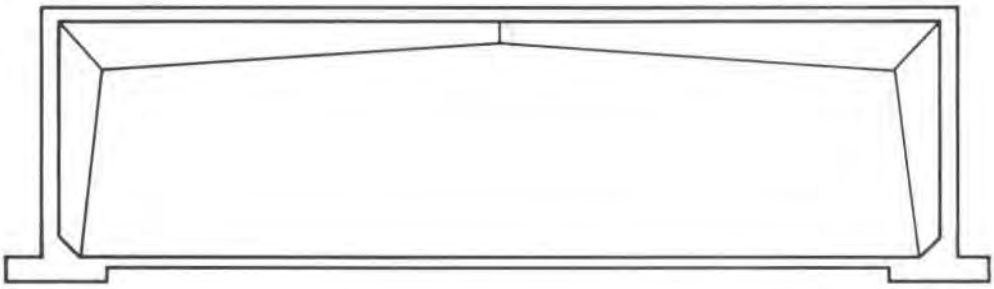


Fig. 12
Plan at −5 m showing the area occupied by the basement

Fig. 13 Basement

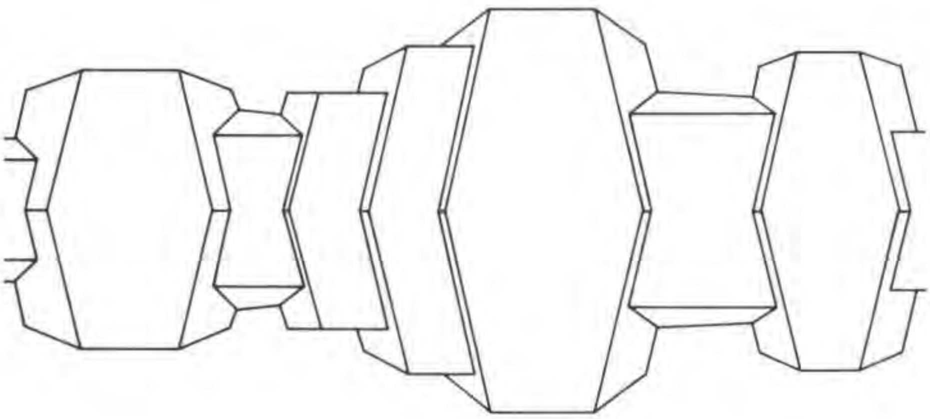


a Elevation of the basement wall

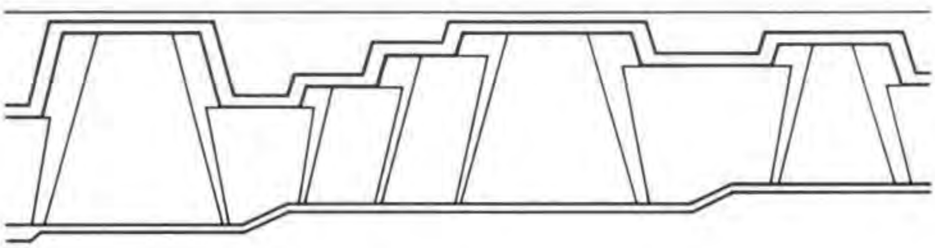


b Cross section of the basement

Fig. 14 Entrance Tunnel



a Plan of the entrance tunnel



b Longitudinal section through the entrance tunnel