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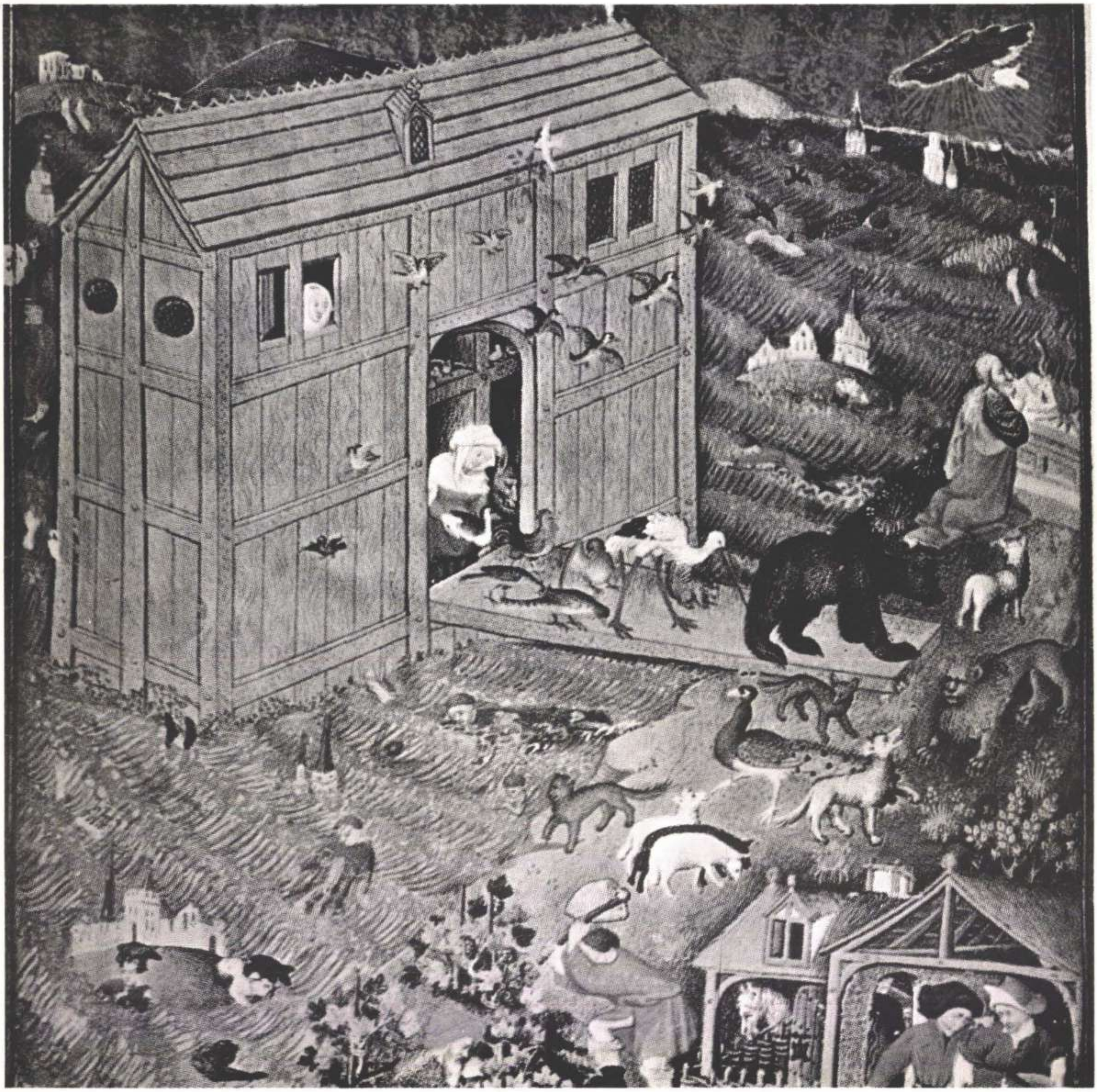

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Front Cover: Detail of *Leaving the Ark* from the *Bedford Book of Hours*. (Reproduced by courtesy of the British Museum)
Back Cover: Stainless steel sculpture by William Pye, South Bank Development Scheme. (Photo: Harry Sowden)

The structural use of stainless steel

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Mike Eatherley
Turlogh O'Brien

Introduction

This paper is a summary of an investigation recently carried out to establish the most suitable materials for the fabrication of a tubular lattice structure which forms part of a design by Arup Associates.

Although related directly to one job's particular set of constraints, the comparisons of materials and manufacturing processes are relevant to other possible uses of stainless steel.

The problem in evaluating the feasibility of constructing the lattice in structural stainless steel was to find an alloy of adequate strength and corrosion resistance that could be readily formed into tubes and could be fabricated or formed to make the 'nodes' and the complete lattice units. Availability of material and of fabrication facilities had to be assured and because of the unusual nature of the work, the expertise of the companies involved in supply and fabrication had to be of a very high order.

The initial performance requirements for the alloy were expressed in terms of corrosion resistance, mechanical properties and weldability. Not all stainless steels can withstand atmospheric exposure without rust staining. Clearly, we required an alloy with total external corrosion resistance which would give a minimum of surface staining. Internal-corrosion resistance was also required.

Mechanical properties in the parent metal, welds and heat-affected zones of the welds were required to be as follows:

0.2% proof stress	309 N/mm² minimum
0.5% proof stress	386 N/mm² minimum
Tensile strength	541 N/mm² minimum
Elongation	15% minimum
Charpy V-notch	27 Joules at −20° C.

The material had to be weldable by the standard processes used for stainless steels such as TIG, MIG or manual metal arc, using readily-available consumables and requiring no preheating or post-welding heat treatment.

The tubular lattice members were required in three diameters, 194 mm, 324 mm and 512 mm. The 194 and 324 mm tubes were required with 9.5 mm wall thicknesses. The 512 mm tubes were required with varying thicknesses from 12.5 mm to 30 mm but with special collar sections of even greater thickness.

Apart from the detailed requirements for fabrication it was a clear principle that the final lattice must feature nodes which were architecturally compatible with the tubes, and also that the surface finish of the completed lattice units should be uniform in texture and colour and free from defects and scratch marks.

Types of structural tube

Wrought tubes

The rolled hollow sections normally used for structural work are made by a rolling and drawing process.

Stainless steel tubes can be formed by the same process. Preliminary investigations revealed that they were available only in a limited range of standard sizes and materials and there would be difficulties in meeting the mechanical property requirements. The large thicknesses required in some places would not be readily available. Supply appeared to be a serious problem with the largest mill closed down due to lack of demand (!) and other mills dislocated by a protracted strike.

Seam-welded tubes

Seam-welded tubes are formed on a press brake machine using indentor tools which match the roundness of pipe required. Stainless steel sheet and plate material is received in the annealed and descaled condition. It is then formed on the press brake and the seam welded by a variety of techniques including TIG and submerged arc. Weld inspection follows and finally the tube is passed through rolls to ensure its roundness is within tolerance.

Tubes are manufactured in accordance with two standards, *ASTM 312* and *ASTM 358*.

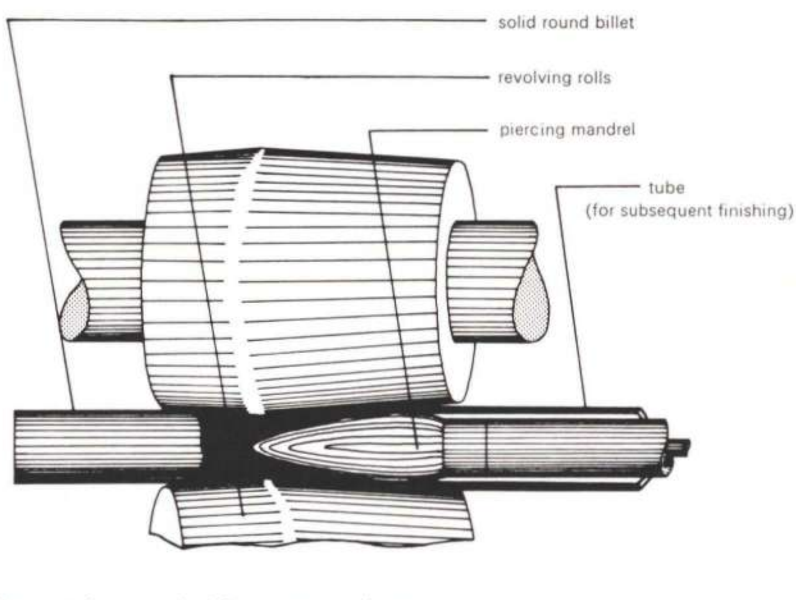

Fig. 1
Seamless tube manufacture: the Mannesmann piercer

Both standards deal with welded tubes and give tolerances, manufacturing materials, rates of testing, and surface finish. *ASTM 358*, the more stringent, was considered to be a suitable standard in terms of tolerances for this application.

However, seam-welded tubes are supplied in the same standard pipe sizes as the wrought tubes. The available sizes closest to those required for the lattice were found to be 168 mm and 219 mm overall diameters with wall thicknesses of 6.35 mm, 7.94 mm and 9.53 mm.

Cast tubes

The manufacturing process consists of pouring molten metal into a hollow, refractory-lined, cylindrical mould spinning about a horizontal axis at speeds sufficient to develop a centrifugal force of 50–150*g*.

This force picks up, distributes, and holds the molten metal in the shape of a hollow cylinder. While the mould is still subject to centrifugal force, directional solidification progresses from the outside surface of the cylinder towards the inside surface against extremely high radial pressures.

Cast tubes are manufactured with a calculated allowance on the internal diameter to allow for dross and porosity which is concentrated there by the casting process.

This layer is generally bored out since the material recovered is of considerable value; in practice this becomes more difficult with increases in the sizes of the tubes.

Following casting the external surfaces of tubes have a layer of adherent refractory wash. This can be removed by blasting, the usual grit used being ferritic steel. Any adherent grit can be removed by a suitable pickling treatment. The resulting surface has the coarse texture characteristic of a cast-steel tube. For normal uses of such tubes, such as in gas reformers, this surface is satisfactory. However, if required, it can be removed by machine to give a bright smooth finish.

Manufacturing tolerances of centrifugally-cast tubes were found to be as follows:

On external diameter	+1.5 mm and −0 mm
On machined bores	+0.0 mm and −1.5 mm
On unmachined bores	+0.0 mm and −5 mm
Straightness Concentricity and thickness	Not more than 2 mm out of straight in a length of 2.5 m
	Not varying by more than 1.5 mm.

Centrifugally-cast tubes were found to be available in most of the sizes required and were found to have the added advantage that wall thicknesses could be readily varied if required.

Alternatives for the nodes

The design featured two basic node types:

(1) Intermediate cruciform inter-sections at half levels in the lattice

(2) The main connections where adjacent frames of the lattice are bolted together.

Both node types could be made either as a direct-welded fabrication from tube or as static castings.

Fabricated nodes

Fabricated nodes would be made out of tubes, sheet and plate, which would be cut, shaped and welded to give the required form. When one considered the structural requirements on these elements and their inherent complexity of form, it became clear that their fabrication would be a most complex task. In addition the number required was such that they could be produced only by a factory of considerable capacity. However, technically they could be fabricated by the specialist companies employed in the production of stainless steel vessels for the nuclear, chemical and food and dairy industries.

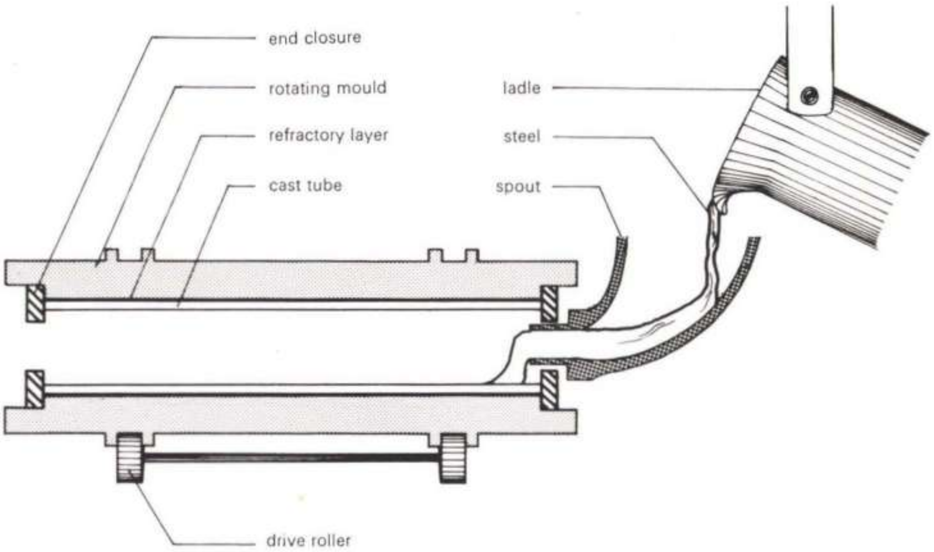

Fig. 2

Centrifugal casting: diagram of principal features

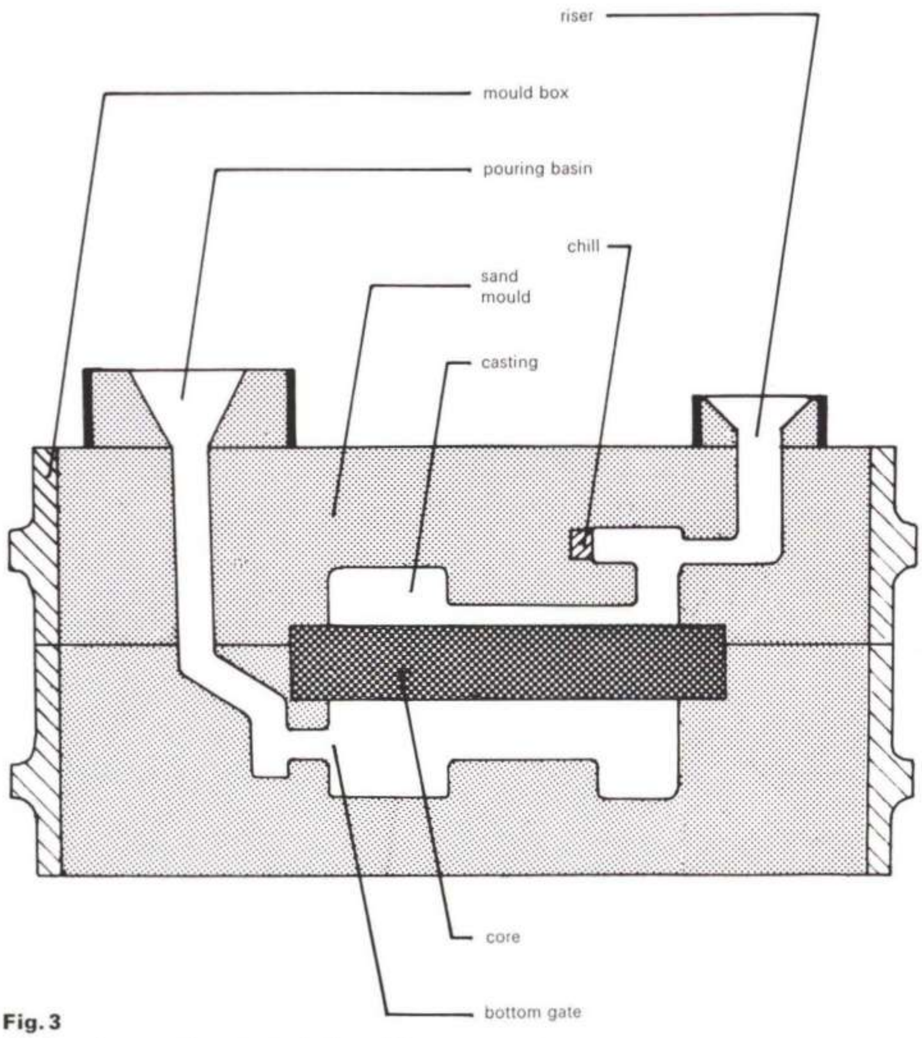

Fig. 3

Sand casting: section of a typical mould arrangement

Cast nodes

The different casting processes were first investigated and it was decided that the normal sand-casting process was the most suitable, both in terms of the sizes involved and the numbers required of each pattern type. Sand castings are, of course, widely used for such items as valve bodies and have been used for massive and complex sections including railway bogey frames weighing up to 50 tonnes.

Design of castings, patterns and cores must be carried out by skilled men, otherwise porosity and blow holes can occur in the final castings.

The principle of the process is that molten metal is poured into a mould constructed out of sand bound together by clay.

The mould is designed to allow the metal to flow into all the interstices and then progressively solidify so that contraction of the cooling metal is made good by continued feeding of liquid metal through the system of 'gates' and risers. When the casting has solidified and cooled, the sand mould is destroyed and the extraneous bits 'fettled' and remelted. The casting is then grit blasted to remove refractory sand and superficial oxide inclusions. Sand castings have a distinctive rough texture.

In making the original sand mould, wooden patterns are used, around which the sand is tamped. These are removed and re-used many times over. Thus, whilst each mould may be used only once, the process is economic.

One disadvantage of sand casting is the lack of precision of the finished casting. For our fabrication, projecting spigots and faying surfaces would require machining to ensure the accuracy required for welding and bolting. One advantage was found to be that, if one combined sand castings and centrifugally-cast tubes, they could be produced by the same company from the same alloy.

Selection of type of stainless steel

There is a wide range of stainless steels to choose from with very significant differences in properties. Fortunately perhaps, the choice for structural uses is limited by mechanical property and corrosion-resistance requirements.

Stainless steels are of three main types, classified according to their metallurgical structures; austenitic, martensitic and ferritic. Alloys of mixed type (duplex structure) are also possible and fall between the two 'pure' types in their properties. The main factor which decides the structure of a particular alloy is its composition.

Martensitic steels

The martensitic steels are generally the lowest-alloyed stainless steels and the least expensive; they have relatively poor corrosion resistance and also have poor ductility. Welding requires pre-heating of the order of 200°C and post-heat treatment of 650°C–700°C. They were consequently considered unsuitable for the lattice. However, one principal advantage is that they achieve very considerable mechanical strength, and their applications are ones which principally exploit this property.

Ferritic steels

The ferritic stainless steels contain either more chromium or less carbon than the martensitic steels. They have better corrosion- and oxidation-resistance than the martensitic steels and are extensively used for chemical plant applications. Totally ferritic stainless steels suffer some embrittlement during welding caused by grain growth, with consequent loss of notch ductility and lack of resistance to crack propagation. Methods of avoiding this include pre-heating the weld-region, thus slowing the rate of cooling the weld. They were not considered suitable for this project.

Austenitic steels

The normal range of stainless steels used in buildings are the austenitic stainless steels based either on 18/8 Cr/Ni or 18/10/3 Cr/Ni/Mo compositions. Such steels have very distinctive characteristics. They are non-magnetic and have high coefficients of thermal expansion. Examples of austenitic stainless steels are the types 316, 315 and 304 to *BS 970; Part 4* which are well-known to many engineers.

In less corrosive atmospheres than London the cheaper non-molybdenum-bearing grades such as 304 have been successfully used architecturally. However, of the austenitic steels, only the 316 and 315 molybdenum-bearing grades are suitable for near-total resistance to corrosion in an industrial atmosphere, and on this requirement these two steels would be appropriate.

Unfortunately the mechanical properties of the ordinary grades of 315 and 316 steels would be inadequate for this application. For wrought- and seam-welded tube, a high-proof grade of 316, designated 316 S66, was found to be available at a projected cost of little more than normal grade 316 material.

Austenitic/ferritic steels

If the chromium content of a 18/8 stainless steel is increased beyond 20%, the fully-austenitic structure reverts to a mixture of austenitic and ferrite. Associated with this change of micro-structure is improved corrosion resistance in certain media and a dramatic increase in the yield point of the steel. There are also welding advantages compared with the fully-austenitic grades.

The duplex austenitic/ferritic stainless steels are very suitable for foundry use, producing outstandingly sound castings combined with good mechanical properties. Casting defects can easily be made good by welding without the needs for post-heat treatment. The duplex steels have thus found numerous applications in the form of cast tubes, particularly for pressure vessel use.

Detailed investigations

It was clear at this stage in the investigation that the tubular structural members could be formed either from seam-welded tube of a high-proof, austenitic, stainless steel, or from a centrifugally-cast, duplex material. The nodes could either be fabricated or cast.

Each alternative combination was examined in respect of corrosion resistance, mechanical properties, weldability, availability of appropriate sizes, delivery programme, appearance and cost. Various surface finishes were examined for each.

The results of these studies are presented below.

Of the high-proof stainless steels, that designated type 316 S66 was the most readily available but even this is not a commonly-used steel. Of the castable duplex stainless steels, a proprietary alloy known as *Paralloy 3FL* was available and appeared to be the most promising. These two materials are the ones examined below.

Analysis and mechanical properties

The analyses of the composition of the two alloys are shown in Table 1 above.

Table 1: Analysis of stainless steels

	Type 316 S66	*Paralloy 3FL*
Carbon	0.07% maximum	0.08% maximum
Silicon	1.0% maximum	2.0% maximum
Manganese	2.0% maximum	1.5% maximum
Chromium	16.5–18.5%	21.0–24.0%
Nickel	10.0–13.0%	4.5–6.5%
Molybdenum	2.5–3.0%	1.75–2.5%
Sulphur	0.04% maximum	0.04% maximum
Phosphorous	0.04% maximum	0.04% maximum
Nitrogen	0.15–0.25%	not specified
Iron	remainder	remainder

The high-proof, stainless steels are classified according to *BS 1501: Part 6*. They differ from the parent grades (*BS 970*) in containing nitrogen additions. This nitrogen is held in solution in the interstices of the metal lattices. It does not cause porosity and is retained regardless of thermal treatment. Thus the mechanical properties are improved with no loss of corrosion-resistance.

The physical properties of the two materials differ slightly, as shown in Table 2. Data for mild steel are included for comparison.

Table 2: Physical properties

	Type 316 S66	Paralloy 3FL	Mild steel
Density (kg/m³)	7920–7980	7700	7930
Specific heat (Joules/kg°C)	486–499	460	501
Thermal conductivity (W/m°C)	12.6–13.8	21.0	35–55
Mean coefficient of thermal expansion ($\times 10^{-6}$ per °C)	15.2–15.8	11.3	11.0

The lower coefficient of thermal expansion of the duplex alloy is advantageous, although it is still higher than normal structural steels. The higher thermal conductivity is also useful in conjunction with the water cooling for fire protection.

The mechanical properties are summarized in Table 3, which includes data for Grade 50C to *BS 4360* for comparison purposes. Grade 316 S16 to *BS 970* is a normal (i.e. non-high-proof) grade of austenitic stainless steel.

Weldability

High-proof stress stainless steels

At one time, welding austenitic stainless steels was considered a difficult proposition. However, with developments in welding processes,

Table 3: Mechanical properties

Grade	0.2% Proof stress (N/mm²)	Tensile strength (N/mm²)	Elongation on 5.65$\sqrt{So}$ (%)	Charpy temp. (°C)	V-notch energy (Joules)
50C to *BS 4360*	356* (minimum)	494–620	18 (minimum)	−5 −15	40.5 (minimum) 27 (minimum)
316 S16 to *BS 970: Part 4*	170 (minimum)	464 (minimum)	40 (minimum)	not specified	
316 S66 to *BS 1501: Part 6*	317 (minimum)	619 (minimum)	35 (minimum)	20 −196	54 27 (minimum)
Paralloy 3FL	309 (minimum)	541 (minimum)	30 (minimum)	10 −50	34 26 (minimum)
	340–480 (typical)	650–780 (typical)	32–44 (typical)	−5 −20	44 34 (typical)

*This value is the yield stress for thicknesses up to 15.9 mm. When yield is difficult to identify *BS 4360* proposes the use of the 0.5% proof stress as equivalent.

consumables and alloys, there are now considered to be few problems with the commonly used commercial alloys.

The weldability of austenitic steels is related to the quantity of delta ferrite in the weld metal. Since nitrogen is an austenitic stabilizer and since this element is the one which gives the high-proof properties, there would be a tendency for it to reduce weld ductility, particularly in welds formed by high heat input processes and with high restraint. The British Steel Corporation state that with manual metal arc welding, there would be no problem. However, with submerged or MIG welding, when used under the most unfavourable circumstances, one may require weld consumables which will increase the amount of ferrite in the weld. A list of recommended consumables is available.

The British Steel Corporation have presented a considerable amount of test data on these steels. For manual metal arc welds on 6 mm-thick plate using consumables to *BS 2926: Grade C** the following properties were obtained for the worst run in each weld.

1.0% proof stress	Tensile strength	Elongation on $5.65\sqrt{So}$	Fracture position
348 N/mm²	668 N/mm²	38%	in parent plate

**BS 2926:1957 Chromium nickel austenitic steel electrodes manual arc welding.*

Impact values for 2 mm Charpy V-notch specimens tested at $-196°C$, welded by manual metal arc from 19 mm welded joints, show recorded values between 34 and 121 Joules.

MIG welds in thinner plate materials show relatively poor notch ductility, due presumably to the consumables used. The implication is that, if MIG is considered as the welding process for the high-proof steels, consumables with extra delta ferrite should be used.

Cast austenitic/ferritic duplex stainless steels

It became clear from discussions with the Welding Institute that the austenitic/ferritic steels would present no inherent problems of weldability. The most serious consideration was that purely ferritic stainless steels do suffer grain growth if welded by high heat-input processes. These steels contain about 30% ferrite and may be prone to the same problem on a reduced scale.

Since grain growth leads to reduced ductility and notch ductility, it is necessary to ensure that we use a weld process and procedure which limits the opportunity for this to occur.

It was agreed that full procedural testing of the welds should be carried out. The criteria established for acceptance levels in these tests were judged to be at least equal to the requirements of a welded carbon/manganese structural steel.

Since *Paralloy 3FL* is a duplex alloy with an austenitic/ferritic structure, it was questioned whether welding this alloy would lead to an increased percentage of ferrite in the heat-affected zone of the weld, and if so, what result this would have on the fracture toughness. It is, of course, an acknowledged fact that the fully-austenitic stainless steels have much higher fracture toughness than structural steels over the range of normal or possible service temperatures. Crack Opening Displacement tests (COD) were initiated on samples machined from 194 mm overall diameter cast tube which had been circumferentially butt-welded using actual consumables for the job. The joint preparation was of the single-bevel type with an integral spigot which provided alignment and backing for the weld. 10 mm × 20 mm test pieces were prepared with a notch sited away from the weld zone. Three specimens of each type were produced for testing. They were carried out at $-20°C$, and the COD measured by means of a clip gauge. The report prepared by the Welding Institute says that: '. . . at $-20°C$, both the pipe material and the heat-affected zone are giving upper shelf values of crack-opening displacement and this indicates that fracture is unlikely to be a problem.

The Welding Institute computed the maximum permissible flaw size. The report concludes: 'For the minimum values of COD, measured in the heat-affected zone, a maximum allowable flaw size of approximately 7.5 mm is determined. This is comparable in size to the plate thickness and indicates that through-thickness flaws of lengths greater than 12.5 mm would cause failure. These flaws are large in relation to thickness containing them and indicate that it is unlikely that a fracture will occur provided that normal standards of fabrication and inspection are employed'.

Surface finishes

Wrought steels

Wrought stainless steel initially has extensive scale covering from the rolling process; this is normally removed by pickling. A matt grey finish results. Such a surface is very easily marked by handling and is not usually considered suitable for architectural applicatons. Most commonly, various machined, etched and polished finishes are given, those with a higher degree of polish generally giving better long-term results.

An alternative to polishing is shot peening (a light form of shot blasting), using non-metallic grits or glass beads. A uniform texture is obtained.

Cast steels

A number of finishes can be given to cast stainless steels. The moulds for centrifugally-cast tubes are lined before casting with a refractory wash. Two types of wash are available: 'cold' and 'hot'. The former gives a uniform, rough-textured surface with a limited proportion of 'pimple' blemishes, which are considered acceptable. The latter gives a regular 'pimpled' surface. Similar effects are not observed on static sand castings. Following casting, shot blasting with ferritic grit or aluminium oxide is employed, followed by pickling in a hydrofluoric acid/nitric acid mixture. These treatments remove the adhering refractory sand and most of the surface inclusions and impurities. The resulting surface is known as the 'as cast' finish. Despite the pickling process, this finish may be subject to limited rust staining due to surface inclusions.

In view of this, improvements in surface finish were investigated. The tubes could be machined, resulting in a slightly striated pattern in a relatively simple operation. Two standards were considered: 125 CLA ('gramophone record') finish or 63 CLA which was then linished (lightly polished with emery cloth) to give a smooth finish. However, to achieve as smooth an effect on cast nodes requires a very lengthy hand scurfing and linishing process. Whilst these finishes are very good on each member, it was found to be difficult to get complete compatibility between tube and node. Greater compatibility could be achieved by subsequently shot blasting. This would also reduce surface brightness, giving a satin-like sheen.

Three media were available for this blasting: glass beads, aluminium oxide and British Industrial Sand. Glass beads provide a smooth satin texture with greatest resistance to finger marking, but at highest cost. Use of sand requires very special conditions in order to comply with Factories Acts. Aluminium oxide is a compromise on appearance and cost grounds.

Corrosion resistance

The corrosion resistance of the molybdenum bearing type 316 stainless steels has been proven through exposure to city atmospheres over many years. The studies of corrosion resistance were therefore primarily concerned with the performance of cast duplex alloys in comparison with type 316 wrought material.

In this application, the corrosion resistance was examined from both the basic durability and surface tarnishing aspects. The former, affecting the inherent suitability of the material for structural use, is examined first.

Atmospheric corrosion

Stainless steels are corrosion-resistant due to the protective surface oxide layer, notably chromic oxide, which forms on the steels when exposed under suitably oxidizing conditions, such as in a moderately pollution-free atmosphere.

Rainwater in industrial or marine atmospheres contains pollutants or dissolved salts which are capable of breaking down these passive films, the most common aggressive ions being sulphate and chloride. The usual form of attack is very localized pitting. This does not normally occur to such an extent that it is of any structural importance but these pits can be visually very deleterious.

There is considerable experience of the influences of individual alloying elements and prevention of pitting is now largely a case of choosing the correct type of stainless steel. For atmospheric exposure in a purely rural environment, alloys of slightly lower relative corrosion resistance have been found to perform adequately. For example, an 18/8 Cr/Ni stainless steel would be considered adequate. An example of this type of steel is 304 S15 to *BS 970: Part 4: 1970.*

For industrial atmospheres, it is generally recommended that a stainless steel containing molybdenum is necessary to promote resistance to pitting.

Within the range of castable austenitic/ferritic stainless steels it was thought that an alloy could be chosen having corrosion resistance both against general corrosion and local pitting of the same order as type 316 materials when exposed to the atmosphere.

The alloy *Paralloy 3FL* conforms to certain American and Swedish standards and is widely used in highly-corrosive situations such as in the paper-pulp industry, where extensive use of sodium chlorite renders other stainless steels prone to pitting and stress corrosion, and in the gas and chemical industries.

This steel has acknowledged resistance to reducing acids where its corrosion resistance is equivalent to type 316 and in oxidizing acids it is better than type 316.

Intercrystalline corrosion

Fully-austenitic wrought stainless steels which have been heated in the region of between 550°C and 850°C are sensitized to a form of corrosion which can occur if exposed to a highly corrosive environment. This is intergranular corrosion. In the heat-affected zone of a weld, there will be a region which has been sensitized to this form of corrosion, and where this is observed, the result is termed 'weld decay'.

The corrosive environment is a major factor here, combined with the carbon content of the steel. It is true to say that the conditions in an industrial atmosphere do not approach those in which it is normal to get intercrystalline corrosion. Where there is a risk of intercrystalline corrosion, special steels can be used to avoid trouble. Normal techniques employed are:

(a) The reduction of carbon content,

(b) The use of stabilising elements, such as niobium or titanium, or

(c) Annealing the stainless steel product.

It is for this reason that wrought stainless steel tubes for use for chemical processing plants are usually supplied in the annealed (fully softened) condition.

The susceptibility of a stainless steel to intercrystalline corrosion or weld-decay is assessed by the following standard tests:

(a) Boiling copper sulphate/sulphuric acid tests as specified in British Standards

(b) Boiling nitric acid test as specified in *ASTM A262 – 64T*, Practice C.

Austenitic/ferritic stainless steels are recognized as being free from this problem.

Stress corrosion

Stress corrosion of austenitic stainless steels occurs when they are operating in tension in highly concentrated chloride environments at temperatures in excess of 60°C. The stress may either be applied or arise from cold working operations. Stress corrosion will be no problem in an urban structure with *Paralloy 3FL* (the duplex structures having a high resistance to stress corrosion).

Internal corrosion

As the structure was to be filled with a solution of potassium carbonate in water for fire protection, this aspect was considered. The potassium carbonate is present in solution, principally as an anti-freeze. However, the carbonate ion is one which promotes the formation of a passive film on stainless steels, and thus acts as a corrosion inhibitor. The solution should contain 25 per cent potassium carbonate and 1 per cent of potassium nitrite as an inhibitor for the mild steel piping within the building.

The water supply likely to be used has been analyzed and found to contain less than the 50 mg/litre of chloride ion which could be present without problems.

Crevice corrosion of welds

If the weld is inadequately de-slagged, the surface profile may be such as to promote the formation of 'crevices' between beads. This is more likely to be the case if the welding is done using manual metal arc rather than TIG. Such defects, although of no structural importance, can be prevented by following a proper descaling procedure.

Surface effects

Although there is ample evidence of the suitability of *Paralloy 3FL* for structural use, investigation has shown that it has not been used in conditions of exterior exposure with an architectural finish. The alloy has therefore been evaluated for its resistance to pitting, rust tarnish and discoloration due to inclusions.

Local pitting due to breakdown in the protective oxide layer is initiated by deposited pollutants from the atmosphere, notably chlorides and sulphates. For stainless steels used for architectural applications, this pitting is barely visible to the naked eye even on polished surfaces. On textured surfaces, the pitting is not visible.

Associated with the pitting on some stainless steels there may be a rust tarnish which forms with time. The appearance of the tarnish is difficult to predict and is not related to inherent corrosion rates of the material. For this reason, the choice of stainless steel for architectural applications is based on experience in use and not purely on corrosion data. The surface finish has an influence, as the rougher the surface, the more tarnishing becomes a problem.

Surface disfiguration or staining may also result from inclusions in the casting, from embedded ferritic grit in the blasting or from steel pick-up from machining operations. These disfigurations result in either rust streaks or general tarnishing. Any procedure for surface finishing should aim to eliminate these problems.

Corrosion tests

In order to investigate atmospheric corrosion further, a series of tests on samples of *Paralloy 3FL* were initiated by Arups, and carried out at the corporate laboratories of the British Steel Corporation (formerly BISRA). Five samples of as-cast pickled tube and five samples of tube machined to *125 CLA* were supplied. Small sheets of type 316 stainless steel were used for comparison. The exposure environments were:

(a) Battersea atmosphere

(b) Shoreham atmosphere

(c) Internal (Battersea office) atmosphere

(d) Continuous salt spray

(e) Immersion in 3 per cent chloride solution.

After 14 days, the machined samples with their bright finish showed about the same dirt pick-up as the '316' control samples and no tendency to discoloration, even in the aggressive environments of tests (d) and (e). Similarly, the as-cast surface showed no general corrosion. However, the as-cast sample in test (d) showed a rust stain from a small surface inclusion.

After two months' exposure, a general surface dulling was noticed, which on examination under a microscope was seen to be due to fine rust spotting. Despite this, the overall appearance, when viewed with the naked eye, was similar to the control sample of type 316, which showed similar rust spots. No pits could be detected.

The initial conclusions reached following examination of cast valve bodies in aggressive chemical environment was that as-cast surfaces were not more prone to rust staining than machined surfaces. However, the presence of surface inclusions inherent in the casting process could be a disturbing factor.

From a theoretical corrosion viewpoint, the smoother the surface the more self-cleaning it will be and the less liable it is to rust spotting or tarnishing. From this point of view, the 63 CLA finish, followed by glass-bead blasting, was to be preferred to either the coarser machining (125 CLA) or to the as-cast surfaces.

Mock-up

Once a decision had been taken to proceed with the building using stainless steel, it was possible to use a full-size mock-up of one bay of the building as a test bed for surface finishes, and the differences in behaviour could be observed through one winter, spring and summer. For corrosion the winter months are the worst, with high humidity and high levels of atmospheric pollutants. From the observations made, it is clear that the maxim about the smoother the surface the less corrosion is true, at least in this case. Another observation which may come as some surprise, is that it is the sheltered surfaces on stainless steel which may show some rusting. This is because the rain does not wash them and spotting occurs behind the clinging dirt. Even so, the observations suggest that, provided the structure is washed down once or perhaps twice a year, no readily visible spotting will occur.

It does appear, however, that *Paralloy 3FL*, whilst having excellent corrosion-resistance, is in fact slightly inferior to type 316 and is closer to the lower molybdenum bearing type 315 material.

Conclusions

From these investigations there were four alternative solutions:

	Tube	Node
A	Seam welded 316 S66	Direct fabrication
B	Seam welded 316 S66	Static casting *Paralloy 3FL*
C	Centrifugally cast *Paralloy 3FL*	Direct fabrication
D	Centrifugally cast *Paralloy 3FL*	Static casting *Paralloy 3FL*

Each alternative was found to be satisfactory as regards corrosion-resistance, mechanical properties, weldability and delivery programme.

A and B had a disadvantage in that seam-welded tubes for the diagonal members were not available in the preferred 194 mm overall diameter size.

Cast nodes were favoured architecturally over fabricated intersections with a preference for textural compatibility of tube and node.

A textured shot-blasted finish was preferred architecturally to the smooth machined surface, which was only practical with fabricated nodes; however, a smooth surface is less susceptible to dirt collection and to tarnishing and it is easier to clean.

Cost studies showed that the as-cast solution was cheapest.

This investigation has shown that alongside the general upsurge of interest in the structural use of steel castings, cast stainless steel can be considered for structural and semi-structural applications.

Construction of the new King's Reach river wall

John Crouch
John Tyrrell

The southern bank of the River Thames in central London was first developed haphazardly for warehousing and wharfage some 200 years ago, and since that time the area has remained more or less as it was initially laid out.

However, with the current trend for moving the shipping activity down to the lower reaches of the river, much redevelopment is now either in progress or is planned to take place during the next few years.

One such project, immediately upstream from Blackfriars Bridge, is known as King's Reach Development, an extensive scheme consisting of an hotel, luxury flats, offices, shops, car parking facilities and various pedestrian amenities.

The Greater London Council also have plans for developing parts of the South Bank and they intend having a riverside walk, 7.5 m wide, along the whole length of the river between Waterloo Bridge and Tower Bridge. It is to be very similar in appearance to that now existing at the South Bank Development adjacent to the Festival Hall.

The Port of London Authority have proposals for improving the contour of the river wall along certain stretches so that a clean smooth line is obtained in place of the present indented one.

Hence, as the river frontage of the King's Reach Development is a scheduled improvement line, this realignment has been incorporated to considerable advantage within the development. The proposed improvement has meant the construction of a new river wall in the bed of the river some 200 m long and 15 m away from the existing masonry wall.

An area of approximately 3000 m² of land was therefore reclaimed from the river which quite naturally broadened the scope of the development, making it possible for a 50,000 m² hotel and a 10-storey block of 87 luxury flats to be accommodated on the enlarged plot size.

The existing river wall

The existing river wall is of masonry construction varying in thickness up to 2 m in parts. It rises 4 to 5 m above the bed of the river and penetrates nearly 2 m into it, but, as the material at that level is unsuitable for taking any load, the base of the wall is carried upon timber cross bearers supported by timber piles driven into the gravel layer lower down.

The top of the wall was constructed to about + 5.41 m OD (minimum) so that it complied with the original flood defence level requirements.

Much of the wall is about 200 years old, although portions of it have been renewed from time to time. Now it is in very poor condition with numerous large cracks and would almost certainly have been condemned by the Flood Defence Office of the GLC as a water-retaining structure had a new wall not been built to replace it.

Soil conditions

Prior to its development in the early part of the 18th century, this area was part of the marshes alongside the River Thames. The geological succession is therefore typical for this kind of situation, with about 2 m of soft grey silty clay overlying a layer of sandy gravel, also about 2–3 m thick. The London Clay starts at − 4 m OD and extends down to approximately − 36 m OD where it meets the Woolwich and Reading Beds.

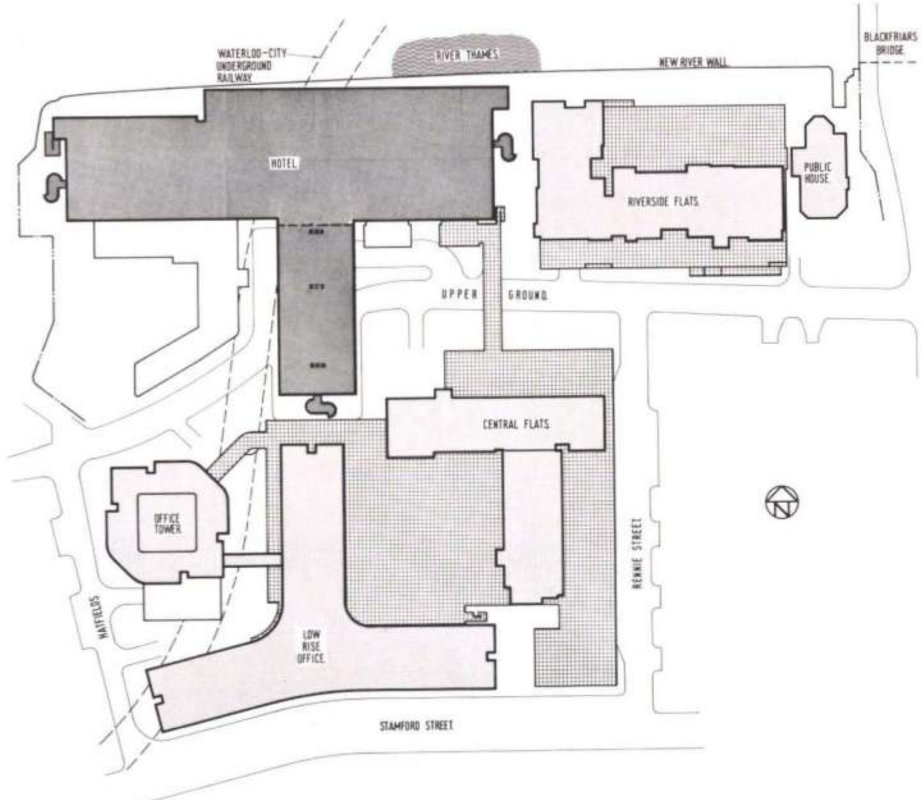

Fig. 1
Site plan

The site investigation carried out by George Wimpey and Co. Ltd. for the entire development included four boreholes (up to 25 m deep) put down in the river bed along the line of the new river wall, the level of the river bed along this line being approximately 0.75 m OD.

Design considerations

The overriding factor was speed of construction since it was intended that the hotel should qualify for a grant under the *Development of Tourism Act 1969*.

It was therefore decided to use a steel sheet pile wall, the finished appearance that the GLC demanded being achieved by cladding it with precast units with a simulated-granite finish. These would have to be fixed at a later date and under a separate contract.

Because of the long delivery period required for the steelwork at that time and urgency of the operation, it was necessary that a provisional order be placed by the client with the steel supplier, prior to the appointment of any contractor. It would have been preferable for Frodingham Sections to have been used, because of their superior individual section properties, but due to problems of delivery which would have caused further delay to the start of construction, Larssen Sections were employed.

The possible chance of meeting extensive obstructions and other forms of delay during construction in this commercially-important stretch of the River Thames was considerable and therefore a large contingency sum was allowed for within the contract documents.

Tidal data for this stretch of the river were assumed to be maximum and minimum tidal variations of 7.6 m and 5.3 m respectively, and maximum tide level of + 6 m OD.

Structure

Although the same type of sheet-pile section, Larssen 3/20, has been used throughout, the support systems used to stabilize it vary to suit the differing requirements of both the hotel and flats.

(a) *Frontage to the riverside flats*

The scheme selected was a conventional

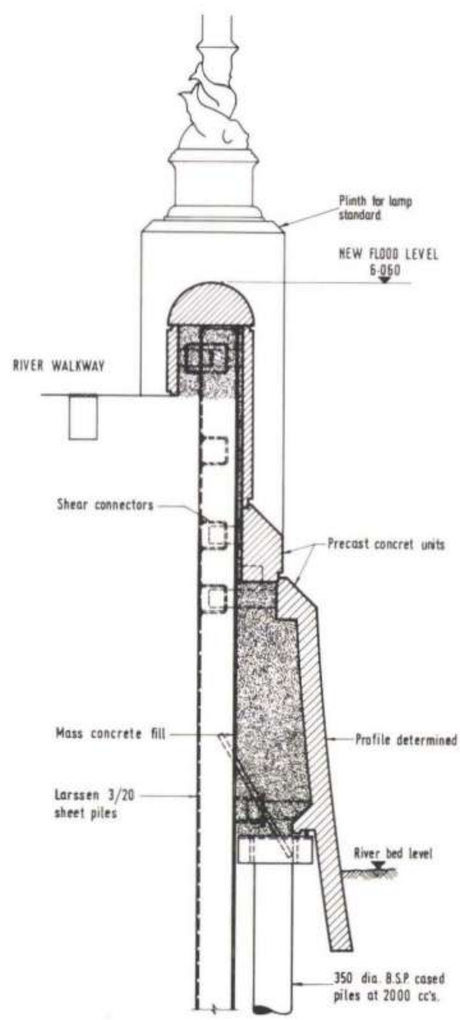

Fig. 2
Section through cladding

anchored sheet-pile wall, stabilized by a relieving platform constructed on the landward side of the new wall. Vertical and horizontal support were provided by vertical and raking British Steel Piling cased piles (350 mm diameter) driven into the London Clay.

This form of construction was chosen because the basement of the riverside flats was not being incorporated into the reclaimed land and also due to the inadequacy of the existing wall to resist long-term, concentrated, horizontal forces.

Two important factors arise with this form of construction. Firstly, the structure is designed to accommodate both the temporary and the permanent conditions. Secondly, due to the presence of the relieving platform construction and the use of clinker backfill, the stability of the wall is greatly enhanced, particularly during low water, because the horizontal forces that the wall has to sustain are minimized. Sulphate-resisting cement had to be specified, however, because the clinker, being combustion waste, was found to have a relatively high sulphur content.

Tidal flaps were incorporated to reduce any head differential between tides.

(b) *Frontage to the hotel*

As the basement was to be built partly on reclaimed land, it was necessary for the thickness of the river-wall construction to be kept to a minimum. A relieving platform was therefore not acceptable and use had to be made of the permanent-basement construction.

This does, of course, present certain constructional problems whilst the new basement is being built. Initially it had been proposed to provide some temporary propping between the sheet-pile wall and the existing river wall but a survey of the latter indicated that it would be unwise to subject it to any concentrated horizontal loads without fairly extensive remedial work having to be undertaken. The propping scheme was therefore rejected in favour of using a rubble berm, 3 m high, which was economical and was found to adequately stabilize the wall in the period prior to the basement construction.

Furthermore, it had distinct advantages over the propping scheme, as it enabled construction to the main basement to proceed unimpeded by the presence of the existing wall.

Weepholes were provided to permit water to flow freely in and out of the coffer dam during construction.

The twin tunnels of the Waterloo and City Railway (which pass under the site) were crossed by the line of the sheet-pile wall, but, as the clearance between the toe of the sheet piles and the crowns of the tunnels was 8 m, which was much greater than the minimum clearance of 3 m required by British Rail, it therefore had no effect upon the construction.

The other three sides of the hotel basement were formed by a diaphragm wall, short return walls of sheet piling being used to join them up to the new river wall.

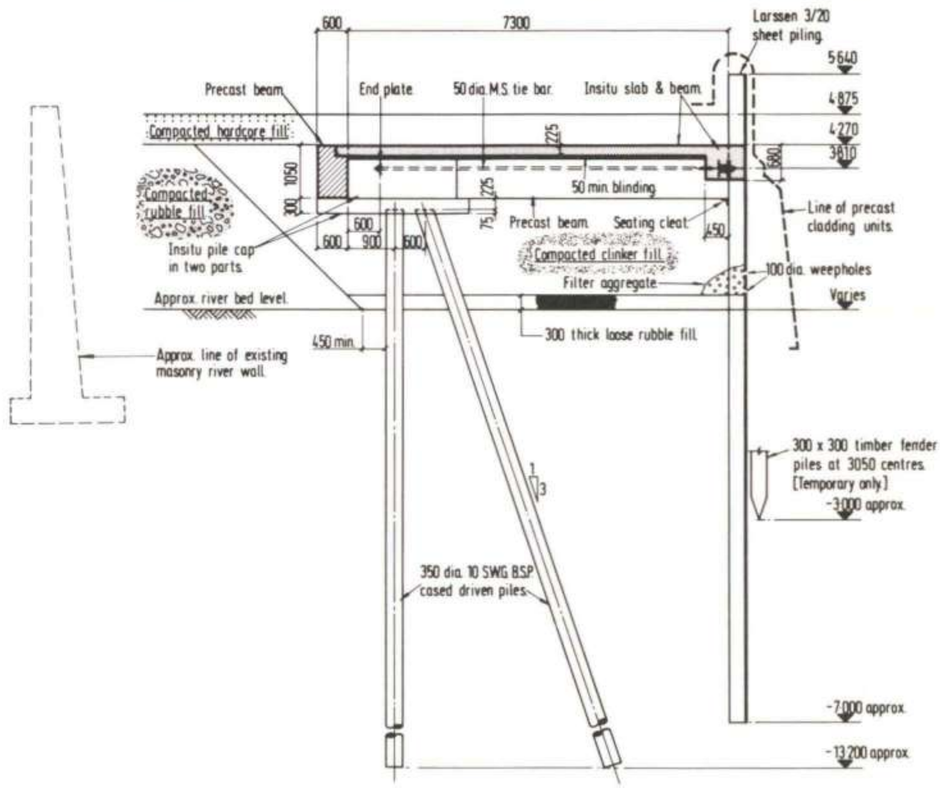

Fig. 3

Typical section through relieving platform structure

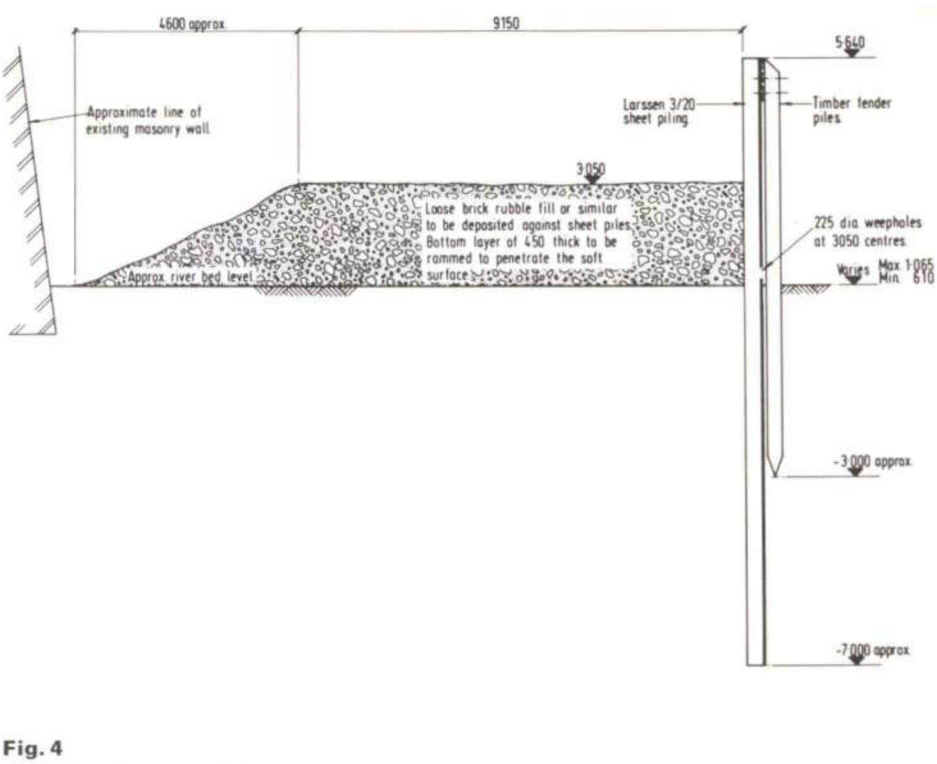

Fig. 4

Typical section through berm

Construction

The contract for the sheet piling and the ancillary works was carried out by Peter Lind & Co. Ltd. under the ICE conditions of contract. Starting in September 1970, it was completed in six months.

The cladding contract has also been let to the same contractor and work started during September 1973.

Installation of the sheet piles took place from the river by means of a barge-mounted derrick, driving being carried out without difficulty through the gravel and about 3 m into the clay. Although the sheet-pile wall and the British Steel Piling cased piles were driven into the river bed beyond any existing building line, some concern had been expressed over the possibility of striking unknown obstructions which might have been encountered, especially when the return walls were driven. A grillage of timber members was known to exist as foundations for the old river wall, but, as it happened, few problems were experienced when breaking through this area to link up the sheet-pile wall with the diaphragm wall.

The sheet piles forming the returns were temporarily omitted to enable the piles and precast reinforced beams of the relieving platform to be installed while the water level within the coffer dam was still tidal. This condition was desirable since the wall would not have been stable under an excessive head of water without extensive temporary works. Similar precautions had to be applied to the hotel basement area until the berm had been constructed.

When the hotel coffer dam had been closed, it was found that the head of water inside it was still subject to tidal effects, due to leakage through the clutches of the sheet piles. Even though this leakage was marginal, it had to be stopped before the probationary period, which was necessary to establish that the new wall would act as a satisfactory flood defence, could begin.

Several methods were considered for making a satisfactory seal, but *PC4* caulking (an asbestos cement rope), welding and *Plastiseal* sealing strip appeared to be most promising. The problem was complicated by the effects of the tides, the flow of the river in both directions, and the wave action caused by the passing of boats.

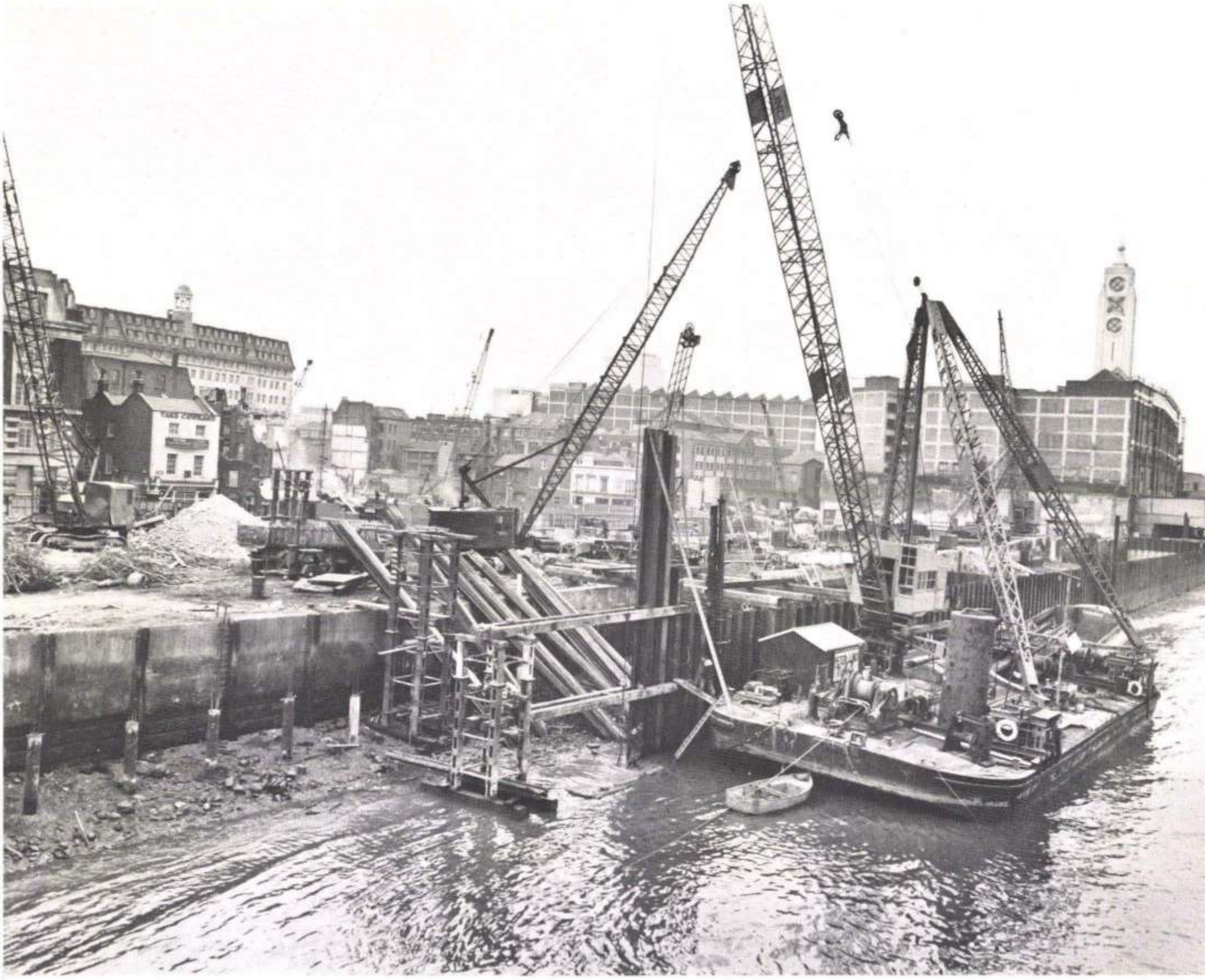

Fig. 5

General view of site.
Installation of sheet piles
using a barge mounted rig.

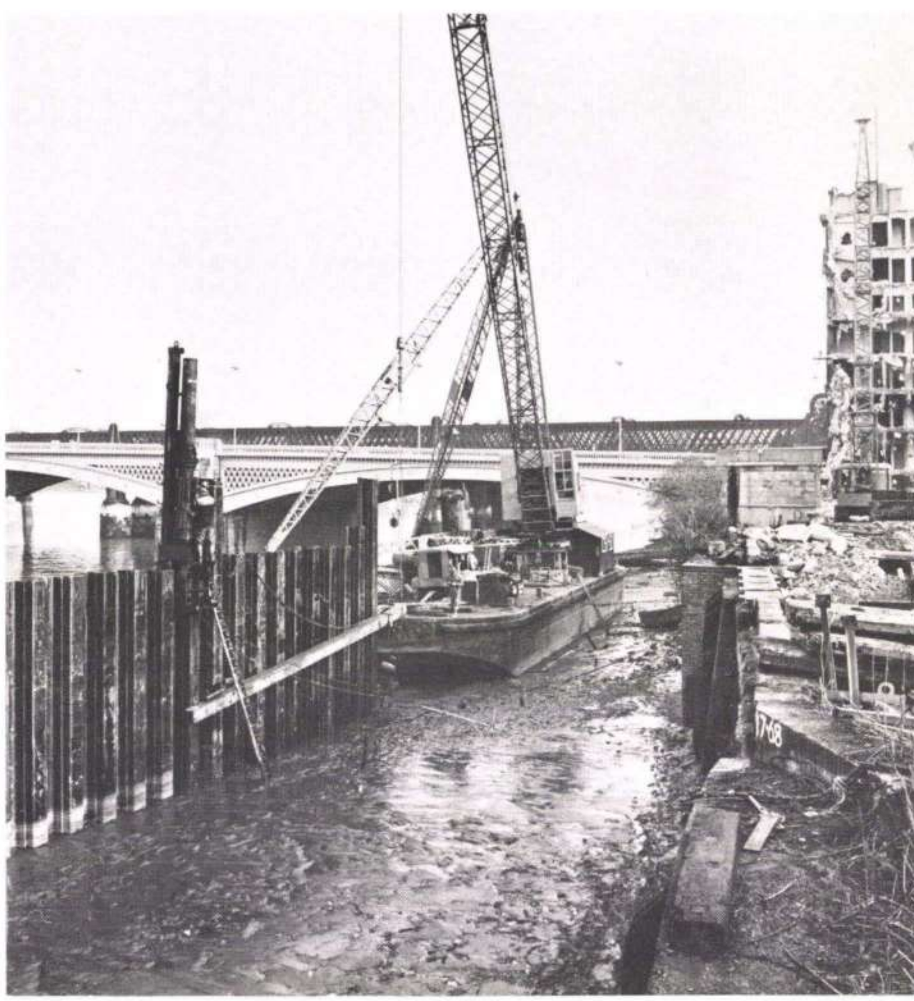

Fig. 6

Installation of sheet piling
by barge mounted rig.
Blackfriars Bridge in background.

Plastiseal was the most efficient but the cost was comparatively high. Welding in wet conditions proved to be difficult to carry out (cleaning was a problem), was expensive and was not recommended. Thus *PC4*, being cheap and fairly easy to apply was used and now after about two years is still performing satisfactorily, although requiring occasional maintenance.

Timber fender piles, 300 mm square, were driven at 3 m centres as temporary protection to the sheet-pile wall until the precast cladding had been fixed. The sheet piles were protected by high-build bitumen, two coats being brush-applied before driving and two coats after, giving a minimum total thickness of 5 mm.

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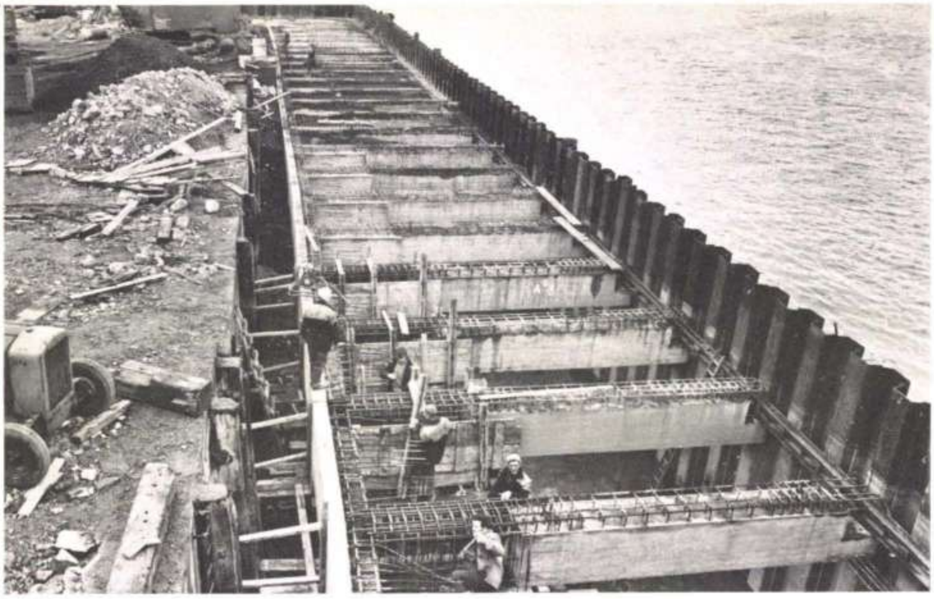

Costs

Based upon the actual costs for the job (1970) an apportionment for the two sections shows:

Construction of a new sheet-pile wall together with the relieving platform (flats area) approximately £1000 per metre run;

Construction of a new sheet-pile wall with only temporary support (hotel area with berm) approximately £500 per metre run.

The final costs for cladding the wall with precast units are not yet available, but based upon the tender rates (1973) the cost is expected to be about £900 per metre run of walling.

These figures for cost can, of course, only be indicative and must therefore not be used in isolation. The very nature of this type of work demands that much consideration be given to the unknown cost factors that come within the areas of preliminaries and preambles – removal of landing stages, obstructions and other hazards, maintenance of flood defences, daywork items, and so on. Luckily, the provisional sum of £50,000 set aside on this job was not needed.

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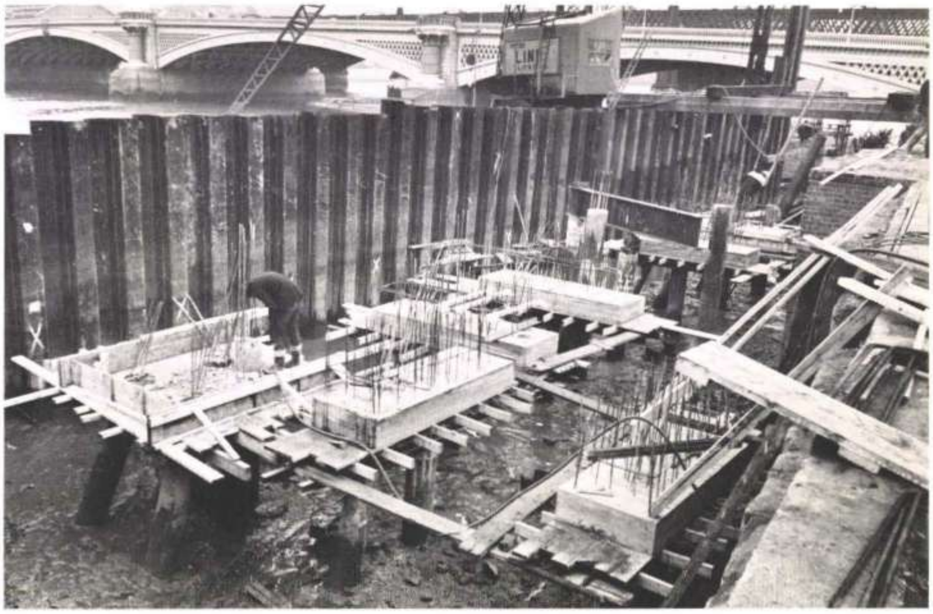


Fig. 7

Precast beams to relieving platform

Fig. 8

Cased piles and pile caps to relieving platform

Fig. 9

Western end before installation of temporary rubble berm

All photographs in this article are by Sydney W. Newbery

9


A concrete platform for the northern North Sea

Tom Ridley

This paper was given at the Offshore Scotland Conference, held at Aberdeen in March 1973. In a future issue of The Arup Journal *we hope to publish a more detailed account of Arups' involvement in the ARK project.*

Introduction

In choosing the theme for this conference to be 'Oil from deep water' the organizers are clearly highlighting the significant new feature of recent offshore oil developments. The deep waters we are considering are furthermore linked to 'Offshore Scotland' where a great many new situations have arisen to confront the oil companies with fresh challenges. These situations are concerned not only with the extremely difficult environmental character of the northern North Sea, but the whole 'state of the art' of the offshore engineering industry.

To enter into this area of development one must approach with a definite sense of pioneering, where there are virtually no fixed attitudes or solutions to given problems (assuming that the problem itself can be reasonably stated). This paper deals with one such approach to the design of offshore fixed platform structures. Although such structures have been constructed during the last 25 years or so, there is a recognized need for new ideas to suit the new circumstances.

The evolution of a design for a fixed offshore platform must first of all concern itself with the way such platforms are used in oilfield development. Some of the features of the oil reservoirs already found in the North Sea suggest that changes in the methods of drilling, production, storage and transport will significantly change the traditional idea of a deck, jacket and piled steel structure.

It is probably true to say that the use of total subsea development could be the most important long-term change to be expected. The immediate future suggests the need for much larger platform areas with drilling and production equipment concentrated on as few platforms as possible consistent with economic recovery from the oil reservoir. The ease of installation of these platforms and their subsequent safety and maintenance will greatly influence the commercial viability of the oilfield.

Platform considerations

From the above general comments it is clear that before design of a fixed platform structure can be started, it is necessary to carry out adequate planning to enable a workable and economic solution to be offered for a given function. This planning must include evaluating the criteria on which the success of the design will be judged. A summary of the factors to be considered, and the results of recent work to establish the criteria, are as follows:

Table 1: Summary of environmental criteria for northern North Sea

Location (latitude)	Design wave (metres)	Max. wind gusts (knots)	Hourly mean wind speed (knots)	Depth (metres)
56°	24.4	90	62	75
57°	25.9	97	62	90
58°	27.4	100	66	105
59°	29	102	66	120
60°	30.5	105	70	135
61°	33.5	108	70	150

Type

The platforms needed are to be used for any combination of development drilling, production, storage, materials handling and living quarters. The desirable layout of these functions can give an area for the supporting structure of 1900/2800 m² and overall applied loading of up to 20,000 tonnes.

Locations

For the northern North Sea, UK locations can be considered as from 56° to almost 62° latitude and 6° West to 2° East longitude.

Depths

For the locations in mind, depths vary from approximately 75–180 m.

Existing designs

Basically steel deck, jacket and piled structures of a total weight varying from approximately 9,000 tonnes in 75 m depth, up to 20,000 tonnes in 180 m depth. Costs are very roughly between £500/£900 per tonne installed on location for this depth range, i.e. £4.5m. to £18m. per platform.

Construction

Steel structures use onshore fabricating yards which need temporary flotation devices to transfer the jacket to a vertical position before sinking with subsequent pile driving and deck installation by crane barges.

Programme

Delivery time for existing designs suggest periods for construction and installation of approximately nine months to 24 months can be allowed for the extreme 75 m/180 m depth range considered.

Environmental conditions

Table 1 summarizes basic criteria being used for platform designs in the northern North Sea. In addition, each location presents its unique problems from the geotechnical point of view.

Regulations

Compliance is necessary with Department of Trade and Industry Construction and Survey regulations for *Mineral Workings (Offshore Installations) Act 1971*. At present these deal mainly with designs using steelwork structures.

Engineering design

In taking account of the platform considerations it can be deduced that any design must fulfil the following order of priorities:

Safety in use

Adequacy for operational purpose

Certainty of completion on time

Speed of construction

Minimum maintenance

Least cost in place.

The extreme environmental conditions to which the platform structure is exposed create a unique problem for both the completed structure and constructional hazards to be catered for in design. Different concepts to date have catered for the conditions by:

Avoidance (sub-surface ideas)

Yielding (buoyant and semi-buoyant)

Resistance (rigid transfer to seabed).

Avoidance by subsurface ideas brings about a much greater change in operational working methods than could be justified in the depth range being considered in this paper. Yielding by buoyancy or semi-buoyancy has attractive possibilities but is considered to require long-term development work before its suitability could be confirmed. The established method of providing resistance by the transfer of forces to the seabed is the most obvious concept for immediate interest. This indicates a structural form which provides the minimum sectional size to reduce the wave action and required seabed anchorage resistance.

For the northern North Sea there is a major influence on design arising from the constructional problems. These impose severe constraints in the availability of good weather on location, and the reduction of as many erection operations at sea as possible is most desirable. Thus the elimination of piling for the essential seabed anchorage of a platform design is to be welcomed, if alternative transfer of forces can be found. A gravity-type structure offers this possibility, where its resistance is achieved by the stability of its own weight acting on the seabed.

From an engineering design point of view this leads logically to the idea of using concrete to provide its quality of very economic weight/strength property to offer a valid platform design. The features of temporary flotation and offshore location are the critical innovative aspects to be dealt with for any valid concept based on concrete construction.

Concrete platform concepts

To utilize the most speedy form of concrete construction for platform structures based on proven experience, a form of wall structure built completely onshore vertically will give maximum scope for rapid execution. Such structures are seen to offer very economic designs as 'boxes' in the form of tanks, caissons, high buildings, etc. and as 'towers' in the form of masts, chimneys, lighthouses, etc.

Large box structures would appear to offer an advantage in providing for easy flotation. With their large water-plane area they give apparent stability during towing and sinking. However, due to the depth ranges being considered up to 180 m, their difficulty in catering for this height introduces major problems. These arise when the structure is floating in shallow draught with the need to add vertical height beyond the vertical limit of economic box construction. If the additional structure is to be added prior to towing, this requires heavy ballasting to maintain overall stability in this condition.

High tower structures offer the most obvious way of achieving the necessary range of vertical dimensions for platform structures. Their construction by slip-form techniques has been widely demonstrated to give very speedy execution to much greater heights than presently contemplated. However, there is little experience with them as floating structures and to establish their stability in this condition they require submergence in deep water due to the relatively small water-plane area. If this can be achieved they give very good stability characteristics, and towing is required with the structure in this deeply submerged condition.

Both the box and tower concepts thus appear to offer valid alternatives for offshore platform structures. The necessary weight of such structures is derived from the dimensions they are given to suit a particular location and environmental conditions. Using a depth of 144.2 m and extreme wave of 30.5 m with 15-second period and 200 km/h wind speeds, each alternative is examined in Figs. 1 and 2.

In Fig. 1 a box structure is shown with a vertical tower to achieve the required height and platform support. With the dimensions stated, it contains 160,000 tonnes of reinforced concrete and a further 140,000 tonnes of rock ballast for stability during flotation. It floats in a depth of about 33.5 m and will thus need a construction site offering water depths of 20/25 fathoms. In Fig. 2 a tower structure is shown which provides a concrete shaft for its full height with foundation slab supported from radial ribs. It contains 90,000 tonnes of reinforced concrete and is given stability by water ballasting in a floating depth of approximately 105 m. This requires a construction site offering water depths of 60/70 fathoms and similar depths during towing to location.

As can be seen from their dimensions each of these alternatives contain a large volume with-

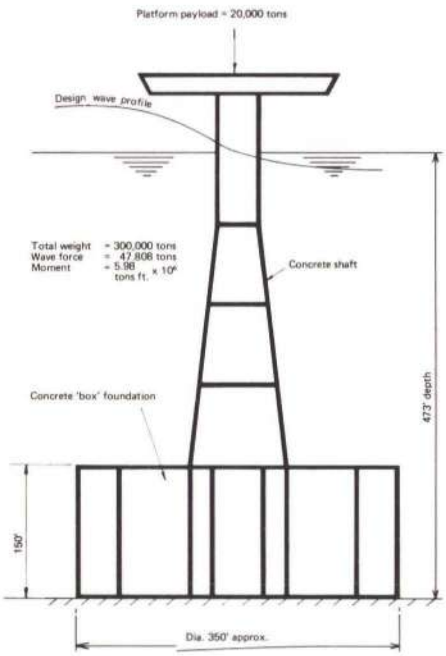


Fig. 1
Box structure

in the structure which is filled with water ballast on submergence. This volume offers the possibility of also catering for crude oil storage on location, which introduces a form of 'bonus' for the oil company! Depending on the circumstances, it may be this feature alone which will determine which alternative concept of platform structure is most appropriate. In the case of the box structure, storage of approximately 1.6 m. barrels would be offered. The tower structure offers a smaller volume of approximately 300,000 barrels. The negative buoyancy of crude oil stored in a submerged structure must be allowed for as must the additional operational requirements to handle the oil in storage and transfer to tanker loading.

The use of concrete structures for offshore locations gives the promise of long life with little or no maintenance. Durability of concrete is a matter of concern for safety conditions, and this arises particularly with the possible risk of corrosion of steel reinforcement in the structure. This problem has been well studied by both the cement and concrete industries and durability of concrete in exposed marine conditions has been reasonably well established. By careful design and supervision of construction, it is believed that this experience could offer another 'bonus' solution to the major problem of maintenance for offshore structures.

The floating stability of the concrete concepts in a vertical position also offers further possibilities for improved platform construction operation. It is clearly better to locate as many erection operations onshore and such elements as deck structures, equipment, cranes, deck modules, etc., could be usefully erected prior to towing the platform structure. With its configuration available for this to be done, each specific application could be explored for further economies to be achieved from this point of view.

Time and cost factors

From a production point of view the concepts described for concrete platform structures will require quantities of concrete and steel reinforcement to be placed with considerable speed to achieve delivery times comparable with steelwork structures. For the box type structure of 160,000 tonnes weight, there is needed 67,000 m^3 of concrete and approximately 20,000 tonnes of steel reinforcement. For the tower type structure of 90,000 tonnes weight there is needed 38,000 m^3 of concrete and approximately 10,000 tonnes of steel reinforcement.

These quantities pose a formidable problem in estimating, from both the time and cost point of view. Bearing in mind that the onshore construction probably would take place in areas remote from material and labour supplies, the

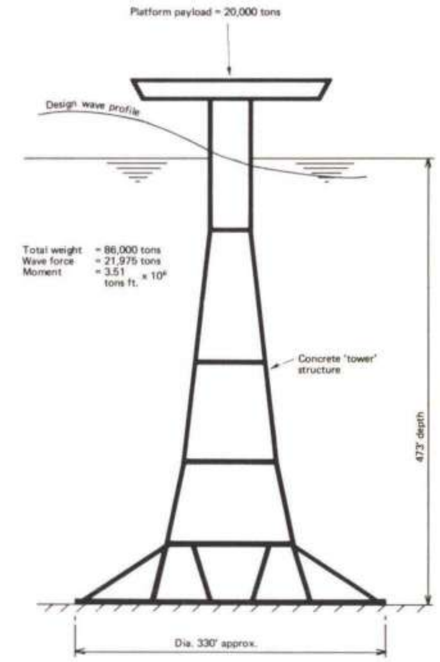
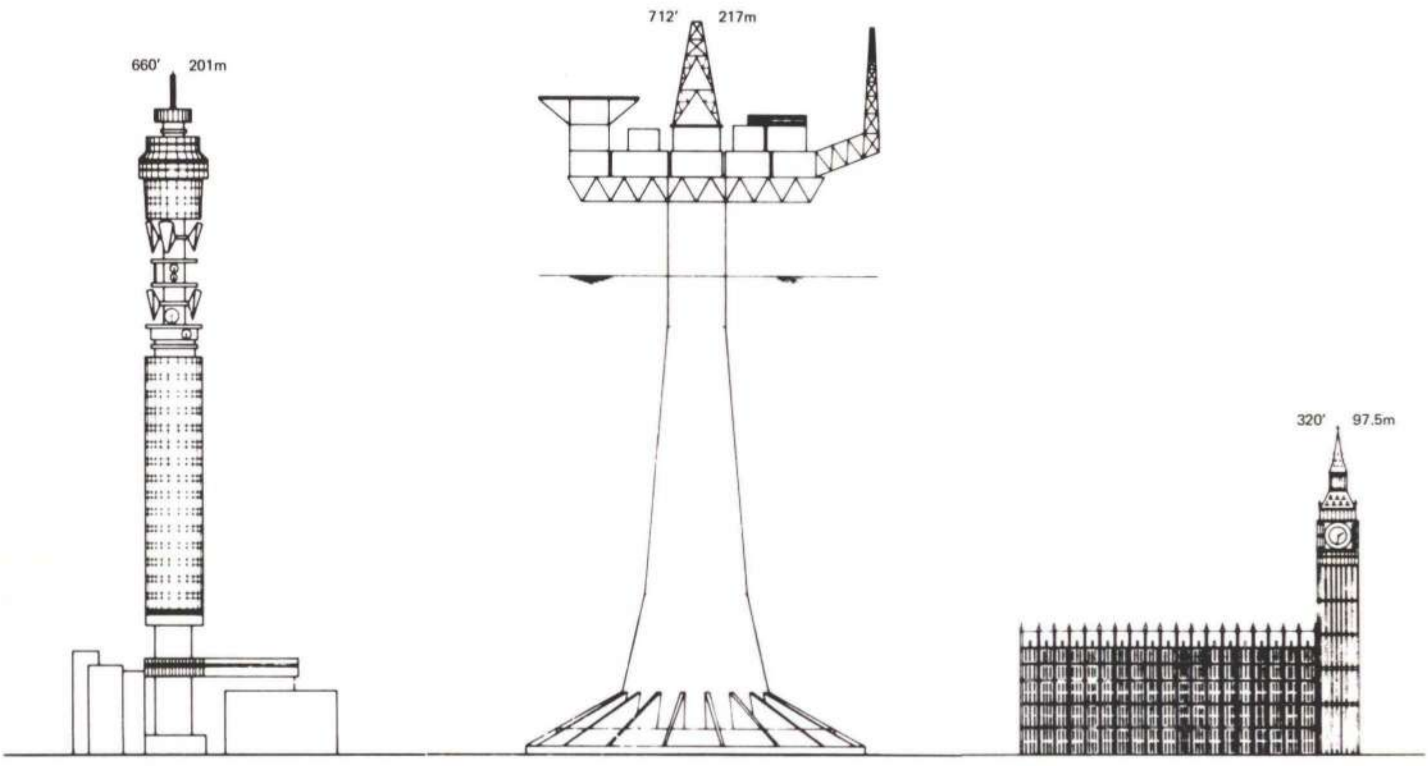


Fig. 2
Tower structure

volume and rate of production to be achieved can be compared with the highest known targets set for any major concrete civil engineering project. Where very rapid concrete production has been achieved, this has been closely linked to innovative developments in formwork techniques such as the slip-form method of continuous vertical construction.

From this experience it is reasonable to consider possible average rates of production of 760 m^3 per week for the concrete structures being studied. This rate gives a period for the box structure of 88 weeks and for the tower structure of 50 weeks. In actual practice it is

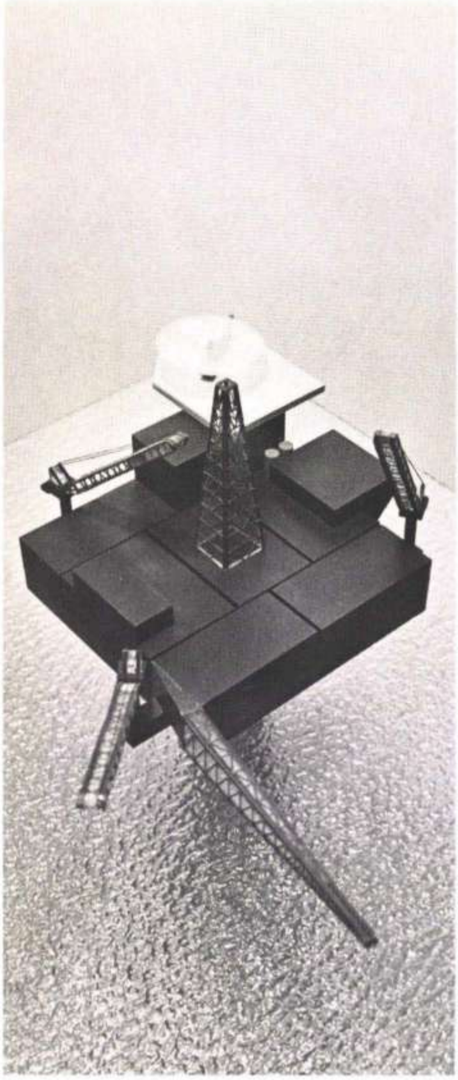


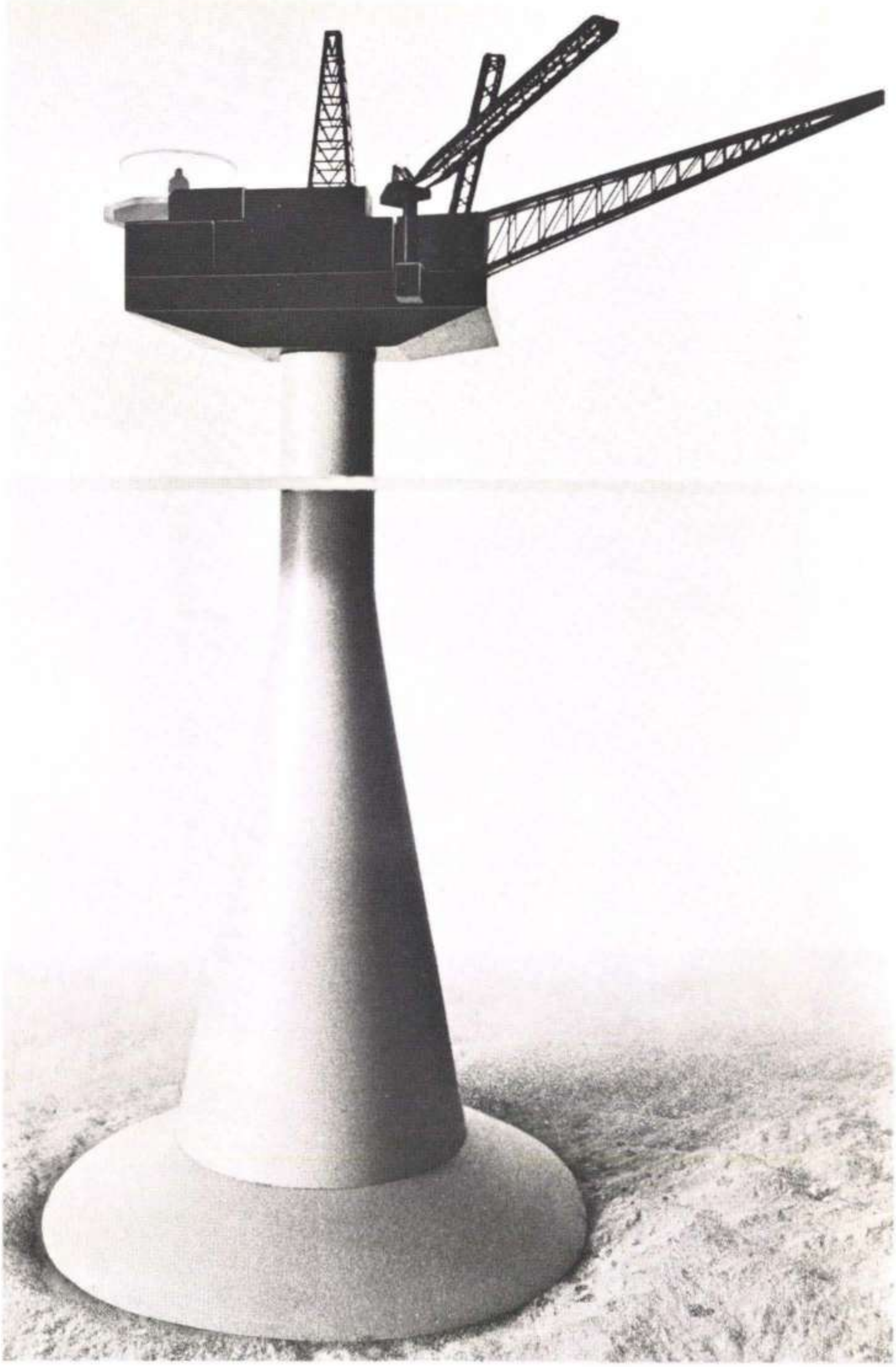
GPO Tower, London

Ark platform

Big Ben and Houses of Parliament

Fig. 3





Figs. 4 and 5
Model of Ark platform

(Photos: Harry Sowden)

considered that this period for concrete work will require approximately 50 per cent increase of time to take account of setting-up and dismantling all the associated temporary works. The overall delivery times thus appear to be of the order of 132 weeks to 75 weeks depending on which basic concept is adopted for the concrete platform.

While it is unrealistic to use these figures to make a definite claim that concrete structures can offer to meet the type of programmes adopted for oilfield developments in deep water, it is clear that sufficient experience exists to be optimistic that they will do so. A specific example is shown in Fig. 3 which is a development known as ARK for a 90,000 tonne concrete platform in 150 m depth with an installed delivery time of 82 weeks from date of order.

To assess the economics of these concrete structures is also not easy from the installed cost point of view. The onshore work is sufficiently straightforward to be able to estimate costs from previous experience. As with steel structures the cost figures to be allowed for overcoming the hazards of flotation and submergence on location are very difficult to assess.

For this part of the work it has been thought that a sharing of risks with the oil company concerned may be to the best advantage, so that unrealistic prices are not used which will prejudice offshore operations. In this way experience will be obtained which can be met at cost in dealing with the unknown environmental state of wind and weather when platform structures are transferred to their offshore locations.

Nevertheless, some cost figures are required for development planning, and guidance on concrete structures is necessary if they are to meet the challenge of being accepted as valid platform concepts. A convenient basis for consideration may be the installed cost per ton weight of structures material used. Bearing in mind the difficulties in estimating outlined above, a reasonable target for the installed cost rating for the box and tower designs described earlier could be taken as £120–£150 per tonne.

Conclusions

This paper has attempted to show that a concrete platform offers a valid concept for the deep water conditions of the northern North Sea. At this stage in the development of such offshore structures there is little evidence of their use being seriously accepted by the oil companies concerned. A considerable amount of development planning is being undertaken which must take account of the overall situation being faced with oilfield commercial viability. It is hoped that the information in this paper may prove useful to such planning studies, and may help to realize the potential contribution of concrete platform structures in future offshore engineering work.

Events have moved rapidly since this paper was prepared and concrete platform structures have now made a breakthrough into the offshore scene. The first platform contract has been awarded by Mobil (North Sea) Ltd. to the Norwegian *Condeep* design, the promoters of which were involved in the construction of the prestressed concrete oil storage tank for Ekofisk Oilfield. The same group have been awarded a second contract by Shell Exploration Ltd. for the Brent oilfield east of the Shetland Islands. Competition is now very keen between rival promoters of concrete platforms, and the ARK concept has been offered in two firm bids to Shell Exploration Ltd. with other oil companies expressing serious interest.

Review of awards gained in 1973 for jobs designed by

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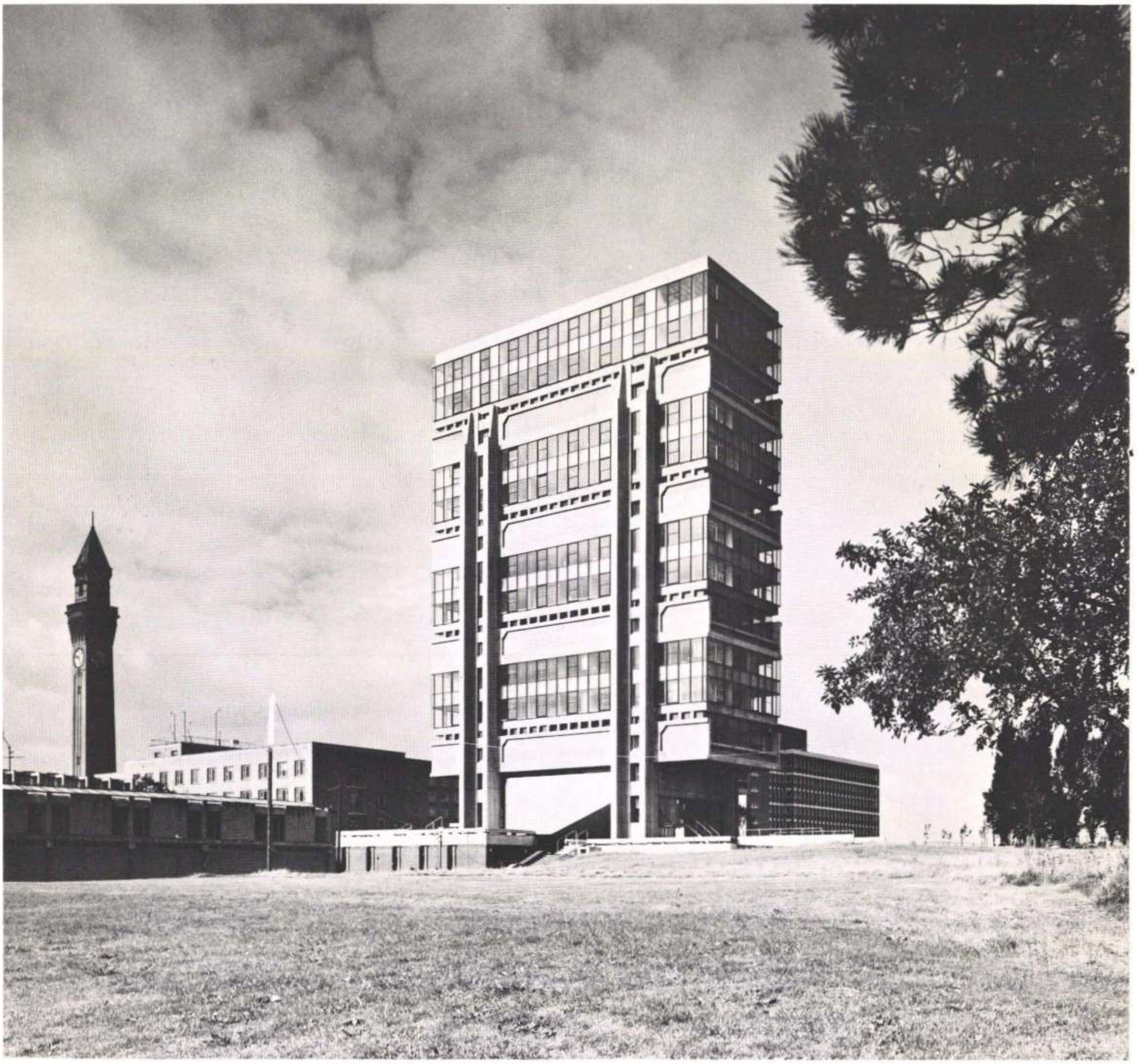

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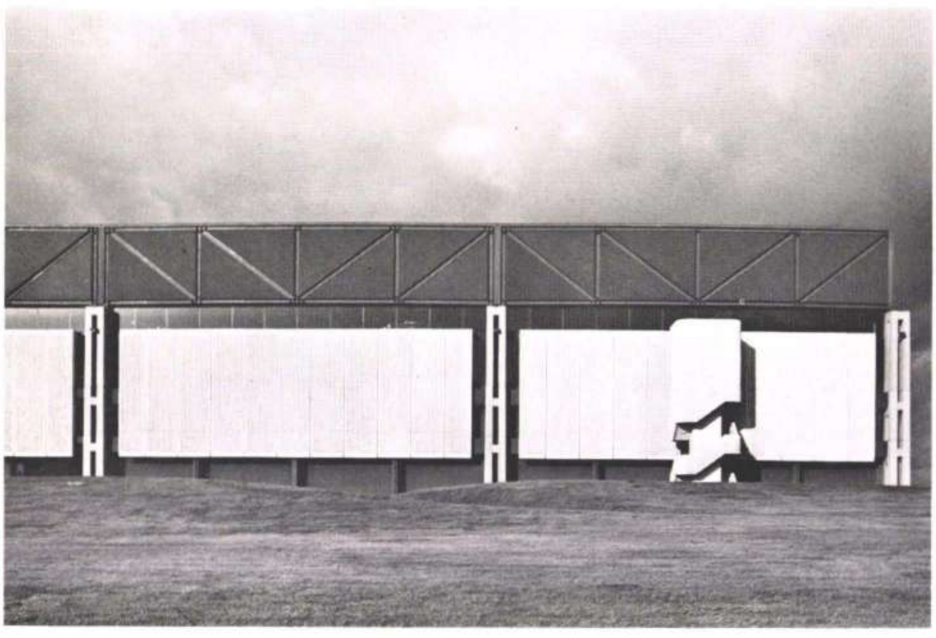

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Arup Associates Architects + Engineers + Quantity Surveyors

Fig. 1 Arts & Commerce Building, Birmingham University (Job no. AA 101)
Civic Trust Commendation 1972
(Photo: Colin Westwood)

Fig. 2 Arts & Social Sciences Building, Leicester University (Job no. AA 103)
Civic Trust Commendation 1972
(Photo: Colin Westwood)

Fig. 3 Player's Factory, Nottingham (Job no. AA 175)
Civic Trust Award 1972
Financial Times Industrial Architecture Award 1973
(Photo: John Donat)

Fig. 4 Oxford Mail & Times Ltd., Offices (Job no. AA 126)
Civic Trust Award 1972
RIBA Architecture Commendation 1973
(Photo: Thomas Photos)

Fig. 5 Penguin Books Offices, Harmondsworth (Job no. AA 162)
RIBA Architecture Commendation 1973
Concrete Society Commendation 1973
(Photo: Arup Associates)

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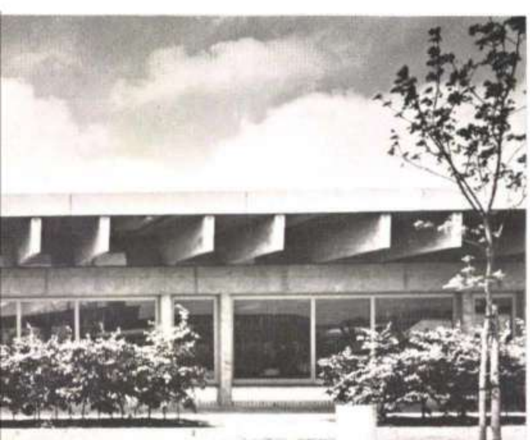

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Review of awards gained in 1973 for jobs with

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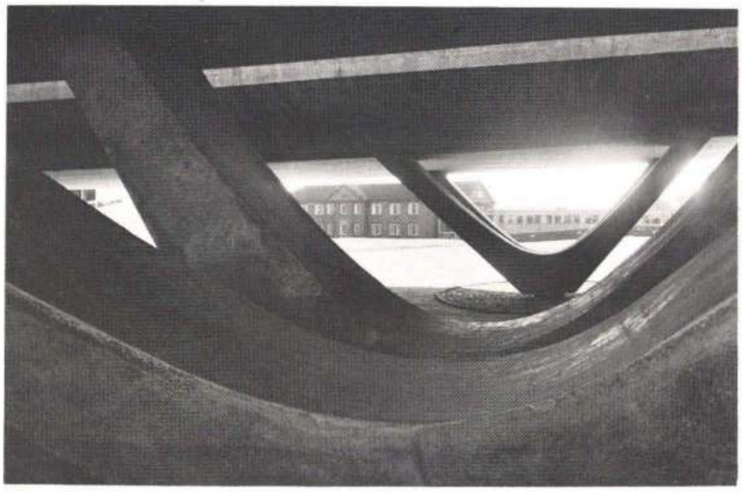

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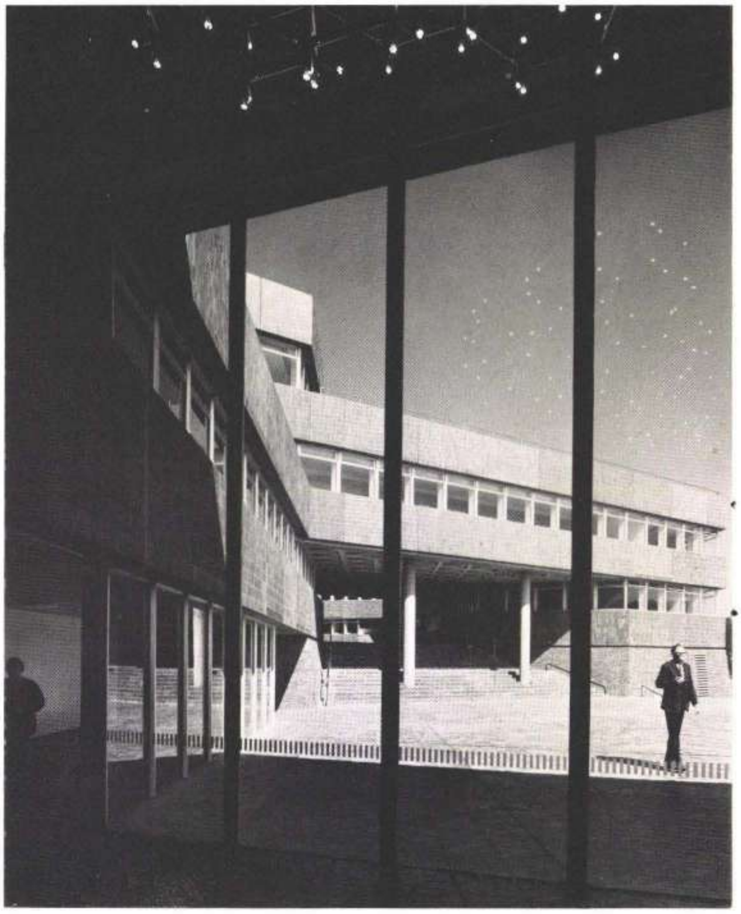

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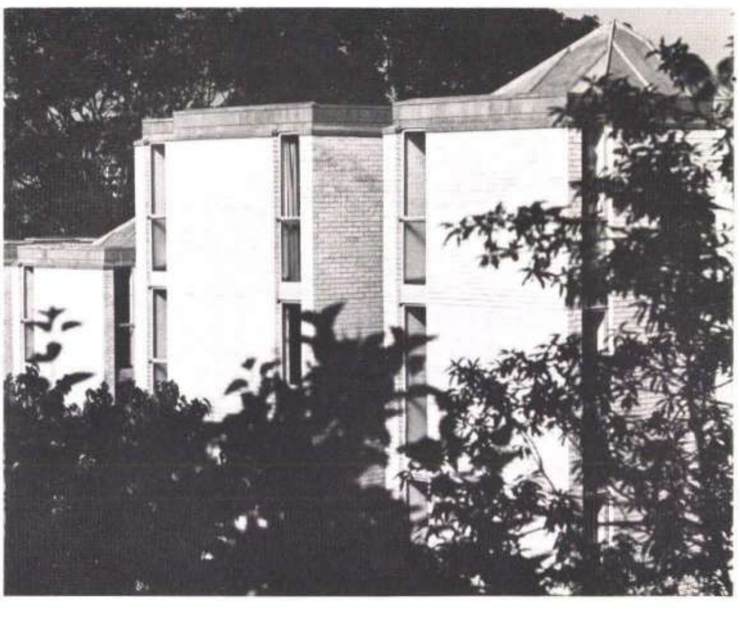

Fig. 1 Western Bank Bridge, Sheffield (Job no. 2266)
Designed by Ove Arup & Partners
Civic Trust Award 1972
(Photo: Henk Snoek)

Fig. 2 Sunderland Civic Centre (Job no. 2118)
Architects: Sir Basil Spence, Bonnington & Collins
Civic Trust Award 1972
(Photo: Henk Snoek)

Fig. 3 Crucible Theatre, Sheffield (Job no. 2798)
Architects: Renton Howard Wood Associates
Civic Trust Award 1972
(Photo: John Donat)

Fig. 4 Dumbarton Town Centre (Job no. 2400)
Architects: Garner, Preston & Strebel
Civic Trust Commendation 1972
(Photo: Courtesy of architects)

Fig. 5 Royal Commonwealth Pool Edinburgh (Job no. 2363)
Architects: Robert Matthew, Johnson-Marshall & Partners
Civic Trust Award 1972
(Photo: Henk Snoek)

Fig. 6 Norrie Miller Walk, Perth. Footbridge (Job no. 3675)
Architects: Derek Lovejoy & Partners
Civic Trust Award 1972
(Photo: Courtesy of architects)

Fig. 7 Goldney House, Bristol University (Job no. 2037)
Architects: Architect's Co-partnership
Civic Trust Award 1972
(Photo: Michael Springer)

Fig. 8 Sydney Opera House (Job no. 1112)
Architects: Jørn Utzon (Stages 1 & 2)
Hall, Todd & Littlemore (Stage 3)
ISE Special Award 1973
(Photo: Michael Andrews)

Fig. 9 Bernat Klein Studio, near Selkirk (Job no. 3684)
Architect: Peter Womersley
RIBA Architecture Award 1973
(Photo: Courtesy of *Scotsman*)

Fig. 10 Medical School, Southampton University (Job no. 2957)
Architects: Sir Basil Spence, Bonnington & Collins
Concrete Society Commendation 1973
(Photo: Henk Snoek)

Fig. 11 St. Peter's Church, Dumbarton (Job no. 2828)
Architect: Garner, Preston & Strebel
RIBA Architecture Award 1973
(Photo: Henk Snoek)

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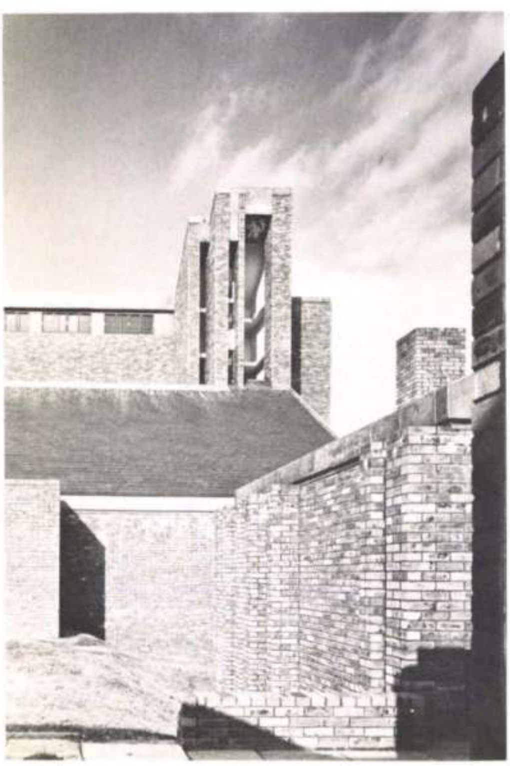

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Kazerne Viaduct

Cliff McMillan

Introduction

On Tuesday, 10 July 1973 the Kazerne Viaduct was opened to traffic. This key section of the M2 East was the last remaining link preventing motorists from travelling uninterrupted for 18 km on Johannesburg's motorway system from the Germiston border to the northern suburbs.

Kazerne Viaduct takes its name from the coalyard over which it passes. It is located three kilometres to the south-east of the city centre and forms part of the east-west M2 motorway crossing the city. The route of the motorway was dictated largely by the low expropriation costs in the area due to the existence of gold mining workings at relatively shallow depths.

The increased cost of designing for mining subsidence movements was justified by this reduction in land cost. The line of the viaduct is crossed by roads and railway tracks along its entire length.

In January 1968, Ove Arup & Partners were approached by the contracting company of Piovesan SA (Pty) Ltd. to prepare a design for their tender submission to the Johannesburg City Council.

In the event of the tender submission being successful, the brief was to extend to the preparation of full working drawings and details, but excluded site supervision, which would be carried out by the City Engineer's Department.

Spans of the viaduct vary considerably from a minimum of 35 m to several of approximately 60 m. The maximum of 65 m is somewhat longer than the normal economic limit for this type of work. The span lengths were largely dictated by the position of the existing tracks and other facilities of the goods yard. The Kazerne Marshalling Yard is a key link in the distribution of coal to Johannesburg and consequently all tracks had to remain open during construction.

Design conditions

The design conditions were fully prescribed in the documents issued by the City Engineer's Department. They called for the design of a double carriageway structure approximately 33.5 m in total width and 564 m long, with two ramps at the east end an an on-ramp at the west end. A maximum structural depth of 2.4 m was prescribed in order to accommodate headroom restrictions. Because of the ramps, very little of the viaduct was parallel-sided.

The construction, which was not to interfere with the traffic in the coalyard, was to be completed within two years. The viaduct was to be designed for HA loading and other loads such as impact and wind were defined. The design was to comply with specified standards and codes of practice.

Substantial mining subsidence movements were specified which had to be accommodated by the structure without overstress or damage. The specified movements relate to an assumed subsidence bowl and amount to relative horizontal movements between points at the surface of 75 mm in 25 m and 250 mm in 210 m. In addition, a relative movement of 150 mm in 25 m was to be considered with remedial jacking. Substantial ground rotations and relative vertical displacements had also to be accommodated under full live load.

A geological report accompanied the tender documents. The site is overlain by rubble, silt and slime from the adjacent mine workings to a depth of between 10 m and 17 m. The report recommended an allowable bearing pressure on the overburden material. Below this level there is hard fissured quartzitic sandstone and towards the east end of the motorway this rock outcrops in a shallow cutting.

At both ends of the site, the viaduct terminates on existing mine dumps which are being cut down to the motorway level. The silty slime material in these dumps is retained by abutments.

Fig. 1

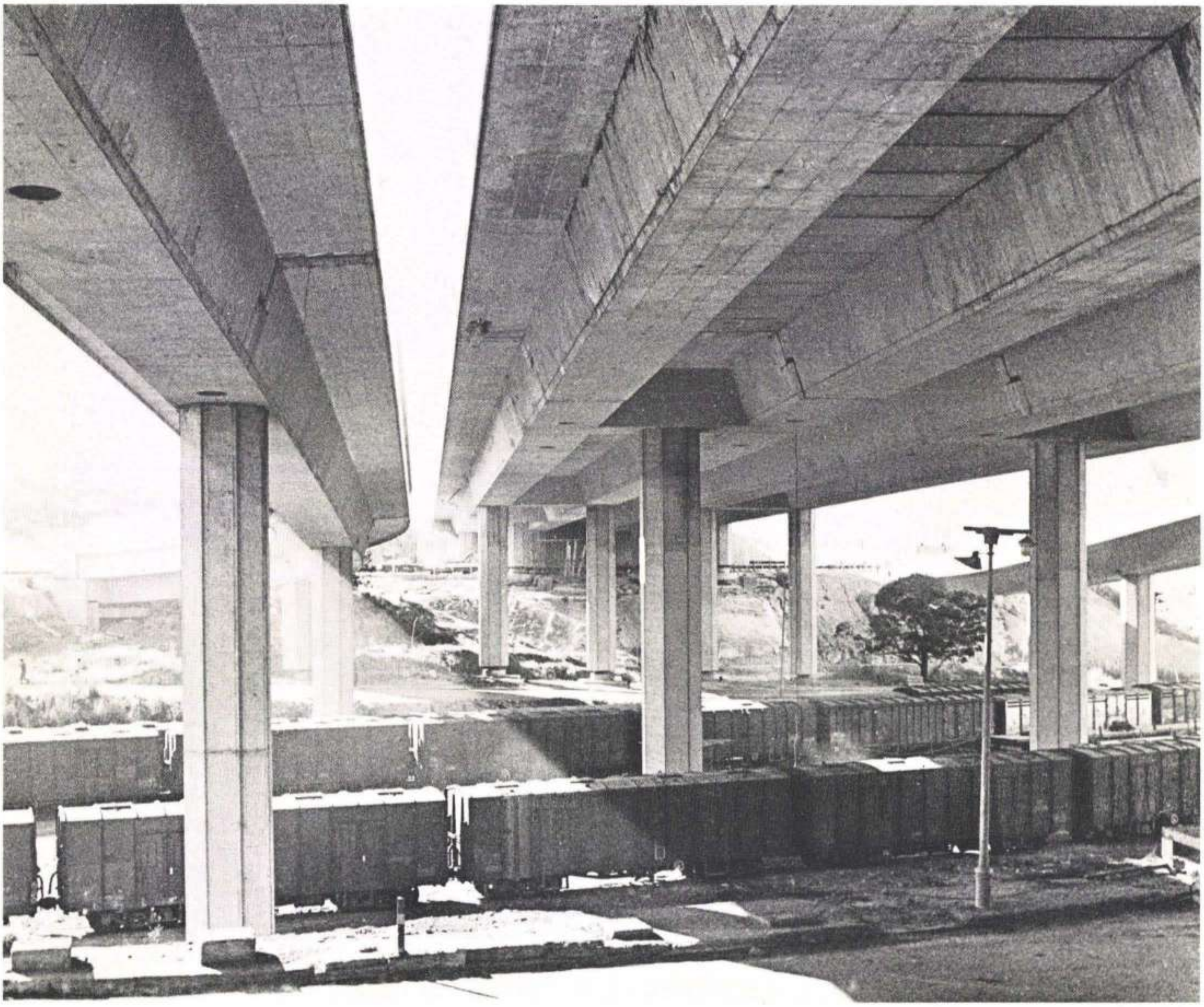


Fig. 2

Fig. 3

The design conditions led to a number of decisions:

In the first place, it was decided that the deck should incorporate some continuity in order to control deflections. It was felt that the discontinuity of slope in a simply-supported structure with the spans and depths specified would give an unacceptable riding surface.

Secondly, the long and variable spans and non-uniform viaduct width permitted little repetition and led to the acceptance that extensive use would have to be made of in situ concrete, despite the requirements of minimal interference with surface traffic.

Finally, it was agreed that stability should be incorporated into the deck and column structure, which should be isolated as far as possible from the ground by bearings under the columns at ground level. This eliminated the magnification of ground movements by columns fixed at the base.

The solution adopted divided the motorway into two independent carriageways, each of which consisted of a series of three-legged continuous tables with drop-in spans between. The tables cantilevered beyond the line of the columns in order to obtain the desired continuity and improve the smoothness of the deflected shape and thereby the riding quality. By rigidly connecting the columns to the deck structure and providing bearings at ground level, ground movements and particularly rotations were eliminated at source.

The typical deck structure of each carriageway comprises two longitudinal box beams carried on transverse cross-beams at the column lines. The cross-beams are supported on one or two columns to suit the nature of the three-legged tables. The box beams have sufficient torsional stiffness to transfer the transverse overturning forces to the twin-column frame for stability.

The three columns under each table are supported on spherical bearings at ground level, the one under the single column being pinned to the foundation and the other two sliding, with one guided in the longitudinal direction. In this way physical movement of the table is prevented but no ground movement can induce forces larger than the limiting bearing friction. The bearings chosen for the contract were purpose-made *Glacier* spherical bearings with *Teflon* sliding surfaces.

The drop-in panels are similar in construction but in this case each box beam is supported on three bearings – two at one end and one at the other. The effect of the twisting between the tables due to movement is thus largely eliminated with only the slab between the box beams in the drop-in panel becoming stressed.

On the ramps, which comprise a single box structure, the three-point support principle is maintained by incorporating two bearings under a cross-beam below ground at one of the columns of a table. The deck section chosen facilitates a smooth merging from the ramps, with three box beams side by side, to the typical section of two beams.

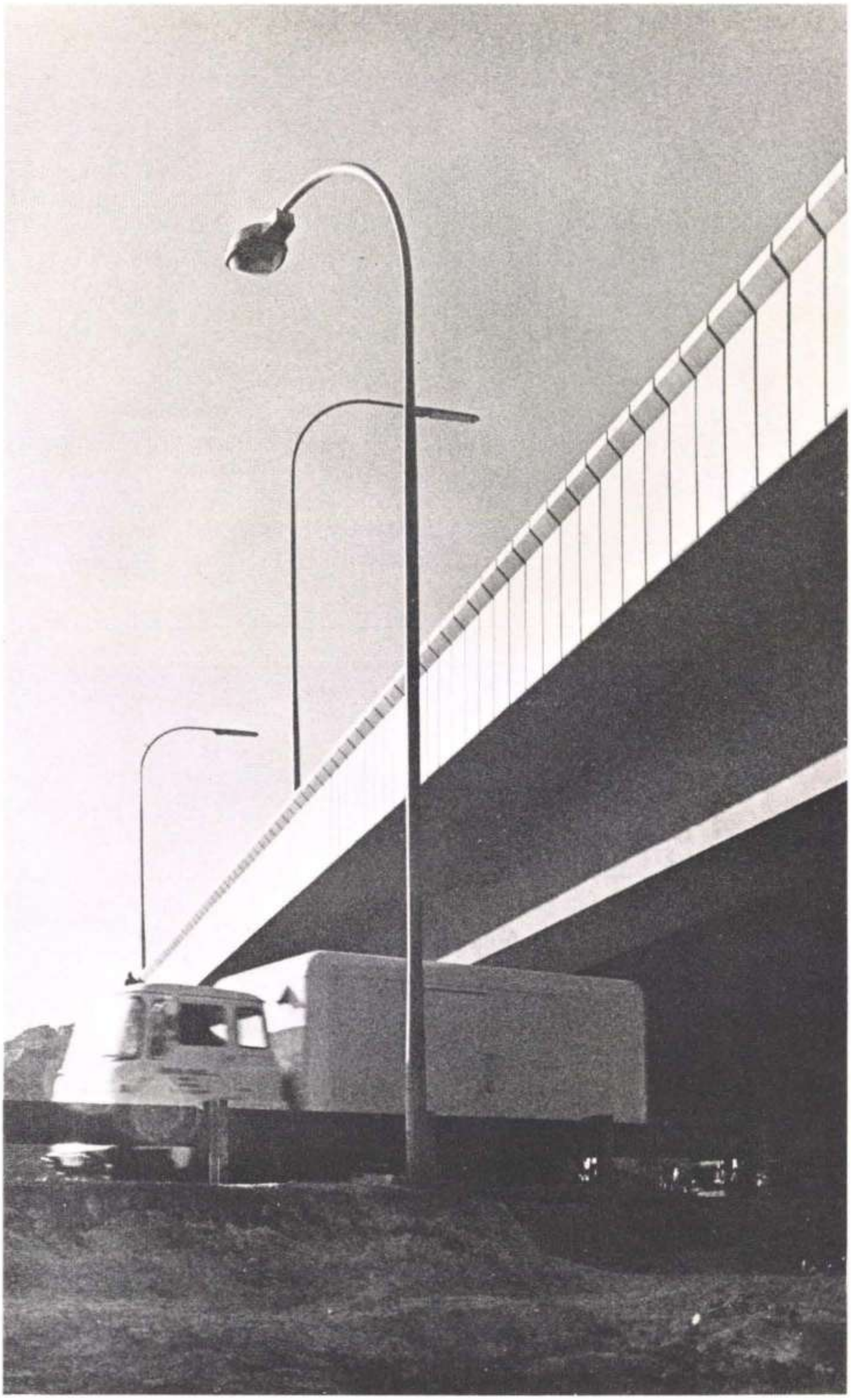


Fig. 4

It was desirable to reduce the loads to be propped from the ground during construction and necessary to limit the duration of propping. As a result a stage construction procedure was adopted. In the first stage the open through sections and cross-beams were cast and stressed to carry their own weight and the weight of deck to be added. Then the propping could be removed. The deck on top was added in the form of precast slabs which were jointed with in situ concrete. The side cantilevers were cast in situ. A final stage of stressing completed the process.

The prestressing system chosen by the contractor was the *Morandi M5* system. Because of the necessity to keep the dead load to a minimum over the long spans, it became apparent that it would be difficult to accommodate all the cables within the concete section. An application was made to the City Engineer's Department to use unbonded cables for part of the prestressing.

This was accepted after details of the corrosion protection of the cables, and the effect on the ultimate strength of the beams of unbonded cab'es, were satisfactorily explained. For the long spans all the first cables are normally unbonded, while the second and third stage cables are fully bonded.

Diaphragms are located in the box beams at approximately 7.5 m centres along the spans, and these serve to control the unbonded cable profiles. The cable is laid in a metal sheath, which after stressing, is grouted and later wrapped with a corrosion-protective material. There is access to the unbonded cables for inspection throughout the life of the structure.

Vertical loads to be carried on the three bearings of each table ranged from 4500 kN to a maximum of 30,000 kN with longitudinal sliding movements of up to 150 mm being required in both directions. Contact pressure between the bearing place and the concrete were restricted to 21,000 kN/m^2, giving a maximum bearing diameter of 1.5 m.

Purpose-made *Glacier* spherical bearings were chosen. The sliding surfaces were *Teflon*, and the recommended design co-efficient of friction under maximum normal pressure was $3\frac{1}{2}$ per cent, with a likely range of between 1 per cent and 5 per cent. The drop-in spans were supported on similar spherical bearings, with a pinned bearing at one end of the beam and two sliding bearings, one permitting only longitudinal movement, at the other end.

Columns varied greatly in height between 4.5 m and 15 m. An octagonal column shape was chosen with a cross-section of 2.1 m across the flats. This meant that the line of the edge of the soffit of the box beam and the cross-beam always ran past the column without interruption.

Two types of expansion joints have been provided on the viaduct. The small joints at the end of the drop-in span which is pinned to the adjacent table only have to accommodate vertical and in-plane rotations. These movements are accommodated by *Pressfit* neoprene inserts.

The large joints, at the end where sliding occurs between a drop-in span and the table, have to accommodate inward and outward movements of up to 215 mm. A further special requirement of the design was that these joints should be accommodated within the depth of the 230 mm slab between the box beams and the tapering slab of the side cantilevers. After considerable research it was decided to accept the comb type of expansion joint for the large joints.

The City Engineer's Department have experienced difficulties with drainage of motorway decks, and requested an arrangement of exposed bolted flanged pipes. This was accepted with reluctance because of the unfortunate aesthetic consequences.

For the viaduct foundations, the overburden material was such that it would have been impractical to carry column loads as high as 30,000 kN on spread fottings. It was therefore decided to found each column on a single 3 m diameter hollow shaft underreamed into rock at a bearing pressure on the rock of 1,400 kN/m². The shafts were sunk as caissons through the overburden material.

The abutments posed an interesting design problem. It was decided to divorce the viaduct from the mine dumps and cantilever the deck from the last row of columns to the abutment, thereby isolating the deck from the effects of possible subsidence of the abutment. The west abutment is 14 m high at a slope of 1 :1.

A conventional rigid abutment would have been subject to exceptionally high lateral forces as a result of ground movements. Instead, a flexible beam and slab abutment structure was constructed on the slope and anchored to the rock below with permanent rock anchors to provide the necessary stability. The length of the anchors is up to 21 m and they can accept the elongation or shortening due to minining subsidence with an acceptable and predictable increase or decrease in anchor force.

Detailed design

The contractor moved onto site less than a month after the contract was finally awarded. Thus, from the outset, information such as foundation and column details relating to parts of the viaduct were required on site before that part had been fully designed. This situation persisted for almost a year before the design advanced ahead of the immediate demands on site.

Although the structure was statically determinate the detailed design was complicated. The extreme slenderness of some of the spans, with a span-to-depth ratio of 30.5 during the first stage of construction created exacting design parameters.

The two-stage construction procedure envisaged in the original concept demanded that in order to limit the range of stresses on the first stage section for the long spans, the precast deck units had to be added and jointed in a predetermined sequence. Great care had to be taken to ensure that unacceptable stresses or unstable conditions did not exist during any one of a number of construction stages.

Computer programs were developed to cover almost every portion of the design which could be automated. The objective was to take maximum advantage of the fact that a large amount of the work, though non-typical, possessed a high degree of similarity. Use of computer programs had the added advantage of standardizing and improving the reliability and presentation of calculations. Without these programs it would have been impossible to meet the site deadlines for information.

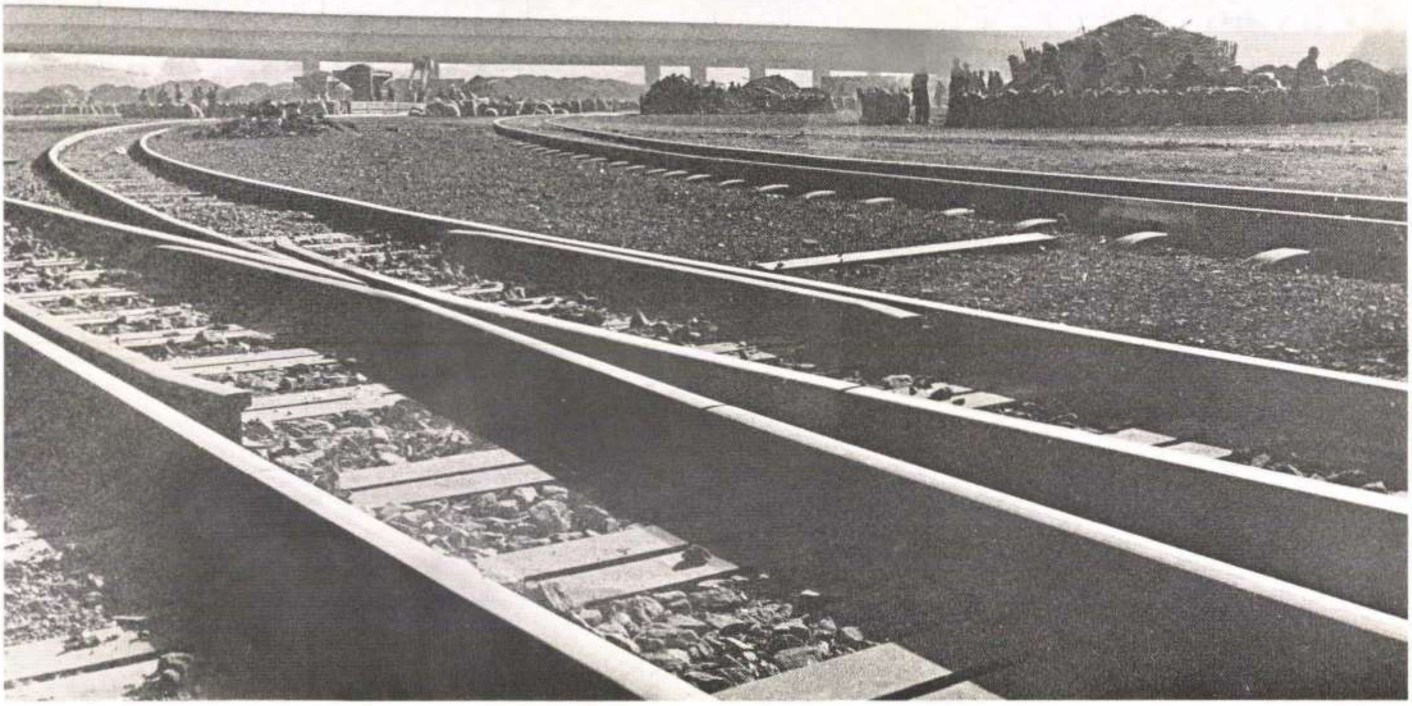
Programs were developed, for instance, to carry out the repetitive calculations involved in determining the foundation diameters and reinforcement, and the beams soffit levels and wall heights at many cross-sections along the deck.

In the analysis the symmetrical tables were assumed statically determinate for dead loads. For live load distribution determination, a space frame analysis of each table was performed. For the unsymmetrical tables, space frame analyses were required for dead loads also, to model the sequence of construction.

In the limited time available, no attempt was made to develop a fully-automatic program for determining the cable profiles. Instead, a simple program was used which, for given moment ranges, shear stresses, prestressing forces, cable eccentricities and losses, could determine the extreme fibre and interface stresses at all stages of construction. Cable profiles were assumed, and then adjusted until all stresses were acceptable.

Despite the difficulties encountered, the project has been very stimulating. Grateful acknowledgement is made to Piovesan SA (Pty.) Ltd. and the City Engineer's Department for the co-operation received on this project with such outstanding technical demands. The viaduct is now complete and its quality of finish and elegance ensure that, despite the dreary surrounding of the Kazerne goodsyard, it is a tribute to all associated with it.

Fig. 5



Looking forward . . .

The next issue of the *Arup Journal* will include a series of articles on the Carlsberg Brewery

Interior view of Carlsberg Brewery. Architect: Knud Munk. (Photo: Colin Westwood)

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