THE ARUP JOURNAL

JUNE 1972

THE ARUP JOURNAL

Vol. 7 No. 2 June 1972
Published by
Ove Arup Partnership
13 Fitzroy Street, London, W1P 6BQ

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

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Front and back covers: The Horizon project

(Photos: Richard Einzig, Brecht-Einzig Ltd. Courtesy of *Architectural Review*)

Structural analysis and recording of ancient buildings

Poul Beckmann

This article originally formed the basis of a lecture at a course, Conservation: Analysis Recording and Documentation Techniques in Existing Buildings, *University of York, 27 September – 1 October, 1971.*

Introduction

Structural analysis and recording are aids to assessing the health of a building as it stands, and the possible effects of changes in its environment.

The two operations must go hand in hand as without records no meaningful analysis can be made, and, without analysis, records do not have any meaning.

For our purposes, structural records include any information describing the structure of the building in the three dimensions of space and the fourth of time. This therefore includes the state of the structure as it exists.

Structural analysis will include all methods by which the available data can be processed to produce an assessment of strength, stability and deformations, past, present or future.

If an ancient building appears in good repair and a visual inspection gives no cause for concern, one would only resort to structural analysis in order to assess the effects of a proposed change of environment such as may be caused by major construction or demolition works in the vicinity.

When an ancient building is visually in a poor state of preservation, the structural analysis can help in assessing

(a) the present structural safety, including the effects of any observed defects on this
(b) the possible cause of the observed defects
(c) the remedial works necessary, if any.

The record of the structure as found

The first requirement is a complete description of the structure as it stands 'warts and all'. The most complete record is, of course, the building itself, but this is in most cases too big and too complicated to provide a picture which can be grasped as an entity. This is most important as it is in most cases impossible to arrive at a sound assessment of the causes and effects of defects observed in isolation. What is needed before a safe diagnosis can be made, is a complete pattern of symptoms. Hence drawings and models are essential. The drawings should be sections and elevations of small enough scale that the whole of the building can be shown on one sheet. On these should be superimposed all significant defects: cracks on two sides of a wall should be plotted on the same elevation with two different types of line so that one can distinguish between surface cracks on one face only and cracks showing on both faces, which are likely to go right through the wall.

In some instances it is necessary to have a three-dimensional picture of the structure with its cracks, and here a fairly crude *Perspex* model may be of assistance as the cracks can be drawn on the individual components of the model before assembly, and subsequently be assessed three-dimensionally in the completed model.

Whereas the graphical presentation of cracks is thus fairly straightforward, deformations are more difficult to depict in a clearly comprehensible manner. Relative levels can be shown under an elevation as a line diagram with exaggerated vertical scale. Out-of-plumbness can be shown with arrows indicating the direction with the magnitude at the level of the arrow superimposed in figures, and where the deformation of a particular line is of special interest, this can be plotted to exaggerated horizontal scale away from the main elevation.

Cracks are, on the face of it, easily observed, but it is important to ascertain, as far as possible, what the structural gap is, as this is often masked by past repairs so that what now appears as a 5 mm crack may in effect signify a structural gap of 25 mm concealed by pointing and/or stone replacement.

Measurement of past movements

As regards deformations which have taken place between the original date of construction and now, one can obviously only achieve a fairly low degree of precision in the measurements due to the inevitable inaccuracies in the original construction. For this reason, fairly crude methods of measurements suffice: an ordinary levelling instrument will do but it should have the best possible telescope to assist reading in poor light. As one may have to take levels in triforium galleries and other hard-to-get-at places, the instrument should be as light and as compact as possible. When taking and recording levels it should be remembered that any one section of the building which was built as an entity in one period would have been finished to within ±10–15 mm on horizontal features such as string courses, but anything built 50 years before or after may have been effected by intervening differential settlement. For plumbing, simple plumb lines or theodolite-plumbing will be chosen on the basis of which is the easier to do, remembering that plumb lines are not easily fixed to great heights, and they require a calm day to be of any use externally.

Where there is more than one storey, and particularly where the work above a floor is of a different vintage from that below, fairly accurate (±25 mm) correlation of pier shapes are however necessary to ascertain possible eccentricities.

Having thus obtained our 'description' of the structure in three dimensions, at this particular point in time one can perform structural analyses to various degrees of refinement, about which more later.

Measurement of ongoing movements

It will, at times, be desirable to know whether cracks or deformations are still 'live', or, having resulted from some cause in the past, are now static and therefore require nothing but 'cosmetic' treatment. Likewise, one may have carried out structural repairs which are compromises for economic and other reasons and would like to know whether they have been effective in arresting movements. In this instance one is dealing with slow movement (250 mm settlement in 500 years averages 0.5 mm per year) and to get answers within a reasonable time, one must therefore employ measuring methods of high precision.

Level measurements should preferably be based on a deep datum bench mark which can be assumed to be unaffected by movement of the structure or the soil underneath it and likewise be independent of structures in the vicinity. Such a bench mark may take the form of a steel tube capped with a stainless steel dome, with its lower end grouted into the bottom of a bore-hole, sufficiently deep to ensure that no movement is likely at that depth, with the lining tube of the borehole left in place but terminated clear of the grout so that it can settle with the soil without dragging down the inner tube which forms the bench mark. The space between the two tubes should be filled with a rust-inhibiting compound.

The levelling instrument should be capable of reading 0.2 mm or perhaps even 0.05 mm. A self-levelling 'automatic' instrument may have advantages because it is less likely to drift slightly out of adjustment than the very accurate bubble levels with opposed-screw adjustment. The levelling points should be either grouted-in sockets into which a removable ball-headed bolt can be fitted for supporting a precision levelling staff or be short lengths of non-tarnishing metal scale, permanently plugged and screwed to the fabric, preferably at re-entrant corners where they are unlikely to be damaged.

When checking variations in out-of-plumbness with the accuracy required, plumb bobs may have to be furnished with vanes and suspended in buckets of oil or water to dampen oscillations, and measurements must be taken with a micrometer between the plumb wire and round metal studs permanently fixed in the masonry. Whilst cheaper in prime cost, this method is labour expensive when a number of points have to be checked and optical plumbing should be considered. Instruments are available which will read to an accuracy of 1 mm out-of-plumb in a 100 m height, but for the smaller heights in question here, the accuracy will be affected by the precision with which one can focus on the target. High level targets should be fixed by stout three-legged brackets out of reach of ordinary maintenance ladders. They should have graduated scales printed on glass and be capable of illumination from the ground. The low level bulls-eye target should be set below floor level, preferably in a pocket in a mass of concrete large enough to remain undisturbed by floor renovation, etc. For ease of operation it would be worth setting it to correspond to the centre of the high level target at the beginning of the exercise so that no zero corrections need be made to future readings. The alternate way of checking changes in slope would be to employ precision spirit levels with graduated tilting screws such as are used occasionally in mechanical engineering. These have plane machined bases which have to be placed on special machined reference plates fixed on brackets. This method will, however, only measure the change in slope at each reference plate position and the change in shape of the member will have to be re-constructed geometrically, thus losing a large degree of accuracy. The same applies to the more sophisticated electronic levels which, however, may be easier to read.

Linear dimensions can be checked with *Invar* tapes which are practically unaffected by temperature variation. It would be possible, if they were permanently installed on pulleys with tensioning weights, to read to an accuracy of say 0·2 mm with a vernier device. However, if they have to be taken down and re-hung every time, the inaccuracy of the measurements is likely to double.

The traditional way of checking movements at cracks has been the use of tell-tales which have taken the form of either glass slips cemented on either side of the crack, or specially z-shaped pieces of pipe-clay with a wasted portion in the centre. Experience at York

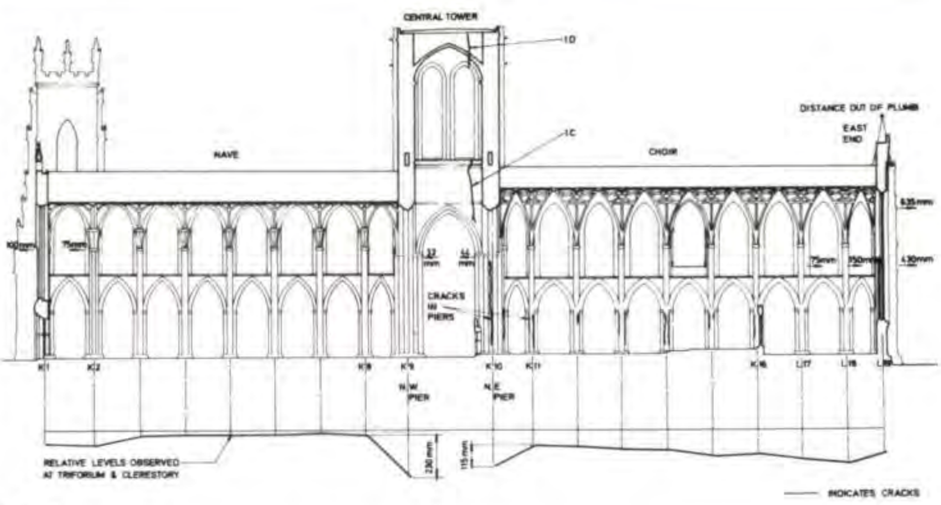


Fig. 1
Section through nave and choir showing movements and cracks

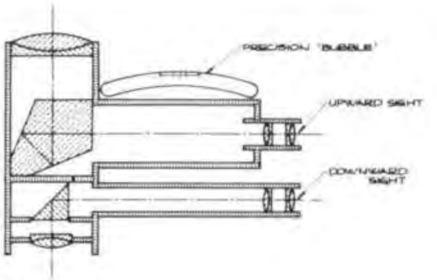


Fig. 2
Principle of optical plumb

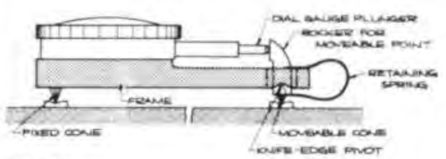


Fig. 3
Principle of *Demec* gauge

Minster indicates that failure of glass tell-tales often occurs where they are cemented to the structure and may in some instances be produced by shrinkage of the polyester cement. Even if the failure is due to structural movement there is no way of ascertaining the magnitude or the direction of any further movement with adequate accuracy. This applies also to the pipe-clay variety.

Where the main direction of a suspected movement can be foreseen, one can use a pair of small oblong brass plates which are placed with their long edges touching each other, and parallel to the expected movement. These plates are cemented or rawlplugged, one to either side of the crack. If a line is scribed at right angles to the touching long edges, future movement of significant size can be directly measured from the off-set in the scribe line.

At St. Paul's a very ingenious system has been operated for the last 50 years: this entails the cementing in of two cylindrical metal studs either side of the crack and the use of a set of specially constructed micrometers (of which only one set is in existence) which will measure movement across the crack, at right angles to it, and out of plane movement of the structure either side of it to 0.001 mm. This hardware is highly sophisticated and in consequence very, very expensive.

At York Minster a demountable mechanical strain gauge is used to measure the variation in distance between three metal studs glued to the structure in the pattern of an isosceles triangle so that two studs are situated on one side of the crack and one on the other. If measurements are taken of the variations in distance between the single stud on the one side of the crack and the two on the other, a simple calculation can produce the movement parallel to, and at right angles to the crack. The gauge length used is 100 mm and a setting template, which is part of the outfit, ensures that the studs are set at the correct centres. The instrument (trade name: *Demec*) is moderately expensive (about £80) but comparatively easy to use. A screw micrometer may be used in lieu of the *Demec* gauge but is much slower and more dependent on the operator's skill.

A somewhat cruder instrument would be a magnifying glass with a graduated reticule commonly sold as a 'crack microscope'. The reticule is available divided into tenths of mms. and provided that a sharp pencil line is drawn around the circular base of the instrument initially and the instrument is always applied, concentrically with the

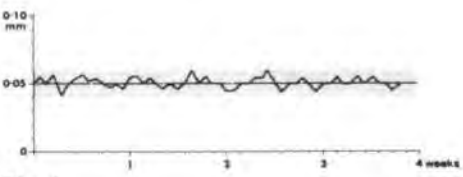


Fig. 4
Noise

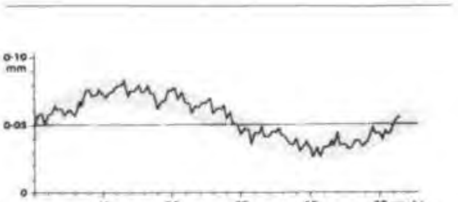


Fig. 5
Noise + periodic oscillation

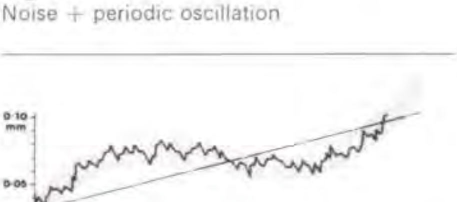


Fig. 6
Noise + periodic
oscillation + structural movement

marked circle, a fair degree of accuracy can be achieved.

There is always a possibility that movement will occur as a result of remedial measures being carried out. These movements may be indicative of certain unforeseen reactions of the fabric or the foundations to the building operations and should therefore be checked in a similar way to that described above. They do, however, tend to be more rapid and in consequence the instrumentation can be slightly cruder than that described for checking ongoing movement, but nevertheless considerably more accurate than that which suffices to establish what movement has taken place in the past.

Recording and interpretation of readings

It is essential that readings are recorded in books kept specifically for the purpose in such a way that a newcomer can, by studying the introductory notes in each book and the setting out of the results, immediately see what the figures represent. It is tempting to let 'George' maintain his own system of booking because 'he knows what he is doing', but George may walk under a bus tomorrow, and unless his hieroglyphics are decipherable, all his good work will be wasted.

Even the best kept note-book in the world is, however, useless on its own when it comes to interpreting the readings. Basically any series of readings should initially be plotted against time. Only this will enable one to distinguish between the three features which I will call, for convenience 'noise', 'seasonal oscillation', and 'movement'. 'Noise' is the fine waviness of the plot due to unavoidable random inaccuracies in instrument reading inherent in the process employed. 'Seasonal oscillation' is the movement about a steady average position caused by changes in temperature and humidity. If one took hourly readings with a sufficiently fine instrument, one would get a plot showing the diurnal variations. If the readings were continued over a year one would see the diurnal variations superimposed on an annual movement, but over several years there would be no total movement in one direction or another from 'seasonal oscillation'.

'Movement' is shown by a slope of the envelope for the seasonal oscillations which indicates that year after year there is a resultant movement in one direction.

'Noise' is common to all observations where an instrument permits readings which are more accurate than the measurement being taken. 'Seasonal oscillation' is mainly observed in widths of superstructure cracks. The level of both of these 'confusing' phenomena must be established before any significant 'movement' can be deduced from the readings.

Foundation investigations

Foundation investigations occupy a middle position between recording and analysis in as much as the soil strata have to be identified and recorded and their properties ascertained by laboratory tests, but the real benefit of the exercise only becomes realized when combined with calculations of ground pressures and settlements.

The soil properties will usually be determined from laboratory tests on samples in the case of cohesive soils and in situ tests, such as the Standard Penetration Test, in the case of granular soils. In both cases bore-holes are required in sufficient number and on such a pattern that reasonably reliable soil sections can be deduced for the main parts of the building. Ground water levels are obviously of interest and for clay soils the pore pressure, as indicated by piezometers left in one or two bore-holes, is important. For sandy soils the density and presence of subterranean water flows should be checked.

The object of all this is to establish, once the

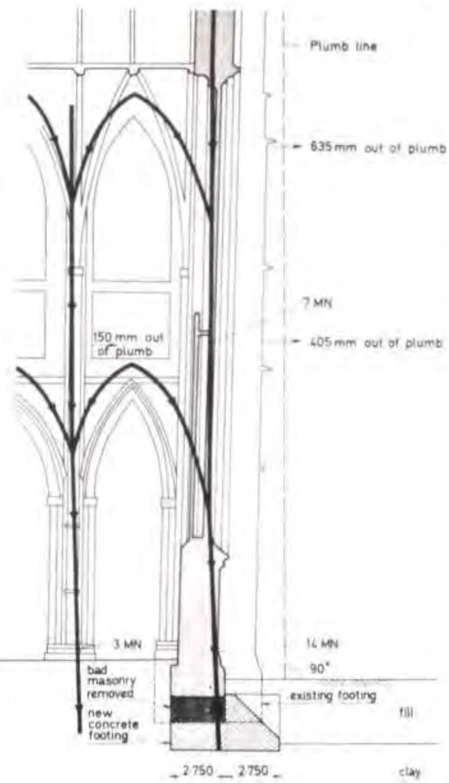


Fig. 7
East end. Diagram of thrusts and movements

size of the foundations and the loads are known, the safety of the existing foundations and their likely response to any change in their environment such as could be brought about by piling and/or heavy traffic consolidating loose sands, deep basements causing washing out of fines through pumping or, conversely, stopping subterranean flows of water. In some instances it may even be possible to get a rough idea of the past behaviour of the foundations of the building as one can, in processing the laboratory tests, 'back-pedal' so as to study the behaviour of the soil strata at the time when the building was first constructed.

Structural analysis for assessment of safety

In the building industry today, structural analysis is normally used for calculating sizes of members in structures to be built or to check stresses against 'permissible' stresses in existing structures. In ancient buildings the sizes are given and, when considering the safety of the structure, the stresses should be evaluated from first principles as the 'safe working stresses' laid down in codes and regulations are often determined from considerations other than those of safety, i.e. limiting initial and long-term deflection, preventing cracking, etc. In an old building one accepts the existing deflections, and there is therefore no need to limit one's stresses below what is necessary to maintain safety and limit further deterioration.

One should also approach with an open mind the problem of what constitutes failure and how it happens. In masonry, failure under compressive load will take the form of shearing, bursting, or splitting, but very rarely crushing of the stone. In fact, stresses in traditional masonry structures, including most Gothic slender flying buttresses, tend to be very low compared with the crushing strength of the stone.

As far as timber roof structures are concerned, one need very rarely concern oneself with the members as it will invariably be the joints that are the weak points.

Very few ancient buildings have 'text-book' structural frames and an accurate elastic analysis can therefore become very complicated, but for purposes of safety one can resort fairly simple limit state assumptions. If equilibrium can be established between the external forces acting on a part of a structure and the internal forces in the structure, and if the internal forces can be mobilized without requiring an excessive stress level anywhere, then the structure will be safe regardless of whether or not the stress distribution assumed is in fact the correct elastic one.

Whilst lintel-and-post structures will therefore not pose any problems, timber trusses can be structurally highly indeterminate with a large number of 'redundant' members, but provided the joints are capable of a certain amount of 'give,' one can assume that all parts which are capable of contributing to the load carrying capacity of the truss do in fact do so. (This may be the time to point out that there has been some myths about hammer-beam roofs, attributing to them some very special load-carrying characteristics which in truth they do not possess as they do not work without exerting an arching thrust against the supporting walls or by developing bending moments in the joints).

The analysis of simple masonry arches can often be carried out as a simple thrust-line construction. Due to the usually low stress levels all that is required for safety is to find a thrust-line which will balance the forces on the arch whilst remaining within a distance away from the boundary of the arch equal to between 5 and 10% of its thickness. The thrust-line is constructed as an inverted string polygon to the forces and the horizontal thrust exerted by the arch can be read off the diagram. One can in fact extend this exercise to construct the thrust-line extended from the arch down through pier and buttress right to

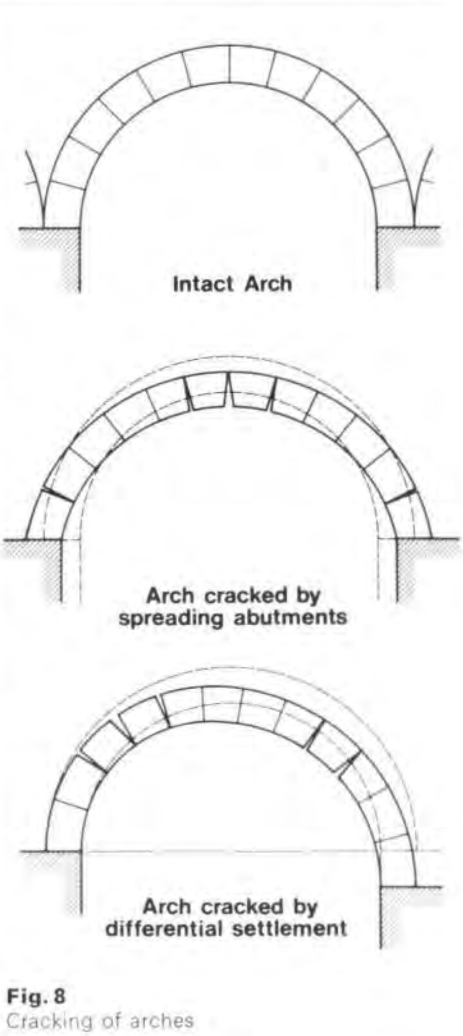


Fig. 8
Cracking of arches

the foundation, and here of course the position of the thrust-line can become much more critical as few soil strata will allow much unevenness of ground pressure before they suffer differential settlement which will result in the foundation rotating with whatever it supports. The east end of York Minster is a prime example of this where high eccentricity of the load on the buttress foundations led to a rotation which in turn aggravated the eccentricity to the extent that the east wall is now about 0.6 m out-of-plumb.

Structural analysis as aid to explaining past defects

Heyman has postulated that if a masonry arch stands up for five minutes after the scaffolding has been removed, it will stand up for 500 years. In this statement, it is assumed that the supports for the arch in question are practically immovable in space and time and the foundation settlements are negligible.

In a less ideal world foundations do settle appreciably and differential settlements give rise to cracks, and so does the spreading of abutments of arches due to the horizontal thrust. But how does one distinguish between the effects of the two different causes?

There is a kind of structural analysis which can be done 'by inspection' which will indicate the likely causes of certain cracks. For instance, if there are cracks on the inside of the arch, near the crown in a symmetrical pattern, the supports are likely to have moved out horizontally. On the other hand, if there are 'anti-symmetrical' cracks at the quarter points, i.e. on the inside of the arch at one quarter point, and at the outside at the other, the cause is likely to be differential settlement of the supports. Similarly, if a wall leaning outwards is concave on the side of the lean, the cause is likely to be horizontal thrust from roof or vault, whilst if it is convex on the side of the lean, the mostly likely cause is an eccentric foundation which started rotating during construction. Another illustration of 'analysis by eye' is given at St. David's Cathedral where a very low-pitched roof 'truss' is supported on walls which lean out alarmingly. There is, however, no indication of movement between the horizontal tie beam of the roof truss and the walls. As a truss of this kind cannot exert any horizontal thrust, it must mean that the lean of the walls was caused by an earlier roof in conjunction with weak foundations and that with the replacement of the roof, the cause of lean has been eliminated and the movement arrested.

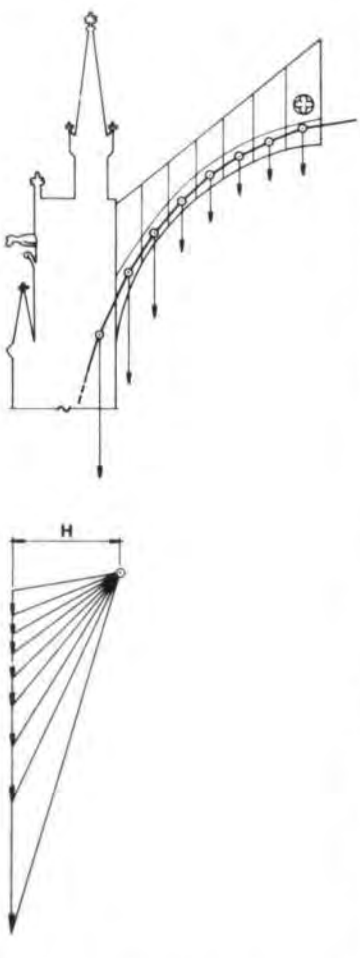


Fig. 9
Geometric thrust – line construction

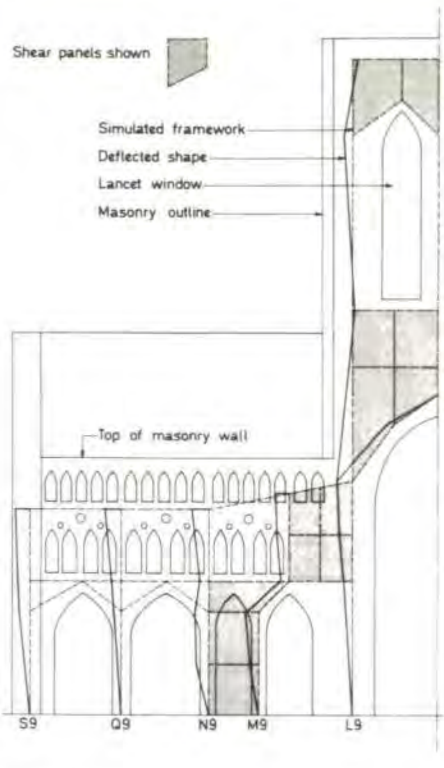


Fig. 10
Mathematical model and calculated deflections from gravity loads

When one comes to multiple bay arcades, with triforia superimposed, where the spandrels over the arches contribute substantially to the strength and stiffness of the whole system, things are no longer simple enough for this kind of deduction. Even the determination of the stress pattern becomes difficult.

One might at this point consider the use of structural models and on the face of it, photo-elasticity is an attractive technique as it is possible to load a three-dimensional model in a heated chamber and let the model cool off whilst loaded. When this has been done, all strains are 'frozen' into the model which can then be cut into slices which can be analyzed on a photo-elastic bench. However elegant in principle, the making and loading of the model is a difficult and expansive task, and the actual photo-elastic analysis is a fairly lengthy mathematical procedure if quantitative results are required. If one could make a model of fairly soft rubbet and coat it with a brittle lacquer one could ger a very easy visual indication of regions of high stress, but there are great difficulties in simulating gravity loads on a model like this. On balance therefore, the best choice is a mathematical model which can be analyzed with the aid of an electronic computer. For the particular example of arches with spandrel panels, the mathematical model can conveniently take the shape of a rigid frame with shear panels.

The objection may be raised that the loads and material properties are not known with an accuracy commensurate with that inherent in a computer analysis. It must however be remembered that, in this instance, the purpose of the computer is not to produce extreme numerical accuracy but to enable one to manipulate a large mass of data and as stress levels are not crucial, reasonably good, approximate figures for loads will be adequate for the purpose. Likewise, as one does not rely on the analysis to produce the exact magnitude of individual deformations as long as the pattern is in proportion, the elastic properties of the material need not be specified with very great precision.

The Central Tower complex at York Minster was treated this way as follows. A plane frame analysis was carried out for a structure consisting of the transept arcade and the Central Tower pier walls in that plane. Gravity loads were first considered and it was found that the calculated movements and stresses were inconsistent with the deformations and cracks observed in the fabric. A similar analysis was then carried out assuming that the Central Tower had settled relative to the rest (as was in fact observed). Here again it was found that the calculated results of such a settlement were in conflict with the observed and measured distortions of the fabric and one was therefore faced with the puzzle of how the transept piers had come to lean the way they did. It was only the 'rediscovery' of the existence of an early English tower built about 1250 and a casual reference to its fall in 1407 that presented the solution to this riddle.

There was still the remaining problem of the large cracks in the tower spandrels under the lantern and whilst analysis had shown small tensions under the windows due to gravity loads, the correlation was not satisfactory.

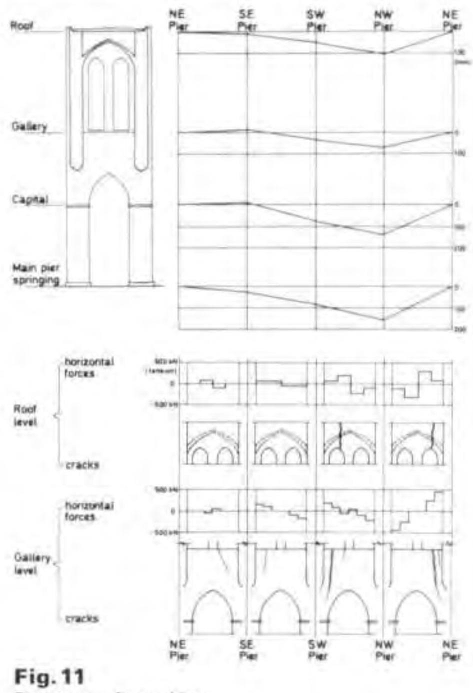


Fig. 11
Patterns of cracking

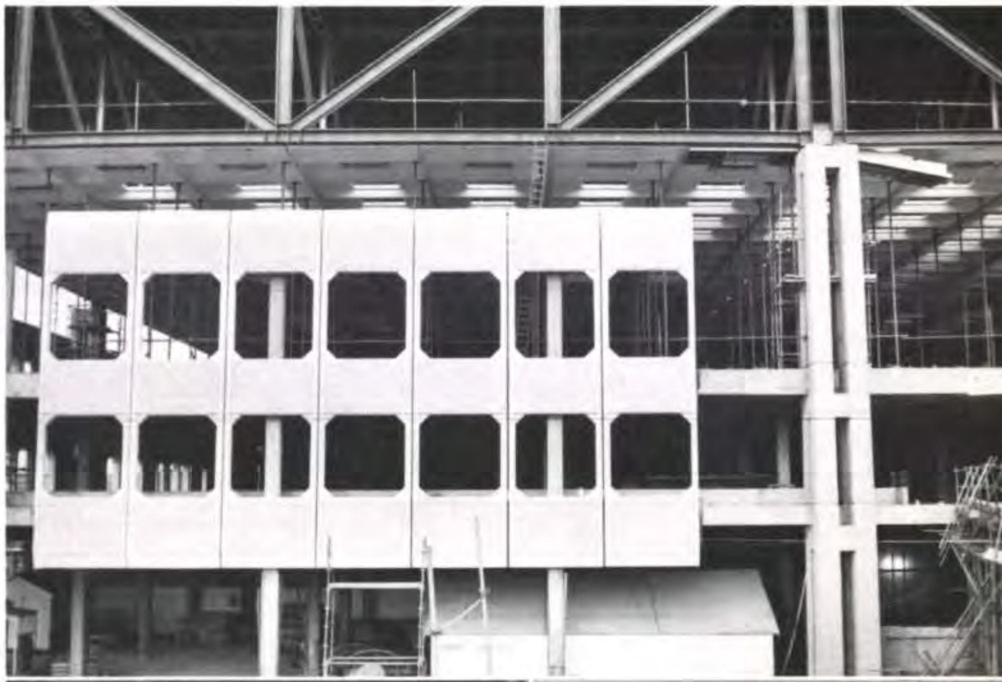
Re-examining the records of the measurements it was found that the north-west pier showed signs of having settled about 120 mm more than the remaining three at the level of the lowest string-course whilst the difference at gallery level was only 50 mm.

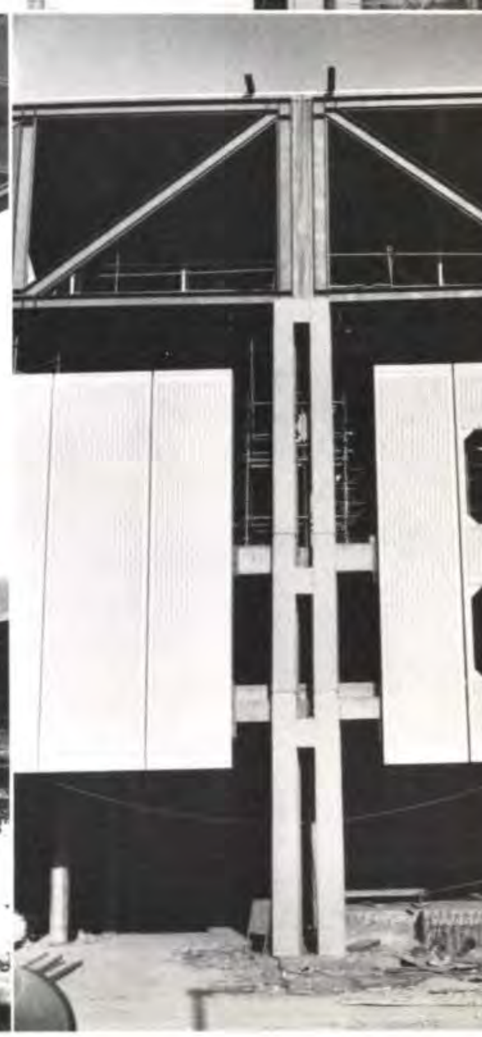
A space frame model was now analyzed on the computer assuming these differential settlements within the tower, and this time the magnitude and disposition of tensile forces checked with the actual position and extent of the cracks.

New factory for John Player & Sons

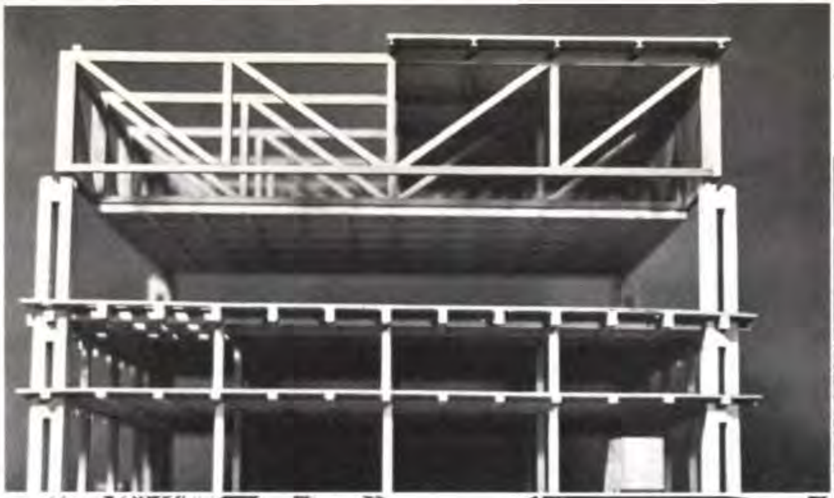
Code named: Horizon

Architects and Engineers: Arup Associates Group 2
Management Contractor: Bovis Ltd.
(Photographs by
Colin Westwood,
Richard Einzig and
Arup Associates Group 2)





The photographs on the following five pages may, at first, give a kaleidoscopic impression of the building. A short study of the photographs, however, should clearly reveal the relationship of design philosophy to concept, to structure, services and enclosure. Photographs of models (made before the building) have been juxtaposed with those of construction and the completed building.

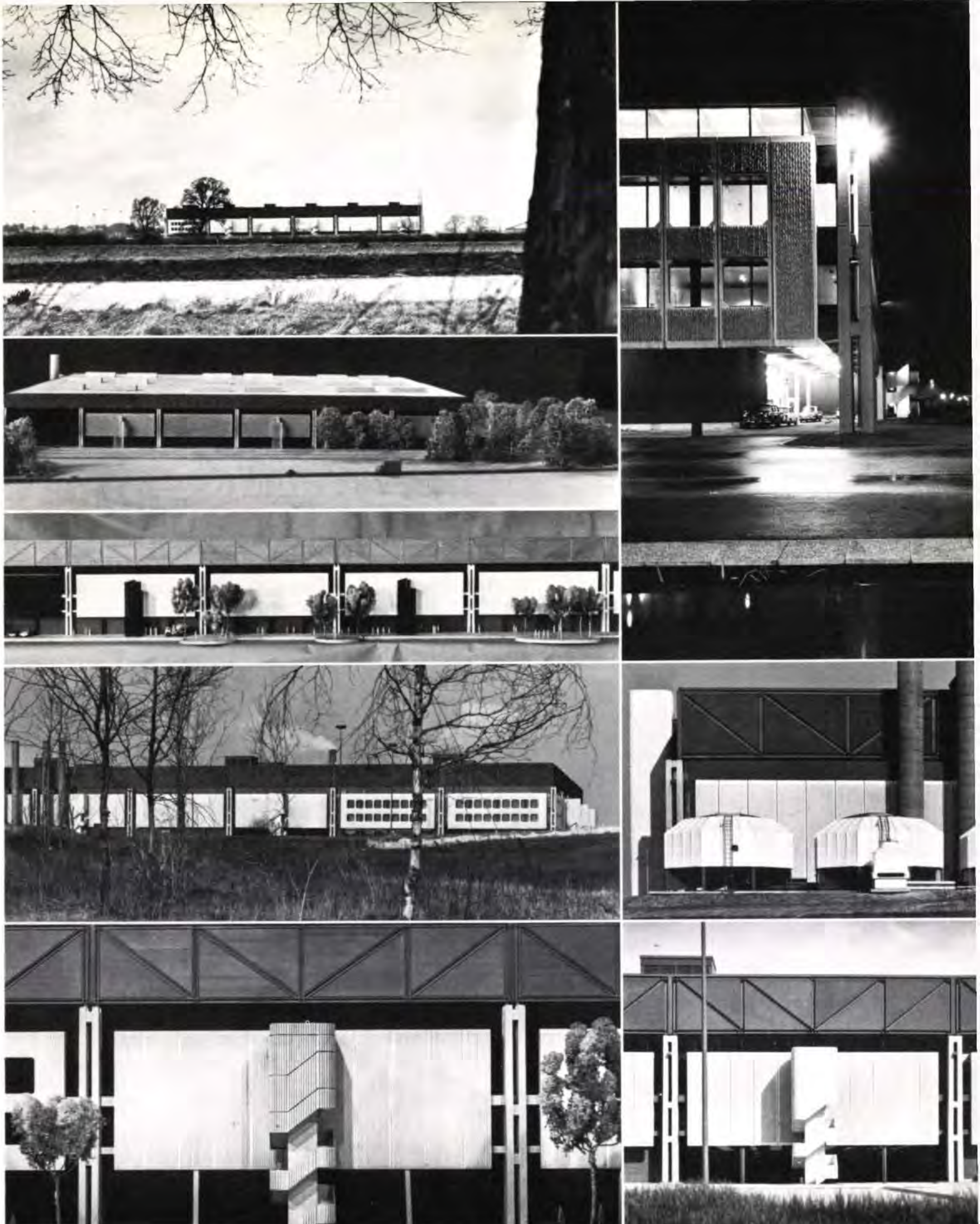






Concrete and steel structure, construction and scale.

Textures and scale (the projecting ribs on the stairs are 40 mm x 35 mm, and on the cladding panels 85 mm x 80 mm)



Power house, inside and out

Horizon services installations

Arup Associates Group 2

The building and process services for the factory are generally located at two levels:

(a) roof void plant room which is 5.5 m high, and

(b) a services void, located beneath the first floor production area, which is 3 m high. The main power house and energy source is situated at ground level in the north-west corner of the building.

Air handling

The factory is fully air-conditioned to provide the environment required for the tobacco as it passes through its various processes from its baled leaf state to its final form as a packaged cigarette.

Apart from the necessity of ensuring a high standard of quality in cigarette manufacture, the needs of taxation alone make moisture control of vital importance. The air-conditioning is controlled to preserve the correct moisture percentage and all changes in moisture content, whether gain or loss, are closely observed. It is essential therefore that the relative humidity is controlled to a very close tolerance.

The conditions required for the process proved in practice to give a comfortable environment to the operatives. Although the wet bulb temperature is on the high side this is to some extent balanced by the considerable amount of air movement.

Some examples of internal conditions are:

Primary Department:	21 °C dry bulb $\pm\frac{1}{2}$ °C 70% relative humidity $\pm$2%
Making Department:	21 °C dry bulb $\pm\frac{1}{2}$ °C 60% relative humidity $\pm$2%
Despatch Department:	21 °C dry bulb $\pm\frac{1}{2}$ °C 60% relative humidity $\pm$2%

The air-conditioning system allows the various departments of the factory to be supplied with conditioned air from the air handling plants located at the roof plant level.

There are 27 air handling plants which together supply a total volume of air in excess of 944 m³/sec. to the factory.

Each plant comprises a fresh and return air mixing chamber, roll type filters, sprayed cooling coil, reheater and centrifugal fan serving a low velocity duct system which terminates in diffusers mounted at ceiling level in production areas and primary department.

Return air from each area is induced via return air grilles through a system of ducting and an axial flow fan. This return air is either discharged to the atmosphere or returned to the plant, depending on the dictates of the controls.

The sprayed cooler coils are supplied with chilled water from the main refrigeration plant via an automatic three-way valve in a locally pumped secondary circuit connected to the primary chilled water circuit.

One of the air handling plants serves a part of the factory known as the Amenity Area which consists of office areas, kitchen and restaurant, shop, medical centre, lockers, toilets and reception area.

The office areas, with the exception of the perimeter modules, are treated by means of a low velocity duct system terminating in ceiling mounted diffusers. These areas are zoned, each zone being provided with its own heater battery.

Air is returned from the offices through return air diffusers mounted adjacent to the supply air diffuser in each module, thereby catering for the maximum degree of flexibility in planning the office partition layouts.

The perimeter modules of the office area are served by a non-changeover induction unit system, which is zoned and served from the air handling plant by means of a single high velocity duct system which terminates at the perimeter of the building in sill-type induction units mounted two per module.

The shops, medical centre, lockers and reception area are air-conditioned by a system of high velocity ductwork, distributed horizontally through the void above the false ceiling over the restaurant and terminating at a number of single duct high velocity terminal units housed within the ceiling void and discharging air into the void. The ceiling is a ventilating type used for the distribution of conditioned air supplied by a pressurized plenum.

Air is returned to the plant in the roof by means of a continuous return air grille fitted with volume control dampers, fixed in the ceiling to a system of low velocity ductwork.

The restaurant is similarly treated but air is extracted from the area via the servery and kitchen area through the kitchen hoods and their associated extract fan.

Total energy

A single fuel is used for the provision of all the energy and service requirements of the factory. Use of waste heat recovery is made wherever possible to improve the operating economy and produce energy at the lowest possible cost.

This total energy concept was recommended for the factory because the feasibility study, carried out in the early stages of design, indicated that savings in operating costs would be of the order of £100,000 per annum.

North Sea Gas provided the single fuel in adequate quantities and at a relatively low cost and, as the factory is not connected to the National Grid, it was essential that a prime mover be employed which had a long proven record of reliability. Also, because gas could be purchased at a cheaper rate if the client negotiated a contract for the supply of gas on an interruptable tarriff basis, it was necessary that the prime mover be a dual fuel machine.

To allow for the 30-day interruption period written into the client's contract with the East Midland Gas Board, a standby fuel, in the form of distillate oil, is stored in a 909,000 litre underground storage tank.

Investigation and inspection of a number of total energy installations showed that the Ruston gas turbine had been providing an extremely high degree of reliability and economy in various applications since the first was installed in 1956.

Ruston gas turbines were therefore selected for the total energy plant, which comprises eight TA1500 gas turbine generating sets, four exhaust heat recovery boilers, interconnecting ducting, control systems and ancillary equipment.

The eight generating sets each comprise a Ruston TA1500 twin shaft, open cycle, axial-flow gas turbine having a continuous brake horse power output of 1080 kJ at 16.5 °C ambient air temperature. The power output coupling, running at 25 revs/sec., is solidly coupled, via a short extension shaft, to an English Electric AEI 1152 kW, 50 Hertz, 11,000 V., three-phase, four-wire, ac generator.

All eight gas turbines and associated generators are installed in the turbine room which forms part of the services void immediately above the main ground level power house. It was anticipated that the factory power and heat requirements would increase in the future and space was left at each end of the existing line of tubines for a total of four further generating sets.

Exhaust gas from each pair of turbines is ducted to a *Spanner-Swirlyflo* exhaust gas boiler, four of which are located in the main power house immediately below the turbine hall. Each boiler produces at a pressure of 1343 kN/m², 2.2 kg/sec. of steam when fired by the exhaust gases of two turbines and 3.8 kg/sec. with auxiliary firing. Two after-burners or combustors are fitted for this purpose, one to fire into the gas stream of each turbine.

The combustors are automatically controlled to maintain the required temperature of gas to give the required output of the boiler and compensate for any fall in gas temperature from the turbine due to reduction in load.

If at any time the steam requirement falls below the amount of steam that can be produced from the turbine exhaust gas, then a portion of the gas is automatically diverted to atmosphere via the external chimneys.

In the event of an interruption in the gas supply, or fall in pressure, the gas turbines which are running will automatically change over to the distillate fuel and the boilers associated with them will also automatically change over to the standby fuel.

The turbine can be started locally or remotely by a selection switch at the turbine which transfers controls to the grouped control panel in the main power house control room. This panel provides grouped supervisory controls and fault monitoring for the turbines, generators and boilers and automatic synchronizing of any turbine generating set with other generating sets. It has four turbine and boiler control sections and one central common control section. Each turbine/boiler section contains the grouped main alarms, instrumentation and controls for two turbines and their associated boiler. The common control section contains busbar instrumentation and a gas main pressure gauge.

Heat emission from a turbine is 102 kW and alternator operating conditions must not exceed 39 °C if high efficiency of operation is to be maintained, so it was necessary to provide adequate ventilation to the turbine room. This was achieved by installing extract ducts along one side of the room at high level and fresh air inlet ducts at low level on the opposite side of the room. An extract fan is fitted into the extract duct immediately above each turbine and is connected into the turbine control system so that when it is started the fan draws fresh air across the alternator and turbine to discharge to the atmosphere.

The noise created by the turbines is of the order of 100 dB and to confine it to the turbine room it was necessary to install silencers in the supply air and extract air ducts adjacent to the external louvres. Rustons fit a silencer to the turbine inlet connection and on the exhaust side the boilers and chimneys give a good degree of attenuation.

One of the features promoting the viability of total energy was the even balance of loads, electrical and steam for both air-conditioning and process throughout the operating schedule of the factory. This was achieved when an imbalance of loads (weighted on the electrical side) was turned into a balanced condition by the use of steam-driven refrigeration machines rather than electrically-driven machines.

Refrigeration equipment

The refrigeration equipment is integrated with the total energy plant and the whole of the chilled water for air-conditioning purposes is provided from the steam produced.

A steam turbine-driven centrifugal/absorption machine combination system was used which can operate for nominally 1.76 g/sec. of steam per ton of refrigeration (1 TR = 3.516 kW). This economic use of steam is achieved because, by using a single centrifugal machine driven by a back pressure steam turbine, exactly the same flow rate of steam at the back pressure can be matched to the operation of a pair of absorption machines.

The refrigeration plant for the factory consists of one external drive *Centravac* manufactured by Trane Ltd., rated at 4180 kW, driven by a Greenwood and Batley multiple stage horizontal steam turbine developing a bhp of 961 kJ with a steam inlet pressure of 1308 kN/m², and exhaust pressure of 220 kN/m², and two Trane absorption machines each rated at 3670 kW when operating on steam supplied at 186kN/m². All machines are located in the main power house.

A total of 417 l/sec. of chilled water is pumped to the air-conditioning air handling plants in the roof void plant room.

Cooling water to the condensers of the refrigeration machines is pumped through three Thermotank fibreglass-clad cooling towers located external to the power house and each capable of cooling 189 l/sec. through 11.5°C.

Other items of equipment housed within the power house are:

Five Hick Hargreaves water-cooled rotary air compressors, each having a capacity of 0.23 m³/sec. at 792 kN/m².

Five vacuum pumps of the same manufacture, each capable of pulling 0.43 m³/sec. at 508 mm Hg.

Hot water supply calorifiers, auxiliary cooling water pumps serving turbines, air compressors, boiler afterburners and vacuum pumps, air compressors for automatic controls, water treatment plant, daily oil service tanks and pumps, sprinkler protection pumps and cold water suppy pressurizing units. Space has been provided for two future boilers and an additional refrigeration machine.

The process services are distributed to the production and primary floors via a deep services void located between these floors which is primarily for the use of machine services to each floor.

Steam at 1343 kN/m² and 517 kN/m² and compressed air and vacuum at the aforementioned conditions are taken through the void, with plugged connections provided at frequent intervals in all service pipes to allow the client maximum flexibility when making final connections to the process machinery.

Electrical services

The electricity generated by the total energy plant is distributed at 11 kV 50 Hz from a main switchboard located in the power house. From this switchboard, two 11 kV ring mains feed six substations, each with a capacity of 2 MW, located on the ground floor of the building. The substations comprise an 11 kV switchboard and three 1,000 kVA oil transformers, two duty, one standby, serving an associated main medium voltage switchboard located in the service void area. Both the high and medium voltage main switchgear is arranged for remote indication and control from the power house control room. The electrical load of the building is approximately 8 MW comprising 1.0 MW lighting, 2.5 MW air-conditioning plant and auxiliaries and 4.5 MW process plant.

Distribution to process plant equipment is from a network of 800 amp. busbar trunking located at high level in the service void and covers the entire area of the building, allowing the user to 'tap in' at close centres to feed machinery located on the production floor. Associated with the busbar trunking is a cable trunking allowing flexibility in arranging control and supervisory cabling.

Air-conditioning plants located in the roof void account for the main electrical load other than the process equipment. Medium voltage cables rise vertically from the MV switchrooms in the service void, via the cluster columns and terminate at a motor control centre associated with each air handling plant. Lighting to the production floor is by fluorescent tubes, recessed into special service panels located in the coffered ceiling, allowing maintenance to be carried out from above.

In other areas lighting is by fluorescent tubes surface mounted on trunking or within the coffered ceiling throughout the factory, with the exception of the amenity area which is lit by tungsten 'downlighter' fittings recessed into the suspended ceiling, and in the power house by colour corrected mercury vapour lamps.

A system of battery operated emergency lighting covers the entire factory in the event of an electrical supply failure. Illumination levels in production areas and service areas are 600 lux and 250 lux respectively.

Externally the lighting is by high pressure sodium lamps in special fittings attached to the major columns of the building, illuminating the perimeter road and mounted on shrouds on four 24.4 m high masts in the external car park.

The building is equipped with three lifts, one passenger duplex and a kitchen service lift in the amenity area, and a 10,000 kg goods lift serves all floors of the factory including the roof void.

Throughout the entire factory a public address system provides for announcements and also continuous music.

The Horizon management contract

Arup Associates Group 2

The need for change

In the early stages of our appointment, it was clear that time and cost pressures were such that a study of alternative forms of contract was essential.

The client made clear the emphasis that he placed upon completion by the end of 1971 and his preference for competitive tenders. It seemed to us that traditional forms of contract arrangement would not meet the needs and so we investigated a new form of arrangement which we termed a management contract. At that time we were not a little awed by the size and complexity of the project and very doubtful that it could be designed, built and commissioned in the three years that remained until D-Day. The management contract principle seemed at that time the arrangement most likely to help in meeting the hand over date, in that it would permit us more easily to overlap the design and construction phases.

Objectives

The main objectives we hoped to achieve in formulating the management contract were to overlap the design and construction phases and to gain the advantage of specialized programming advice in the early stages of design by completing the client/designer/contractor team.

In working towards achieving the main objectives we also tried to arrange matters so that the following requirements were satisfied:

1 To gain advice in the early stages of design to help us arrive at construction-time efficient solutions

2 To extend the time available for preplanning before start on site without increasing total project times

3 To maintain the initiative in design and cost at all times

4 To obtain competitive bids for at least 95% of the works

5 To select a main contractor:

(a) Appointed in the earliest stages of design (i.e. before the expenditure and cost plan was settled)

(b) Who could provide a clearing house for placing firm contracts for long-term critical elements requiring early orders

(c) Who would accept all subcontractors in direct contract (i.e. no nomination veil).

(d) Who would do no construction and apply his full efforts to planning and organization

(e) Who would *always* be on our side

(f) Who would take contractual responsibility for organizing the works on site and ensuring that the work was carried out in accordance with our specifications.

Most of these objectives are applicable to all projects regardless of the form of contract arrangement, but at that time we felt the management contract principle gave more chances of success and our recommendations to the client, particularly with regard to programme time, were accordingly weighted in favour of this form of arrangement.

The client was obviously sympathetic to the principle of team approach and joined us in exploratory interviews with selected national contractors.

Selection

The plan was to select the national contractor most sympathetic to our objectives, who would help us in defining his responsibilities at the time when the design was merely an idea, and who would enthusiastically undertake his part for what we considered to be a fair fee. The selected management contractor was guaranteed and paid his direct costs plus fee up to scheme design stage, to which the employer's liability would have been limited had the project been stopped at that point.

Upon approval of scheme design, the management contractor's costs for his involvement before scheme design approval were absorbed into the total project cost plan.

Contractual arrangements

The management contractor was in direct contract under seal with the employer for the construction of the whole of the works to the agreed programme as in the normal situation. The bias of the main contract was however on a 'cost-plus' principle in that the employer paid the management contractor his net costs plus only a fixed percentage fee. 90% of the costs, however, consisted of provisional sums covering the elemental subcontracts and the provisional sums were calculated and competitive tenders for the subcontracts obtained by us. The remaining 10% of the costs consisted of the management contractor's estimate for providing defined preliminaries, such as temporary site roads, central canteens, temporary lighting and power, etc., and fees. All costs were subject to agreement with our quantity surveyors.

The contract was worded such that the employer could not be called upon to pay for extra costs caused by the management contractor's negligence. Any cost which the management contractor might have had to pay, and to which the quantity surveyor might not have agreed (i.e. subcontractor's claims), would be payable by the employer only after the management contractor had lost his case at law.

The subcontractors were each in direct contract under seal with the management contractor under terms and conditions similar to those applicable to a nominated subcontractor under the RIBA form. Each subcontract was placed on a lump sum basis after competitive tender submission, and tenders were generally invited on completed design information. Extensive use was made of performance specifications which helped in placing specific items of plant on order at the earliest possible date and gave industry greater scope for contributing towards the design. Several subcontracts, necessarily forming specialized integral sections of the major services contracts, were placed on firm orders by the management contractor several months before tenders for the services subcontracts were invited and privity of contract was subsequently transferred to the successful services subcontractor after his appointment. All subcontract tenders were invited against specific programmes which were extracted from the management contractor's master. The specific programmes were included in the tender documents and subsequently written into the formal subcontract agreements.

Conclusions

Differing conclusions are drawn from those who have worked on the project and the emphasis would vary depending upon who was asked. A subcontractor, having sustained a substantial loss and having been subjected throughout his subcontract works to the concentrated and combined efforts of the employers, designers and management contractors, would find it difficult to say much in favour of this way of working. Perhaps one should not draw precise conclusions until the other projects being handled in a similar way are finished. In the case of the Horizon Project, however, it can be said that the objectives were in the most part achieved on the critical items, in that the building was virtually completed by the due date after an extremely intensive construction programme and subjected to very tight cost targets. We didn't always agree with the management contractor. Perhaps it was naive to expect otherwise, but, compared with the normal main contractor situation, we achieved a much higher degree of team application without losing the initiative in design and cost control.

Some of the original objectives we now know could not have been achieved. We have learned that it is unreasonable to expect a contractor to programme design work when the production of this information cannot be either under his control or his responsibility. We also found some difficulty in sharing our responsibilities of quality control on site and that, whilst the early appointment of a management contractor does help to settle the early logistics of project organization, one should not put too much reliance upon advice related to detailed tactics for construction, which in the event will be executed under commercial pressures by another firm. There may, on other projects, be a strong case for the management contractor to execute some parts of the construction if the inherent dangers of his commercial involvement can be avoided. The kudos for the management contractor is in completing the project on time and, although this problem did not arise on the Horizon Project, difficulties might be met if a management contractor pressed for accelerated working to achieve his programme at the expense of quality and cost.

Many of the minor difficulties given were not the fault of the management contractor but simply teething troubles of a new way of working to be ironed out in subsequent management contracts.

There will be subsequent management contracts. As projects become more complex and the demand for faster construction programmes increases so too will the need for changes in contract arrangements become more critical. The Horizon management contract was a reaction to this need and a progression toward completing the team and joining with the main contractor.

Gartnavel General Hospital, Glasgow

Jim Shipway

Introduction

The first of the new District General Hospitals in the west of Scotland is located at Gartnavel, Glasgow (job no. 1818). It is to serve community needs in the north-west of the city and has been built in the grounds of the existing Gartnavel Royal Hospital close to Great Western Road, about three miles from the city centre. The hospital occupies some 10.1 hectares of these grounds and is well placed for convenient access from the surrounding residential areas.

The first stage, which is very nearly complete, includes 576 beds in an eight-storey ward block rising from a three-storey podium. The podium contains out-patients, diagnostic and therapeutic facilities, together with seven operating theatres, a central supplies department, staff changing facilities, kitchen and dining-room. Laboratories are contained in a separate building with a link to the main hospital.

Nursing and medical residences and the nurse training school are located in a separate area of the site to the west of the main building. The boiler plant is placed some distance to the south, and is oil-fired.

There is provision for expansion of the hospital, and future phases of work may include homeopathic, geriatric and obstetric units, as well as a health clinic. (See Fig. 1).

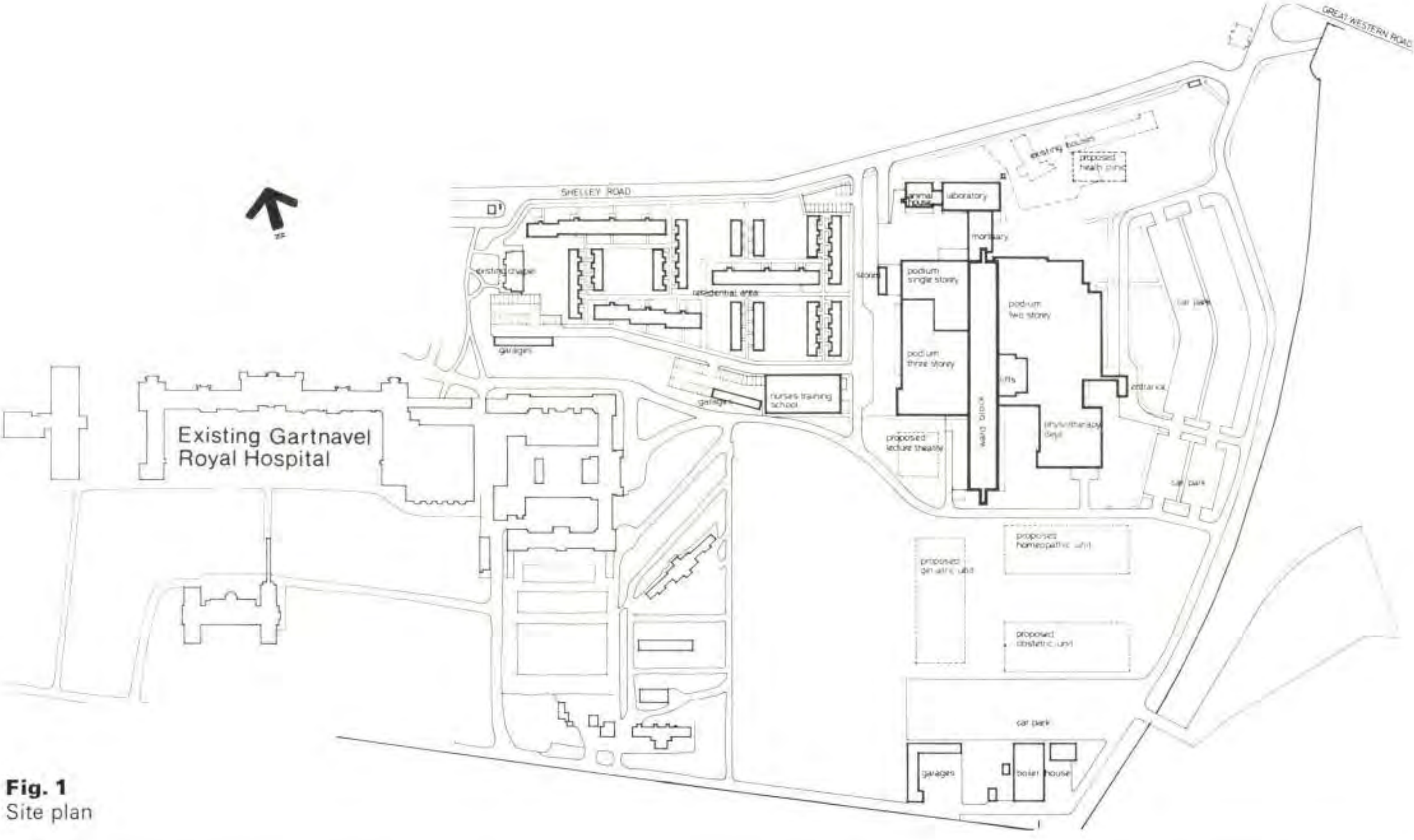


Fig. 1
Site plan

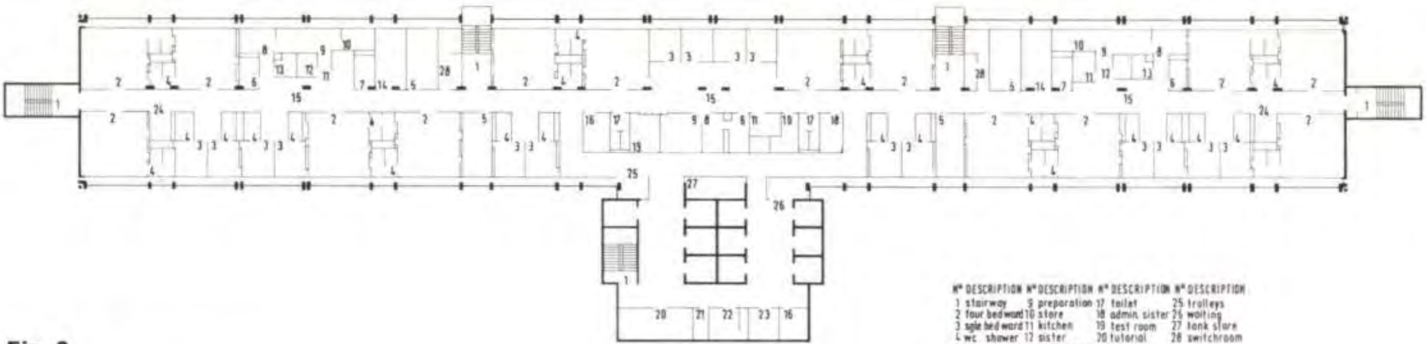


Fig. 2
Linear plan of ward block

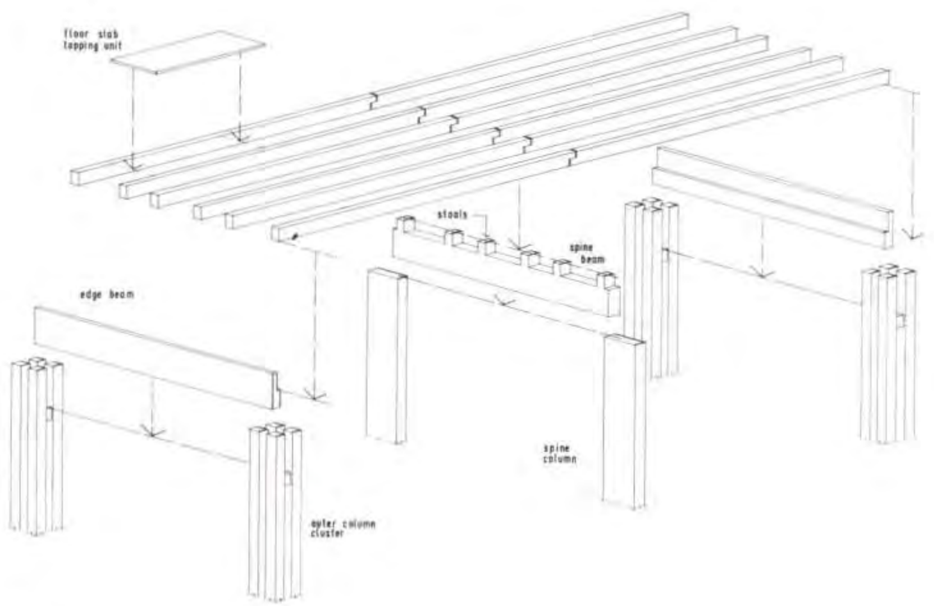


Fig. 3
Arrangement of precast structure

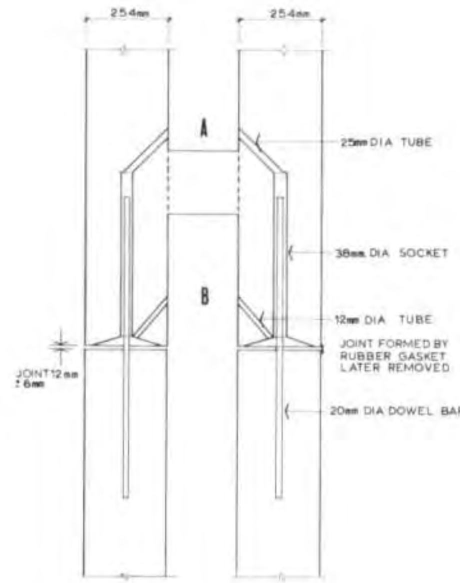


Fig. 4
Column joint

Programme

The architects, Keppie Henderson & Partners, received an early brief in February 1964, and the final brief in January 1965. From the first, the accent in building Gartnavel was to be on speed, because of the necessity to accommodate needs resulting from hospital demolition and rebuilding elsewhere in the city. It was aimed to complete design work in two years and build the hospital in three years if possible, achieving completion in 1970. The design programme was largely met, but for various reasons the date of the award of the contract and the agreed programme of building have resulted in a later completion date.

The design team considered, in 1964, that the programme time required a non-traditional approach, and that management techniques should be introduced into the design process to eliminate bottlenecks and aid co-ordination. They also recommended having a nominated contractor and a negotiated contract. The client accepted these proposals, and John Laing Construction Ltd. were appointed, on the basis of a letter of intent, in December 1965. One of the factors in the choice of this firm for main contractor was that they had a wholly-owned subsidiary, H. J. Cash Ltd., who would be subcontractors for the services engineering.

PA Management Consultants Ltd. were appointed by the client to advise on networking techniques, and in time a master network covering the design process from sketch plans to start-on-site was produced. This was the joint effort of all members of the design team and the tender date was programmed for June 1967. A programmer was appointed by the team to service the network and advise on up-dating, etc., and the client also appointed a member of his own staff to liaise with us on this aspect. No-one at this time, 1965, had much experience of networking, and it took time to learn that the network was only as good as the information fed to it. Generally, however, it proved a useful tool and the tender date achieved in June 1967 was only three weeks late on the programme.

At this point there was some delay in resolving both the final cost limit allowed by the client, and the tender figure submitted by the contractor. Eventually a final cost limit of £5.272 m. was agreed in December 1967 and the job started on site on 1 April, 1968.

Gartnavel was therefore largely designed between 1965 and 1967 and has taken four years to complete on site.

Structure

Form of the ward block

The main aspects of interest of the structural work were in the ward block. (See Fig. 2). The 72-bed linear ward plan limits the mechanical ventilation required, and generates the plan shape of the block, 129 m long by 17.9 m wide. Lifts and stairs are located in a tower linked to the centre of the block, with additional stair towers at each end.

With the exception of the lift and stair towers and spine columns, which are of in situ concrete, the entire block is built of precast concrete units stitched together with in situ concrete joints. The elevation presented to the westerly prevailing winds is substantial as are the wind forces. The resulting horizontal shear forces are resisted by the in situ lift and stair towers at the centre and ends of the block, with the precast floors acting as lateral girders transmitting the forces to them.

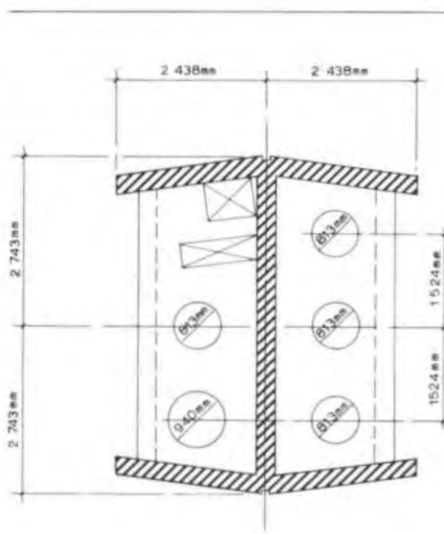


Fig. 5
Plan of chimney flue arrangement

Omission of movement joints

Very early in the design period a decision had to be made regarding the provision of movement joints in this 129 m long structure. The following points were relevant:

1 The wards, once built, would provide a controlled environment with little temperature variation within the cladding.

Fig. 6
East elevation of main hospital block
(Photo: W. Ralston Ltd.)

Column joints
A critical feature of the design is the joint between storey heights of the precast columns. A few of these joints on the lowest floor are stressed to around 13.75 N/mm² and it was essential that the jointing material should be easily placed and free from shrinkage, as well as of high strength. There were approximately 900 joints to be constructed, and any difficulty in making them would be a major cause of delay.

Several materials were tried experimentally, including *Embeco* mortar, which was used successfully for a similar joint on the London Stock Exchange building. However, the Stock Exchange joint detail incorporates a permanent rubber gasket which contains the mortar within the joint and prevents staining. The detail at Gartnavel employed a temporary gasket and the concrete joint was exposed. High strength grouts were rejected because of shrinkage and cold weather difficulties. Dry-packed materials were also rejected because of the difficulties of ramming them effectively without access by scaffolding or staging.

The material finally chosen was pourable *Certite* grout in a mixture specially prepared for us by the manufacturers, SBD Construction Products Ltd., of Rickmansworth. This grout consists of a polyester resin, hardener and filler in the proportion of 1.15:1:1 by volume. The columns to be jointed were 250mm x 250mm in plan, in groups of two and four. (See Fig. 4). The material was poured from the top at A, and air was allowed to escape at B. When the grout appeared at B the hole was plugged and the reinforcing bar was finally grouted to the entrance at A.

The *Certite* mixture has an ultimate strength of approximately 114 N/mm² at an age of four days, and the vast majority of the joints in the structure are stressed to less than 10% of this figure. The percentage of shrinkage is about 0.6% and the specified tolerances in

2 The block has its longitudinal axis almost exactly north-south, and therefore neither face would receive the full influence of the sun. A hysteresis effect would also assist in this case.
3 Shrinkage in the precast units would be much reduced compared with normal in situ concrete.
4 The external support structure outside the cladding would be simply-supported beam units carefully detailed and not tightly butted.
5 There would be stiff in situ construction at the ends of the building tending to restrict movement.
6 The roof would be insulated internally and externally, and if no joints were incorporated, movement would not build up at any point.
7 If no movement joints were incorporated it would save money, maintain the harmony of the column arrangement and simplify the structure to give a maximum of repetition.

It was therefore decided to omit movement joints in the ward block. So far, two summers and two winters have passed since the roof was completed, and the internal heating has been turned on to its full extent. No adverse effects have been observed due to the omission of joints.

Perhaps we tend to be too joint-conscious these days. In the past, long Georgian and Victorian terraces had no movement joints as we know them today, and precast concrete construction is somewhat analogous to stonework in that the structure is composed of a large number of small units allowing it to breathe.

Arrangement of the precast structure
Three lines of columns carry the floors. The outer two lines positioned outside the cladding, are of precast concrete, but the inner line, known as the spine columns, is in situ. They are at centres varying from 6.7m to 7.13m and carry a heavy precast spine beam in simply-supported sections running the full length of the building.

The spine beam carries precast ribs which span transversely to the external edge beams. The latter in turn are supported on the outer lines of precast columns mentioned above. The floor ribs are bridged by shallow slabs, and these complete the precast structure. (See Fig. 3).

Fig. 7 left
Boiler and chimney (Photo: W. Ralston Ltd.)
Fig. 8 below
West elevation of main hospital block
(Photo: W. Ralston Ltd.)

construction avoided cracks due to this cause.

Full-scale joints were made and tested to prove the design. The force involved to test the joint to a suitable factor of safety was too large for the machine available in Glasgow, and this was carried out at UMIST in Manchester. Spalling began when the factor of safety reached 2.6 on the design load, and the tests were discontinued at this point. Models of joints were also tested, but there was some difficulty in scaling down the reinforcement accurately to size in the models, and these tests were not considered entirely satisfactory.

Tests of mixing and pouring the *Certite* were carried out beforehand in a deep freeze with the object of studying joint construction under cold weather conditions. Cubes were cured at temperatures of —5°C and strengths of 105 N/mm² at five days were obtained.

The material remained pourable, and no heating or any frost protection was required. Construction of the joints was also favoured by mild weather conditions during the winter of 1969/70.

Fig. 9
Erection of precast spine beam
(Photo: Keppie Henderson & Partners)

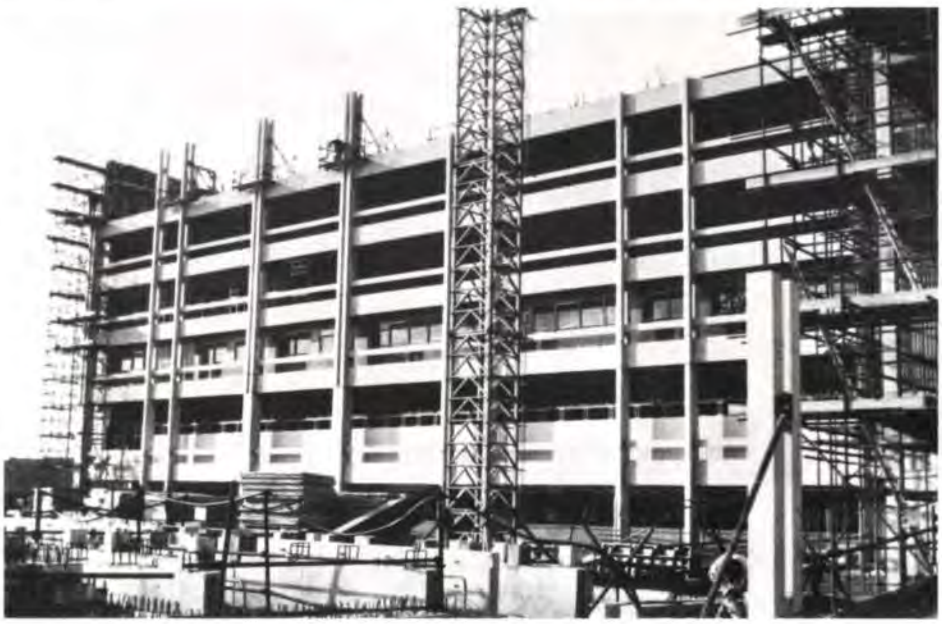


Fig. 10
Precast ward block under construction
(Photo: Keppie Henderson & Partners)

Fig. 11
Operative pouring *Certite* column joint
(Photo: Keppie Henderson & Partners)

Provision for the integration of engineering services

Early assessment of the service requirements led to the choice of plate slab floors throughout the podium areas where the out-patients' department, operating theatres, etc., are located. There are no drops to the column heads and there is a maximum of free unimpeded space for the pipe and duct-work. In the linear-plan ward block the services run longitudinally above the ceiling in the main corridor, branching out as necessary to serve the wards on either side. One side of the corridor is entirely free of vertical structure and there is no impediment to the passage of services. On the other side, however, lie the spine columns and spine beam, the beam forming one side of the service space above ceiling level. To allow the services to branch out on this side the beam is reduced in depth, and the precast floor ribs are carried on individual stools sitting on top of the beam. The space between the stools allows the services to pass above the spine beam but below the floor, and the reinforcement in the ribs is detailed for easy fixing of the services to soffit and sides.

Finishes

Much of the in situ structure has a chipped rib finish. This is applied over the full height of the ward block, lift tower, the stair towers on the gables and elsewhere on the podia, nearly all the work being done by only two trained men. No concrete was chipped until at least seven days old.

Precasting of columns, floor ribs, biscuit slabs, edge beams and balcony handrail units was undertaken by John Laing Concrete Ltd. at Heywood, Manchester. The edge beams and handrail units have an exposed aggregate finish of Skye Marble chips trowelled on to the surface forming the elevation. Stiff brushing was required to free the units from loose chips before they were set in position, otherwise patients might be troubled by the Skye falling about them, particularly from eight storeys above.

Foundations

A conventional borehole investigation in 1964 showed the presence of soft laminated clay above harder clay overlying rock at depths of up to 9.1 m. The rock level in the boreholes over the 10-hectare site varied within limits of ± 3 m.

The heavy column loads from the ward block required large diameter bored piles to rock, and these were put in by Whatlings Ltd. As explained earlier, the horizontal shear forces arising from the wind loads are substantial, and are concentrated at lift and stair shafts at the centre and ends of the ward block. These forces were found to be too great to be conveniently taken on piles, and mass concrete foundations to rock were adopted. The central foundations were the most massive of the three, and at this point the rock level, while within the limits proved by the site investigation, was some 3 m lower than shown by the nearest borehole. The additional excavation and infilling required to overcome this and the resulting side effects, caused delay to the contract in the early stages.

Other column loads were taken on small diameter bored piles by Pressure Piling Ltd., subcontractor to Whatlings. To avoid differential settlement, no footings were placed in the soft clay, and all the columns were piled. The depth to rock was on average 5.8 m and no other difficulties were encountered.

Boiler plant chimney

The boiler plant is situated to the south of the ward block, and to avoid enveloping the wards in smoke, the chimney rises to a height of 64.5 m. It serves three boilers and there is space for three boiler flues, one incinerator flue and two future additions. (See Fig. 5).

An early decision was made by the design team that the flues would be non-structural, and there are floors within the chimney at 4.6 m intervals vertically, which carry the moler concrete rings forming the flues. The floors provide platforms for access and maintenance.

The structural shell of the chimney is I-section in plan. Two walls and the spine are of structural concrete, and the other two are *Galbestos* cladding. The chimney is founded on a slab bearing directly onto rock.

Wind-tunnel tests were commissioned by the heating and ventilating engineering consultants to ensure that the smoke would be efficiently dispersed. These tests determined the height of the chimney and the shape of the flue outlets, but did not affect the structural form.

Acknowledgements

This project was commenced in earnest in 1966 and it has thus taken just over six years to design and build £5.25m of hospital. Members of our staff involved have been Jack Carcas and Arun Save, successively project engineers, and Roy Sigerson and Phil Speakman, successively senior engineers on the site.

Client: Western Regional Hospital Board.
Architects: Keppie Henderson & Partners in association with the Regional Architect, Mr. T. D. W. Astorga
Quantity surveyors: Dansken & Purdie
Electrical consultants: Ramsay & Primrose
H. & V. consultants: Smith, Seymour & Rooley
Main contractor: John Laing Construction Ltd.

The rebuilding of the London Stock Exchange: Market roof steelwork

Jim Hannon

Photo: Sydney W. Newbery

Introduction

The new Stock Exchange Market, known as Phase B of the rebuilding of the London Stock Exchange, is located in the island area bounded by Threadneedle Street, Throgmorton Street and Bartholomew Lane. The market area is sandwiched between the Stock Exchange tower and the Sun Alliance buildings on Bartholomew Lane, with accesses on Threadneedle Street, Throgmorton Street and a court approach off Bartholomew Lane.

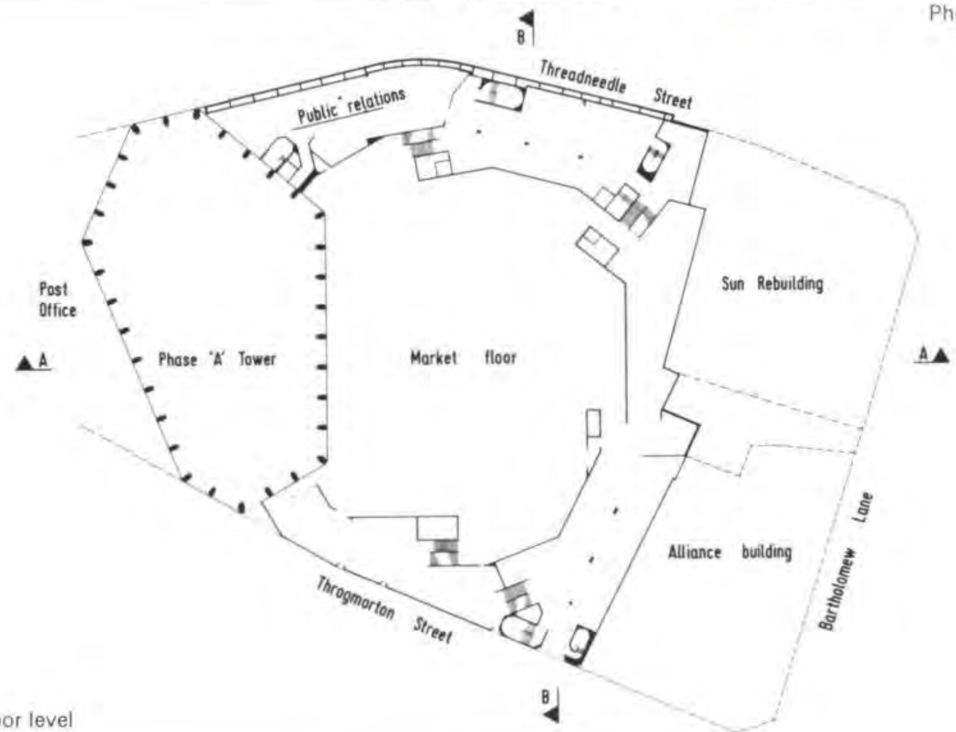


Fig. 1
Plan at market floor level

The market floor provides a clear area of about 1860 m², forming an irregular hexagon on plan. The market area is generally surrounded, apart from the tower side, with toilets and offices (known as boxes) and by a banking house on part of the Threadneedle street frontage. (See Fig. 1).

A public relations section, adjoining the tower on Threadneedle Street, contains a cinema which cantilevers into the market area above the main entrance.

The central area of the market is three storeys high, comprising a basement car park, a service floor and the market. Around this central area, which is about 840 m², is a perimeter strip with reduced ceiling heights varying from 6 to 9 m. Above these perimeter areas are more plantrooms and an access corridor. The ceiling height in the central area is about 16.8 m with a 1.2 m clerestory on five of the six sides. (See Fig. 2).

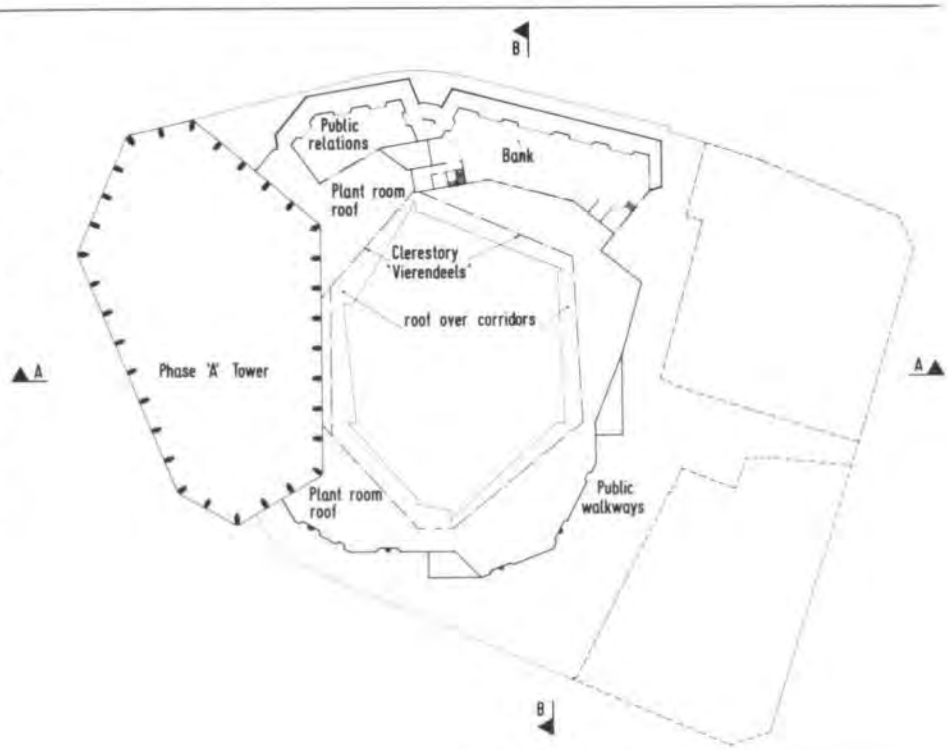


Fig. 2
Plan at clerestory level

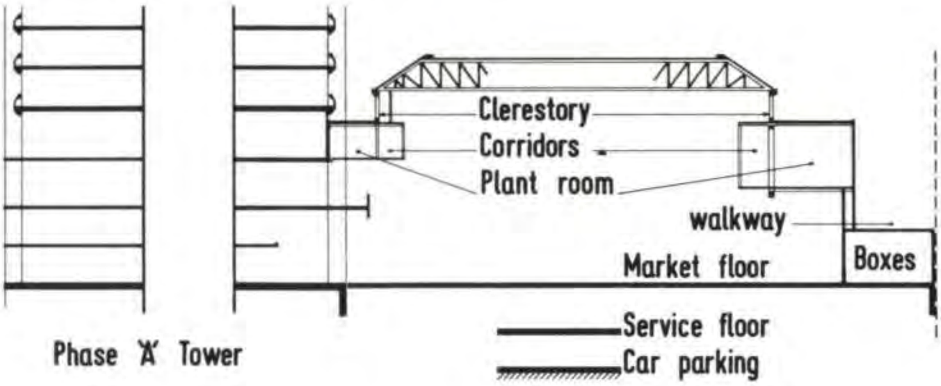


Fig. 3
Section AA through market

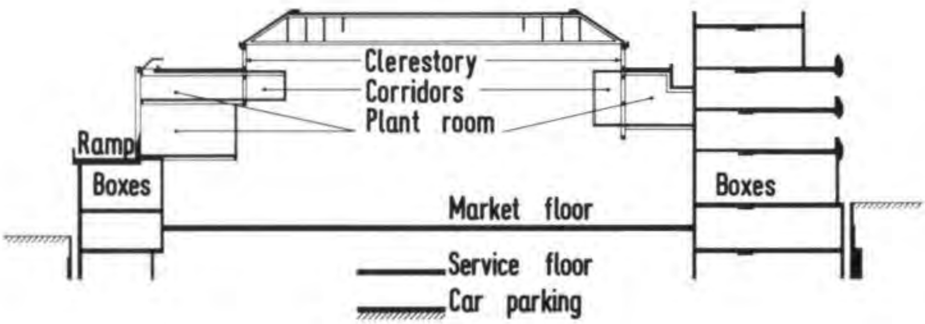


Fig. 4
Section BB through market

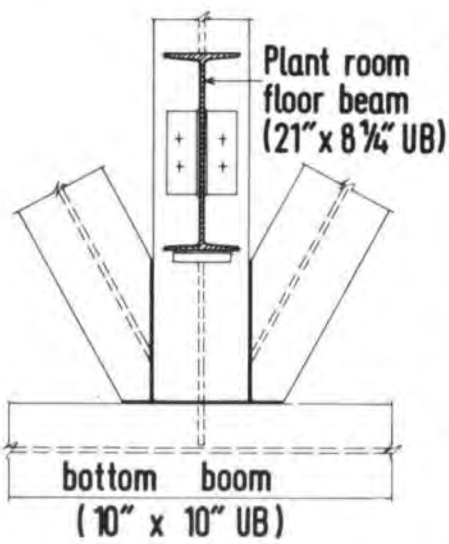


Fig. 5
Typical girder detail

Figs. 3 and 4 show cross sections of the market with the plantrooms, clerestories and the roof structure.

Roof steelwork

The hexagon has vertical structure at four apices in the form of concrete service and lift shafts, the remaining two points of support being preformed pockets in the tower.

These six points are joined together at high level with steel lattice girders, the depths of which are governed by the clerestories at the top booms and the perimeter ceilings at the bottom booms.

Above the lattice girders are the clerestory frames, looking like vierendeel trusses but supported continually by the lattice girders. The roof trusses, which span across the market floor, vary in length to suit the hexagon, the maximum length being about 33.5 m. These trusses at 4.3 m centres are N frames of 2.6 m structural depth supporting, via 203 × 133 mm universal beam purlins at 1.83 m centres, a roof of *Durox* planks, asphalt, and large pebbles set in a cement/sand screed. The end slope of the trusses form a mansard all round the market which is lead sheeted on *Durox* planks.

The bottom booms of the roof trusses support, via 127 × 64 mm channels at 1.83 m centres, a fibrous plaster ceiling incorporating hexagonal fibrous plaster lighting boxes. Above the ornamental ceiling and lights is a plaster fire break ceiling.

The design load for the combined roof and ceiling is 4.79 kN/m² and the weight of the steel in the plantroom and roof is 386 tonnes.

Plantroom lattice girders

The lattice girders, forming the perimeter of the approximate hexagon, support the market roof, via the clerestory frames, on their top booms. The latter also have the roofs of the surrounding plantrooms spanning on to them. The floors of the plantrooms span onto the bottom booms and the access corridor is carried by the roof beams cantilevering over the top boom and supporting hangers at their extremities.

Support for the outer ends of the plantroom roof and floor beams is provided by reinforced concrete walls, generally spanning between the previously mentioned concrete shafts.

Although the market area has been described as a hexagon, it must be made clear that no geometrical symmetry exists in the plan, the angles varying as do the lengths of the sides. However, the span of the lattice girders is generally about 21.3 m and the load carried about 400–500 tonnes. Although the depths varied from about 2.6 m to 6.1 m, so that each was a separate design exercise, an attempt was made to produce some standard details for member connections, splices, etc., and this was achieved to a degree on five of the six girders. Briefly, the 'standard' girder comprised 305 × 305 mm universal column sections of varying weights in Grade 50C steel, joined at the nodes by full strength butt welds. The splices, made necessary mainly by the limitations of the site cranage, were plated joints using general grade 25 mm HSFG (High Strength Friction Grip) bolts, usually through two interfaces. (See Figs. 5 and 6).

T5A

The sixth girder, which should be the subject of a paper itself, is known as T5A.

During the design stage, the bottom level was raised so that a pressed sheet metal duct could pass straight under, rather than be cranked over, the bottom boom. Thus the design depth became 1.98 m. This meant that the standard truss details would not apply, the member property requirements indicating a column core section.

It was decided to use 25 mm plate material of Grade 50C to fabricate a welded box section top boom and a 6 ply bottom boom, the internal members being a single serial size universal column section of varying weight to suit the structural requirements, the joints being butt welds and HSFG bolt groups. (See Fig. 7).

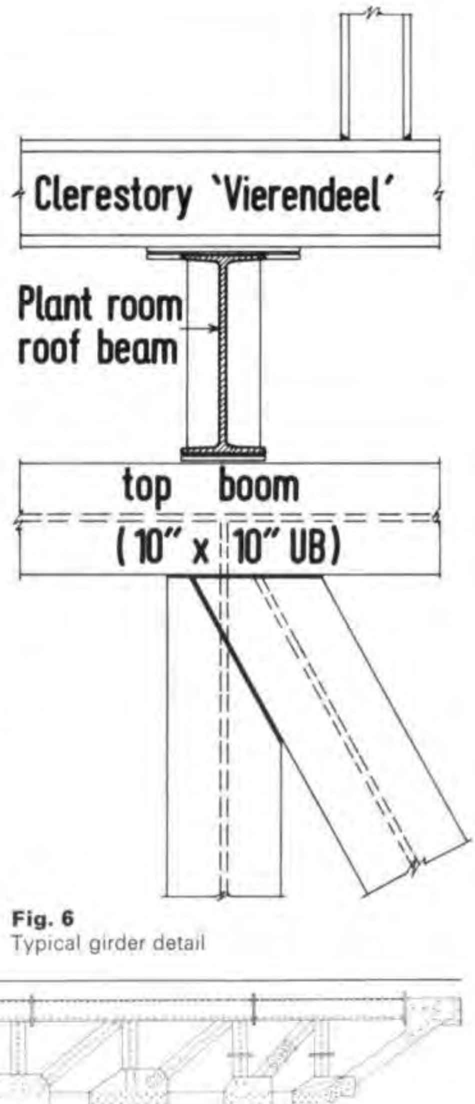


Fig. 6
Typical girder detail

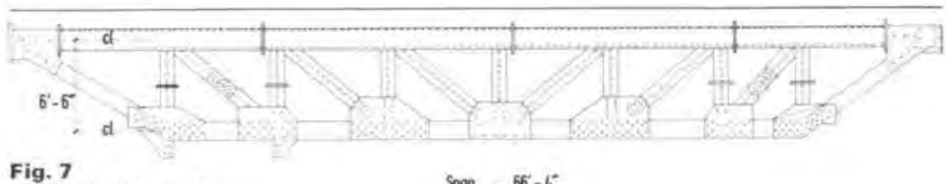


Fig. 7
Elevation of truss T5A

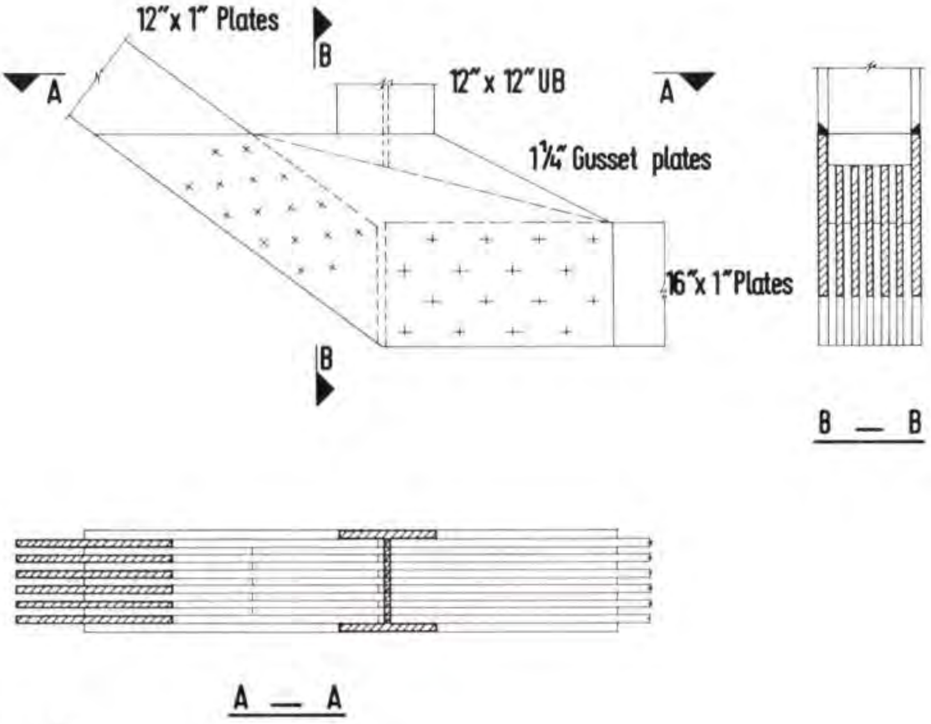


Fig. 8
Bottom tie splice of truss T5A

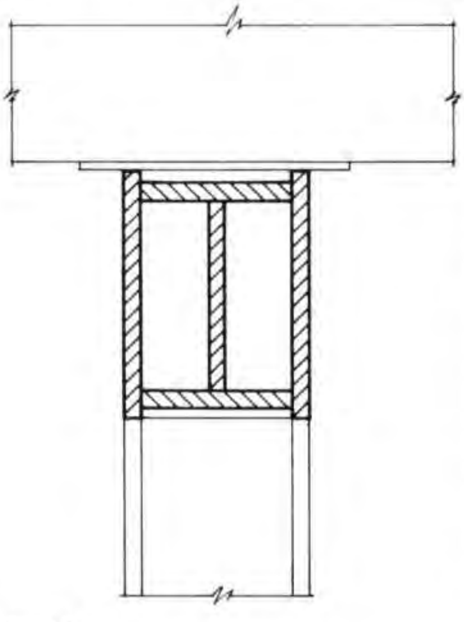


Fig. 9
Section through top boom of T5A

At this stage the details of these heavy lattice girders were discussed at length with the Welding Institute, the British Steel Corporation, the district surveyor, and Sandbergs. The aim of these discussions was to establish welding procedures, non-destructive testing requirements and to eliminate as far as possible the usual errors in weld details which lead to crack formation and inclusions. Having satisfied ourselves on the viability of the welded joints, we carried out a similar but more lengthy exercise on the HSFG bolt groups in the bottom boom of T5A. (See Fig. 8).

Multiple interface friction

The bottom boom comprises six 25 mm thick plates, 406 mm wide, generally with a 25 mm gap between the plates. At splice positions, which were also node points, splice plates were introduced between the main plates. 32 mm gussets, attaching the internal truss members, were placed on either side of the member. The 25 mm diameter HSFG bolts, employed to clamp together all these plates and gussets, thus passed through 12 interfaces and required a grip length of some 343 mm. (See Figs. 9–11).

Some doubts were expressed as to whether the 12 interfaces could be considered to have 12 times the value of a single interface when computing the value of a bolt. To prove one way or another the viability of this joint, tests were instigated to pull multiple plate splices until slip, and then failure, occurred, the test plate sizes and bolt sizes being related to predictable failure loads in both slip in the bolt group and tensile failure of the holed plates.

These tests indicated that the assumption of the equal value of multiple interface friction was justified. In addition, they indicated that, within the limits of proportionality, the gross area of the plate could be considered as effective. However, this last point should be further investigated before embarking on a gross area tensile design in an HSFG bolted connection.

A more important result of these initial tests concerned the inconsistencies found in the bolt tightnesses related to the applied torque. We instigated a further set of tests, using the actual length of bolt to be used on the job, and a set of 11 25 mm thick plates through which the bolt was passed. A method of testing the slip value of any one plate was incorporated into the test which confirmed the previous test results regarding the interface friction values.

The tightness of the bolt, defined as the measured tension in the shank, was equated to an applied measured torque and the rotation of the nut. The test results enabled a specification to be produced for tightening the bolts by the part-turn method using a torque check.

Manufacture

The roof trusses were made at T. C. Jones' works at Treorchy. The trusses were set up on a 33.5 m jig to the specified camber and welded in the assembled position. These trusses of Grade 43A steel were subjected to spot checking for dimensional accuracy and freedom from weld faults, prior to leaving South Wales.

The six lattice girders were manufactured at the United Steel works at Greenwich, South London. All welds in the girders, together with two spreader plate girders which sit in the tower pockets, were tested as manufacture progressed. Few faults were found and generally a high standard of welding was produced.

Two cases of lamellar tearing occurred which necessitated the replacement of the end plates of two of the trusses. However, apart from some signs of panic on site, this did not affect the progress of the job.

Erection

The Redpath Dorman Long branch of the British Steel Corporation was used for the erection of the steel with a programme time of 12 weeks including the erection and dismantling of a 10 tonne guyed derrick and winch.

The erection went well and excellent co-operation was experienced, particularly in the bolt tightening procedures and sequences in the site joints.

A high degree of accuracy in the manufacture contributed to this very complicated structure being completed on programme.

Credits

Architects: Fitzroy, Robinson and Partners.
Main contractor: Trollope and Colls Ltd.
Quantity surveyors: Gardiner and Theobald.
Structural steelwork: T. C. Jones Ltd., United Steel Ltd., and Redpath Dorman Long Ltd.
Testing consultants: Messrs. Sandberg.

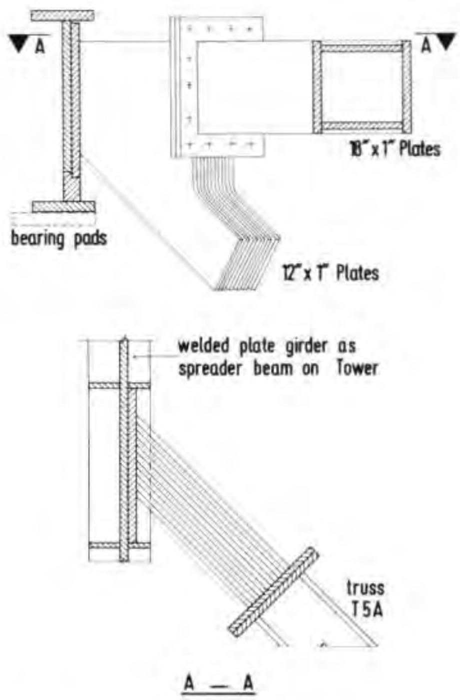


Fig. 10
Truss T5A connection to tower spreader beam

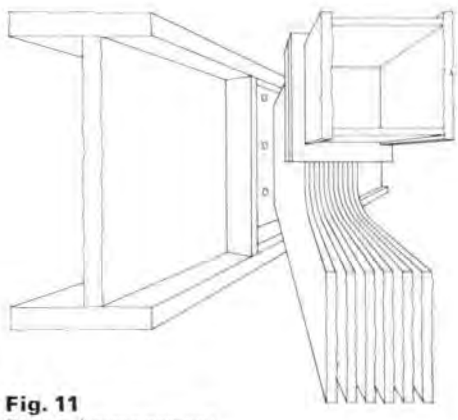
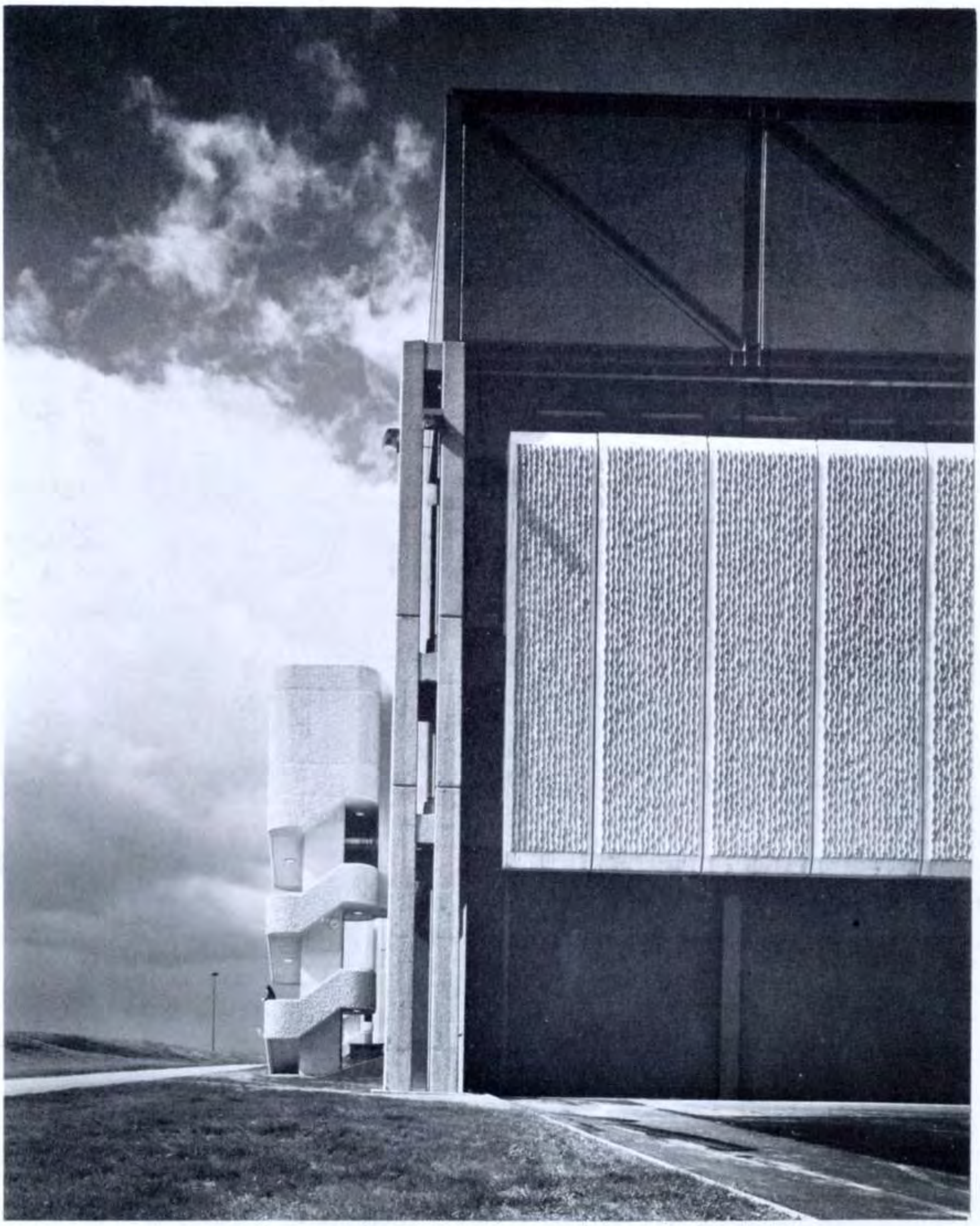


Fig. 11
Pictorial arrangement
of T5A and tower junction



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