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Front cover: Stained glass at the Chester Bell Tower. (Photo: Harry Sowden)
Back cover: Berry Lane Viaduct: trial section of beam

Co-operation between architects and allied professions

Ove Arup

I am invited to write a short piece (600–800 words) on the above theme. Not just a quart, but a couple of cubic metres into a pint bottle! A tough proposition! But I'll try.

First the theory.

It is a fact, that any building or construction nowadays must be preceded by a design or a number of designs, if you like—what I call a Total Design—which is the key to what it is intended to build and how it is going to be built.

It is also a fact that such a Total Design can rarely be produced by one man. It requires the collaboration of several people who have been trained to deal with particular aspects of this Total Design—a spacial arrangement, structure, services and so on. But if every specialist produced the perfect solution to his particular problem, these part designs would simply not fit together to produce a successful whole. And of course it is the whole we want. In fact we want more and more things, as many as we can get for our money. And we want to satisfy not only the client, but also the people using or having to live with the building or bridge, or whatever it is. We want it to harmonize with the environment and we want to ensure that there are no destructive or anti-social side effects.

That this calls for close integration of the part designs and close collaboration between all the people who should or who do influence the design is obvious, and is all the theory that is needed, as far as I can see.

This consideration of every aspect of the design and fitting the bits together to produce the best possible total solution is the real art of designing. And as in all art, there is more to it than meets the eye. It is not only a question of meeting all the different requirements in the fullest possible way at the least cost, it depends also on the simplicity or elegance of the solution, the felicity of design which has the power to inspire those who comprehend it, and which is the reward the designers hope for. It can rarely be produced without taking great pains and it cannot be defined or measured.

How to ensure that every design gets the care and attention needed to produce a thing of quality is, I suppose, the question covered by the second part of the title: 'the effective practice of co-operation between planners, architects, etc.' That is a much more difficult question to deal with because there is obviously no general rule and none that by itself would guarantee the desired result. There is even no generally accepted criteria which can decide whether it has been achieved or approached in a particular case. But whereas abundant lip service has been paid for years and years to the need for such co-operation in all its forms (in fact the energy spent in moving all those thousands of lips for the last 40 years must have contributed to our present energy crisis), the progress in 'effective practice' has been disappointing.

The reasons for this are manifold and well known. The different professional institutions have nursed their separateness for too long to be able to change their attitude in a few years. Educational establishments have concentrated on turning out specialists, thereby fostering a narrow outlook. The whole organization of the building industry has its roots in the past and cannot adapt itself so quickly to the vastly changed social and technical climate of today. It is fragmented, full of vested interests, of rivalries between professions and trades about status, money, responsibility and influence. It clings to outworn practices and is governed by partly irrational legal restrictions. There is also the traditional chasm between design and execution and the stranglehold of the costing system leading to the costing of non-existing designs as if quality and integration did not matter. This is not the whole picture, of course, but it is things like that which make it difficult to create the the conditions where collaboration can take place between those technicians who really understand the business of building and who are eager to get on with the job of finding the right answers, i.e. the right designs at the right price.

But what then is 'the effective practice of co-operation' and how can it be brought about?

My short answer is that you learn co-operation by practising it. You will then learn to understand the other fellow's point of view and the value of his contribution and you will learn to see your own work as part of a whole, which will gradually tend to make your collaboration more effective. But, of course, it depends on the quality of the people in the team, and especially on the leaders inside each of the disciplines represented. If they have the necessary enthusiasm and will to produce a work of quality, half the battle is won.

Talking or writing about collaboration and integration is not nearly as effective as we have seen. But that does not mean that it is useless—on the contrary—it is a necessary preparation. But also here it depends on the quality of the talk. The tendency is for talking to be done by those who are good at talking. But it is much more important that it is done by those that know how to build well, and who understand the need for integration. Each job is in many ways unique, but designers can help each other by explaining how they have surmounted certain difficulties on their job, especially in the domain of integrating the various disciplines, including the method of construction or manufacture which has or should have a major influence on all designs where economy is of importance—which in varying degrees applies to all of them. And, even after 40 years of talking about collaboration and integration, it is still necessary to convince authorities, private clients and their legal and financial advisors that it will repay them handsomely to ensure that they have got the right design before they start to build and that this means that the advice of all those who have a significant contribution to make to the Total Design should be sought *before* the basic design decisions are taken. This means, at the very least, that they must have the opportunity to meet and harmonize their requirements on the basis of priorities established by the leaders of the team and the clients. This can be a long process but out of this melting pot a purified design should emerge which will more than repay the effort expended.

It is a sad fact that most of the people who are not designers but have the power to get things built, do not understand this. They think that designing is a routine matter. It can be reduced to that and that is what is the trouble with our environment.

Construction of the Berry Lane Viaduct

Jørgen Nissen
Peter Renouf

The Berry Lane Viaduct is now being built as part of a contract for the construction of the London North Orbital Road from Hunton Bridge to Maple Cross.

We were appointed to design this viaduct in April 1971 and had completed the design in the summer of 1972. We wrote about it in this Journal in September 1972 and expected then to call tenders a month or two later and to start on site early the following year. The planning procedures were, however, held up at this stage and the scheme was delayed.

Tenders were invited in April and submitted in July 1973 for the whole contract. In a case like this, where we are responsible for only a small part of a large scheme, the contract can be arranged in various ways. The possibilities are: two contracts with two engineers (and perhaps two contractors), one contract with one engineer and one contract with two engineers. We much prefer the last alternative and this was chosen for this scheme with the Eastern Road Construction Unit acting as the engineer for the main part of the contract and with Ove Arup and Partners as the engineer for the Berry Lane Viaduct.

Five contractors were invited to submit tenders. Prices for the viaduct varied more than the prices for the whole contract and there was no obvious correlation between the total tender figures and the prices submitted for the viaduct. The tenders were evaluated for the whole contract and it was awarded to Costain Civil Engineering Ltd. Their tender for the viaduct was very close to our estimate.

Costains started work in October 1973. Whilst they did not expect to need the full 24 month contract period to build the viaduct, they started almost immediately, in order to be able to complete the viaduct early; at least to the stage where it could be used for construction traffic. They would in this way gain easy access between two sections of the road works which were otherwise separated. At the time of writing (February 1975), the tabletops have all been built and four out of the seven suspended spans have been completed. No finishing works have yet been started. Costains expect to have the viaduct open to construction traffic by May, with completion in the autumn.

Fig. 1
The site in August 1974 (Photo: Theodore Greville Studios)

Fig. 2
The site in December 1974 (Photo: Theodore Greville Studios)

Fig. 3
Delivery of precast beams (Photo: Harry Sowden)

Construction method

The viaduct was designed around a particular construction procedure which we had chosen because it would work within the restrictions imposed by the site. We described it in the earlier article but it might be useful to repeat the main argument.

The viaduct is placed in a deep wooded valley. When built it should not act as a barrier and the trees should be preserved as much as possible. It should be built in such a way that the working area is the smallest possible and the natural character of the area could easily be restored, so that a scar would not remain for long. The only direct access to the bottom of the valley is on narrow roads through sensitive residential areas and major constructional traffic should be encouraged to use the line of the new road to the northern end of the site at the top of the valley. Finally, the structure should be capable of being built over the busy railway line. These objectives led to ideas about the construction which again led to the proposed design.

For us, the objectives were important, not the way in which the structure is built; not in itself at any rate. We cannot, and should not, tell contractors how to build the structures we design, but we should know at least one good way of building them particularly if we limit the contractor's choice. In this case, we restricted him to a very limited works area under the viaduct and we put restrictions on his traffic on the more sensitive roads near the site.

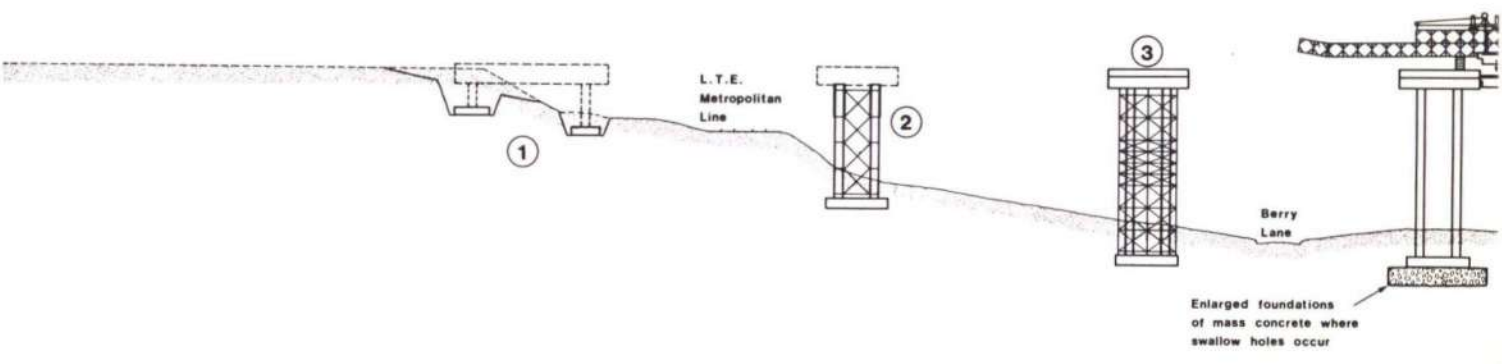
This has worked out well. Costains have developed a method of construction which has allowed them to build the viaduct in an efficient manner within the restrictions imposed on them and at the same time has met the objectives we had in mind. The construction sequence is shown in Fig. 4. Work started at the north abutment at the main entry point for access and progressed towards the south abutment. The various activities in the sequence are described in more detail in the following.

Foundations

The foundations did not present any special construction problems. They are all spread footings on chalk. We had extended the original site investigation to search for swallow holes and had found some. In those cases we enlarged the bases to span across the holes. The excavation confirmed our earlier findings and the bases were built as designed.

Columns

The columns are quite tall, up to 18 m in height and it would obviously not be sensible to cast them in one. We did not specify the size of lift but asked that all construction joints should be at the same level. We wanted a rough finish and specified a boarded finish as produced by rough sawn Douglas Fir boards 75 to 90 mm wide, but we did not want any particular feature at the joints. Costains chose to start with a 2 m lift on all columns,



1 Column bases founded on chalk 2 Columns 3 Table-tops 4 Precast beams placed by gantry

this always being below ground level, and then to use a standard shutter system with a pour height of 5.6 m. This then gave three further lifts for the large columns. The shutters were made as two semi-circular steel frames which were connected by hinged bolts. The frames were designed to clamp onto the previous pour and did not need further support from the ground. Erection and striking were easy using a crane to place and remove the half shutters. Construction joints were formed by just leaving the concrete after vibration and a grout-tight seal was formed using a 50 mm wide foam strip stuck to the preceding lift. Concrete was pumped for all pours using a boom pump.

The boards were tongued and grooved and treated with a proprietary protective coating similar to polyurethane. A shutter release agent was sprayed on before each use. Striking times were often around 24 hours and with a minimum of maintenance the shutters were still producing an excellent finish after the 27 re-uses required for completion.

Tabletops

The construction of the tabletop presented the contractor with the problem of supporting 500 tonnes of concrete some 18 m above ground. The plan area of each tabletop is greater than the area of the foundation and to cope with this, Costains chose to use falsework with raking towers made from military trestling. The falsework supported a grillage of steel beams which formed the base for the shutter panels. The side shutters were propped against the steel grillage and a system of through ties was used to stabilize the form. Sand jacks were used to release the shutter panels after the concrete had set. The trestling was erected and dismantled in sections which were handled by cranes with telescopic jibs.

The shutter lining was similar to that used for the columns but wider boards were used. The boards were treated in a similar way and the same excellent finish was produced.

The tabletops were cast in one pour which usually took some nine hours to complete. This meant that a large face of concrete had to be kept alive throughout this period. The pour was not large enough in the contractor's opinion to justify the use of two pumps and a breakdown in supply would inevitably lead to the formation of cold joints. To overcome this Costains proposed a retarding plasticizer and this proved so successful that it has since been used in all concrete for the viaduct. This additive meant that the concrete could be kept alive, even on a hot summer's day, long enough for a standby pump or an alternative pouring method to be arranged. The tabletops were cured using polythene sheeting and straw mattresses.

Precast beams

The precast beams have been manufactured for Costains by Ferro-Concrete and Stone Ltd. in their works at Retford. From there they have been transported by road to the site.

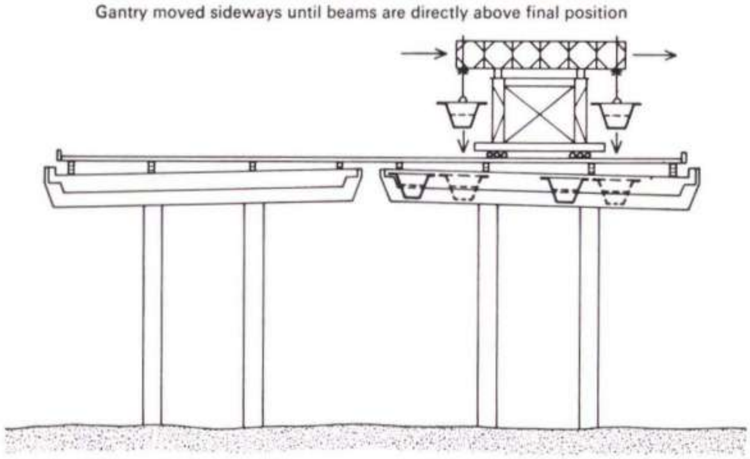
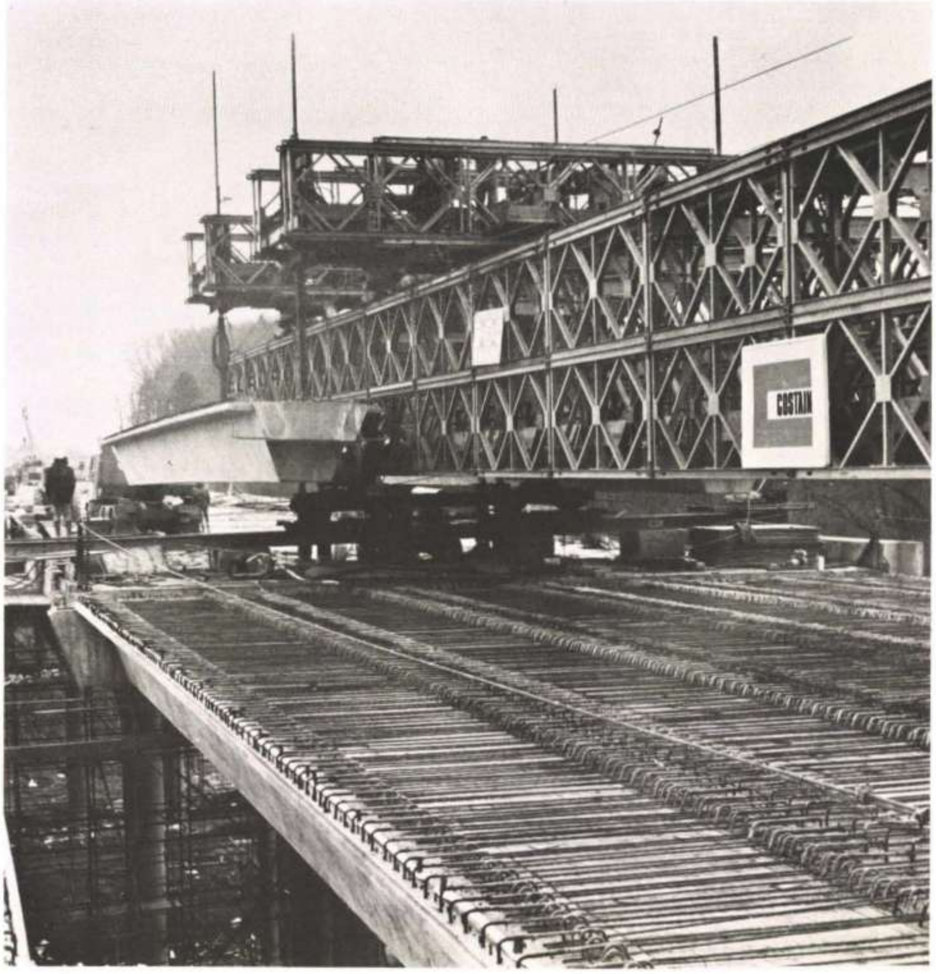
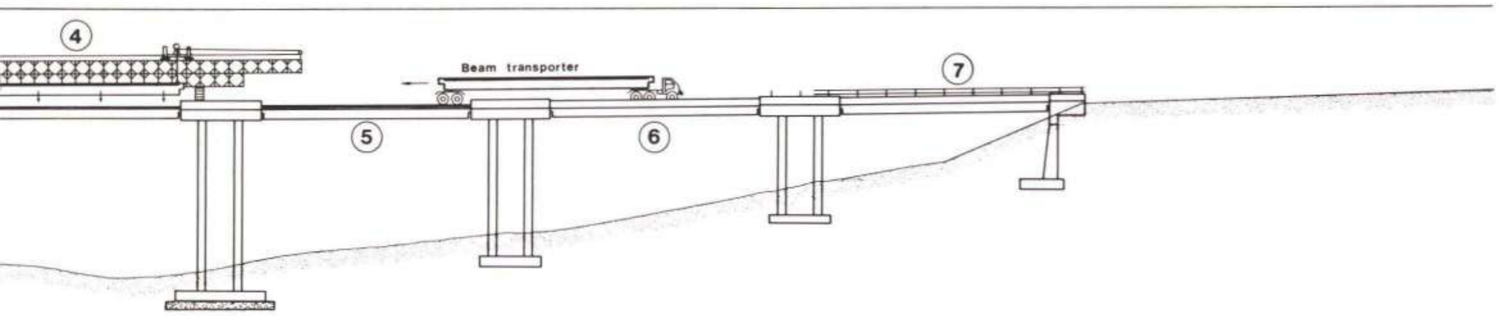


Fig. 5
Section through viaduct with gantry positioning beams

Fig. 6
Launching of precast beams (Photo: *Construction News*)





5 In situ deck slabs 6 Concrete edge beams on spans 7 Aluminium parapets and surfacing

All 84 beams have been cast in one steel mould. We wanted a smooth finish for the beams and we had trials made of a plywood shutter faced with glass fibre during the design period. This gave a very good finish and we specified this finish or a similar one. In the event, the contractor chose steel and we have no reason to regret his choice. The problem of glazing has generally been overcome and a good finish has been produced.

The production of the beams was very slow in starting up. The sub-contractor proposed originally to cast three beams per week and the erection programme on site was arranged around this production rate. He was, however, held up by the industrial dispute in early 1974, which caused a delay in the supply of steel, and by problems in aligning the segments of the shutter and fabricating the reinforcement cage. At this stage, the programme for the whole viaduct was threatened and we agreed to allow the sub-contractor to spot weld some areas of the reinforcement. Whether it was this small change or something else, it certainly had its effect. Within 19 weeks the remaining beams were cast and we had moved from the problems of scarcity to the problems of abundancy. The beams were now stored for longer than expected and we therefore had to cope with a greater upwards deflection due to creep than we had anticipated.

Beam erection

Each beam is 24m long and weighs 50 tonnes. After having studied a scheme for placing the beams with cranes, Costains chose to use a beam launcher made from Bailey Bridge panels. The launcher is shown on Fig. 6. The beams arrive on lorries from the factory and are picked up two at a time by lifting trollies which run on the top member of the launching girder. The trollies are winched across the span, and the launching girder, complete with beams, is then rolled sideways until it reaches the right position. The beams are then jacked down the necessary 3m onto their rubber bearings.

The in situ slab must be cast before the launching girder can launch itself across the next span. The contractor used at first a recoverable shutter system and cast the slab in two parts but to save time, he changed to a lost shutter system and he now pours the whole span in one. As soon as the slab has achieved sufficient strength, the girder is lowered onto rollers and it is then launched across the next span. Both carriageways are being constructed at the same time; first the beams are placed in the east span and this is then prepared for concreting while the beams are placed in the west span. Construction of the two decks of each span takes about four weeks.

The launching method has had its teething troubles but no major snags have developed. A high level of supervision has been maintained throughout all critical operations, both by us and by the contractor. At the time of writing the fifth of the seven spans is being constructed. The erection of the last span,

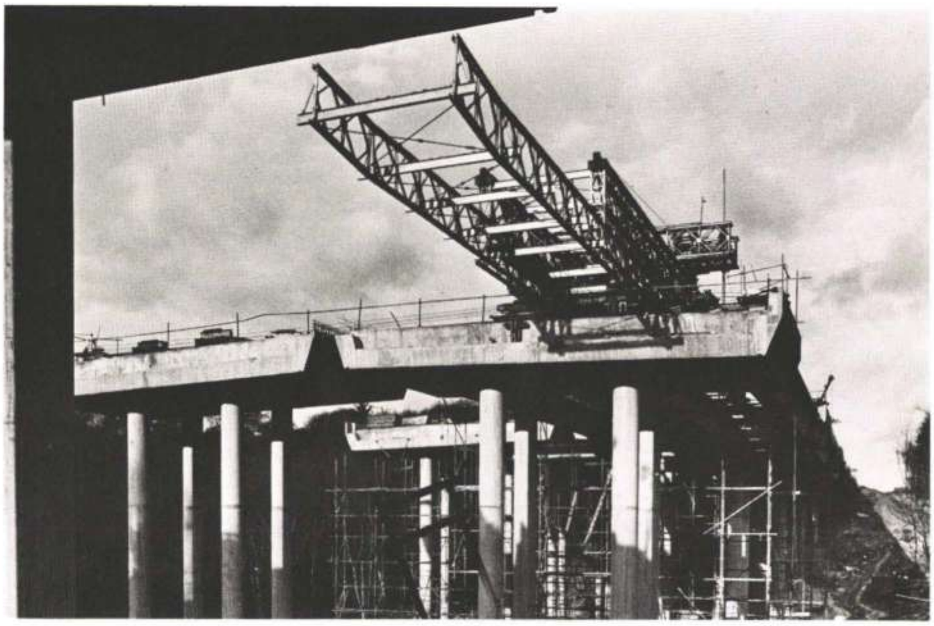
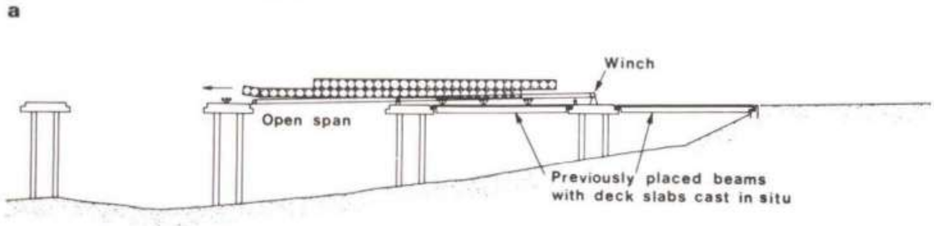


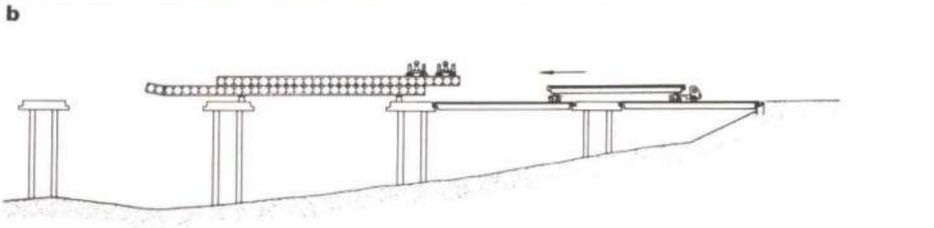
Fig. 7
The launching girder
is moved forward
(Photo: Harry Sowden)

Fig. 8
Part of nearly completed viaduct
(Photo: *Construction News*)

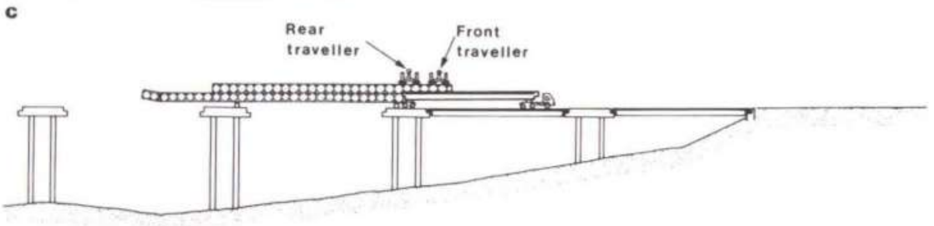
Fig. 9
The viaduct from beneath
(Photo: Harry Sowden)



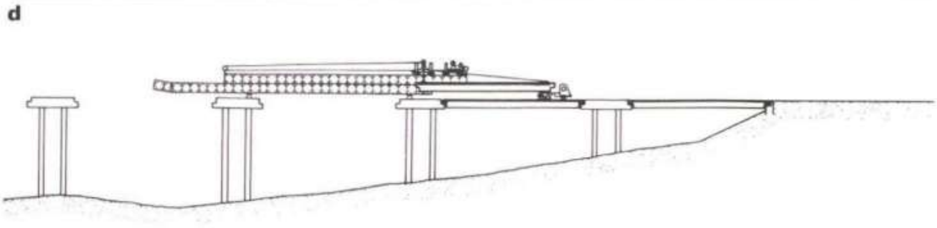
Gantry winched into position across open span



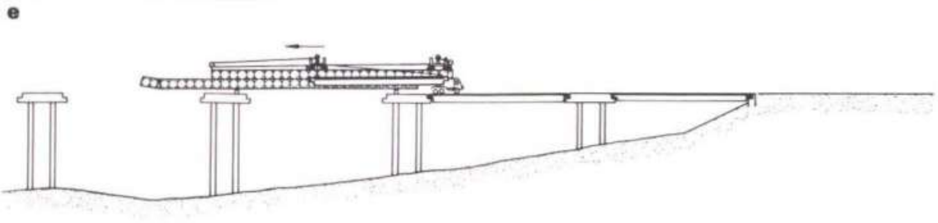
Beam transporter reverses on to bridge....



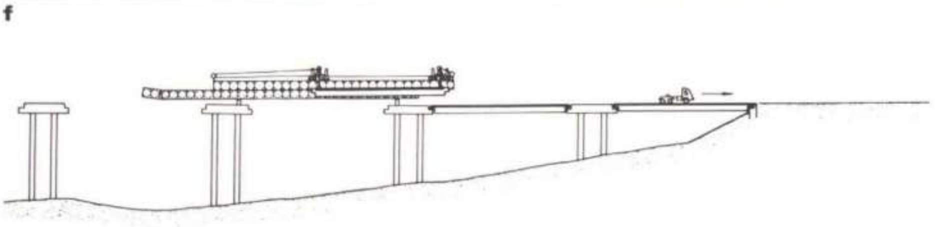
. . . and stops when rear end of beam is beneath rear traveller of gantry



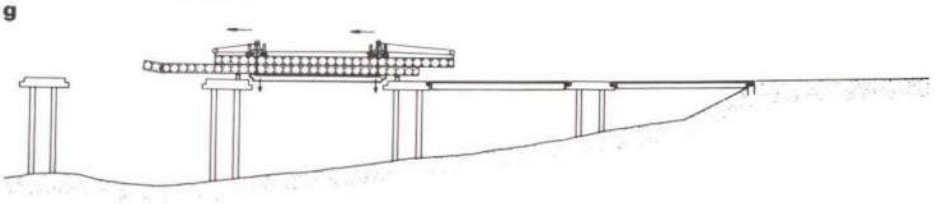
Rear traveller lifts rear end of beam; winching ropes positioned



Rear traveller moves along gantry until front end of beam is beneath front traveller



Front traveller lifts front end of beam off transporter



Both travellers move along gantry and lower beam on to bearings

Fig. 10, a to g
Construction of Berry Lane Viaduct by launching method.

that over the railway, is expected to start at mid-March and procedures have been agreed with London Transport. Most of the work on this span will have to be carried out during short two to three hour night sessions and this span will take somewhat longer to complete.

Site organization

The concept of one contract and two engineers carries with it some of the problems of divided responsibility and it is important that good communication is established between the two engineers. In this case our resident engineer has kept close liaison with the ERCU resident engineer and a good working relationship has been established. This has ensured that similar attitudes have been adopted to common problems and that decisions were uniform on the two parts of the contract. Many of the decisions taken by one of the engineers affect the whole contract and in these cases, joint action has been taken.

A third engineer is also involved in this contract. The London Transport Executive have an engineer resident on site and all works adjacent to and over the railway are carried out only after obtaining his approval and under his supervision.

Testing

Messrs. Sandberg were appointed as testing engineer for the whole contract and they have an extensive laboratory on site. Costain's tender was based on using excavated material as a source of general fill material, sub-base for the road and aggregates for structural and pavement quality concrete. With this in mind they set up a washing and screening plant and concrete batching plants. Sandbergs have provided a service for assessment of the suitability of all 'as dug' products as well as the usual tests for black top and concrete cubes, etc. Concrete was supplied by ready-mix companies at the beginning of the contract and this was used in most of the bases and in four sets of columns. The local suppliers could not, however, produce a consistently good concrete and by the time the Costain batcher came into operation in April 1974, only one ready-mix supplier remained on the approved list. With concrete now coming from the contractor's own plant, control of the batching process was much easier. The 'as dug' material could not, however, be sufficiently cleaned by 'single washing' and a thin coating remained on the aggregates. This impaired the bond strength between cement paste and coarse aggregate and the full design strength could not be reached for the class 45/20 mix. The problems were solved by 'double washing' but, coupled with the discovery of a 'shortfall' in the quantity of suitable material on the site, Costains decided that this was not economical and that they would import all aggregates. Supplies from the batcher have since then been very consistent.

Perhaps the most interesting part of the testing programme has been the use of 'Fresh Analysis', a new test method which enables the aggregate grading, water ratio and cement content to be known fairly accurately within one hour of placing. With this test, unsatisfactory concrete can be identified before test cubes results are available and before the concrete has set. On smaller pours faulty concrete can then be more easily removed and on larger pours, where the incorporation of a rogue batch has a less damaging effect, the fault in the mix can be located and corrective measures taken.

This test was introduced as a non-contractual test and it did meet with some scepticism on the part of the contractors. The test was, however, so successful that they later used it themselves for quality control of the batching plant.

With this higher level of control, they were able to reduce the cement content with obvious cost benefits.

The location of backfilled quarries by high resolution gravimetry

Carl Poster
Chris Cope

Fig. 1
The view northward along Millers Road (Photo: Carl Poster)

Fig. 2
The gravity meter, levelled upon its tripod, being read at this site's base station (Photo: Carl Poster)

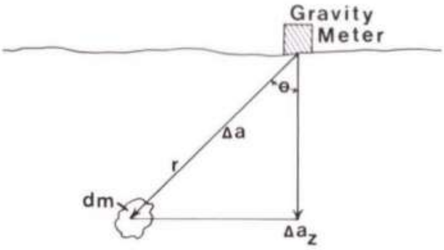


Fig. 3
Measurement by gravity meter

Introduction

During the initial site appraisal for the Byker section of the Tyne and Wear Metro in Newcastle-upon-Tyne, it was found that a substantial section of the route encountered a built-up area containing backfilled 19th century quarries (Fig. 1). As the design of this section of the railway included a cut-and-cover tunnel, an important consideration was the accurate location of the quarry faces.

The preliminary information on the site consisted of an archaeological desk study and a single borehole (Borehole A on Fig. 5). The desk study showed that sandstone quarrying had commenced prior to 1861 and by 1894 had reached its maximum extent. Backfilling began in the same year and was completed by 1938 when building upon the area started. This study also revealed the presence of a smaller quarry, to the south of the main workings, which had been used as a reservoir prior to being backfilled in 1894.

The borehole revealed 12.45 m of loose ash fill above a mudstone within the Carboniferous coal measures that stratigraphically underlie the sandstone.

Suitability of method

Because of the unreliability of old maps and plans, it became apparent that the position of the quarries would have to be confirmed.

A site investigation that involved considerable drilling would provide this confirmation, but would also involve substantial costs. Therefore, a geophysical survey was considered in advance of the main investigation to optimize the drilling and to provide preliminary data for design.

The location of shallow, vertical sandstone faces in contact with loose fill is not normally a difficult geophysical problem. Usually, contrasts in the electrical resistance or magnetic susceptibility of the different materials can be measured, or seismic techniques, in which changes in the velocity of elastic wave propagation are detected, can be used.

The choice of a suitable geophysical method is often limited by the site conditions. To assess these conditions, a brief site reconnaissance was made by a geophysicist. This visit confirmed that a majority of the area was covered by metalled roads, pavements and buildings and underlain by numerous services. Parked vehicles were found to restrict access along the streets during daytime and traffic was heavy during daylight hours. Traffic and industrial activity in the area also produced considerable ground vibrations. Finally, exposures of the fill material showed that it contained large amounts of metallic debris.

These conditions precluded the methods previously noted. Resistivity and seismic techniques rely on systematic contact with the earth, which would have been impractical in this area, due to the extensive building cover. The underground services and buried metal debris would have adversely affected resistivity and magnetic methods, while ground vibrations would have complicated or rendered useless the employment of seismic techniques.

However, it appeared that a gravity survey, in which changes in subsurface density are detected, might be applicable.

A gravity survey would not be affected by the presence of pavements, as it would not require contact with the earth, but the reading of the meter, which operates like a sensitive 'spring balance', would be affected by strong ground vibrations. For this reason the survey would have to be conducted between 2400 hrs. and 0600 hrs., when traffic and industrial plant operation would be at a minimum. The number of parked cars would also be reduced during this period, which would facilitate access. A further consideration was the magnitude of the density contrasts at the site.

At several outcrops around the site, the sandstone appeared to be extremely strong, buff cross-bedded and arkosic. A density of approximately 2.1 gm/cm³ was determined from three samples. Joints were in the order of 1 m apart and both horizontal and vertical joint sets were closed. Although high angle cross-bedding was prevalent, the true bedding seemed to be sensibly horizontal and the stratum to be essentially homogeneous across the site. In contrast, the fill material seen in exposures was quite variable in composition. Its density was estimated to be 1.5 gm/cm³, by analogy with the figure commonly estimated for dry unconsolidated clastic rocks. (This value was later confirmed by in situ density measurements.) It was therefore assumed that there was a density contrast of about 0.6 gm/cm³ across a 12 m deep contact and calculations showed that the resulting gravity anomaly was within the detection range of most gravity meters.

The gravity survey

The use of gravity measurements as a geophysical exploration technique has generally been confined to the structural study of large sections of the Earth's crust. Only recently, with the development of highly sensitive and portable gravity meters (Fig. 2), has it become feasible to employ the method for surveys of small scale, low amplitude gravity anomalies. [1], [2] The details of the theory on which gravity exploration is based are not included here and can be found in most geophysical texts.[3]

Exploration gravity meters detect relative changes in the vertical acceleration of the Earth's gravity field. An object's gravitational acceleration, a, is described by the equation

$$a = G\frac{m_e}{r_e^2} \qquad (1)$$

where G is the gravitational constant, m_e the mass of the Earth and r_e the distance of the object from the centre of the Earth. If the meter is moved from one observation point to another station nearer a feature of either lower or higher mass, a change in the acceleration of gravity, Δa, will be measured,

$$\Delta a = G\frac{dm}{r^2} \qquad (2)$$

where dm is either the deficiency or excess of mass of the feature and r is the distance from the meter to the centre of the anomalous mass. Actually, the meter only detects the vertical component, Δa_z, of this anomalous acceleration, which acts along radial lines (as shown in Fig. 3), thus:

$$\Delta a_z = G\frac{dm \cos\theta}{r^2} \qquad (3)$$

Exploration gravity meters do not measure the absolute acceleration of gravity; instead, they measure changes in this acceleration relative to an arbitrarily chosen base station. Measurements at a site can only be referred to a base station in the vicinity and, if comparisons of different sites are required, they must have a common base station.

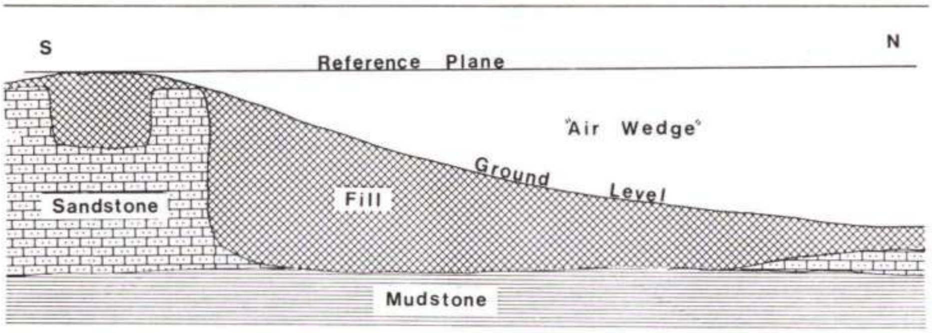


Fig. 4
Schematic north-south section through the site

The unit of acceleration of an object in the Earth's field is the gal (after Galileo), which is equal to 10 mm/s^2, and the Earth produces an acceleration of 980 gals. The milligal (1 mgal = 10^{-3} gal) is the commonly used unit, since the accelerations produced by anomalous masses are very small compared to that of the Earth. Modern gravity meters can detect changes of 0.01 mgal, or about one part in 10^8 of the Earth's field.

To produce a meaningful gravity map, the raw gravity readings must undergo a series of corrections. Initially, the meter readings are multiplied by a calibration constant to convert them to mgals. The next correction is an instrumental one: an adjustment of the survey for 'drift', the time variant mechanical response of the gravity meter. This correction arises because the mechanical systems of the meter are susceptible to thermal and vibrational influences, which to a large degree are impossible to avoid during a survey. The magnitude of this correction is determined by either closing the survey line or regularly returning with the meter to the site's base station.

The values must then be reduced with respect to the altitude of the measurement station to a common reference plane, since the Earth has a vertical gradient of gravity of 0.31 mgals/m. This is termed the 'free-air correction'. Elevation effects can easily mask small anomalies, which may have amplitudes equal to the vertical gradient over 1 m.

Next, the values have to be corrected for the mass of the material between the altitude of the measurement station and that of the common plane, since this surplus or deficiency of mass will account for some of the variation in acceleration measured by the instrument. This is the Bouguer correction and its value is approximated by the gravitational attraction of an infinite slab with a thickness, 'h', equal to the above mentioned elevation difference and with a density contrast, $\Delta\rho$, with respect to air. This produces a correction, 'c':

$$c = \pm 2\pi\Delta\rho \text{ Gh gals.} \qquad (4)$$

Two further corrections are usually made in large scale surveys for changes in latitude and the presence of nearby topographic features. The first of these is made because the gravity meter's upward angular acceleration, which is caused by the Earth's rotation, will tend to oppose the downward pull of gravity and will vary with latitude. The value of this correction will increase as one nears the equator. Secondly, the close proximity of major terrain features, either hills or valleys, will effectively decrease the gravity reading and therefore a correction, which is always positive, must be made for these. (An adjacent hill will exert an upward attraction on the instrument, while a valley—a mass deficiency—will decrease the downward attraction; both will tend to lower the observed gravity measurement.)

The selection of the appropriate interval between measurement stations is important. The waveform of the gravity anomaly produced by a buried feature is a function of the contrast in density of the anomalous mass with the surrounding material and its geometry: shape, size and depth. The features at the site were known to be shallow and of short duration (vertical contacts of sandstone with the backfill) and therefore fairly steeply sloping, short wavelength anomalies would have to be resolved. An interval of between 2 m and 4 m between measurement stations was accordingly chosen and used on the traverses.

At the site, the arrangement of fences and buildings made an evenly spaced grid survey impossible. Instead, a series of six traverses was made, with regular returns to a base station to check for instrumental drift. The directions of the traverse lines were chosen to cross at right angles the edges of the quarries as indicated by the archaeological work. Over 180 readings were taken at stations along the traverses. Following the gravity measurements, the elevations of the stations were levelled.

Results

The resulting Bouguer gravity map for the site is shown in Fig. 6. The latitude and terrain corrections were not applied and therefore it is necessary to estimate briefly the effect that the absence of these two corrections may have prior to interpreting the map. Because of the changing latitude, the observed acceleration of gravity will increase northward, approximately along Millers Road. The horizontal gradient of the acceleration for this latitude is approximately 0.8×10^{-3} mgals/m. Therefore, for points at either end of the site's north/south length, this effect will contribute about 0.20 mgals to the observed difference. The effect is planar over the whole site and, since the investigation's primary concern was the location of features with short wavelength anomalies, this planar effect would not conceal them. The terrain correction is only applied when the topography in the vicinity of a measurement station is significantly non-planar. The site was, however, sufficiently planar to exclude this correction as well.

Bouguer gravity maps are usually reduced to sea level, in order to study crustal features. In this instance, however, the required reference plane had to lie above the level of the structures being sought and, therefore, it had to be at the same altitude as the highest point encountered on the survey. This point occurred at the southern end of the site, as the ground surface slopes down northwards. This meant that, when the gravity readings were reduced to the reference plane, there was a relative mass deficiency in the north arising from the wedge of air created between the reference plane and the ground level (Fig. 4).

This raised difficulties in studying only buried features at the site, since the 'air wedge mass deficiency' was being included in the northern readings but not in the southern ones. A negative Bouguer correction was therefore made by adding an approximation for this missing mass, which was assumed to have a density equal to that of the backfill material. The approximation for each station was made with the infinite slab formula described earlier (Equation 4).

As can be seen in Figs. 5 and 6, the resulting gravity map is in general agreement with the archaeological work and shows three main features: southern and northern 'low' areas and a gentle rise in the gravity values along Millers Road towards the northern part of the site. The position of the quarry faces should approximately coincide with the inflection points of the anomalies' slopes and it was on this assumption that the position of the quarry faces were located. The steepness of the gravity contours should also reflect the steepness of the faces, assuming the thickness of the overburden to be approximately constant. The southern basin is bordered to the south and east by low gravity gradients and on the west by a steep gradient. Its northern limit, which probably consists of a rock ridge separating it from the deeper northern basin, is not well defined. It appears then that the southern face at least is a sloping one. This interpretation is in agreement with the findings of a subsequent trial pit, which uncovered the top of a 70° sloping face.

The southern wall of the northern basin produced the site's largest anomaly, probably because of its greater height. The location of this face was indicated by an outcrop in the car park and it was possible to trace the extension across the rest of the car park and the neighbouring road. An accuracy of ± 1 m was confirmed by later trial pits. The western face appears to terminate at the adjacent building or just beneath the edges of it. To the east, the anomaly was not well defined and the face again appears to be sloping.

Proceeding northward, the lowest point of the gravity values is reached about 100 m from the road junction, after which there is a rise at the rate of about 3×10^{-3} mgal/m. This gradient is nearly four times that calculated for the increase in latitude and so it appears that there is an increase in mass beneath this section. The reason for this increase cannot be deduced from the gravity alone; it may have been caused by either the level of the top of the sandstone rising, an increase in the density of the fill material, or a combination of the two. Boreholes have subsequently indicated that the sandstone floor of the quarry rises at the northern end of the site and also that the penetration resistance of the fill is higher in the north than in the south, indicating an increase in density of the fill.

It was desirable to obtain from the gravity data some idea of the depth of the quarries. However, any depth figures based on the geophysical work alone must be treated as a very rough approximation and cannot be considered as approaching the value or accuracy of borehole information.

A very simple approach was made to these calculations. From the infinite slab formula (Equation 4), one can calculate the thickness of a slab, with a density contrast estimated from the site's geology, which would produce an anomaly equal to that observed. This thickness is an approximation of the quarry's depth. Anomaly amplitudes were determined from the western edge of the southern quarry and the southern edge of the northern quarry. Using the density contrast of 0.6 gm/cm^3 between the sandstone and the fill, a depth for the southern basin of 8 m, ± 1.5 m, was calculated. Subsequent drilling met sandstone at a depth of 7 m near this edge. Using the same density contrast for the northern quarry, the depth of fill beneath the car park was calculated to be 16 m, ± 2.5 m. A borehole was not placed in this area, but further to the east, at a ground level of about 2 m beneath that of the car park, a borehole proved a depth of fill of 13 m. When the 2 m difference in altitude is added, an approximate total depth of 15 m is obtained for the fill under the car park.

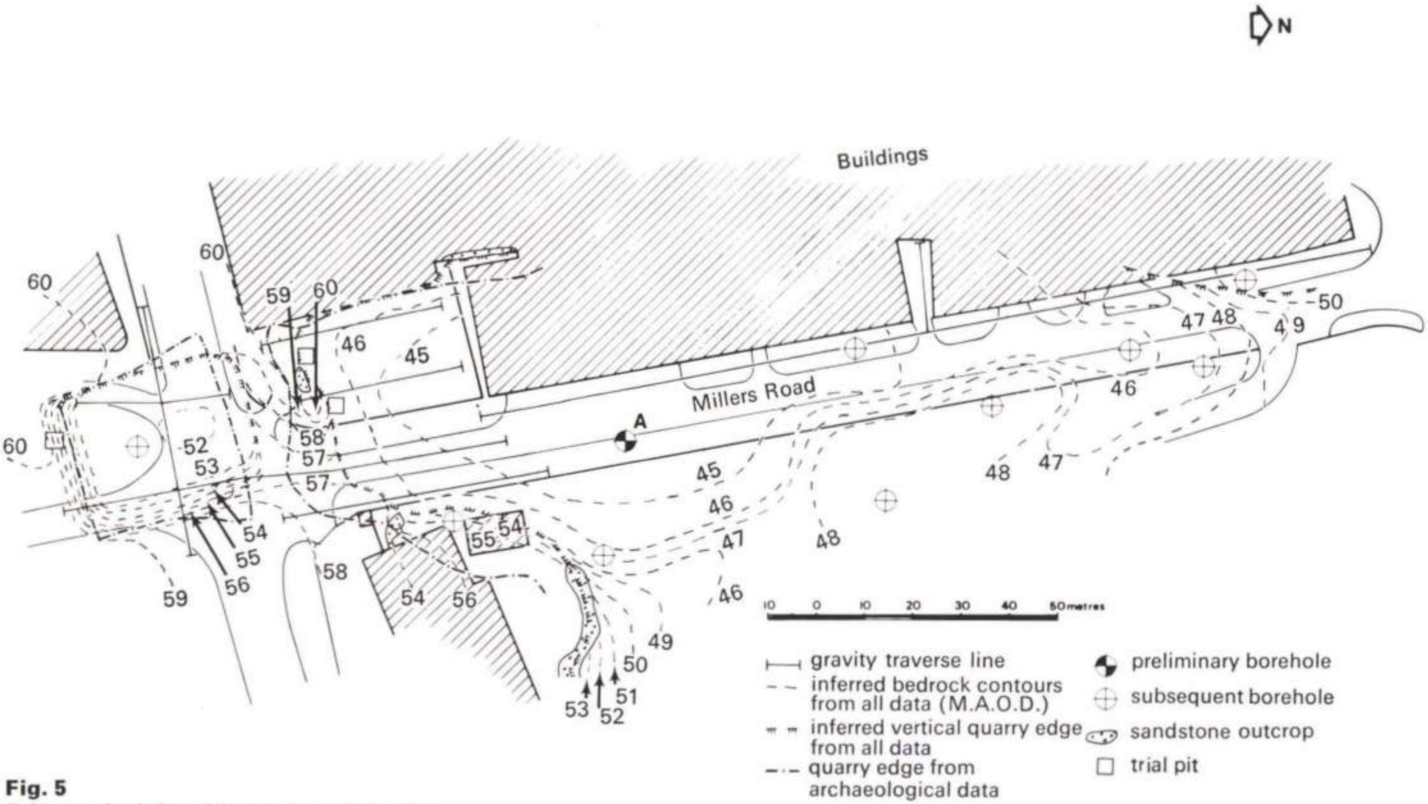


Fig. 5
Evidence for inferred bedrock configuration

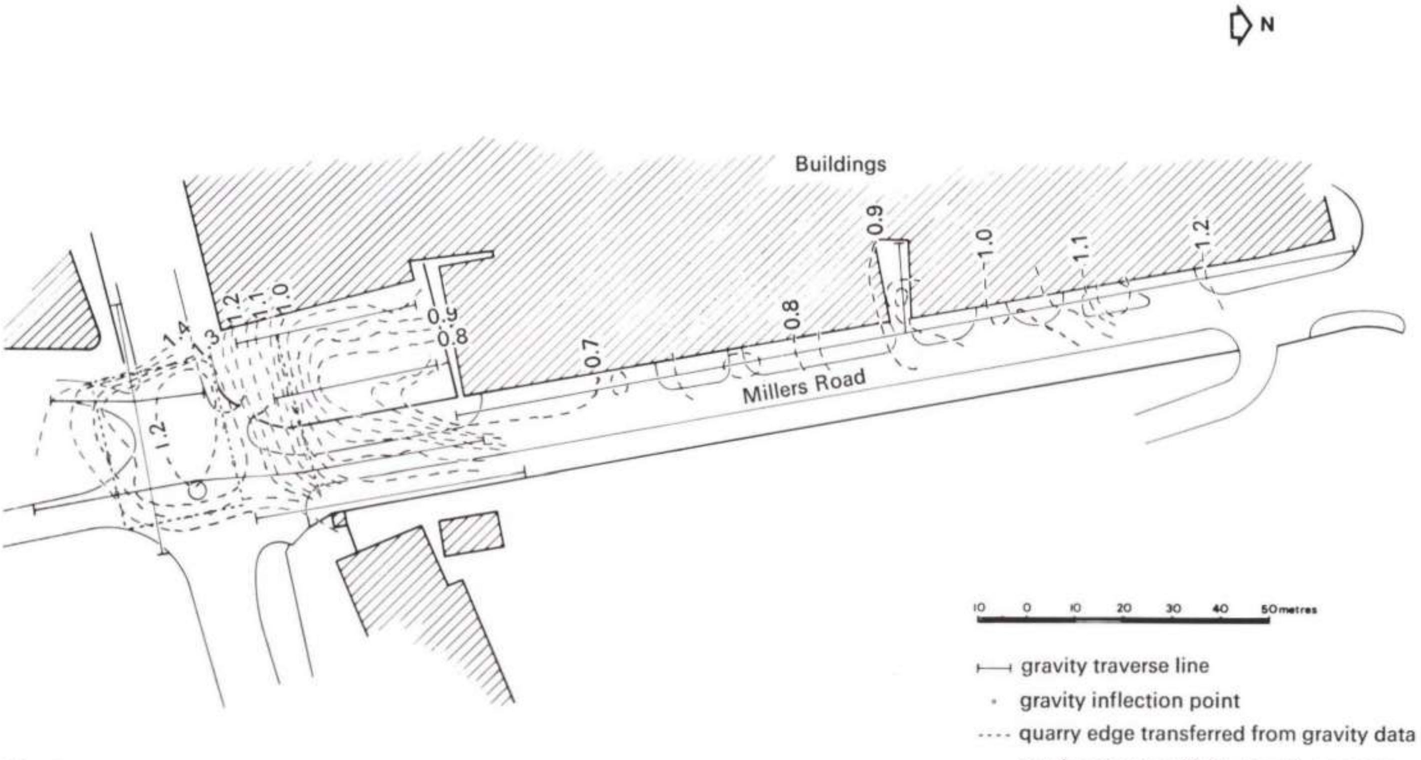


Fig. 6
Bouguer anomaly map

Conclusions

High resolution gravimetric surveying was successfully used to confirm the position of buried quarries outlined by archaeological research. The method may also be used for site reconnaissance when little previous information is available, but the interpretation of the gravity data will become more ambiguous. In both instances, the gravity maps can prove useful in defining targets for drilling and later for interpolating between boreholes. They can augment, but not replace, conventional site investigations.

Before a gravity survey is considered, the conditions at a site must be checked to ascertain whether anomalies above the detection limit of the method are likely to be present.

The survey fieldwork described above was carried out by a geophysicist and a geologist in three nights and the total cost of the exercise, of which a major part was the rental of the gravity meter, was approximately equivalent to that of the preliminary borehole.

Acknowledgements

We wish to acknowledge the co-operation of the Geotechnics and Civil Engineering Divisions, particularly the archaeological work of Richard Hughes and the editorial assistance of Felicity Hood.

References

(1) SUMNER, J. R. *and* BURNETT, J. A. Use of precision gravity survey to determine bedrock. *ASCE Proceedings: Journal of the Geotechnical Engineering Division, 100* (GT1) pp. 53-60, 1974.

(2) COLLEY, G. C. Detection of caves by gravity measurements, *Geophysical Prospecting, 11* (1), pp. 1-9, 1963.

(3) GRIFFITHS, D. H. *and* KING, R. F. Applied geophysics for engineers and geologists, Pergamon Press, 1965.

Transportation Surveys in the Design Process

Malcolm Simpson
Mike Sargent
David Johnston

Introduction

The layman's contact with transportation surveys rarely extends beyond an occasional glimpse of a bedraggled figure, his fingers twitching on a tally counter as he stares gloomily at passing pedestrians or vehicles. The passer-by, after feeling a brief interest in this unfortunate creature and his activities, hurries on without understanding what the poor wretch is recording and the use to which his endeavours will be put.

The intention of this article is to describe why surveys are required, the problems encountered in undertaking and analyzing surveys and the role these surveys play in design and investment decisions.

The necessity for surveys

Transportation surveys, which are studies of existing movements of persons or goods, are undertaken for a variety of purposes. The existing situation may be studied to see how it can be ameliorated by changing external conditions; or to ascertain the effect of superimposing further movements on the system. An example of the first case would be the examination of a road junction to see if it could be improved by altering its layout and of the second, the assessment of the future adequacy of the junction if a major traffic generating development were to be built nearby.

Both these exercises assume that there has been sufficient data collection and analysis under various combinations of external conditions for formulae to have been established by which the effects of altering the junction layout or of increasing the flow through it can be assessed. In the field of transportation, such standards, statistics and formulae are available in Europe and the United States for many aspects of vehicular movement and for some aspects of pedestrian activity. However in many countries in which the Group is now operating, indigenous standards and statistics do not exist and it is not acceptable to use European or United States values and experience. Differences also exist within the UK; when examining escalator capacities, it was found that to use London standards in Birmingham would overestimate capacity by some 50%. Even if some allowance could be made for more obvious differences in characteristics influencing behaviour, such as income levels, adjustments for social attitudes and organization, culture and history would not be possible without in situ surveys.

Universities in the Middle East

Over the last two years the Group has been effecting transportation studies for universities in Saudi Arabia, Libya and Doha in the Persian Gulf. The differences between these countries and the UK are obvious but the differences amongst them which affect movement patterns are also very considerable.

Apart from the obvious climatic differences, examples are: the situation of the campus relative to the existing built up area, the status of women students, the existence of public transport and the proportion of staff and students who will be resident on the campus. These factors are summarized in the table below:

Factors	**Riyadh**	**Tripoli**	**Doha**
Campus situation	Virgin desert site 12 km from city	Expansion of existing university on edge of city	Virgin desert site 6 km from city
Women students	Not admitted	20% of students	33% of students**
Public transport	Service taxis only	Town bus service*	Non existent
Percentage of population resident on campus	not decided but 50–75% approx.	20%	66%

* Women students do not travel in public buses
**Women students do not drive cars.

Other social characteristics, such as the attitude towards car sharing, which in Doha is confined to members of the same family, are also very significant when assessing the future demand for transportation facilities. The organization of the University will affect the demand and its distribution; the adoption of the American style credit system as proposed in Doha, can substantially reduce the peak traffic generation.

The studies of these universities are of value in making initial assessments for further schemes in the Arab World but it will be necessary to have studied more universities before firmer proposals can be made without recourse to on-ground surveys. The greatest advantage gained for the Doha study from the earlier studies at Riyadh and Tripoli has been in the organization and implementation of the surveys.

External considerations affect the decisions on the type and the depth of the surveys to be carried out. Amongst these considerations are: programme, the degree of detail to which other disciplines within the design team are working and the quality of planning data. In Tripoli, where the University is being dramatically expanded, the rate of change is so rapid that plans for the City transportation network have not been drawn and yet the University must be integrated into the system. It was therefore decided that studies stretching outside the Campus to assess in detail the effect of the University generated traffic on the City would be of little value.

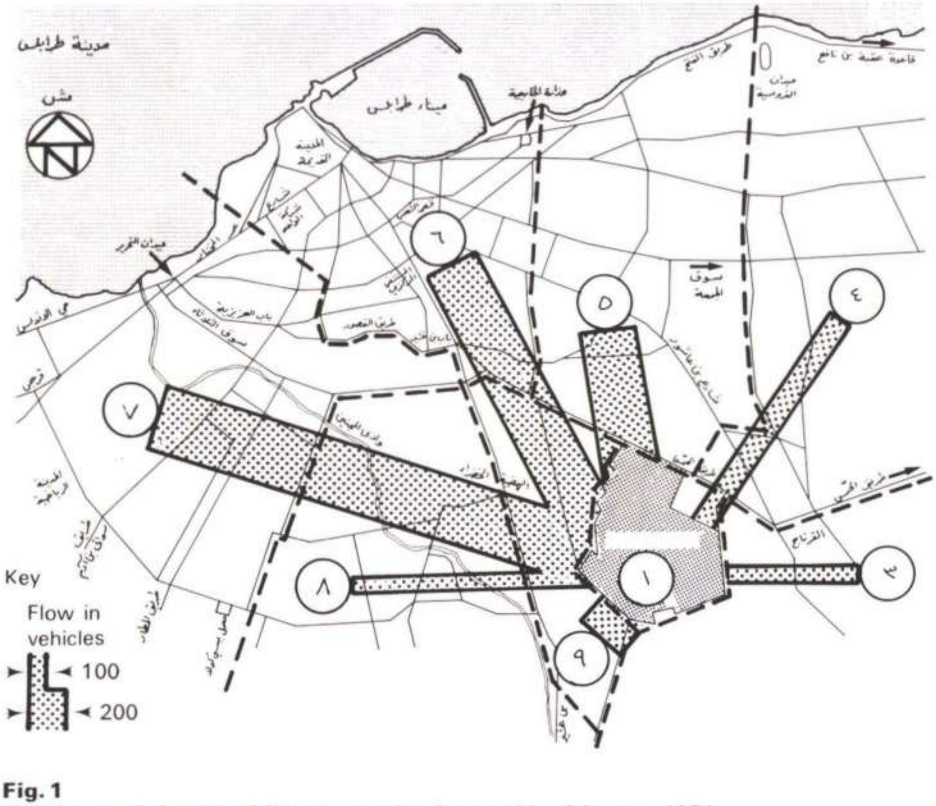


Fig. 1
Zoned map of the city of Tripoli; car trips from residential zones 1974

Accordingly an *a priori* decision was taken on the location of the connection points between the University and the new City network and surveys were designed to give a directional approach split at these junctions for their detailed design and to assess internal movements within the Campus. Another problem in carrying out origin and destination surveys had been identified previously in Riyadh where it was found that many places in the City did not have commonly accepted addresses which made subsequent zonal identification very problematic. This was overcome in Tripoli by attaching a zoned map of the City to the questionnaire form. Even this map had to be produced by updating available maps which did not take account of the large amount of new development and road construction. (Fig. 1).

If time permits it is always preferable to undertake a pilot study to test the organization, method and validity of results. Questionnaire surveys should where possible, have some results which can be checked by direct measurement. The Tripoli questionnaire included car occupancy and time of arrival questions which were checked by visual observations at the entrances to the University. The pilot survey in this case proved successful and a full-scale one was launched a few weeks later.

The questionnaire and observation count results are compared in Fig. 2 which shows satisfactorily correlation between the two for the arrival and departure patterns. The difference in University hours between the UK, where staff and student arrival and departure is spread, and Tripoli, where it is very concentrated, is very important. In Tripoli, and to an even greater extent in Riyadh, the generated traffic flows are extremely peaked, creating a very high demand for road space which would be unused throughout the major part of the day. The results of the questionnaire survey on residential location and desire lines of car drivers are shown in Fig. 1.

The objects of these surveys were to establish the behaviour pattern of the existing University and to project them for the extended University design. They would then serve as a base for testing design policies. In Riyadh, for example, the surveys showed such a high car generation and time concentration of use that it was decided to reduce the effect by putting more students' residential accommodation within the campus. This was accepted by the University and represented a fundamental change in the Master Plan.

Application to the design of buildings

The process of using surveys within the more detailed design process can be demonstrated by the work carried out for the now abandoned Channel Tunnel Rail Passenger Terminal at White City.

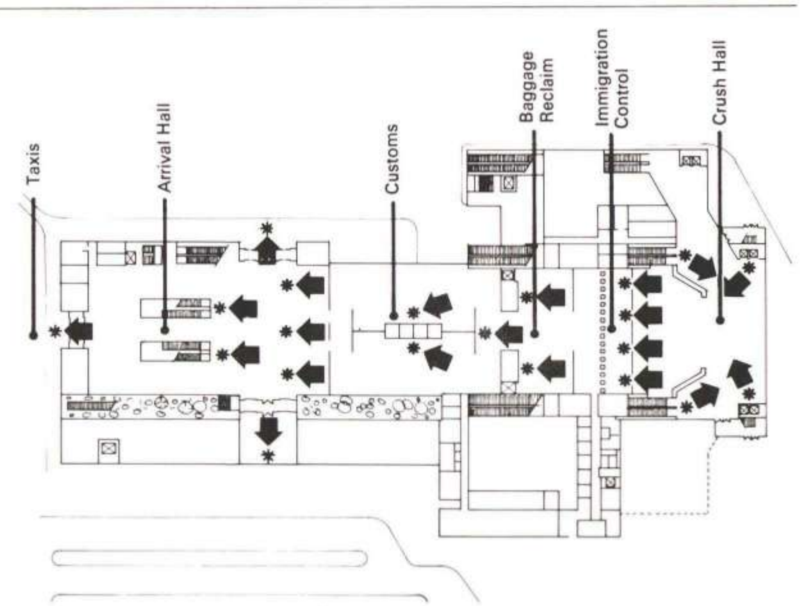


Fig. 3
Channel Tunnel Rail Passenger Terminal, White City

The main pedestrian and vehicular movement paths of Terminal users were evolved in an initial architectural planning scheme and the points of interruption to pedestrian flow within the Terminal were identified (see Fig. 3). The next objective in the design process was to size the pedestrian activity halls, the pedestrian vertical movement facilities and the vehicular movement network and waiting areas. In order to carry out this exercise, it was necessary to determine the number of people present in the halls and pedestrian network and the number of vehicles using the roads and waiting areas and then to assign space standards to the various pedestrian and vehicular activities.

The level of service concept was used for determining the acceptable space standards. For 90% of the time people were to be provided with a comfortable standard of environment and a check was carried out for peak hour activity to ensure that conditions were not unacceptably congested. These standards are reasonably well documented and their values are not discussed here. However some survey work on the proportions of passengers carrying luggage, using personal trolleys and common trolleys was necessary to establish space standards for each area of pedestrian activity.

Very little information was available on many aspects of the likely movement characteristics of Terminal users. To overcome this, a number of surveys were undertaken. No parallel situation could be found to the White City Terminal either in terms of function, location or clientele. It was necessary to study the nearest parallel situations available and to adjust the results by reference to other published information and weighting of the survey results.

The surveys were a mixture of questionnaire and observation. The list of surveys and the results obtained from them are given below.

Boat train passengers

Questionnaire

1 Mode of travel to terminal
2 Number of people seeing off or greeting passengers
3 Occupancy of cars and taxis
4 Time of arrival at station before train departure

Observation

1 Rates of ticket inspection
2 Time of arrival at station before train departure (check on questionnaire)
3 Time to vacate train and platform
4 Influence of luggage

Euston Station surveys

1 Time of arrival at station before train departure
2 Occupancy of cars and taxis
3 Taxi and car pick up/set down times

Various rail and air termini

1 Influence of luggage
2 Taxi handling methods and throughput

New Street Station, Birmingham—

1 Escalator capacity

The Boat Train and Birmingham passengers were predominantly on holiday while those at Euston were on business. In the use of the survey results or other research findings the characteristics of businessmen and holidaymakers were weighted according to the proportions expected at White City.

The uses to which the survey information was put in the design of facilities for departing passengers is shown in Fig. 4.

The method of calculating queue lengths on the platform and in the Crush Hall is shown in Fig. 5. The survey results provided the information regarding platform vacation and escalator capacity. This area proved to be the most critical part of the design as the area requirements were very susceptible to train interval, capacity and occupancy, platform scheduling, nationality split and the number of immigration desks and escalators. The entire departure and arrival sequences were computerized so that area requirements could be rapidly calculated for different combinations of the aforementioned variables. The ability to provide a rapid response to changes in train timetable became particularly valuable when British Rail were assessing the line operating benefits of running groups of high speed trains at small headways against the additional terminal construction and running costs involved.

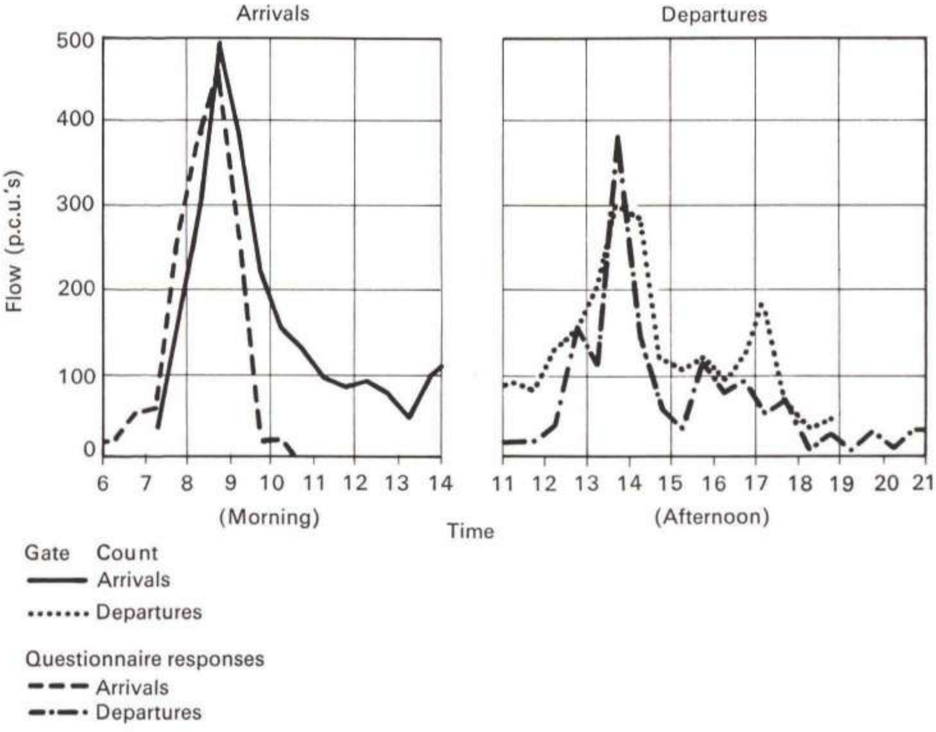


Fig. 2
Comparison between Gate Counts and Questionnaire results 1974

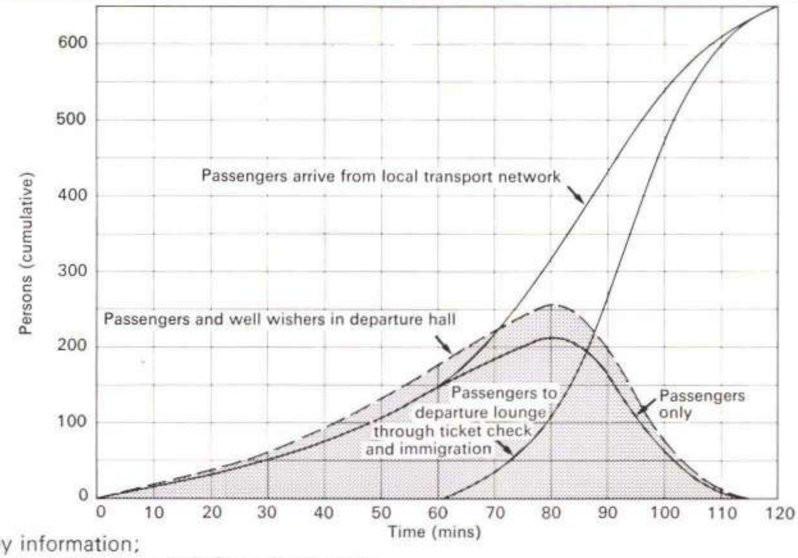
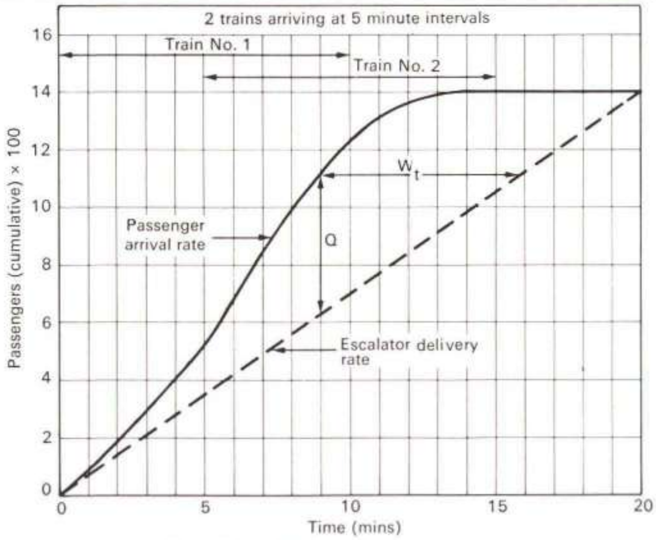


Fig. 4
Uses of survey information;
Movement through departure hall for a single train



Q = Queue W_t = Waiting time for escalator

Arrivals – platform escalator

Fig. 5
Method of calculating queue lengths

Economic assessment

Transportation surveys are now regularly used in assessing the economics of schemes which range from single buildings to local and regional development proposals, and to compare alternative designs. The decision whether to implement developments should, of course, take into account social effects and the transportation benefits form only a portion of the overall benefits accruing.

The economic benefits provided by a road itself, rather than the development benefits it allows to be realized, are measured by decreased vehicle running costs and personal time costs. These benefits, when compared to the capital costs of providing the road, allow an economic cost benefit calculation to be made. To compute these benefits the traffic patterns and running costs under different conditions must be established. This information is often unavailable and ground studies are therefore required. This was the case in the recently completed feasibility study for the Jerangau-Jabor road on the eastern side of Peninsular Malaysia (see Fig. 6).

The objective of the road was to open up the inland forest areas for agricultural development and exploitation of the rubber and timber resources. These developments will require a considerable work force which would be accommodated in townships sited near the major route or its feeder roads. The projected population of the eight major towns is 55,000 in 1980 rising to 130,000 in 1995. The road will also provide long distance traffic with an attractive alternative to the existing coastal route which is subject to annual flooding.

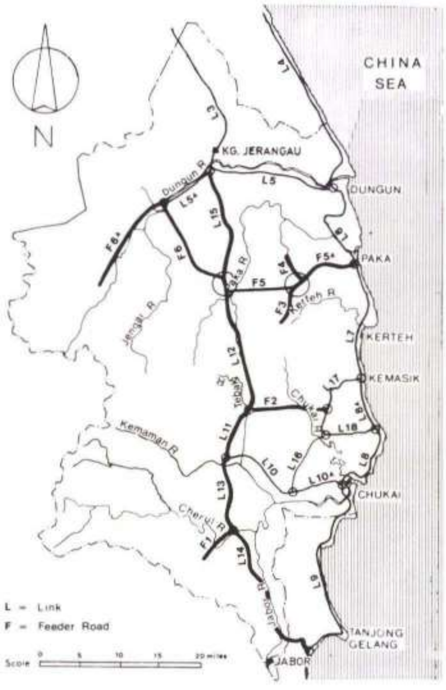


Fig. 6
Jerangau-Jabor Road network

The brief which developed as the study proceeded was to examine alternative routes for the major road and feeder roads, to compare these with a road network based on the existing coastal road with new inland feeder roads to the development areas and to produce standards for the horizontal and vertical alignment of the highways which would optimize their economic benefit.

Traditional traffic counts were carried out to establish average daily volumes and these were compared with past records to obtain estimates of traffic growth. Simultaneously origin and destination surveys were undertaken. In addition to obtaining details of movement patterns of persons, information was collected from these surveys concerning the movement of all produce from its production area to processing plant to consumption area or export point. Of particular importance was the collection and analysis of the lorry weights, loads and axle configurations, so that the highway pavements could be designed.

Journey times were measured to establish running speeds on roads of different geometric standards. These surveys provided the base data for simple mathematical models to determine the future traffic patterns in the study area for different new road networks. The future traffic was divided into three categories.

Firstly the 'Diverted Traffic' which is the existing traffic adjusted for growth.

Secondly the 'Development Traffic'; this is the new heavy lorry traffic which is generated by the new production areas and moves to consumption area or export point via processing plant.

Thirdly the 'Residential Traffic' which is the traffic generated by the new towns. The model for these movements was derived from the origin and destination surveys of two towns of 3,000 and 25,000 population.

The combination of origin and destination matrices and running speeds enabled traffic assignments to be made onto the various networks to be tested by assuming drivers take minimum time routes. In order to take into account drivers' differing opinions to travel time, a multiple routing assignment was used. Applying running costs to 'Development Traffic' and 'Diverted Traffic' assignments, the benefits of the alternative routes were compared and each was measured against the proposal to serve the new developments by feeder roads from the existing coast road. The combined assignments of 'Diverted', 'Development' and 'Residential' traffic were used to establish the road widths and pavement designs and thus enable the construction and maintenance costs to be calculated. The vehicle operating costs and time costs were based on data prepared during another study for the Malaysian Government.

Surveys were also carried out to assess the effect of horizontal and vertical alignment on vehicle speed and thus running cost. The speeds of different types of vehicle were recorded.

Very little work has been done on this subject anywhere in the world and we have developed a technique to estimate costs. We hope that this will be used by the TRRL in a road optimization programme that they are developing. The principal result that has emerged so far is that operating costs for very different standards of roads alignments are virtually constant.

Increased speeds, and hence cost, in straight and level routes, appear to balance the additional costs due to very low design speed alignments. From the operating cost point of view it therefore appears that we do not need motorway standard alignments! This result could fundamentally affect the standard of road being built, particularly by the Asian Development Bank where most occupants' time savings are not included in a cost benefit analysis.

Chester Bell Tower

Roy Cowap

Introduction

This is the first free-standing bell tower designed for full change ringing to be built in England since the Middle Ages. The tower stands 24.4 m high to the apex of the roof, in the south east corner of the existing cathedral grounds over an old burial ground adjacent to the Roman City wall.

Design problem

This job arrived in the London office early in 1968 from the architects George G. Pace of York, with a fairly predictable brief for a rock bottom structural cost. The money was to be provided from a Restoration Fund to be launched later in the same year. The main design problem was stated in Mr. Pace's original letter thus:

'As you will know, in England, when a peal of bells, in this instance 13, are sounded by what is known as "change ringing"—in this each bell is "rung up" before the "change ringing" starts—this means that all the bells are upside down with their mouths facing the sky and each pull of the rope turns the bell completely round so that it comes up again with its mouth towards the sky. A considerable force, of course, is exerted by this process. You will note from the plan the way in which the bells are placed in the bell frame so that they ring against each other and thus to some extent neutralize the forces.'

A quick look in *Encyclopaedia Britannica* will show that for 12 bells, there are 479.001.600 different possible 'changes' or sequences in which the bells can be rung. Fortunately it was not necessary to carry out a dynamic analysis for each of these in order to ascertain the maximum response as there are certain festive occasions when the bells are 'fired', that is swung in unison, and this clearly produces the maximum lateral thrust on the structure.

The architect originally envisaged a steel-framed structure and the bell founders provided us with an equivalent static lateral thrust which we used in our initial check on the sway of what we considered a reasonable structure.

Calculation

Those were the days when the computer analysis of a plane frame with 26 joints required you to complete four A4 pages of input forms, in return for which we were told that the ringing floor would move 0.0061787 ft. horizontally. Half a page of hand calculation, allowing 25% for the contribution of the lattice members to the deflection from the flanges, produced a figure of 0.0054.

This was close enough as a check but too much for the comfort of the bell ringers, so the steel structure was abandoned for a reinforced concrete frame with stiffening panels of brickwork. This provided a tower of the required stiffness which also gave the necessary planning flexibility. There was no further activity on the job until the middle of 1971, at which time it was passed to Wales and West in Cardiff. After several false starts the job finally went out to tender in mid-1972 and, following negotiations on cost, A. Monk & Co. were appointed main contractors with Frankipile Ltd. as their direct sub-contractor for the piled foundations. Work finally started on site at the beginning of March 1973.

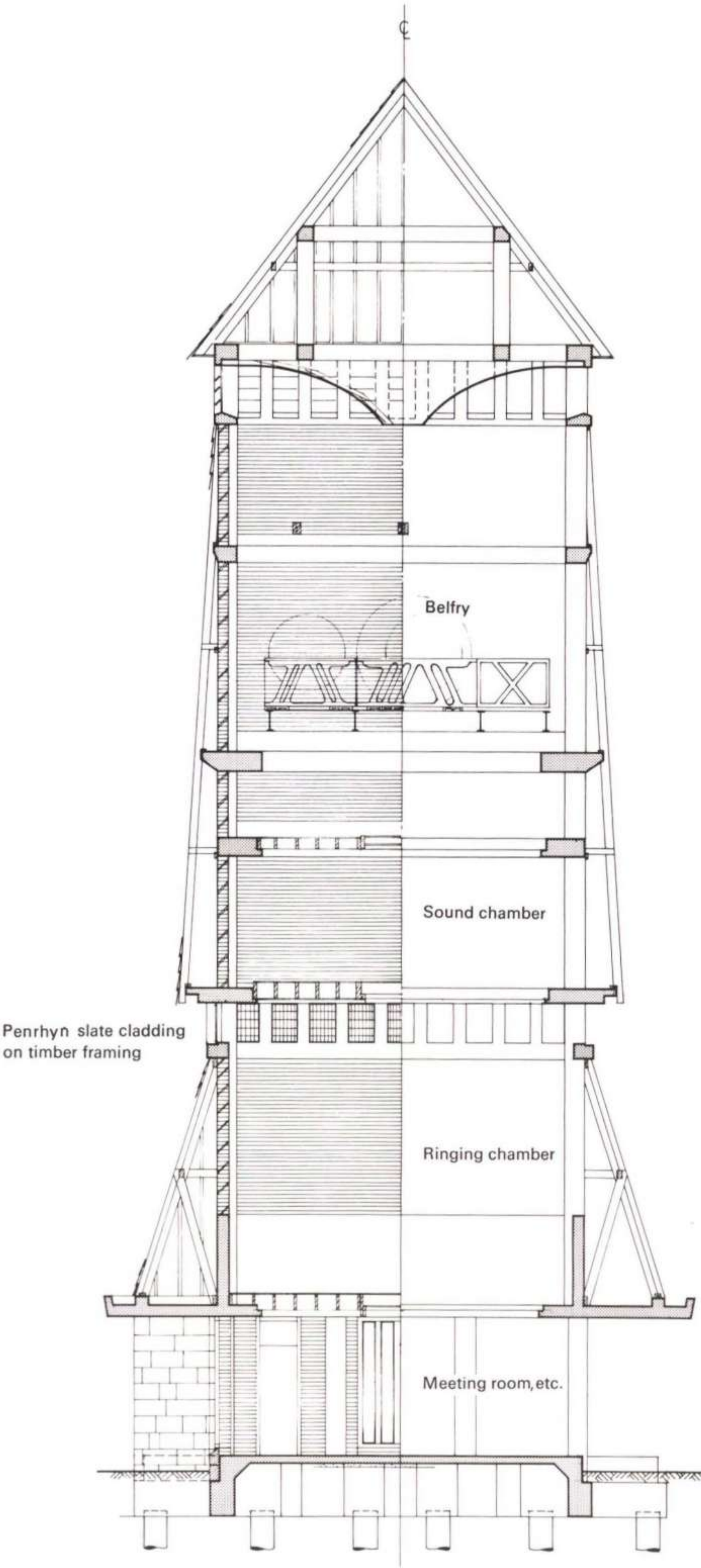


Fig. 1
Section through Chester Bell Tower

Fig. 2
Exterior of bell tower
(Photo: Harry Sowden)

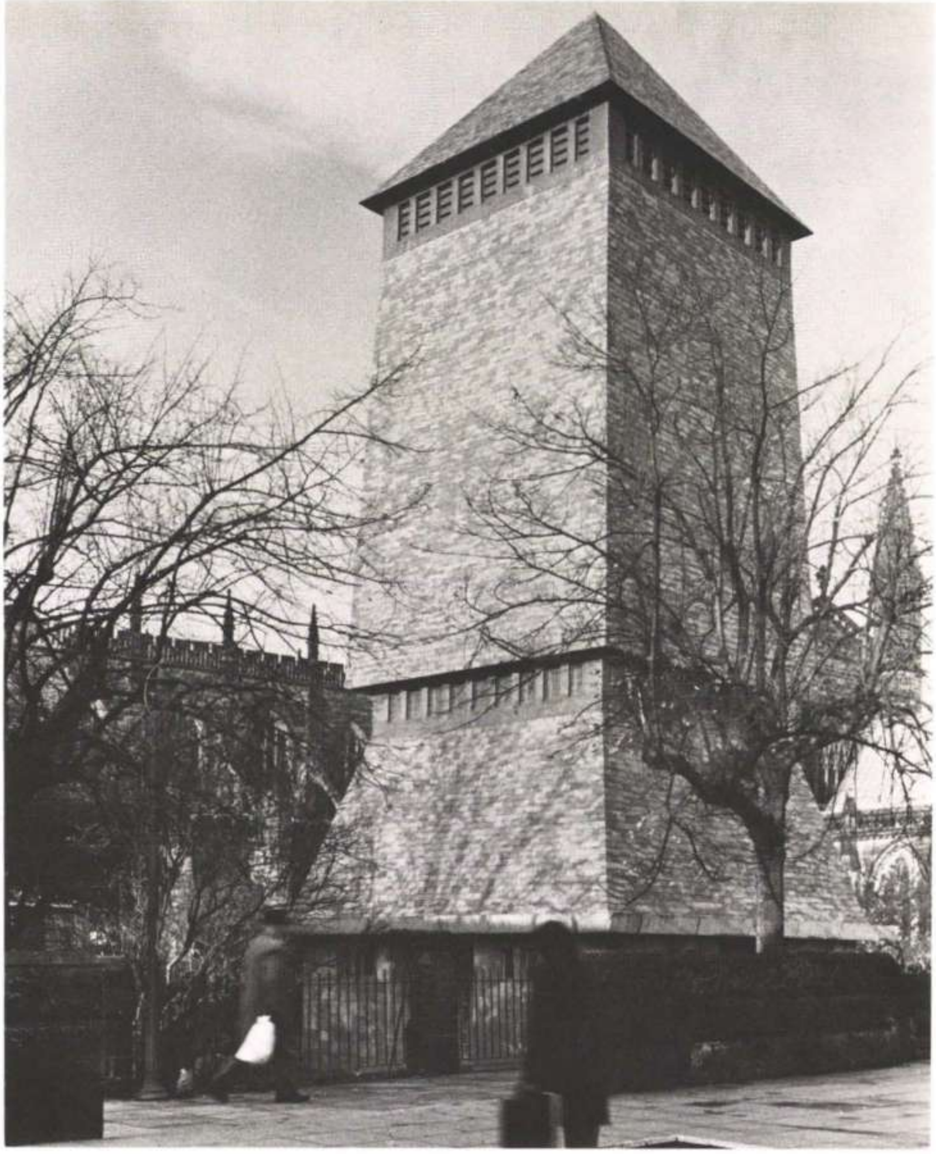


Fig. 3
Bell ropes
in the ringing chamber
(Photo: Harry Sowden)

The Chester Bell Tower site

The site itself was very restricted and access to it was difficult. This precluded the use of any large items of plants such as tower cranes, piling rigs and concrete weigh batch equipment on site. Access for Readymix trucks onto the site was impossible. Also, the standing time required in the narrow road at the back of the site for discharge of concrete could have seriously affected unloading of goods to the adjacent shops.

Due to the small pre-contract budget, no site investigation had been possible. A piled solution had been chosen largely because of founding over an old burial ground and a recent archaeological dig on the site. Also from reference to the geological maps of the area and foundation information received from adjacent sites, we knew that the sandstone bedrock was fairly near the surface. The first pile, therefore, acted as both our site investigation and agreement for the pile design.

Bored piles were used, as only a small tripod rig would be needed. The piles were 480 mm in diameter, of 30 tonnes capacity and approximately 6.7 m long, end-bearing onto the sandstone. The superstructure was an in situ reinforced concrete frame, stiffened against the ringing forces from the bells by infilling with 230 mm loadbearing brickwork. All the concrete was volume batched and placed by a hoist and wheelbarrows. The walls were pinned tight within the frame by using *Abbey* anchor slots cast into the columns and brick ties inserted into these and bonded into the brickwork. Slate dry pack was used between the top of the brickwork and reinforced concrete beams at each level. This made a very stiff construction with a high natural frequency, much higher than the ringing frequency of the bells, so that resonance could not occur. The tower was clad externally with Penrhyn slate nailed to a timber framework.

The 13 bells were re-cast from the existing bells in the original cathedral bell tower, which had not been used since 1963, and weighed a total of 7 tonnes. The bell founders were John Taylor & Co. of Loughborough. The tower was substantially finished early in 1974 but the bells were not hung until the end of the year. The official opening is on 25 June 1975 and it will be performed by the Duke of Gloucester.

Credits:
Architect: George G Pace
Main Contractor: A Monk

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