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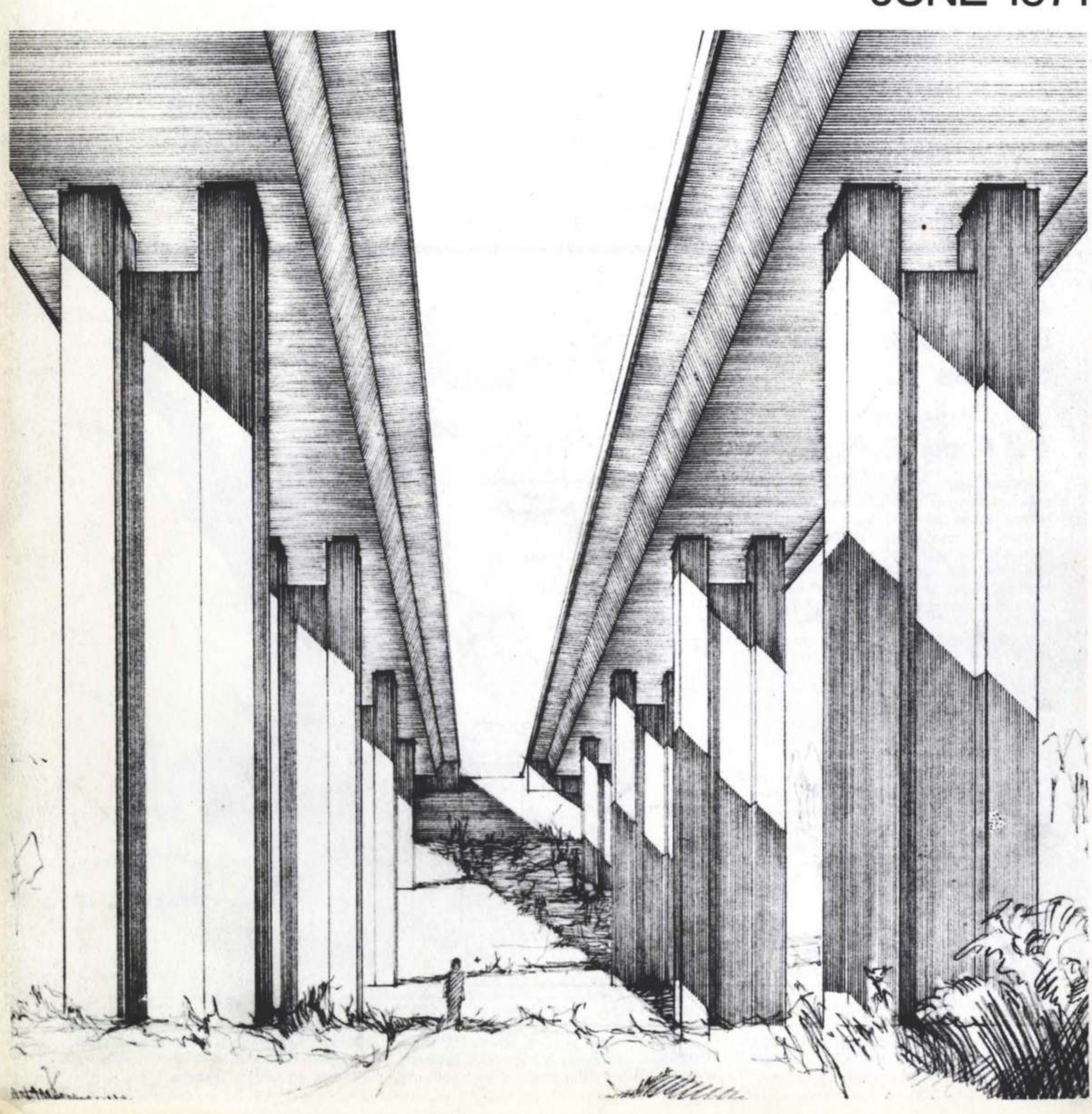

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Editorial Assistant: David Brown

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Front cover: Doornhoek bridge 3078 (drawn by Georg Rotne)
Back cover: MIG welding of aluminium plate (reproduced by courtesy of Alcan Ltd.)

Design and analysis of Doornhoek bridges

Victor Nassim
and Robert Benaim

Introduction

The construction of a dam across the Great Letaba River, near Tzaneen in the North Eastern Transvaal (Fig. 1) entailed the rerouting of the local road network. The Transvaal Provincial Authority asked our Johannesburg office to design the bridges; the roads were being designed by another consultant. As the Johannesburg office was at that time heavily committed, bridges 3077, 3078, 3080 and 3081 were designed in London by Highways and Bridges Division in consultation with Peter Ferrett in Johannesburg. Bridge no. 3078 was to form one contract, while bridges 3077, 3080 and 3081, all of modest size, were to be grouped in another contract.

During the initial design stages, Ron Heydenrych and Peter Ferrett of the Johannesburg office took part in the preliminary discussions in London. Our architectural advisers were Georg Rotne for bridge 3078 and Humphrey Wood of Renton Howard Wood Associates for bridges 3077, 3080 and 3081.

Bridge 3078

The client required a twin bridge carrying dual carriageways, each 14 m (46 ft.) wide with a 9.9 m (32.5 ft.) separation. Each carriageway consisted of 11.6 m (38 ft.) wide roadway and two footpaths, 1.2 m (4.1 ft.) and 480 mm (1.6 ft.) wide.

The valley width at road level was approximately 400 m (1300 ft.) with a maximum depth of about 34 m (110 ft.).

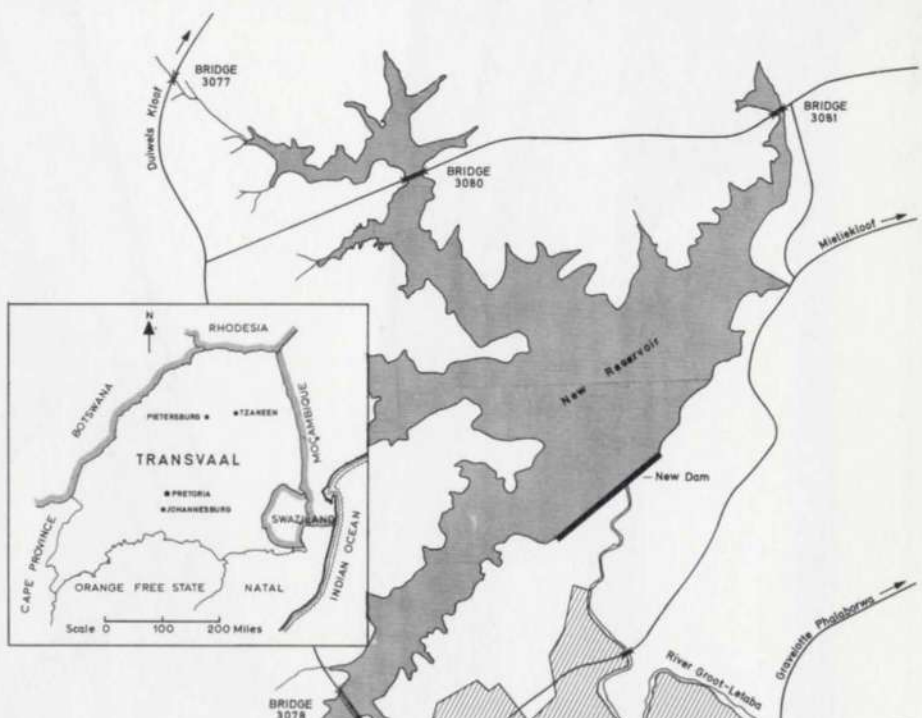


Fig. 1
Location map

The geology of the site was rather complex. The soil consisted of up to 30 m (100 ft.) of augerable weathered granite with boulders, overlying broken and fresh diabase bed rock.

The weathered granite had a collapsing grain structure which decreased in volume when in contact with water, making it totally unreliable as a founding material.

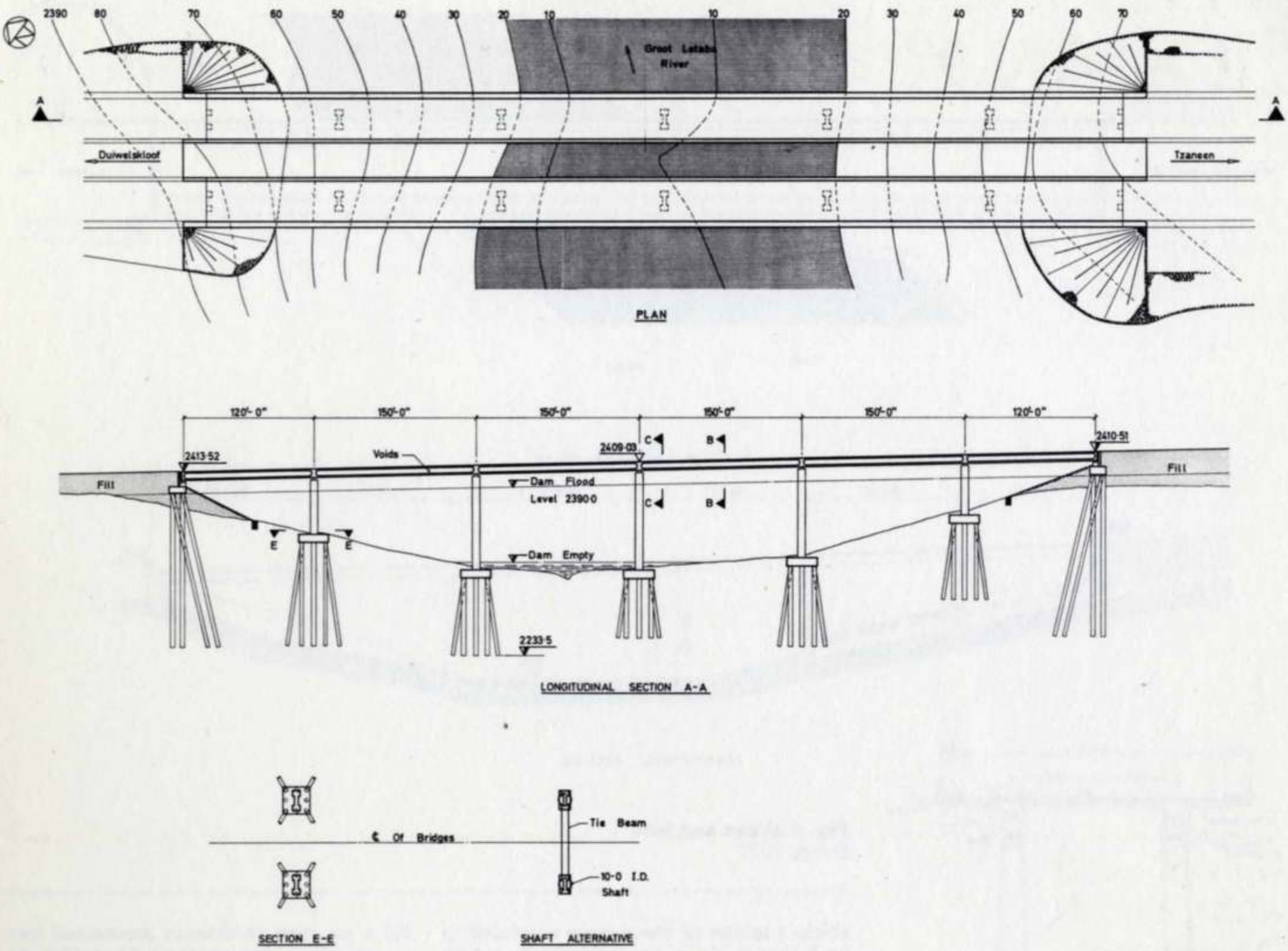


Fig. 2
Bridge 3078

Preliminary schemes

i) *Foundations*

Two types of foundation were finally considered as being suitable for the site. The first consisted of 0.9 to 1.2 m (3 to 4 ft.) diameter steel tubes driven to bedrock with the aid of an auger and cutting head; material within the tubes would then be excavated and replaced by concrete.

The second type of foundation consisted of 3 m (10 ft.) internal diameter shafts excavated to bedrock and lined with 300 mm (1 ft.) thick concrete as the excavation progressed. On completion of excavation the shafts would be filled with concrete. As shaft sinking is a technique widely used in the mining and construction industry of this region, it was felt that this type of foundation would make use of the local expertise in this field. Both foundation systems were also designed to take the additional load due to negative friction caused by the collapsing weathered granite. The two foundation schemes were included in the tender documents and contractors were asked to price the scheme favoured by them.

ii) *Superstructure*

Four basic methods of construction were investigated; in situ prestressed box girder, precast prestressed I girders, high yield steel I girders and twin steel box girders, the latter three schemes with in situ deck slabs. Altogether 10 schemes were investigated for an overall length of bridge of 388 m (1275 ft.). Because of the height and the expensive foundations the spans investigated ranged between 58 and 41 m (190 and 135 ft.) and the gross estimated cost (1969 prices), using shaft foundations, was between £9 and £8 per ft.² for both the prestressed and the steel schemes. It was found that a prestressed concrete in situ box with a span-to-depth ratio of 19 and a maximum span length of 46 m (150 ft.) would be the most economical solution. Further cost investigation showed that it would be more economical to fill part of the approaches, thus reducing the overall length of bridge. This resulted in a bridge of overall length of 256 m (840 ft.) consisting of four spans of 46 m (150 ft.) and two end spans of 37 m (120 ft.) (Fig. 2). The total estimated cost of the chosen scheme was approximately £600,000.

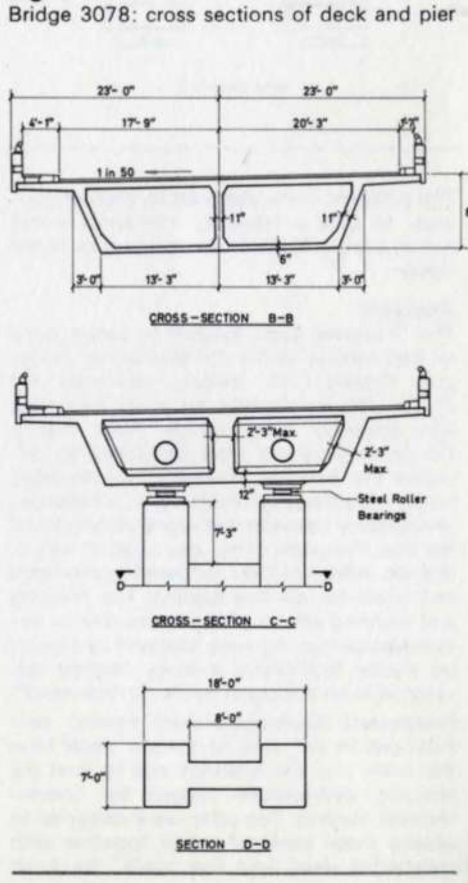
Deck structure

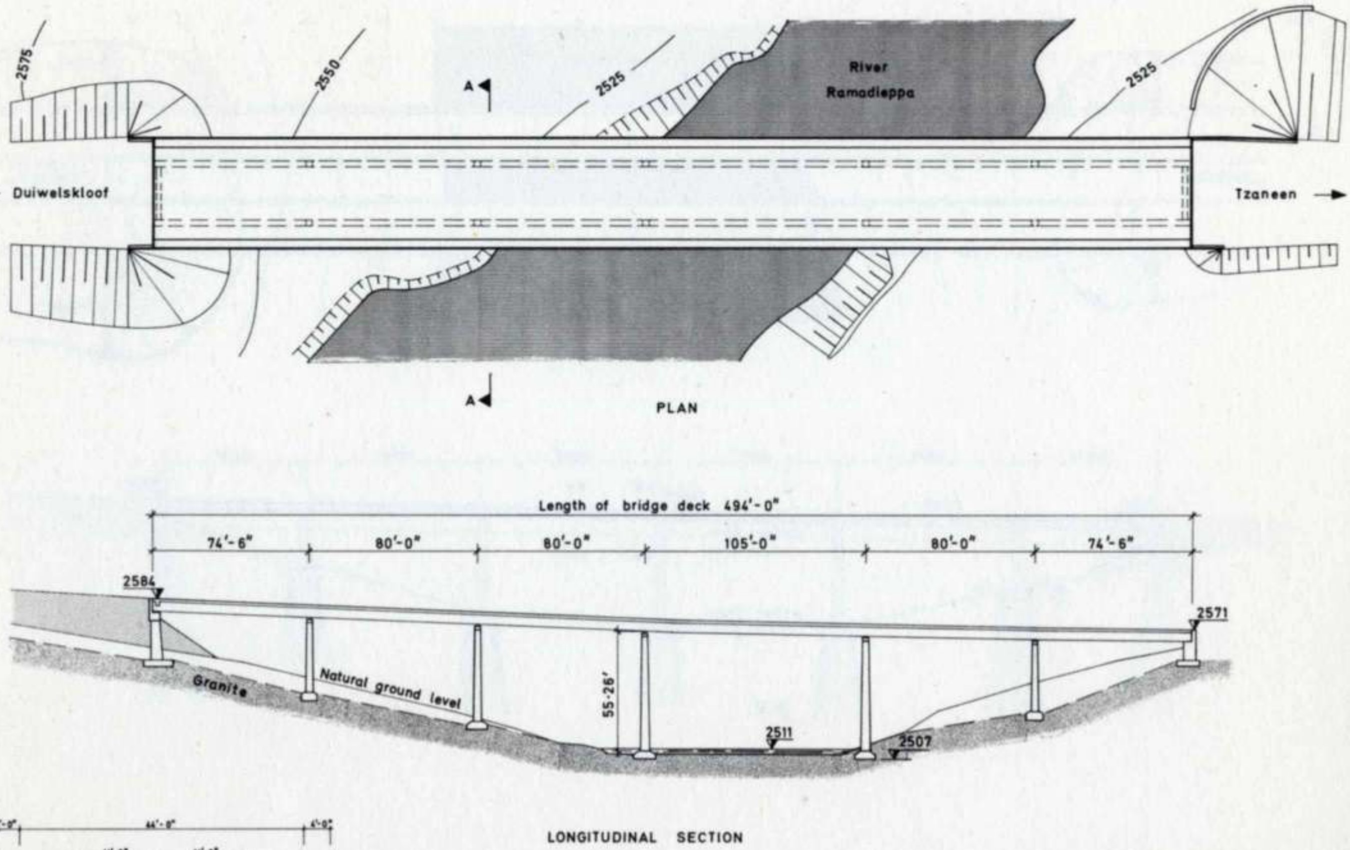
The deck consisted of twin cell boxes having walls 280 mm (11 in.) thick in the central portion of the spans and increasing to 686 mm (2.25 ft.) thick over the piers to accommodate the shear forces and the coupling anchorages. (Fig. 3).

It was decided to design the bridge for erection using the span-by-span method of construction, with joints at the 0.2 span positions, with the aid of an erection girder. The reasons for using this method were repeated use of formwork (12 uses are possible for both boxes), and the erection girder avoids the use of falsework built from the valley floor which would be expensive considering the depth of the valley.

The prestressing was detailed using the CCL system 12/0.6 strand with a working load of 428 kips per cable, but other systems were permitted in the tender.

Fig. 3
Bridge 3078: cross sections of deck and pier





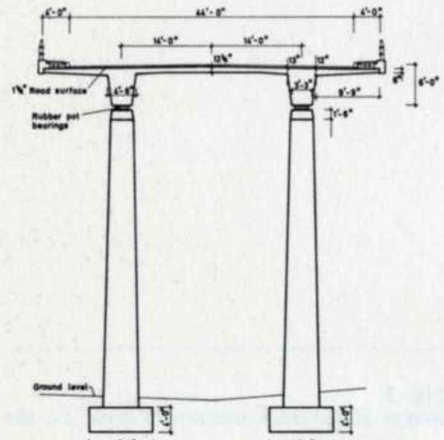


Fig. 4 above and left
Bridge 3077

The parapets were designed in precast concrete to give a relatively maintenance-free barrier compatible with the appearance of the bridge.

Analysis

The Transvaal deck loading is based on a revised version of the old Ministry of Transport loading. HA loading governed the design. The longitudinal action of the bridge was analysed using simple beam theory. Computer programs were developed to calculate the bending moments due to dead loads, prestressing loads and differential temperature between the top and bottom of the box. Programs were also used to calculate the influence lines for bending moments and shear for the live loading. The twisting and warping effects of the boxes due to unsymmetrical loading were analysed by a beam on elastic foundation analogy method developed in an ICE paper by Dr. B. Richmond[1].

Prestressed diaphragms were needed over each pier in the deck to transfer shear from the webs into the bearings and to limit the warping deformation caused by unsymmetrical loading. The piers were designed to absorb these torsional forces together with substantial dead and live loads; the most efficient section for the purpose was found to be I shaped. As it was essential to reduce bending moments in the longitudinal direction, steel roller bearings were used in the piers to reduce friction. The bridge was pinned at the north abutment and was made free to roll over all the piers and south abutment. The horizontal forces, including longitudinal wind, braking and friction, were absorbed by the north abutment.

Bridges 3077, 3080, 3081

Design

Details of road alignment first became available for bridge 3077, and this was chosen as the prototype design. It was accepted as an act of faith that the other two bridges would be similar enough in scale and situation to permit the same aesthetic and technical treatment as the prototype. As it happened this assumption was proved correct.

Bridge 3077, the prototype, will be fully described, the two others being similar in most respects.

This bridge carried a 13.4 m (44 ft.) roadway and two 1.2 m (4 ft.) footpaths over the Ramadieppa River. The overall width of the bridge was 16.5 m (54 ft.) while the overall length was 150 m (492 ft.). The tallest piers were 17 m (55 ft.) high. (Fig. 4).

The area was considered a beauty spot and something of a playground for the inhabitants of Johannesburg, despite the fact that the city was some 200 miles away. The Merensky dam (Fig. 5), just upstream of the bridge, is used for sailing, and there is a golf course nearby, while the Ramadieppa River itself, virtually dry for most of the year, is paved with smooth flat rocks ideal for sunbathing. (Fig. 6). Cost estimates were made for three preliminary schemes:

(a) a 'control' project consisting of a 750 mm (2.5 ft.) thick reinforced concrete slab on rectangular columns at 15 m (50 ft.) centres.

(b) a six span continuous prestressed twin rib bridge over spans of 22 m–24.5 m–24.5 m–32 m–24.5 m–22 m (72.5 ft.–80 ft.–80 ft.–105 ft.–80 ft.–72.5 ft.).

(c) a reinforced concrete arch of 36.5 m (120 ft.) span.

Scheme (b) was considered to be the cheapest, as well as the best looking, and was adopted.

The cross section of the deck consists basically of a top slab varying in thickness from 150 mm (6 in.) to 330 mm (13 in.) and of two solid concrete ribs, spaced at 8.5 m (28 ft.) centres, tapering in width from 1.37 m (4.5 ft.) at their junction with the slab to 1 m (3.25 ft.) at the soffit. This type of solid twin rib bridge section has recently been used quite extensively in Germany and in France, although there do not seem to be any recent examples in this country.

Although this cross section has not the bending efficiency of the more traditional box or I sections, and cannot match a box section for its load distribution characteristics in a transverse direction, it is found frequently to offer the cheapest solution to moderate sized bridges (spans from about 23 m–40 m (75 ft.–130 ft.)).

The omission of the bottom slab is one of the reasons for the low cost of the section. The bottom slab of the box section has generally to be poured in a separate operation. Only when it has been poured can the formwork for the webs and for the top slab be set up, the reinforcement and prestressing cables placed and the concreting finished. It is also general practice in box construction to pour the webs in approximately 10 m (33 ft.) lengths, with joints between them to avoid cracking due to the differential shrinkage between the relatively old concrete of the bottom slab, and the fresh concrete of the webs. This leads to a slow and piecemeal construction process, very labour intensive and expensive.

However, for these medium span bridges, of a span/depth ratio under 25, a web designed to resist maximum shear will necessarily have sufficient width to render a bottom slab superfluous for resistance to hogging bending moments. This was amply borne out on this bridge as, despite the small width of the bottom of the web, the bottom fibre stress at the support due to maximum hogging moment was 9.3N/mm² (1320 lb./in.²), far short of the limiting value of 14N/mm² (2000 lb./in.²) under HA (standard) loading, or 17.5N/mm² (2500 lb./in.²) under HB (exceptional vehicle) loading.

The omission of the bottom slab allows the webs to be considerably thickened without increasing the cross sectional area. As the webs are the most difficult and time consuming part of a bridge deck to build, due to the relative complexity of their reinforcement and due to the difficulty in placing and adequately compacting the concrete, this thickening results in a major simplification of the construction process.

There is also, however, the debit side to the balance sheet.

The two main sources of extra cost in twin rib bridges are firstly the longer span, and hence heavier reinforcement of the top slab, and secondly a tendency to use slightly more prestressing steel than a comparable box. As the section lacks overall torsional rigidity, a load placed directly over one rib, for instance, is largely carried by that rib (Figs. 7, 8 and 9), while in a torsionally stiff box, this load would be distributed more or less evenly between the two webs. However, most concrete box beams for bridges of the span range under consideration are not stiffened by diaphragms between supports, and are thus relatively flexible distorsionally, losing a good deal of their load distributing characteristics. For a typical concrete bridge in which live load moments are approximately ⅓ of total moments, the twin rib bridge would need some 10% more prestressing than a comparable box section.

When finally comparing the credit and debit entries, quantities of materials for comparable box and twin rib solutions are generally about the same, but in most cases lower unit costs of materials, due to ease of construction, favour unequivocally the twin rib solution. This latter section is also very well adapted to mechanized construction methods, such as the bridge building machines much used on the continent. Bridges of this type have been built at the rate of two spans a month, the concreting of the deck for one span being carried out in less than 24 hours. With further development it is certain that construction rates of a span per week are possible with highly mechanized techniques reducing drastically the labour content of the work. Economy in poured in situ medium span bridge building appears to lie in this direction, rather than in efforts to reduce quantities to an absolute minimum.

A second departure from common practice in this country was the omission of all diaphragms, except for a relatively flexible cross beam at each abutment.

This omission served the two-fold purpose of simplifying notably the construction, and reducing the torsional stresses to which the ribs are subjected. Diaphragms over intermediate supports would have acted as torsion sumps, giving unacceptably high torsional shearing stresses in the ribs.

Each pier consisted of twin columns centred under the ribs and founded on pads bearing on granite. The three central pairs of columns were pinned to the deck by rubber pot bearings which allowed relative rotation. These columns were sufficiently flexible to accept the dimensional changes of the bridge deck. The two outer pairs of columns were capped by free sliding rubber pot bearings, while on each abutment there was one guided and one free sliding bearing.

The bridge was thus not absolutely fixed longitudinally. Any longitudinal force was not taken out by any one fixed point, but was distributed between all the support points by the small induced longitudinal movement of the deck.

One abutment is of the 'spill through' or trestle type, while the other had to cope with a very steep cross fall of the terrain, and is of the retaining wall type, with one free standing 15 m (50 ft.) long wing wall.

Expansion joints, formed with epoxy mortar nosings protected by steel angles, were sealed with extruded neoprene sections.

Doornhoek Bridges 4 and 5

These were similar to Doornhoek 3, but carried an 11 m (36 ft.) road and two 450 mm (1.5 ft.) footpaths. Their overall width was 12.5 m (41 ft.).

Doornhoek 4 had four spans of 30.5 m (100 ft.) and two end spans, each of 27.5 m (90 ft.). They rested on columns up to 27.5 m (90 ft.) long.

Doornhoek 5 had four spans of 26 m–29 m–29 m–26 m (85 ft.–95 ft.–95 ft.–85 ft.).

The only modification necessary to adapt the formwork from bridge 3077 to the narrower 3080 and 3081 was to shorten the cantilevers

Fig. 5
Bridge 3077
Aerial photograph of bridge site
(Photo: Courtesy of
Map Studio Productions (PTY) Ltd.)

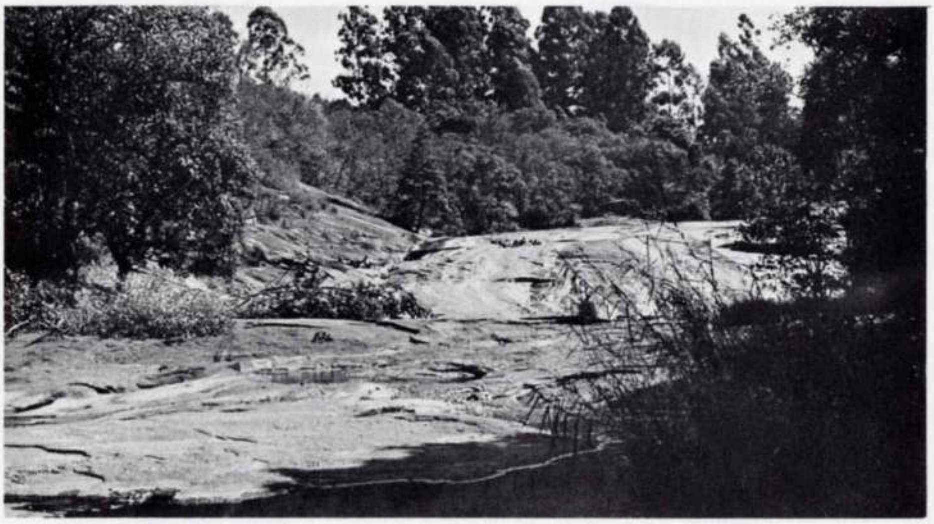


Fig. 6
The Ramadieppa River
(Photo: Tom Makin)

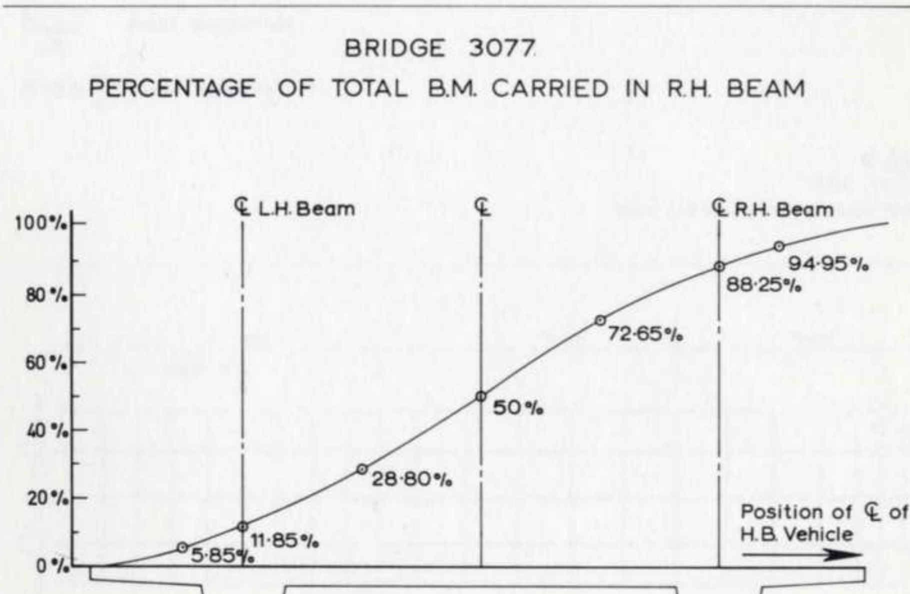


Fig. 7
Bridge 3077
Load distribution curve

BRIDGES 3080 AND 3081

PERCENTAGE OF TOTAL B.M. CARRIED IN R.H. BEAM

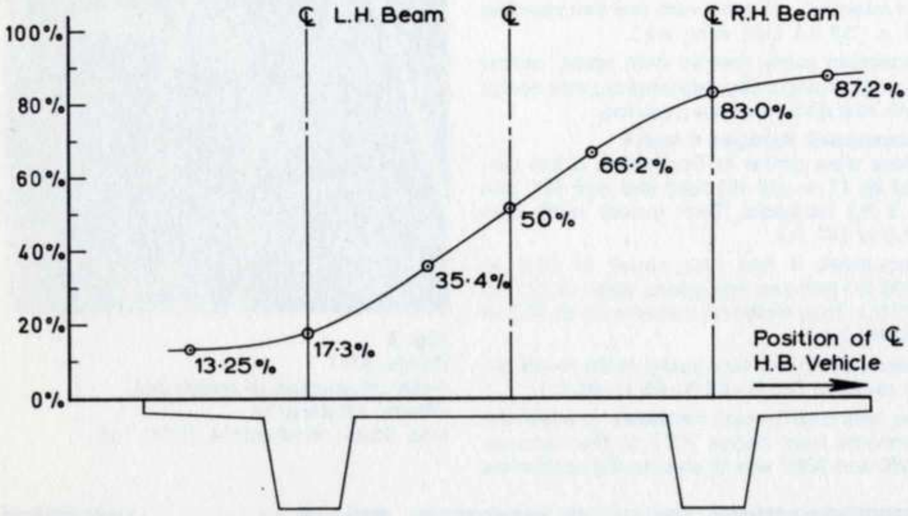


Fig. 8
Bridges 3080 and 3081
Load distribution curve

BRIDGE 3077

DEFLECTION UNDER AXLE OF HB VEHICLE AT MIDSPAN

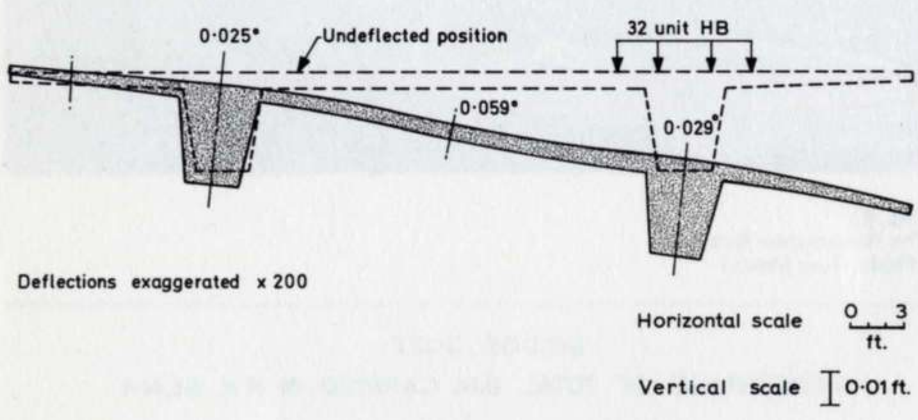


Fig. 9
Bridge 3077
Deflection under eccentric load

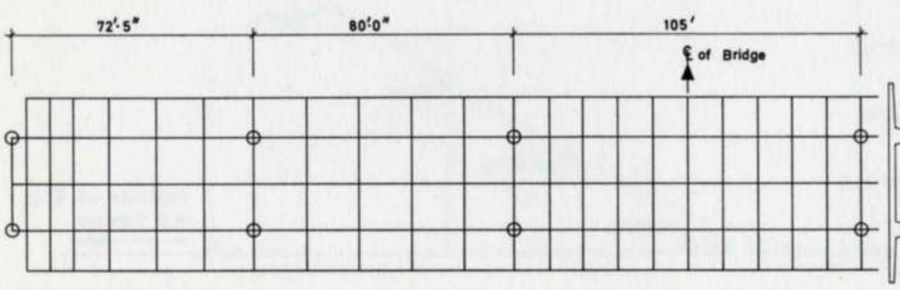


Fig. 10
Bridge 3077
Equivalent grillage for computer program

and to shorten the horizontal slab soffit form between ribs.

Both these bridges were founded on granite. However, at the abutments and at some pier locations, the depth of overburden made the use of piles necessary. 900 mm (3 ft.) diameter bored caisson piles were used, with a working load of 155 t. At the abutments the horizontal forces were resisted by raking piles, while horizontal forces on the piers were resisted by bending in the vertical piles.

Construction sequence

Bridges 3077 and 3080 were initially designed to be built two spans at a time, while 3081 was to be built in one stage. The prestressing system was to be Freyssinet 12 x 15 mm (0.6 in.) multi-strand cables. However, the successful tenderer requested that all the bridges be redesigned to a span by span sequence and BBRV cables substituted for Freyssinet multi-strand.

Analysis

As the bridge was to be checked for only 32 units of HB loading, HA was for every action the governing load case. HB loading was, however, also examined in order to obtain a greater insight into the functioning of twin rib cross sections under eccentric loading.

The longitudinal action of the bridge was analysed by simple continuous prestressed beam theory.

The transverse action of the bridge was analysed using the plane grid computer program OA101 (Fig. 10).

The six span unsymmetrical bridge was idealized to a five span symmetrical structure for simplicity (spans 22 m–24.5 m–32 m–24.5 m–22 m (72.5 ft.–80 ft.–105 ft.–80 ft.–72.5 ft.)). As the only regions investigated were the centre span and the areas near the end diaphragms, there was no appreciable loss of accuracy.

As the bridge has no diaphragms between the abutment positions, and the deck rests on flexible columns through rotationally free bearings, the ribs are virtually free of any outside restraint to their rotation about a longitudinal axis. The deck slab acts as a plate, supported at the centre line of the ribs, prolonged by two cantilevers. The degree of rotational fixity afforded by the ribs depends on the length of bridge loaded. When the full length of the bridge is loaded, the slab is simply supported, whereas for short loaded lengths, the slab is virtually fixed at its supports.

Similarly, the torque in the ribs is zero when the whole length of the bridge is loaded, and increases as the loaded length decreases. Various loaded lengths had to be tried with their corresponding intensities of loading in order to ascertain the greatest values for the action being analysed. For instance, maximum sagging bending moment in the slab occurred when the area between rib centre lines was loaded over three continuous spans.

Conclusion

The successful tenderers were Murray & Roberts (Pty) Ltd.

They are now on site, and have started work on bridge 3081. The tendered price was Doornhoek 3: £150,000; Doornhoek 4: £160,000; Doornhoek 5: £110,000.

Bridge 3077 was also priced by our quantity surveyors as if it were to be built in this country. The cost for mid 1970 worked out at £6.22 per ft.² including finishes, 10% general items and 10% contingencies.

Reference

(1) RICHMOND, B. Trapezoidal boxes with continuous diaphragms. *ICE Proceedings*, ***43*** (August), pp. 641–650, 1969.

Building satisfaction: the user's viewpoint

David Whitton

It has been suggested that the architect, like the cuckoo, lays his eggs in other people's 'nests' and never returns to see into what they hatch.[1] A rather more mundane way of expressing this is to suggest that many architects do not learn from their mistakes because they do not carry out any systematic survey of what errors they have made in their buildings or their users' reactions to them. If they were to specify in detail what they were attempting to achieve in any particular building and were then to examine the building after it had been open for some time, they would have a more detailed understanding of how successful they had been in achieving what they had intended.

What is to be measured?

The concept of the *Plan of Work*[2] published by the RIBA and related to building design of individual projects is well known. The orderly progress of work from feasibility through detailed proposals to tender action is well defined, but after stage G (receipt of tender), the stages tend to become blurred and by the time stage M is reached, interest in the orderly progress of the job has faded in the desire to see the client satisfied and concentration passes to the next job. But the design process is an iterative one, in total as well as in part, to which analysis–synthesis–appraisal is applicable. Stage M is designated as 'feedback'; the appraisal part of each job cycle, unfortunately, still all too frequently, inadequately pursued or quietly forgotten. Input for each job design cycle then becomes the optimistically anticipated results, usually deduced from past jobs and not carefully recorded experience of achievement or failure in meeting past design aims.

The complete process of design can, under this system, approach experience, a closed loop with the same data constantly fed in for new projects and the 'memory' of past experience not updated with fresh criteria. Further, the experiences of designers are not available for the benefit and education of others. To break out of this closed loop the method of analysis of past projects had to be more rigorously defined.

In 1967, Professor Markus,[3] with financial assistance from a group of 11 architectural practices, an architectural journal and the government, set up the Building Performance Research Unit at the University of Strathclyde. The intention of creating the research group was to measure the success of various completed buildings against the original design criteria. From the outset the Unit was intended to be interdisciplinary and to bring people not necessarily trained in the architectural or building fields to analyse specific problems encountered and measure the success of the solution by the criteria of their own field and not of existing architectural practice.

The Unit was initially established with an operational research scientist, a psychologist, an architect and a physicist. The Unit's composition changed later with the physicist's place being taken by a quantity surveyor. Work initially concentrated on creating a conceptual model of a building and its user. The model in Fig. 1 shows the four basic systems identified. The interrelationship of the systems is shown by the cylinder representing the total interdependence and the fact that any variation must have repercussions on all the other four systems. The primary concern of the BPRU was the interaction between the activity system and the environmental system. The dynamic balance is a concept which in any specific case can only have momentary existence, as any change in any one of the four systems must have immediate repercussions on the other three systems or unbalance of the total.

The specific interests of the Unit within the conceptual model centred around the environmental/activity interface. Fig. 2 shows the conceptual model applied to an educational organization. The objectives and building systems were considered in this study but not in so much detail.

What can we measure with?

The difficulty of knowing when the model is balanced now becomes obvious as the units of measurement of each of the systems, and the subsystems within them, are completely different. Another level was therefore added to the conceptual model, the cost system (Fig. 1). Each of the four systems could be measured in financial terms from which a position of balance could be identified.

The appraisal of a building in respect of any variable is impossible without reference to other buildings. Some yardstick is then needed for each method of measurement of comparison. The simplest are mandatory maxima or minima which are capable of physical measurement, such as room size or lighting level. Other more nebulous criteria cannot be related to so specific a measure and must be referred either to a criterion, i.e. the best possible within the prevailing conditions, or to a norm, such as the average result over an adequately sized sample.

The examples now given were chosen to show the varied pattern of the work within the unit. Criteria needed were taken from a building type initially selected for the homogeneity of the requirements and come from 48 comprehensive schools built during the last 10 years in the central belt of Scotland (between Glasgow and Edinburgh basically). Most were new schools, although a few were major extensions to existing buildings.

Spatial environment and the activity system

Planning of a building can in very crude terms be described as an identification of the requirements of spaces and the placing of these spaces in juxtaposition. One of the earliest things noticed in the visits to schools in the sample was the number of individual desks that were empty although functionally the school was classified as full. On analysis this was traced back to the requirements of the clients who had specified classrooms of a certain standard size. This requirement fitted the teaching pattern of one teacher and a class of 30 pupils.

Comprehensive education had produced a different activity pattern of children being put in 'sets' for subjects and granted many options. This produced a complete range of class sizes from 100–1 which, if they were to be most adequately provided for, meant a complete range of room sizes. Fig. 3 shows a typical schedule of room sizes and the activity pattern, the unshaded area under the existing schedule graph being the redundant space provided. The optimum schedule for that activity pattern is also shown.

Fig. 3 only shows the present activity pattern and an individual solution. In the future

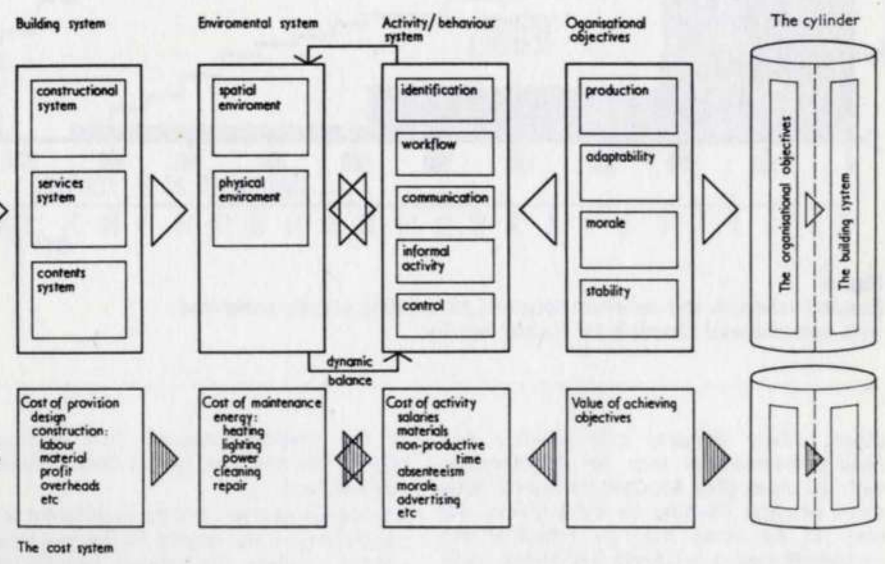


Fig. 1
The building-environment-activity-objectives system: a conceptual model

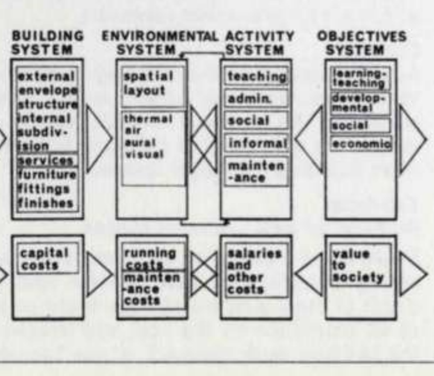
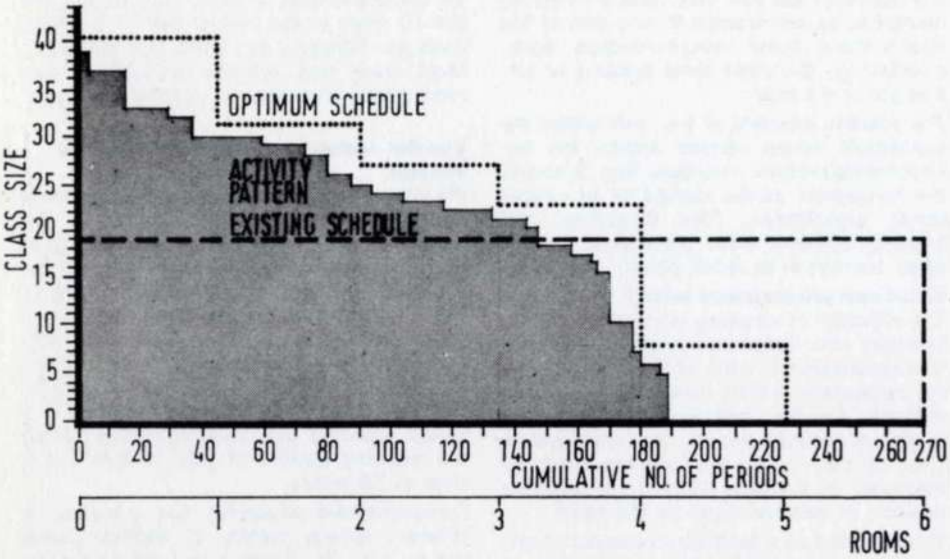


Fig. 2
BPRU conceptual model applied to a school

Fig. 3
Existing schedule and activity pattern compared, showing very bad misfit



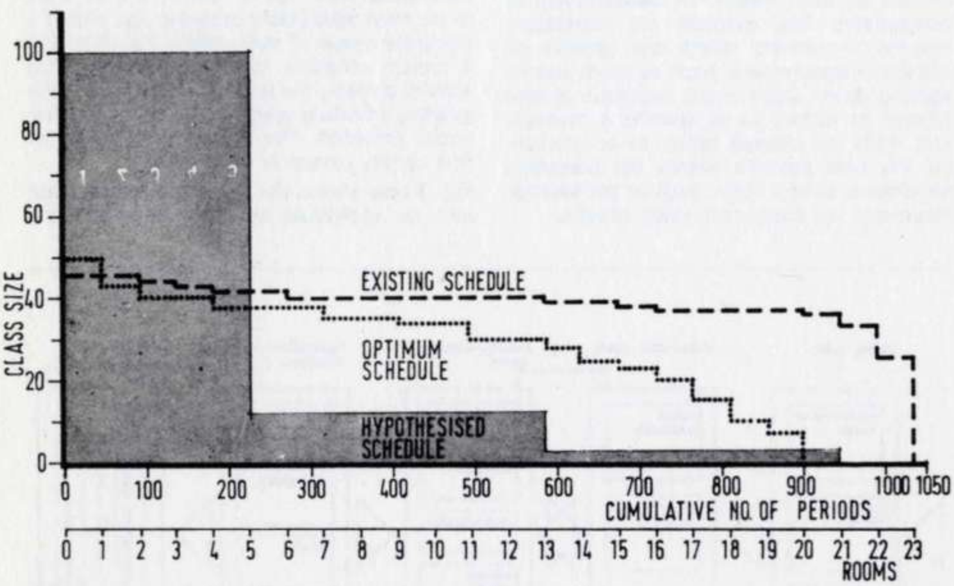


Fig. 4
Existing schedule and optimum schedule for existing activity compared with hypothesized schedule for further activity

other activity patterns may develop and various possibilities may be hypothesized with the client (Fig. 4). Over a period of time some physical changes to the building will have to be made but, by hypothesizing several alternative solutions, a schedule minimizing the possible costs for various strategies can develop (Fig. 4). The strategy followed by the schools visited had been to provide redundant space, but a more suitable solution might have been a building designed with a range of room sizes and more easily capable of physical adaptation. This scheduling is a very laborious task and the Unit developed several computer programs for analysis of activity patterns and generation of schedules which have been used by the Scottish Education Department.[4]

Organizational order

Any organization occupying a building has to fit its activities to the physical structure. In the case of a perfect design and an exact brief the 'fit' will be exact but organizations are, by definition, organic and constantly restructuring themselves. They have either to adjust their changing growth requirements to the physical structure they occupy or change the structure to suit their anticipated requirements.

The degree of either will be on different levels, dependant on the source of the pressure for change. Within the schools sample, three distinct levels were isolated:[5]

Improvisation
Any alteration in mode of use of the building or a physical change of a temporary or makeshift nature, carried out without much effort or cost and easily reversible.

Change
Any adaptation of the building system involving expense and which is not easily reversible. An example of this would be installation or removal of partitions to provide more numerous or larger spaces.

Extension
Addition of new enclosed spaces.

Evidence of change at all three levels was strong, even in very new schools, and the depth of organizational change might be out of all proportion to the cost and trouble of the building work involved. It was hypothesized that improvisation was a symptom of need for structural re-planning and a survey pinpointed two main areas; improvisation in the circulation system and in the use of individual spaces for different activities from those planned.

Within the circulation system visible improvisations such as arrows and direction signs were comparatively rare. But almost all the schools had found it necessary to introduce formal rules to smooth the flow of people around the building and many of these rules were akin to traffic rules in congested cities. Examples of one way traffic, tidal flow and speed limits (e.g. no running) were common.

16 major rules were isolated and it was found that these could be placed in an ascending order giving a scale of formality of circulation. It was proved statistically over the whole sample that, if a school selected at random had rule 4 operating, it was highly probable that rules 1 to 3 also operated and that, if rule 5 did not operate, neither did 6 to 16.

Having established a scale, the question was 'did it measure something about the building or its occupants?' A correlation against a number of characteristics of the buildings such as area per pupil, design roll and circulation area percentage, were tested but the most statistically significant result came with the headmaster's attitude to the education system. This was even more significant in buildings less than three years old. The inference drawn from this is that the headmaster's attitude affects the number of rules imposed when the building is first occupied but the reactions of occupants to buildings increase and ultimately submerge this authoritarianism.

If, at an ideal school, the pupils should be free to move from point to point, by the most direct route and without restriction, the scale may be used to measure how far from ideal is any existing coeducational comprehensive school. The higher the 'score' of any school, the more formal the circulation order.

Satisfaction of the user

Critical to an understanding of the satisfaction of the user is an understanding of the relationship between users' reactions to buildings and the buildings themselves. The theoretical methods of doing this in the laboratory have been widely explored but actual field measurement not nearly so fully. The unit decided to attempt this, regarding it as more useful to the design professions. It was also the way in which further studies could be carried out by people with relatively little psychological background knowledge.

It is highly probable that the effects of building on behaviour are very subtle and we need to know not only how, but why people behave as they do. One of the easiest ways to do this is to ask what they feel and think about various aspects of the building. Words are the most easily used tool for this purpose, although other techniques could have been applied. Words have richness and varying dimensions of meaning to different people, but this was overcome by clustering groups of the descriptive adjectives used.[6]

Two parallel studies were made, one on the effect of buildings on their users and the other on the reactions of possible users to posed alternative designs. Expression of satisfaction may be thought of as interpretation by users of their own interaction with the environment. It was therefore decided to develop a set of scales that would give an accurate indication of reactions of users to their environment.

Scales for measuring user reactions to buildings need many properties but in this case two of the most important properties were robustness and reliability. Robustness means

Fig. 5
Questionnaire to measure teachers' reactions

UNIVERSITY OF STRATHCLYDE
SCHOOL OF ARCHITECTURE GLASGOW C1 telephone BELL 4400
Building Performance Research Unit
Director: Thomas A Markus MA MArch(MIT) ARIBA Professor of Building Science

PAT/948

This questionnaire has been developed to measure your reactions in general to three places which you use.

Please indicate where EACH of these places comes on EACH of the scales below by putting the appropriate number (1 to 7) in the box below the place. Do not ponder too long over any one question. Please treat each response separately, any apparent repetition of questions is for statistical control. Please ensure you have completed EVERY item.

	X. THE CLASSROOM IN WHICH YOU USUALLY TEACH	Y. THE STAFF ROOM YOU USUALLY USE	Z. THE SCHOOL BUILDING AS A WHOLE
ADEQUATE 1 2 3 4 5 6 7 INADEQUATE			
SUITABLE 1 2 3 4 5 6 7 UNSUITABLE			
ACCEPTABLE 1 2 3 4 5 6 7 UNACCEPTABLE			
PLEASANT 1 2 3 4 5 6 7 UNPLEASANT			
COMFORTABLE 1 2 3 4 5 6 7 UNCOMFORTABLE			
GOOD 1 2 3 4 5 6 7 BAD			
INTERESTING 1 2 3 4 5 6 7 UNINTERESTING			
STIMULATING 1 2 3 4 5 6 7 DEPRESSING			
BEST POSSIBLE 1 2 3 4 5 6 7 WORST POSSIBLE			
ABOVE AVERAGE 1 2 3 4 5 6 7 BELOW AVERAGE			

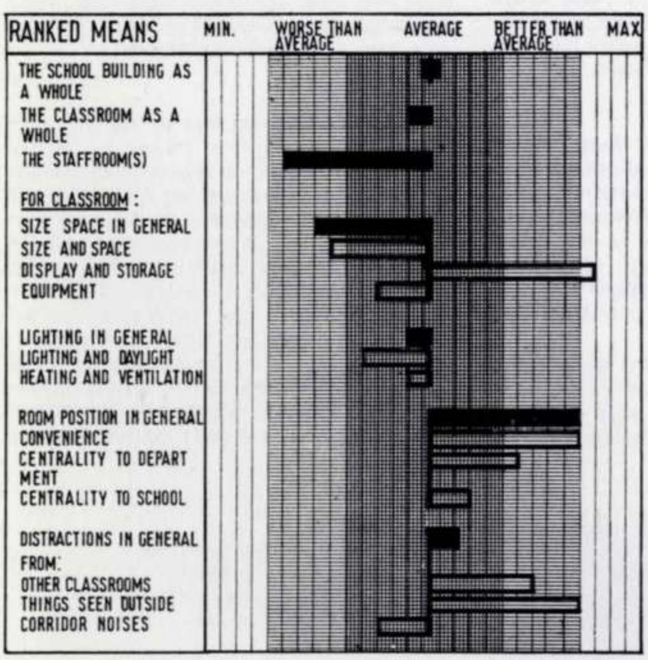


Fig. 6
Psarchigraph.
Black bars represent aspects: the major categories.
Hollow bars represent facets: the underlying influences

difficulty of misuse of the scales while reliability means consistence. One of the questionnaires, used by 493 teachers, is reproduced in Fig. 5.

The results from this type of study enable a profile of a building relative to a larger sample to be established. Fig. 6 shows a psarchigraph for one school out of a group of 28. This particular example shows a school, average in the measure of general satisfaction, but very unsatisfactory from the viewpoint of staffroom position and general space, although overall planning was much better than average. Experience of the use of this tool would allow a designer to build up an image of what the working environment was like. The psarchigraph gives an insight into a building's personality.

The other study of the reactions of users to possible design changes was approached in a different manner. This was done either by showing teachers drawings of possible alternative designs or asking for their value judgement in terms of salary adjustment for certain changes. Both of these methods prove reasonably consistent, although some teachers found difficulty in answering questions regarding environments in which they had no experience of teaching. Their consistency in selecting the most expensive alternatives was remarkably high!

The value judgement exercise provided some very interesting results in that, although the effect of the environment was a major element in satisfaction with a school, the effect of the personality of the headmaster presented the major element of influence.

Conclusions

The three examples quoted show the development of three measures of a building in use and the success of the design as far as the user is concerned. In some cases the satisfaction of the user is regarded as unimportant and that of the owner viewed as omnipotent. In these cases mechanical or human productivity are the critical criteria and the methods of arriving at these are mechanistic and well explored. Many buildings cannot be satisfactorily explained or measured by these techniques and the methodology outlined with the examples quoted is an attempt to solve some of those problems.

The examples have been deliberately stripped of all statistical material and in this I must apologize to my colleagues Professor Markus, Tom Maver, David Canter and Peter Whyman and hope that I have not misrepresented their work too irresponsibly.

References

(1) WOOLS, R. Assessment of room friendliness. *In:* Architectural Psychology; proceedings of the conference held at Dalandhui, University of Strathclyde, Glasgow, 28 Feb.–2 Mar. 1969. Edited by D. V. Canter, pp. 48–55. RIBA, 1970.

(2) ROYAL INSTITUTE OF BRITISH ARCHITECTS. Plan of work for design team operation. RIBA, 1967.

(3) MARKUS, T. A. The role of building performance measurement and appraisal in design method. *Architect's Journal Information Library*, ***146*** (25), pp. 1567–1573, 1967.

(4) MAVER, T. W. *and* FLEMING, J. SECS: a program package for accommodation scheduling in comprehensive schools. *ABACUS occasional paper no. 2.* Department of Architecture & Building Science, University of Strathclyde, Glasgow, Dec. 1970.

(5) WHYMAN, P. Movement of pupils in comprehensive schools. *Building,* ***216*** (6574), pp. 20/86–20/90, 1969.

(6) CANTER, D. *and* WOOLS, R. A verbal measure for buildings. *Building,* ***218*** (6631), pp. 25/73–25/76, 1970.

Some aspects of welding aluminium

Alan Denney

Introduction

The existence of a metal related to alumina (otherwise known as aluminium oxide) was first proved in a series of chemical tests by Sir Humphrey Davy in 1809. He suggested the name 'aluminum', which is the present day American name for this metal, but did not succeed in extracting it in its pure metallic form. This was later achieved by Hans Christian Orsted, a professor at the University of Copenhagen, better known for his discovery of the relationship between electricity and magnetism. He produced impure metallic aluminium by using an amalgam to extract the metal from aluminium chloride which he then refined to remove the mercury.

The next developments took place in France where a successful but expensive extraction process was used to produce sufficient aluminium for an exhibition in Paris in 1855. One can conjecture that the marble-sized lumps of this remarkably light metal were received in the same spirit as that in which we would regard moon rock.

Interest was maintained in France and in 1886 yielded the first workable electrolytic process, but the material was still comparatively expensive and widespread application of aluminium did not follow rapidly. It was the search for new light materials for aircraft which really gave aluminium alloy development its great boost, and the aerospace industry continues to be the major user of aluminium alloys.

In view of its light weight, corrosion resistance, good conductivity, ease of working and forming and the availability of higher strength alloys, it should continue to find many future applications. We can anticipate seeing some considerable developments in the production of high strength alloys which are capable of being welded without loss of strength. When these have become accepted we should see a tremendous increase in structural applications where the strength to weight ratio of these alloys can be fully exploited.

Aluminium alloys

Wrought aluminium alloys fall into two distinct categories, which are called heat-treatable alloys and non heat-treatable alloys.

The heat-treatable alloys

The heat-treatable alloys for both wrought and cast alloy products are mainly based upon the aluminium-magnesium-silicon alloy system. Such alloys combine moderately high strength and good corrosion resistance. The strength of these alloys is achieved in a two stage heat treatment process; solution treatment and ageing. The solution stage involves heating to a temperature which is in the region of 520° C (968° F) for the majority of the alloys. During heating, the alloying elements are taken fully into solution by the aluminium and are retained in solution by giving the alloy a rapid quench in water.

Ageing is the gradual precipitation of distinct phases such as Mg_2Si which then takes place over a period of time. It is the separation and dispersion of this phase which causes the increase in tensile strength of the metal. This process can take place at room temperature in a few days or over much longer time periods, or it can be accelerated by heating. When it occurs rapidly without applied heat it is termed 'naturally aged' and when accelerated heat is applied at a specified temperature (160–180° C (320°–356° F)) for some hours it is termed 'artificially aged'.

The non heat-treatable alloys

The N alloys (non heat-treatable wrought alloys) are initially hardened by cold work. A partial annealing treatment is then carried out to remove some of this work hardening. The final condition of the plate is known as its temper, and this is given as part of the classification system. Metallurgically, these alloys are based upon aluminium–magnesium and aluminium–manganese phase systems. Simple aluminium magnesium alloys do not show precipitation hardening in concentrations of magnesium of less than 7%, but show high work hardening characteristics which are the basis of this class of alloys. Small percentages of manganese and chromium are included in the commercial alloys to provide substantial increases in strength. Alloys containing 5–7% magnesium are susceptible to inter-crystalline corrosion and to stress corrosion. The commercial aluminium/manganese alloys contain about 1% manganese and only very small quantities of magnesium. Additions of modifying elements are made to develop specific properties in each of these alloys. None have outstanding mechanical properties but they generally have better corrosion resistance than the heat-treatable ones.

The structural alloys

Both heat-treatable and non heat-treatable wrought alloys may be used structurally. The emphasis in *CP 118: 1969* is on the more usual alloys H30, N8 and H9, but provision is made for design with other alloys of the British Standards Institution general engineering series. A selective list of mechanical properties for the more commonly used structural alloys is given in Table 1.

The heat-treatable alloy H30 is the principal high strength structural alloy. It is used in the heat-treated and artificially aged form, i.e. H30–TF. The alloy is weldable but, like the majority of age-hardened alloys, it suffers from reduced strength in the welded zone. For design purposes the reduced strength zone is assumed to stretch over 25 mm (1 in.) in all directions from the centre line of a butt weld and from the root of a fillet weld[1].

The alloy N8 is normally used where it is intended to weld the structure. The parent metal is less strong than H30–TF but does not suffer from reduced strength in the weld zone when used in the as-manufactured (M) temper or fully annealed (O) temper. Welded joints are hence stronger in N8 alloy than in H30–TF. This alloy is available in all plate thicknesses and in extruded sections, and is one of the most durable. Alloy H9 is notable for its ease of formability into intricate shapes, its high durability and anodizeability. It suffers from loss of strength during welding for the same reason as H30 and is also susceptible to hot cracking.

Some new strong weldable alloys

A conference* in 1969 and a subsequent paper[2] drew engineers' attention to some aluminium alloys which showed considerable potential in terms of welded strength, and lightness. The research was primarily done because of the need for a light transportable military bridge (Fig. 1), and was undertaken by the Military Engineering Experimental Establishment (now called Military Vehicle and Engineering Establishment or MVEE). It was the first research and development done in Britain on these alloys despite their growing use in the United States, Canada and Europe.

They are composed of aluminium, zinc and magnesium and have 0.2% proof stresses of 230 to 310 N/mm² (15–20 tons/in.²). It has been known for some time that alloys in this system with 3–7.5% zinc are age-hardening due to the formation of $MgZn_2$ and that the highest strength aluminium alloys available come from this system, but until the publication of the MVEE research it was not fully appreciated in Britain that alloys in this system have considerable advantages in the as-welded state over other age-hardening alloys. These advantages over a normal structural hardenable alloy are summarized below.

Weld strength

It will be remembered that an age-hardened alloy loses its strength in the welded region. This is because welding reverses the changes which took place in hardening. A high strength aluminium alloy acquires its strength due to the fine dispersion of precipitated particles in the metallurgical structure during ageing. When such an alloy is welded, these precipitated particles are redissolved in the heat affected zone and coarsened just outside this zone. This considerably weakens the alloy in this region, the beneficial properties of the ageing action being completely lost. Further, the weld metal contains no pre-

Fig. 1
High strength welded aluminium alloy bridge spanning 148 ft. (Photo: MVEE (Christchurch) Crown copyright reserved)

*The Welding Institute. Select conference on weldable aluminium alloys, Cambridge, 23-25 September, 1969

Table 1 Mechanical properties of structural alloys.

Alloy	Condition	Form	Thickness (mm) from	to	0.2% tensile proof stress N/mm²	Tensile strength N/mm²	Elongation % on 5.65 area
H30	TF	E	6.4	76	270	309	8
		S	—	6.4	255	293	8
		T	—	1.6	255	309	7
		T	1.6	—	239	309	9
		S	6.4	13	239	293	6
		S	13	25	239	293	5
N8	M	E	—	—	131	278	10
		S	6.4	25	131	278	10
N8	O	E	—	—	131	263	14
		S	6.4	25	131	278	14
H9	TE	E	—	3.2	147	170	10
			3.2	13	116	154	8
H20	TF	E	—	151	239	278	8
H15	TB	E	—	9.5	232	371	11
	TB	E	9.5	76	247	386	11
H15	TF	E	—	9.5	386	432	6
	TF	E	9.5	25	417	463	6

cipitate and is also weak. However, *Al: Zn: Mg* alloys are unlike the other alloys in that they do not form a coarsened zone, they recover their properties on ageing at ambient temperatures, and weld metal from the system ages to similar strengths to the parent metal.

Resistance to solidification cracking
It has been found that where high strengths welds are to be made, *Al: Zn: Mg* weld metal will give the required properties without cracking.

Weld toughness
Loss of toughness in the weld region is not significant for the majority of alloys in this system.

Stress corrosion
It has always been considered that *Al: Zn: Mg* alloys are prone to stress corrosion. This can, however, be prevented by careful choice of composition and control during manufacture.

Production of standards
At present the situation regarding the use of these alloys is that they are not covered by standards or codes of practice and hence are virtually unusable by people who like to work to the book! The need for such standards has fortunately been recognized and the Welding Institute has agreed to act as secretariat for the production of a draft standard and code specifications relating to *Al: Zn: Mg* alloys. In the more immediate term it appears likely that data will soon become available on the proof stress and ultimate tensile strength as a function of ruling section, the choice of suitable filler materials, and the use of the various welding processes with these alloys.

Many of the likely applications of these alloys are outside the sphere of our work. Current uses in other countries appear to be transport applications where currently 10,000 tons a year are being used in Europe, cryogenic vessels and bulk containers. However, these alloys show potential for bridge superstructures, especially bascule and other movable bridges, cranes (particularly welded plate gantry cranes) and space frames.

Welding processes
In the past few years there has been a striking change of attitude over the advisability of welding aluminium and its alloys. The older processes, oxy-acetylene welding, atomic hydrogen or manual metal arc, all suffered from the disadvantage that they required the use of highly corrosive fluxes to remove the refractory oxide layer which covers any aluminium or alloy surface. The effect of non-removal of the oxide was to prevent proper fusion between parent metal and the filler wire. Also, unless the flux was completely removed following each pass, localized corrosion could be initiated. It follows that it was essential to avoid any type of joint in which flux could be trapped. There were further problems caused by the heat input during welding which could cause buckling. It is understandable that the idea of welding aluminium was rarely popular, and it was considered a slow and skilful business to produce full strength welds even if this was metallurgically possible.

Aluminium is now considered to be weldable by a whole range of modern welding processes, including such exotic ones as ultrasonic welding, explosive welding, electron beam welding and laser welding. However, the whole story of welding aluminium is bound up with the experiments which were carried out by the Northrop Aircraft Company in the United States in 1944 and the subsequent production of workable inert gas welding processes.

Since then a considerable amount of development has been done in many countries, and we can now identify three processes which have resulted from this work. These are *inert-gas tungsten-arc* (occasionally called tungsten argon arc or more commonly TIG), *inert-gas metal-arc* (MIG) and *carbon dioxide metal-arc* (CO_2). TIG and MIG are both used for welding aluminium, although MIG is the later development and has many advantages. CO_2 welding is a modified form of MIG and obviously uses carbon dioxide instead of an inert gas as the shielding agent for the weld pool. It was developed as an economic and practicable means of applying gas-shielded metal-arc welding principles to the jointing of steel.

TIG and MIG processes with aluminium alloys have many advantages over the processes which they have displaced. They are easier to operate in all positions and can be used on a wide range of metal thicknesses giving high quality welds. They are also considerably faster and require no flux to remove the oxide film, because the arc itself is capable of providing the cleaning action.

The cleaning action of the arc
In the early days of inert gas welding it was discovered, during experiments into the significance of arc polarity, that the arc was capable of dispersing the refractory oxide surface films which invariably cover aluminium and some other metals. Dispersal occurs only on the negative pole of the arc and hence, for the cleaning action to work on the workpiece and weld pool, the electrode must be positive. Where a.c. current is employed the oxide dispersal takes place only on the half cycles when the work is negative.

Two theories have been widely accepted to explain cleaning action. The first regards it as a consequence of 'ion bombardment' and the second theorizes that the cathode spot from which the arc originates has preferential action for the oxide particles and other impurities. Vaporization of oxide and underlying metal is believed to occur and some freed oxide floats away to the edges of the weld pool.

This cleaning action is fundamental to the growth in the application of inert gas welding

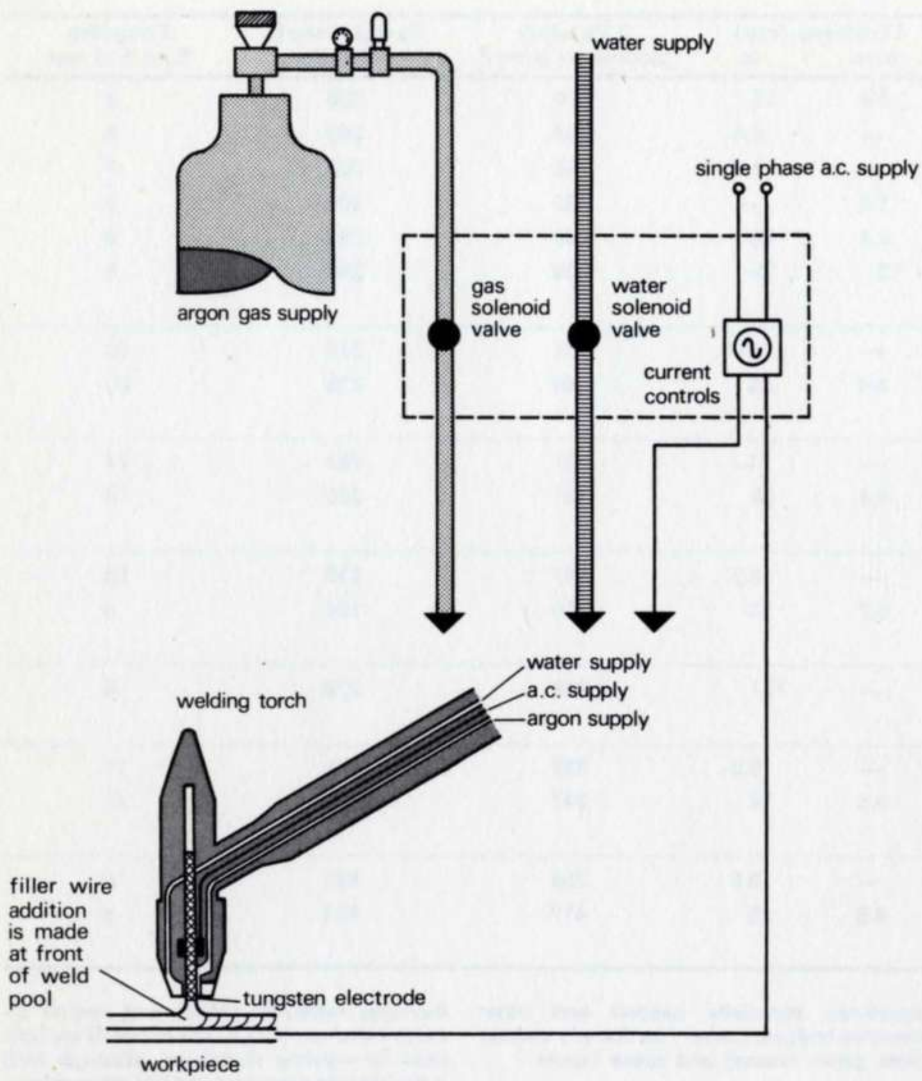


Fig. 2
Diagrammatic representation of inert-gas tungsten-arc process

processes to aluminium, since by doing away with the necessity for fluxing of the work, speed of operation is increased, the chance of corrosion due to the fluxes is prevented and one no longer has to design joints in order to prevent entrapment of the flux.

The inert-gas tungsten-arc (TIG)

The basic principle of this process, represented in Fig. 2, is that an arc is struck between a non-consumable tungsten electrode and the workpiece. The welding torch containing the electrode also contains the inert gas supply which flows from the nozzle and maintains an envelope of gas around the electrode. This prevents oxidation of the weld pool. In the UK the gas used is high purity (welding quality) argon, but in the United States helium or helium/argon mixtures are commonly used, principally because of the natural occurrence of helium there.

The torch is held at an angle of about 80° to the workpiece, sweeping the argon ahead of the arc. TIG is mainly recommended for situations where no filler metal is required. Where filler is used with TIG, great care must be taken to prevent disturbance of the gas envelope. It is generally advisable to keep the filler rod within the gas shield at all times, and where this addition is carried out mechanically it is fed into the leading edge of the weld pool. The filler is in the form of bare rod, of a suitable composition.

Welding aluminium alloys by TIG is invariably carried out using a.c. current. This is essentially a compromise between the requirement of the cleaning action of the arc which is explained above, and the heating action of the arc on the workpiece and welding torch.

The arc does not heat uniformly at each pole, the majority of the heat being generated at the positive side. Consequently we would normally require the workpiece to be positive and the electrode negative, and this is the case in the majority of electric-arc welding operations. But there is no cleaning action on a positive workpiece and hence welding with direct-current electrode negative is unsuitable. Nor is direct current electrode positive acceptable because the torch would be far too hot, even with arrangements for cooling, and hence alternating current is used. Even so the nozzle of the welding torch gets very hot and this is either air-cooled and made of ceramic material or is water-cooled in equipment sets intended to operate at high currents (more than 150 amps).

The use of a.c. is not without its own problems, the first one of which is called 'inherent rectification' (Fig. 3). Here the a.c. wave is unbalanced with the cleaning portion of the arc decreased for the workpiece. The reason for this is a fundamental difference between the emissivity of the tungsten electrode and the aluminium. There are several consequences of this, including 'arc blow' (the deflection of the arc by a magnetic field) and transformer inefficiency. For maximum weld quality it is vital to overcome inherent rectification, and this can be achieved by the incorporation of a bank of electrolytic capacitors in the circuit or by using d.c. storage batteries to compensate for the overall d.c. component of the waveform.

Another problem is the difficulty of maintaining the arc, which arises from having to extinguish it twice in every current cycle when the current falls to zero. When the electrode becomes negative the arc ignites satisfactorily but when positive it does not re-ignite until the current has reached an open circuit voltage sufficient to break down the oxide film. The standard method of overcoming this is to provide an arc generator which is synchronized to provide high frequency sparks at the moment of zero current. The arc is then termed 'stabilized'.

Inverse rectification is a further problem encountered during starting the arc. It is the failure of the arc during its negative half cycles and results in total failure of the arc. It does not occur when the electrode has warmed up and the usual way of preventing it is to switch the suppressing capacitors out of circuit during striking and warming up. Great care must be taken to ensure that the electrode is well maintained. It should be protected from oxidation by allowing argon gas to flow during the cooling period (most sets do this automatically) and should not be allowed to touch the weld pool (or a carbon block for starting the arc). When welding aluminium by a.c. the tip will superficially melt and acquire a hemispherical shape instead of the usual pointed one. This is acceptable.

The TIG process does not achieve the same penetration as MIG and for practical and economic reasons it is normally limited to welding sheet metal or small parts in thicknesses from 1 mm to 10 mm (.04 in.–0.4 in.) and for single or double sided butt joints and edge joints. It is a process which is readily mechanized and, when used in this way, automatic addition of filler wire may be used. This is termed 'cold wire feed'. The currents used may be from 25–350 amps for a.c. work, depending on metal thickness and joint profile.

The inert-gas metal-arc (MIG) process

As examination of Fig. 4 shows, MIG superficially bears a considerable resemblance to TIG, from which it developed. The fundamental difference is that here, unlike the TIG process, the electrode is consumable and is the filler rod for the weld. It is of related composition to the parent metal. The fact that the electrode is consumable is implied by the term 'metal-arc' which appears in the full name of the process.

The equipment of the process basically consists of the torch, the wire drive unit, the gas supply, the water supply, and the power source. The torch is usually water-cooled and incorporates a guide tube and electrical contacts for the electrode wire. This wire feeds through the tube, picking up current in a conduit as it goes. The wire drive unit is usually mounted separately with the control unit, although some equipment has the drive rollers mounted on the welding torch. The gas used is argon, as in TIG. Both the argon and the cooling water supply are controlled by valves in the control unit. All controls are preset before welding commences and the welding operation is controlled by the trigger of the welding torch.

Power sources

In MIG welding we do not need to use alternating current with its inherent disadvantages. Direct current with electrode positive has the required cleaning action on the aluminium surface and generates heat at the positive electrode. The heat melts the electrode, which is also the filler wire and the filler is transferred to the workpiece by the action of the arc. There is considerable argument about the power sources available.

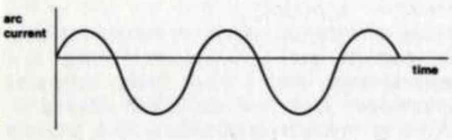


Fig. 3a
Inherent rectification due to characteristics of electrodes

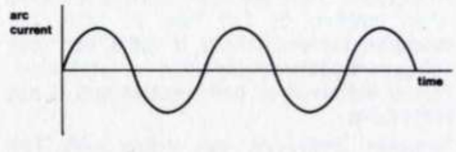


Fig. 3b
Balanced wave following incorporation of condensers

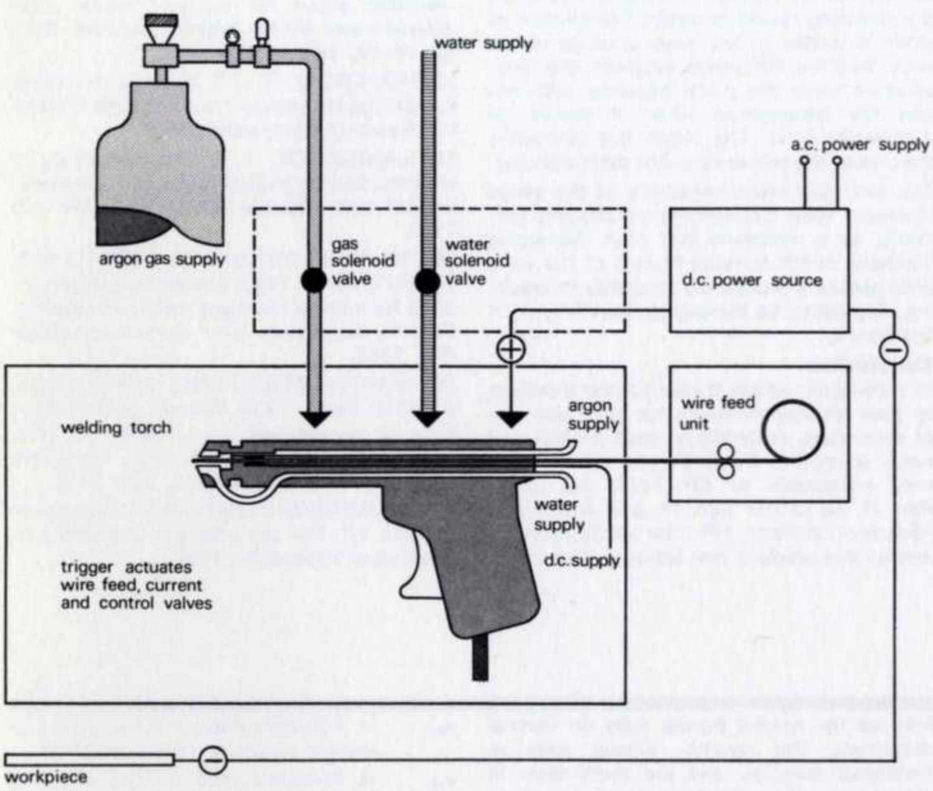


Fig. 4
Diagrammatic representation of semi-automatic inert-gas metal-arc equipment.

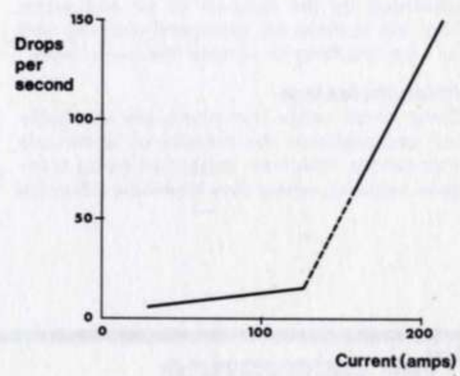


Fig. 5
Effect of current on the frequency of drop detachment with 0.064 in. diameter aluminium wire in argon.

These centre around the relationship between arc voltage and current, and it is this relationship which determines the behaviour of the self-adjusting arc. These characteristics are normally referred to as drooping, flat, or slope, depending on the shape of the voltage/current curve. Power sources may be of motor generator or transformer-rectifier type.

The self-adjusting arc

MIG equipment for manual use is classed as semi-automatic because of the incorporation of a mechanism for automatic control of arc length. In outline what happens is this; it is accepted that best quality welds are produced when arc length and arc voltage are maintained constant. However, changes in the gun to work distance are inevitable during manual welding and preventing this is not a means of maintaining constant arc length. There are two other principal variables, and these are the rate of feed of the wire and the burn-off rate of the wire. By suitable control either of these could be altered to maintain arc length constant. In practice it is difficult to make the wire drive respond instantaneously to the movements made by the welder and so control of burn-off rate is the more popular operational system.

The principle is that the wire feed works at a predetermined constant rate. The power source is chosen to give a large change in current to a change in arc voltage (arc length, arc voltage and arc current are all related). Movement by the welder causes an instantaneous change of length of arc which causes a corresponding change in arc voltage and hence current. The current change alters the burn-off rate and arc length changes accordingly, restoring arc equilibrium.

Metal transfer

The type of metal transfer that occurs in the arc is one of the fundamental factors in the application of MIG to aluminium, and is of as much importance as control of arc length. Aluminium is melted off the wire and is transferred as drops to the weld pool. Fig. 5 shows the relationship between the number of drops transferred per second and the welding current for 1.63 mm (0.064 in.) diameter aluminium wire with argon gas.

From the curve it is apparent that there is a clear transition point and that two types of metal transfer occur. Below the transition point the tip of the electrode develops as a large molten blob and transfer occurs by gravity. This condition is entirely unsuitable for welding.

Above the transition point the drops are transferred by plasma jet at a considerable speed, with the direction determined by the magnetic field of the arc. This mechanism is referred to as spray transfer and, due to the directional qualities of the arc, makes overhead and vertical welding possible. The upper limit on useful current range for aluminium is set by the force of the arc. Above about 400 amps, air is drawn into the gas envelope and the bead becomes partially oxidized.

Summary

In practice, the main advantages of MIG are that it is easier to work than TIG when filler additions are to be made and is hence much faster. It is more amenable to positional work and high quality welds can be made for all types of joint. Welding conditions are pre-set on the equipment and this is designed to maintain these conditions as far as possible. The process gives deep penetration (using current densities of 5–17 times those of TIG), and is applicable to sections over 7 mm (0.3 in.) thickness with efficient heat transfer. Hence it gives high output of good quality welds with minimum distortion at lowest cost. The equipment is, however, rather sophisticated and hence expensive.

Pulsed-arc welding

This is a later development of MIG and is used for welding the thinner gauges of aluminium. Its range of application normally lies between TIG and MIG. It is designed to overcome the problem that thin gauge aluminium must be welded using either thin wires which are difficult to feed through usual MIG equipment, or high currents with welding speeds which are difficult to maintain.

The current in pulsed-arc welding has two components; a background current which is used merely to maintain the arc, and superimposed pulses of high current at 50 to 100 cycles per second. These pulses melt the wire and transfer the globules to the weld pool. This transfer occurs only at the periods of peak current.

This process allows normal MIG wires and equipment to be used with the addition of a pulsed-arc source, and allows good control of welding small thicknesses.

Defects in welding aluminium

Nearly all the aluminium alloys can be welded to produce acceptably low levels of defects, but proper control of surface preparation and welding variables is necessary. The most common weld defects are porosity, non-metallic inclusions and cracks.

Porosity

Porosity is caused by inadequate pre-weld cleaning or by incorrect weld procedures, incorrect gas flow, damp work or damp wire. The risk of porosity occurring is greater during stopping and starting and with positional welding. The source of gas is hydrogen which may come from surface lubricants, from moisture on the surface or from the oxide film, which is partially hydrated. The solubility of hydrogen is high at the temperatures of the weld pool but with a fall in temperature this hydrogen is rejected with a corresponding fall in solubility. MIG is particularly prone to porosity problems because of the use of a thin wire with corresponding large surface area and hence area of oxide. Porosity is prevented by the removal of oil and water from the surface, by detergent washing and by wire brushing to remove the oxide layer.

Oxide inclusions

Some small oxide inclusions are inevitable and acceptable in the majority of aluminium applications (the main exception being cryogenic vessels), where they have little influence on strength. They are more commonly found when welding by TIG than by MIG, but excessive current density in MIG can give rise to particularly severe oxide contamination. This is the result of bad practice and is not acceptable.

Tungsten inclusions can occur with TIG welding. They may be due to welding with incorrect current settings but can be unavoidable at times.

Cracking

Aluminium alloys can be particularly prone to hot short cracking which is caused by compositional reasons.[4] During solidification the shrinking results in strain. The amount of strain is related to the freezing range of the alloy and the difference between the temperature when the metal becomes coherent and the temperature when it would be completely solid. The larger this difference the greater the tendency to hot short cracking.

The hot short crack sensitivity of the alloys increases with the amount of alloying elements to a maximum and then decreases. Elements which increase fluidity of the weld pool generally reduce the tendency to cracking, apparently by increasing the filling-in of the cracks.

Conclusion

In writing this article it was not the intention to give recommendations for structural use of aluminium, or details of weld profiles and weld strengths. Such things are covered very adequately in *CP 118*[1], *BS 3571: Part 1*[5], *BS 3019: Part 1*[6], and *Aluminium Federation Bulletin 19*[7]. The justification for writing this article is that although aluminium alloys have not often been used by us in structural applications, they are one of the range of materials which we should consider as part of our armoury. In writing it I acknowledge that I have freely borrowed information from various British Standards, Welding Institute publications and journals which are not listed in the references.

References

(1) BRITISH STANDARDS INSTITUTION. CP 118: 1969. The structural use of aluminium. BSI, 1969.

(2) BAKER, K. G. Al:Zn:Mg Light strong weldable alloys for industry. *Metal Construction and British Welding Journal*, **2**(2), pp. 65-68, Feb. 1970.

(3) HOULDCROFT, P. T. Welding processes. Cambridge University Press *and* British Welding Research Association. 1967.

(4) LANCASTER, J. F. The metallurgy of welding, brazing and soldering. Modern metallurgical texts. George Allen and Unwin Ltd., 1965.

(5) BRITISH STANDARDS INSTITUTION. BS 3571: Part 1: 1962. General recommendations for manual inert-gas metal-arc welding. Part 1. Aluminium and aluminium alloys. BSI, 1962.

(6) BRITISH STANDARDS INSTITUTION. BS 3019: Part 1: 1958. General recommendations for manual inert-gas tungsten-arc welding. Part 1. Wrought aluminium, aluminium alloys and magnesium alloys. BSI, 1958.

(7) ALUMINIUM FEDERATION *Information Bulletin 19*. The arc welding of aluminium. Aluminium Federation, 1968.

The relevance of laboratory measured parameters in field studies

David Henkel

Introduction

This paper was delivered at Cambridge University on Wed 31 March 1971 as part of the Roscoe Memorial Symposium.

It is always difficult to relate laboratory tests directly to field problems. We are often faced not only with obvious non-homogeneity but also with the less obvious anisotropy which results from both microstratification and stress history.

The practice of geotechnical engineering requires that both non-homogeneity and anisotropy are handled in such a way that satisfactory engineering solutions are achieved in both the design and construction processes. In order to obtain these solutions we need some basic understanding of the way in which anisotropic materials behave and it is often more useful to carry out a few tests designed to throw light on the basic behaviour rather than produce a mass of test data of a routine nature.

The writer has recently had the opportunity to review critically our knowledge of the London Clay in relation to the design and construction of deep basements. In this process the value of selected tests in throwing light on the degree of anisotropy in London Clay has become apparent.

In the study of the problem the results of three types of undrained tests have been used. The first are the normal triaxial tests on vertical specimens, the second, triaxial tests on horizontal samples, and the third, tests in plane strain where the direction of zero strain is horizontal.

In order to understand the significance of the results of these tests it is necessary to develop theoretical relationships between the physical parameters and the restraints imposed by the test procedures.

Theoretical background

In the absence of significant tectonic movements the vertical direction will be the axis of symmetry for any anisotropy which might exist in a sedimentary soil. In heavily overconsolidated clays the initial portions of the stress strain curves and the stress paths in undrained tests are essentially linear and reversible. It is therefore possible to examine the relationships between stress path inclination and apparent Young's moduli within the framework of the theory of elasticity. Some aspects of the behaviour of anisotropic soils have been discussed by Pickering.[2]

The strains in the vertical and horizontal directions may be written as follows:

$$\varepsilon_v = \frac{\Delta\sigma_v'}{E_v'} - \nu_{vh}\frac{\Delta\sigma_{h1}'}{E_h'} - \nu_{vh}\frac{\Delta\sigma_{h2}'}{E_h'} \qquad (1)$$

$$\varepsilon_{h1} = -\nu_{hv}\frac{\Delta\sigma_v'}{E_v'} + \frac{\Delta\sigma_{h1}'}{E_h'} - \nu_{hh}\frac{\Delta\sigma_{h2}'}{E_h'} \qquad (2)$$

$$\varepsilon_{h2} = -\nu_{hv}\frac{\Delta\sigma_v'}{E_v'} - \nu_{hh}\frac{\Delta\sigma_{h1}'}{E_h'} + \frac{\Delta\sigma_{h2}'}{E_h'} \qquad (3)$$

Where:

ε_v is the strain in the vertical direction

ε_{h1}, ε_{h2} are the strains in the two orthogonal horizontal directions

E_v' is the effective stress Young's Modulus in the vertical direction

E_h' is the effective stress Young's Modulus in the horizontal direction

ν_{hv} is Poisson's ratio for the effect of vertical strains on horizontal strains

ν_{vh} is Poisson's ratio for the effect of horizontal strains on vertical strains

ν_{hh} is Poisson's ratio for the effect of horizontal strains on horizontal strains in the orthogonal direction.

From considerations of energy it can be shown that

$$\frac{\nu_{hv}}{E_v'} = \frac{\nu_{vh}}{E_h'} \qquad (4)$$

$$\text{if} \quad \frac{E_h'}{E_v'} = R \quad \text{then} \quad \nu_{vh} = R\nu_{hv} \qquad (5)$$

ν_{hh} is an independent parameter but it will be assumed that

$$\nu_{hh} = \tfrac{1}{2}(\nu_{hv} + \nu_{vh}) = \nu_{hv} \cdot \frac{1+R}{2} \qquad (6)$$

The strain equations can now be rewritten as follows:

$$\varepsilon_v = R\frac{\Delta\sigma_v'}{E_h'} - R\nu_{hv}\frac{\Delta\sigma_{h1}'}{E_h'} - R\nu_{hv}\frac{\Delta\sigma_{h2}'}{E_h'} \qquad (7)$$

$$\varepsilon_{h1} = -R\nu_{hv}\frac{\Delta\sigma_v'}{E_h'} + \frac{\Delta\sigma_{h1}'}{E_h'} - \nu_{hv}\cdot\frac{1+R}{2}\cdot\frac{\Delta\sigma_{h2}'}{E_h'} \qquad (8)$$

$$\varepsilon_{h2} = -R\nu_{hv}\frac{\Delta\sigma_v'}{E_h'} - \nu_{hv}\cdot\frac{1+R}{2}\cdot\frac{\Delta\sigma_{h1}'}{E_h'} + \frac{\Delta\sigma_{h2}'}{E_h'} \qquad (9)$$

The volume change of an element is given by

$$\Delta_v = \frac{1}{E_h'}\left[R(1-2\nu_{hv})\Delta\sigma_v' + \left(1-\nu_{hv}\cdot\frac{1+3R}{2}\right) \times \Delta\sigma_{h1}' + \left(1-\nu_{hv}\cdot\frac{1+3R}{2}\right)\Delta\sigma_{h2}'\right] \qquad (10)$$

Three types of undrained tests will now be considered:

I Triaxial tests on vertical specimens

II Triaxial tests on horizontal specimens

III Plane strain tests with $\varepsilon_{h2} = 0$

In all the tests the requirement is that $\Delta_v = 0$ while an additional constraint in the plane strain test is that $\varepsilon_{h2} = 0$.

Stress path inclination

The inclination of the stress paths in the tests may be written as follows:

Case I Triaxial tests on vertical specimens

Here $\Delta\sigma_{h1}' = \Delta\sigma_{h2}'$ and, from the requirement of no volume change,

$$\frac{\Delta\sigma_v'}{\Delta\sigma_h'} = -2\,\frac{[1 - \nu_{hv}\cdot(1+3R)/2]}{R(1-2\nu_{hv})} = M_{\mathrm{I}} \qquad (11)$$

Case II Triaxial tests on horizontal specimens

Here $\Delta\sigma_v' = \Delta\sigma_{h2}'$ and, for no volume change,

$$\frac{\Delta\sigma_v'}{\Delta\sigma_h'} = -\frac{[1 - \nu_{hv}\cdot(1+3R)/2]}{[R + 1 - \nu_{hv}\cdot(1+7R)/2]} = M_{\mathrm{II}} \qquad (12)$$

Case III Plane strain tests with $\varepsilon_{h2} = 0$

Here $\varepsilon_{h2} = 0$

$$\text{and } \Delta\sigma_{h2}' = R\nu_{hv}\,\Delta\sigma_v' + \frac{1+R}{2}\cdot\nu_{hv}\,\Delta\sigma_{h1}' \qquad (13)$$

For no volume change

$$\frac{\Delta\sigma_v'}{\Delta\sigma_h'} = -\frac{[1 - R\nu_{hv} - \nu_{hv}^2\cdot(1+3R)/2\cdot(1+R)/2]}{R[1-\nu_{hv}-\nu_{hv}^2\cdot(1+3R)/2]} = M_{\mathrm{III}} \qquad (14)$$

Undrained Young's Moduli

The value of the undrained Young's Modulus for the three cases may be expressed as follows:

Case I Triaxial tests on vertical specimens

$$\varepsilon_v = R\,\frac{\Delta\sigma_v'}{E_h'} - 2R\nu_{hv}\,\frac{\Delta\sigma_v'}{E_h' M_{\mathrm{I}}} \qquad (15)$$

$$\Delta\sigma_v = \Delta\sigma_v' - \Delta\sigma_h' = \Delta\sigma_v'\left(1 - \frac{1}{M_{\mathrm{I}}}\right) \qquad (16)$$

$$\text{and} \quad E_{u\mathrm{I}} = \frac{E_h'}{R}\left(\frac{M_{\mathrm{I}} - 1}{M_{\mathrm{I}} - 2\nu_{hv}}\right) \qquad (17)$$

Case II Triaxial tests on horizontal specimens

$$\varepsilon_{h1} = \frac{\Delta\sigma_{h1}'}{E_h'} - \nu_{hv}\cdot\frac{1+3R}{2}\cdot M_{\mathrm{II}}\,\Delta\sigma_{h1}' \qquad (18)$$

$$\Delta\sigma_{h1} = \Delta\sigma_{h1}' - \Delta\sigma_v' = \Delta\sigma_{h1}'(1 - M_{\mathrm{II}}) \qquad (19)$$

and

$$E_{u\mathrm{II}} = E_h'\left(\frac{1 - M_{\mathrm{II}}}{1 - \nu_{hv}\cdot(1+3R)/2\cdot M_{\mathrm{II}}}\right) \qquad (20)$$

Case III Plane strain tests with $\varepsilon_{h2} = 0$

$$\varepsilon_v = R\,\frac{\Delta\sigma_v'}{E_h'} - R\nu_{hv}\,\frac{\Delta\sigma_v'}{E_h' M_{\mathrm{III}}} - R\nu_{hv}\,\frac{\Delta\sigma_v'}{E_h'}\left(R\nu_{hv} + \frac{\nu_{hv}}{M_{\mathrm{III}}}\cdot\frac{1+R}{2}\right) \qquad (21)$$

$$\Delta\sigma_v = \Delta\sigma_v' - \Delta\sigma_h' = \Delta\sigma_v'\left(1 - \frac{1}{M_{\mathrm{III}}}\right) \qquad (22)$$

$$\text{and } E_{u\mathrm{III}} = \frac{E_h'}{R} \times \left[\frac{M_{\mathrm{III}} - 1}{M_{\mathrm{III}} - \nu_{hv} - \nu_{hv}^2[M_{\mathrm{III}}R + (1+R)/2]}\right] \qquad (23)$$

Note that in plane strain tests the E_u values in the horizontal and the vertical directions are the same.

Relative numerical values

In order to calculate relative numerical values for the stress path inclinations and undrained Young's moduli, a value for ν_{hv} of 0.12 deduced by Wroth[7] from the data in a thesis by Webb[6] has been used.

In Table I that follows the values of $\Delta\sigma_v'/\Delta\sigma_h'$ and E_u have been computed for the three cases for various values of R.

Table 1

Case I

R	$\Delta\sigma_v'/\Delta\sigma_h'$	E_{uI}
1.0	−2	1.340 E_h'
1.25	−1.51	
1.5	−1.175	1.024 E_h'
1.75	−0.941	
2.00	−0.764	0.879 E_h'

Case II

R	$\Delta\sigma_v'/\Delta\sigma_h'$	E_{uII}	E_{uII}/E_{uI}
1.0	−0.5	1.34 E_h'	1.0
1.25	−0.430		
1.5	−0.370	1.220 E_h'	1.19
1.75	−0.320		
2.0	−0.276	1.143 E_h'	1.30

Case III

R	$\Delta\sigma_v'/\Delta\sigma_h'$	E_{uIII}	E_{uIII}/E_{uI}
1	−1.0	1.786 E_h'	1.33
1.25	−0.769		
1.5	−0.611	1.460 E_h'	1.42
1.75	−0.499		
2.0	−0.413	1.305 E_h'	1.49

Test results

The linear portion of the average stress paths for the three types of tests are shown in Fig. 1. The data on which these stress paths are based have been obtained from tests carried out at Imperial College by Ove Arup & Partners, and from data published by Bishop, Webb and Lewin,[1] and Som.[3]

For the vertical triaxial samples the slope of the stress path, $\Delta\sigma_v'/\Delta\sigma_h'$, is about 1.0 while for the horizontal triaxial samples and plain strain samples the slopes are 0.35 and 0.60 respectively.

In Fig. 2 the variation of stress path inclination with changes in R or E_h'/E_v' given in Table I have been plotted. The points at which the experimental values of stress path inclination intersect the respective curves are also indicated. It will be noted that the test data implies a value of E_h'/E_v' of about 1.6. This value is in agreement with the results of consolidation tests on vertical and horizontal samples described by Ward *et al.*[4] in which the average implied value of E_h'/E_v' was found to be about 1.6.

For a value of E_h'/E_v' of 1.6 the ratio of the undrained Young's moduli in plane strain and vertical triaxial tests should be about 1.4. The test data ranged from 1.3 to 1.5. On a far wider range of tests Ward *et al.* (1959 and 1965) found that the ratio of undrained Young's moduli for horizontal and vertical triaxial samples varied between about 1.2 and 2.4 with an average value of 1.6. These results are to be compared with the theoretical value of about 1.2.

Conclusion

The simplified theoretical treatment of anisotropy used in this paper has enabled the behaviour of London Clay in various types of tests to be related. In spite of some anomalies it appears that the assumption that E_h'/E_v' is approximately 1.6 enables reasonable predictions of the relative behaviour in a variety of circumstances to be made. The magnitude of the anisotropy is such that, if reasonable predictions of field behaviour are to be made on the basis of laboratory tests, the anisotropy must be taken into account.

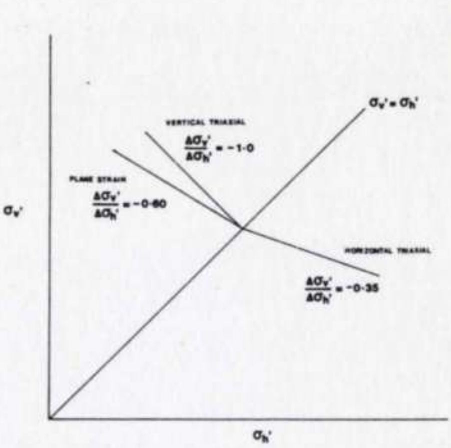


Fig. 1
Stress paths in undrained tests

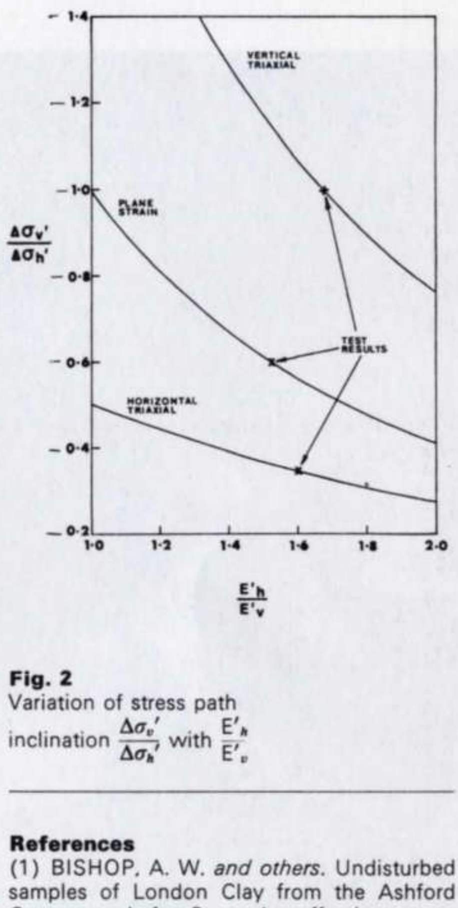


Fig. 2
Variation of stress path inclination $\frac{\Delta\sigma_v'}{\Delta\sigma_h'}$ with $\frac{E'_h}{E'_v}$

References

(1) BISHOP, A. W. *and others*. Undisturbed samples of London Clay from the Ashford Common shaft. Strength—effective stress relationships, by A. W. Bishop, D. L. Webb and P. I. Lewin. *Geotechnique*, **15**(1), pp. 1–31, 1965.

(2) PICKERING, D. J. Anisotropic elastic parameters for soil. *Geotechnique*, **20**(3), pp. 271–276, 1970.

(3) SOM, N. N. The effect of stress path on the deformation and consolidation of London Clay. PhD Thesis, University of London, 1968.

(4) WARD, W. H. *and others*. Further studies of the properties of London Clay, by W. H. Ward, S. G. Samuels and M. E. Butler. *Geotechnique*, **9**(2), pp.33–58, 1959.

(5) WARD, W. H. *and others*. Properties of the London Clay at the Ashford Common shaft. In situ and undrained strength tests, by W. H. Ward, A. Marsland and S. G. Samuels. *Geotechnique*, **15**(4), pp. 321–344, 1965.

(6) WEBB, D. L. The mechanical properties of undisturbed samples of London Clay and Pierre shale. PhD Thesis, University of London, 1966.

(7) WROTH, C. P. Private communication, 1970.