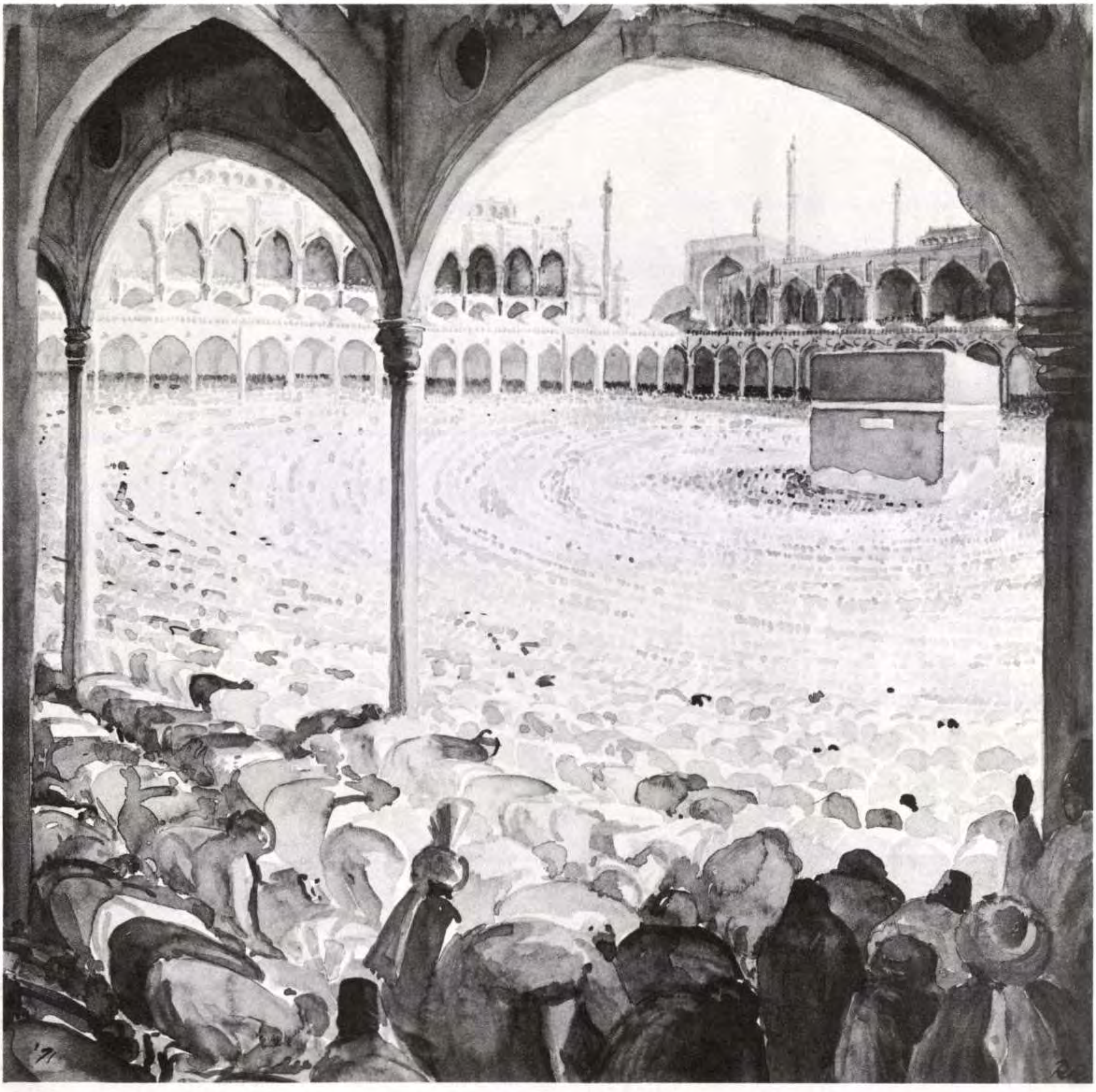
THE ARUP JOURNAL

SEPTEMBER 1971



THE ARUP JOURNAL

Vol. 6 No. 3 September 1971
Published by
Ove Arup & Partners Consulting Engineers
Arup Associates Architects and Engineers
13 Fitzroy Street, London, W1P 6BQ

Editor: Peter Hoggett
Art Editor: Desmond Wyeth FSIA
Editorial Assistant: David Brown

Contents

Front cover: Artist's impression of the Kaaba in Mecca
Back cover: Hyde Park Barracks seen from the Park (Photo: John Dangerfield)

Hyde Park Cavalry Barracks

Ted Happold

Architect: Sir Basil Spence
Main contractor: Sir Robert McAlpine & Sons Ltd.

The new Hyde Park Barracks were occupied by the Household Cavalry Regiment late last year. The site, magnificently situated but of awkward proportions, ¼ mile long, wedge shaped in plan and with marked falls in length and breadth, has been the home of cavalry regiments for almost 200 years. The 1959 brief to the architect required that, within the standard scales of expenditure, quarters be provided for some 500 officers and men, with associated messing and recreational areas, with store, office, workshop, educational and other facilities and with a riding school and stabling for some 270 horses. Like the British Embassy, Rome, the design was intermittent. These requirements divide into three distinct groups: areas for the horses, the soldiers and the married families. The groups are obviously inter-related but they have separate individual needs and these, together with the limitations of the site, fixed the plan.

To reduce disruption to traffic and when exercising in the park, the mounted cavalry must leave the barracks on the Hyde Park side. For this reason the stable yard was made level with Carriage Road and this enabled the fall across the site to be used to separate horses and vehicular traffic by placing the normal day to day vehicular and pedestrian entrances at the lower level on the Knightsbridge side. The stable yard is the focal point of the barracks. Surrounded on three sides by blocks housing barrack rooms, offices, messes, stores and workshops, it is linked by ramps to the stables at the east and widest end of the site. Here, two squadrons of cavalry are housed on two levels in a fully enclosed, artificially lit, mechanically ventilated and heated environment. Stores for saddlery and forage and an underground removal system are all part of the design.

A ramp to the west of the stable yard leads to the riding school and thence to the tower in which are 112 quarters for married soldiers. In its basement is a central boiler house serving the site by a service tunnel. Beyond this again are quarters for the married officers, their mess, 120 pram stores and parking for some 50 cars.

The tower, of strong vertical emphasis in contrast to the essential horizontality of the greater part of the development, is square in plan, measuring 20 m x 20 m and rising 31 floors to a height just above 118 m from ground level. It has a basement, ground floor, mezzanine level and 28 floors above which are similar. From the 29th floor to the roof the floor layouts differ.

The slenderness of the tower is such that the stresses which wind forces induce are large. The design of the tower was carried out nine years ago and was based on work on the wind pressures on buildings available at that time. It was similar to that now suggested in the *BRS Digest*. Pressure exerted by the wind is a function of its velocity and the shape of the building, and so a velocity profile was assumed over the height of the building. The variation of mean hourly velocity with height follows a power law. The mean hourly velocity is related to the gust factor. Gusts of short duration, however, will be spent before the pressure has built up, so because of the dimensions of the building, a gust with a duration of 10 seconds has been used. The gust factor is based on the height. These two assumptions were combined and, taking the gradient velocity over London as being 176 km/hr. mean hourly velocity at a gradient height of 457 m for a return period of 100 years, the velocity profiles, and thus the pressure profiles, were calculated. Checks were made in an ultimate load condition for gusts occurring once in 2,000 years. The pressure profile taken gives a higher overturning moment at the base than designing from Table 3 of *CP 3 Chapter 5**. The forces the structure is designed to resist during the period of construction can obviously be less than those it has to resist in its final condition. A gradient velocity of 122.5 km/hr. for a return period of 100 years was assumed and checks were made in an ultimate load condition for a return period of 2,000 years and assuming 2–3 second gust period.

The solution to the structural core required for the wind and direct load forces and its relation to the architectural requirements, can be seen on a typical floor plan. There are four flats on each floor and it is essential from an architectural point of view that the walls are soundproof. This suited the engineering problem as the walls could be made of reinforced concrete to form an extremely effective core to resist wind forces. In the north/south direction wind is resisted by the walls sub-dividing the flats and these walls are connected across the central lobby by other walls, which provide fire protection, separating the lobby from the stairs on one side and the refuse shaft landing on the other. In the east/west direction the wind is resisted by the walls separating the flats from the lobby and the lift shaft, which run completely across the building presenting their edges to the outside elevation. Torsional forces on the tower are resisted by the lift lobby walls. Thus in plan the wind core is similar to two Ts placed back to back with two links connecting their tops. To improve continuity in these link walls as they rise up through the building, the doors through them are alternated in position so that they do not occur directly above each other. This core carries load as well as wind forces and, in addition, there are four large columns, two on the north side and two on the south side 3.05 m in from the corners.

For a long time it was intended that this core would carry all the wind forces which, suitably reinforced, it was well able to do. Two architectural changes then occurred. Firstly the east/west wall was stopped at the 31st floor to allow two squash courts to be placed

Fig. 1
The original barracks
(Photo: Axel Poignant)

within the top of the tower. Secondly, the architect sculpted the top of the tower which took up the ends of the east/west walls and produced very deep beams across the columns in the north/south direction. This developed a secondary stiffening to the action of the tower under wind forces, enabling the core in the north/south direction to be only reinforced for the maximum forces assumed during construction. In the final condition the assumed forces are reduced by the deep beams, inducing a portal action.

A typical floor has 180 mm thick reinforced concrete floor slabs throughout and is supported on the wind core and round the perimeter on 1.06 m deep by 150 mm thick reinforced concrete edge beams, which carry the load to the four columns and the edges of the wind walls. These edge beams cantilever out 2.4 m and 6.7 m to their corners and act as fire barriers between the floors.

The base of the tower is a traffic circulation area. At mezzanine level, reinforced concrete bridges link the tower to the adjacent buildings and roads. At ground level two approach roads run below the tower through large openings in the north/south wind wall which extends in length to allow for these.

Investigations into possible 'systems' certainly affected the design and, though traditional in situ methods of construction were finally favoured, considerable attempts were made to enable speed to be achieved. The spacing of supports was adjusted to enable the floor to have a level soffit to assist the contractor with forming and thus enable prefabricated reinforcement cages to be more easily used. It was originally intended to use lightweight concrete for the floor slabs but a detailed investigation showed that the economies of saving in weight were offset by the problems of finding space for two mixers on such a confined site. The finish of the exposed columns and wall ends was important to the architecture of the building. To prevent the disruption to construction caused by surface defects in these areas, concrete boxes, of one storey height, were precast and used as permanent formwork. Window frames and stairs were also precast to ensure maximum re-use of formwork and minimum interruption to construction.

At the completion of the design, the one element in a typical floor likely to slow down erection was the in situ construction of the edge beams. We had considered designing

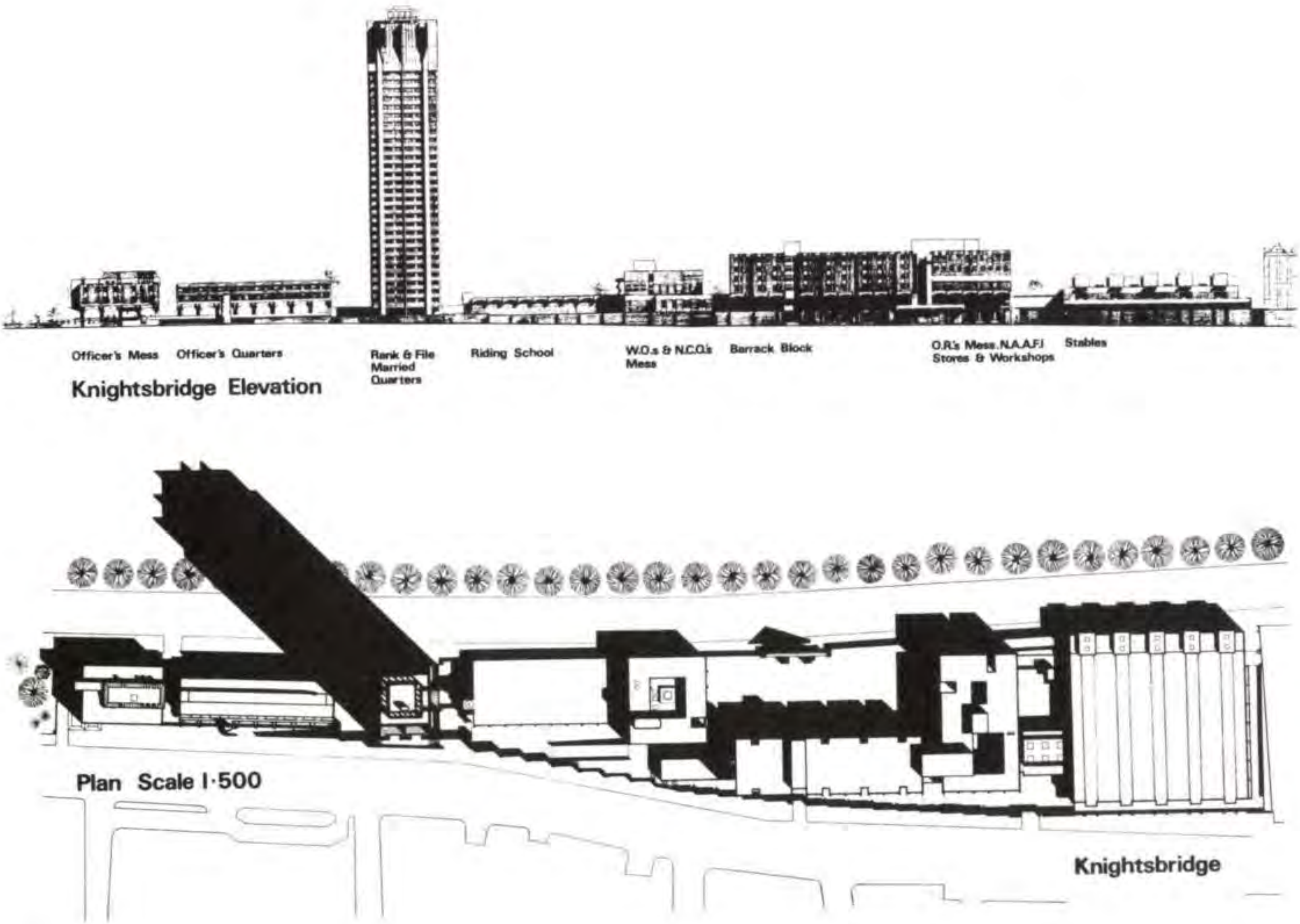


Fig. 2
Site plan and elevation

Fig. 3
Progress photo taken in December 1967 showing the awkward shape of the site (Photo: The contractors)

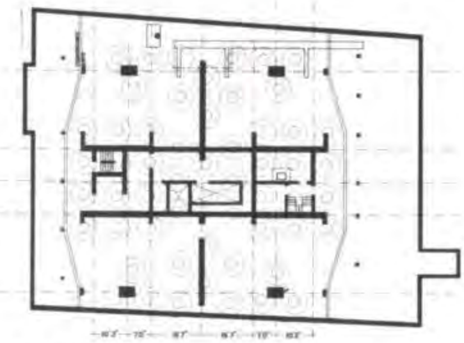
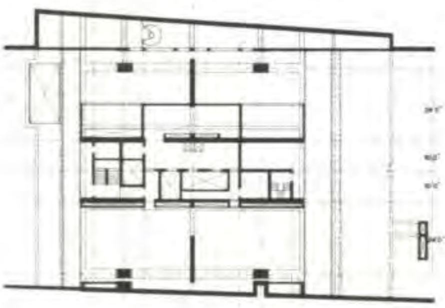


Fig. 4 above
Basement plan of tower block

Fig. 5 right
Plan of typical floor of tower block

Fig. 6 below
Ground floor plan of tower block



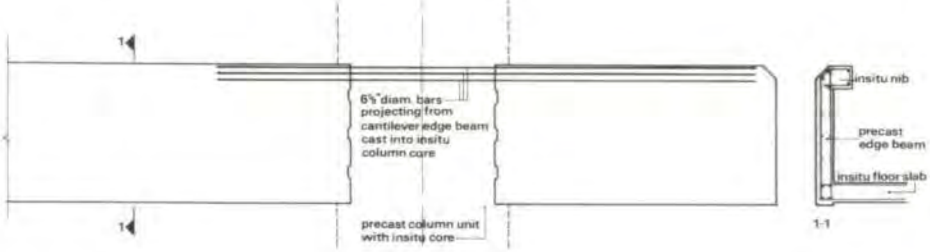


Fig. 7
Detail of McAlpine stitch for edge beams of tower block

them for precasting with poured in situ joints at the corners but were unable to design a detail which could take the torsion and bending without exposing the joints externally.

When Sir Robert McAlpine & Sons Ltd. were appointed contractors, it was this element that most worried them, not so much from the effect on the construction time, but because the beams were cast with a ribbed finish which was then hammered and the risk of injury to passers-by from falling aggregate was serious. They proposed to precast the full L-shape of these beams on the ground to the west of the tower and developed a 'stitch' capable of carrying the negative moments at the columns and walls.

The weathering of the exposed concrete on the tower was of concern. The permanent concrete box forms used on the tower columns and walls were precast using an all-in Capstone aggregate with a lightly ground surface because it was felt to keep its colour in the London atmosphere.

The edge beams have a strongly textured finish with tapered ribs 38 mm deep by 38/13 mm wide at 50 mm centres running in a vertical direction. The ribs formed on casting were manually hammered to produce broken vertical bands.

Considerable study and testing was done before the finish of the edge beams was finally decided on. The client made special funds available for this work. Some self-cleansing properties were sought and it was felt that a flint content in the aggregate might achieve this. Small samples, using different aggregates, were cast at the C & CA Laboratories at Wexham Springs and treated. Equating cost with appearance, Thames Valley sand and gravel were found to have enough flint in them to give the slightly glassy surface required and then a full scale typical corner beam was cast in order to see what the difficulties were in making and striking the shutter and in hammering the concrete. Finally, to try and ensure that the tenderers were fully aware of the standards required of the concrete finishes, photographs were included in the tender documents.

The tower supports bear on to a 1.5 m thick pile cap which is carried on 51 25 m long, 0.9 m diameter, 2.4 m under-reamed piles bored into London clay. The primary problem in designing a pile raft for any such tall building is that no-one knows exactly how the load is distributed between the raft and the piles, i.e. how much is carried on the piles and how much on the raft above. A secondary problem is that there is little knowledge of the distribution of load among the piles. It was felt that evidence should be collected on these two problems. In collaboration with Sheffield University and with the help of the Ministry of Public Building and Works, three pressure cells were installed in the raft and load cells in three piles. These load cells are formed with two steel plates held apart by eight cylindrical columns which incorporate eight glass stress plugs. Reading is by periscope and continues at intervals. Fred Butler and John Hooper are preparing a paper on that work for publication later.

The other buildings on the site, each of differing function and scale, are unified by a common structural system comprising an in situ reinforced concrete frame carried on piled or strip foundations. This is partly clad with very precise brickwork and, where exposed, is fairfaced both inside and out. At the level dictated by the mounted soldier, further unity is given to the various blocks by the use of cylindrical concrete shell units. The shells actually hang off secondary beams which bear on the main beams; they are not themselves connected to the main beams. The hanger beams were precast first and then cast into the shells as they were poured. The concrete used was lightweight *Lytag* and the

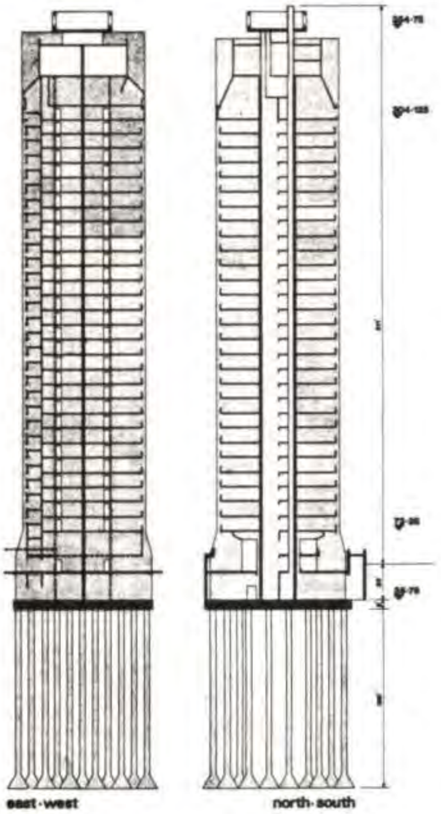


Fig. 8
Diagrammatic section through tower block

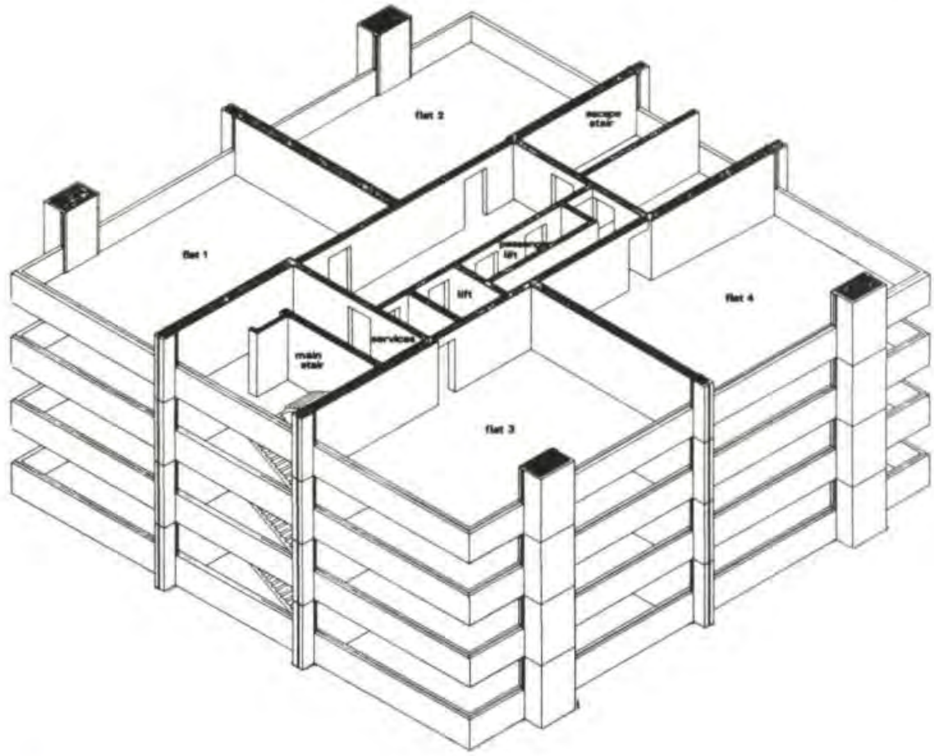


Fig. 9
Isometric showing structure of tower

finish on the soffits is fairfaced. The floor above the shells is made of precast concrete planks spanning between the hanger beams and services can run between the shells and the precast plank floors. The ends of each row of shells are closed by an in situ cast reinforced concrete edge beam. The design of the shell units was first developed for Sussex University. The main beams, which project between the shells, are of reinforced concrete except in the barrack block where they were partially prestressed in order to maintain a constant depth of 1.6 m throughout the site.

The building has caused a lot of controversy because it is claimed that its height excessively dominates the park. There is justification for this as people in the park certainly have a growing consciousness of the buildings around – the Hilton, Royal Lancaster, Hyde Park Corner, Knightsbridge and others. But before blaming the architect for this growing intrusion, perhaps one should enquire who did away with the height restrictions in London or why this site should have such a high density.

I believe the Cavalry Barracks is a good building because it is a building occasion. There is a whole picture in everyone's mind of Changing the Guard, of ceremonial, of Trooping The Colour. People expect the Household Cavalry to have a building of value – individual,

Fig. 10
Exploded isometric of tower block showing in situ splices

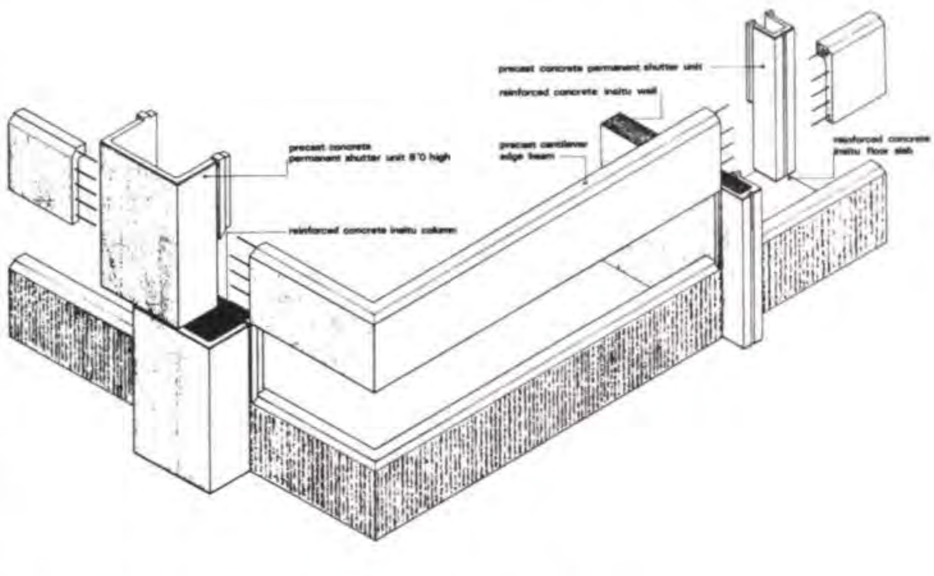


Fig. 11
Part of north elevation of barrack block

idiosyncratic and unique. Scale is part of that; in a way the tower is necessary for the imagery people need. Yet it is the small scale invention and complexity which creates enjoyment for people and this building has it.

Many people have worked on this building in Arups – the time scale ensured that. Povl Ahm, of course, was involved from the beginning. Joe Yu, now at Imperial College, and Clement Chan, now in Hong Kong, had major parts in the early days. Klaus Roos and Peter Peterson, both now in Denmark, worked on the tower. Peter Woodward, now in Saudi Arabia, was the project engineer for a long time and then was resident engineer on site for the entire construction of the structure, Tony Langford taking over as project engineer. Fred Butler worked on the foundations and John Hooper, initially at Sheffield University, on the foundation loading experiments on the tower.

The architect is Sir Basil Spence, OM RA, with John Dangerfield taking a leading part and Anthony Blee and John Church being prominent. Reynolds & Young were the quantity surveyors, Donald Smith Seymour and Rooley the services consultants. Last, though in the quality of their work certainly not least, Sir Robert McAlpine & Sons Ltd. were the main contractors.

Fig. 12
The south side of the site from Knightsbridge

Fig. 13
The tower block.
Detail of cladding

Fig. 14
North elevation of the stables from Hyde Park

Fig. 15
Racks for saddles inside the stable block

Fig. 16
A typical soldier's bedroom

Fig. 17
The tower block

Fig. 18
Courtyard area of stable block

Fig. 19
One of the ramps to the stable block

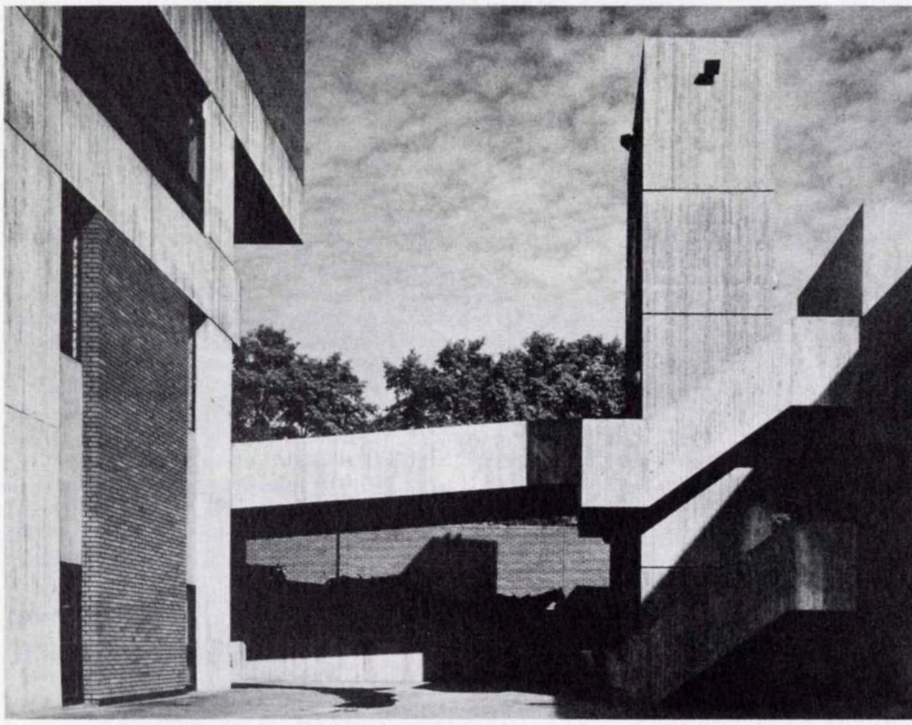
15

12 16

13 17

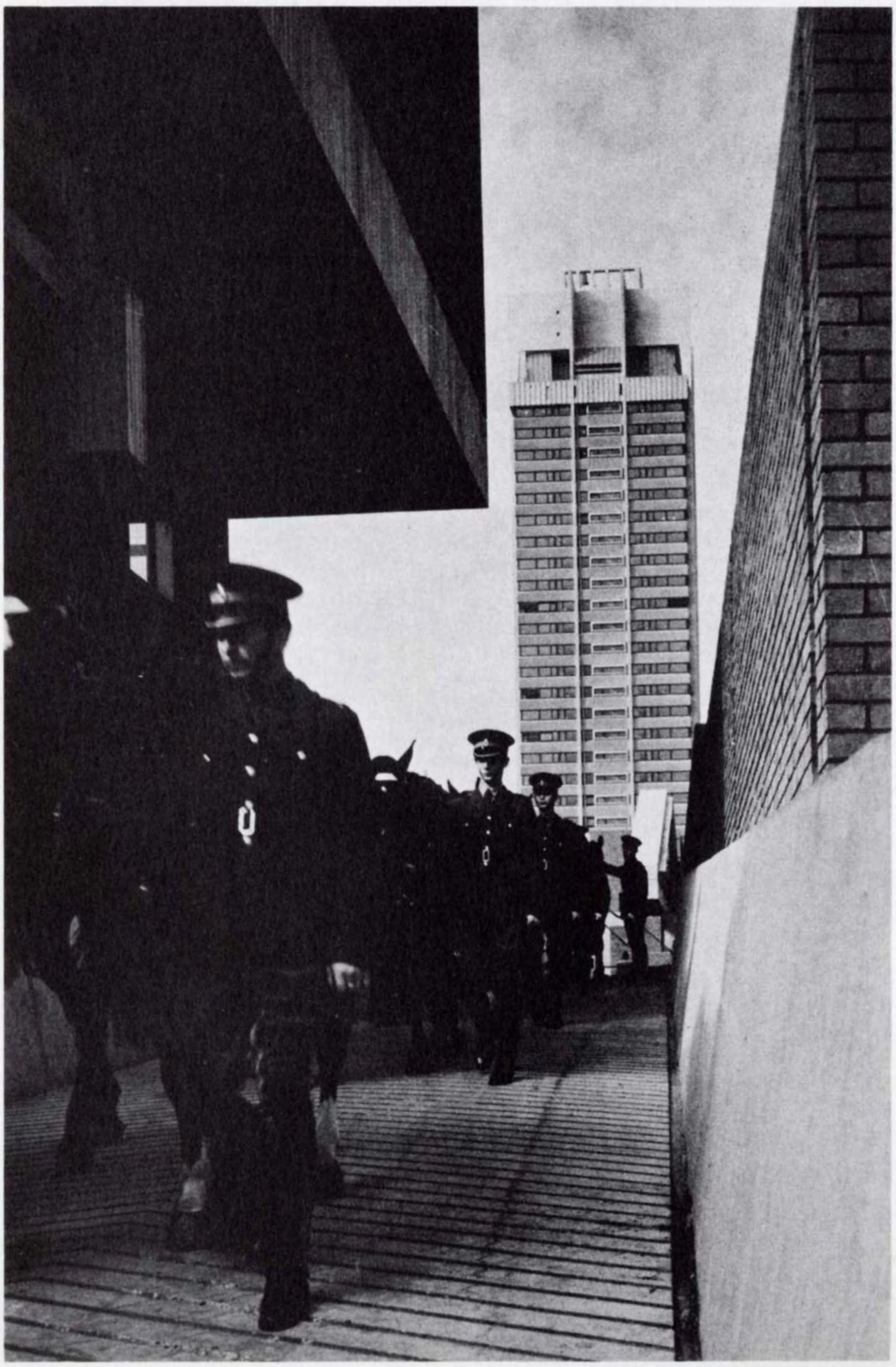
14

Figs. 12, 13, 15 to 19 (Photos: Henk Snoek)
Fig. 14 (Photo: The contractors)



18

19



Archaeology and the engineer

Norman Ross

Introduction

The indications are that, like it or not, engineers and certainly building contractors are going to have to pay far more heed to the claims of archaeology than they have to date. As a nation we have, on the whole, been irresponsible in the way that we have abused sites of historical importance for no other reason than economic expediency. The blame lies not primarily with developers and contractors but rather with local and national government, and even, as will be discussed later, with antiquarians themselves. As far as the building industry is concerned, the reasons for this irresponsibility are the common ones of ignorance, indifference, and a lack of guidance from those who should know better, i.e. architects, engineers and the planning authorities. We can start to make amends by educating ourselves.

Archaeology is defined by Professor V. G. Childe in his book *A short introduction to archaeology*. He begins 'Archaeology is a source of history, not just a humble auxiliary discipline. Archaeological data are historical documents in their own right, not mere illustrations of written texts. Just as much as any other historian, an archaeologist studies and tries to reconstruct the process that has created the human world in which we live—and us ourselves in so far as we are each creatures of our age and social environment. Archaeological data are all changes in the material world resulting from human action or, more succinctly, the fossilized results of human behaviour. The sum total of these constitute what may be called the *archaeological record*.'

This record is therefore a *material* record of soil layers, pottery, walls, floors, graves, etc., and is unique to any one site. To destroy it before it has been properly recorded would be rather like burning an ancient manuscript before reading and photostating it. In each case we would lose forever one fragment of communication with the past. Moreover the archaeological source of history of any particular site might be the one and only source. Those of us involved with the restoration work at York Minster have been made aware of one archaeological 'problem period'—the Dark Ages, which span the 5th to 9th Centuries A.D. Any area that is known to have been inhabited in pre-Norman times could be of vital importance to us. The history of the York Minster site begins in the Roman era but the strata being most diligently excavated are

Fig. 1
Plan of York Minster

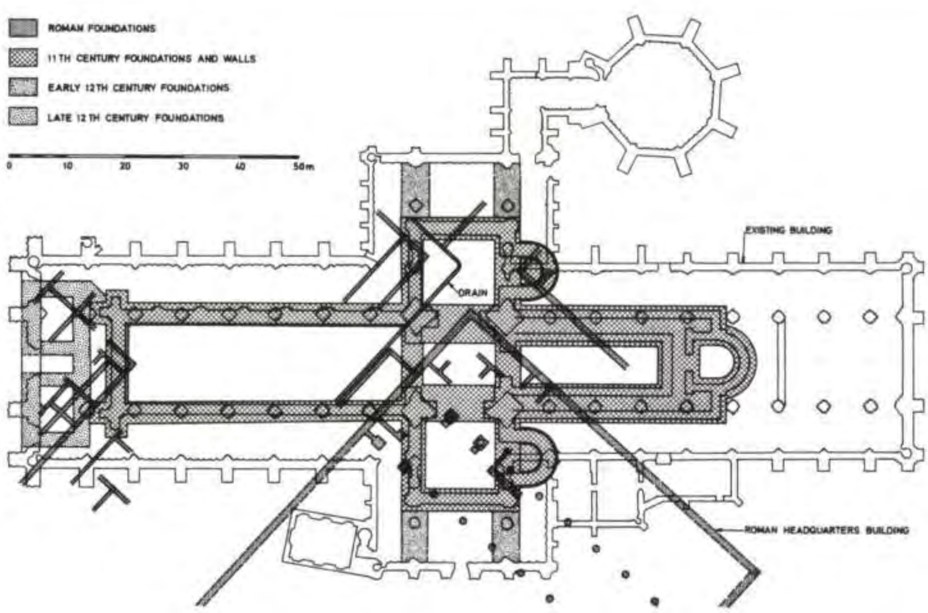


Fig. 2
Plan of a Saxon hearth discovered under the Nave of York Minster

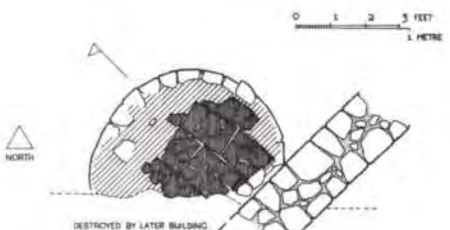
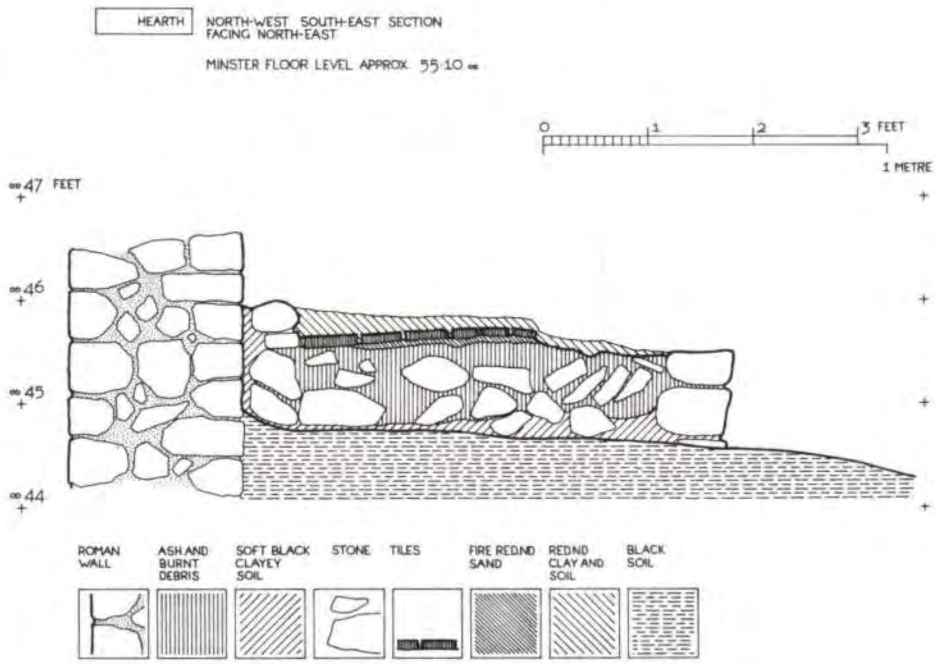


Fig. 3
Cross-section of a Saxon hearth



those which form the post-Roman occupation levels.

Once we have appreciated the importance and relevance of archaeology we must come to an understanding of the techniques which an archaeologist uses to extract the maximum amount of information he can from a site. It is the sense of incomprehension which leads to feelings of frustration in a contractor as he watches an archaeologist at work. What is the man doing? Why is he excavating with a trowel in layers only 5 mm thick?

The only thing that an archaeologist and a contractor have in common is the removal of soil. The methods differ because the aims differ. Archaeology, in essence, is the removal of deposits in the reverse order of their deposition. If any deposit leaves slender evidence, careful investigation is called for.

Contrary to common belief, the study of ancient structures or monuments in itself is not the primary function of an archaeologist. It is the history of the structure that interests him, and its history is contained in the layers of soil which surround and cover it. If the occupants of the building were very houseproud, the layer of deposits left by them might be very thin and contain very little evidence of habitation such as potsherds, coins, and metalware. The archaeologist depends heavily on such artifacts for dating the period of occupation and consequently would wish to dig very slowly and carefully. On the other hand a deposit might be fairly thick, perhaps even several feet, and could be dealt with more quickly. Examples include areas of demolition and thick layers of silt deposited during extended periods of flooding. If, for one reason or another, time is against him, the archaeologist must make a value judgement about the time he spends on any particular area or level of the site. Ideally of course, such a situation would never arise.

It is fascinating to watch an archaeologist at work. What appears at first sight to be nothing more than a pile of builder's rubble is immediately classified by the archaeologist as a 'feature'. Careful scraping with a trowel (sometimes a spatula and paintbrush) reveals it to be a Roman wall or a Saxon fireplace or even a group of tightly-packed graves each with ornately-carved slabs and headstones. In association with the Roman wall there might be an area of *opus signinum* floor of crushed red tile and mortar which would be painstakingly cleaned and recorded; or possibly some slabs of painted plaster which at one time would have decorated a room. Where it is possible, all finds are preserved and conserved for exhibition either on site or in a museum.

Archaeologists prefer to excavate a site in a manner which enables them to see the whole site at once and simplifies the photography and surveying which is a very important aspect of their work. Consequently they can leave high soil sections around the perimeter of their dig, useful to them as references, but which raise the whole question of safety.

In the past, archaeologists have relied on their own common sense in this matter, receiving professional advice when the site falls within the domain of a city engineer's department or similar body. Unfortunately commonsense occasionally takes second place to stronger emotions and over-zealousness has led to tragic endings. On the site of York's city wall, not 300 yards from the Minster, a Royal Commission archaeologist was killed when a 15 ft. high trench collapsed and buried him. He was taking a calculated risk with the pressures of time and money against him which makes his death even more needless and adds to the pertinence of this article.

The subject of safety is one of many which bring us as engineers into the field of archaeology. We can give advice, for example, on matters like surveying techniques and mechanical means of soil disposal. We can offer opinions on structural problems. Is an arch within a foundation there as a relieving arch or for another purpose altogether? Did these 12th Century foundations ever carry a substantial load? The benefits of a close relationship are, of course, mutual, since the engineer or the contractor could receive valuable advice during the site investigation stage resulting in reduced costs. But perhaps most important of all we can help the archaeologists to organize themselves efficiently and to plan the best way of dealing with a particular site.

At present there are two possible methods for providing for archaeology on any development site. One is to allow for it within the contractor's programme of work. In other words the site remains the total responsibility of the contractor from the start of the contract and archaeological activity is permitted only within a set period and under terms set out in the contract. The other method is to hand over the site initially to the archaeologists for research, either until they have finished or for an agreed duration, after which the contractor takes possession.

York Minster archaeology

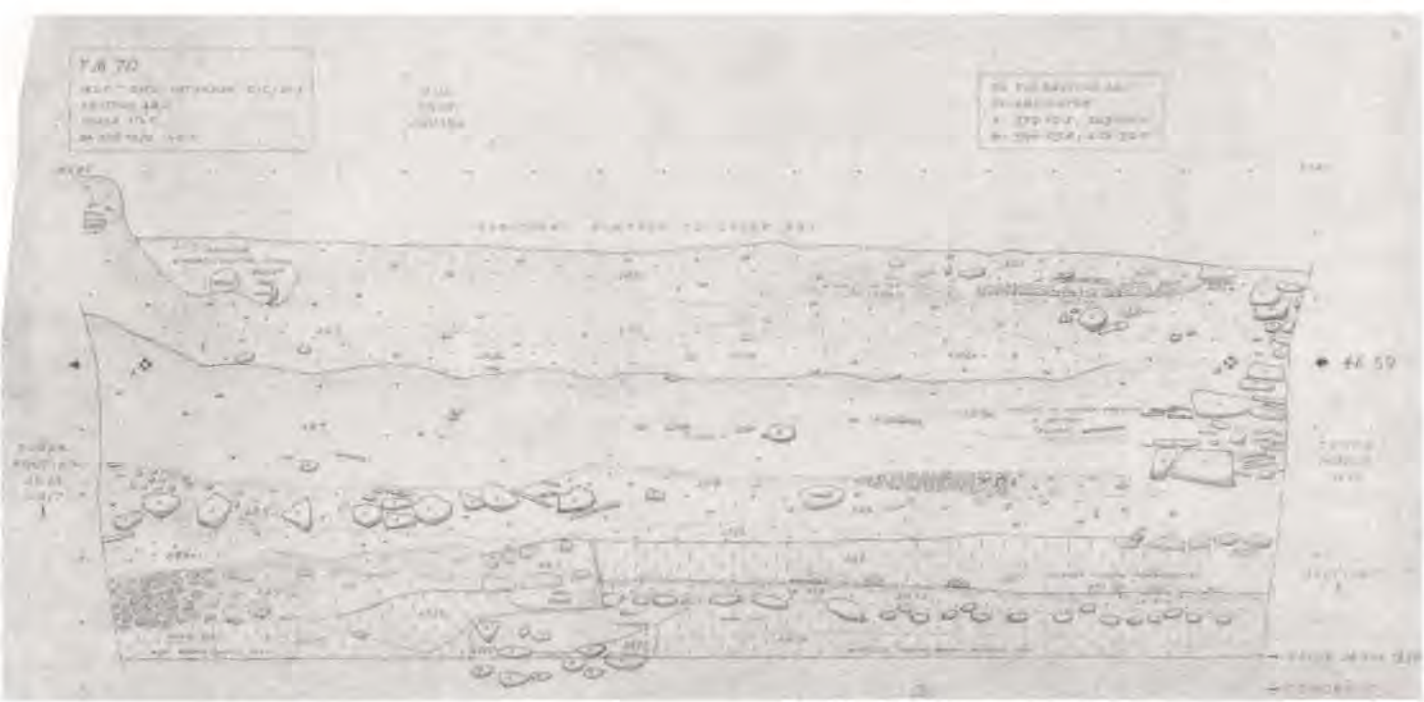
At York Minster the situation has slowly evolved, after a struggle, into something approaching the former method. The site is said by some to be the most important in this country. It is certainly one of the most important, situated as it is in the centre of the Roman fortress of Eburacum—the capital of *Brittannia Inferior;* the Roman 'Whitehall'; after the departure of the legions, a religious and political centre with great European standing. Under the floor slabs of the Minster lies part of the *principia*, possibly the Commandant's house, Saxon remains, and parts of the huge Norman cathedral.

When it was first realized that something was drastically wrong with the foundations of the Central Tower, an initial inspection of part of the substructure was carried out under the guise of an archaeological dig. As soon as the contract for the restoration work was signed however, archaeology came a very poor second to the need for rapid excavation. This began in a very unarchaeological manner, despite the setting up of an advisory committee under the chairmanship of Sir Mortimer Wheeler.

The contract work at the Minster thus began with the premise that archaeology was only to function within the limitations of a watching brief. Initially, only one archaeologist was appointed on secondment for a year with a possibility of continuing to try and record as much as possible of a mountain of material and, inevitably, he was forced to withdraw by the pressures of the task. Archaeology on the site has been struggling ever since to recover from that inauspicious start.

It has been a sad situation in many ways, particularly as much of the difficulty was unnecessary. Now *with the benefit of hindsight*

Fig. 4
Typical example of an archaeologist's working drawing



we can question why the difficulties arose, not in malice but in the hope of easing the path of others in the future. There seems little doubt that the main responsibility lies with the archaeological community, and in particular with the Advisory Committee, for they failed on two crucial counts.

First by not making it sufficiently clear right from the outset to all concerned; the client, engineer, contractor and, particularly, the general public, just how important the site was historically. The client, in the form of the Dean and Chapter of York together with the trustees of the appeal fund, thus were not made to fully comprehend and acknowledge that the site must contain an invaluable record of early British history. In fact the client tended to play down the fact that archaeologists were present, apparently in the mistaken belief that this was in the best interests of the project.

For our own part we ought to have insisted more strongly than we did on a sequence of excavation to give as much time as possible for archaeological research. Regular, precise measurements are taken of the movements of the fabric. These give an indication of what Bernard Feilden calls the patient's temperature. Together with visual inspection and sound judgment the measurements enable us to be in control of the situation and to advise on the safe limits for excavation.

The contractor must also be included in any list of culprits. He deserves some sympathy as he has probably suffered financially as a result of the general shortsightedness. However, whilst admitting that he received inadequate instructions, he could and should have been more understanding of the difficulties facing the archaeologists. He could have been much more flexible in his methods of excavation and shoring. We all might also have made better efforts to try and interest his workmen in antiquities. One of his labourers might then have experienced a feeling of dismay when the point of his pneumatic drill pierced an exquisitely carved Northumbrian hogback tombstone!

By no means all has been lost. The publications which will follow the completion of the work will be eagerly received by historians all over the world. The Saxon church has not been found as had been hoped, but the findings include an extensive Saxon graveyard on an alignment parallel with the axis of the Roman fortress which runs northeast to southwest. This circumstantial evidence has led to a concensus of opinion that the Saxon foundations were not simply incorporated into the Norman work. Much has been learned or confirmed about the Roman buildings and the Norman cathedral. In addition, archaeology has demonstrated that it is a source of history by the discovery that, in the 13th Century, York Minster had two twin projecting towers adjoining the 11th Century Norman West Front, a fact not previously known or even suspected. A very large number of artifacts have been extracted, among them some of the finest Saxon tombstones in this country. Another valuable find was a large quantity of painted Roman plaster, once a wall decoration, showing scenes from life 1600 years ago. Part of the plaster has already been assembled like a gigantic jigsaw into a panel measuring 8 m long by 3.4 m high. It is intended for display in the undercroft which is being formed around the new foundations. The preserver is Dr. Norman Davey, a retired civil engineer!

Conclusions

What are the lessons we can learn from the experience of York Minster? There is patently a need for much better communication. If a site is archaeologically important, let the fact be stated unequivocally by all interested parties, in particular by the antiquarians. Let the Government take steps to ensure that everyone is informed and let Parliament legislate, as others have, to ensure that the claims of archaeology are not brushed aside. Sufficient funds must be forthcoming to deal fully with the research. If they do not come from private resources then the Government must provide them. Let there be an adequate team of professional archaeologists who are engaged full-time on a proper contract basis with a brief to study and record a site in as short a time as is practicable and to publish their report. The team should receive advice from the professions concerned with building and the backing of experts in fields like soil analysis and conservation. They should be able either to hire their own plant and labour, or to have it seconded to them by the contractor. The ramifications of insurance, compliance with building regulations, etc., should be worked out thoroughly so as to avoid a situation where there could be divided responsibilities.

It is time that we all awakened to the danger of being described by future generations as barbarians. In this conservation-minded decade we must make every effort to preserve our heritage and our history.

In conclusion, I should like to acknowledge the assistance given in the preparation of this article by Derek Phillips, Director of Archaeology at York Minster.

References

(1) MITCHELL, D. York Minster. Part 1—York. *The Arup Journal*, **3**(3), pp. 50–58, 1968.

(2) MITCHELL, D. York Minster. Part 2—The Minster. *The Arup Journal*, **3** (4), pp. 70–82, 1968.

Fig. 5
York Minster: Saxon grave slab and skeletons from South transept central area excavations (Photo: Royal Commission on Historical Monuments (England). Crown copyright)

Figs. 6 & 7
York Minster: Remains of a Roman column base and podium with Saxon grave. (Photos: Royal Commission on Historical Monuments (England). Crown copyright)

Projects in Saudi Arabia: an introduction

The story of our entailment in Saudi Arabia dates back to 1966 when Trevor Dannatt joined a group of us in entering for the United States Steel bridge competition. Trevor must have enjoyed it because when, immediately afterwards, he was asked to take part in a limited competition for the design of a conference centre and hotel for Riyadh, in Saudi Arabia, he asked us to join him.

The United Nations planning adviser to the Government of Saudi Arabia was a well-related Egyptian architect called Dr. Omar Azzam. He had organized the Saudi Government, through the United Nations, to ask the UIA (Union Internationale des Architectes) to run the competition and the latter appointed Theo Crosby as technical adviser. The competition defined a site in Riyadh, but subsequently we discovered that they intended to build three; one at Riyadh, one at Mecca and one at Dhahran.

Our entry went off and we half forgot about it until a cable arrived, rather to Trevor Dannatt's horror, asking him to build the Riyadh one and to come out to discuss it. So on New Year's Day 1967, Trevor Dannatt and Ted Happold went out. They were in Riyadh about three weeks and there met Professor Rolf Gutbrod, with some of his staff, who, in partnership with Professor Frei Otto, had also been a competitor and who was asked to build his scheme in Mecca. Obviously many of the problems of the two projects were the same and when Rolf Gutbrod and Frei Otto asked us if we would be their engineers as well, we agreed and jointly signed the contracts on both buildings. One other firm cannot be missed out from this story. It was suggested to us that Widnell & Trollope should be appointed quantity surveyors and Chris Meyer, their senior partner, and his colleagues became an integral member of two very closely knit design teams.

Arabia is an amazing country. Nearly a million square miles in area, it has a population estimated at 6 million. Half the population are still nomads; of the rest a million are farmers. Largely desert (temperatures of 120°F in the shade are common in the summer), it has an estimated 10% of the world's oil supply.

Land of Abraham and Mohammed, and once the centre of an empire that stretched from Spain to India, it was broken up by the beginning of this century, into a whole series of tribal areas. Riyadh was a small town held by a northern family, the Rashid, while the Saud family, the traditional rulers, were in exile in Kuwait. In January, 1902, Abdul Azziz, the son of the Saud family, led a raiding party of 40 men to try to recapture the town. With six of his men he climbed over the mud brick walls of Riyadh, opened the gates to the rest of his followers and hid for the night. Next morning they attacked the Rashid Governor as he came out of the fort with his bodyguard and, though heavily outnumbered, won a short, fierce battle to regain the town.

In the years that followed, Abdul Azziz consolidated most of central Arabia. A deeply religious man, he founded religious co-operative farming communities in order to reduce the tribal divisions and to provide a readily available army. In 1913 he drove the Turks from the Eastern Province; in 1925 he drove the Hashemites out of the Hejaz.

In 1938 the Standard Oil Company struck oil near Dhahran but the King was slow to let Westerners freely into the country. He died in 1953 and was succeeded by his son, Saud, a profligate man who was deposed in 1964 in favour of his brother, Feisal, who rules now.

Under King Feisal's rule the country has been opened more to the West though tourists are still not allowed, only men on business and pilgrims being granted visas. The King's liberal policy led to this competition. An attempt had been made to hold an international conference in Riyadh and the facilities had been totally inadequate. Linking this need with a desire to set a higher level of building standards led to our entailment.

Both projects went out to tender in 1968 and COGECO, an Italian firm, won the Riyadh project and Entreprise Thinet, a French firm, in partnership with Joseph Khoury, a Lebanese firm, won the Mecca project. Together with the architects, we have the responsibility of supervising the work on site.

In Riyadh Peter Woodward is in charge of the site staff and has been there two years. Nick Madinaveitia started the site off and has been there two and a half years. Ted Cook has been out there about three months as the Clerk of Works and now Richard Clark, of Trevor Dannatt's office, has joined as site architect. All have families out there, three of them living on the site and Peter Woodward in the town.

Site meetings are held at about six week intervals and regular visits are made.

In Mecca the site itself can only be supervised by Moslems. Saad Kudsi and Muhsin Dharamsi have been out there for two years. In the last year, two architects, Amine Charif and Malik Evrenol, have alternated there. All of them are Moslems and can enter the site. There is also a meeting place at Hadda, just on the Mecca boundary, which is the nearest point non-Moslems can go and Brian Duck is about to go out to help to co-ordinate the supervision work.

Travel on both jobs ranges widely. The site investigations were carried out by Basil-TAC, a firm based in Athens. On the Riyadh project the main contractor's office is in Rome and the structural steel space frame roof has been fabricated at Metalfer, a firm just outside Rome. The mechanical services sub-contractors are centred in Beirut and the electrical services sub-contractors in Eindhoven.

On the Mecca project the architectural design centred on Stuttgart where both Rolf Gutbrod and Frei Otto had offices. Now Büro Gutbrod has moved to Berlin. The main contractor is centred in Paris, his partner in Beirut, the mechanical services sub-contractor is in Jeddah and the electrical sub-contractor in Eindhoven. The cable roofs are a major feature in this project and the steelwork is being carried out by Voyer & Company from Tours, the masts being fabricated in Lille and the cables by Baudin at Chateauneuf.

Figs. 1 & 2
Pilgrims assembling outside Mecca for the Hajj (Photo: Courtesy of the Saudi-Arabian Embassy)

Fig. 3
Three of the seven minarets of the Haram (Photo: Muhsin Dharamsi)

Mecca: Saudi Arabia

Muhsin Dharamsi

The city of Mecca is in the western part of the kingdom of Saudi Arabia. It has a good communication system with the rest of the kingdom and all parts of the world. There is a three-lane motorway connecting Mecca with the Red Sea port of Jeddah 75 km to the west. Mecca is also connected with good tarmac roads on which one can easily cruise at 140 km/h to Medina, about 400 km to the north, and on to Jordan, Syria, Lebanon and Europe. Eastwards one can go to the mountain resort and summer seat of Government of Taif 100 km away and on to the capital, Riyadh, 1000 km away. East of Riyadh are the rich oil producing areas of the Kingdom. One can go on further by road to Kuwait, Iraq and Iran. With such a good network of roads it is no wonder that a good percentage of pilgrims to the Holy City still come by road. Others come by sea to Jeddah or to the international airport at Jeddah or to the local airports at Medina and Taif. Comfortable taxis take travellers from one city to the next.

Mecca is built over large rocky hills and the houses cling precariously to the hillsides. The city has been extensively developed in the last decade and huge multi-storey, high density blocks of flats have sprung up in the city centre. The town planning office in Jeddah is trying its best to guide the development of the city. There are plans to construct a large pedestrian precinct around the Holy Mosque. In the meantime development is taking place at a fast rate. As the small, old and beautiful houses are demolished one by one, tall and extremely slender blocks of flats spring up in their place. Many of these towers are only 15 m square and rise to 12 or 15 storeys high.

The Conference Centre and Hotel in Mecca is being constructed next to the Mecca-Jeddah highway, 7 km from the Haram (the mosque which houses the Kaaba, the central Muslim holy place). The site is at the junction of several valleys and is in the path of flood waters. A rip rap embankment has been built to divert any flood waters coming from the north. After any rain, flood waters also flow past the site, on and along the highway. Part of this water spills into the site and has to be diverted away. The site has also been generally raised about 1½ m above existing ground level.

In the past year it rained on three different occasions for one hour each.

Because of the rocky nature of the soil in Mecca, almost all the water collects in flood waters and disrupts communications for several hours. Soon after any rain storm, the city council brings out an army of sweepers who clean the city very rapidly. Otherwise the climate is dry, although occasionally humidity rises to 80% for a day or two at a time. During summer the shade temperatures are usually 25°C minimum and 45°C maximum, occasionally rising to 48°C.

In winter the shade temperatures are usually between 30°C and 18°C, occasionally dropping to 13°C minimum.

The project has been conceived as an oasis springing out of the desert. There are plans for an extensive gardening system to provide shade and beauty. Because of the herds of goats and sheep roaming around, the site may finally have to be protected with a boundary wall or a thick growth of bushes as a fence. There is a basement service tunnel connecting the whole project. The basement had to be of water tight construction because of the extensive amount of water that will be used in the gardening system. Two existing wells will be used to water the gardens, the top of the water in them being about 6 m below the existing ground level.

Mecca Hotel and Conference Centre

Mick Courtney

Architects: Rolf Gutbrod and Frei Otto
Main Contractors: Entreprise Thinet (Paris) and Joseph Khoury (Beirut)

Description:
The project is a complex of low rise buildings; a 200 bedroom hotel, of approximately 22,600 m² floor area, of three and four storey blocks, with restaurant and reception areas, and an adjacent service block, grouped round a central artificial oasis and a conference centre of approximately 9,000 m² floor area containing a 1,500 seat main auditorium 21 m high overall, forming a continuous walkway round another central oasis, with the two storey foyers to the three seminar rooms which each seat 200 persons. These two groups are linked to form the entrance at the junction, and look in plan rather like a figure '8'. A separate mosque and religious authority office block of approximately 3,170 m² floor area has a sunshaded marble floored courtyard. Three villas for staff and external works, consisting of roads, landscaping and car parking, form the rest of the project.

The oases and the entrance drives are covered with wooden open lattice work sunbreaker screens suspended on cables. The main auditorium roof is aluminium cladding on insulation, supported by a suspended cable net.

Saudi Arabian scenery has a harsh desolate charm, and this quality has been picked out by the architecture of the hotel and conference centre at Mecca. The desolation is reflected by the way the buildings huddle round the oasis pools and planted areas, drawn together by the radial lines of the cables of the sunbreaker screens and surrounded by the small, grassless, gravel strewn hills which look rather like abandoned slag heaps. The harshness, beaten out by the sun, is reflected in the stark, severe concrete finish of the walls and slabs.

However, once within the complex of the hotel buildings, the cool shading of the sunbreakers, the patterned marble pavement, the gentle waterpools and streams, the planted areas, the rich roughness of the wood-framed glazing, and the natural stone walls will give the feeling of the quiet, rich hospitality of the country. The massive auditorium and the seminar areas of the conference centre provide a contrast. The conference buildings will be much higher than the hotel, and will be highlighted by the sun shining on the aluminium cladding. Thus the visitor arriving will see them glinting behind the hotel buildings.

This distribution of the buildings is shown in the site plan, and is becoming apparent in the latest photographs of the site. These photographs show how the building will be in harmonious contrast with the surrounding scenery.

Key
1, 2, 3 Seminars
4 Conference oasis
5 Hotel entrance
6 Hotel oasis
7 Hotel
8 Conference entrance
9 Auditorium
10 Villas
11 Restaurant
12 Mosque
13 Car park

Fig. 1
Site plan of Mecca project

Fig. 2
General view of the Mecca site from the east (Photo: The architects)

Structure: general

The site required roughly 1½m of fill over three-quarters of its area to raise it above flood level. The ground was sand, fine and well compacted, so the foundations were designed as reinforced concrete pads just below original ground level. The range of temperatures was considered to be from 50°C to 5°C so the position of joints to accommodate temperature movements was an important part of the design. Joints were detailed at approximately 20 m spacing and at junctions between buildings of different structural type or loading. No provision was made for earthquakes as Mecca is outside the recognized earthquake zones. The loadings and the design were according to British Standard Codes of Practice. The concrete was generally of 21 N/mm² cube strength at 28 days.

Hotel oasis

The hotel is constructed in reinforced concrete of slabs spanning on to cross walls at 4 m centres. The hotel terraces step back from the central oasis as the building rises and the corridors running continuously through the buildings at each level do likewise. There are no downstand beams in the corridor areas to obstruct service runs so the cross walls above are designed as beams spanning the corridors. The corridor openings form a line of weakness in the cross walls and checks were made to ensure that this did not create areas of local overstressing of the walls or footings. The concrete cross wall construction permits local high stresses to be accommodated and a certain flexibility in working out final details. The stability of the building in the longitudinal direction is ensured by frame action between the walls and slabs and in the transverse by the rigidity of the cross walls.

The restaurant area is the ground floor of the linking part of the complex behind the entrance and has hotel rooms on the upper floors. To create the change from the spaced cross walls of the hotel rooms to the large open spaces of the restaurant, the walls are carried by Y-shaped columns. Each pair of walls is carried by three columns. The walls are positioned at the ends of the arms of the Y and are supported by compression and bending in the arms and tension in the slab tying the arms together. The walls span as deep beams between each column. Stability is ensured by the slabs, columns and walls, acting as a frame.

The service block design is slab, beam and column to maintain open areas for the siting of plant. Originally the floors were designed as flat, in situ concrete. However as the appointment of the hotel lessee by the client was delayed, in fact has not yet been made, the positions of equipment and the associated builders work are unknown. To facilitate the installation, the slab was changed to precast beams with a topping and the load carrying capacity of the slab between these beams was ignored. Thus holes can be punched anywhere in the slab in the area between these precast beams.

Conference Centre oasis

The structure of the auditorium reflects the architectural planning of the interior. The banks of precast seating steps are carried on portal frames on lines radiating from the stage. To accommodate services and movements twin parallel portals are used and the services carried in the gap between. The auditorium is enclosed by featured concrete walls which form the VIP rooms, staircases and balconies.

The roof is a suspended cable system independent of the structure. The front edge support is a portal frame of a beam and two raking masts in structural steel. This frame also provides stability both during erection and in final position. The back edge support is a series of three boundary cables carried by six raking masts. The roof is formed of steel cables, spanning between the boundary cables and the portal frame, forming single curvature planes, carrying transverse T-section steel members to support the roof cladding—sandwich panels of wood insulation and aluminium sheeting. The portal frame and the masts are stayed by other cables which are tied to gravity anchors below the ground. The design method of this roof is too complex to describe in this article but it will probably be dealt with in a separate article. To simplify the erection process the need to prestress any of the cables has been avoided. However, this erection is still a difficult process to which a lot of thought has been given to forecast and solve problems. The worst difficulty was to convince the contractor that he would encounter site problems and that these problems must be conceived and solved before he got on site. Our achievements in this way are described in another article.

An interesting structural problem was tying the ends of the cables to the gravity anchors. The anchors are formed of reinforced concrete boxes filled with sand, of which there is a cheap and plentiful supply in Saudi Arabia. To provide continuous access to the cable ends and for economy, the anchorage tie points are formed above ground. Essentially the attachment is made by an inverted L-shaped piece of reinforced concrete projecting from the structure of the box. The plug on the end of the cable catches on the arm of the L, which acts as a corbel, and the force is taken down the leg into the box in tension and bending. As the forces are of the order of 70 tons some concern was expressed over load carrying capacity and factors of safety in the design of the concrete corbels. After some research the conclusion was that a lot of reinforcement was required and that if there was sufficient room to get this within the concrete member then the stresses on the concrete were within acceptable values.

The seminar foyers and entrance which form a two-storey ring round the oasis are a flat slab and column construction with column heads. These foyers are the entrance to three seminar rooms. These are, essentially, large (13 x 19 m) reinforced concrete boxes forming the upper storey and supported along two sides only by columns and walls and covered by a suspended roof. The roof structure is similar to the auditorium with cladding, T angles and cables, but the cables are attached at both ends to boundary cables. These are anchored to each end of the cross wall end beams of the seminar box which are supported by the cantilevering ends of the side walls. The cross wall beams therefore span 19 m, are deep (4 m) and have a pre-compressive force at their upper edge. Some agile structural analysis was required before it was established that the structure was safe. The steel sculptures, which convert the tensile force of the boundary cables to compression thrusts in the cross wall and side walls, weigh 1 ton each and their geometrical analysis and stress determination were complex.

The mosque has a 'flat slab and column' office block and a high wall surrounding the prayer courtyard. The wall is of reinforced concrete, faced with stone and the courtyard has a wooden open-lattice work sunbreaker roof supported on circular steel columns.

Sun shades

The open-lattice work wooden sunbreakers are a feature of the project. Each hotel room terrace has a small individual one and the oases and entrances are covered by vast canopies.

Each canopy has a boundary cable supported by 3 or 4 steel masts at a distance of about 4 m from the central point of the oasis or entrance. As the canopies are not closed circles the boundary cables pass over the last mast and are anchored down to the ground by a buried concrete slab. From the boundary cable, beams formed by upper and lower cables joined by vertical spacers radiate as if from the centre point and are anchored to the first and second floors of the buildings. The loads carried by the cables are determined by the geometry of the system and the geometry of the system is determined by the loads carried. This is a vicious circle and the design is accomplished by making an estimate of the geometry and then letting a computer do a reiterative reduction of error program until there is compatibility of loads and geometry.

Fig. 3
Mecca: auditorium seats over Mechanical Room (Photo: The architects)

Fig. 4
Mecca: restaurant and King's Place (Photo: The architects)

Fig. 5
Mecca: stone walls in hotel terraces (Photo: The architects)

The design team

Professor Rolf Gutbrod is the architect for the Hotel & Conference Centre at Mecca for which Structures Division 3 are responsible for the structural design. Professor Gutbrod has a very strong philosophy of an architect's function, which he has instilled and enthused into his whole office. It is not just about architectural design, but is a philosophy of total application to building. He believes that the architect should be fully committed with the contractor in constructing a building, and that the contractor should be completely involved with the architect in the design. That is, he believes that the architect and the contractor are a team, working together, so that the one can correctly translate into a building, in time and place, the ideas of the other.

This entails tremendous co-operation, from a very early stage of the design, between the architect and the contractor. To obtain this, Büro Gutbrod believes that the architect should have an idea of the form and grouping of buildings, which he evolves and completes on drawings, and should explain the texture and type of finishes he wants to harmonize the building and the environment. The contractor then experiments with materials to provide samples of different finishes, done in different ways, until a mutual agreement is reached between the architect and the contractor that the finish shown by one of the samples is the correct one. This procedure is then repeated for each of the finishes suggested by the architect. While producing samples, the contractor is expected to actively explore different methods and materials, which will give the expression wanted by the architect, preferably using materials and methods indigenous to the country of the building.

This active co-operation is not always easy to achieve. The contractor may refuse to do work which he believes is not in his contract, and want everything precisely defined before he starts work on any area. This causes immense difficulty with Professor Gutbrod's work. It may be argued therefore that the standard forms of contracts are not the best way to manage this type of project. The necessity for a team with a common interest should be recognized.

On this particular contract there are other factors which have hindered the smooth application of this philosophy.

It is very frustrating for most members of the design team that, as non-Moslems, it is illegal to go on to site and see their creation taking physical shape. However, we do receive photographs and reports from our Moslem site staff, which keep us in touch with the state of the site. At present the job has reached a transitional stage. The structure is nearly complete, and the finishing trades are starting to become active, but the event which is eagerly anticipated, is the erection of the first of the cable roofs. It will cause a great deal of excitement, and possibly a few anxious moments, as the steel portal frame, 20 m high, is lifted up ready to receive the cables.

The erection will be later than originally programmed, and the story of the material for these roofs illustrates some of the difficulties a contractor in Saudi Arabia faces. The average time for materials to clear customs, measured from the time the ship or aircraft arrives in Saudi Arabia, is 35 days. The roof material was no exception. It spent six weeks floating in barges in the middle of Jeddah harbour, in spite of special efforts to get a quick passage for it through the port formalities. However, it has now arrived on site, and checking and preparatory work are proceeding.

It is during this time in port that damage is liable to occur. In a shipment of pressed board panels 80% were destroyed, during the only rainstorm this year. On another occasion electrical parts, urgently required for a broken tower crane, were air freighted from Paris to save the time of the six weeks sea voyage. However on arrival at Jeddah airport they were broken by an over zealous official, who moved them with a bulldozer!

The contracting work is being carried out as a joint venture; the technical expertise being provided by a French firm of contractors, and the labour by a Lebanese concern. At the beginning of the contract the French tried to supervise from outside the forbidden zone, but this proved impossible. They now have several Frenchmen in Saudi Arabia, who are Moslems, and these form the supervisory team, from project manager to trade foreman. Early this year, without warning, the Saudi Arabian religious police stopped these men from entering the Holy Zone, and thus prevented them going onto the site. The site stuttered to a stop, like a headless Hydra, and things looked desperate. However, once the seriousness of the position was pointed out to the client (the Government of Saudi Arabia), the religious authorities agreed that those Frenchmen who had 'Moslem Religion' stamped on their passports could enter the Holy Zone. This was quickly arranged and construction gathered pace again.

This is the sort of problem which causes the contractor far more trouble and money than the technical difficulties of the job, large as these are. These and other problems, place an immense burden on our resident staff, particularly as they are so far from the supervising office in London.

Trial structure at Tours

Lennart Grut

Mini-Mecca lies in the water-laden fields bordering the middle reaches of the River Loire in France, ironically not far from where Charles Martel in AD 732 at last put a stop to the northward drive of the followers of Mohammed. More precisely it lies behind Voyer's aluminium factory at Tours. Here, sustained by the cuisine and wines of the Touraine, both we and the contractors hopefully learned the essentials of erecting single-curvature cable roofs.

Initially thought of as a means to train Moslem erection personnel and to test the erection methods and handling techniques for the cables and fittings, the trial erection structure also turned out to be a very useful political weapon in our relations with the contractor. Up till the time of the trial erection we had spent innumerable fruitless meetings trying to persuade the contractor to treat the problems differently from conventional structures. It was only after the first attempt at erecting the cables had to be abandoned, owing to some most visible inadequacies, that our dialogue improved.

Perhaps the single, most important object of the trial erection, for us, was to convince ourselves that the roof could be prefabricated, transported to Mecca and erected there with a minimum of adjustment and without specialist supervision. To ensure this we felt it was necessary to have defined the exact sequence of erection, to anticipate what errors could occur at each stage and understand their effect on the overall structure, and, not least, to obtain an idea of what it would be like working 20 m in the air on a flexible medium. The problem of working on the cables was one to which we had given a lot of thought. This, together with the problem of having to absorb incompatibilities between adjacent cables arising from the geometry of the roof and inaccuracies in manufacture and erection, had determined our attitude to the methods we had proposed for the erection.

The trial erection structure itself was not a scale model of the auditorium roof; instead, we had designed an independent structure including as many of the peculiarities of the auditorium roof as possible, within our estimated budget. The cable sizes and cable fittings were full scale. The layout of the cables was chosen to illustrate the problems arising from varying cable lengths and changing roof shape, thus anticipating the inherently incompatible shape of the roof.

Before erecting the cables, the supporting steelwork would be hoisted and guyed in position. This we obviously could not adequately test on the trial structure. The first cables to be erected were the boundary cables, which would be hoisted individually and propped in their approximate positions.

Our proposed method of then erecting the roof was to preconnect large segments of the roof off the site with a sufficient number of transverse members to give the shape and stiffness necessary when erected. These units were to be packed and transported to the site where they would be hoisted directly into position from their cases. By this we would minimize the work to be done on site, especially in the air, at the same time ensuring compatibility between the cables. This method did require large cases and difficult manoeuvring on site and we might have damaged some of the transverse members.

The contractor, on the other hand, preferred to erect everything item by item on site. To obtain an approximate shape and sufficient

Fig. 1
Connecting cable to boundary cable

Fig. 2
The cable net with sandbags

stiffness, the cables would be loaded with sacks of sand and the transverse members placed one by one. He believed that it was cheaper and that the other problems could be tackled on site, although the prospect of working on the cables before any transverse members were attached did worry him.

Both the methods were tried out on the trial structure and both the methods in the end worked satisfactorily. Two simple but relevant points were clearly demonstrated. Firstly the stiffness of the roof is a direct function of the relationship between the load in the cables and the load you apply. Thus as long as there is a certain minimum load on the roof and major effects such as wind are accounted for, the roof will be stiff relative to the weight of the erection personnel. Secondly, and conversely, using the correct methods one can always apply a concentrated load locally sufficient to force a cable into its required shape; again the force required relating the load already in the cable. This indicated that, with the correct preloading of the cables, it should not be too difficult for workmen to force the roof into a compatible shape in the air.

The contractor has stuck to the item by item method, but, to reduce excessive work at the highest levels, it was decided to attach all the cables to the portal frame on the ground so that they were hoisted with the frame; the so-called spaghetti method. The preconnected system is being kept as a stand-by solution and, however much some of us would like to see it used, it being more in character with the medium of cables (and also, we believe, cheaper) we will be satisfied not to use it.

The final roof was then covered and proved invaluable in solving some of the very difficult cladding details at corners and joints. Standing under this small completed structure and trying to imagine the whole auditorium, I really felt quite attached to it, especially as I realized that this was to be the only Meccan roof I would ever stand under.

Figs. 1-11
Photos: Serge de Naglowsky (Thinet)

Fig. 12 right
Model of conference centre and hotel which was wind tested at the National Physical Laboratory (Photo: Lennart Grut)

Fig. 3
Attaching the double angles

Fig. 4
Attaching the timber panels

Fig. 5
Connecting the *Sillan* fibre glass panel

Figs. 6 & 7
Views of the roof with side walls

Fig. 8
Detail at the boundary cable

Fig. 9
Detail at the mast head

Fig. 10
Detail of the aluminium edge cladding

Fig. 11
Detail of aluminium roof cladding

The Riyadh Project

Ian Liddell

Architect: Trevor Dannatt
Main contractor: COGECO

The site

The site is situated to the west of Riyadh, towards the airport on the Chara Maathea. There is very little development on this street which was recently constructed as part of the city roads programme, and our site was completely clear of any previous buildings.

Geologically the ground is limestone covered with weathered limestone and fine silt. The latter becomes sticky when wet and a fine dust when dry. At about 1 m below the existing ground level, hard rock is encountered which continues down indefinitely. The Geological Survey of the area reported that cavities were quite usual in this rock and they mentioned four large sink holes in 80 km of Riyadh in the same strata. We arranged for a drilling team to probe the rock to verify that it was sound. When this was done we, in fact, only discovered small cavities in the rock.

The site has a 400 m frontage to the road, is 260 m deep and slopes down towards the road. It was covered with small hills and hollows, the difference in level being as much as 3½ m in 80 m.

The buildings

The main buildings are the conference centre and the hotel. In addition there is a mosque, three hotel staff villas, staff quarters and other service buildings.

The conference centre covers an area of 100 m² and includes a 1,400 seat auditorium, a large foyer on three levels which has space for exhibitions, five committee rooms of various sizes and the necessary offices and plant rooms. Outside the conference centre is a porte-cochere where the cars arrive and there are roads and ramps to the VIP entrance and car park.

The hotel has 200 bedrooms arranged in a W-shaped block on six storeys. Each bedroom has a north facing balcony on the outside, the entrances being from access balconies, which run round two internal courtyards in which the lounges are situated. The lifts are in the central point of the W and escape stairs are situated at the ends. The restaurant and administration offices are in a two storey area to the south of the bedroom block.

The mosque is at the front of the site next to the hotel entrance ramp. The villas are adjacent to the mosque.

The structure of the buildings

General

The buildings are constructed largely in reinforced concrete, blockwork being used as a partition material except for the two-storey buildings. To a considerable extent the concrete surface is used as the finished surface. Smooth, boarded, exposed aggregate and bush hammered finishes are used. In addition, the concrete details are all carefully and intricately thought out and the services are integrated into the building.

Expansion joints

Both the main buildings are over 100 m long and therefore require expansion joints. In addition, the extremes of temperature of the Riyadh climate imply a greater thermal movement than normally allowed for. We took a maximum length of 30 m for flat slabs on single columns and 15–20 m for areas more heavily restrained. The complexity of the buildings made it difficult to incorporate the joints without affecting the plan. This difficulty meant that some parts of the structure were designed specially for movement and the expansion joints always increased the complexity of the building details.

The Conference Centre structure

The auditorium is enclosed by concrete walls of 250 mm minimum thickness with piers to a total thickness of 450 mm. The seating steps are in in situ concrete spanning onto concrete beams or walls running longitudinally. The spaces below the steps and between the beams are sealed to form ducts for extracting the air from the hall. Where the back of the auditorium cantilevers over the foyer an in situ ceiling slab is cast below the beams to form these ducts. It was not possible to incorporate expansion joints in the auditorium structure which is 50 m square. A special casting sequence was therefore followed to reduce the effects of shrinkage.

Around the auditorium are the foyer spaces on the ground floor and one suspended floor. These suspended slabs are generally 300 mm deep and have expansion joints on the grid lines of the main quadripartite columns supporting the foyer roof.

The soffit of the foyer roof has been designed to give an expression of the moments in the segmented flat slab construction; the slab being at the bottom of the beams of the column and side panels to take the compression of the negative bending moments. The centre panel, being entirely in positive moment, is of coffered slab form. There is a lap joint between each panel to allow for temperature movement, so the centre panels are supported on four sides by the side panels which span onto the column panels. As the lap joints cannot take moments, eccentric loads on the column panels produce moments in the quadripartite columns. These moments are taken by direct up and down forces in the individual columns. The roof is covered by light weight steel sheeting laid to falls to a wide gutter around the auditorium which also serves as the escape walkway.

The roof over the auditorium is a two layer space frame on a square 4.33 m grid 3.06 m deep; the top nodes being centred on the square formed by the bottom nodes. The roof is 56 m square overall, supported on four columns. Each column is at the corner of a 39 m square and supports a top node. The top of the space frame is closed in by a concrete slab 100 mm thick. As the building is quite close to the airport, it was considered necessary to have this slab to provide sufficient noise reduction inside the auditorium. The space frame is constructed out of high yield steel grade UNI 52C equivalent to BS grade 50C. The members are all made out of angles in pairs or fours, each angle being bolted by one leg to a vertical plate at each node. The nodes therefore consist of eight vertical plates arranged radially, four having bolt holes for horizontal members and four having bolt holes for diagonal members. The plates are formed out of 20 mm or 30 mm high yield steel plate and are welded onto a 180 mm diameter tube with a 25 mm wall thickness at the centre and to horizontal top and bottom plates. There are 365 nodes in all and about 30,000 bolts. This design was arrived at after prolonged discussion with the contractor and was influenced by the need to use Italian steel sections and by the contractor's demand for fabricated sections which could be easily shipped.

The committee rooms

The committee rooms are in a separate structure from the remainder of the conference centre. The roof consists of pairs of beams 0.7 m apart at 3.25 m centre to centre. Each beam is 20 x 1.2 m and runs into a column of similar section at the window end. The other end is propped by the wall between the committee rooms and the foyer.

The hotel structure

The bedroom block is of concrete cross wall construction six storeys high. The whole of the hotel is set out on a 60° triangular grid including the bedroom walls, and so the setting out of the walls on plan becomes very complex. The bedroom cross walls have in some places to resist the horizontal forces from the sloping roof.

The hotel public areas are flat slab supported by hexagonal columns. The latter are on the 3.25 m triangular grid generally at 7.5 m c/c. The shapes of the individual pieces of slab are irregular. The highest of the public area roofs is a triangulated coffered slab with ribs on the 3.25 m grid. The space between the public area roof and the bedroom roof is covered with the sloping roof. This consists of skew half portal frames hinged at the top which support a series of concrete panels arranged to give indirect lighting to the space below.

Progress on site

Contractor's organization

COGECO is a subsidiary of the large property company Beni Stabile. The Riyadh contract is in the charge of the small and recently formed foreign division and they also have the contract for a smaller building, the radio centre, also in Riyadh.

The contractor started work on the site in April 1969 with a foreman and a few men camping in the desert. There are now more than 600 people employed on the site including the Italian staff, foremen and tradesmen from the nucleus of the labour force, the remainder being local men. The Italians live

Fig. 1
Architect's drawing for conference centre, Riyadh

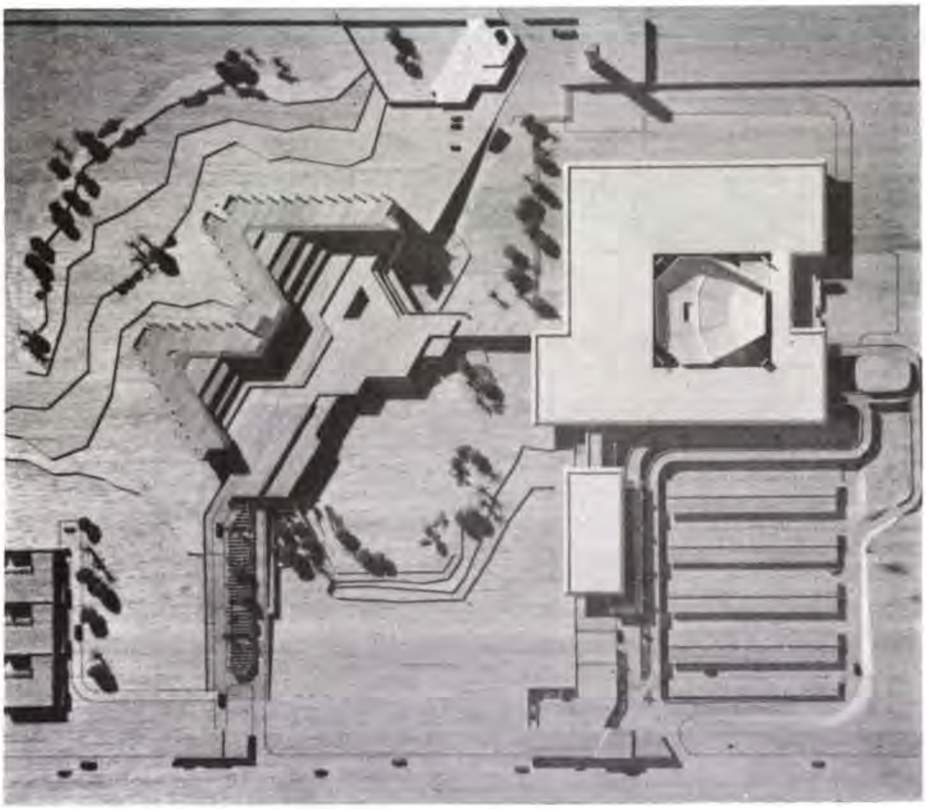


Fig. 2
Model of Riyadh project
(Photo: Mann Brothers)

Fig. 3
Plan view of Riyadh model

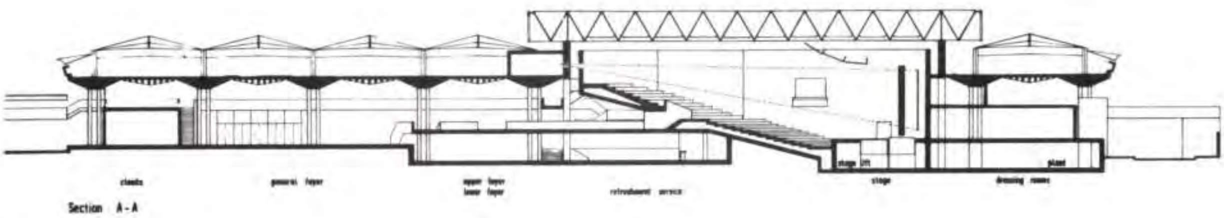
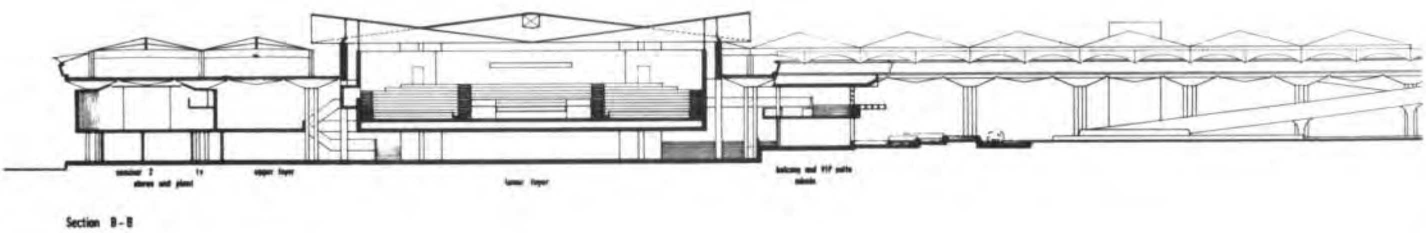
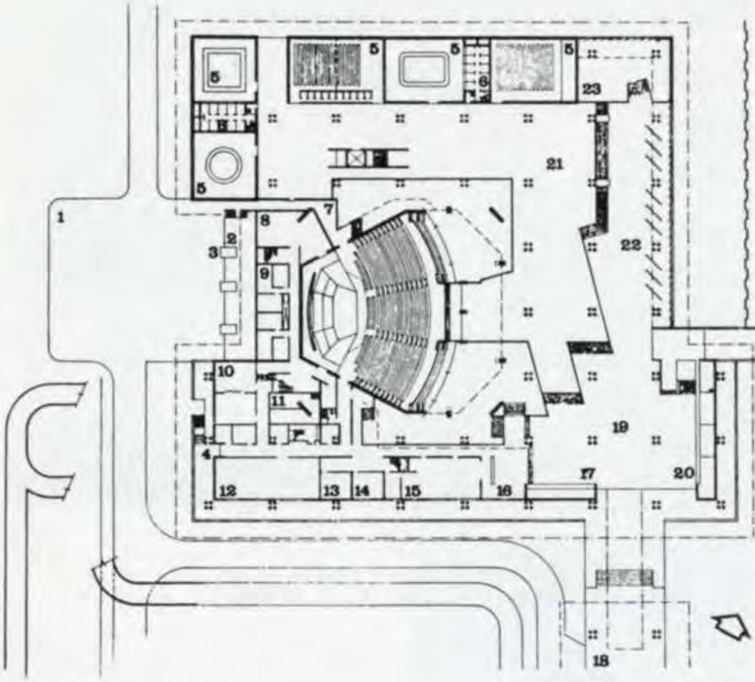


Fig. 4
Conference centre, Riyadh: Section A-A

Fig. 5
Conference centre, Riyadh: Section B-B





Key
1 service yard
2 loading bay
3 air intake
4 staff and artists' entrance
5 committee room
6 interpreters
7 escape
8 orchestra store
9 dressing rooms
10 dressing room
11 stage cloaks
12 temporary offices
13 assistant director
14 director
15 general office
16 enquiries
17 ticket office
18 *porte cochere*
19 general foyer
20 cloaks
21 upper foyer
22 exhibition centre
23 coffee bar
24 link to hotel

Fig. 6
Riyadh: lower-level plan of conference centre

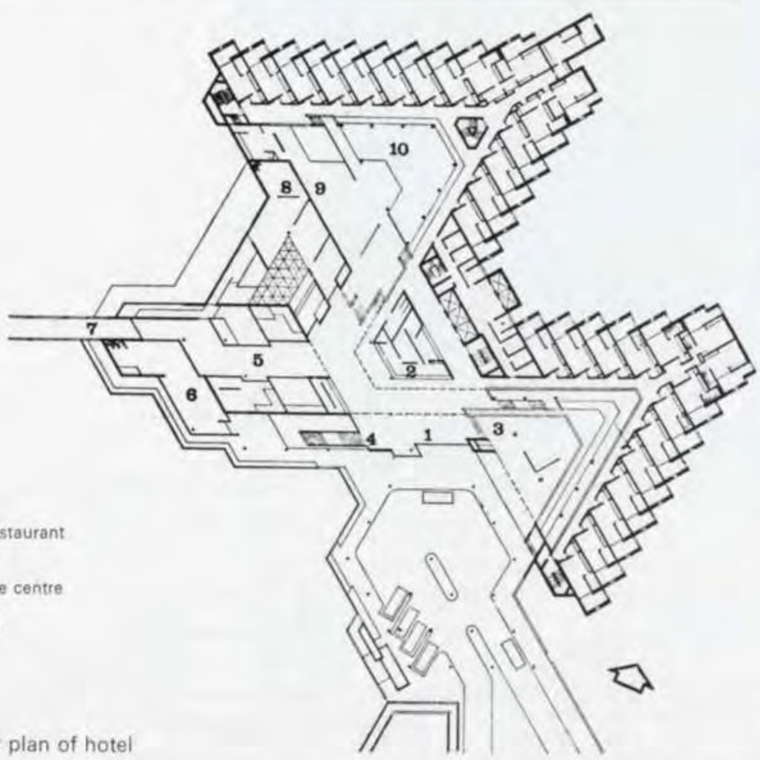
in a well-appointed camp at the back of the site where there is a swimming pool surrounded by well-cared for grass and bougainvillaea. Films are shown most evenings in the cinema or recreation room. The most important feature of the camp is the canteen where the best pasta in Riyadh is served.

One of the first things to be established was the metal workshop which is fully equipped with a huge lathe and a planing machine as well as the usual tackle. In this workshop they have made bogies for transporting concrete and parts for the tower crane, as well as many special shutters and temporary supports which have proved to be a great success.

Off-site work

The structural material most readily available in Saudi Arabia is concrete. The sand and aggregate is gleaned from the desert and there is a cement factory in Riyadh. Certain other allied materials are produced locally. There is a gypsum plaster factory and there are assorted concrete block factories varying from the hand tamped mould in the back yard to a brand new totally mechanized factory capable of producing first class blocks. All other materials are imported and much of the work of the contractor's head office in Rome is related to the supply of imported materials.

Some materials, such as marble, arrive in bulk and are cut and polished in Riyadh. Others, like joinery, are worked into the finished articles off site. The most important structural item being fabricated off site is the steel-



Key
1 hotel entrance
2 reception
3 waiting lounge
4 stairs down to restaurant
5 shopping arcade
6 breakfast room
7 link to conference centre
8 administration
9 residents' lounge
10 internal garden

Fig. 7
Riyadh: first floor plan of hotel

Fig. 8
Riyadh: view from conference centre
(Photo: The contractors)

Fig. 9
Riyadh: back of service core
(Photo: The contractors)

Fig. 10
Riyadh: conference centre, foyer roof
(Photo: The contractors)

Fig. 11
Riyadh: general view of auditorium
(Photo: The contractors)

work for the auditorium roof. This is being done at the Metalfor workshop outside Rome. The fabrication of the nodes from high tensile steel plate requires careful welding and accuracy of assembly. When there is such a quantity of welding to be done in this country it would be normal to have welding inspectors in the fabrication shop to watch the work at all times. This is generally admitted as being the best way to ensure that all the welding is up to standard. In Italy the practice of employing welding inspectors is not so common and it proved difficult to find an independent firm who could supply this service. Eventually we discovered that Lloyds Register had offices in Milan and Naples who were able to provide this service and we persuaded the contractor to employ them.

Work on site

In Italy it is not usual to leave the concrete surface as the finished surface. The custom is to build a rough form with *Blocko* linings and cover everything with applied finishes. As a result the resident engineer experienced some difficulty in persuading the contractor's staff to produce the desired standards of concrete finish. Once they got the idea of what was wanted, the Italians' pride and skill came to the fore and the results are generally very good. Certainly in the conference centre, where there is a great deal of board marked concrete finishes, there is a uniformity in appearance which provides the desired results.

For the smooth finish work, where repetition warrants it, the contractor fabricated steel shutters in his site workshops and these give excellent results. *Wisa form* ply is also used when it is available. For other parts of the work the contractor has also fabricated special shutters or used precasting techniques to cut down on the labour of stripping and fixing shutters. In particular the bedroom walls are cast in purpose-made shutters which are handled by crane and the side and centre panels of the foyer roof are cast on formwork suspended from the column panels.

Progress

The contract period is three years from July 1969. Now, two years later, the contractor has nearly reached the end of the structural work. We expect the roofs to be completed by October. The contractor's rate of progress has never been sufficiently greater than the required average to give him sufficient in hand to cover the difficult periods when production drops. There is the month of Ramadan when the Muslims fast from sunrise to sunset without even taking any water. During this period production is halved. Then there are always difficulties in importing materials; there are delays in shipping, customs or loss of documents and the lack of materials sometimes delays the site. As a result the contract is now about three months behind on average and whilst the contractor is making efforts to catch up, it is unlikely that he will do so.

Services, lifts and lighting in car parks

Derek Ball and Hans Nordentoft

Introduction

The detailed design of mechanical and electrical services for car parks has been dealt with comprehensively from time to time in a variety of technical papers.

The purpose of this paper is to indicate some general principles related to engineering services in car parks and offer some suggestions for improvement in the services engineering content as it affects the environment.

A car park is part of our environment and, just as it has external features, it has its internal features, in which the services play an important role.

It is certain that the environment will benefit in the coming years from the rising general wealth and standard of living. People want clean, nice looking and comfortable surroundings. How can the services engineer contribute to this?

First of all, he must join in the basic planning at a very early stage to allow the services to be integrated with the architectural and structural elements of the building. Only in this way will it be possible to get both an overall economic solution in relation to investment and running costs and, at the same time, a more attractive end result.

The services engineer, in his whole attitude to design, must also try to influence the daily running of the car park, by ensuring that the ventilation and lighting systems are efficient, reliable and easy to operate, and that the sanitary systems are easy to clean and maintain.

Lighting

Good lighting, both in the interior and along access routes, is essential if car parks are to present a reasonably attractive appearance commensurate with the environment normally enjoyed in associated or adjacent buildings. Many present day car parks present a rather shabby, badly illuminated appearance to which one feels no attraction to park, particularly in relation to the security of the vehicle and its contents. In a number of parks visited it was most revealing to find that the degree of vandalism directed against fittings and equipment appeared to be a function of the illumination provided.

A guide to the levels of illumination which should be provided is contained in *CP 1004: Part 9: 1969*,* but it is important to remember that the quantitive criteria are recommended minima only and that the whole subject of lighting levels, glare and design appearance is rather more complex than is generally appreciated and is under constant review, resulting in progressive improvement in these standards. Any installation should, therefore, be designed to anticipate this trend, so far as is practicable within the cost constraints imposed, by providing either a high standard initially or by building in the necessary flexibility to permit future improvement.

Car parks warrant far greater attention and expertise in the design of lighting than they are presently attracting if the best possible use is to be made of materials and finance for improving the visual scene.

Current lighting practice is to generally provide surface mounted batten fittings with bare fluorescent tubes. These fittings are usually low cost units suitable only for installation within temperature controlled, relatively clean air environments. These unprotected fittings corrode and their performance deteriorates quickly with accumulations of dirt, insects, and moisture in the more aggressive environment found typically in car parks. One should question whether something better could not be provided, especially since the fittings themselves are so vulnerable to vandal attack. In this regard, the possibility of using tubular fluorescent or high pressure colour corrected mercury lamps in optically controlled sealed fittings recessed into the slab offers both visual and practical advantages, albeit at some extra cost.

In the context of improved lighting, a very significant contribution could be made to the visual scene by providing light coloured concrete surfaces to improve the ceiling reflection and hence background luminance. It is obviously impractical and also undesirable to attempt to provide even illumination over the whole area, due to height and functional limitations.

In particular, lighting of access points and ramps should be to a higher standard than elsewhere and can be employed as route markers by suitable arrangement and possibly by optical control of the fittings.

Back illuminated advertisement displays fixed on the walls can yield revenue and enhance the appearance by adding interest, colour and lighting to the walls.

The installation of illuminated informatory and directional signs is a complementary but essential extension of signing which, from observation, is rarely given the prominence which it warrants in relation to route marking and space availability.

Secondary supply

A secondary electricity supply should be considered an essential part of the services installation. This requirement can be satisfied by either duplicated incoming supply feeders provided by the Electricity Board, normally stationary generation plant, or batteries including a combination of two or more of these alternatives.

Whichever alternative is adopted it is essential that operation of the secondary supply is completely automatic on mains failure in order to maintain continuity of essential service installations including lighting, fire alarms, communications and mechanical ventilating plant necessary for the preservation of safety and health.

Lifts

It is common to find that lifts in car parks are undersized, poorly illuminated and provided with utility finishes giving rise to a claustrophobic atmosphere. Normally lifts should be rated for not less than 10 persons which would involve relatively small extra cost over the 6 or 8 person lift more usually employed.

Lifts should be of the residential type with a deep platform so as to accommodate a wheel chair or shopping trolley, be cheerfully illuminated with attractive anti-vandal finishes and operate at a speed properly related to the traffic flow and travel.

Communications

Car parks are often provided with a limited voice communication system which is usually of the cheapest possible type, commonly known as squawk boxes. It appears from observation that the staff do apparently become accustomed to interpretation of the gross distortion of speech emanating from these boxes, but it is suggested that a system giving an acceptable standard of reproduction could be provided at modest extra cost.

The system could then also be usefully employed to communicate with users at all levels above or below ground, or perhaps even provide pre-recorded background music commonly used in department stores.

Closed circuit television would be complementary to a sound system and might in the future prove to be a justifiable expense in providing supervisory scanning over the parking areas on all levels, working in conjunction with a visual and/or voice direction system. Such an installation would also provide facilities for exercising maximum security from a central control position.

Ventilation

Ventilation is required to avoid risk of fire and explosion arising from petrol fumes and to

*BRITISH STANDARDS INSTITUTION. CP 1004. Street lighting. Part 9: 1969. Lighting for town and city centres and areas of civic importance (Group G). BSI. 1969.

prevent injury to health from engine exhausts.

As mechanical ventilation is expensive to install, natural ventilation should be provided wherever possible. This is normally the case in multi-level car parks above ground where adequate cross ventilation through openings in outside walls is obtainable.

Statutory regulations require, to ensure the effectiveness of natural ventilation, that the minimum free area of ventilation openings is 2½% of the garage floor area so arranged as to induce cross-ventilation.

For light traffic, three changes of air an hour is regarded as sufficient and for more intense traffic six air changes per hour are required. This means that enclosed car parks, both above ground and underground, have to be provided with mechanical ventilation, or a combined system including natural ventilation. Normally one third of the extraction is taken at high level and two thirds at low level.

The supply of fresh air is normally ensured by using gates of the open lattice type with the ventilation system itself divided into two separate sections, each providing half the required air changes. The sections should be controlled independently and automatically so that continuous running is ensured in the event of a failure in one.

To achieve a better environment we suggest that more attention is paid to the following points:

1 All enclosed car parks should be provided with a system capable of operating at two speeds, thereby giving at least three air changes at all times and increasing automatically to at least six air changes per hour when called for by air sensitive devices.

2 Air changes per hour provide useful guidance, but should not inhibit the designers in their thinking of how air is to be provided and distributed. The fresh air intake is normally the car park entrance which is not a good position as it is the site of maximum exhaust discharge and is the prime area of occupation for attendants and customers and should therefore be avoided.

3 All ductwork should be arranged so as to relate clearly to the building fabric and the other services ducts and inlets at floor level should be protected against damage from vehicles.

4 Outlets should be situated away from the openings in the building to prevent recirculation.

Heating

Generally the heating system in car parks is ramp heating which should be installed normally in cases of exposed gradients exceeding 6%. MICC heating cable to provide 150-200 W/m² is undoubtedly superior to any other alternative for this particular application and it is essential that a comprehensive control system be provided if the heating system is to work effectively.

Alternatively, salt can be used to combat ice and snow but the difficulty in storage and application coupled with the possibility of damage to the structure and vehicles will often justify the extra expense involved in the electrical system.

Normally the car park itself is not heated but underground parks are to a certain extent warmed from the surrounding earth and heat from adjacent buildings is supplemented, if necessary, by a unit heater to temper the incoming air.

Toilets, waiting and staff rooms are most important places from the point of view of people working in and using the parks and should, in our opinion, be provided with heating and ventilation to a higher standard than is presently considered acceptable.

Fire protection

The requirements for underground car parks are clearly defined by the licensing departments of the local authorities who can require any standard of fire precautions that they deem to be necessary in any particular circumstance.

Basically, the requirements are for automatic sprinkler installations and back-up first aid fire protection in the form of hose reels, hand extinguishers and buckets of sand. Sprinklers are not normally required in above ground levels, with the exception of drenchers at the edge of the car park when the building is close to neighbouring properties. A dry riser for the use of the fire brigade is sometimes required.

Whilst this paper is primarily concerned with the environment, we would like to add some suggestions regarding fire protection in underground car parks.

It is apparent that there is a greater risk to people in underground parks than to those in above ground parks but the requirements for above and below ground appear to be disproportionate. We feel that fire fighting requirements, particularly below ground, could stand re-appraisal, based partly on the findings published in *Fire and car park buildings* (Fire Note no. 10) issued by the Ministry of Technology and the Fire Officers' Committee Joint Fire Research Organization. Although referring to open sided parks it concludes that the fire load in car parks is considerably less than was previously thought. It also states that an outbreak of fire within a single parked vehicle is unlikely to result in uncontrollable fire spread, even without the assistance of the fire brigade. In any event we feel that a re-appraisal is necessary since, in our opinion, water is the wrong agent with which to fight petroleum fires unless used by experts, i.e. the fire brigade.

The provision of sprinklers in underground car parks would appear to provide less protection than was first thought, the reasons being that, as previously noted, water is not the best agent for fighting garage fires, and the sprinkler discharge will not get at the seat of the fire inside the vehicle but will only cool the external surfaces of the car and soffit of the structure.

Money spent on providing sprinkler installations could well be put to more effective use, possibly by providing detectors, linked directly to the fire brigade, which would give warning of fire long before the temperature built up sufficiently to activate a sprinkler head.

In our review of existing car parks, hose reels were provided, but it would seem that their prime function was for washing down floors and ramps and many were in a sad state of repair.

We suggest that the most effective way of fighting fires by untrained persons is by means of hand fire extinguishers, which should be provided on a generous scale and be carefully maintained. Any fire not capable of being dealt with by an extinguisher should be left to trained personnel. Buckets of sand, despite their continual misuse, are a very real asset when dealing with spilt petrol.

The requirements for smoke outlets in underground car parks are less clearly defined than most facets of design. The Greater London Council's Code of Practice asks for an adequate number of smoke outlets and their positions suitably indicated. Discussions with Fire Prevention Officers at various times suggest that the most satisfactory solution would be to provide controllable smoke outlets that can be regulated by the fire brigade. It has been suggested that the mechanical extract system could be used as smoke outlets by providing a fireman's control in a safe place.

Drainage

The requirements for drainage of underground car parks are clearly defined by the licensing departments of the local authorities, based upon the requirements of the Petroleum (Consolidation) Act 1928.

Briefly the requirements are that no spilt petrol is to lay on the floor, with gullies to be provided at set distances and the floors to be laid to falls to them. The gullies are usually required to be trapped and sometimes to have silt buckets and the entire petrol drainage system has to pass through a petrol interceptor before discharging into the public sewer.

With garages above ground there are no general regulations regarding petrol drainage although some local authorities do have some requirements. In effect, petrol can lay in the floor of naturally ventilated car parks until it evaporates (or catches fire).

It is essential to provide some floor drainage at the edge of open sided car parks to receive any blown-in rain and to ensure that the floor slab is waterproof and falls to the outlet. This will avoid water dripping through cracks in the floor, picking up impurities on the way and dropping onto the cars parked below, possibly causing damage to the paintwork.

Sanitary accommodation

From our visits to existing car parks it was apparent that normally the standard of sanitary accommodation provided was minimal, and that the standard of cleanliness and general decor left a great deal to be desired. Whilst it is appreciated that toilets do not help to make the car park more profitable and are targets for vandals, we feel that in the interests of people, better toilet facilities with hot water, soap and towels, conscientiously maintained and cleaned, are essential to the general environment.

Operation and maintenance

Within the parks we have inspected, the standard of plant maintenance is often poor and it is worth recording that some operators in particular resorted to switching off ventilation and lighting, including the removal of some fluorescent tubes, in order to effect economies in electricity consumption. In one car park, an enquiry revealed that the staff below ground found the absence of air movement quickly promoted intolerable working conditions.

In our opinion it is essential that a planned maintenance programme is instituted and adhered to throughout the life of the installations.

Whilst we have not mentioned mechanical car parking it is worth stating that full-time attendance of a first class technician is essential to maintain the mechanical plant whilst the park is in use. This may imply a shift system of working and, where staff are employed on the premises, adequate workshops and rest room facilities should be considered a necessity.

Summary

The aim of this paper has been to look both into the present situation and also into the future, but undoubtedly the desire for better, people-orientated facilities in all aspects of life will become an increasingly important part of the rising standard of living. To this, one day could be added a competitive element between neighbouring car parks, so that besides better standards for lighting, ventilation, heating, lifts, sanitation, etc., could come, for example, a development of waiting rooms with communication facilities, etc. What about television for parked children? Has anyone asked the customers what they want or feel about car parks? Talking to a lady parker we received the immediate reply that she had too far to walk, often in unpleasant surroundings, and that there were inadequate lifts and toilet facilities.

Even though cost has not been mentioned very much in this paper it is obvious that it must always be the aim to get the best possible car park, operating as smoothly as possible at the most economic price. This can, in our opinion, only be obtained through a totally integrated engineering service working in the closest possible collaboration with the clients and architects.

An alternative approach to calculation of soil movements in deep excavations

Alistair Day

As the project to which this work will be applied is being designed in imperial units and the experimental data obtained to date are also in imperial, no metric equivalents are being given in this article.

The state of the art

A lot of work has been done in the past on developing various finite element programs for elastic analysis of plane stress and plane strain problems.

These methods are at present used to calculate stresses and displacements in soil profiles such as deep excavations, etc., despite the fact that they cannot adequately allow for two of the most essential phenomena in certain soils, particularly clays, i.e. the porewater pressure and the non-linear stress-strain relationship.

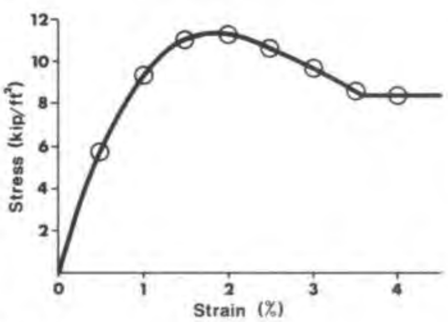


Fig. 1
Total stress-strain curve

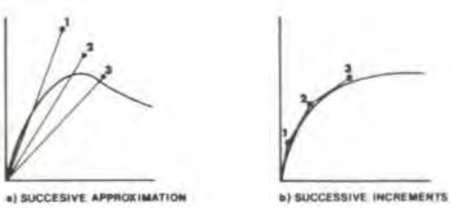


Fig. 2
Alternative methods of calculation

The problem of the porewater pressure is usually dealt with by working with total stresses and assuming a Poisson's ratio of almost 0.5. The non-linearity is typified by the stress-strain curve for an over-consolidated clay, shown on Fig 1, which also indicates the characteristic trait of a drop in strength from a peak value to a lower figure at higher strains, the falling branch of the diagram.

Duncan and Chang[1], describe an approach in which the stress-strain curve is taken as a hyperbola, but this explicitly excludes the possibility of calculations which take account of the falling branch of the stress-strain diagram. There appear to be no published details in the UK of generally available programs which allow for these factors.

Current methods of solution

To use the linear elastic programs for non-linear problems, two approaches are possible: either by successive approximations using the full load, or successive increments of load up to the full load. These two methods are shown diagrammatically in Fig. 2. After each approximation or increment, the elastic coefficients are modified, and the linear program re-used for the next stage. Where there is a drop in stress from a peak, the incremental method cannot be used, as the calculation cannot follow a falling stress-strain curve.

In summary, the methods adopted for non-linear work have been to modify the linear analysis and the effort has gone into suitable linearization of the elastic coefficients.

Alternative method proposed

The alternative of using iterative methods operating directly on the non-linear soil parameters does not seem to have been used so far. Dynamic relaxation had been found to have the capacity to accept any form of stress-strain relationship so it was a logical extension to apply this solution technique to the soil mechanics plane strain problem. Given a sufficiently adaptable program, no limitations needed to be imposed on the equations describing the soil behaviour and therefore any relationships found experimentally could be reproduced in the calculations.

Equations proposed for overconsolidated clay

For the specific problem of the immediate deformation of over-consolidated clays, the equations had to reproduce the effects of the porewater pressure and the non-linear stress-strain curve of the soil.

The most obvious set of equations related the horizontal (x) and vertical (y) effective stresses to the respective strains, the shear stress to the shear strain, and assumed the pore pressure to be a function of the volumetric strain. This latter assumption was doubted at one stage but appeared justified by the later results.

$$\sigma_x' = f(\varepsilon_x, \varepsilon_y)$$
$$\sigma_y' = f(\varepsilon_x, \varepsilon_y)$$
$$\sigma_{xy}' = f(\varepsilon_{xy})$$
$$p = f(\varepsilon_v)$$

Where σ_x', σ_y' = effective x and y stress
$\varepsilon_x, \varepsilon_y$ = x and y strain
σ_{xy}' = shear stress
ε_{xy} = shear strain
p = pore pressure
ε_v = volumetric strain

The total stresses at a point in the soil (σ_x and σ_y) equalled the sum of the effective stresses i.e.

$$\sigma_x = \sigma_x' + p$$
$$\sigma_y = \sigma_y' + p$$

These equations were tried but did not seem to be able to give the correct form of the stress path described later. (In retrospect it appears that a different set of coefficients for the functions may be more successful and it may therefore be possible to use these equations later.)

Looking at the way in which the failure develops in a clay specimen it appears that equations relating the effective average stress, the effective shear stress and the pore pressure to the shear strain and volumetric strain would be the most promising, i.e.

$$\sigma_a' = f(\gamma, \varepsilon_v)$$
$$\tau' = f(\gamma, \varepsilon_v)$$
$$p = f(\varepsilon_v)$$
$$\sigma_a' = \frac{\sigma_1' + \sigma_2'}{2}$$
$$\tau' = \frac{\sigma_1' - \sigma_2'}{2}$$

Where σ_1', σ_2' = principal effective stresses
γ = shear strain
ε_v = volumetric strain

Although other forms of the equations could have been used, the easiest one to try first was a set of polynomials:

$$\sigma_a' = D_0 + D_1\gamma + D_2\gamma^2 \ldots D_n\gamma^n + E_0 + E_1\varepsilon_v + E_2\varepsilon_v^2 \ldots E_n\varepsilon_v^n$$
$$\tau' = G_0 + G_1\gamma + G_2\gamma^2 \ldots G_n\gamma^n + H_0 + H_1\varepsilon_v + H_2\varepsilon_v^2 \ldots H_n\varepsilon_v^n$$
$$p = K_0 + K_1\varepsilon_v + K_2\varepsilon_v^2 \ldots K_n\varepsilon_v^n$$

Fitting to the experimental stress-strain curves is easier if the polynomials are made discontinuous. It should be noted that the coefficients cover all the elastic constants, e.g. for linear elastic calculations the coefficients E_1 and G_1 can be related to Poisson's ratio and the Young's and shear moduli respectively.

The computer program

A finite element program was written to incorporate the equations. The polynomials were restricted to cubics as this gave sufficient degrees of freedom for the initial investigations.

In its final form this program uses rectangular elements. In typical sections which occur in design, rectangular elements tend to form grids, so provision was made for changing of the grid spacing between groups of elements. Uniform-stress triangular elements were tried but, as they behaved in a very unsatisfactory manner in some cases, their use was abandoned. When the opportunity arises, more work will have to be done to determine the suitability of the various types of elements.

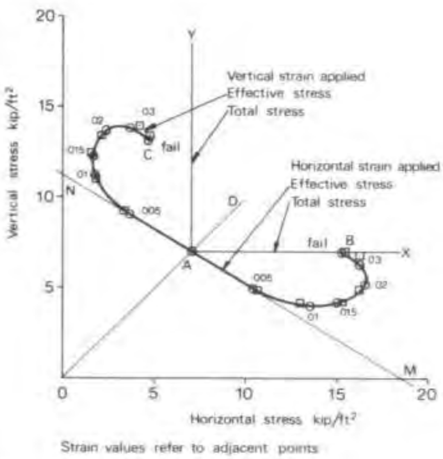


Fig. 3a Equal initial stresses

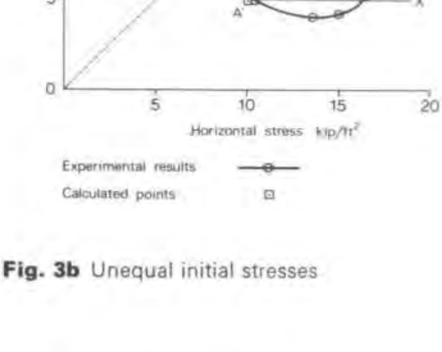


Fig. 3b Unequal initial stresses

Figs. 3a & 3b Stress paths

Testing of the program

When the program had been written it was tested to show that:

(a) it reproduced known answers sufficiently accurately, and that

(b) the non-linear equations reproduced experimental data from soil tests.

For the linear elastic case, test calculations were made for various simple structures and satisfactory results were generally obtained, confirming that it satisfied criterion (a) above.

The stress paths for soil samples taken from a site in the City were chosen for testing the program regarding criterion (b). The stress path in Fig. 3a is a graph of the two principal stresses for London clay at about 40 ft. depth with the strains marked on it. If, in the un-undisturbed state, a specimen of clay is at, say, Point A (equal vertical and horizontal stresses) and it is then strained, the two stresses will always correspond to values given by a point on the graph.

When comparisons are being made between calculated and experimental results, it is essential that the relevant constants are derived, and that these are the correct ones for the actual excavation. The experiments were carried out on 3 in. cubes in triaxial equipment, modified for plane strain and the data obtained from them were the average stress, strain and pore pressure for the samples.

If we were investigating the behaviour of a soil sample, the calculations would use elements which formed a fine grid, say 10 x 10 elements of 0.3 in. sides, to represent the 3 in. sample. When constants had been derived so that, with strains applied to the grid of elements, the total experimental stress strain curve is correctly reproduced, then the constants would be those of the 0.3 in. x 0.3 in. segments of soil. This is essentially the approach used by Perlof and Pombo[2]. However, in the calculations for any actual excavation, the smallest elements will be of the order of 2 ft. 6 in., and it is unlikely that the properties obtained for the 0.3 in. elements would represent the properties of 2 ft. 6 in. elements.

It is more likely that the average values obtained from the experiments would be the more appropriate. For this reason the calculations to simulate the experimental results were made on a single element of 3 in. square section so that, when the constants were derived, they were the average constants for a soil volume of 3 in. x 3 in.

From the σ' and τ' curves derived from the stress path in Fig. 3a the following equations for the soil parameters were found:

$$D_1 = -41.6 \quad D_2 = 12861 \quad D_3 = -260500$$
$$G_1 = 718 \quad G_2 = -28300 \quad G_3 = 324600$$
with a discontinuity at $\gamma = .036$ giving
$$G_0 = 4.24 \quad D_0 = 3$$

and the remaining coefficients equal to zero.

Data using these soil parameters for a single element were set up and the experimental work was simulated by applying increments of deflection (equivalent to increments of strain) to the element. For the first test it was assumed that the vertical and horizontal effective stresses were both equal to the value of 7 kip/ft.^{2} at point A (Fig. 3a). As the soil was orthotropic, tests with both vertical and horizontal strains were made.

The effective stresses calculated for each strain level are plotted on Fig. 3a. As arbitrary constants, which may be adjusted indefinitely are used, then in principle the calculated curve may be made as close to the experimental as desired. As the discrepancies between the calculated points and the curve on Fig. 3a are of the order of the experimental accuracy, there seemed little point in modifying the coefficients. It should be noted that this fit was obtained using cubic coefficients only.

In the experiment, when the sample was being strained in one direction, the free face moved with a constant total stress on it. This meant that the total stress paths were vertical or horizontal lines (AY and AX) as shown on Fig. 3a and the difference between the effective and total stress paths was equal to the pore pressure. The calculated total stresses were on the lines AY and AX with the pore pressures equal to the experimental pressures.

As London Clay can have initial horizontal stresses which do not equal the vertical ones, it is important that any calculation for an actual design case takes this into account. This meant that the stress path must be followed from whatever initial stress point represented the soil conditions. To test this a second run was made with the initial stress corresponding to point A' (Fig. 3b) (vertical effective stress = 5 kip/ft.^{2}, horizontal = 10 kip/ft.^{2}).

Again, vertical and horizontal strains were applied and the resulting values are plotted on Fig. 3b which shows satisfactory accuracy. From these calculations criterion (b) appeared to be satisfied.

Having obtained a calculated curve which was sufficiently close to the experimental stress path, it was interesting to plot various curves which could be obtained from the calculations. First a plot of the total stress against axial strain was drawn, as shown on Fig. 1, and gave a typical shape for over-consolidated clay. This seems to be a significant result as this curve was not directly related to the input data.

Plots of the vertical and horizontal effective stresses against strains are shown on Fig. 4. The tangents to the two curves at the origin would be the elastic moduli, if the clay was linear elastic, and the corresponding stress path would be the straight line NAM on Fig. 3a. If the clay is orthotropic the vertical and horizontal moduli are not equal and the ratio of the vertical to the horizontal modulus depends on the inclination of NAM to the diagonal OD. (If NAM is normal to OD, the moduli are equal.) For the inclination of NAM on Fig. 3a the ratio was 1.66. It was found that the ratio of the slopes of the tangent to the calculated curves at the origin in Fig. 4 was 1.66. On Fig. 3a it can be seen that there is a section of the stress path adjacent to the point A where the soil has linear constants and

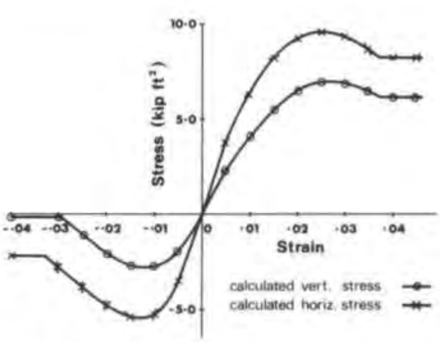


Fig. 4
Calculated vertical and horizontal effective stresses

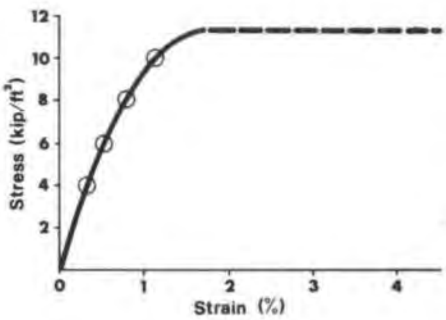


Fig. 5
Total stress strain curve for applied stress

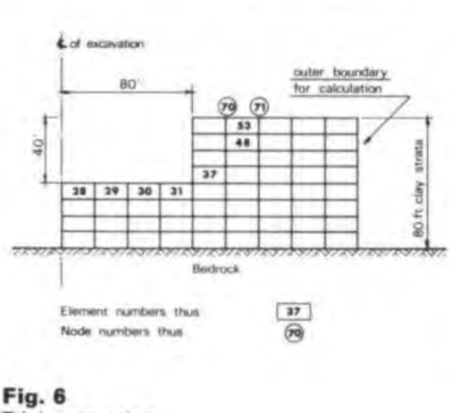


Fig. 6
Trial excavation

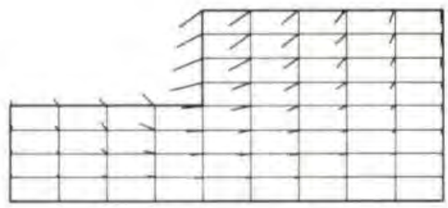


Fig. 7a
Deflection

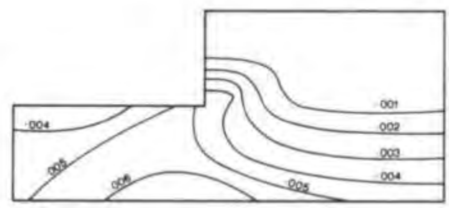


Fig. 7b
Shear strains

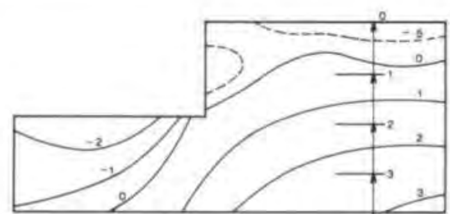


Fig. 7c
Pore pressures (kip/ft.^{2})

this is reflected in the portions of the curves adjacent to the origin on Fig. 4. An unexpected feature was the small strain at which the effective stress curves on Fig. 4 departed from the tangents at the origin. This was as low as 0.005 for one branch of the vertical stress-strain curve.

When the sample is being strained, by applying strains in one direction, the strain in the other is nearly equal to the applied strain but of opposite sign. On Fig. 4 the vertical stress reaches a minimum at −.01 strain. For applied horizontal strains between .01 and .018 the calculations imply a negative modulus (i.e. rising stress for decreasing strain) for the vertical stress despite an increasing total stress-strain curve (Fig. 1).

The curve on Fig. 1 is plotted from the total stress obtained for applied strains. In the converse case, i.e. when stresses are applied, the resulting strains are shown on Fig. 5. This curve is identical to the curve on Fig. 1 up to the maximum, but beyond this point the element collapses producing large strains. In a multi-element calculation, such elements will shed part of their load to other elements until their stress drops to a reduced value, unless, as found in a later section, a continuous zone of failed elements develops which constitutes a slip surface.

A trial calculation

Before embarking on calculations for an actual design, a trial calculation for a simplified excavation was made.

The strength of London Clay has been found to increase linearly with depth. However, for the trial calculation it was decided that it would be more instructive to take a uniform material so that the varying strengths would not affect the stress pattern. An 80 ft. thick stratum, assumed to be resting on rigid bed-rock and having the properties obtained from the single element calculations, was used. A 40 ft. deep and 80 ft. half-width excavation shown in Fig. 6 was assumed to be made sufficiently rapidly for undrained conditions to persist in the clay.

To calculate stresses and deflections for this excavation, the elements shown in Fig. 6 were used. This figure also shows node and element numbers referred to later. The boundary conditions were taken as fixed in both directions at the top of the bed-rock and fixed in the horizontal direction at the centre line of the excavation and at the outer boundary.

The initial stresses before excavation were assumed to be:

1 Pore pressure equal to three-quarters of the hydrostatic pressure

2 Effective horizontal stresses equal to twice effective vertical stresses.

These initial stresses approximate to those of the actual site. The pore pressure and total stress, which equals the total overburden weight, at the mid-height of each layer of elements, was found and the effective vertical and horizontal stresses derived from them. The initial pore pressures are shown on Fig. 7c.

Before excavation the stresses and the self-weight are in equilibrium. After excavation the calculations assume that there are unbalanced vertical stresses on the bottom of the excavation and horizontal ones on the sides and these cause movement of the soil. (An alternative way, which is sometimes used, is to assume external forces applied to the nodes around the perimeter of the excavation, with zero initial stresses in all elements[3]. The external forces equal the weight of the soil removed and the unbalanced horizontal stress. This is possibly satisfactory if the initial vertical and horizontal stresses are equal but if they are not equal, e.g. Pt.A' (Fig. 3b), then there is less strain capacity to failure if the soil is strained horizontally, i.e. along the path A'B, than vertically along A'C. Assuming that zero initial stresses implies equal strain capacity in both directions, the calculated failure load for (say) a surcharge adjacent to the excavation could be different from that calculated using unequal initial stresses.)

The results of the calculation gave all the horizontal and vertical stresses and strains, the principal stresses and strains, the deflections and pore pressure. The most interesting of these are the deflection, shear strains and pore pressure which are shown on Fig. 7a, b and c.

It can be seen on Fig. 7a that there was a heave on the bottom and an inward movement of the sides with the remaining deflections forming a consistent pattern. As the weight of the soil adjacent to the excavation was supported before excavation, the downward movement of the vertical face and adjacent soil was due to the reduction of the pore pressure near the face, seen on Fig. 7c.

The maximum shear strain was 0.0066 in element 37. From Fig. 7b it is seen that most of the elements had a shear strain of less than 0.006 so that they would be in the linear stress range and have small changes in effective stresses. This was because most of the load was carried by changes of pore pressures. The stress reduction on the bottom due to the excavation was 4.8 kip/ft.². The initial pore pressure in the first row of elements below the bottom (Nos. 28 to 31) was 2.2 kip/ft.² and the average after excavation was —1.7 kip/ft.² (Fig. 7c), i.e. a total change of 3.9 kip/ft.², so that the change of effective stress was only 0.9 kip/ft.². A similar effect is seen on the vertical face.

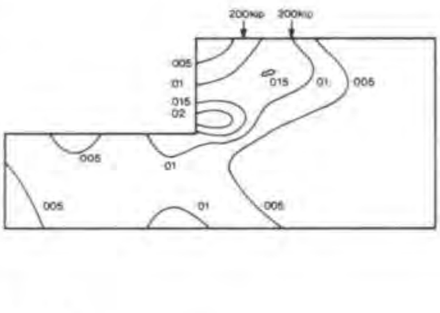


Fig. 8
Shear strains for 400 kip load

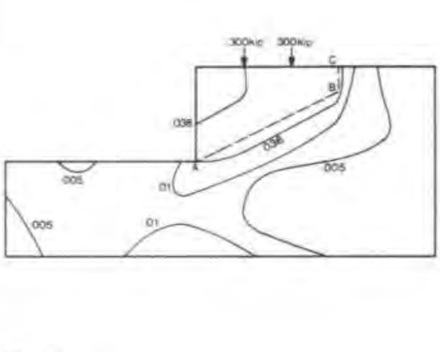


Fig. 9
Shear strains for 600 kip load

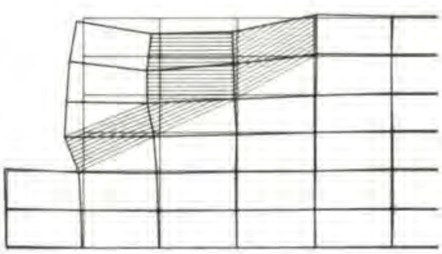


Fig. 10
Part section showing deflection adjacent to face

A total load of 400 kips equivalent to a surcharge was then applied to nodes 70 and 71 indicated on Fig. 8 which also shows the resulting shear strains. It is seen that high shear strains have developed below the load and at the vertical face of the excavation where element 37 is at a strain of 0.0267, indicating that it has passed its peak strength.

The load on these nodes was then increased to 600 kips. It was found that equilibrium could not be obtained, so that the figures for an intermediate stage of the calculation were taken from the computer. The shear strains for this stage of the calculation have been plotted in Fig. 9. In addition, the program gave the out-of-balance force of each node due to the incompatibility of the stresses in adjoining elements. Examining these residual forces it was found that all the nodes on and below the line ABC on Fig. 9 were in equilibrium but not those above the line. It is seen from the shear strains (Fig. 9) that above this line there is a zone of elements which have a shear strain greater than 0.036 so that in this zone all the elements are at the reduced strength and there is no additional strength which can be mobilized.

The form of the failure can be seen in the plot of the deflections on Fig. 10. From the strains in the elements it is found that those which have been hatched horizontally, elements 48 and 53, are failing by direct crushing due to the local direct stress of the applied load. Although the program assumes yielding of the whole element, it can be seen from the deformed shape shown in Fig. 10 that the shear strain in the remaining yielding elements has been developed in the cross-hatch zone and that this area forms a slip-plane where the failure is occurring.

Conclusions

Whilst this method is still in early stages of development it seems to have a number of points to commend it. It enables more realistic assumptions to be made about the mechanical properties of the soil and, by separating out the effects of the porewater pressure, opens up the possibilities for a more detailed assessment of the behaviour of soil profiles with time. At the moment an undrained plane strain problem can be solved at a reasonable cost, admittedly with some special know-how on how to run the program. The immediate next requirement would be to make this program user-proof. After that there is a whole range of problems to tackle: two-dimensional fully drained sections, porewater dissipation calculations and three-dimensional analysis of finite excavations to mention a few. All these can, in principle, be solved this way but the details will require some working out.

References

(1) DUNCAN, J. M. *and* CHANG, C. Y. Analysis of soil movement around a deep excavation. *American Society of Civil Engineers Journal.* Soil Mechanics Division, ***96*** (SM5), pp. 1655–1681, September 1970.

(2) PERLOF, W. H. *and* POMBO, L. E. Proceedings of Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969, Vol. 1 pp. 327–333.

(3) DUNLOP, P. *and* DUNCAN, J. M. Development of failure around excavated slopes. *American Society of Civil Engineers Journal.* Soil Mechanics Division, ***96*** (SM2), pp. 471–493, March 1970.

Printed in England by Roystan Printers Limited