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Front cover: Gateshead Western Bypass, horizontal alignment showing Consett and Lobley Hill interchanges
Back cover: Leptis Magna, Libya, columns to ornamental wall at the rear of the stage to the theatre. (Photo: Colin Wade)

Roofs and roofing

Ove Arup

This article was the opening speech at the International Symposium on Roofs and Roofing, Brighton, 9–13 September, 1974.

It's nice to have a roof over one's head as a protection against inclement weather. In fact, it is more than nice, it is a basic need of man, and has been so for tens of thousands of years. All this time man has built roofs and has handed the experience gained to his offspring, so you would have thought that by now we would know how to build roofs. But not a bit of it. There is more to a roof than meets the eye. Of course what meets the eye is also important, for the character of the urban landscape is often dominated by the type of roof which has been developed through generations in a particular district. But the present generation has discarded tradition in exchange for maximum performance at minimum cost, scientifically measured and sponsored. Tradition produced a pattern of affinitive units effected with local materials. The result has often been praised as *great*, albeit anonymous, architecture, conferring identity on a locality and possessing an internal harmony as well as harmonizing with the surroundings.

But Architecture is not anonymous any more, whatever it is. It is personal, maybe, sincerely believed in by dedicated artists, but it is also subject to warring ideologies and changing fashions, to personal vanity and pursuit of notoriety, and transcending tradition and locality. An international muddle, you may say.

This is not to say that the form of a roof is not influenced by fashions and ideologies or artistic considerations. On the contrary, that influence can be decisive and may override considerations of efficiency. When the flat roof was launched by the modern movement in Europe, it was, to begin with at least, as an article of faith, and it was violently opposed by the old guard as an affront to decency or national dignity. In other words, the shape of the roof may be governed by idiosyncrasies outside the scope of rational argument, but nevertheless powerful, and indeed often justified on artistic grounds.

But of course, it is mainly the technological revolution which has brought about the change.

With, or rather without your permission, I will quote from a paper I gave to the British Association in January '42—during the war, at the instigation of Julian Huxley:

'The problem is the same here as in other spheres of human activity—a wealth of new knowledge, new materials, new processes has so widened the field of possibilities, that it cannot be adequately surveyed by a single mind. Corresponding to this increase of means there are increased or entirely new requirements to be satisfied. Our needs increase with the means. Standards are raised, new services introduced.

This produces the specialist or expert, and the usual problem arises how to create the organization, the 'composite mind' so to speak, which can achieve a well-balanced synthesis from the wealth of available detail. This is, I suppose, one of the central problems of our time.

How then can we overcome this difficulty?'

This was 32 years ago—and the situation is still essentially the same, only more so.

At the time I pointed out the need for a register of reliable data about the properties and costs of new materials and systems, what they can do and what they can't do, as proved in theory and practice. The bewildering variety will remain, but the practitioner, architect, engineer or builder will at least have the possibility of searching for a solution which will solve his particular problem, without being misled by over-optimistic advertisements and high-powered salesmanship. Unfortunately it is *not* possible to produce such a register. The Building Centres which sprang up in various countries, made a feeble attempt to do so, but they could only provide a forum where exhibitors could show their wares. They could not check the information provided and could not, or dared not, give any guidance to customers. Not only because they were financially dependent on the exhibitors, but also because they lacked the necessary expertise. Even if the State supplied the cash and the experts, experts cannot be umpires as well, they have experience in depth—but not over the whole field. *Nobody* has that, because the field is ever-widening. Standards are raised, new problems and needs are created. One of the latest additions is the need to conserve sparse resources, a matter which has perhaps been given too little attention by the contributors to this conference, although it is of course, mentioned by several of them. As one contributor remarked, we are now only beginning to know how a roof should be designed. We are only beginning to get the measure of what takes place in a roof, the stresses and strains and variations in temperature and moisture content caused by varying conditions and the effect this has on the materials employed. But as soon as we cure one problem, two others appear—just *because* we have cured the first.

And of course, the various demands we make on our roof often conflict. New materials spring up everywhere. A wealth of research is going on, much of it irrelevant. Conflicting claims are made and some are spurious. There is nobody who can classify this information, who can separate the wheat from the chaff.

The practitioner has to effect a synthesis appropriate to his special case. He needs not only one specialist, but several. He needs all the facts so that he can make his selection. He may be prejudiced, like the Scot who kept on changing his doctor until he found one who recommended whisky as a cure—but for that he must himself take the responsibility.

The search for the perfect roof goes on. Any designer has an inbuilt desire to find the best possible solution, otherwise he would not be a designer. Invention never stops, this is fortunate I suppose—although sometimes one is inclined to doubt it—but this is another matter. And invention and design are closely related. Invention changes the rules, so the search goes on. And, presumably it will never stop. We will never know how to design the perfect roof. For in fact there is not such a thing as a perfect roof which will fit all cases. So we have to find what suits us, our purse, our local conditions, our aesthetic aspirations.

An international conference where all those who can make a contribution to the design of roofs in one way or another, can exchange information, could be the best possible way to enable the designer to learn about and evaluate the various possibilities open to him. For then he can confront the experts with the opinion of other experts, and hear all sides of a question being argued. But it all depends on the contributors to the conference. They produce the bulk of the evidence, and the quality and relevance of that evidence makes the conference. It is, of course, unavoidable that some of the evidence is repeated unnecessarily or is of minor importance, and conferences would be less time-wasting if the bulk of evidence and the number of points to be discussed could be cut down to essentials by a kind of censorship —but that is, I am afraid, not possible.

The forum must be open to all opinions. What seems far-fetched today may be vindicated tomorrow.

Having gone through the contributions in the large blue book and read through those I considered the most important, I feel hopeful about this conference. That the subject is important cannot be disputed. Moreover it is one in which much progress has been made in many different centres and it is very necessary that this information should reach all concerned in this field in various countries. And it seems to me that there are many very valuable contributions. Most of them deal with the flat roof and this is reasonable, because this is both the most difficult and one which will be forced upon us in large scale multi-level developments. If we can make flat roofs waterproof, keep the heat in when we want it and out when we don't, prevent noise from penetrating or spreading, prevent moisture from spoiling its efficiency, allow air to enter or not, as it pleases us, a roof which will not crack or expand unduly, and yet be tolerant of the movements of adjoining building elements, and which needs no attention for a generation or more, well then we have a useful roof—provided it does not cost too much, doesn't take up too much room and provided we like the look of it. And if we can design such a flat roof, then we can indeed design any kind of roof.

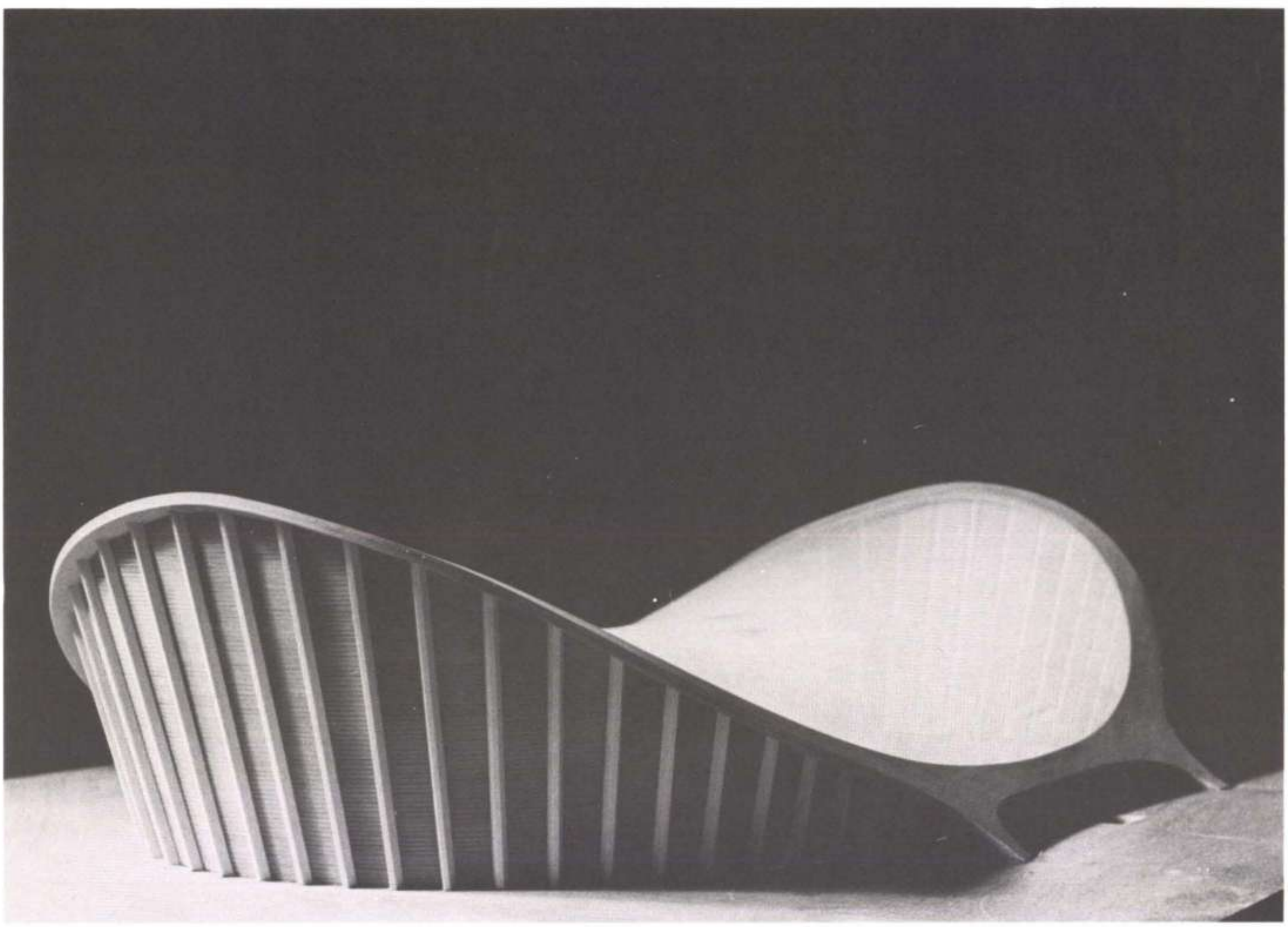
This conference is not concerned with the structural aspect of roof design. This is as it should be for this is a large field in its own right, or rather a part of the general domain of structural design, and it would be counterproductive to deal with that at this conference.

But obviously the shape of the roof is a basic factor in the total design of a roof, including the structure. And it is probably safe to say that an inclined roof, which will get rid of rainwater quickly, is easier to deal with than a flat roof and should therefore be preferred where it is otherwise appropriate.

I have learnt a good deal from reading the proceedings of the symposium, and I expect and hope that it will generate a lively and useful exchange of opinions and information. Some of the authors range over a wider field and give a useful overview—what the Germans call überblick—over the development in a particular country, or over a group of materials doing the same job. Others specialize on a particular system, or material, or research programme.

I got the impression from my reading that the future of waterproofing and heat insulation of flat roofs may rest with the plastic materials, the "phase 4" in Ingenieur Hechler's useful review. (Paper. 2). And it seems that "Umgekehrt is auch was wert." The other way round may be equally sound, a tag I learnt in my childhood in Germany, and the truth of which has often impressed me. For one of the more promising developments appears to be the 'Umkehrdach' or 'upside down' roof, where the thermal insulation is placed above the structure and the water-proofing layer. But it depends on whether the right material can be developed for this top insulating layer, if it hasn't quite been developed yet. The material we need, and which I have been looking for for a couple of generations to protect concrete walls and roofs, is a material providing high thermal insulation which will remain stable under the influence of water, heat and frost and which weathers well. Perhaps it will be developed one day.

And now it is time to open this conference. It has already been done by the Mayor—but just in case it isn't quite open yet, I have pleasure herewith to open it again, as I always do what I am told to.



Job no. 3587—Model of Sølrod Sports hall roof

The North Seaton Bridge

Victor Nassim
Bill Smyth

Introduction

North Seaton Bridge is a two-lane, prestressed concrete viaduct about 180 m long which is being built across a pleasant tidal valley in Northumberland within sight of the sea.

The bridge carries the South East Northumberland Spine Road, which is being constructed to give access to industrial sites in the north of the county, across the estuary of the Wansbeck River. The road and the other bridges on it are designed by the Northumberland County Surveyor who appointed us to design the North Seaton Bridge in January 1970.

The road will eventually be a four-lane dual carriageway. At this stage only one of the carriageways is being built to be used as a two-way road. The bulk earthworks for the full width of dual carriageway have been carried out and the bridge is designed so that a second bridge can be built alongside.

The site

The bridge crosses the valley at a skew of about 25°. The valley is about 140 m wide with banks which are about 15 m high and fairly steep. It is tidal and is flooded at high water while at low water the floor consists of sand flats through which the river finds its way to the sea. The north bank of the valley is covered with colliery shale and the south bank with grass and bushes. There are high tension electric cables, overhead and underground, crossing the river at the site and there were also several piers from a partly demolished colliery tramway viaduct.

In spite of all this the valley is a nice seaside valley especially on a sunny day.

Geotechnical investigation

The original soil investigation for Northumberland County Council consisted of 26 shell and auger holes, some of which were deepened by rotary coring, and two rotary cored holes. Most of these boreholes were arranged in two rows along the outer kerb lines of the proposed carriageways at a spacing of 20 m.

We asked for the investigation to be extended and, as a check on the shallow mining situation, one borehole was deepened to intersect the Moorland coal seam. Nine more rotary cored holes were drilled on each bank in order to define more accurately the profile of bedrock near the abutments.

The general geological sequence is of boulder clay overlying coal measures. The surface of the sandstone bedrock is nearly horizontal on both banks. It falls steeply, forming buried cliff faces, and between these there is a buried channel cut about 15 m into rock. This channel is filled with alluvium and has an axis slightly skew to the present river valley. The dominant rock type in the floor of the channel is mudstone.

Two faults with downthrows of 5.5 m and 2.5 m to the north are inferred to cross the centreline near the north end of the bridge.

The site lies in an area of abandoned deep coal workings. The Low Main, Yard, Bottom Main and Top Main Seams have been mined beneath and close to the bridge site. There is also a number of shallow seams which are thin or of poor quality and these are not likely to have been mined. Most of the workings which could affect the bridge have been abandoned for many years and all surface subsidence is now likely to have ceased. Workings in the Top and Bottom Main Seams are relatively recent, but they are not likely to affect the bridge because there is almost certainly a fault between them and the bridge and they are long wall workings which subside rapidly.

The National Coal Board's programme does not include proposals for mining which could affect the bridge.

The principal conclusions from the geotechnical study were as follows:

(1) The bridge should not be affected by subsidence due to past, present or future coal mining.

(2) The valley contains a buried channel cut some 15 m into rock which has been infilled with alluvium. The spans of the bridge should be related to the profile of the river valley and the buried channel, and foundation locations chosen to limit the practical difficulties of constructing foundations.

(3) Except for the foundations at the south end which might be constructed as pad or strip foundations on rock or boulder clay, all foundations should be of steel H-section piles driven to effective refusal in the underlying rock.

(4) The environment is corrosive to steel and concrete and precautions against corrosion would be necessary.

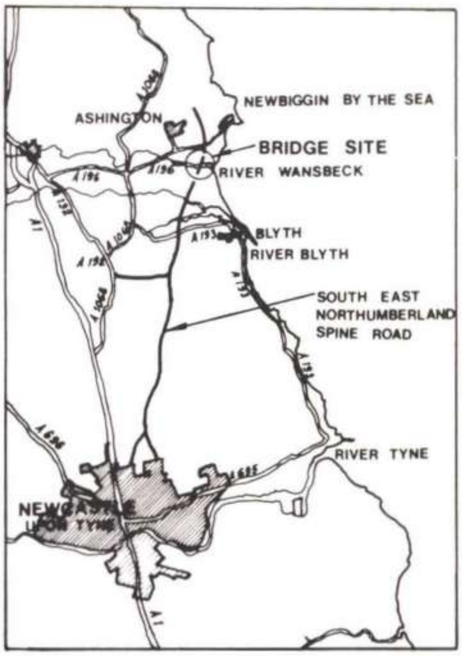


Fig. 1
Location plan.

Fig. 2
The estuary from the bridge, with the sea beyond. (Photo: Bill Smyth)

Loading

The bridge is designed for HA loading and 45 units of HB loading.

Design studies

Some of the principal factors which we had to remember during the preliminary studies of structural schemes were the tidal site with its access difficulties, the steep-sided buried valley filled with highly compressible silt, the skew crossing, the corrosive environment, and, of course, the character of the valley itself.

Access could be either by means of a bund which would have to be constructed in sections to avoid blocking the valley, or by a temporary jetty, as working between tides is not practical.

The foundations had to avoid the steeply sloping sides of the buried valley, so that the range of possible spans was not continuous but strictly limited. The foundations would be relatively expensive, which tended to favour longer spans; this was partly offset by the need to provide proper foundations for any temporary works where settlement could damage the structure; for example, settlement of the falsework during placing of concrete in situ.

The piers have to be skew to the bridge to minimize scour.

The bridge should respect the character of the valley, and if possible people crossing it should be able to see the river valley and the sea.

The access problem and the problem of temporary foundations suggested that a largely prefabricated scheme might be economic, although it would still have been necessary to provide access for constructing the permanent foundations. We investigated schemes using composite construction in steel and concrete with spans ranging from 15 m to 45 m, but they were not competitive in cost.

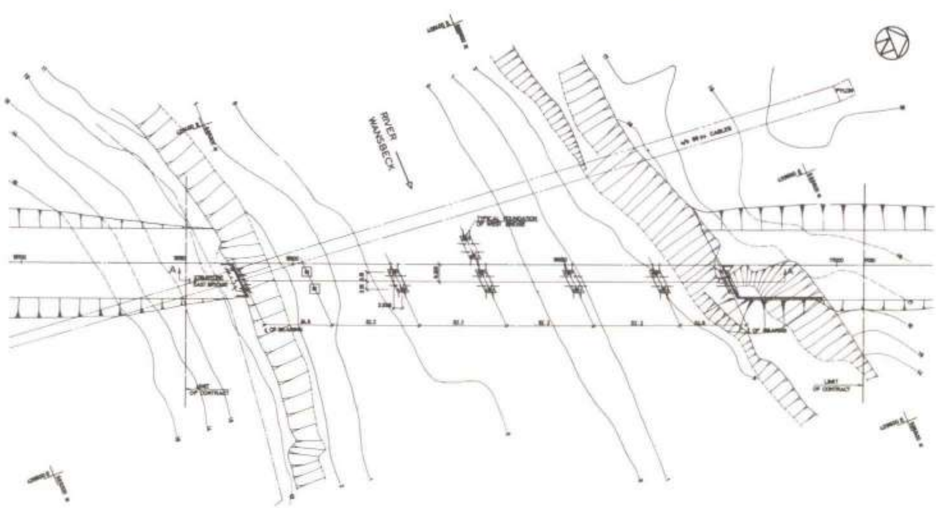


Fig. 3
Foundation plan.

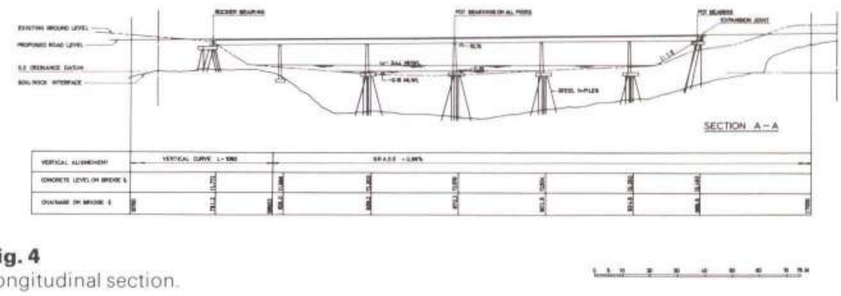


Fig. 4
Longitudinal section.

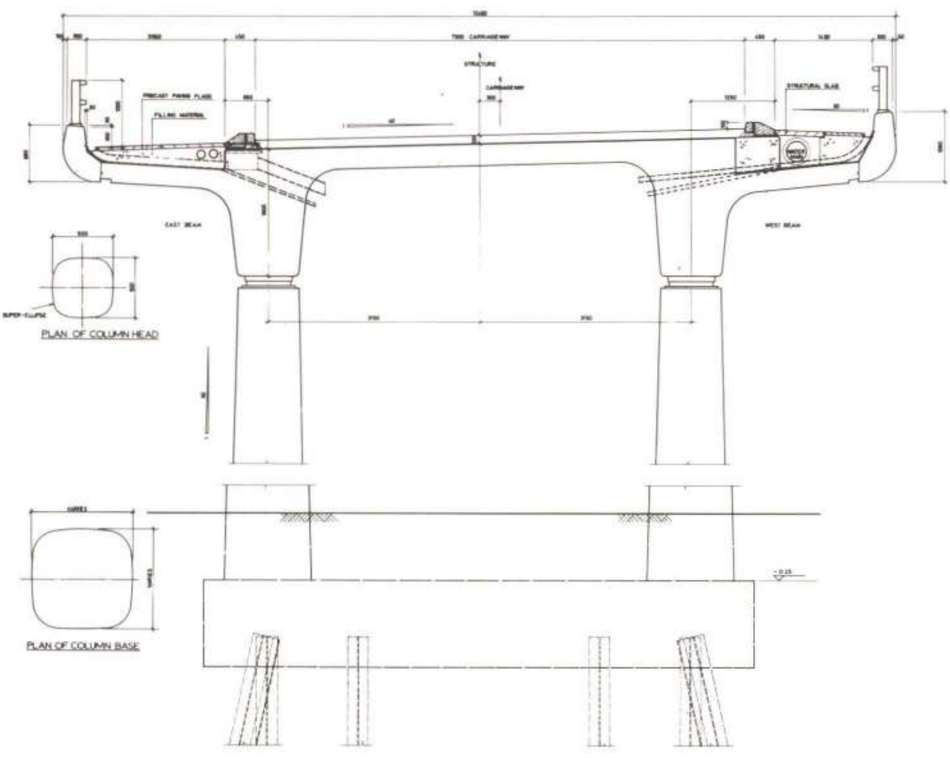


Fig. 5
Cross section and column sections.

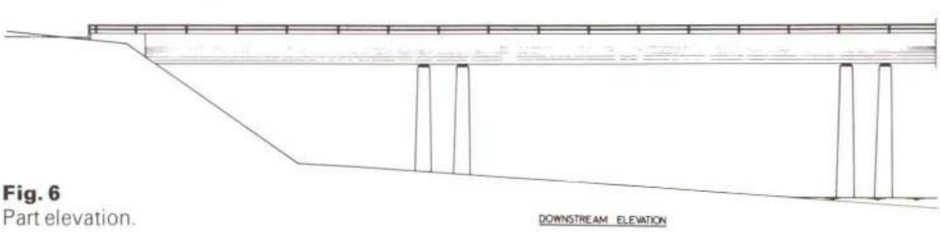


Fig. 6
Part elevation.

All the short span schemes, whether in situ or partly prefabricated, were found to be expensive because of the high foundation costs. They also tended to have a 'forest of columns' effect which would not be right in this valley.

After the range of spans and of structural solutions had been narrowed, we studied two types of cast-in situ, post-tensioned structures, both with spans of 45 m and 32 m. (Because of the buried valley there is no possible span between the two). The first was a hollow box which is structurally very efficient. The second was a double tee beam structure which is structurally less efficient than a box but simpler to construct. A scheme using double tee beams with 32 m spans was found to be the cheapest and was also very suitable in relation to the site.

The design

The deck structure of prestressed concrete consists of twin tee beams in six spans. Each tee beam has its own independent line of supports, set skew to each other so that each pair lines up with the river flow. The interior spans are 32.2 m and the end spans 24.8 m. The ends of the bridge are supported on bank seat abutments, hinged at the south end, with an expansion joint at the north end. There are no transverse diaphragms except at the abutments. The deck is supported on slender, tapering, reinforced concrete columns through bearings which allow sliding and rotation.

One pair of columns is founded directly on rock. All the other columns and the abutments are carried by steel H piles driven through the alluvium into the underlying rock. The reinforced concrete pile caps are skew to support the pairs of columns, and are buried to reduce scour and obstruction and because it looks better. One of the pile caps conflicted with a foundation for a coal conveyor and we decided that it would be simpler to demolish the old foundation than to have an irregular arrangement of spans.

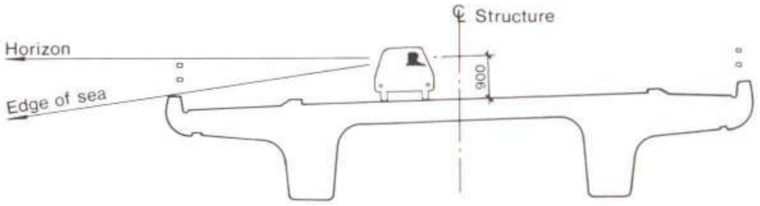
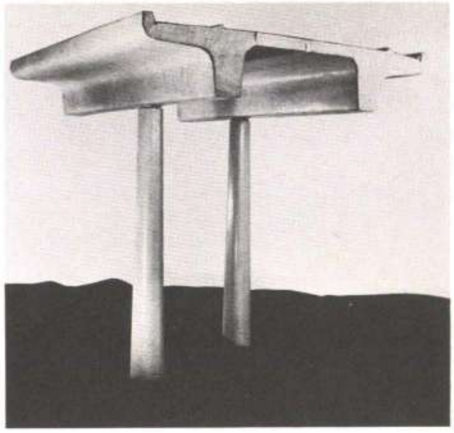


Fig. 7
Plesner's Principle (see also Tunnard & Pushkarev, *Man Made America*.) The view of the sea is considerably better than the minimum shown here. The 900 mm viewpoint implies a very low vehicle.

Fig. 8 (right)
Cardboard model made to study the effect of the curved deck soffit. These columns are more rounded than the ones used in the actual bridge. (Photo: Harry Sowden)



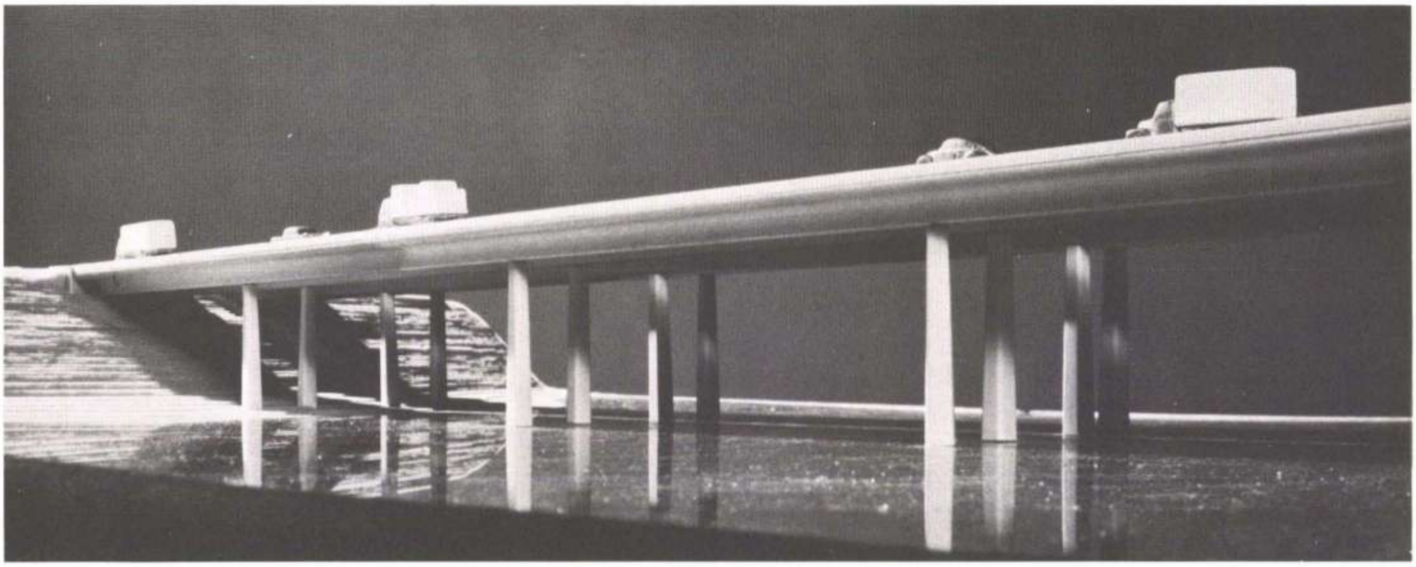


Fig. 9
Presentation model showing the second bridge. (Photo: Henk Snoek)

The architect who worked with us on this job was Ulrik Plesner. He came on to the job after the basic structural idea was already more or less there so that the architectural contribution is more easily recognizable than usual.

Ulrik had two basic ideas. One was that the footway of the bridge should be depressed so that the top rail of the bridge could be below the horizon in order that people driving on the road could see the edge of the sea over the solid part of the parapet. This means that the kerb is an upstand between the footway and the carriageway. There was of course a certain amount of discussion with our clients and the Department of the Environment about this, but we were able to persuade them that for this particular road it was an acceptable solution.

The other idea was that the corners of the deck should be rounded to give the effect of one integrated structure rather than of a collection of beams and slabs and we had a lot of argument about this. In particular there was the question of how much the bottom corners of the beam should be rounded because aesthetics said a lot and engineering said not too much.

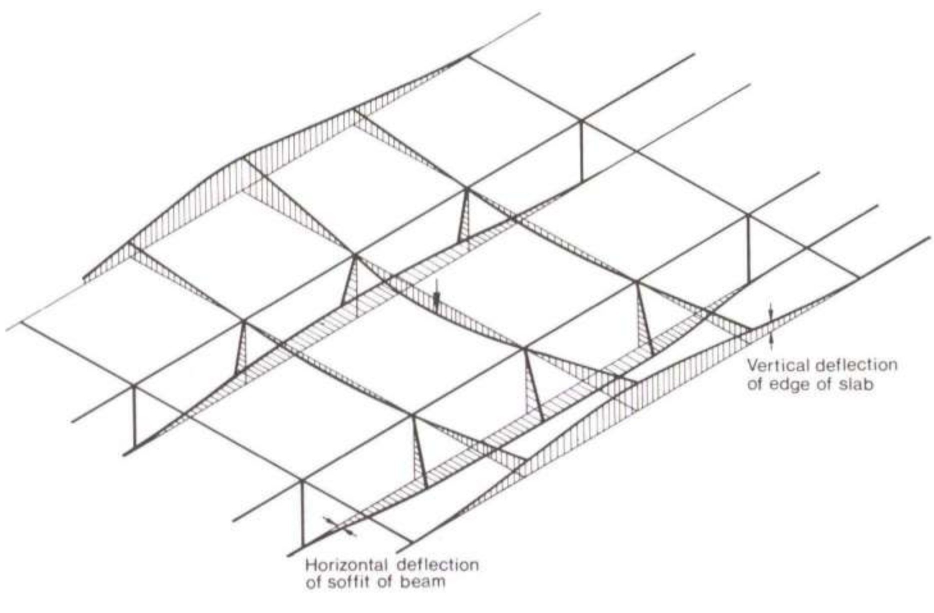
In order to find out whether what engineering thought was all right also looked all right, we made a simple cardboard model of a slice of the bridge in the drawing office and the final decision about what was satisfactory was based on this.

This model also showed us something else. Originally the columns were conceived as square in plan but when we made up columns to match up to the model deck we saw that they were basically incompatible with the curved cross-section and that we would have to have rounded columns. Circular columns did not work nor look right. We then tried a super-circular shape which was still too rounded. The eventual shape is a super-circle which is, from an engineering point of view, not all that different from a square with rounded corners but looks infinitely nicer in practice because the way the light falls on it is surprisingly subtle.

The abutments have been kept very simple so that the valley slopes continue without much interruption and the bridge seems to spring from the banks.

When we discussed surface finishes we came to the conclusion that all the surfaces except the edge of the bridge would have to be smooth because we could not justify doing anything

Fig. 10
Deflections of a section of deck loaded by a single point load. The beams are as if supported continuously by bearings which allow horizontal movement and prevent vertical movement; this is done simply so that one particular action of the deck can be seen without the complication of the vertical deflections of the deck as a whole.



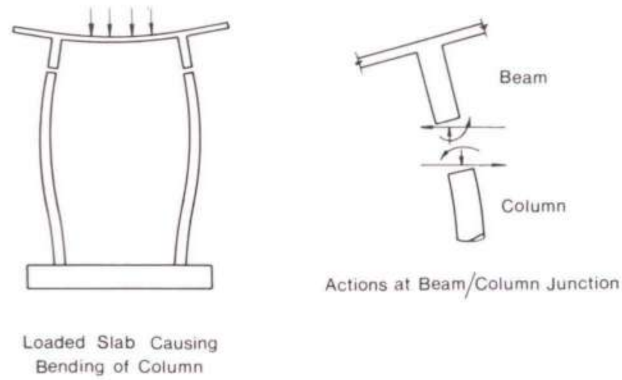


Fig. 11
Interaction of deck and columns; the longitudinal and transverse actions of the deck are eccentric to one another. There are spherical bearings between the columns and the beams, but friction stops them rotating in the transverse direction. Theory aside, it doesn't matter if they do.

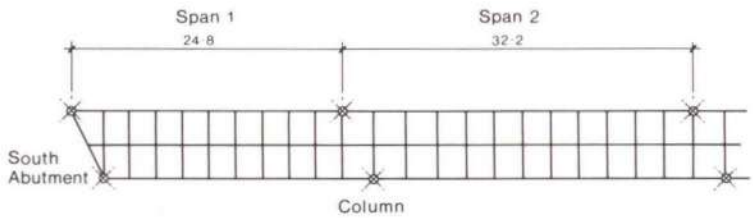


Fig. 12
The rather crude grillage used for analysing the deck slab. The transverse bending stiffness of the columns is simulated by torsion springs. The longitudinal stiffness of the columns is not significant.

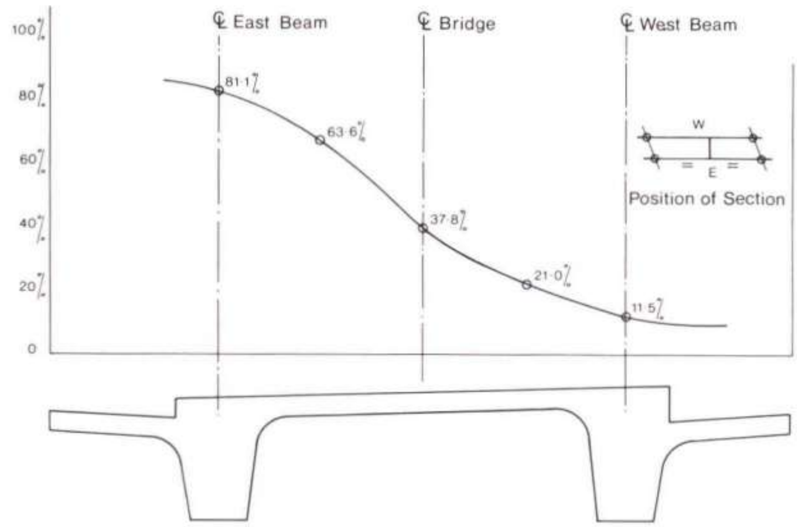


Fig. 13
Percentage of total bending moment in span plotted against transverse position of point load. The values on the centrelines of the two beams do not add up to 100% because the bridge is skew. See the diagram for position of section.

special to them. The edge of the bridge, where the streaking is most likely to be a problem, is cast on afterwards and gives a lot of re-use of formwork so we felt able to put vertical grooves in it.

Structural action

Although the bridge structure is very simple it is necessary to look carefully at the structural action.

The effects of live load on the deck slab are interesting. Suppose we load the bridge in between the main beams with a live load of limited length and neglect for the moment the effect of the columns and the longitudinal deflections of the main beams—we could consider the beams continuously supported by rollers and hinges to allow unrestrained sideways movement.

The slab will deflect under the load and the beams will twist and rotate so that the slab will not be fully restrained by them. At either end of the loaded length the slab will be acting as a restraint on rotation of the beams. The loaded portion of slab twists the beams and the unloaded portion tries to untwist them again. If the whole length of the bridge were loaded and if there were no diaphragms at all, the beams would exercise no restraint on the slab and the slab would span between the beams as if simply supported. For a very short loaded length, on the other hand, the beams will almost fully restrain the slab.

We had to ask ourselves whether in this situation it was safe to apply the HA load in the normal way—decreasing with increasing loaded length—and we came to the conclusion that it was conservative to do so.

For this particular bridge when two spans are loaded, the sagging moment in the slab is a maximum. For short loaded lengths (including the HB vehicle) the slab is virtually fixed at its supports and this gives the maximum hogging moment in the slab. The torsion in the beams is small for long loaded lengths and greatest for short loaded lengths.

The structural analysis of the deck falls into three parts:

(1) An investigation of the transverse action of the structure to take account of the interaction between the columns and the deck and of the effect of the eccentricity between the longitudinally and transversely acting parts of the deck.

(2) An analysis of the deck in which the slab and beams are replaced by a grillage and the bending flexibility of the columns in the transverse direction is simulated by torsion springs. This is used to analyze the deck slab and to find the torsion in the beams and also to assign loading to the beams.

(3) A longitudinal analysis of the beams using loadings from the previous analyses.

Initial rough calculations showed that the beams could be treated as if they were independent under the dead load and prestress, so that the grillage analysis was only required for live load.

Construction

The bridge is just one part of a road contract for the construction of Stage V of the South East Northumberland Spine Road. The County Surveyor is the engineer under the contract and we have no contractual status but simply act as advisers to him. This is a situation we do not like. It makes it more difficult for us to act effectively if something is going wrong (as it has on this contract). However it is the arrangement that our local authority clients seem to prefer. In similar circumstances the DoE tend to go for a 'two engineer' contract.

The contract went out to tender in August 1972. The successful tenderer was Tarmac Construction and the contract started in October 1972. The contract period is 24 months but it is running extremely late and will probably finish in April 1975.

The first problem in building the bridge was the access. The contractor originally hoped to build a bund of shale halfway across the valley to construct three spans of the bridge, then to remove the bund and to build another one across the other half of the valley to construct the other three spans. This was not allowed by

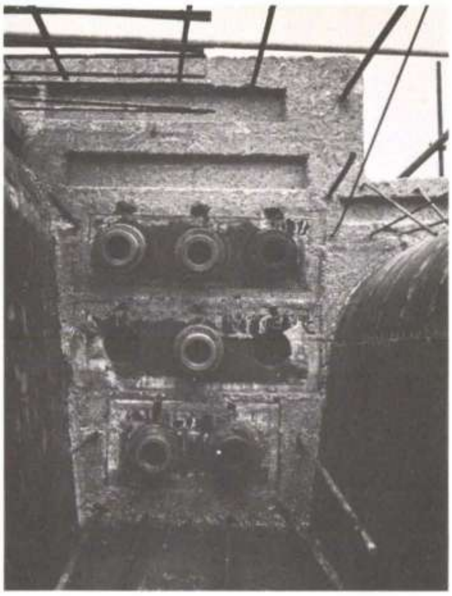


Fig. 14
Construction joint showing cable anchorages. (Photo: Harry Sowden)

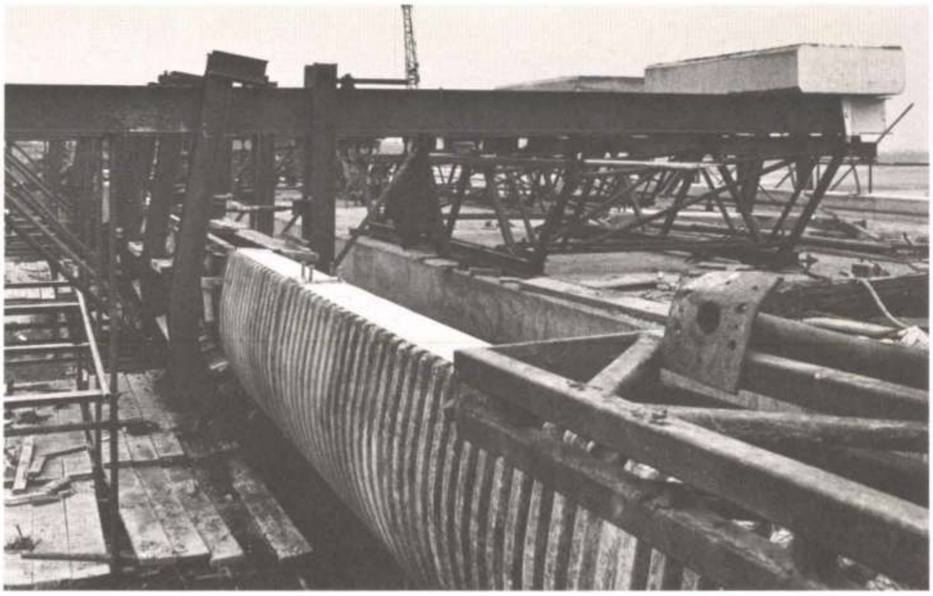


Fig. 15
Ribbed edge & steel frames for edge forms.
(Photo: Harry Sowden)

the river authority on the grounds that it blocked too much of the valley. (Subsequently construction of a barrage was started upstream of the bridge and this completely blocks the valley anyway). The contractor had also hoped to support his falsework on the bund and this raised problems of excessive settlement due to consolidation of the silt during construction of the deck.

In practice a bund was built for the first two spans from the south bank and access across the rest of the valley is by a temporary jetty which also supports the falsework for the deck. The jetty consists of steel H piles supporting steel beams and a timber deck.

The problem with supporting the deck falsework on the bund was that when the deck was concreted, the thick silt layer would settle and could cause cracking and misalignment of the deck. In order to reduce this effect, the bund was built with a 2 m surcharge which was removed after six weeks. The settlement was monitored and the surcharged bund settled up to 98 mm. When the surcharge was removed the monitoring points were disturbed but the recovery was believed to be about 30 mm. We estimated from this that the main beams of the second span would suffer a permanent deformation of about 10 to 15 mm between the time they were cast and the time they were stressed.

Falsework settlement is more of a problem with prestressed concrete than with reinforced concrete because there is very little reinforcement in prestressed concrete and therefore there is nothing to take the tensile stresses due to settlement until the concrete has gained enough strength for the cables to be stressed. The unavoidable settlements due to tightening up of the falsework during concreting (in this case about 10 mm) should occur while the concrete is still plastic if the pour has been properly planned. The problem in that case occurs at the junction between new and old concrete. Settlement, such as foundation settlement which occurs more slowly, is more likely to cause cracking generally and this was what we were worried about.

After the casting of the first span the bund settled 9 mm and since the silt layer under the second span is much thicker, we expected considerably greater settlement and were fairly concerned about it. However the second span only settled 5 mm during the three days before the deck could be stressed. This was what the contractor had said it would settle, which shows that it is better to be lucky than clever.

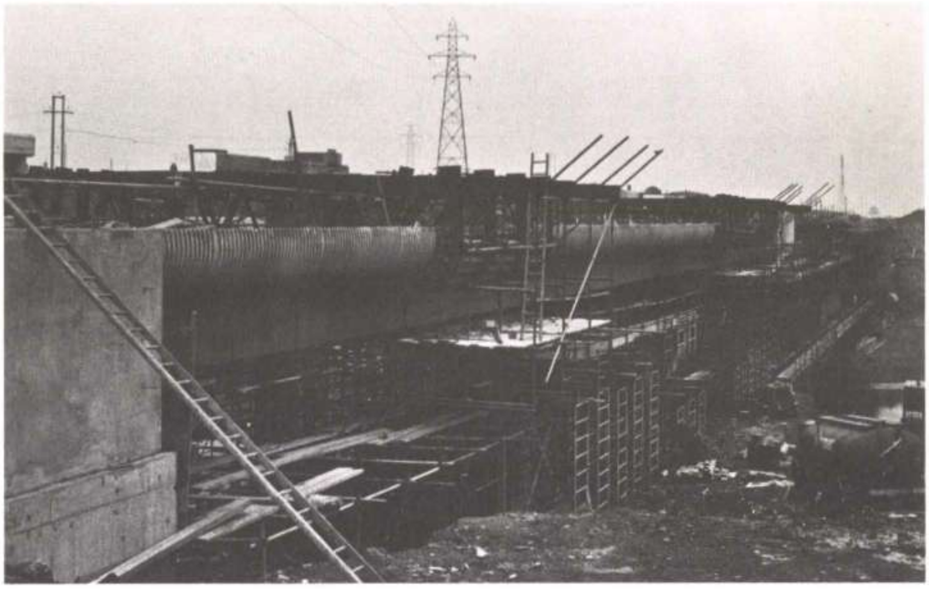


Fig. 16
From the south bank. The bridge looks sawn-off without the two-rail parapet which is going on top. The bank seat will be buried. (Photo: Harry Sowden)

Fig. 17
Part of the edge has been cast and the frames for the edge shutters can be seen.
(Photo: Harry Sowden)

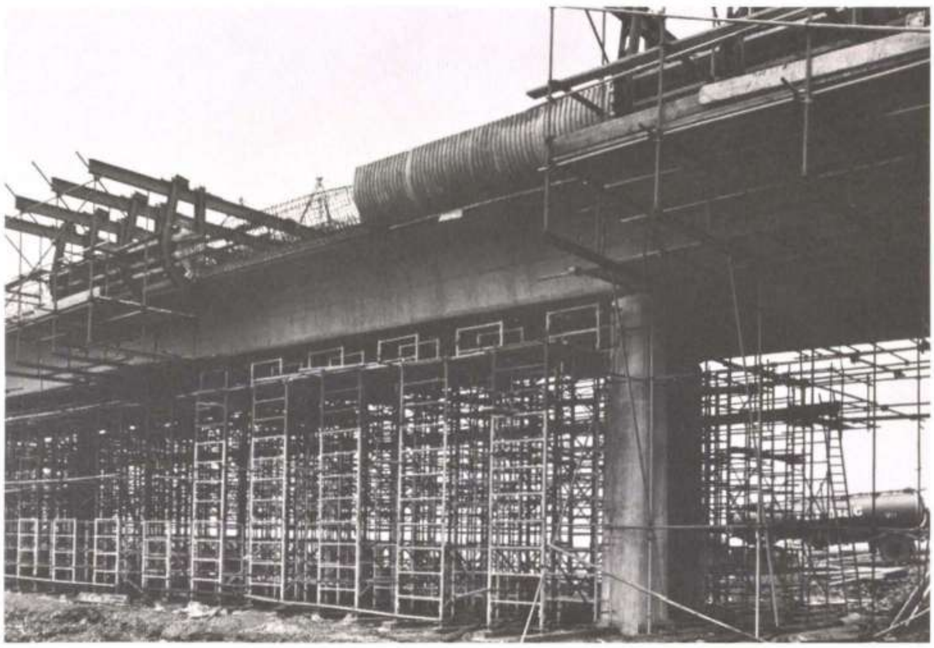


Fig. 18
From the north bank. Stage 4 has been stripped and the shutters for Stage 5 are partly ready. There is a fine contrast between the industrial landscape above and the peaceful valley below. (Photo: Harry Sowden)

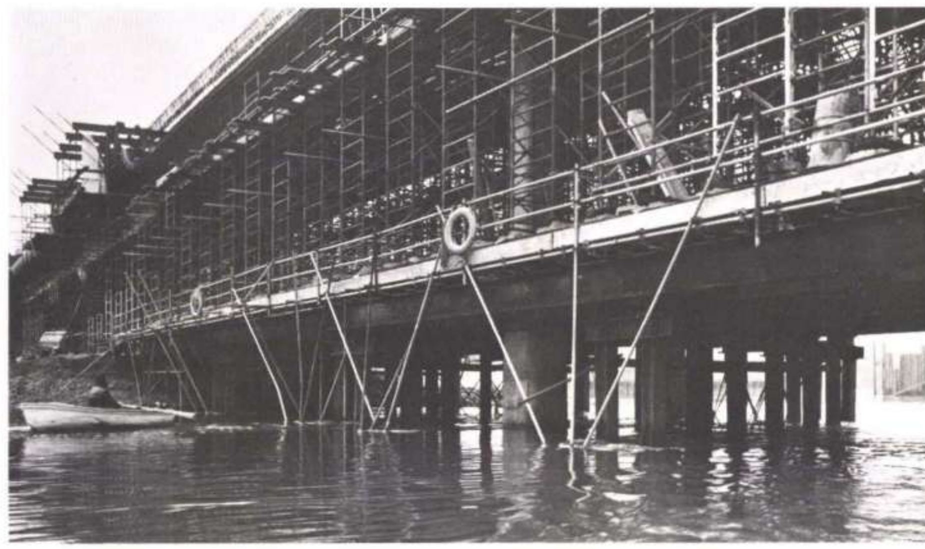


Fig. 19
The temporary jetty with the bund to the left. (Photo: Harry Sowden)

Pier 1, which is founded directly on rock, was excavated within the bund, pumping continually. The foundations for the other four piers were constructed inside sheet pile cofferdams. Pier 3 conflicts with one of the piers of the old waggonway. The excavation for the pile cap revealed two caissons of about 3.5 m diameter with cast iron casings and filled with colliery waste. The pile arrangement had to be revised so that there were vertical piles inside the caissons and raking piles outside.

The excavation for the north abutment and the necessary PFA filling were carried out before piling. Rock points were slotted and welded to the piles to help them to penetrate the inclined faces of the buried cliff. These piles were jetted in to preserve the protective coating in the highly corrosive shale bank. The south abutment is sited below the 132 Kv cables. The piles had to be made of two lengths which were welded together after the first length had been driven.

All the columns were cast in one form which was constructed of timber with a fibreglass lining and made in two halves joined across the diagonal. The columns were each cast in one pour.

The deck formwork consists of plywood lined shutters, with hardboard curved to form large radius corners, and small radius corners formed out of solid timber pieces. The shuttering is carried by a grillage of timber joists supported on shoreload falsework. Special precautions had to be taken to minimize the effects of settlement, particularly at the junction between a pour and the section previously cast.

Each stage of the deck was cast in one continuous pour of 230 m^3. During the first couple of pours great difficulty was experienced with the ready-mixed concrete which was very inconsistent. Due to the difficulties with the concrete mix and the shuttering not being as firm as it might have been, the finish to the first two spans has needed a good deal of remedial work.

The parapets are being constructed after the main deck is stressed and a span behind in order to get a good edge line. The forms are made from fibreglass in 6.4 m lengths with timber stiffening. They are held in position by steel frames travelling on the deck.

The prestressing system is a BBRV system using 55 or 7 mm wires in each duct. The deck was designed for span by span construction in which each stage is cast and fully tensioned before the next stage is constructed. The cables for each successive stage are connected to those already tensioned by means of couplers.

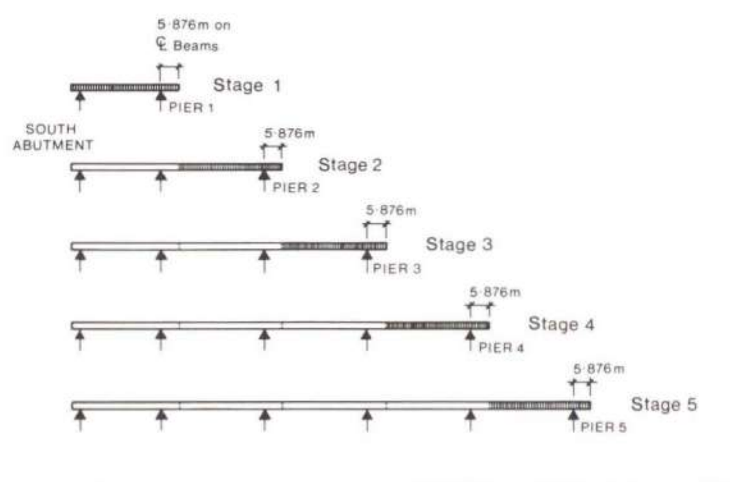


Fig. 20
Deck construction sequence; each stage is cast and fully stressed before the next stage is constructed.

When span 4 was being stressed on 14 September, the coupling connecting one of the cables to its mate in span 3 failed at a load of 230 tonnes. (The working load is 244 tonnes). There was a sound like a pistol shot, a violent tremor, and a large chunk of concrete over the coupler spalled off. The failure was due to a screwed connection having too few threads engaged, and we are still not quite sure how this happened.

At the time of writing, the cable is being replaced and proposals for repairing the damaged beam are being considered.

Apart from the coupler failure our current concern is whether or not the contractor can get the cables for the remaining two spans. We are hoping that these issues will have been resolved satisfactorily by the time this is published.

The University of Libya, Tripoli Phases 1 and 2

Colin Wade

Introduction

Although this article is primarily concerned with the first two phases which are under construction, a brief mention will be made of the subsequent phases and general scope of the works. Except for the general statistics regarding Libya, a deliberate attempt has been made to avoid too much technical data, details and problems associated with the buildings themselves; instead the approach has been to give a general description to show the size and extent of the contracts which to date make up one of Arups' largest jobs.

Libya

Although Libya has gained international prominence over the last few years it is probably a relatively unknown country to most people and it is worthwhile to mention here some basic details.

It is a country of about two million people and although split into 10 districts for administrative purposes there are three basic areas: Tripolitania to the west with the capital city Tripoli on the Mediterranean, Cyrenaica to the east with the second major city Benghazi also on the coast, regarded as the 'second' capital, and Fezzan to the south with its major township, Sebha.

Approximately 800,000 people live in the two capitals; of the remainder, even until a few years ago most were either nomads, semi-nomads or agricultural workers.

The area of Libya is approximately 680,000 square miles (about one quarter the size of the USA). About 85% of this is desert, there are no rivers and rainfall is irregular and scanty; the temperature is usually high and often increased by the Ghibli—a wind which blows from the Sahara.

Historical background

There have been many civilizations which have occupied Libya for the past 2,000 years; Phoenician, Punic, Greek, Roman, Byzantine periods, Ottoman dominance, Italian colonization until final independence in 1951, have all contributed to its history. Tripolitania, as its name implies, supported three major Roman cities—Leptis Magna, Sabratha and Oea (Tripoli); Cyrenaica has the mainly Greek city of Cyrene. Leptis Magna and Sabratha which are on the Mediterranean are amongst the best preserved Roman relics anywhere in the world.

Present day situation

From 1958 onwards major oil strikes were made and by 1968 Libya was the world's fourth largest exporter. Oil revenues have risen from LD 14m. in 1961 to LD 594m. in 1971 (the present exchange rate being approximately £1.4 sterling=LD 1.0 Libyan Dinar).

The upsurge in oil production caused many people to leave their occupations as farmers and

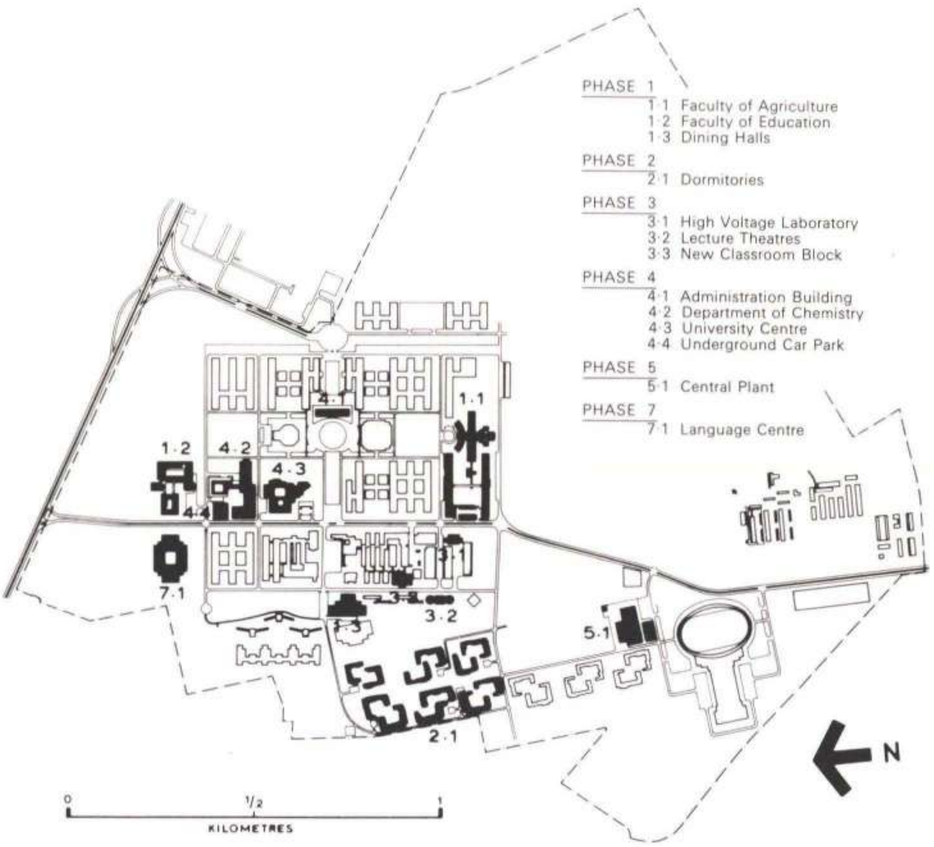
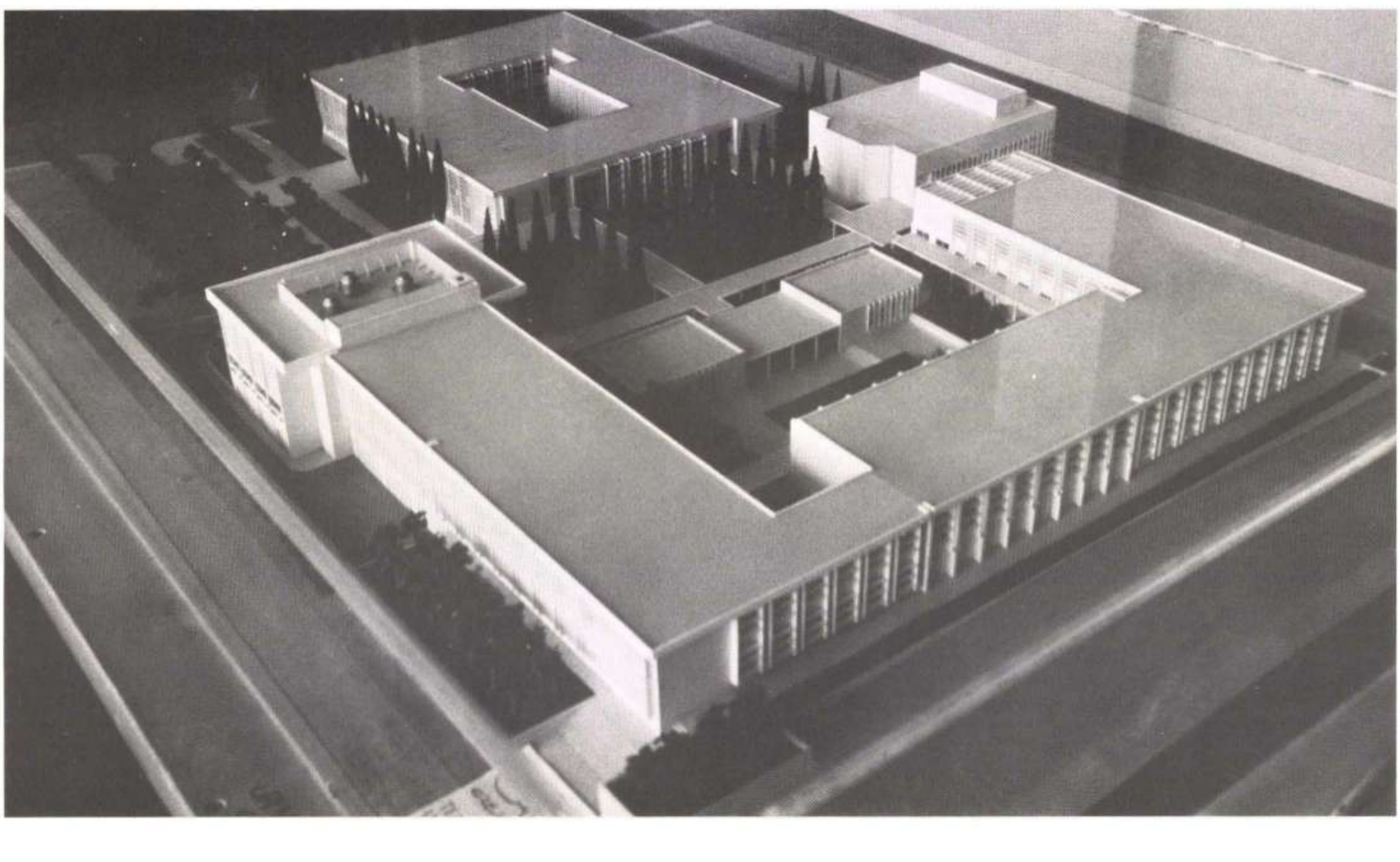


Fig. 1
Site Plan

Fig. 2
Faculty of Education—model.
View from south east



caused a serious decline in agricultural areas. Since 1971 however, more investment in agricultural aspects has begun with massive development schemes. For the 10 year plan from 1973 to 1983 the Revolutionary Command Council allocated nearly LD 800m. for agricultural developments, mainly spread over five zones throughout the country.

The construction industry

The present situation can probably be best described as being in 'frenetic haste' despite labour and basic materials shortages. Most towns are dotted with buildings of all shapes and sizes being erected with varying degrees of skill. The semi-official news sheets publish almost daily reports of multi-million dinar contracts won by local, neighbouring Egyptian and Tunisian, or international contractors.

British consulting engineers, most of whom were firmly established in Libya before the 1969 revolution and who weathered the subsequent difficulties, now have a position of respect for their expertise and experience. There is however little corresponding activity by any British contractors; this is perhaps because in a labour intensive situation there is fierce competition between local firms, the neighbouring countries and some Iron Curtain countries who are chasing 'hard' currency. An exception to this rule is a large German contractor, Philipp Holzmann, who has won large contracts all over Libya and who has obtained part of Phase 3 of the University project.

The universities

It is not surprising in the light of the above brief description of the Libyan economy that the pre- and post-revolution governments embarked on a vast educational programme.

Accordingly in 1964 the architects (then) James Cubitt & Partners were asked to plan and design a full university at Benghazi and a college of advanced technology at Tripoli. The Benghazi project is well under way and parts of it are now fully operational.

Arup involvement

In 1968 the architects asked Ove Arup & Partners to provide a full structural design service plus the necessary services design for the Tripoli scheme which comprised initially a teachers training college. As Arups were not then able to deal with the services an outside firm of consultants, Kenneth Stead & Partners were engaged to do this for us. This arrangement has held up to the present Phase 7 with the exception of the central plant and site distribution duct which is being designed by Building Engineering Division Section B.

Part way through the design, the project was re-named the Faculty of Education and was incorporated in a far more ambitious scheme which became The University of Libya, Tripoli. Subsequently additional faculties were added to comprise Phase 1. The design of Phase 2 which comprises the dormitory complex began in 1971.

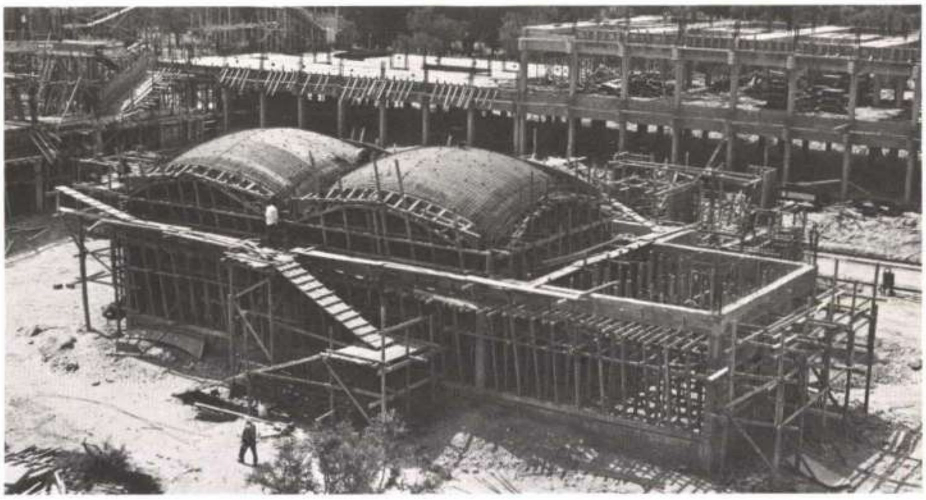
Design considerations

Arups had worked in neighbouring Tunisia and Algeria, but Libya was a new country. Our investigations took in geography, climate and the state of the construction industry and was dealt with by field trips and by liaising with the architects who had a small resident team of site staff both in Tripoli and Benghazi.

Although Tripoli is in a low activity seismic zone a more serious earthquake in 1963 at Barce near Benghazi proved that buildings should be designed to have some degree of resistance. There is no Libyan code for earthquake resistant design and it was decided after more detailed investigations to adopt the SEAOC (Californian) Code using a 'static' analysis. The design guides and standard earthquake details produced early on in the design have been used to obtain a uniform approach for the whole university project especially now that the job is being designed by four 'separate' Arup sections. All design has been done to British Codes and Standards.

Fig. 3
Dining halls – interior at ground floor

Fig. 4
Faculty of Agriculture – Recreation Centre D6 – view from north



The site

The area of the campus (see Fig. 1) is approximately 246 hectares and for those who are familiar with London geography its extremities would extend from Euston Road to Buckingham Palace north to south and from Charing Cross Road to Marble Arch east to west.

A site investigation was made in 1969 by George Wimpey and Co. Ltd. and showed there was a reasonable sand stratum allowing us to design conventional pad and strip footings for all buildings, the water table being about 20m down. Due to the other phases a more extensive site investigation has recently been completed by an Egyptian firm called ECG, and shows a similar stratum.

Brief description of phases

During the period between our appointment and the signing of the Phase 1 Contract, Colonel Qadhafi came to power and because of the political changeover there was a loss of continuity. This has been overcome and Phases 1 and 2 are now well under way on site.

Phase 3 is split into several sub-groups, the major part of which has just begun on site; as well as the buildings shown on the site plan this phase includes a poultry and research station, sheep farm and food technology plant and Stage 1 of the site distribution duct.

Phase 4 is also partly sub-divided and not shown is Stage 2 of the distribution duct. All buildings are well into the final design stage and are out to tender.

Phase 5 is the central plant and was described in *Newsletter 76*.

Phase 6 comprises a marine biology station and, although a relatively small building, is perhaps one of the most interesting in that it is situated away from the Main Site on the sea shore and has a small jetty carrying sea-water intake pipes to the underground aquarium.

Phase 7 is the language centre and is the newest addition to the project. As well as providing 500 audio-visual carrels (individual teaching booths) making it to date the largest of its kind in the world, it will have two full size, fully equipped television studios.

Resident staff

As can be expected with such a large project the site commitments are heavy. The consultants all have their own site staff and to date there are 20 supervisory staff including secretaries. Bob Knight, John Thornton and Paul Duizend are looking after the structural work at present with more arrivals planned as phases start on site.

Phase 1

Description

This phase comprises four distinct groups of buildings in separate areas on the site and is valued at approximately LD 11 m.

Faculty of Education

(Fig. 2) consists of a group of 13 inter-related buildings of one and two storeys with some basement areas. Main functions are classrooms, lecture theatres, studios, library, administration, etc. This is the furthest advanced on site with finishes and services well under way.

Dining halls

(Fig. 3) are contained in a single storey 'T' shaped building with a basement, which will be mirrored in a future stage. Services are being fixed in all areas and finishes, windows, etc., are well advanced.

Faculty of agriculture

(Figs. 4 to 7) This constitutes the bulk of Phase 1 both in cost and size. Out of the six

Fig. 5
Faculty of Agriculture –
General view west from Laboratory Block D1

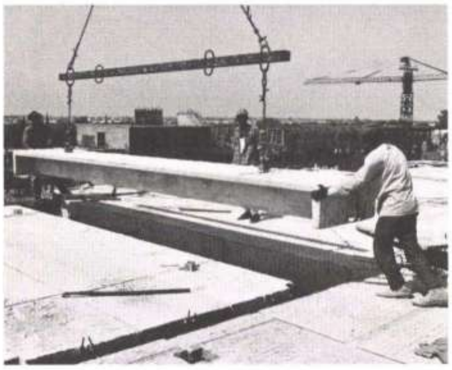


Fig. 6
Faculty of Agriculture – Laboratory Block – precast T beam floor unit being placed

buildings there are three 3-storey laboratory blocks with basement areas. These blocks are very long (136 to 175 m) and to make them more 'manageable' they are split into approximately 28 m long earthquake bays. The remainder are a cluster of four lecture theatres, a 2-storey library/administration building and single-storey recreation centre and store blocks. Structural work is expected to be completed by mid-1975.

All three groups are fully landscaped and the two faculties have covered walkways between the buildings with services distribution ducts beneath them.

High voltage laboratory

This is basically one large enclosure 30 m × 25 m × 14 m high with a folded plate roof. Intermediate floors cover part of the plan to form ancillary rooms and observation galleries. The building's main function is to test large pieces of electrical 'plant' such as transmission tower insulators. All the reinforcement in the structure is welded together to form a 'Faraday Cage'—this prevents any outside electrical interference from affecting monitoring instruments being used on experiments inside the building. Due to client requirements, this building was switched from Phase 3 contract and given to the Phase 1 contractor as a variation order to the contract. Structural work on site is progressing reasonably.

Structural form

All the buildings are of in situ reinforced concrete and are generally framed structures with additional stiffening from reinforced concrete walls in some blocks. Floors are mainly solid reinforced concrete but the laboratory blocks make extensive use of precast T beam floor units with a structural screed. (Fig. 6) There are also large numbers of precast sunbreaker and eaves units for these blocks. All foundations are simple pad and strip footings interconnected with tie beams where necessary. Most walls and partitions are constructed from hollow blockwork, many areas such as external walls and gable ends have been designated as 'structural'. These are being constructed as hollow blockwork reinforced vertically and horizontally with some cavities fully concreted, the whole wall being tied into the structure to give some earthquake stability. The use of standard details has also been adopted to cover the different situations. Other features which are of added interest and have given problems are some interconnected shell roof structures for the recreation centre (Fig. 4); the administration block which has a reinforced concrete dome supported by a stiffened coffered slab with an elliptical plan shape and assymetrical support conditions (Fig. 7); and many long span (12 to 18 m) roof beams on various buildings.

Tendering

In 1970, when Phase 1 went out to international tender, a very limited response was obtained. After some changes to the contract documents requested by the client, the same scheme was sent out to tender again in 1972 and in August that year a contract was signed with an Egyptian firm, Osman Ahmed Osman & Co. This firm has been responsible for much of the heavy civil engineering work on the Aswan High Dam and they work extensively in the Arab world.

The services sub-contractors employed by Osman are a Tunisian company called Ikdam for the mechanical work and a British firm, Balfour Kilpatrick International, for the electrical installations.

Siteworks

Work began in December 1972 and, after a fairly slow start, the contractor has been making good progress and has started all the buildings. The services are also progressing but at a fairly slow rate. Workmanship on the whole is good, even though the contractor is mainly known for his civil engineering work. All the precast elements are being manufactured in a yard set up by the contractor on site. The whole job should be complete by mid 1976.

There are a variety of Arab nationalities being employed as well as the local workmen. Osman is using mainly Egyptian labour especially recruited, including his key site management staff; Ikdam, however, employs mainly Tunisian workers. Nearly all the men live on the university site in specially built camps set up by the main contractor.

Phase 2—Dormitories

Description

The dormitories (see Figs. 8 to 10) consist of six colleges and will accommodate a student population of 2,000. In addition there will be a laundry block and water tower. Each college is made up of two 'houses' U-shaped on plan placed toe to toe. The houses are 4 storey blocks with some basements, enclosing formal gardens and courtyards. This Phase is valued at approximately LD 10 m.

The architectural concept was to take the various rooms and treat them as standard modules. These would then be linked together in different combinations giving a wide variety of configurations. The main repeating elements were the one person and three person study bedrooms. All the study bedrooms have balconies and these have been an important elevational influence (see Figs. 9 and 11). With the exception of the laboratories ventilation is natural and is achieved by extensive use of *brise soleil*, double doors and windows. The finishes are all hard, durable surfaces, using glass, travertine, render, and mosaic-finished precast cladding units.

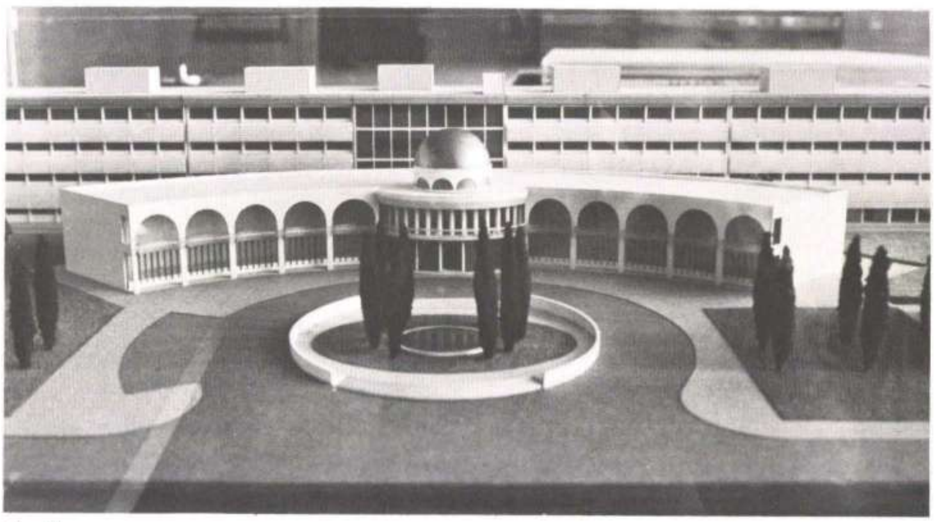


Fig. 7
Faculty of Agriculture – Model – Administration Building D5 & Laboratory Block D1 in background. View from north.

Fig. 8
Dormitories – General view – houses 9 & 10. View from east

Fig. 9
Dormitories – House 1 – North elevation

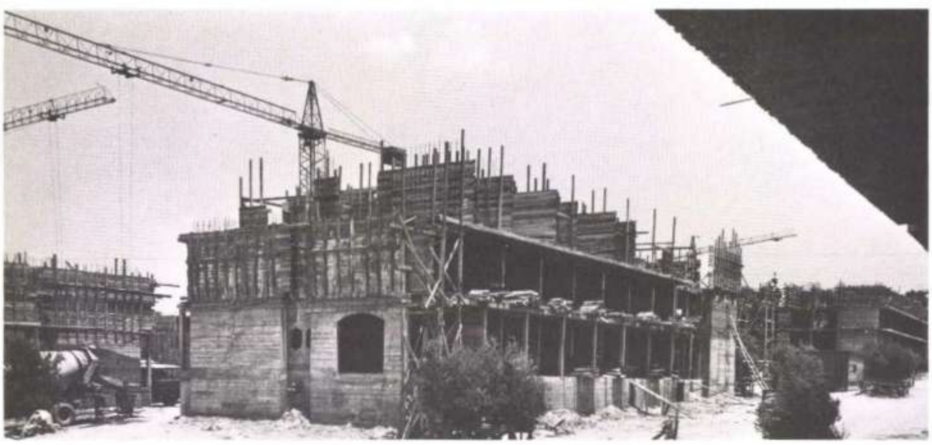
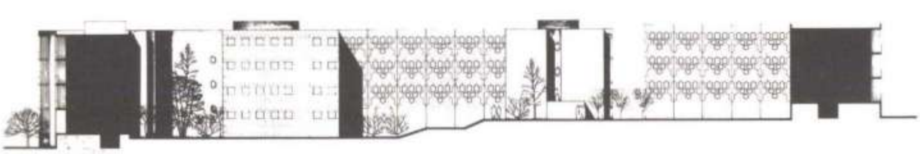


Fig. 10
Dormitories – House 6 – Block 5. View from south west



(a)

(b)

Fig. 11
(a) Dormitories – Typical Elevation-House 3 from West
(b) Dormitories – Typical Elevation-House 3 from East

Structural form and design

To help with the earthquake resistance, symmetry of plan was required and to achieve this the Houses were each split into between 4 and 6 earthquake blocks. The structure is very simple; flat slab and shear walls, including some decorative arches at ground floor level, all carried on strip footings. The soffits of the rooms except for external balcony and similar areas have been shuttered using woodwool slabs as permanent formwork.

Tendering

The contract was let in February 1973 to a local firm, Ben Sassi Contracting Co. This firm has been built up in one generation and has a wide and varied range of business interests both in and outside Libya as well as being a building contractor. The founder is still in complete charge of all aspects of the business.

At the time of writing Ben Sassi is carrying out all the services installations but has been considering sub-letting some aspects.

Siteworks

Work began in March 1973 in an impressive way as all 12 houses were begun simultaneously and a total of 13 (12 of them brand new) smallish size remote control tower cranes were erected—one for each block. Four of the blocks are being built using steel pan forms and props and the remainder using traditional timber boards.

There have been difficulties in getting the work up to the required standard but building is now continuing at a reasonable rate. Services installations are only just beginning and at the time of writing one house is structurally complete. The structural work should be finished by mid-1975 and the whole job by mid-1977.

The labour force on this contract can be said to be truly international with workers from at least 17 countries all around the Mediterranean, and, as for Phase 1, they are all housed on the site.

Credits

Architects:
James Cubitt, Fello Atkinson & Partners

Quantity surveyors:
W. J. F. Tillyard & Partners

Photographs:
Colin Wade

Central Hill Lambeth

Peter Buckthorp
John Morrison

This article is based on the paper by Peter Dunican, Fred Butler and the above authors, which appeared in the November 1974 issue of The Structural Engineer

Introduction

Early in 1964 we were asked by Lambeth Borough Council to design a block of flats in Lunham Road, Central Hill. This block, which was subsequently named Pear Tree House, was the forerunner of a long and interesting yet sometimes frustrating period of design. The design of this single small block of flats was to give a glimpse of the problems which lay ahead when Central Hill itself would be developed.

At the time of Pear Tree House, the total development (Figs. 1 & 2) was just a planner's dream, however 374 housing units, garages, a doctors' group practice, a district heating boiler house, an old peoples' day centre, a community centre, and a nurses' hostel were envisaged.

The Central Hill site is in a bowl-like depression bounded by Central Hill, Highland Road, Lunham Road, Hawke Road and Roman Rise and has a gross area of 6.02 hectares. Being a sloping site, superb views to the north are offered and on a clear day St. Pauls, Westminster Abbey, the Houses of Parliament and Highgate Hill may be seen. The development was consequently very attractive and would make a substantial contribution to the housing programme for the area. There was good reason why the site had not previously been redeveloped and related to the slopes. The general development of the area commenced around 1850 when Central Hill was built upon as a cross-town route when the Crystal Palace was moved from Hyde Park and the railway opened at Gipsy Hill. Ribbon development for the prosperous middle class started along Highland Road and Lunham Road around 1874 and was completed around 1890. The houses were built at the top of the slopes, their large gardens encompassing the steep gradients. The Victorians knew when to let well along!

In the 1930's the GLC built further down the hill towards Streatham, no doubt due to the pressures upon them for new sites, yet a visit today will show that this development of 2-storey terraced houses was not entirely successful, even though this was on the flatter slopes. At that time, however, Central Hill was allowed to sleep on a little longer undisturbed and later no doubt the stories of the treacherous ground at Sydenham Hill and some 2f the projects that came to grief on the southern side of Central Hill kept the developers at bay.

Even before the formation of the new London Borough of Lambeth in 1966, the old Borough had been buying up such sites with a view to their development. However it is not known whether any engineering appraisal of the likely problems of building on these sites was obtained at that time.

Pear Tree House

The scene is set. The one small block, Pear Tree House (Fig. 3), was to prove to be a relatively simple problem compared with that which was to follow. Our knowledge then of such sites was limited but we knew that Skempton had stated that London Clay sites steeper than 10-12° are potentially unstable. But the site of Pear Tree House was in London Clay as shown by the geological map and the slope was steeper than 20°. We could not purchase the surrounding area and regrade all the land on the uphill side of Lunham Road to ensure a stable site.

Fortunately at that time Civil Defence was in vogue and if a 2-storey bomb-proof basement was constructed for Civil Defence, our problems would be solved, not only from the point of stability but also from cost, since government grants were available for the expense of providing such structures. Thus a rigid 2-storey basement was designed which locally supported the hillside on one side with the flats built in brick above. The temporary works consisted of a 6.1 m high retaining wall formed of 0.6 m diameter bored piles at 0.91 m centres, the soil pressure for the temporary work being specified as 30H (lbs/ft²).

Site investigation and geology

Several years then passed before the Central Hill project proper got under way and fortunately our knowledge of London Clay advanced and we felt more confident in tackling the full site. We commenced with a full site investigation in 1966 in which 20 boreholes were sunk, distributed along five cross-sections approximately normal to the contours (Fig. 4). A simple Casagrande piezometer was installed in each. The investigation was entirely conventional for that time and consisted primarily of obtaining 'undisturbed' samples at intervals in each borehole.

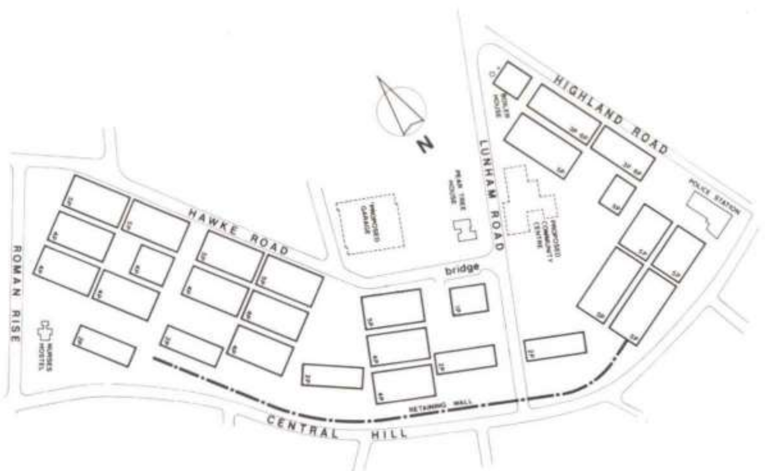


Fig. 1
Development plan

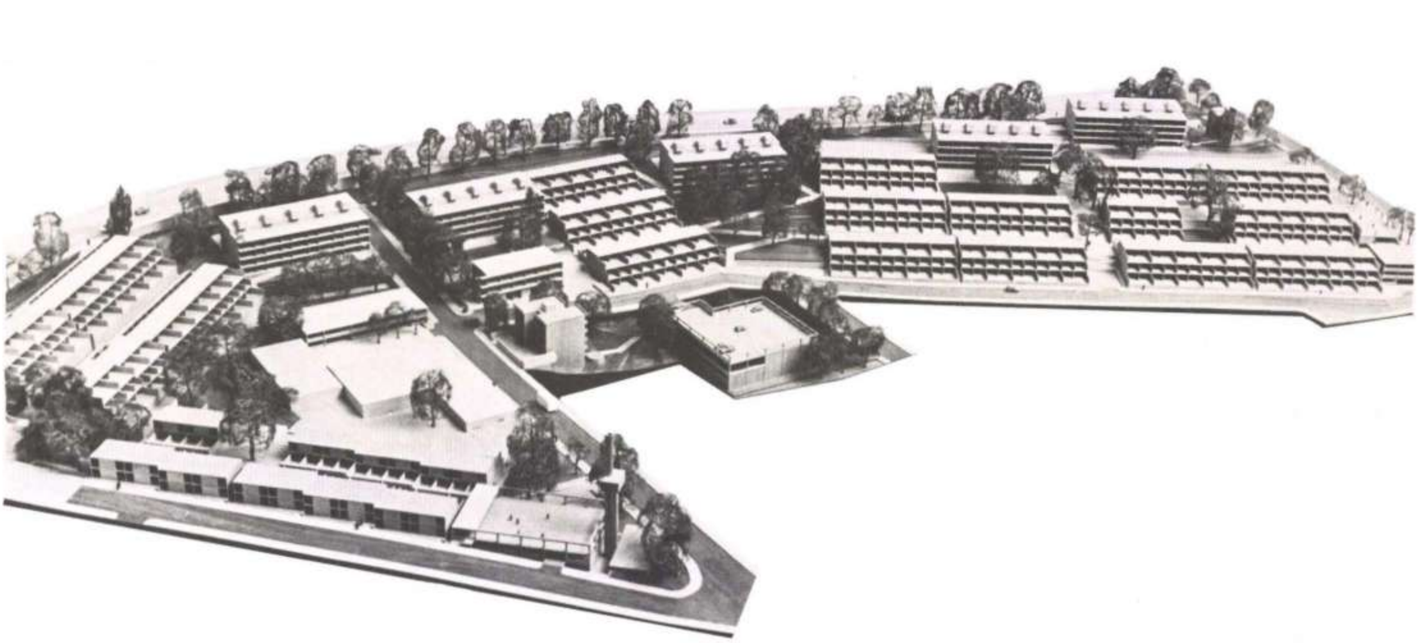


Fig. 2
Model

Regular readings of the piezometers were taken from June 1966. A check in April 1968 showed that the majority of them appeared to be working satisfactorily. All the piezometers showed quite marked seasonal variation, the level rising and remaining high between September and March and falling thereafter until late September.

Classification tests were performed on a large number of the samples. These tests showed that the upper slopes consisted of a silty sandy clay of similar engineering properties as the Claygate Beds. The evidence seemed to indicate that the material was lying comformably as the result of sedimentation rather than solifluction, suggesting that the boundary between Claygate and London Clay was within, rather than to the east of, the site. A very large variation in the classification tests was revealed but nevertheless there appeared to be a trend, contrary to the geological map of the area, that the interface between the base of the Claygate Beds and the upper London Clay was at a level of approximately 90 m Above Ordnance Datum. Peak and residual effective strength tests were undertaken on specimens of Claygate Beds and London Clay.

A simplified cross-section of the hillside was then assumed as a model for stability analysis purposes (Fig. 5). The Claygate Bed yielded an average clay fraction of 36 per cent, residual

Fig. 3
Pear Tree House

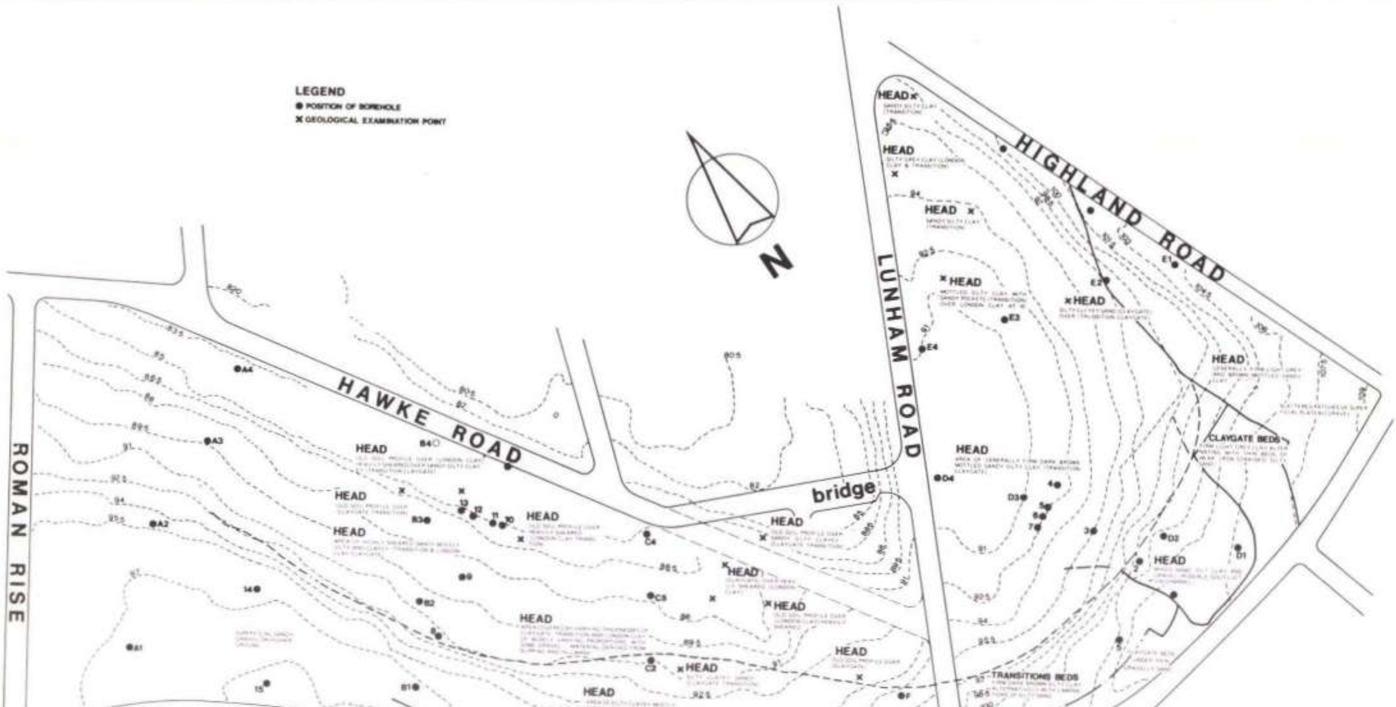


Fig. 4
Plan showing site investigation, borehole locations and surface geology as exposed

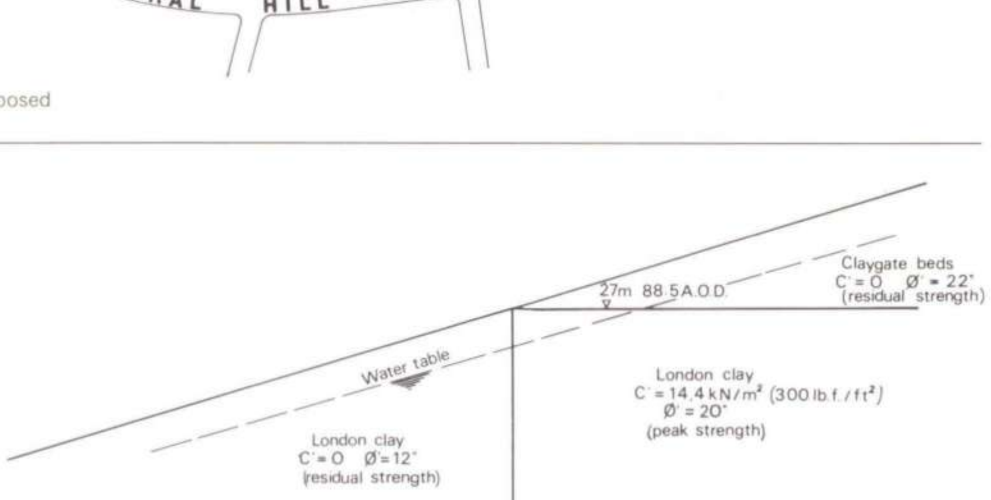


Fig. 5
Initial analytical model

effective stress properties of cohesion, $c'_r=0$ and residual shearing resistance, $\emptyset'_r=20\text{-}22°$: the London Clay was typically 55 per cent with $c'_r=0$, $\emptyset'_r=12°$. In the model the residual shear strength parameters established from laboratory tests were assumed in Areas I and II while in Area III, using the argument that no progressive deterioration could have occurred due to the buttressing effect of the slope, the peak strength parameters $c'=14.34$ kN/m², $\emptyset=20°$ were used. The stability of the existing and intended profile with and without building surcharge was examined using Bishop's circular slip and Kenny's non-circular slip methods. The model had been evolved after discussions with Professor Morgenstern.

Whilst the site investigation data were being evaluated, debate with the architect commenced and the merits of various solutions were considered. Point blocks were mooted but got a very flat reception. The idea of terraces acting as buttresses up the hillside was also turned down. Clearly what was wanted was a contoured, stepped hillside development realizing the full potential of the site. Aesthetically and environmentally this was of course the correct solution and it was up to Arups to solve the stability problem.

Sub-soil drainage trenches

There existed two classical solutions to improve the stability of the site; either the water table could be reduced by installing land drains or the top of the hill could be cut off. The latter solution was not possible without re-routing Central Hill road and consequently the former was proposed, envisaging a total of 1860 m of drainage trenches. The trenches were to be 0.45 m and 0.60 m wide and ran downhill at approximately 15 m centres, to a maximum depth of 6.0 m-7.5 m joining a collector drain along Hawke Road and Lunham Road (Fig. 6). This was the general practice at the time based upon empirical rather than on definitive methods of design. In order to simplify the installation and allow rapid refilling without timbering, the buttress drains consisted of graded aggregate to within 0.91 m of ground level. The remainder could be backfilled with excavated material, provided not less than 1.5 m of aggregate had been placed. Only the collector drains, feeding via silt traps into the public main drainage system, contained open jointed land drain pipe sections for the placing of which planking and strutting were necessary. The grading curve of the aggregate was specified to lie between the limits shown in Fig. 7.

Trenches were to be formed within 0.3 m of the line and within +0 and −0.15 m of the specified invert level. The surplus excavated material had to be removed from the site as dug; stockpiling on the site was not permitted. In order to provide the tenderers with confidence in what they were being asked to undertake, a trial trench was excavated, using two demonstration back hoes manufactured by Poclain and Hymac, to which all tenderers were invited. All attended. This demonstration proved a very useful means by which to ensure very competitive tenders.

The final cost of the drainage was £34,000 compared with the estimate of £38,990 and was completed by early 1968. Advantage was taken of the trench excavations for the drains to obtain a closer impression of the geology of the site.

The design and development of the site was going to take time but the politicians wanted to see some action. It was known that a diversion of Hawke Road would greatly improve the site layout and it was therefore decided that a contract for the construction of Hawke Road Bridge should be let as soon as possible.

Hawke Road Bridge

Hawke Road Bridge (Figs. 8 & 9), a platform rather than a bridge, was constructed in

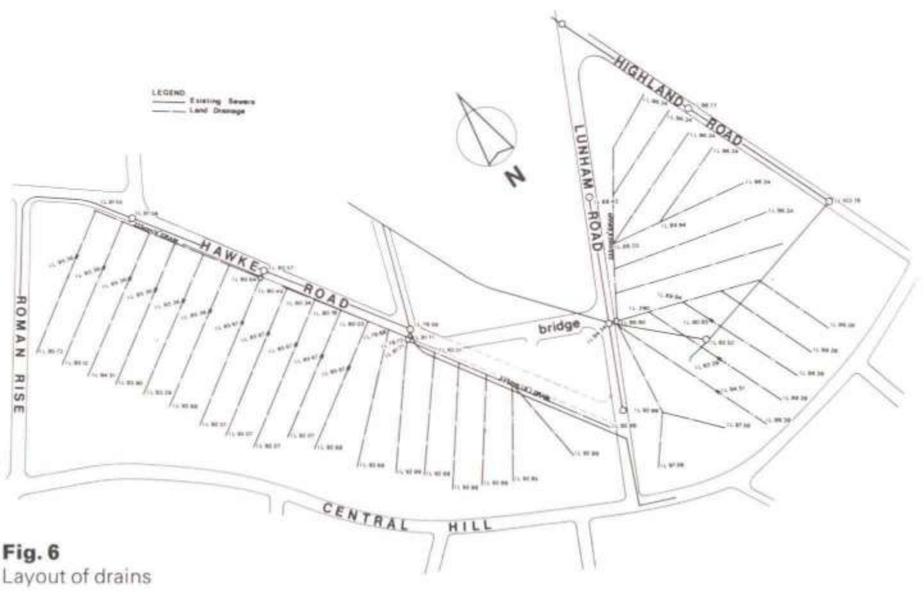


Fig. 6
Layout of drains

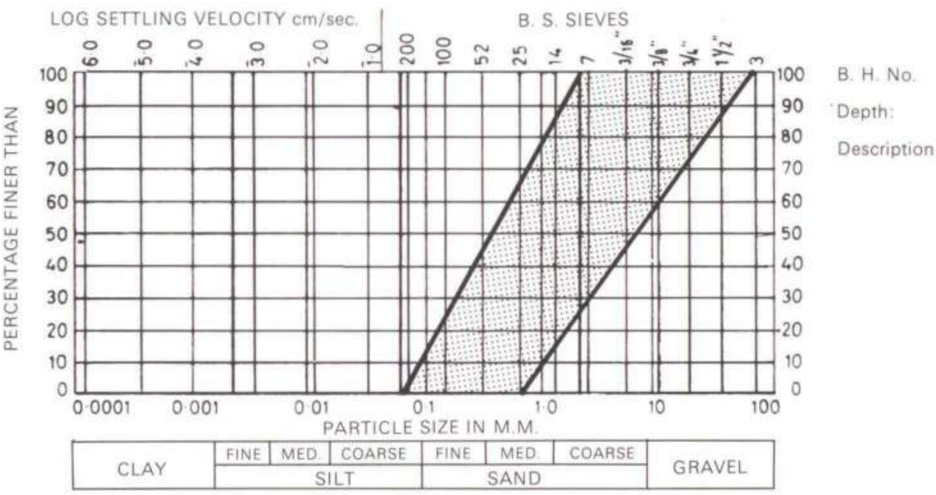
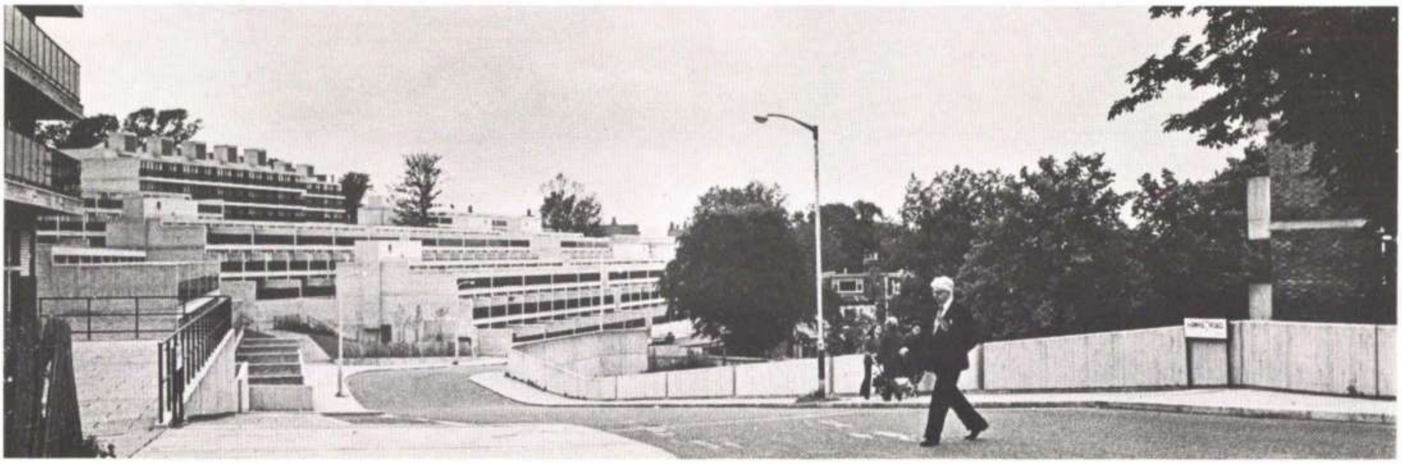


Fig. 7
Limits of grading for drain aggregates

Fig. 8
Hawke Road Bridge



reinforced concrete and founded on piles, the piles to the lower abutment being raked to resist braking loads. The road deck was approximately 43 m long by 14 m wide and was supported by pairs of columns at 8 m centres. The deck was fixed to the lower abutment and freely seated on bearing pads at 1.47 m centres to the upper abutment. The design of the deck was that of a flat slab with waffle soffit. The waffles were purpose-made since no proprietary moulds were available that would support the weight of wet topping required. As a car park was planned beneath, the columns had to be of reasonable size so their tops were freed from the deck by the insertion of bridge-bearing pads. This enabled the deck to move without bending the columns and avoided the large bending moments that would otherwise be induced in a monolithic junction.

The bridge contract was let to John Laing with a very strict specification requirement that all temporary works should be designed for a soil pressure of 60H (lbs/ft.^{2}). The abutment walls themselves were also designed for this pressure as this was felt to be sufficiently conservative to allow for any possibilities that should arise if a slip plane was found. The backfill wedge behind this wall was also specified as pulverized fuel ash as its low density reduced overturning – the PFA was also tied into the back of the wall by projecting reinforcement to try and achieve a gravity wall effect to ensure added stability.

Everything proceeded well with the excavation for the abutment retaining wall taking place in a braced trench – with the excavation in what was thought to be brown London Clay. Then there it was, a huge blue mass of London Clay in the side of the excavation resting on top of the brown clay. Until we expressed our concern, the contractor had been perfectly happy, but now he asked such questions as, why hadn't he been informed of the dangerous condition of the site while tendering? Our reply that this was considered to be one of the safer sections went nowhere towards calming a situation which he saw as a basis for a potential claim.

However the work continued and was completed without mishap.

The efficacy of the drainage system

It had been agreed prior to the installation of the land drains that they should be allowed to function for a minimum of six months to enable them to become effective in lowering the water table before further work was undertaken. Major site clearance operations were carried out immediately prior to the installation of the drains, and virtually all the vegetation was removed. Only after this operation had been undertaken did the full significance of the transpiration effect of the vegetation become apparent. Regular monitoring of the piezometers revealed the unpleasant fact that the land drainage system had contributed little more than to maintain the *status quo*, the ground water levels six months after installation of the drains appeared to vary little from those recorded in the initial site investigation. Monitoring was made difficult because some piezometers were excavated in placing the drains; others were interfered with by vandals even though they were protected by steel caps and padlocks! The stabilized water table, when applied to the initial analytical model, gave unacceptable factors of safety; in some cases apparently below 1. This was clearly not so if only because the slopes had stood for many years and were still standing. Therefore, if this impasse were to be resolved, the physical facts had to be questioned and the assumed model based on them had to be challenged. This was a time of some strain for the design team.

Reappraisal and further site investigation

By now there was tangible evidence that the geology of the site was extremely complex and a further extensive site investigation was carried out. This first entailed the installation of more piezometers set both across the slope between drains as well as down, in order to see the effect

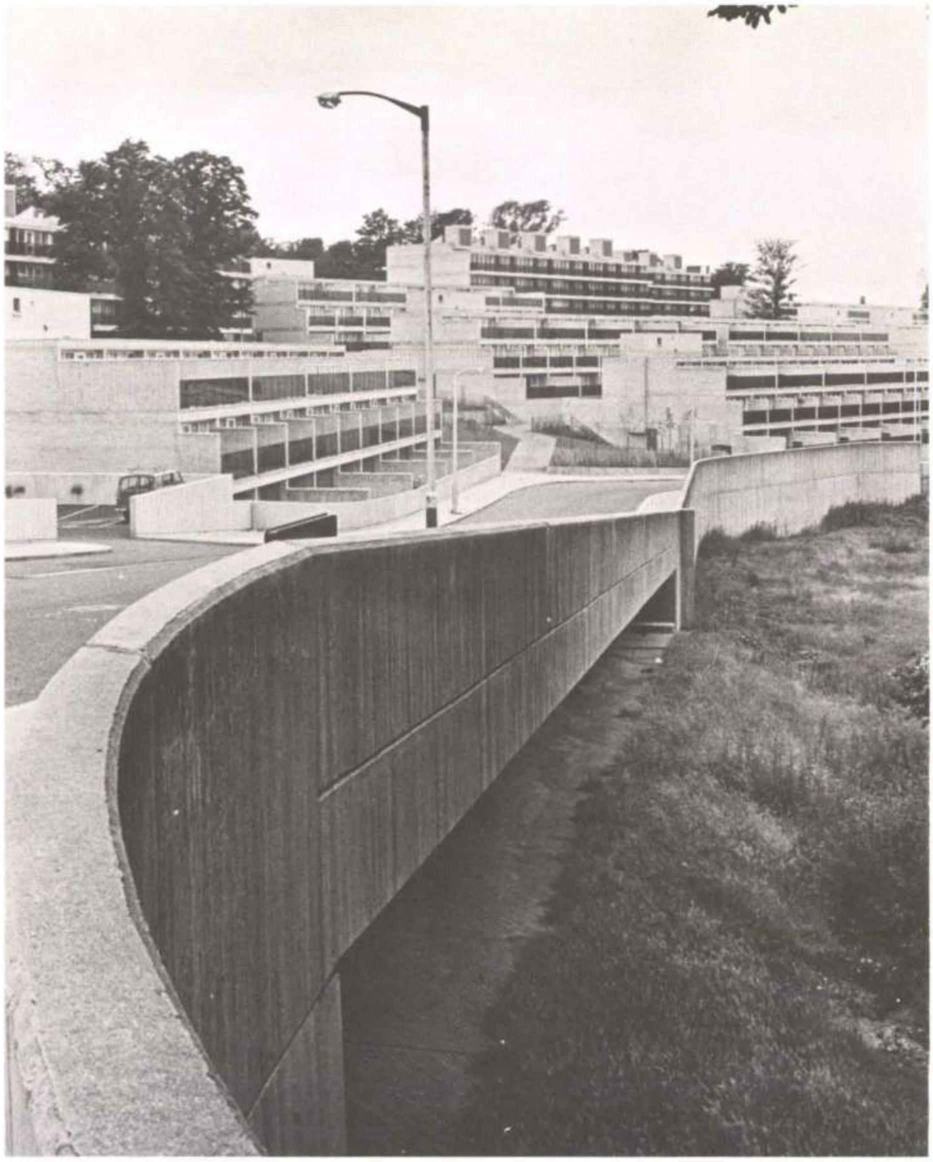


Fig. 9
Hawke Road Bridge

of the drawdown over the slopes as a whole and also to determine the variation of water table between the trenches as it was not known which of the conditions shown in Fig. 10 was actually occurring (it was actually found to be b). Secondly, a complete reappraisal of the geology was undertaken in the light of all the information that had been accumulated by then. It is almost certain that the River Effra rose on the hill, probably consisting of two tributaries each emerging from the gravel capping at the top. The first tributary is thought to have flowed along the south side of Central Hill ridge, the other in a northwards direction down Lunham Road adjoining the main stream near West Norwood Cemetery. During the glacial periods of the Pleistocene, when the sea levels were much lower than now, the Thames and its tributaries eroded deeply into their channels. It seemed likely that the hillside was over-steepened during this period, causing slipping of the London Clay. Towards the end of the glacial periods successive seasonal freezing and thawing of the upper strata occurred, concentrating water near the surface. This had the effect of creating a sludge of very high water content which moved down slope during the thaw periods (solifluction). Much of this soliflucted material is derived from the sandier beds at the top of the hill, together with the gravel from further to the south east. The materials identified on the site were Claygate Beds, 'Transition' London Clay, London Clay, Head comprising land slip colluvium, solifluction deposits and hillwash.

Along with this clearer understanding of the site,

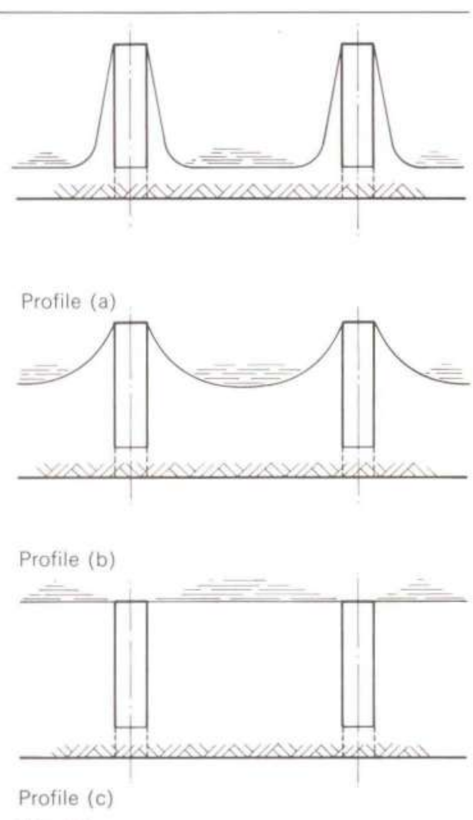


Fig. 10
Possible water table profiles

discussions were also held with Professor R. E. Gibson concerning the application of the residual shear strength parameters measured, as described by Skempton. Following these discussions and additional information from other sources, it was agreed that the direct application of the laboratory values was unnecessarily conservative. Field residual values of $C'r = 3.6\ kN/m^2$ and $\emptyset'r$ = (laboratory value +2°) were probably supportable on the available evidence.

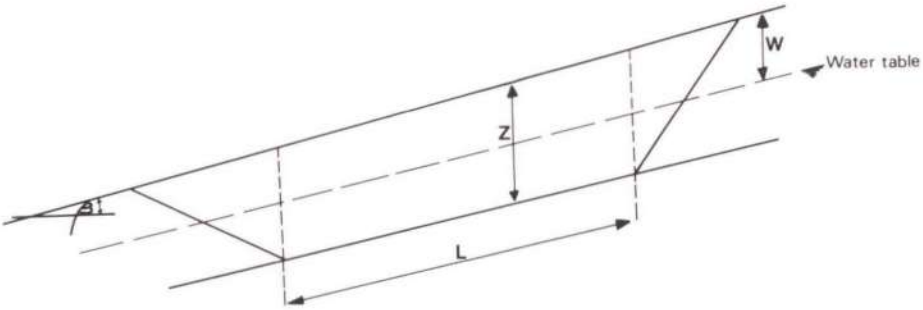


Fig. 11
Stability analysis

Fig. 12
Section through typical block

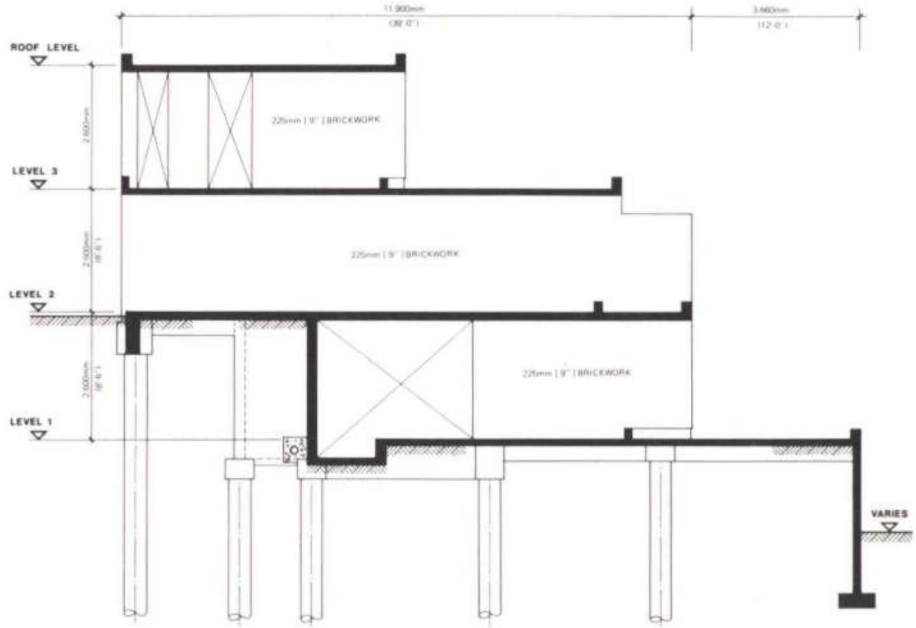


Fig. 13
Typical block – end elevation

In analytical terms the simplified model used in the first appraisal of the site was clearly a substantial over-simplification. Fortunately, in view of the disturbing lack of positive contribution from the drainage system, the geology, which was more complex, was in general also more favourable. The mantle of Head over most of the site, and particularly the steeper slopes appeared to be primarily Claygate-derived material, and therefore had somewhat better residual shear strength characteristics than those of the London Clay assumed earlier. However, our computer program at that time did not facilitate the modelling of a complex multi-layer system and particularly in the case of the regraded slopes, where there were rapid changes in the ground profile, non-convergence problems gave a lot of trouble.

A different form of stability analysis amenable to hand computation was therefore adopted. Its assumptions were closely akin to the probable worst cast of non-circular slip, postulating a plane with active and passive wedges. It was clear that the inhomogeneous nature of the site was such that continuous observation of all the ground work was necessary. Construction could only proceed on the basis of the newly assumed conditions and, where the actual conditions proved to be worse, appropriate measures would have to be initiated on a site instruction basis.

The result of this work led to very stringent site supervision for the main housing contract. This took the form of:

(1) A geotechnical engineer logging all excavations

(2) Design aids prepared for critical zones of site, i.e. length of slip

(3) Adjustment of housing levels

(4) Avoidance of surcharge due to fill, i.e. voids left under some blocks

(5) Use of PFA fill in trenches transverse to slopes

(6) All trenches deeper than 1.83 m to have temporary works capable of resisting 60H (lbs/ft.²)

(7) All blocks piled to avoid surcharge loads.

The model assumed in order to provide site control is shown in Fig. 11.

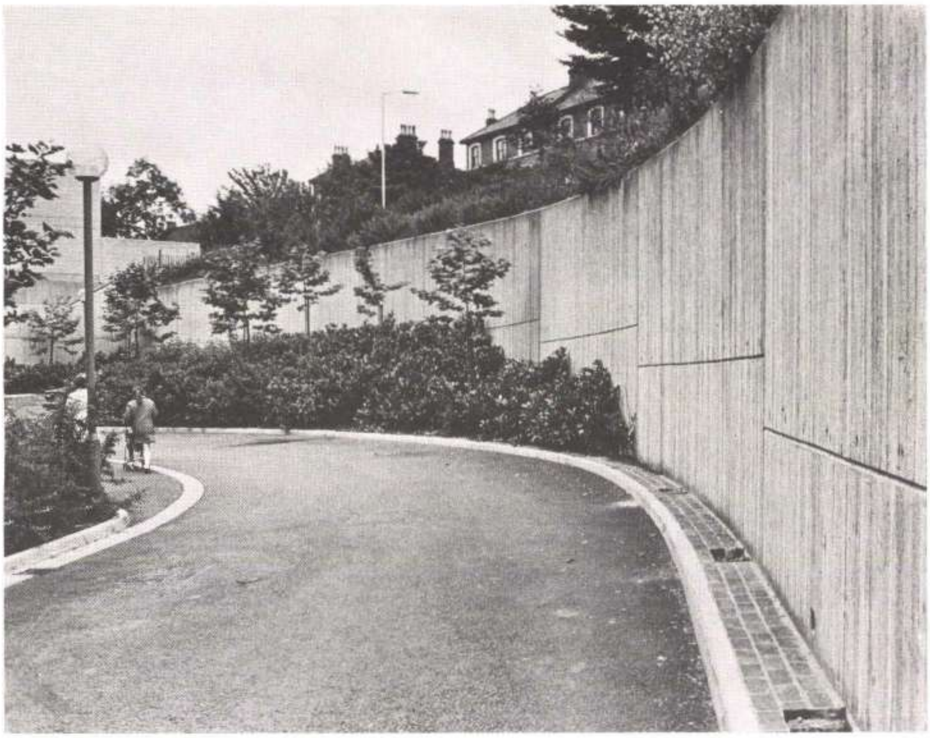


Fig. 14
The main retaining wall

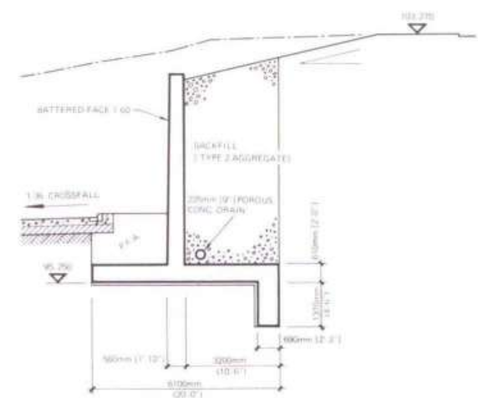
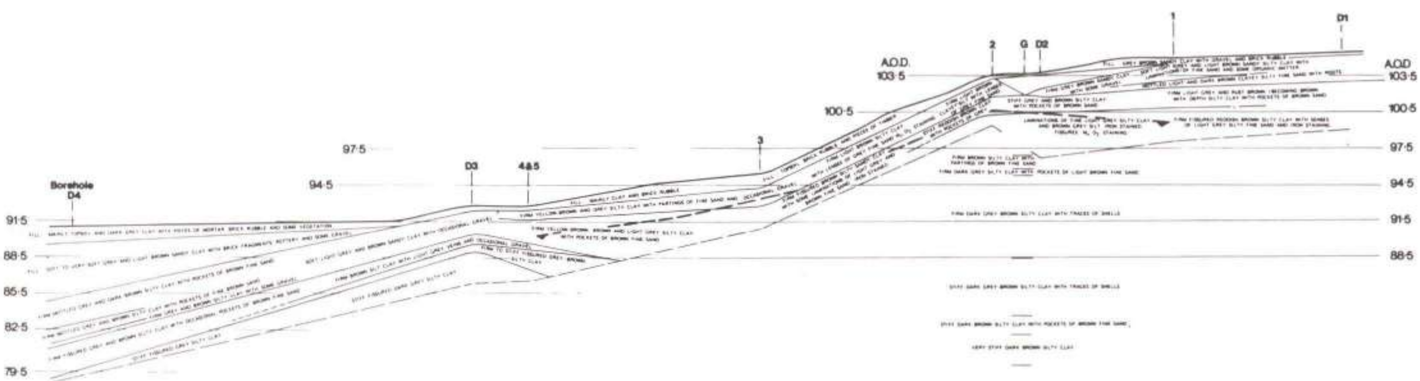


Fig. 15
Section through main retaining wall

Fig. 16
Actual geological cross-section



The housing units

All the structures are simple in design and fall into several distinct categories:

(a) Stepped housing (three, four, five and six person blocks)

(b) Flats with ground level parking (two person block)

(c) One block with flats over shopping and car parking beneath (one person block).

Cross wall construction was generally employed (Figs. 12 & 13) except for the one person block which had a reinforced concrete frame. The grid for all the blocks was 3.9 m and 0.15 m concrete floor slabs were used. To avoid surcharging the hillside with areas of fill, precast ground floor slabs and patios spanning between crosswalls were adopted in some areas of the site. Holes were left for trees! Cross ventilation of the voids, below the ground where the service pipes ran, was allowed for to avoid the risk of gas collection and explosion. All services above ground level were contained in a continuous, longitudinal, walk-through, service duct in each block. All external concrete was boardmarked. Staircases were precast wherever possible.

The main retaining wall

To enable a main service road to serve the top of the development and accommodate the levels of the upper blocks, it was necessary to construct a retaining wall 331 m in length, broken only by a single stair access to Central Hill (Fig. 14). The wall was analysed as a free-standing cantilever, using our standard retaining wall program, with the surface of the backfill sloping upwards to Central Hill at an angle of 15° (Fig. 15). The maximum height of retained material was 4.9 m and the largest section had a stem thickness of 0.6 m base 6.5 m wide and 0.69 m thick with a heel shear key 1.22 m deep. The construction began by reducing the original ground levels to form a V, having side slopes of 15°, the slopes intersecting at the level at the top of the wall to be constructed. The wall was then constructed in a braced trench achieved by driving sheet piles and bracing them by steel frames. The bases and walls were poured in lengths of 7.6 m separated by an expansion/contraction joint. All exposed faces were board-marked. The rear of the wall was drained by backfilling with an aggregate base down to a 0.225 m porous collection drain. Front fill was either PFA or aggregate.

Fig. 17
The boiler house

Fig. 18
Site retaining walls

Piled foundations

The construction programme determined a need for the bored piles to be installed from existing ground level, that is, prior to the benching of terraces. On a sloping site this meant that tripod rather than auger equipment would be necessary. As a part of the control measures to monitor the geological conditions, each pile was to be logged to determine the nature of thickness of Head material and the level of the in situ London Clay.

Initially, six pre-contract piles were installed and tested with the objective of establishing suitable pile lengths for working load of 100 kN 300 kN, 400 kN, 500 kN, 600 kN and 650 kN. The piles were installed in an identical manner to that intended for the contract piles, an additional 90 kN being added to the nominal working loads in order to allow for the top 4.7 m of Head material at the test pile site which, since it was contained within the critical slip zones, was assumed would be non-active in the final construction.

The piling for the 29 blocks and the boiler house was carried out during the period May to October 1970 using eight tripod rigs with 0.46 m diameter temporary casing. Concrete was mixed on site. A total of 15 working pile tests were undertaken to a sequence of loading cycles requested by the district surveyor. All behaved within acceptable performance limits although it became apparent that those in the middle of the slopes consistently gave performances which were indicative of rather lower factors of safety than piles in the upper and lower areas. Although no entirely satisfactory explanation of this observation has been found it is believed that it was due to the fact that under relatively rapid loading conditions, such as those used in a pile test, the Claygate-derived head, which was thickest in the middle of the slopes, produced less effective shaft adhesion than the London Clay-derived Head on the lower slopes.

A contoured plan of the surface of the London Clay was drawn up as part of the final assessment of the stability of the site based on the information gleaned when installing the piles. Similarly, geological sections were drawn, one of which is shown by Fig. 16.

The boiler house

The boiler house is located at the most easterly corner of the site at the intersection of Lunham Road and Highland Road (Fig. 17). The site sloped steeply at this point and it was necessary to set the structure deeply into the hillside. Advantage of this siting together with its large floor to ceiling height was taken by providing covered car parking on the roof, with direct access from Highland Road. Here the solution for Pear Tree House was repeated and the structure is that of a concrete box, the rear and side walls retaining the road and sloping hillside. Internally circular concrete columns support the concrete deck of the car park. Foundations are strip and pad footings throughout. Construction was carried out in sheet piled excavation. The sheet piles were propped from anchor blocks formed by driving secondary sheet piles. Work then proceeded normally until the box was complete. The void between the box and the sheet piles was filled with aggregate before temporary supports and sheet piles were withdrawn.

To complete the boiler house complex, two concrete chimneys, 23.5 m and 26.2 m high, were constructed. Both chimneys were cast in situ and founded on piles.

External works

It was inevitable that in a densely developed sloping site, very many retaining walls were necessary (Fig. 18). An extension of the analysis of the main retaining walls along Central Hill showed that, if constructed wholly in London Clay Head (that is, $C'r = 3.6\,kN/m^2$, $\phi'r = 14°$) with a case 1 water table, a wall over 1.83 m in height would be potentially unstable. Since the walls are employed solely to protect access roads and footpaths, this was considered an acceptable risk even though walls of such a height could occur in combination with adverse soil and water conditions. A plan showing the layout of all retaining walls and identifying possible critical sections was prepared for site control purposes. The exposed excavations for these sections were carefully logged. Had large slipped masses of London Clay been revealed, some measure of protection was to be provided by more drainage and by additional lateral reinforcement to give support at the flanks. In the event, the only wall excavation which showed evidence of this kind was in the abutment of Hawke Road Bridge. In this excavation a substantial slip surface in displaced London Clay was encountered. The excavation was fully planked and strutted and the contractor was restricted to a maximum excavation of 6 m. This wall remained free-standing only in the temporary condition.

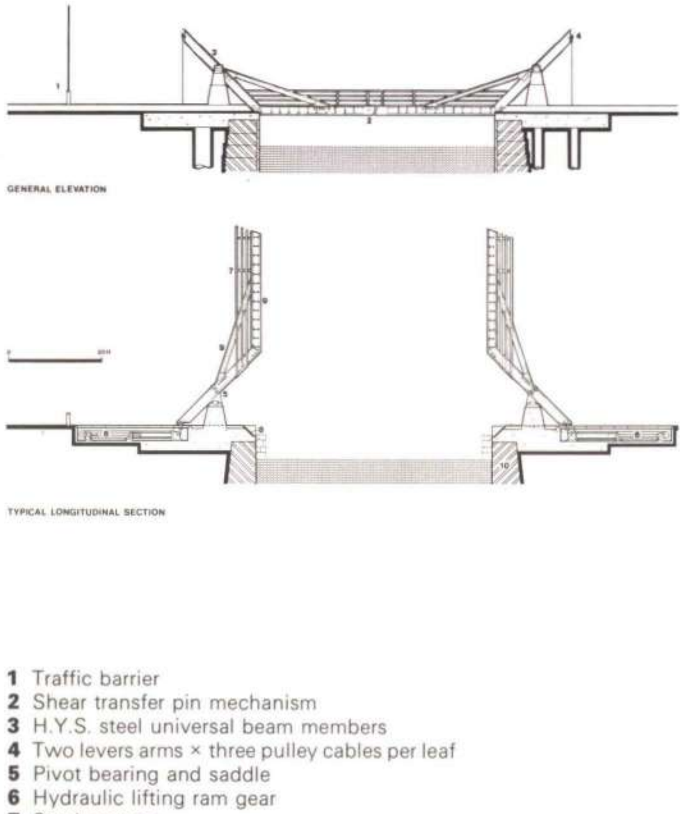
Credits

Architects and Quantity Surveyors:
Lambeth Borough Council

Main contractor:
John Laing Construction Ltd.

Photos:
Harry Sowden

Structural Steel Design Award 1974



1 Traffic barrier
2 Shear transfer pin mechanism
3 H.Y.S. steel universal beam members
4 Two levers arms × three pulley cables per leaf
5 Pivot bearing and saddle
6 Hydraulic lifting ram gear
7 Steel parapet
8 Rubber bearing pad
9 Universal beams
10 Existing wall

St. Katharine Dock Lifting Bridge (designed by Ove Arup and Partners) which has been awarded a 1974 Structural Steel Design Award

Gateshead Western Bypass

Bill Smyth

The Gateshead Western Bypass, which is our first job of its kind, was formally opened by Princess Margaret on 17 October. It is a dual carriageway with grade separated interchanges and is about 8½ km long. The carriageways are mostly two-lane but there is a dual three-lane section in the middle.

As well as the design of the road and the 11 bridges on it we carried out the traffic studies to determine the width of the road and the forms of the interchanges. There were a lot of interesting geotechnical problems and we had to deal with the results of a mine subsidence. Humphrey Wood of Renton Howard Wood Levin Partnership was our architectural adviser on the structures and the consulting landscape architects were Derek Lovejoy and Partners in association with Renton Howard Wood Levin Partnership.

We were appointed to design the road in 1966 by Durham County Council and Gateshead County Borough Council in conjunction. Due to local government reorganization our client is now the Tyne and Wear County Council. The project report was presented in October 1966 but due to lack of money the construction did not start until September 1971. The Bypass was well built by Mowlem and it was opened to traffic last June.

References

(1) RANAWAKE, K. *and others.* Bridges on the Gateshead Western Bypass. *Arup Journal, 5* (1), pp. 2-15, 1970.

(2) LAW, K. Computer-assisted road design. *Arup Journal, 3* (1), pp. 16-17, 1968.

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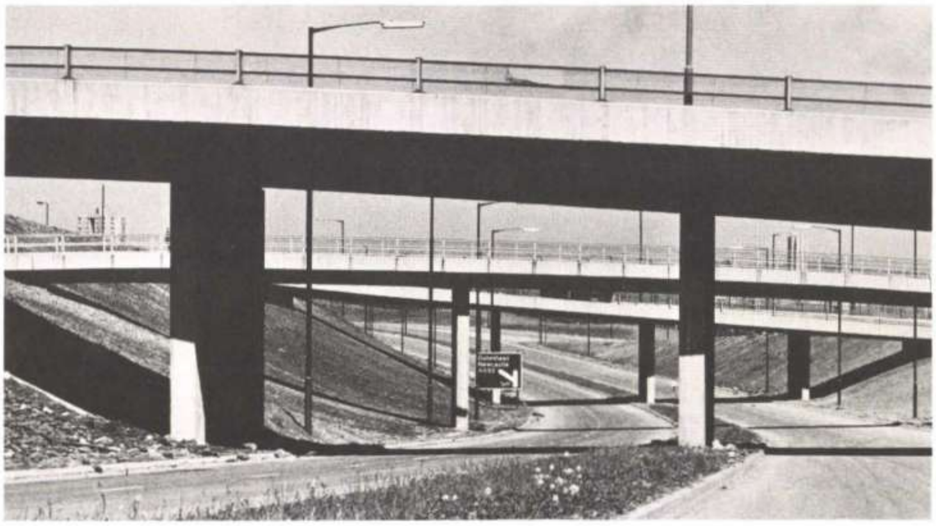
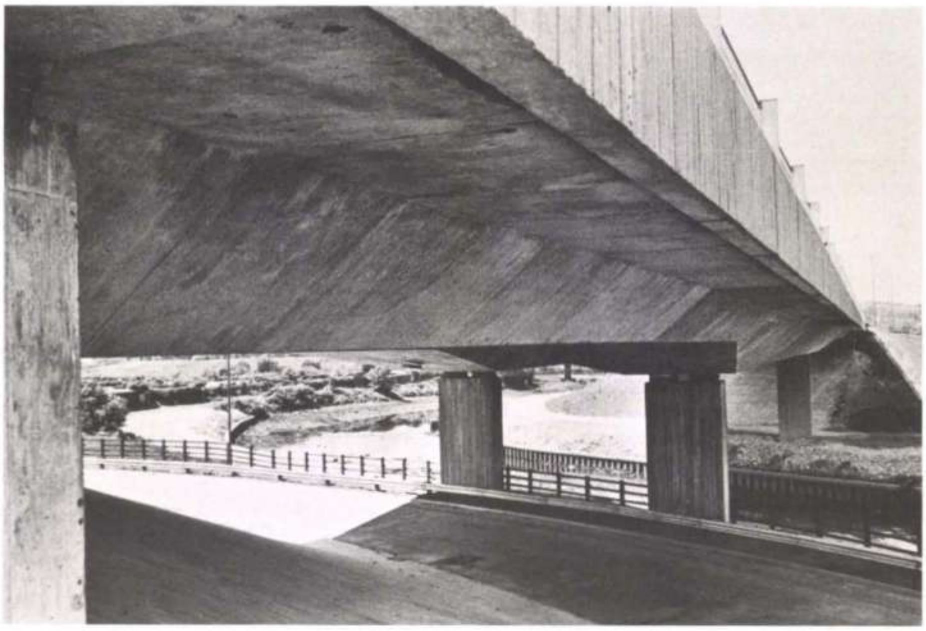


Fig. 1 Lobley Interchange

This part of the road is kept down to reduce the effect on a residential area.

The Bypass bridges are all different from each other and most of them are curved and skew but they all recognisably belong to the same family. We were not responsible for the lighting.

2



Figs. 2 & 3 Derwent Bridge

The main span of 50m over the River Derwent is a prestressed concrete box. The river is trained around the main piers.

This bridge is much larger than the others but it is consistent with them in character and in detail. (All photos: Harry Sowden)

3

Letter to the editor

Essex County Council
Planning Department
County Hall Chelmsford

Dear Sir,

The wind environment of buildings

It is thought that you may be interested to compare the picture depicting the post mill at Mountnessing which appeared on the front cover of your *Journal* in September with the enclosed photograph showing how it actually looked at the time of publication.

As this issue of the *Journal* concerned itself in particular with the wind environment of buildings, it may also be of interest to consider the reasons why a pair of skeleton sails should fall away from a post mill in gusty high winds.

As you will know, the body of a post mill is pivoted on a centrally placed tree or post and is free to rotate thereon. For efficient working this had to be turned into a prevalent wind by hand until the year 1745.

The crown tree or post rested on the post and the timber framing of the whole body was built up from this on side-girts, corner posts, breast and rear beams, etc.

When such a building is retained as a visual amenity in the landscape it is important that it should be stationed in the direction of the prevalent wind.

After the passing of a cold front, the wind normally veers with gusts of very high velocity particularly in a location which was chosen for that very reason. In such conditions due to its form of construction the body becomes unstable and with lateral winds a cycle of vibration is then set up. The stocks of the sails may also be in a saturated condition near the boss or pollend, and this is where rot can form and a fracture eventually occurs.

My Council, who has been managing 4/5 windmills for the past 25 years, is well aware of the causes but with unoccupied buildings, occasional fractures have proved difficult to prevent.

A Millwright who has recently been appointed by this Council will, it is hoped, be able to give more time and personal supervision to prevent further such occurrences.

Yours faithfully,
M. T. Rowland

Before: (left) and after: (below)
(Photo: Courtesy Essex County Council)

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