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SESSION 4

The Influence of technical, economic and cultural factors in different design situations

Influence de facteurs techniques, économiques et culturels sur le choix du système et de la forme des structures

Einflüsse technischer, ökonomischer und kultureller Faktoren auf die Wahl des Systems und der Form von Tragwerken

Chairman:	Werner von Olnhausen, Sweden
Co-ordinator:	W. H. Arch, UK
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I

History of construction in different geographical environments

L'histoire de la construction dans les environnements géographiques différents

Geschicte der Tragwerke in verschiedenen geographischen Umgebungen

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SUMMARY
This paper is divided into three parts. The first discusses the huts of primitive civilizations. These were built before the classical periods in Egypt, Greece and China, and they are still constructed in various parts of the world, for example in Papua New Guinea. The second part deals with the more monumental buildings of the classical civilizations, prior to the development of iron and reinforced concrete structures. The final section examines some geographical influences on modern architecture, which has often been described as uniformly and monotonously international.

RESUME
Le rapport est divisé en trois parties. La première partie discute des huttes des civilisations primitives. Celles-ci ont été construites avant les périodes classiques en Egypte, en Grèce et en Chine, et elles sont toujours construites dans diverses régions du monde, par exemple en Papouasie. La deuxième partie s'occupe des bâtiments plus monumentaux des civilisations classiques, avant le développement de l'acier et des structures en acier et en béton armé. La dernière partie examine les influences géographiques sur l'architecture moderne, laquelle a souvent été décrite comme internationale, uniforme et monotone.

ZUSAMMENFASSUNG
Die Abhandlung ist in drei Teile aufgeteilt. Der erste bespricht die Hütten der primitiven Zivilisaitionen. Diese wurden vor der klassischen Zeit in Aegypten, Griechenland und China gebaut, und sie werden auch heute in verschiedenen Gebieten der Welt noch errichtet, zum Beispiel in Papua, Neuguinea. Der zweite Teil behandelt monumentale Gebäude der klassischen Zivilisationen, bevor Strukturen aus Eisen oder aus Stahlbeton entstanden. Der letzte Abschnitt untersucht den Einfluss der Geographie auf die moderne Architektur, die oft als international einheitlich und monoton bezeichnet wird.

Primitive Buildings

Lest I be accused of condescension, may I quote the Oxford English Dictionary, which defines primitive, *inter alia,* as "original, from which some construction begins, from which another is derived". The buildings of primitive civilizations are small and simple, compared with those of the classical period, because there is not yet any demand for great religious buildings or for palaces for powerful rulers.

Most primitive buildings leave no ruins, so that we have only limited information about the structures of the pre-classical civilizations, but they were probably not unlike those of present-day primitive societies. Until the late 19th century there were numerous tribes whose construction procedures had not been affected by the iron age. There are some people in the New Guinea Highlands whose first contact with the outside world was made within living memory, and whose buildings are therefore of particular interest. Apart from the use of steel tools for cutting the materials, they have not changed significantly since that time (Fig. 1).

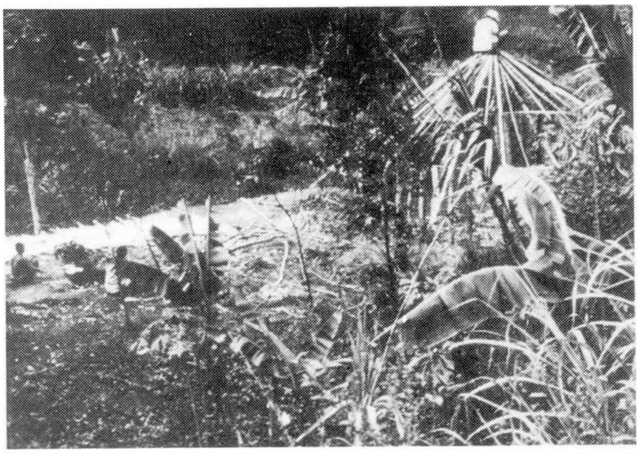

Fig.1 Circular hut in the New Guinea Highlands under construction. Once the materials have been assembled the hut can be easily be erected by two people in one day. The structure consists of bamboo tied with vegetable fibres. The walls are plaited mats.

In the twentieth century primitive societies have been confined to the hottest and coldest parts of the earth's surface. The former may be divided into hot-humid and hot-arid zones, of which the hot-humid is the more hospitable. It produces an ample supply of readily worked structural material in the form of bamboo, reed, and tree saplings. These can be cut at leisure. When there is an adequate supply of structural materials of the right length, a house can be built easily in one day.

The resulting house is not entirely weatherproof, it becomes infested with vermin, and it is easily destroyed by fire. However, it does not cost much to build, even in terms of the resources of a primitive society, and it is easily replaced. Indeed, in many regions it is treated as a disposable building, to be abandoned or destroyed when it no longer meets the sanitary standard of its owners. It is interesting to note that in the 1960's, when the consumer-oriented society was at its most fashionable, a disposable house was regarded as a desirable modern objective.

The construction is environmentally suited to the climate. It provides ventilation in a hot-humid climate; insulation would be of little value when the difference between the daily maximum and minimum temperature is small.

The type of building illustrated in Fig. 1 is still erected in many of parts the Pacific region, even in islands, such as American Samoa or Fiji, which have been greatly influenced by European-American concepts. There are regional differences, notably the use of two vertical poles and a ridge beam to produce a large hut (Fig.2). There are modern improvements, such as the use of concrete floors in Samoa. The tradiational method has been totally abandoned in some islands, for example Rarotonga, where timber-framed huts with iron roofs have acquired the characteristics of an indigenous architecture.

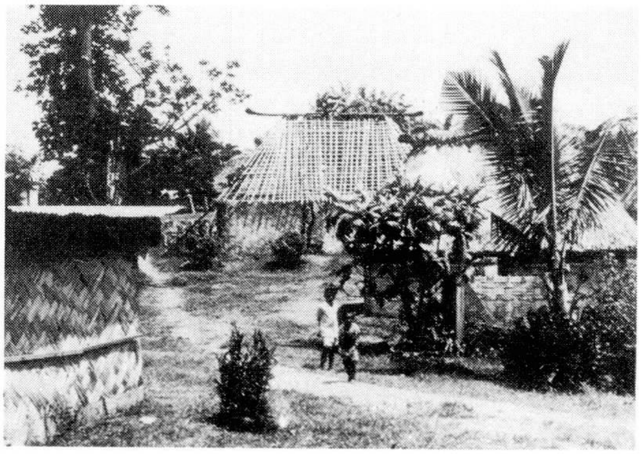

Fig. 2 A large bamboo hut in Fiji, which has two vertical poles and a ridge beam.

The primitive architecture commonly associated with hot-arid regions has thick walls of local materials, such as mud, mud-brick or stone which provide good insulation and thermal inertia. The real structural problem is the construction of the roof. One common solution is a curved roof, but the attainable span is usually very small in primitive buildings and the interior space is limited accordingly. Another solution is a timber roof, where timber is available. A roof of light materials largely negates the insulation and thermal storage of massive walls; however, in some parts of the world with great temperature variations, notably in the Middle East, timber roofs are covered with a thick layer of soil, and sometimes also with grass, to provide the roof with the necessary thermal insulation and heat storage capacity.

In the 1930's, and again in the 1960's, it was fashionable to proclaim vernacular architecture as the true guide to the correct use of local materials in conformity with the local climate. Primitive man, if given many centuries to experiment, would unerringly discover solutions in conformity with the best modern precepts of building science. Looking at the surviving methods of vernacular construction, there is much evidence in support of this thesis. However, the keyword may be "surviving". When a primitive society comes into contact with another that has a higher technology it naturally adopts many of its techniques. The old methods are more likely to survive if they perform satisfactorily.

Even with that proviso there are a number of vernacular structures which are not well suited to their geographical region. Central Ghana may serve as an example. The northern part of the country is hot-arid, and the southern part is hot-humid and has ample resources of timber and reed. However, the people of a large part of the forest country live in mud-huts which are ill-suited to the climate. It may be that these people migrated from the arid north, but this would have happened several centuries ago. Attempts by various foreign experts to persuade the forest people to use the readily available timber for more comfortable buildings have not, as far as I know, been successful. This emphasizes the role played by tradition.

Classical Timber Structures

The influence of tradition is shown particularly clearly by Ancient Egyptian architecture which copied timber structures in stone for more than two thousand years with only minor variations in the structural concepts.

The emphasis on stone as the material for important Egyptian buildings is probably due to the importance attached to life after death, and thus to permanence of construction. In primitive civilizations durability is generally not a major consideration in the design of buildings; indeed, disposability may be considered desirable for reasons of hygiene.

We therefore enter the era of conflict between simple and economical long-span structures using timber, and durable structure using stone or brick, which could be made to bridge long spans only with difficulty and at great expense. We know very little of the, possibly long-span, timber structures of the early classical period, except in East Asia. The remains of the great masonry structures survive, and are well-known.

The Japanese had a interesting solution to the lack of durability of timber structures; they re-built important structures at regular intervals. Thus the present Ise shrine is allegedly an exact copy, the 59th copy, of the original shrine built in the 7th century (Ref. 1).

This interest in timber structures possibly accounts for the development of an ingenious structure for sloping roofs used in China and in Japan (Fig. 3). As far as I am aware this structure which consists of a series of beams of increasing length has never been used in the Middle East or Europe, nor has the triangulated roof truss been used in East Asia until introduced from Europe.

It is likely that the Romans understood the advantages of triangulation, because a picture of a triangulated truss appears on Trajan's column in Rome. It shows a bridge in Roumania across the Danube, by any strandards a big river. In medieval times the advantages of triangulating trusses were not understood in Europe, but they were rediscovered in the 16th century, probably by Andrea Palladio (Fig. 4).

The Japanese type of roof structure uses more timber, probably not scarce at that time in either Japan or Europe, but it requires far less labour for cutting the joints.

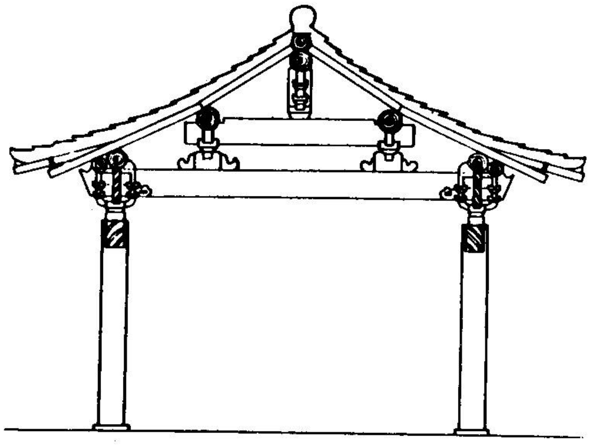

Fig. 3 Framing of traditional timber roofs in China and in Japan.

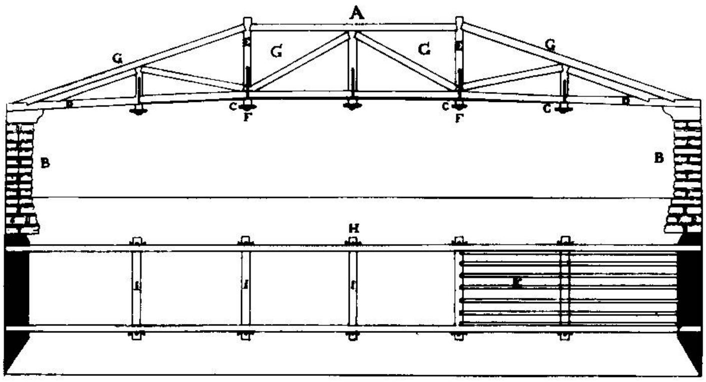

Fig. 4 Andrea Palladio's bridge over the River Cimone, with a span of about 30 m that was large for the 16th century. In his *Four Books of Architecture* (Ref. 2 p. 65) he explains the structure:- "A, The flank of the Bridge. B, The pilasters that are on the banks. C, The heads of the beams that form the breadth. D, The beams that form the sides. E, The colonelli, F, The heads of the cramps, with the iron bolts. G. are the arms, which bearing contrary to each other, support the whole work..." Evidently Palladio recognized the importance of the diagonal members.

Classical Masonry Structures

The history of the architecture surviving in the Middle East and Europe as distinct from that of Japan, is mainly a masonry buildings, and the roof structures of the most important buildings, since the time of Ancient Rome, were built of stone, brick or concrete. In some buildings, for example in most Gothic cathedrals, the masonry vault was surmounted by a timber roof. The masonry vault protected the interior of the cathedral from fire, and the timber roof kept the rain off a thin masonry vault that, in many cases,could not drain the water because it consisted of a number of curved structures with several low points. Before the age of lighting conductors, invented in the 18th century by Benjamin Franklin, a very tall building was liable to attract lighting, and its timber roof was thus at risk. In addition fire from military action was a constant problem throughout most of Europe's history. The importance of masonry vaults for fire protection was recognized at an early time:-

> "I am entirely for having the roofs of temples arched, as well because it gives them greater dignity, as because it makes them more durable. And indeed I know not how it happens that we shall hardly meet any temple whatsoever that has not fallen into the calamity of fire .... Caesar owned that Alexandria escaped being burned, when he himself took it, because its roofs were vaulted." (Ref. 3, p. 150).

Once the structural designer was committed to build a long-spanning roof with timber, and before the age of iron structures, he had to devise a geometry which took the greatest advantage of the compressive strength of masonry materials, and minimized their low tensile strength.

We know today that a cable hanging under its own weight, that is, a pure tension structure, assumes a mathematical curve

$$y = a \cosh \frac{x}{a}$$

where x and y are the horizontal and vertical coordinates, and a is a constant.

We call this a catenary, and it is similar in shape to a parabola. Evidently if a structure is to be in pure compression, it must have the exactly opposite shape, that is, an upturned catenary.

I do not know who first suggested that this theory was discovered in Mesopotamia 1500 to 1800 years ago, but it may have been James Waller. Certainly it is mentioned in a paper of which he is the co-author (Ref. 4), and a modern thin reinforced concrete catenary vault was named by him "Ctesiphon vault", after the 6th century vault on the outskirts of modern Baghdad.

Elongated arches resembling catenaries were used in Iran and Eastern Turkey in medieval times, and pointed arches existed in the Holy Land at the time of the First Crusade (1096-99), and were copied in the Gothic cathedrals.

The need for a correct solution would be greatest where the tensile strength of the masonry materials was lowest, and the Mesopotamia of the 6th century relied on lightly burned brick as a major building material because of shortages of natural stone, timber and fuel.

It is conceivable that this solution could have been discovered during the Hellenistic era (which made more profound innovations), and used in the Middle East for the construction of arches, vaults and domes. I found the argument sufficiently convincing to reproduce it in a book (Ref. 5, p.104).

I later developed doubt about its validity for two reasons. The catenary concept was discovered in the 17th century. David Gregory, Professor of Astronomy at Oxford University, published it 1697 (Ref. 6), but it was probably known to Robert Hooke several years earlier (Ref. 7, p. 50). It may even have been the idea of Sir Christopher Wren, who preceded Gregory in the chair of astronomy and used a structure resembling a catenary in the dome of St. Paul's Cathedral in London (Ref. 5, p. 180). From the 17th to the early 20th century designers used chain or string models to determine the line of thrust of arches vaults and domes (Ref. 5, p. 196-7 and Ref. 8, p. 87). There is no mention of the use of such models in earlier ages.

Secondly, the shape of the Ctesiphon vault could have developed from that of a timber vault. Reeds can be bent elastically to an arch shape which resembles a parabola or catenary (Fig. 5). A recent book published in East Germany shows a photograph of contemporary vernacular Iraqi roof built from reeds bents into arches (Ref. 9, p. 196). Vitruvius described vaulted roofs built from a structure of reeds bent into arches as a common form of construction in Roman times (Ref. 10, pp. 205-6).

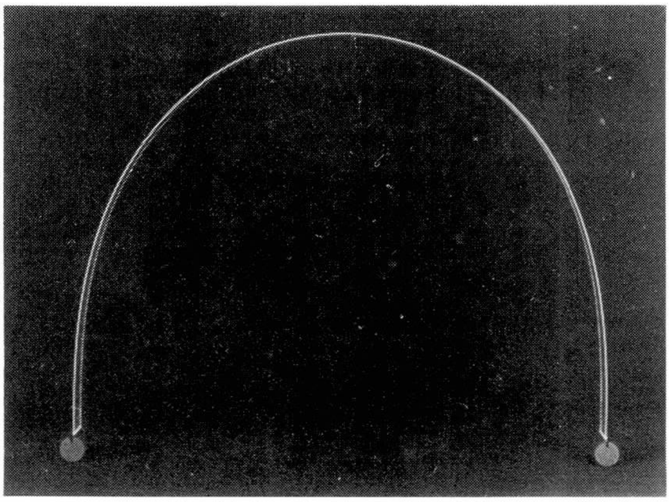

Fig. 5 Piece of perspex bent elastically to form an arch.

The timber column turned into stone persisted in Greek and Roman architecture, for centuries. It is therefore likely that the shape of the elastically bent timber arch would also have been used in masonry. The fact that it is relatively close to the catenary makes it a structurally efficient form, and this accounts for the fact that the Ctesiphon vault, with the large span of 25m, could be built in a weak material and could survive maltreatment for more than a thousand years. It would also account for the relative efficiency of the Gothic arch as compared with the Roman semi-circular arch. The elongated domes of the Renaissance and the Baroque which are much lighter than comparable Roman structures may also derive from the same source.

Modern Structures

In recent times geography was probably a factor in the development of the steel-framed building in the 1880's. Iron was first used for the construction of fireproof floors and roofs in the first decade of the 19th century in the industrial regions of the English Midlands (Ref. 5, p. 25). Until it attracted world-wide attention with the construction of the Crystal Palace, there had been general agreement that utilitarian iron structures were often masterpieces of engineering, but that they were not architecture. Iron might be used to support the masonry of a monumental building, so long as it was not visible.

After the success of the Crystal Palace, some architects began to think otherwise, and this drew a rejoinder in 1872 from Charles Eastlake, at that time the Secretary of the Royal Institute of British Architects:-

"One architect at least .... did not hesitate to avow his conviction that Mr. Paxton, guided by the light of 'his native sagacity', had achieved a success which proved incontrovertibly how mistaken we had been in endeavouring to copy from ancient examples; that the architecture of the future should be the architecture of common sense; and that if the same principles which had inspired the designer of the Exhibition building had been applied to the Houses of Parliament, to the British Museum, and to the new churches in course of erection, millions of money would have been saved and a better class of art secured.

"Sanguine converts to the new faith began to talk as if glass and iron would form an admirable substitute for bricks and mortar, and wondrous changes were predicted as to the future of our streets and squares ....

"It did not take many years to dissipate the dreams of universal philanthropy to which the Exhibition scheme had given rise, and with these dreams to charming visitors of a glass-and-iron architecture may also be siad to have vanished. If the structural details of the Crystal Palace teach us any lesson, it is that they are strictly limited in application to the purpose for which that building was erected". (Ref.11., pp. 281-282).

The prevailing opinion in New York was similar, but Chicago was too remote at the time of the Great Fire of 1871 to be concerned about such matters of taste. The rapid reconstruction of the city, using only fireproof structures, was the urgent priority. It was probably this geographical isolation which caused W.B. Jenney to design the first building with a complete skeleton frame (the Home Insurance Building, Chicago) in 1885, more than 70 years after the first use of iron beams in buildings. Once the economy and efficiency of the skeleton frame had been demonstrated, New York quickly adopted it, followed by Europe and Australia.

The use of a skeleton from did not by itself imply the abandonment of the external masonry wall. It continued in use even for "modern" buildings until the 1920's.

The concept of the transparent curtain wall was essentially aesthetic. The load-bearing wall needed to be thick, particulary if the building was tall; but buildings with skeleton frames did not require load-bearing walls. The lightness of the structural frame made possible by the new technology was obscured by the traditional masonry walls.

Some architects in the 1920's, notably Le Corbusier, thought that this was wrong:-

"The Engineer, inspired by the law of Economy and governed by mathematical calculations, puts us in accord with universal law. He achieves harmony ....

"The Engineer's Aesthetic and Architecture - two things that might march together and follow one from the other - one at its full height, the other in an unhappy state of retrogession.

"A question of morality; lack of truth is intolerable, we perish in untruth." (Ref. 12, pp. 1-2).

The glass curtain wall, while not a structural concept as such, was justified because of the emphasis it gave to the newly discovered light skeleton frame. Ludwig Hilbereimer, an associate of Ludwig Mies van der Rohe, first at the Bauhaus and later at the Illinois Institute of Technology, claimed that in Mies' buildings "the disunity between architecture and engineering had been overcome; the engineer, once the servant of the architect, is now his equal."(Ref. 13, p. 21).

This applies, however, only to the structure. The environmental design is clearly subordinate to the visual effect of the metal-and-glass curtain wall, the essential ingredient of the Miesian style.

It is interesting to speculate to what extent the glass curtain wall owes its development to the relatively severe climate of central Europe and the even more severe climate of the Eastern United States. In both countries proper facilities for winter heating are essential. In the Eastern United States summer cooling is also very desirable. By contrast an English architect might satisfy his client with open fireplaces and no summer cooling, and a Sydney architect might satisfy his client with no provision for heating or cooling, other than good attention to sunshading and ventilation. In neither case is a mechanical/electrical plant required.

In the 1950's energy was cheap and plentiful, and the concept made excellent sense in the climate of North-East America which for reasonable comfort demanded in any case a heat engine and a forced-ventilation system. The events since 1973 have changed the emphasis in architectural design, and post-modernism is, in my opinion, in no small part due to the energy crisis of 1973. It is possible that events would have taken a different turn if the avant-garde of modern architects had worked in a different geographical environment.

The time has come to review the relation between structure and environment. We took it for granted that a well-designed structure in primitive architecture and in the architecture of the classical period would help to provide a favourable thermal environment. The complete separation of the structural environmental functions occurred only in the mid-twentieth century. We should take a fresh look at the load-bearing wall, and particularly the load-bearing wall incorporating sunshades (Fig. 5).

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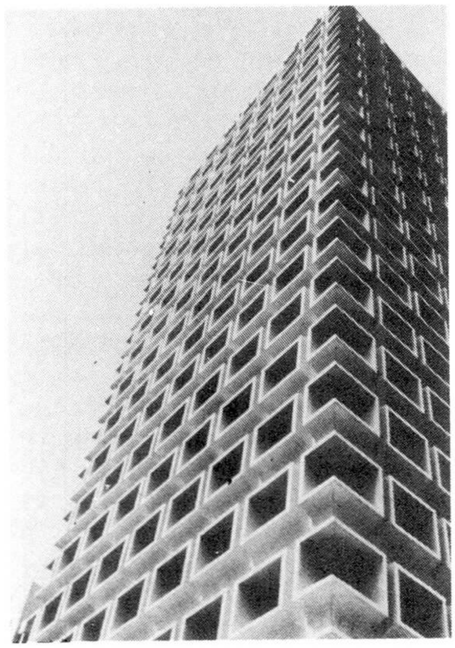

Fig. 6 Sunshades as part of a load-bearing wall (Greater Pacific House, Sydney)

I

Foundations in differing environments

Fondations dans des environnements différents

Fundationen bei verschiedenen Umweltsbedingungen

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SUMMARY
The selection of structural form for foundations is greatly influenced by the techniques available for their construction in the particular ground and environmental conditions. The adoption of a particular design may be determined solely by the availability of a specialized construction method, the structural form being unrelated to that of the superstructure.
The paper describes the effects of the environment on the development of construction methods for foundation works and their influence on structural form. Different environmental conditions will be considered.

RESUME
Le choix du type de fondation est fortement influencé par les méthodes de construction utilisables dans un sol et un environnement donnés. Une conception particulière peut être déterminée uniquement en fonction d'une méthode spéciale de construction, la forme de la structure de fondation étant sans relation avec celle de la superstructure. L'article décrit les effets de l'environnement sur des méthodes de construction pour les travaux de fondation et leur influence sur la forme de ces structures. Différants cas sont présentés.

ZUSAMMENFASSUNG
Die Wahl einer Fundationsart wird beeinflusst durch die verfügbaren Bautechniken und die Umweltbedingungen des betreffenden Baugrundes. Die Ausführung eines Projektes hängt allein von der Verfügbarkeit einer spezialisierten Ausführungsmethode ab, wobei die konstruktive Ausbildung von derjenigen des Ueberbaus völlig unabhängig ist. Der Beitrag beschreibt die Einflüsse der Umweltbedingungen auf Fundationsmethoden und deren Einfluss auf die Form von Tragwerken. Verschiedene Umweltbedingungen werden beschrieben.

THE GREEN FIELDS SITE

The location of a project on a green fields site implies construction on dry land in a temperate region with freedom from the difficulties involved in construction close to existing property. In these conditions the selection of structural form for the foundations is governed by the form of the superstructure and by the ground conditions.

In ground with good supporting characteristics the form of the foundations is simply the downward extension of the superstructure. Load-bearing walls are extended in the form of continuous strip foundations and columns are terminated in simple pad foundations. Design tends to follow traditional lines and is little influenced by construction techniques, although the development of efficient mechanical plant for earthmoving and compaction tends to favour shallow foundations at a nominal depth on graded terraced sites rather than tailoring foundation levels to suit the contours of the ground or the profile of a particular bearing stratum.

In some conditions the structural form of the foundations is determined by conditions other than the layout and loading of the superstructure. Where buildings are sited on expansive clays or shales the uplift forces from the swelling ground can greatly exceed the superstructure loading. The development of rotary auger drilling equipment for clays and shales has favoured the provision of deep pile or pier foundations as a means of isolating the building from the swelling forces by constructing it on beams and slabs supported clear of the ground. The alternative method is to provide a substructure founded at a relatively shallow depth with sufficient stiffness to prevent excessive upward curvature from the uplift forces. This method is more labour-intensive and not amenable to mechanization.

Heavy structures sited over weak ground on the green fields site will usually require some form of piled foundations. The driven displacement pile is usually selected in preference to the bored and cast-in-place pile because it mobilizes a higher proportion of the available skin frictional and endbearing resistance of the soil, and also for reasons of economy and rapidity in installation. The latter consideration is influenced by the development of light easily transportable piling rigs with a diesel or drop hammers capable of being operated by a small crew.

For light to moderate loadings the jointed precast concrete pile is likely to be the most economical type although the driven and cast-in-place pile may be cheaper in conditions where lateral ground displacement is not detrimental to the formation of the cast-in-place shafts. For very heavy foundation loadings the development of large diesel and steam-operated piling hammers has enabled large diameter steel tube piles to be installed with carrying capacities matching those of bored piles installed by plate auger or grabbing oscillator rigs.

As an alternative to piled foundations deep compaction techniques can be considered as a means of consolidating and stabilizing weak granular soils and fill materials thus enabling shallow strip or pad foundations to be used. These techniques comprise either the insertion, at a close spacing, of mechanical vibrating units to form columns of compacted soil or stone in the ground, or the compaction of the soil to considerable depths by dropping heavy weights on to the ground surface.

Where basements are required for buildings, the efficiency and high performance of modern earthmoving equipment will usually lead to the selection of a substructure consisting of free-standing cantilever retaining walls constructed in an open excavation with sloping sides. Even in poor or water-bearing ground it will usually be more economical to slope back the sides of an open excavation to a stable angle rather than to construct the basement in an excavation supported by anchored or strutted sheet piling. Wellpoint or bored-well ground water lowering systemspermit the adoption of quite steep slopes to open excavations in water-bearing granular soils.

Bridges in the green fields environment usually take the form of traffic separation structures with relatively short spans. Continuous deck slabs or girders are frequently used for three or four-span highway bridges and these structures are sensitive to small differential settlements between adjacent piers. This often leads to the adoption of piled foundations in soils of relatively high bearing capacity. Construction of shallow spread foundations is usually much cheaper and far more trouble-free than piling operations. Hambly (1) reported that the cost of piled foundations for a bridge may be comparable to that of the deck. Therefore designers should give careful consideration to the reliability of forecasts of differential settlement, and to the effects of rate of settlement of the foundations at the successive stages of construction of the bridge superstructure before deciding that piled foundations are unavoidable.

THE URBAN SITE

The selection of form of the substructure for projects in an urban environment is influenced not only by the layout and function of the superstructure but also by the proximity of existing bridges, highways, railways and underground services. At the construction stage there is the need to avoid or minimize nuisance to the public by noise, construction vibrations, dust and mud.

In good ground conditions the construction of shallow spread foundations should not present any special difficulty or nuisance and, as in the case of the green fields site, the structural form is usually a simple downward extension of that of the superstructure.

In weak ground which necessitates the provision of piled foundations consideration of noise, vibrations and lateral ground displacement usually rule out the selection of driven displacement piles, although small displacement steel piles have been used on sites surrounded by existing buildings, and considerable progress has been made in developing techniques for absorbing the noise of the impact between pile and hammer.

A wide range of construction equipment is available for installing bored and cast-in-place piles for moderate to very heavy loadings. In particular the auger-injected pile in which cement mortar or concrete is pumped down the hollow stem of a continuous spiral auger is a suitable type for installation close to existing buildings. The barrette foundation, constructed by grabbing under a bentonite slurry to form rectangular, tee-shaped or L-shaped excavations filled with concrete placed by tremie pipe beneath the slurry, is also a suitable method of construction for work close to existing structures.

Piling is not the only method which is available for the construction of deep foundations in the urban environment. In Hong Kong extensive use is made of deep shaft foundations constructed by hand digging in a circular excavation about 1 to 3m in diameter (2). The excavation is taken out in 0.6 to 0.9m deep stages. Concrete cast-in-place behind circular formwork is used to support each stage.

The site of the 50-storey tower of the Development Bank of Singapore was very narrow and was bounded on two sides by heavily trafficked roads and on the other two sides by deep basements beneath the podium structures of the new development. These restricted site conditions together with the presence beneath the site of deep fill and soft alluvium followed by steeply dipping and highly weathered sedimentary rocks led to the selection by the consulting engineers, Steen Sehested and Partners, of hand excavated circular shaft foundations beneath each of the four columns supporting the tower (3). Each column carried a load of 180 MN and the 7.3m diameter shafts were taken down to depths between 40 and 64m to reach moderately weathered rock of the required bearing capacity.

The techniques of constructing retaining walls in the form of a concrete diaphragm cast-in-place or in precast concrete slab form in a trench excavated with support by a bentonite slurry has enabled structures such as deep basements, cut-and-cover underground railways and highway underpasses to be constructed in close proximity to the foundations of existing buildings. These forms of construction are the first choice in weak or water-bearing soils but other methods which are available include ground support by contiguous or secant bored piles, or concrete panels cast-in-place between vertical steel H-section soldiers. Hand excavated caissons were used in conjunction with concrete jack arches and steel Universal beams for the retaining walls of the Diamond Hill Mass Transit railway station in Hong Kong (4). These caissons are constructed by excavation to founding level in stages of 0.6 to 0.9m. The ground at each stage is supported by concrete cast-in-place behind circular formwork.

Retaining walls constructed in a trench or as contiguous bored piles are frequently used in conjunction with permanent support by intermediate floor slabs or by deep horizontal reinforced concrete walling beams in order to provide a clear working space for excavation within the retaining walls of deep basements (Fig 1).

An important adjunct to the design and construction of deep basements in the urban environment has been the development of instrumentation to monitor the vertical and lateral deformation of the ground beneath and in areas surrounding the basement as excavation proceeds (5). Although decisions must be taken on the design and construction method for the basement before the instrumentation is installed, the records obtained of ground movements are an essential check on assumptions made at the design stage, particularly on the effect of the excavation on surrounding buildings. More important, the case records are of value to the future planning and design of similar structures when the engineer can proceed with greater confidence on his ability to predict ground movements surrounding a deep excavation. Such accumulated data from the monitoring of deep excavations in central London enabled engineers to proceed confidently with the construction of an underground car park within an 18.5m deep excavation sited a few metres away from the Big Ben clock tower and the historic Westminster Hall (6).

If possible, piling should be avoided for the foundation of bridges close to existing railways in restricted urban sites. This is because of limitations imposed by the railway authorities on the type of plant which can be operated close to their running lines, and on the possibilities of deformations to rail tracks caused by ground heave when displacement piles are used, or loss of ground with bored pile installations. Shallow spread foundations should be adopted if at all feasible, in conjunction with simply supported bridge decks where settlements are expected to be of appreciable magnitude.

The techniques of pipe jacking have been developed to enable large units to be jacked horizontally over considerable distances thereby providing a means of constructing bridge pier foundations and abutments beneath existing roads and railways without disrupting traffic flow. These techniques were used for the foundations of Old Ford Bridge in London (Fig 2). Three tiers of precast concrete units were used for the abutments and central pier. Each tier was jacked horizontally from thrust pits constructed at the ends of the three substructures (7).

REDEVELOPED INDUSTRIAL LAND

In Britain extensive redevelopment is being undertaken on land previously occupied by heavy industries such as steelworks, foundries and shipyards. These sites are characterised by deep deposits of loosely-placed miscellaneous fill material often concealing massive brick and concrete foundations, pits, culverts, and worked-out quarries. The fill material may include chemical wastes which are aggressive to buried concrete and steel in new foundations, or which may undergo large unpredictable settlements as a result of solution by seepage water or by biological degradation. Some fills may contain materials liable to spontaneous combustion when exposed or disturbed.

Redevelopment of these sites often takes the form of light industries and warehousing, making it possible to construct the new buildings on shallow pad or strip foundations. Settlements of the foundations can be minimized by subjecting the levelled and graded ground surface to heavy dynamic consolidation by the dropping weight method. This avoids difficulties in installing piles through ground containing massive obstructions, and the need for protection of pile shafts against aggressive conditions in the fill. Fills containing degradable waste material may have to be removed and replaced by inert materials beneath the sites of individual structures.

SWAMPLANDS

Swamplands in tidal estuaries or in the deltas of major rivers are characterised by soils of highly variable characteristics and compressibility. Soil characteristics vary at close intervals as a result of changes in the course of the river or its distributaries within the meander belt or in the zone influenced by encroachment of the sea. Deep channels filled with peat and highly compressible organic clays are flanked by sand bar deposits of moderate to low compressibility. The seaward margin of deltas may be occupied by a belt of sand dunes.

Development of swamplands usually involves reclamation to raise ground levels above river flood or tidal levels and to allow effective drainage of building sites. The imposed loading from a few metres of reclamation fill has the effect of accelerating the natural process of consolidation of the swamp soils and large settlements of the ground surface may continue for a period of years. Henkel (8) described how a total thickness of 7.5m of pumped sand fill was required to raise the ground surface by 2 to 2.5m in the reclamation of swampland in Lagos where former tidal creeks had been infilled with very soft organic clay. Settlements continued for more than five years. On the other hand settlements of less than 0.2m have recently been recorded as a result of pumping-in about 3m of hydraulic fill to reclaim swamps in another area of Lagos where the subsoil is predominantly sandy.

Detailed subsoil exploration of a deltaic site can delineate zones of compressible clay soils enabling buildings to be sited on the less compressible sandy deposits. Shallow spread or slab raft foundations can be adopted on the reclaimed ground surface. With careful attention to detailing of the foundations and superstructure high quality buildings can be constructed on shallow articulated reinforced concrete slab and beam foundations even in conditions where the reclaimed ground surface is underlain by soft compressible clays. The foundations of an office building on a reclaimed site near San Mateo, California, are shown in Fig 3. At this site 1 to 1.2m of fill were placed over 12 to 13m of soft bay mud (9). Precast concrete panels were used between reinforced concrete columns to stiffen the walls of the structure. Settlements had exceeded 0.3m after a period of 6 years and were still continuing but with no structural damage being reported.

Piled foundations should be avoided if at all possible because of the need to provide for high dragdown forces on the shafts of piles and the likely need to take the piles down to a considerable depth to reach a competent bearing stratum. Where piles are necessary for bridge foundations severe difficulties can be caused by large relative settlements between the bridge structure and embanked approaches.

When hydraulic fill is allowed to drain above ground water level it becomes compact and capable of supporting shallow spread foundations with moderately high bearing pressures. However if the fill is placed through water, as in the seaward extension of a reclamation project, it remains in a very loose compressible condition. Where fills are dumped through water on to very soft seabed mud the resulting massive lateral displacements (mud-wave formation) can continue over a long period of years, causing severe difficulties with piled foundations in the reclaimed areas. The dynamic consolidation process can be used to compact the ground with depth thereby avoiding the need for piling. If required the dropping weight can be operated through water to compact fill placed in layers by bottom-dumping barges for the foundations of heavy structures such as dock installations and breakwaters.

ARID REGIONS

Intensive urban and industrial development of coastal lands in arid regions is being undertaken in several Middle Eastern countries. Soils in these regions frequently consist of alluvial silts sands and gravels, uncemented or weakly cemented by calcium carbonate or chloride salts. The cemented soils are hard in consistency enabling high bearing pressures to be adopted for shallow foundations. Perversely however the principal difficulty in foundation design in these arid regions is caused by water flow when surface erosion and weakening of the cemented soils occurs under sheet floods originating in mountainous regions inland.

It is necessary to take foundations below the zones of potential erosion in the form of deep strip or pads, and bearing pressures should be based on the characteristics of the uncemented soil layers. The design of piled foundations should take into account the possibility of the installation method, whether by driving or boring, causing breakdown of the cemented structures of the soil and weak carbonate rocks, with consequent loss of skin friction and end bearing resistance. Equipment must be capable of either driving piles to a considerable depth to get below zones of potentially unstable or cavernous soils and rocks, or of drilling deep cased holes in difficult ground for some form of cast-in-place pile.

Collapsing soils may be present in arid or semi-arid regions. These soils, of which loess is a typical example, consist of an open-structured agglomeration of weakly cemented fine particles. When in a dry undisturbed state they have an appreciable strength and low compressibility, but when exposed to flood water or to seepages from leaking drains the cementing medium may be destroyed causing collapse of the soil structure and massive subsidence. Sometimes these soils are treated by prewetting before constructing shallow spread foundations. Abelev (10) described the installation of deep foundations formed from columns of compacted wetted soil placed in holes formed by driving in a mandrel, or by firing a string of explosive charges suspended down a small diameter drilled hole.

ARCTIC AND SUB-ARCTIC REGIONS

A feature of these regions is the depth of soil affected by frost penetration, and the problems caused to the engineer by the effect of freezing and thawing of the soil on the design of foundations. In sub-arctic regions the ground may freeze in winter to depths of a few metres causing substantial heave of the ground surface. The spring thaw causes collapse of the heaved ground. It is the usual practice to take foundations below the zone of seasonal freeze-thaw by means of basements, piers or piles. Where pier foundations are constructed in drilled excavations the uplift ("adfreezing") forces can be minimized by surrounding the piers with a layer of non-frost-susceptible gravel.

Foundation problems are of much greater severity in arctic regions where the ground is permanently frozen. In these regions the "permafrost" is blanketted by a few metres of soil which is subject to seasonal freezing and thawing. The permafrost can extend to very great depths, as much as 1500 metres having been recorded. Permafrost soils are highly compressible and creep substantially under foundation bearing pressures as a result of recrystallization and repacking of ice within the pores of the frozen soil. Permafrost also contains unfrozen water which can migrate from one zone to another as a result of temperature or pressure gradients in the soil mass with consequential large volume changes.

Where structures cannot be founded on stable ground such as rock outcrops, the principle generally followed in foundation design is to avoid, as far as practicable, any change to the regime of the permafrost. This is achieved by elevating the structure above the ground surface so that the soil is exposed to freezing winds, and with the minimum of heat transfer from the building to the ground. The foundations take the form of piles set into holes drilled deeply into the permafrost (11). Driven piles are not used because of the risk of splitting the ground thus forming channels for the flow of unfrozen water. Holes for the piles are drilled by mechanical auger, or in the USSR by reverse circulation drilling with steam jets, or by gas burners to provide the circulating medium in the form of a hot air blast.

EARTHQUAKE REGIONS

Construction processes have an influence on the selection of structural form for foundations in an earthquake region. If, for example, investigations to determine the liquefaction potential of the soil show that the ground may become unstable under earthquake-induced vibrations, deep compaction techniques using the dropping weight method or the insertion of mechanical vibrating units can be used to consolidate loose granular soils. These techniques can eliminate or greatly minimize the susceptibility of the soil to liquefaction, thus enabling conventional shallow foundations to be used. It is then necessary to ensure that there is full continuity between the foundations and superstructure, and that pad foundations or strip foundations in crosswall structures are tied together by transverse ground beams.

Fine-grained soils such as silts and clays cannot be compacted effectively and where heavy foundations are sited on weak compressible fine-grained soils, then some form of piled foundation is required. Driven piles are likely to be the favoured type because of the capability of cyclindrical steel or continuously reinforced precast concrete piles to assume a curved shape caused by variations in lateral ground displacement without failure in bending or shear. As already noted diesel or steam-operated hammers are available for driving large diameter preformed piles. Alternatively steel or precast concrete cylindrical sections can be installed in holes drilled by rotary mechanical augers or grabbing/oscillator rigs. Piling may have to be extended to considerable depths in order to reach a stratum capable of carrying the pile loadings in end-bearing. This is because of possible loss of skin friction on the pile shaft under earthquake-induced vibrations.

RIVER CROSSINGS

Selection of construction methods for the foundations of bridges over waterways is governed by the environmental conditions which in turn influence the design. If the width of the waterway enables the crossing to be made in a single span the piers or abutments can be located on dry ground with the construction site protected as necessary against flooding. Construction on a land-based site permits a wide range of techniques to be adopted. Selection of the appropriate type depends on the depth of excavation to reach foundation level, on the depth of any piled foundations, and on the general form of the substructure.

Where piers are required to be sited within the waterway then the dominating environmental conditions which affect construction methods are the depth of water, the velocity of flow, the depth of bed scour and the need or otherwise to maintain navigation in the river.

A conventional form of construction of river piers provides for steel sheet piling driven around temporary staging to form a cofferdam. Excavation for the pier base is taken after pumping-down the water in the cofferdam, or the excavation and pier base concreting may be performed under water. However in deep fast-flowing water it may be impossible to pitch and drive sheet piling, and in conditions of deep river bed scour cofferdams may be undermined. Although it is feasible to pitch and drive steel sheet piles in long lengths to take them below potential scour zones there is a risk of piles becoming deflected and interlocks becoming separated, particularly when driving the piles through ground containing large gravel, cobbles and boulders. Cofferdams are also liable to damage by collision with ships or barges, and where they form an obstruction to river flow under flood conditions the cofferdams may be carried away by pressure from the build-up of floating debris or ice.

The Japanese steel industry has developed a method of constructing large bases for the piers of river bridges in the form of interlocked large diameter steel tubular piles. The piles are locked together to form a peripheral structure of circular, rectangular or double D-shapes (Fig 4). This form of construction has a high strength to resist lateral forces from river flow and impact from vessels or floating debris, and the piles can carry high axial loads from the bridge piers. The piles, which may be one metre or more in diameter, are provided with circular interlocks and have been driven to depths of more than 40m below the river bed. In cases where the pier is located below water level the piles can be extended above the base to form a cofferdam in which the pier is constructed.

Caissons are an alternative form of construction for bridge piers in wide fast flowing rivers where the bed is subjected to deep scour.

Open well caissons are a traditional method of constructing bridge foundations on the Indian sub-continent, where the method is still in use to the present day. Electric power transmission towers are being constructed over the Jumana (Brahmaputra) River in Bangladesh. The river is about 11km wide from bank to bank at the site of the power line crossing. The main river channels can move considerable distances with islands disappearing and re-appearing elsewhere in one season. The calculated regime scour depth is 46m with local scour around foundations of up to 70m below high water level. For these conditions the consulting engineers, Rendel Palmer and Tritton, have designed circular caisson foundations, 13m in diameter, to be sunk to depths of up to 105m. Sinking the caissons through the sandy deposits is being facilitated by continuous circulation of a bentonite slurry from cutting edge level up the outer skin of the caisson.

The foundations of the main towers and anchorages of the suspension bridge over the Humber Estuary, the world's longest span, are an example of selection of foundations of differing types as determined by the environmental and geological conditions. The river is wide and shallow and the meandering channels are subjected to deep scour. Chalk outcrops on the north bank of the estuary where it was possible to construct the anchorage and main tower pier on conventional shallow spread foundations. The south main pier was constructed within the river and the consulting engineers, Freeman Fox and Partners, designed twin circular caissons 24m in diameter sunk by open-well grabbing through 40m of estuarine sands and gravels to a penetration of 7m into very stiff Kimmeridge Clay (Fig 5). Bentonite slurry was again circulated around the outer skin of the caisson to facilitate sinking. The south anchorage was constructed on land at a location where 27m of soft alluvial clay and glacial deposits were overlying the Kimmeridge Clay. It was necessary to take the foundation down to a depth of 35m to resist the 38 000 tonne horizontal pull from the bridge cables.

The 72m by 44m anchorage block was constructed in the form of a multi-cell structure constructed by diaphragm wall methods. By excavating the soil from the individual cells within the diaphragm walls swelling and softening of the Kimmeridge Clay beneath and around the anchorage was avoided (12).

MARINE LOCATIONS

The environmental conditions of winds, waves and tidal stream flow are the dominating influences in the design and construction of foundations in marine locations. For works near the shore such as jetties or island terminals for tankers or ore carriers the favoured method of construction is driving large diameter steel tubular piles. These have a high resistance to lateral forces from waves and currents, and their flexibility provides a means of absorbing energy from the impact of ships on the berthing structure.

Where lateral forces are limited to those from waves and currents, as in approach trestles to berthing structures, or in ore conveyor trestles, precast concrete cylindrical piles are suitable for easy to moderate driving conditions below the sea bed.

In sheltered waters piles can be driven from pontoons, but the use of jacked-up barges enables large diameter piles to be handled and driven with little interruption by sea conditions. Rotary or grab-type drilling equipment can be mounted on the barges for use where conditions below the sea bed require alternate drilling and driving to achieve the required depth of fixity for piles carrying lateral and uplift forces. These conditions were present in the Firth of Forth, where a tanker terminal for the British Petroleum Company was designed by Babtie, Shaw and Morton (13). Steel tube piles of 2.2m diameter were provided for the berthing structures. These were taken down to rockhead through ground containing large gravel, cobbles and boulders, by alternate driving and drilling out the soil from within the pile tube by a rotary full-face drilling bit and reverse circulation to remove the drill cuttings (Fig 6). On reaching rockhead the drill was used to form a socket for a 560mm tubular anchor grouted into the rock to resist a 7.4 MN uplift force.

The most severe environmental forces on marine structures are those prevailing where petroleum production platforms are sited in deep water. These conditions necessitate constructing the platforms in sheltered locations, towing them as a single unit to the deep water site and securing them to the sea bed as quickly as possible before the onset of storms which might undermine or overturn them.

Two basic methods have been evolved for designing the foundations of the deep water structures in the North Sea oilfields. The first is the open steel tubular structure designed either as a self-floating unit (Fig 7) or to be carried by barge to the site where it is sunk and pinned to the sea bed by tubular piles. The piled structure has the advantage of being capable of installation in a wide variety of soil conditions below the sea bed. It can be used where deep soft clays are present, or on a rocky sea bed where the base structure can be pinned to the rock by drilled-in anchors. The principal disadvantage is the need for attendance by heavy crane barges over the period of pile driving with costly delays in bad weather conditions.

The second basic design approach is the reinforced concrete multi-cell caisson in which wind, wave and current forces are resisted by gravity and the frictional resistance of the massive structure bearing on the soil at sea bed level. Undermining by scour is prevented by a steel skirt around the perimeter of the caisson base, and grout is injected to fill voids between the underside of the base and the sea bed. The principal advantage of the gravity caisson structure is that it can be towed to the sinking site complete with the drilling deck and accommodation units, thus involving the minimum period of attendance by floating construction plant at the deep sea location. However, the sea bed soil must be sufficiently stable to prevent excessive settlement or tilting under the high cyclic bearing pressures imposed by storm waves on the caisson structure.

The gravity base platform illustrated in Fig 8 was designed and constructed by Howard-Doris Ltd for the Ninian Field of Chevron Petroleum (UK) Ltd. The structure is 167.4m high from base level to the top of the deck. The base has a diameter of 140m and the water depth is about 135m at the North Sea location.

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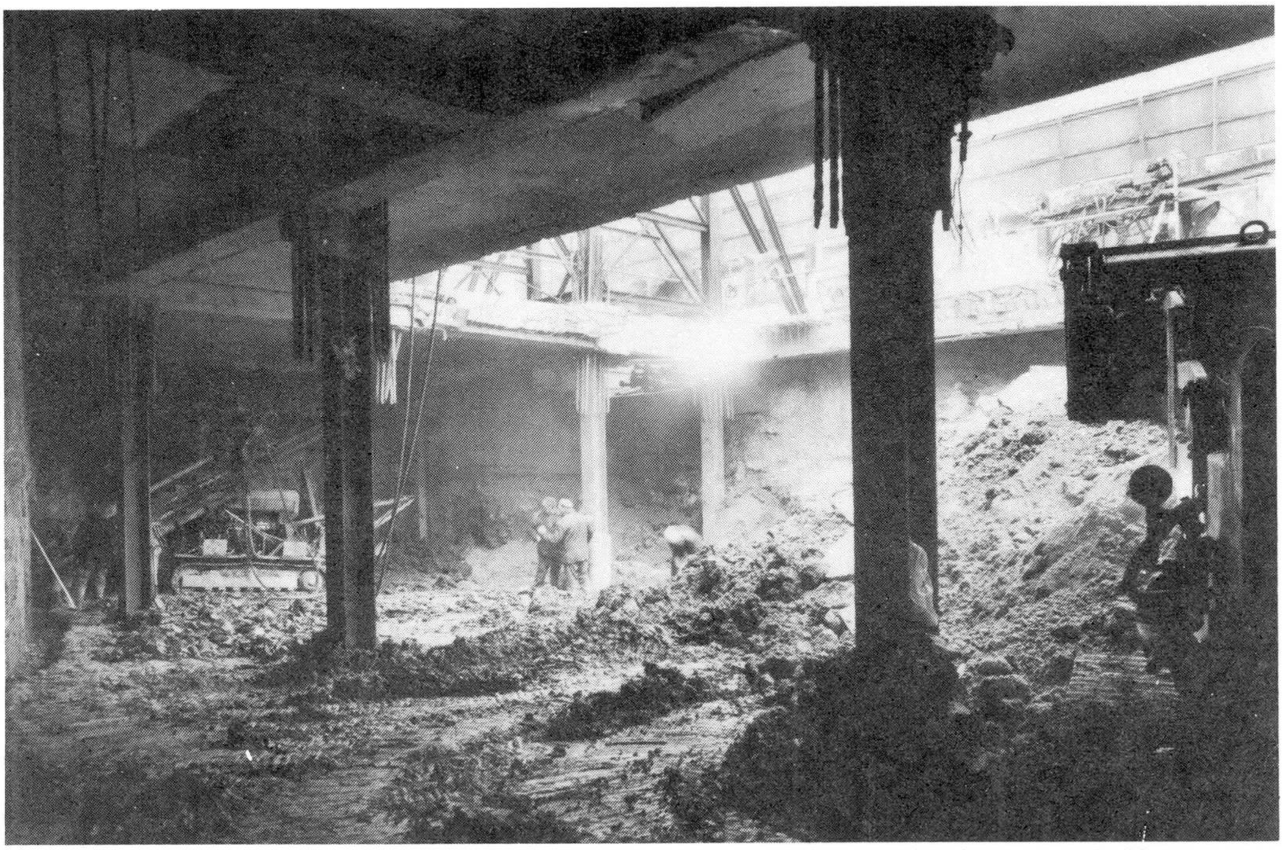

Fig 1 Ground anchor installation and excavation beneath the ground floor slab of a basement substructure, Victoria Street, London

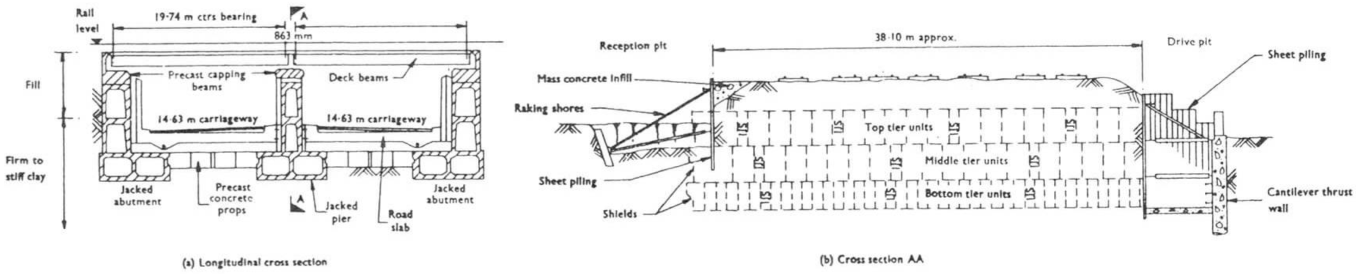

Fig 2 Three tier abutments and piers installed by jacking beneath railway at Old Ford, London

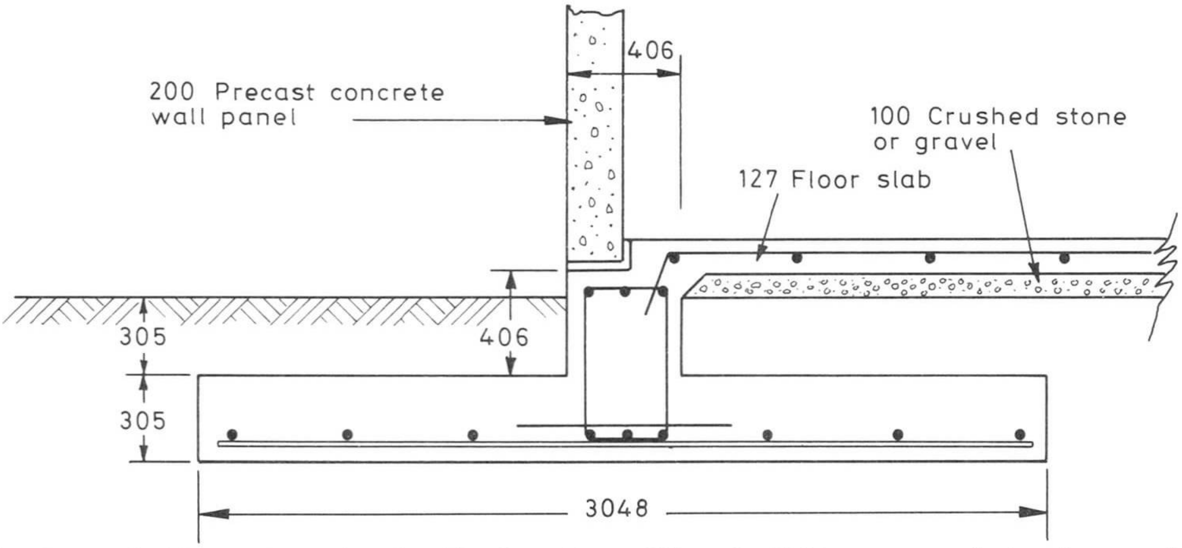

Fig 3 Shallow RC strip foundation for office building on reclaimed swampland at Foster City, California

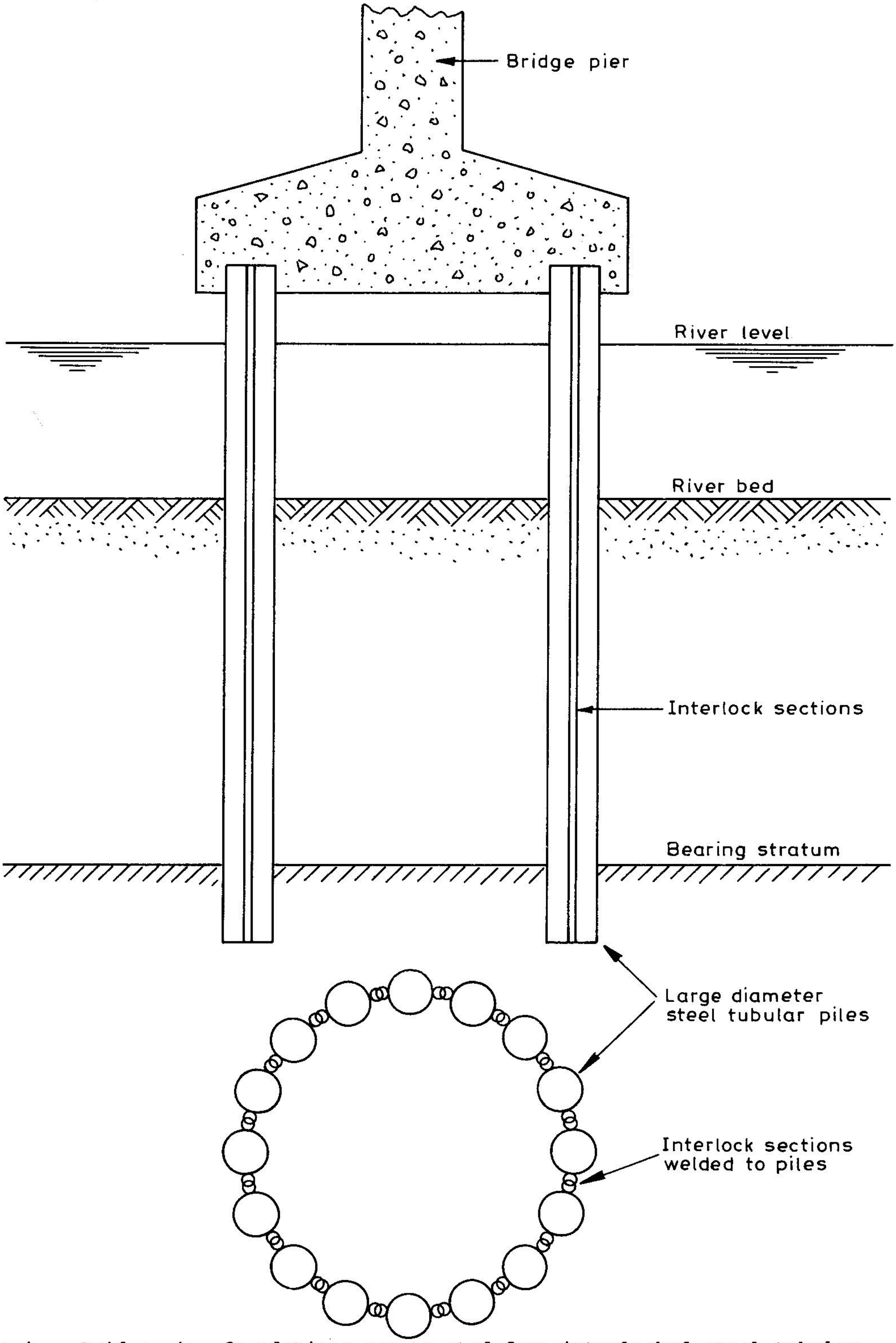

Fig 4 Bridge pier foundations constructed from interlocked steel tubular piles, Japan

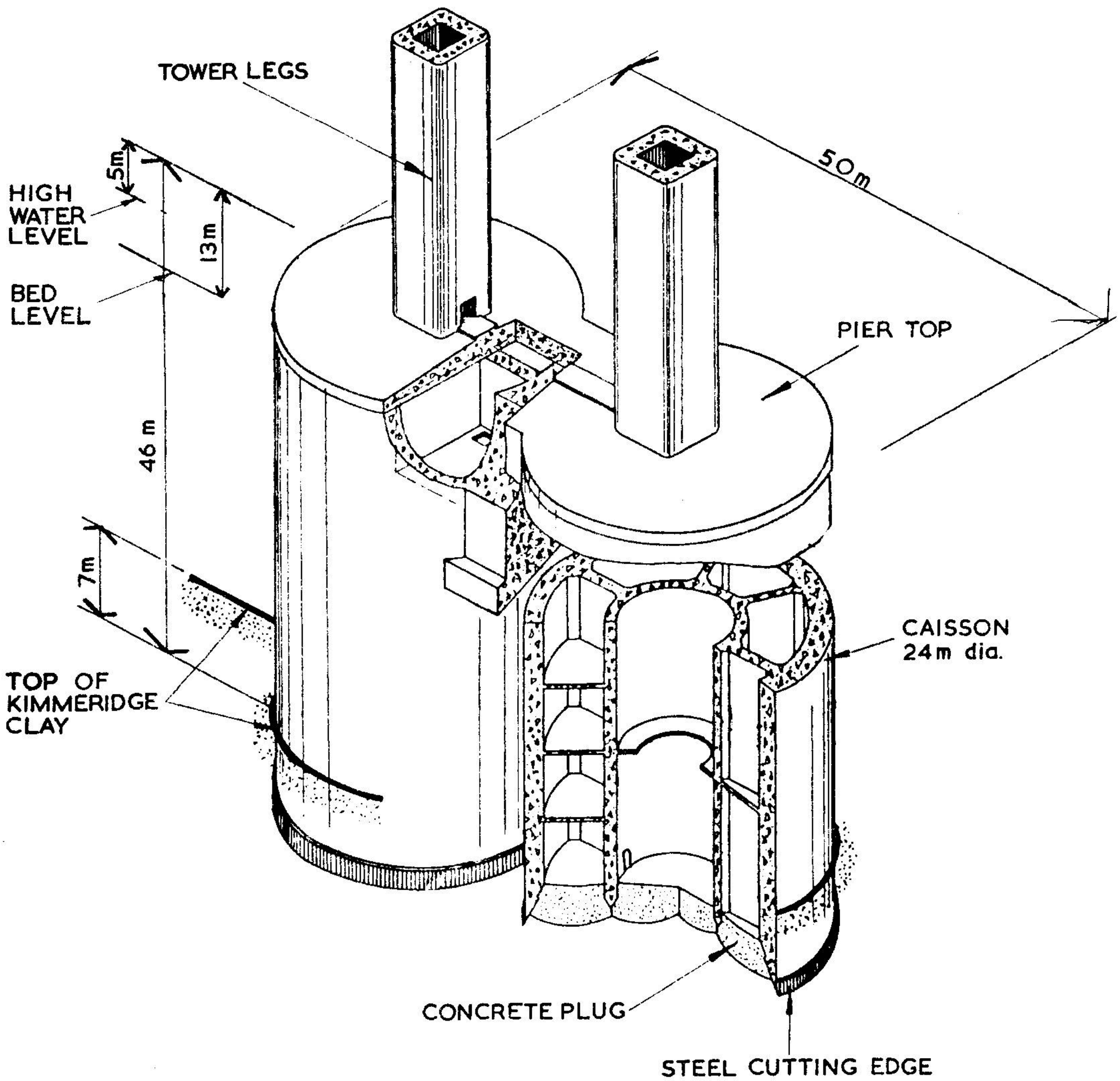

Fig 5 The foundations of the south tower pier, Humber Bridge

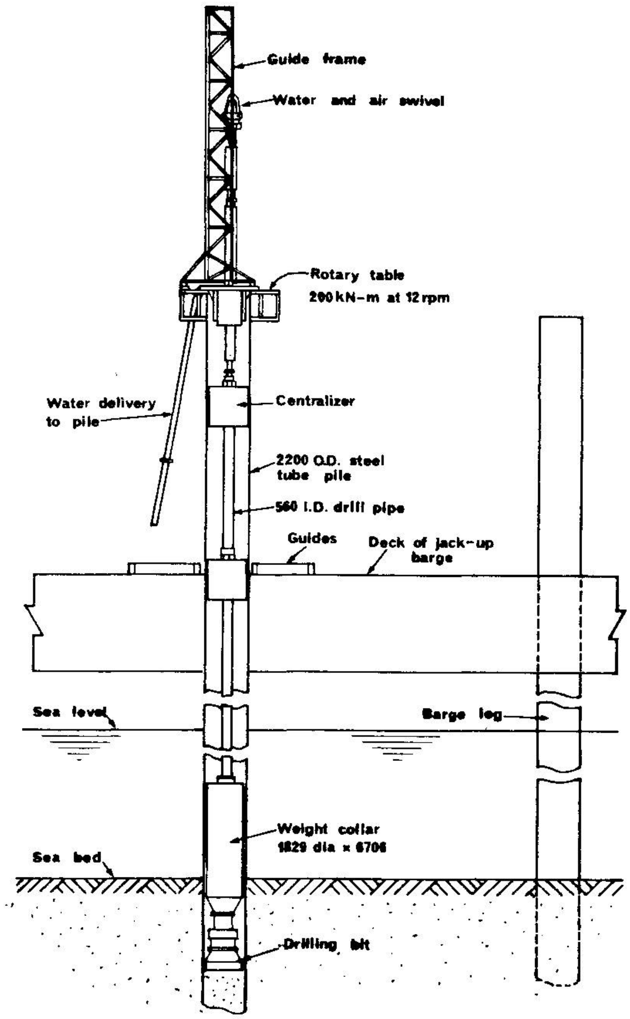

Fig 6 Installing large diameter tubular piles for berthing dolphins, British Petroleum tanker terminal, Firth of Forth

Fig 7 A self-floating petroleum production platform under tow from the construction dock of Brown and Root-Wimpey Highlands Fabricators Limited to the Ninian Field of the Chevron Consortium. The guides for the supporting tubular piles can be seen at each corner of the structure

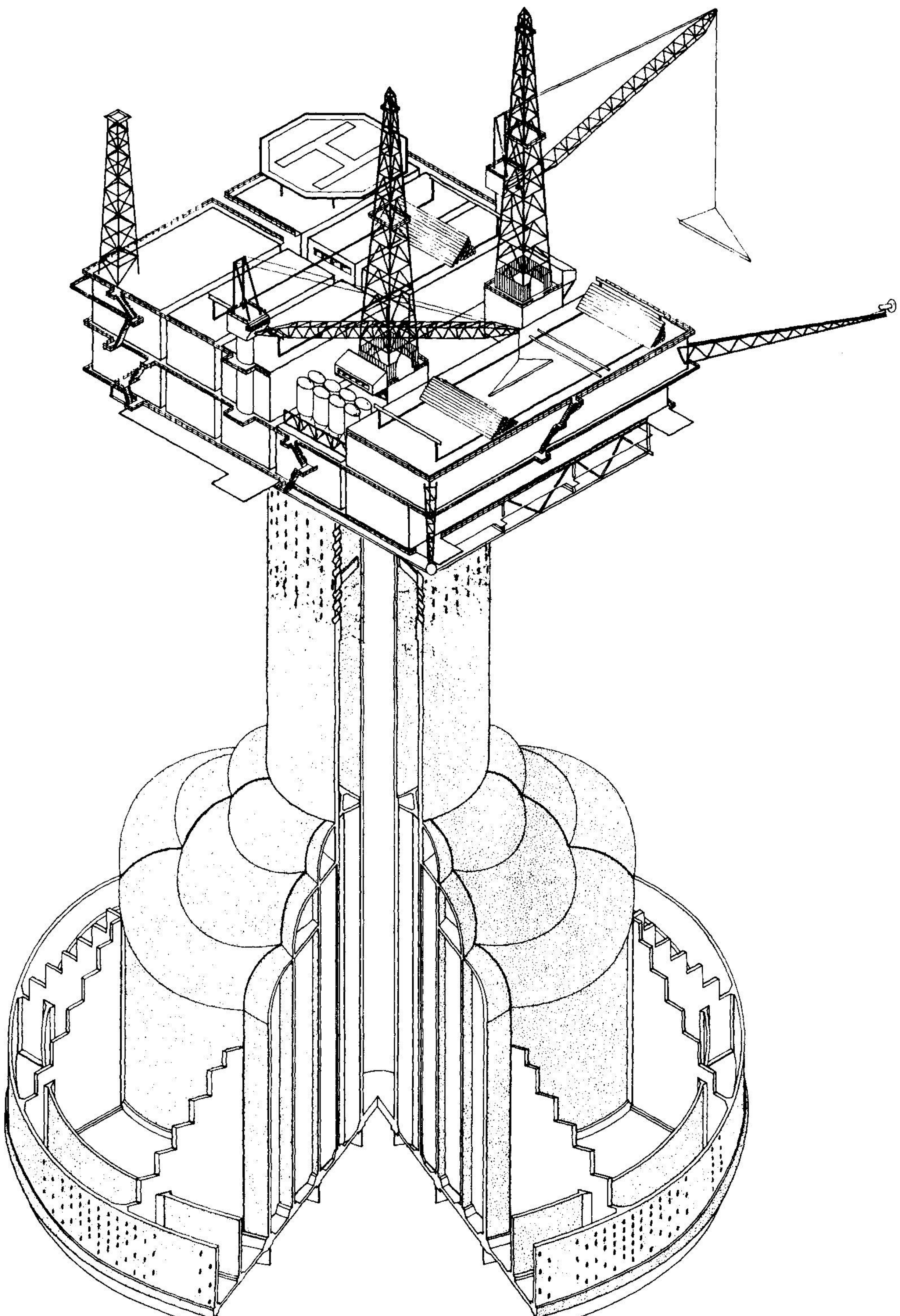

Fig 8 Doris gravity base structure for the Ninian field, North Sea

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Construction of superstructures in different environments

Construction de superstructures dans des environnements différents

Ausführung von Bauwerken in verschiedenen Umgebungen

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SUMMARY
During the construction of bridge superstructures it is possible to come up against problems, which have not been envisaged by the designer, because no definite construction method has been assumed at the design stage. In offshore construction it is the other way round, and it can be clearly seen, that the structures have been designed for transportability or to suit existing floating construction equipment rather than for their final permanent purpose.

RESUME
Lors de la construction de ponts il arrive de rencontrer certains problèmes qui n'avaient pas été considérés par le projeteur, car aucune méthode de construction n'avait été définie dans la phase de projet. Dans la construction en mer (off-shore), c'est souvent le contraire et il est possible de constater que les structures ont été étudiées en vue de leur transport ou de leur adaptation aux équipments flottants disponibles pour l'execution, plutôt qu'en vue de leur utilisation finale.

ZUSAMMENFASSUNG
Während der Ausführung eines Brückenüberbaus kann es vorkommen, dass der Ingenieur mit Problemen konfrontiert wird, mit denen er nicht gerechnet hat, weil beim Entwurf keine definitive Konstruktionsmethode festgelegt worden war. Bei der Konstruktion von Oel-Plattformen z.B. ist es gerade umgekehrt. Es kann mühelos festgestellt werden, dass diese Tragwerke eher für ihren Transport oder aufgrund ihrer vorhandener schwimmender Bauausrüstungen konzipiert sind als für ihren endgültigen Zweck.

1. INTRODUCTION

Superstructures come in many shapes and sizes,and the nicest shape and size from a construction man's point of view is a structure which looks complicated but turns out to be simple. This does not happen too often, but the future looks brighter since I learned that the British IABSE Group last year organised a Colloqium at Cambridge on the topic "Design for Constructibility".

I have limited the talk to bridge superstructures, and limited it even further to a few types, that I know from personal experience. However, since I am supposed to deal with different environments, I will include some thoughts on offshore structures, where it is very clear to see, that it is the environment and the construction method that dictates the design and not the structural engineer. Closer to land and in a friendlier environment it is the designer who starts the creative process, and a good designer given a specific task, of course, sets out to produce a structure which functions as required, is durable and looks good and costs as little as possible. The first three criteria may be comparatively easy to achieve or at least for the design team to agree on, but in a design office separated from the commercial world of construction it is impossible to be sure, that the chosen solution also is the cheapest.

With powerful computers and an abundance of ready-made computer programmes available it is to-day possible for any designer to analyse and optimise practically any structure and minimise the material content. A great number of very important factors are though missing in these programmes; such as labour and plant content, temporary supports, construction risks and - tolerances, complexity, weather dependancy, supervision etc., and any of these factors may influence the construction cost more than any saving in material.

In the old days, when it was a tedious and time-consuming job to solve simultaneous equations, it was a natural act of self-preservation to simplify complex structures to a simple assembly of components with a minimum of indeterminate interactions. With absolutely no fear of the number of simultaneous equations required for the structural analyses there is a natural tendency to ignore the complexity of the calculations and to let every bit of steel or concrete, if possible, play a useful structural role in all three dimensions. This may produce an extremely elegant and safe final product, but during the construction phases, when some of the interacting parts are missing and replaced by temporary supports, and other parts have not yet gained their design strength, the structure may at times be closer to collapse than designers or builders have realised, and I doubt that structures have become any cheaper because of the computer, since the most important cost determining factors have not yet lent themselves to mathematical and statistical determination.

Some years back the British Ministry of Transport undertook a major study of bridge prices in an effort to establish some kind of a price list for bridges. To that end they analysed thousands of Bills of Quantities for all kinds of bridges submitted by a great number of tendering contractors. The fact, that tender prices could be vastly different from the final cost figures, was ignored and only the B.Q. figures were compared item by item from handrailing and kerbstones to concrete and steel in all sorts of places and excavation in all kinds of grounds. The result was most disappointing. The variations of every single bill item were so great and the distribution from one end of the scale to the other so utterly haphazard that no logical conclusion could be drawn from any statistical analysis - with one possible exception. It did appear - to the complete bewilderment of the Ministry - that bridges became cheaper with increasing quantities of concrete per sq. meter of deck area. Having seen a few B.o.QS being filled in the last hour before the tender deadline, the haphazard pricing of individual bill items was not a great surprise, and the tendency to appear cheaper with increased average concrete thickness of the bridge deck was to me a clear indication that simplicity would pay off. Another member of the study team couldn't even draw that conclusion, since he could quote concrete in a 1 m thick flat slab which was priced exactly the same as the concrete in a 6 m deep web only 0.25m thick.

2. CANTILEVER CONSTRUCTION

It may be difficult to decide whether a design is good or bad, cheap or expensive, and I don't think it is possible to lay down simple guidelines for good design, but based upon experience from a few bridges and offshore structures I do think, it is possible to point out at least one approach, which should lead to a competitive and rational pricing process, and that is to have a definite construction method in mind when designing the structure and preferably a method, which permits the structure to grow naturally in size and strength, reaching completion without relying on an extensive system of temporary supports and passing through vulnerable and indeterminate intermediate stages.

Constructing vertical structures as tall tower blocks or chimneys would seem straight forward in a structural sense with the main problem being the logistics of bringing men and material to the very narrow working front on the top. But there is no limit to what the human mind can work out, and there are examples of vertical structures which have been built in reverse, starting with the roof at ground level and keeping on working at ground level while jacking the structure upward; apparently the natural way for grass to grow.

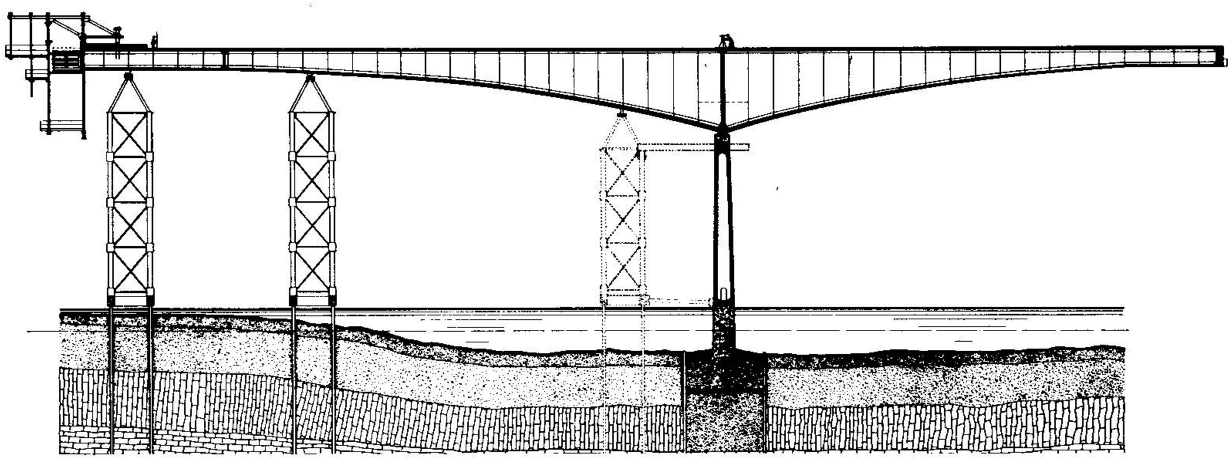
Even in horizontal construction there are examples of bridges, which virtually had to be built from the deck and downwards; but before we look at such examples, let us first look at a major bridge which could be built the right way round - more or less.

2.1 The Medway Bridge

The Medway Bridge on M2 from London to Dover was at the time of its construction the largest of its kind in the world and has been described in great detail in many technical publications. It has since been beaten several times, but there are aspects of the construction worth mentioning.

The sidespans were basically just conventional viaduct construction, albeit rather big. Piled foundations, twin columns, in-situ cross-beams, precast longitudinal beams and in-situ deck slab, with the work starting at the abutments moving out towards the main spans across the river. The edge beams were designed as concrete box girders weighing 200 tonnes. At the time these beams appeared to be super-heavy-weights, and they required purpose-made launching equipment, but it was only a matter of scale. It was exciting to push the launching girder across to the next pier and to roll the precast beams across, but theoretically there was no problem. The structure was at no time stressed more critically than in its completed state.

The main river span of 500' was designed as two cantilevers of 200' and a suspended span of 100'. The construction of the two cantilever arms were obviously intended to be in accordance with the well-established way of free cantilevering, and the suspended span was to be of a similar construction to the viaduct spans. The construction of the two 312' long anchor spans was not so well-defined and caused a few theoretical and practical problems.(Fig.1)



The free cantilevering may appear a daring way of construction, but it does in fact permit the bridge to grow in a simple and natural way. As the cantilever moment grows bigger, the prestressing force is increased, and properly designed one can maintain almost uniform stress from deck to soffit of the cross section, and thus in spite of a very large dead weight moment have rather small resulting deflections.

The longer anchor spans don't lend themselves automatically to cantilever construction, since part of the span is in positive bending in its final state. Furthermore, without the suspended span in place and in consequence without the negative moments over the main piers having attained their minimum design values, the anchor spans were hardly self-supporting. To add to the complications one had to launch the launching girder and the precast beams across the anchor spans at this vulnerable stage, which called for temporary support towers and a very careful control of the indeterminate support reactions.

A more rational and "construction-friendly" and therefore probably cheaper design would have been to eliminate the suspended span and to continue the cantilever construction right to the centre of the main span, perhaps even increased the main span to permit a balanced cantilever construction of the two anchor spans.

2.2 Taf Fawr - a smaller Medway

An opportunity to try out in practice some of the lessons learnt on the Medway Bridge appeared a short while later, when an alternative proposal for one of the bridges on the Heads of the Valleys Road in South Wales was accepted.

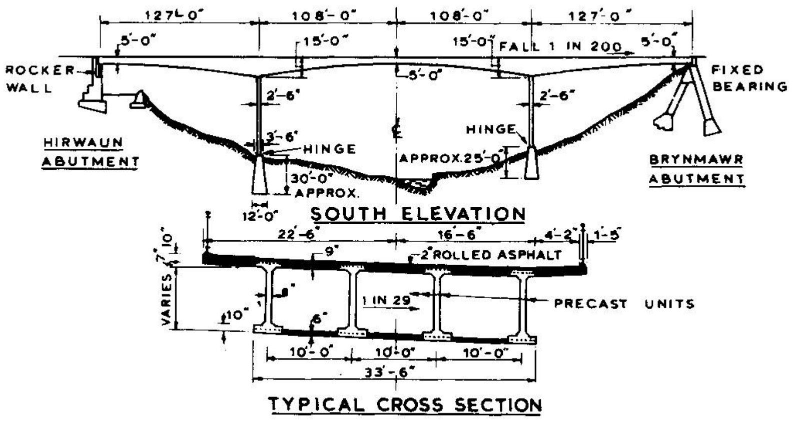
The Taf Fawr bridge crosses a 100' deep valley at Merthyr Tydfil in South Wales and it was decided to design it as a 3-span bridge, 127'-216'-127' to be constructed by free cantilevering from the two piers to meet in the middle and then be stressed together to form a 3-span continuous bridge. (Fig.2)

Another lesson learnt from concrete bridge construction was to be taken into account, and that was to simplify and reduce to a minimum the in-situ construction. The worst parts of a concrete box girder to construct are the thin webs. They are expensive,because they require more formwork than the soffit and the deck, and the concrete is difficult to place, and if anything goes wrong, it is almost impossible to carry out a neat looking repair job. Furthermore, to add insult to injury the webs constitute a major proportion of the structural weight without adding much strength, since steel generally has to take the shear.

As a first step towards the simplification of the web construction it was decided to form the box girder by having precast I-beam sections and casting in-situ only some soffit and deck slab concrete. This has certainly as a result, that the so-called cantilever carriages were reduced to simple strongbacks supporting the I-beam sections, which in turn supported a simple flat soffit shuttering and working platform.

The in-situ construction was certainly simple and the structure behaved nicely as theoretically predicted, but there was obviously room for improvements.

THREE SPAN, PRESTRESSED PRECAST CANTILEVER CONSTRUCTION 667



Each box section consisted of 4 No.I-sections and with the bridge spans being symmetrical it had been assumed there would 16 No.

identical elements in each construction sequence. That was quickly proved wrong. The bridge was on a curve and had a cross fall, and the arrangement of the prestressing tendons caused further complications, so that in fact there were very few identical elements.

It was decided to cast the I-sections in a casting yard some 30 miles from the bridge site, and having completed one set of 16 elements it was of course natural to cut the forms down for the next smaller section further away from the piers. Unfortunately one of the elements managed to fall off the lorry on the way to the site and had to be remade, but by then the formwork had already been modified to the smaller section. Not only became the remade section a very expensive one, but the construction of the bridge came to a standstill, while it was being fabricated.

The real lesson to learn from this bridge is not, that a precast element can be damaged en route from casting yard to the bridge site, but rather that it was a mistake to make the precast elements I-shaped and to cast them vertically. The difficulty of casting webs was not eliminated - only removed from the site to the casting yard - and the quite large proportion of precast concrete in both the deck and the soffit slab was a design disadvantage. The restraining effect of the precast concrete upon the fresh in-situ slab concrete caused tension, which had to be eliminated by additional prestressing. The obvious answer to both of these drawbacks would have been to make the precast web members flat and to cast them horizontally. Not only would that make all the concreting in the superstructure simple and reduce the prestressing but it would also make the entire method much more flexible, make room for all the prestressing tendons in the in-situ parts and permit curved alignments and crossfalls without loosing the benefits of symmetry around the supports.

It is encouraging to see, that a bridge over the river Coquet in North England just has been constructed that way, and it shall be interesting to see, if it has been a success and will be repeated. It may however have to wait a while, when we consider that there is a gap of 15 years between Taf Fawr and Coquet; and I would still like to see one more step forward.

The webs should not be made of concrete at all - but of steel, since steel in any case is doing most of the work. By making the webs of flat steel plates suitably stiffened one would on a major bridge save a considerable amount of weight and be able to span longer spans economically. The fabrication of the web members would be extremely simple and the handling and joining of the members with friction grip bolts would be equally simple, have no serious problems of tight construction tolerances, highly skilled labour content or danger of unknown built-in stresses.

External Prestressing

Still, even with the greatest simplification of concrete box-girder construction there is a limit to it's economic range, and at some stage it is easier to keep the box size constant and increase the possible span of the bridge by introducing intermediate supports from above. One has reached the range of the cable stayed bridge, which from a pure construction point of view is a simple and natural extension of the balanced cantilever method. When the cantilever moment reaches the design limit, the bridge is given a "lift" in the form of an inclined external prestressing cable, and the cantilevering can continue. The bridge is growing longer and stronger in a natural way and at no stage during construction vastly different in behaviour or more vulnerable than the completed structure. In that respect it would appear a much simpler and safer proposition than "big brother", the suspension bridge, which must pass through a whole range of varying weights and stiffnesses and corresponding natural frequences and "resonant" wind speeds.

I have, I am sorry to say, not had the opportunity to construct a cable-stayed bridge or a suspension bridge, but I have tried to support a bridge from above during it's construction.

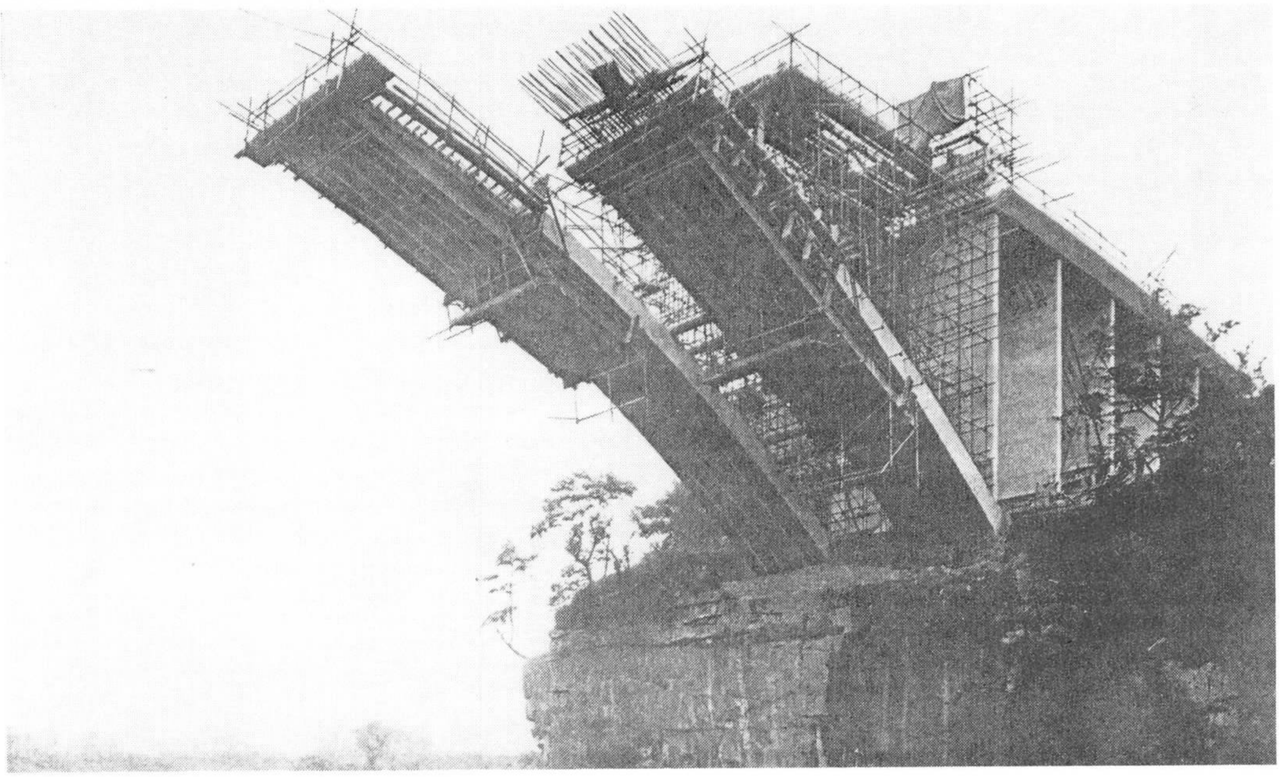
Arch bridges lend themselves best to crossing of deep valleys or rivers with steep and rocky sides, and even if many of the valleys have been very deep and quite inaccessible, there are still plenty of examples of concrete arches constructed on falsework supported on the valley floor. There are also plenty of examples where it was clearly impossible to use or reach the valley floor and the concrete arches were constructed on temporary steel or timber arches. In such cases by a carefully chosen sequence of concreting of the various sections of the arch it is possible to end up with a complete, fully fixed arch rib almost without built-in stresses in spite of considerable deformations of the falsework due to the weight of the concrete. The art is to end up with an arch with as small bending stresses as possible, after the falsework has been removed, and the entire self-weight of the structure has been transferred to the arch.

On the Heads of the Valleys road there were 3 fully fixed concrete arches, one over a water reservoir and two over deep and rather inaccessible valleys. For the largest arch and the one over the reservoir it was decided to try something new, and in the case of the third one the valley was filled up with conventional scaffolding.

The largest of the arches over Faf Fechan at Cefn Coed had a span of 236' and the crown was about 100' about the valley floor, and it was decided to construct the arch ribs by straight forward cantilevering from the two abutments (Fig. 3).

The bridge was supported on two parallel arch ribs, approx. 3 m wide and 0.6 m thick at the springings. These ribs were of course far too thin to be able to cantilever to the middle of the span, so in each construction segment of the arch rib were cast in a number of 0.5" prestressing strand anchors, and 0.5" tendons were taken to an anchor beam on a rocker bearing above deck level at the abutments. Another set of tendons were taken from the same anchor beam to rock anchors beneath and behind the abutments.

When the concrete in a section had attained the required strength, the weight of that section was "eliminated" by the stressing of some of the cast-in suspension cables and the corresponding anchor cables. The cantilever formwork could then be winched forward and another section of the arch be concreted. As the construction approached the crown of the arch, the suspension cables produced increased compression in the arch, and some of the earlier cables could be destressed and removed.



It was very noticeable that the structure became extremely flexible, as the work progressed; and as the entire construction method was a new one for everybody in the team, it caused a certain degree of anxiety, when the man on the level could see the bridge move about, when someone just walked to the end of the arch. It was, however, soon realised that the corresponding stresses were insignificant, and that the levels could be adjusted by applying just a slightly different force to some of the prestressing cables.

It might appear a little worrying, when the free end of the bridge moves, so you can see and feel it; but it certainly makes it easier to make the two halves meet in the middle, and it is very satisfying to know, that one is in charge of the stress distribution in the arch, when the final closure takes place.

From a purely technical point of view there can be very little doubt, that cantilevering and the use of external prestressing is a correct way of constructing an arch bridge, and the step from an arch to a straight cable-stayed bridge would appear quite natural.

From a cost point of view it is possible, that Taf Techan was too small for the method to be really competitive, but for larger spans there can be no doubt, and a much larger arch over Van Staden's Gorge in South Africa has been built since using the same method.

A concrete arch rib may not be the most difficult structure to construct an conventional falsework, but then try an in-situ multi-span continuous prestressed bridge comprising a grid of longitudinal box beams and transverse diaphragms and with prestressing tendons extending the full length of the bridge. Each additional concrete pour causes movements of the falsework, which the neighbouring green sections can't follow and haven't enough strength to resist, until at the very end of the construction the structure is provided with tensile strength through the prestressing. To control the deformations of a flexible arch rib by means of prestressed suspension tendons is absolute child's pay compared to the problem of preventing cracks in a rigid but green concrete structure on conventional falsework.

If it is necessary to cross a valley with a multispan structure and cantilevering from pier to pier may be considered too slow, it may very well be better to think big and provide travelling formwork, which can span from pier to pier and carry the entire weight of a complete span. The specialised formwork is expensive but the saving in labour and construction time by concreting a whole span in one continuous operation and having what amounts to a travelling bridge factory can still make it a competitive proposition.

I have so far concentrated my talk on construction of concrete bridge superstructures in different environments, because my own experience is limited to that field; but as far as I can see, the same principles would apply to steel bridges. If they are designed to grow longer and stronger in a natural way without relying on extreme accuracies, exceptional material strengths and a highly skilled labour force and supervision staff and - of course - without requiring a complex system of temporary supports, they are bound to be safe and cheap to construct.

I am however still looking forward to having a go at, what I would guess to be the simplest superstructure to construct, a box girder with in-situ concrete deck and soffit slabs and flat steel webs with friction grip bolted joints.

Offshore Structures

In the case of offshore structures it is a little difficult to decide what is superstructure, and what is substructure; but to give myself a chance to talk about the most interesting part I will concentrate on the section between the seabed and the water surface.

Most offshore structures are connected with oil production, and it can not be denied, that the development since the early "Texas Towers" has been quite impressive; but if we ignore the North Sea for a moment and compare the steel towers with the wide variety of bridge structures, it is quite obvious, that offshore construction still is in it's infancy. The towers have just grown bigger, until they outgrew themselves and stretched the technology to the limit, so that new ideas had to be tried. The North Sea was the limit, and the giant concrete gravity structures were given a chance.

But can it be said, that these heavy weights really represent a step forward?

To answer that question it may be useful to have a closer look at the pile-supported steel structures, to see what we can learn from them.

Steel Structures on Piles

The foundation method has always puzzled me. Why is one happily prepared to make huge piling hammers, drive piles of colossal dimensions to refusal and finally to rely on underwater grouting to tie piles and structure together instead of trying to tackle the apparently no more onerous task of digging some footings into the seabed?

But that is another story and not the subject of this talk.

Some positive lessons are however to be learnt from this puzzle, and that is, that huge piles can be manoeuvred into the pile guides, and heavy hammers can be landed on top of the piles, and later an modules weighing thousands of tons can be placed safely on top of the tower structure. There are therefore calm spells long enough to make these marriages of floating and fixed objects offshore possible; but, of course, there are also rough spells, which in one day can undo a whole season's good work. One must therefore plan the offshore work to consist of a minimum of short duration mating operations, and each operation should if possible leave the structure safe against subsequent attacks from the environment.

The installation of pile-supported steel structures does not meet this ideal requirement. Sometimes the pile driving takes too long, and the structure has to be left only partly supported, perhaps for months through the roughest part of the year.

Strangely enough, experience shows, that structures do survive such winter seasons without proper support. They have obviously been rocking about, but have remained structurally intact, so if the structure had contained some sort of living quarter, it would have been possible to stay on board, and if the foundation method had been self-contained and required only moderately sized equipment, it would have been possible to carry on working through the rough season. When one furthermore takes into account, that there generally are short calm spells even in the stormy seasons, one would not necessarily be marooned for long periods on the structure.

When the oil in the Northern North Sea was discovered, and it was realised, that one had to build bigger and heavier structures than ever before and during a shorter "working window" than before, it was feared, that existing designs and construction techniques would prove inadequate, and for the first time a completely new approach was considered.

Gravity Structures

From one extreme of having too much offshore work one went for the other extreme of having no offshore work at all, and the aim was now a structure, which could leave the inshore fabrication base complete in all respects and ready for production, as soon as it touched down on the virgin seabed. The concrete gravity structure was conceived, and a variety of proposals were produced, out of which only 3-4 solutions have been successful, in so far as they have been built and installed.

But have they really been successful? Are they really the natural next step forward from the pile-supported structures? Is it sensible to cut the offshore work out all together, when it is an established fact, that there are spells of calm weather offshore, where it is possible to bring a large floating object safely in position on a bottom-supported fixed object?

Some 15 years ago I had the experience of designing and constructing a gravity structure in the Irish Sea off Dublin. It was the Kisk Bank Lighthouse, and it was telescopic and attracted a certain attention at the time. The telescopic design was introduced by necessity to obtain floating stability during all stages afloat from the inshore construction base to the final touch-down on the seabed in 20 m water depth. Floating stability depends upon the relative position of the centres of buoyancy and gravity and upon the "water-plane'inertia". For the lighthouse to be stable in it's extended form it required a large "water-plane-inertia"; i.e. large diameter at water surface level. By pushing it together telescopically the centre of gravity was lowered, and stability could be achieved with a smaller water-plane-diameter, which in turn meant, that the structure attracted smaller wave forces and thus become better suited for the permanent design conditions. One could therefore say, that the telescopic feature was a good idea since it improved the structure for its permanent purpose; but by no means did it make it ideal. So although I am in 100% favour of "design for constructibility", I could not accept, that the design was dictated entirely by the temporary conditions and only afterwards just was checked to see, that it was adequate for the permanent - and very severe environmental conditions. So therefore, having completed the Kisk Bank Lighthouse I thought, I was going to keep a unique record, and that nobody would pursue the gravity principle into deeper water.

It did take me by surprise, when the Condeeps and the Seatanks were accepted by the oil companies; but I did though find some mitigating circumstances, when I got the opportunity to look at the problems a little closer. In very deep water it is possible to lower the centre of gravity so much without telescoping, that stability can be obtained even with a small "water-plane-inertia" and a heavy payload above water. The design is however still governed by the temporary floating conditions, and the price for stability is exceedingly high. For every tonne of payload above water it is necessary to place as much as 10 tonnes below the centre of buoyancy to keep the balance, and the base structure in consequence becomes extremely bulky. This in turn even in deep water attracts large wave forces and imposes high stresses on the seabed, and we are in the same situation as on Kish Bank, that the structure certainly is not the best answer to the permanent design conditions.

Only the fear of too short a "weather window" for the offshore construction and the prospects of enough oil to pay for any structure could make the "heavy-weights" acceptable, and probably only Norway with the ideal conditions for this construction will continue to use them.

After the initial shock from the arrival of concrete on the offshore scene the established offshore contractors have recovered, and now with much larger and more stable floating cranes to rely on the pile-supported steel structures will be pushed into still deeper water and shorter weather windows, until at some stage fixed structures are out, and compliant structures become a must. It would appear, that one oil company already seriously is planning a tension leg platform in the North Sea; but if this plan is carried out, and one jumps straight from the scaled-up Texas Towers to the TLPs leaving the concrete Dinosaurs as a dying-out species, there is most certainly some missing links in the offshore chain of evolution.

"A Missing Link?"

Foundation-wise there must be some stages in between 100 m or more long piles driven to refusal and nothing at all apart from some tiny shear keys biting into the soft surface layers, and structurally there must be some sensible solutions between the towers with their multitudes of braced tubular members and the "Dinosaurs" with their three unbraced cantilever columns. There must also be some safe and practical compromise between a lengthy offshore operation which hardly can be fitted into the "working window" and one single touch-down of a completed structure including all top-side installations.

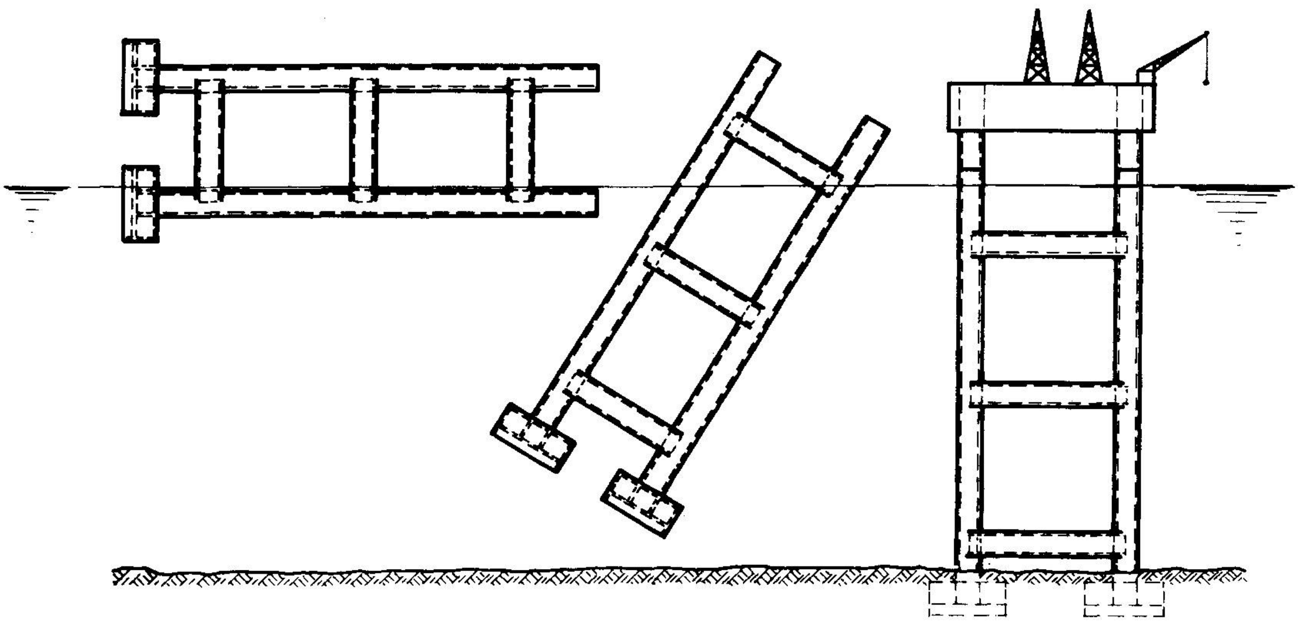
"Design for Constructibility" is a most commendable aim, but in offshore structures it has been taken to such an absurd extreme, that the design has been permitted to be dictated by the transport to the site and not by the permanent purpose of the structure. This is obviously a completely unacceptable situation - and unnecessary, if we only use the experience already gained, imaginatively.

The design and construction of the tower structure is a civil - and structural engineering task and should be completely detached from the design and construction of the top side, which is not a structural design or an offshore problem, but entirely an oil production task.

A tower structure can be designed to float horizontally in shallow water, and vertically in deep water, without getting into conflict with the requirements for the final fixed position.

From the development of jack-up structures it is obvious, that even the most extensive and heavy top-side installations can be housed in a stable and buoyant hull and in just one short spell of calm sea be landed on a previously installed tower structure and jacked to safety above the waves.

Combining the tubular design experience with the knowledge gained from the gravity structures it is obvious, that the tower design could be simplified in the extreme and be reduced to three vertical columns braced together at 2-3 levels by horizontal tubular numbers. This may not lead to absolute minimum material content, but it provides as compensation useful interior living and working space, and knowing that the structure is perfectly safe just resting on the seabed, it should not be beyond the wit of man to devise a practical foundation method. (Fig. 4)



The tower may rock about a bit in the waves; but it is possible to live and work comfortably inside the structure from the moment of touch down, and it would not appear too difficult to develope a firm foundation somewhere between huge piles or nothing, which would make sense in ordinary geotechnical terms. This may require some new and specialist equipment, but surely nothing as outrageous as hammers weighing hundreds of tons or cranes with capacities of thousands of tons.

I do hope that civil and structural engineers will take up the challenge of the sea. Sailors and oilmen are also important; but without a more forceful input of civil and structural design, some of the missing links in the offshore construction evolution will remain missing for a very long time.

I

The influence of the construction methods on the design

Influence des méthodes d'execution sur la conception

Einfluss der Ausführungsmethoden auf den Entwurf

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SUMMARY
The important nuclear power plant programme, under construction in France is performed out of execution drawings which, in theory, must not be modified. New construction methods and especially heavy prefabrication methods—which would enable to save time and money—will probably make the nuclear power plant civil engineering design progress and improve its adaptation to new methods.

RESUME
L'important programme de centrales nucléaires en cours de construction en France, s'exécute à partir de plans d'exécution qui, en principe, ne doivent pas être modifiés. De nouvelles méthodes de construction et notamment l'utilisation de méthodes de préfabrication lourde—qui peuvent apporter des économies d'argent et de délai—permettront sans doute de faire évoluer la conception du génie civil des centrales nucléaires et de l'adapter mieux à ces nouvelles méthodes.

ZUSAMMENFASSUNG
Das zur Zeit in Frankreich im Bau befindliche bedeutende Atomkraftwerkbauprogramm basiert auf Ausführungsplänen, die grundsätzlich keiner Modifizierung bedürfen. Neue Ausführungsmethoden jedoch, und speziell die Anwendung neuer Vorfabrikationsmethoden für schwere Bauteile, die Zeit und Geld sparen, werden es in Zukunft erlauben, Fortschritte im Atomkraftwerk-Ingenieurwesen zu machen.

The aim of this paper is to illustrate how prior choice of construction methods can influence and inflect design.

In France, any such possibility arises solely when the Client puts out an open, competitive call for tenders. In this case, only the bare outlines of the project are laid down by the Client and the Contractor is free - within the set limits- to design and implement the project using the means and methods specific to his company or any others which he may deem suitable.

However, the most frequent case is the call for tenders in respect of a precisely defined project.

The methods by which the contract will be carried out flow from the design without being able to influence it and, if one wishes to use new methods it is necessary to study an alternative, different project which is not always permitted.

It is from the standpoint of the latter case, the most frequent, that I will try to demonstrate that, even when the project is defined very precisely, it is, sometimes, possible to cause it to evolve in line with the construction methods and which differ from those which would normally be applied.

By way of illustration I would like to address nuclear plant civil engineering.

For a number of years now, the French nuclear programme has been one of the main activities of civil engineering in France.

The building of nuclear plants is carried out on the basis of drawings supplied by E.D.F., the French Central Electricity Board, to the Contractor.

These drawings are made by E.D.F. with the aid of outside Consulting Engineers. They remain the same for all plants of the same type, apart from the adaptation -to-site-drawings- which may vary from one site to another, depending on the varying ground conditions.

E.D.F. alone is entitled to make any changes, which usually are minor ones and are normally due to developments in the electromechanical field. The Contractor may not make any changes.

Within the framework of the CP1 programme - which comprises twenty tranches of 900 MW - E.D.F. has contracted our company to construct 4 of those tranches. 2 at Saint-Laurent-des Eaux, and 2 at Chinon. The equipment and working methods which we planned and introduced on the sites did not differ greatly from those used by other companies for the same type of plant on other sites.

Very soon we reached the conclusion that it would perhaps be advantageous to use prefabrication methods - already in use in the building trade - adapting and developing them to our needs, even although the design and implementation methods had not been worked out with prefabrication in mind.

Why use prefabrication ?

Generally speaking prefabrication is envisaged when a number of criteria exist, the two main ones being :

- Repetition : a large number of identical or similar components have to be built.
- Complexity : a number of complicated framework systems, or high-rising framework systems have to be built and which require costly shoring which one would like to simplify or even do away with altogether.

The repetition criterion is not met in the case of nuclear plant civil engineering ; on the other hand, it most certainly meets the complexity one.

What advantages may we expect ?

Prefabrication offers :

- a reduction in the total number of hours to be worked, (time gain)
- quality improvement
- safety improvement.

Time Gain :

Time savings result from the improvement in unit production which one can hope to achieve on those activities amenable to prefabrication, i.e.
. reinforcement,
. frameworking and shoring,
. concreting.

Prefabrication also enables a certain amount of work to be done off-site, or even to be sub-contracted to an outside firm thus reducing the number of workers on the site. This leads to better to relations on the labour front on the site, improved use of manpower and, at the end of the day, improved overall production.

The savings made on time may be used in two ways :

- By shortening the construction time of the plant or part of the plant. That, however, is only advantageous if the plant of the part concerned is on the critical path and if the follow-up work is not behind schedule.

- By smoothing out the site-workers curve. In the case of a nuclear plant site, the curve representing the number of workers as a function of time is a rather sharp one. It tends to flatten out over a relatively short period, of about one year (for a site with two tranches) and which corresponds to the number of workers required, once cruising speed has been attained. This number should be as low as possible. The reason for this is that it is only possible to increase this number by bringing in people from far away - which costs a lot of money - since local labour sources have been used up.

It would also mean setting up reception centres and additional housing facilities (accommodation, cantines, changing-rooms) which could not be amortized over the time during which they would be required.

Quality improvement :

Prefabrication enables a considerable amount of work to be carried out at ground level and not up in the air. It makes for easier working, less difficult supervision and planning. The work is more accurate, therefore of better quality.

Improvement in safety :

Prefabrication enables avoidance of a great deal of work at heights or in difficult access conditions, thus reducing accident risk and in particular, serious falls.

We tried to implement the fore-going ideas on the Saint-Laurent-des Eaux site, then on the Chinon site despite the constraints already referred to, namely :

- Total ban on modification to the plans,
- Equipment and working methods not adapted to prefabrication methods : handling equipment in particular ; traditional equipment for traditional type work. Mainly tower cranes which did not exceed 300 t/m. The weight of loads at the end of the jib could not exceed 7 tonnes, which is very insufficient for concrete prefabricated components.

At Saint-Laurent-des Eaux and at Chinon we therefore limited ourselves to reinforcement trussings and floor units.

Reinforcement trussings were used to produce a great number of units or frames for bedding, slabs, beams and floors of almost all of the buildings. By using weldable reinforcing rods we were able to assemble highly rigid panels or frames which did not deform on handling or when the concrete was poured. At the same time we were able to improve on the quality of trussing.

Taking advantage of the temporary presence of a high-power crane on the Chinon site, we were able to put into position at one go a trussing unit weighing more than 25 tonnes.

Pre-cast concrete slabs :

These were used for the first time when we were constructing the plant's water supply galleries. The conventional method is to pour the roofing slab in situ which entails having a very complicated shoring system bearing on the BONNA pipe. Produced in the form of a self-supporting pre-cast slab, the work involved took very little time and did not require much labour.

Using the pre-cast concrete slab system we were also able to produce a considerable number of the floors for the various buildings, in particular for the machine room and the power installations which normally require high-rising shorings which we were able to cut down on or eliminate altogether.

At the same time, we pursued our prefabrication studies with a view to breaking out of the straight-jacket-like limitations imposed on us at Saint-Laurent-des Eaux and Chinon. Our studies proved that it is possible to envisage two types of prefabrication :

1) - medium-weight prefabricated units with a maximum weight of approximately 50 tonnes : this concerns :

1.1 - prefabrication of trussings - limits being imposed not by weight but by overall dimensions of units.

1.2 - prefabrication of mainly self-supporting pre-cast slabs of large cross-section which eliminates any shoring work.

1.3 - prefabrication of beams, joists or ribs

1.4 - possibly, the prefabrication of finished floor components.

1.5 - prefabrication of non-structural slab components, the weight of which is not too great due to their small size.

1.6 - restoring of continuity between concrete components by using reinforcing rods and not costly special tie systems.

1.7 - use of non-job-specific hoisting apparatus which can be re-used on sites other than nuclear plant sites and which can therefore be more readily amortized - such as American Hoist 9310 or Manitowoc 4100.

2) - heavy-weight prefabricated units up to 250 tonnes and which comprise prefabricated components of all types, including structural slabs, but which require :

- special heavy handling equipment,
- costly special ties between prefabricated components.

The outcome of these studies and their application at Saint-Laurent-des-Eaux and Chinon, although limited, seemed to be promising enough to E.D.F.that they authorized us to run a medium-weight prefabricated unit trial (units up to 50 t) on the 2 x 1,300 MW tranches on the Belleville site, the contract for which we have just been awarded within the context of the twenty tranches programme involving a number of sites.

As was the case in the 900 MW tranches, the working drawings for the 1,300 MW are valid for all the plants of the same type. However they already take partly into account the possibilities of prefabrication techniques developed and adopted for the afore-mentioned sites.

Moreover, by anticipating long enough in advance the start-up of the work, the time necessary for re-thinking the plans which did not take prefabrication into account was able to be found without affecting the stipulated construction time.

There remains however, a certain number of limitations which restrict the full utilisation of the outcome of the experiment :

- We are still prohibited from changing the overall dimensions of the construction, hence of the framework plans. We may modify only the trussing plans.

- The cost of the trussing plans is borne by the company. For each plant, or part of a plant, it is necessary to callate the cost of re-doing the plans with the savings hoped for from prefabrication. This leads to rejecting prefabrication - which might be advantageous from a technical point of view - but which entails costly adaptation of the plans.

The experiment on the Belleville site will require adapting approximately 700 trussing drawings.

The conventional equipment using tower cranes, complemented by a Manitowoc 4100 with a Ringer 2000 t/m. Prefabrication will involve :

- virtually all of the trussing work,
- floors of the main buildings (Nuclear Ancillary Equipment, Fuel Buildings, Reactor Building),
- beams and floors (pre-cast slabs) for the machine room,
- possibly some of the non-structural slab units.

For the two tranches concerned we reckon that the total number of units which will have to be handled will be approximately 1000, i.e. a little more than one unit per day. A figure which is insufficient to permit of full utilisation of the Manitowoc. Additionally, it could also be used for lifting or positioning metal components or heavy electromechanical equipment which normally means bringing in equipment from outside as required, the transport of which, to and from the site being extremely costly.

Conclusion :

From the afore-going - based on an on-going experiment - it can be seen that even in the case of a set project it is possible to use alternative, different means and working methods which, little by little, bring about changes in design. The example of the nuclear plants has, of course, advantages and disadvantages.

Positive aspects : Series construction spread over a number of years thus offering a time advantage to allow the design to evolve in line with new working methods.

Negative aspects : Civil engineering is tied to the available electromechanical equipment and which, because of long leads involved, is ordered well in advance. In the short term, therefore, structural changes are practically impossible.
It is for this reason that prefabrication of reinforcements and the use of precast concrete slabs is now included in the various stage plans of the 1,300 MW plant as a result of the trials in Chinon and Saint-Laurent-des-Eaux. It is quite possible that the medium-weight prefabrication experiment being carried out at the Belleville site will have a certain spin-off on other sites and that despite the reluctance of E.D.F. to require of companies that they use methods involving the use of special lifting equipment. The possibilities of medium and even heavy weight prefabrication are beginning to be taken into account at design level in the studies for the next stage currently being made by E.D.F.

Of course, in order that the medium and heavy prefabrication processes be fully utilised it will be necessary to change the structure of certain constructions, in particular :

- at present, the ribbed joists and slabs used in the floors of the power houses only permit prefabrication of small dimension components. They would need to be replaced by concrete slabs thus enabling prefabrication of heavier units.

- in the machine rooms solid floors would be more easily prefabricated than the present ribbed ones.

- in the Ancillary Equipment building there is no well defined load-bearing structure as all the components are hyperstatic. Because of this, all the units are limited in size and there are not many cost-saving prefabricated components such as concrete partitions and floor slabs on straightforward bearings.

- the present design of the reactor building hardly permits any prefabrication and it is the reactor building which is, as far as the dead-line is concerned, on the critical path. It is the reactor building which determines the start-up of the tranche. It is therefore of utmost necessity to revise the design with a view to using prefabrication methods of construction in order to benefit fully from the substantial time savings which can already be made on the other buildings.

From these examples it is possible, very briefly, to define the most suitable type of structure from the point of view of prefabrication :

- A well defined load-bearing structure formed mainly by continuous shells over the whole height of the building or at least over several levels with large dimension, end-restrained slabs.

- A secondary structure made up of partions or slabs on single bearings, having no structural importance, but which can be easily prefabricated. (uncomplicated joining-up, therefore uncostly).

- New design of the reactor building with a view to using prefabrication methods.

Such changes will require time. They will mean going against the present widespread trend of seeking immediate savings, sure-fire savings, simply by cutting down on quantity and rejecting technical solutions which, even if they involves an increase in quantities, permit time to be saved and less labour to be used : savings which, in the longer term, are incomparable with the former (additional benefits, early start-up etc...).

The evolution has begun. Perhaps it is not asking too much to hope that it will continue.

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I

Structuring the commercial environment

L'aménagment de l'environnement commercial

Planung der kommerziellen Umwelt

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SUMMARY

The architectural and structural engineering design of highrise, commercial office buildings to meet the needs of the commercial environment in the world's major cities is one of the most challenging aspects of building design today. Unlike many other types of construction projects, such as highways, bridges, water-front facilities, and institutional buildings, the commercial highrise office building environment and the facilities built to meet its needs must immediately turn a profit for the owner/developer.

RESUME

Le projet architectural et structurel de bâtiments élevés répondant aux besoins commerciaux est un des aspects actuels les plus passionnants dans le projet de bâtiments, dans les villes principales du monde. A l'opposé d'autres types de construction, tels que routes, ponts, aménagements côtiers et bâtiments gouvernementaux, les bâtiments élevés à l'usage commercial, et leur environnement, doivent remplir leurs fonctions immédiatement, tout en offrant un revenu au propriétaire.

ZUSAMMENFASSUNG

Das Architektur- und Ingenieurprojekt für Bürohochhäuser zählt heute in den meisten Weltstädten als eine der herausforderndsten Aufgaben des Bauingenieurwesens. Im Gegensatz zu anderen Bauten wie Strassen, Brücken, Bauwerke am Meer und Regierungsgebaude müssen Bürohochhäuser ihren Dienst sofort erfüllen und gleichzeitig eine Rendite für den Bauherrn abwerfen.

A. INTRODUCTION

The architectural and structural engineering design of highrise, commercial office buildings to meet the needs of the commercial environment in the world's major cities is one of the most challenging aspects of building design today. Unlike many other types of construction projects, such as highways, bridges, waterfront facilities, and institutional buildings, the commercial highrise office building environment and the facilities built to meet its needs must immediately turn a profit for the owner/developer.

In this kind of economic climate, the building must be designed with the "bottom-line" as the top priority. The pressures that exist in this environment cause the designers of the building to constantly be faced with delivering the most functional, durable, aesthetically pleasing and commercially competitive structure at the lowest construction cost.

Ten years ago, when the prevailing interest rates in the United States were in the single digit range, the time required to construct a building was not nearly as important as it is today with double digit interest rates and inflation. With interim construction financing presently costing approximately 2 to 3 percentage points above the prime lending interest rate (e.g., 20% to 23% in 1980), the shortest construction time is of paramount importance to the owner so that rental income can be generated as early as possible.

All of these factors combine to make the design of the commercial environment for highrise buildings a most challenging experience.

B. BASIC STRUCTURAL SYSTEMS

Highrise commercial office buildings, from a structural point of view, consist of two major systems: The floor and the lateral load resisting system. The analysis, design and cost evaluation of the alternate framing approaches for the floor system for commercial buildings is generally an objective and well established procedure. For a structural steel building, the floor system including the columns to support the floor system usually ranges between 8 to 10 pounds per square foot (psf). For a structure with about 10 stories, the total weight of structural steel including the floor system, columns and lateral load resisting system usually amounts to about 10 to 12 psf. The distribution of steel weight and cost changes when highrise construction is contemplated. The following example shows what happens:

a. Assume that the floor system and columns, i.e. the amount of steel required to support vertical gravity dead and live loads, is 10 lbs. per sq. ft.

b. Thus, all additional steel in the building frame is related to lateral load resisting requirements. Assuming that a braced central core approach is utilized for a series of buildings having 30, 40, 50 and 60 stories, the following becomes apparent:

30 stories - gravity load system - 10 lbs. per sq. ft.
lateral load system - 15 lbs. per sq. ft.

$$\frac{\text{lateral}}{\text{gravity}} = \frac{15}{10} = 1.5$$

40 stories - gravity load system - 10 lbs. per sq. ft.
lateral load system - 20 lbs. per sq. ft.

$$\frac{\text{lateral}}{\text{gravity}} = \frac{20}{10} = 2.0$$

50 stories - gravity load system - 10 lbs. per sq. ft.
lateral load system - 25 lbs. per sq. ft.

$$\frac{\text{lateral}}{\text{gravity}} = \frac{25}{10} = 2.5$$

60 stories - gravity load system - 10 lbs. per sq. ft.
lateral load system - 30 lbs. per sq. ft.

$$\frac{\text{lateral}}{\text{gravity}} = \frac{30}{10} = 3.0$$

From the above, it is apparent that for a low-rise building, the ratio of lateral load resisting steel to gravity load resisting steel components of the building is small while for a 60 story building, the ratio can be three (3.0) or greater.

Thus, for tall buildings the key to an economic solution is the structural system selected by the designer to resist the lateral loads on the building.

The purpose of this paper is to depict the relative comparative approaches that can be utilized in the design of highrise buildings. It is important, however, to remember that the aim in highrise design is to achieve the true cost effective structure which makes a positive contribution to the total building system by minimizing the overall cost and maximizing its efficiency. The lateral load system should not only be a self-serving, low cost support system which jeopardizes the other systems of the building, such as architectural and mechanical and electrical systems.

C. STRUCTURAL AND ARCHITECTURAL FORM

With most major American cities already full of rectangular box-like and prismatic towers, the tendency on the part of today's architectural designers is to introduce variations in form and shape. Curved tops, cuts, sculptured silhouettes and other geometric expressions are utilized to capture a distinctive image for the building and to attempt to reduce the visual impact of these large monoliths very real size to the pedestrian and observer at street level. Architectural commentators are referring to this direction as the "post-modernist" school.

These various changes in geometrical form contribute an additional complexity to the selection of a structural solution to resolve the lateral load on tall towers. Generally, for buildings less than 40 stories, a central core bracing system is efficient and economical to adequately resist lateral loads. When the height of a building exceeds 40 stories, then an exterior bracing solution begins to make economic sense. Once an exterior bracing solution is arrived at as the most economical method to resist lateral loads, variations in exterior geometry of the building introduce significant impact and cost penalty on the building. The best compromise arrived at by the design team is to attain the most efficient lateral load resisting system with the most aesthetically appealing and acceptable exterior geometry.

In 1979, The American Society of Civil Engineers through the Council on Tall Buildings and Urban Habitat published Volume SB, "Structural Design of Tall Steel Buildings". This definitive volume is one of a set of five comprehensive volumes of a Monograph on the Planning and Design of Tall Buildings. This reference classified various structural systems into four types as follows:

Type I - semi-rigid and rigid connected structural frame works generally less than 30 stories.

Type II - braced core and braced core with outrigger and frame with braced core generally less than 60 stories.

Type III - framed end channel or framed middle I exterior braced systems generally less than 70 stories.

Type IV - exterior framed tube, bundled tubes and exterior diagonally braced tubes generally acceptable above 70 stories.

Figure 1 shows a diagrammatic comparison of these four structural systems. Based upon the multitude of possibilities and the subjective aspects of the selection of the lateral load resisting system, there is tremendous controversy among designers surrounding which, if any, system is best. Claims are made by many designers that one system is more advantageous than another. Should a tube or a braced frame be used? Should a steel shear wall system or a concrete shear wall system be used? Is a structural steel system more economical than reinforced concrete system? Should moment connected steel frames be used or should K-braced or knee-braced steel frames be utilized? Unfortunately, no two commercial buildings are alike, and as such, the needs and constraints of each project are never the same. It is often very misleading to utilize facts, figures and quantities from one project as a datum of comparison with another project.

Figure 2 shows the general variation in steel weight for Type I and II systems while Figure 3 shows the general variation in steel weight for Type III and IV systems. For these figures, it can be noted that the economic domain for which each type of system is economically feasible varies from 30 stories for Type I, 50 stores for Type II, 70 stories for Type III and greater than 70 stories for Type IV. It becomes obvious that as buildings grow taller, the only economical way to solve the structural problems and achieve an economical solution is to utilize a Type IV solution; an exterior braced system, or exterior tubular approach.

D. STRUCTURAL STIFFNESS AND DRIFT CRITERIA

In a design of a tall structure for lateral loads, including seismic and wind loads, the structural engineer must design the system to meet three criteria:

1. To provide a structure with adequate strength to safely resist all expected forces.

2. To provide adequate rigidity to prevent damage to non-structural elements due to building motion, thus reducing maintenance costs and improving serviceability of the structure.

3. To eliminate perception to sway (movement) or undesirable drift and unpleasant response by the occupants within the building.

There are no established drift criteria for the design of tall buildings. In general, each structural engineer designing a tall building must develop his own design criteria. The reason for this is that the wide variation in geometrical configurations and the mixture of materials, structural systems and exterior cladding systems can either significantly increase or decrease the amount of drift. Generally, drift is limited to approximately .002 H where H is the building height. Another way to look at drift limitation is to relate it to a relative drift or movement between adjacent floors. This is called interstory drift.

The interstory drift becomes quite important relative to the interaction and impact of structural movement or drift of the overall building on the exterior enclosure system. With the modern trend toward the use of lighter, high performance glass and metal exterior walls, the drift of buildings tends to be much more dependent upon structural stiffness of the major wind resisting system, the structure. Very little resistance results from the exterior wall. As a result, the calculations for drift of buildings become quite important and the actual calculated amounts are very close to reality. Since the normal floor to floor height in highrise office buildings is generally 12'-0" to 12'-6" high, the interstory drift at .002 times the story height becomes approximately 3/8". Exterior wall systems must be capable of absorbing this movement without damage. If the drift of the building were to be significantly higher than .002 H (where H is the story height) which has actually happened in several major highrise buildings in the United States, severe curtain wall deformation and failure occurs.

It is difficult to set a definitive criteria for drift limitation in a highrise building. More important, the drift control and behavior must be related to the natural period of vibration and dynamic motion of the building.

E. EXAMPLES

In order to attempt to show how the above factors have impacted on the design of several highrise buildings in the United States, two projects are described in this section. The Continental Center, a 42-story building located in New York City, contains approximately 1.1 million square feet, and a 54-story United States Steel Realty Corporation, Dravo Building, in Pittsburgh contains 1.7 million square feet. Each of these buildings utilizes a different lateral load resistance system.

1. The Continental Center, New York, N.Y.

The Continental Center is a 1.1 million square foot commercial office building located at Maiden Lane in the business district of lower Manhattan. The owner of the project is a joint venture of Rockefeller Center Development Corporation and the Continental Corporation, a large American insurance company.

The architect for the project is Swanke, Hayden, Connell & Partners and the structural engineer is Thornton-Tomasetti, P.C., the Office Of Lev Zetlin Associates, Inc. The general contractor is Tishman Construction Corporation.

The project is located adjacent to the South Street Seaport and is sited diagonally on the parcel of land in order to allow vistas on the Seaport and the surrounding waterfront. Because of space dedicated to the public at the lower levels and the desire to architecturally express the large atria on three sides, the elevation from the ground to the first occupied office floor is approximately 100 feet. As a result, the unsupported length of structural columns around the exterior of the building is quite long. With long unsupported lengths of exterior columns at the base of building, the utilization of exterior moment connected frames or the use of an exterior tube type structure is not economically or technically desirable. Since the building is only 42 stories high, a braced steel core utilizing knee braces in one direction and K-braces in the other direction is utilized. This corresponds to a Type II building. The weight of structural steel per square foot for this project is 23 lbs. per sq. ft. This compares favorably with the chart shown in Figure 2. Alternate schemes were studied using moment connected frames, exterior tube framing and central reinforced concrete shear wall system but all proved to have no technical or cost advantage.

The typical floor plan of the building is shown in Figure 4 for both architectural and structural systems. The exterior architectural elevation of the building is depicted in Figure 5. The wind loadings for the structural system of this building are shown in Figure 6 along with the curtain wall wind pressures and their correspondence to New York City Wind Pressure Code specified values. A wind tunnel test was undertaken for the building which showed that in all cases the pressures for structural design would be less than the code specified values. This was not true for the pressure distributions developed from the wind tunnel for the exterior cladding or curtain wall design. In this case, the wind pressures were significantly in excess of what would be expected by code. Figure 7 shows a diagrammatic depiction of the wind bracing system utilized in the building. The structural analysis of the structure for the lateral loads on the building showed that the deflection or drift at the top of the building would be 14". This corresponds to a drift ratio of about .0021H. Figure 8 shows a deflected shape diagram, generated by computer, of the structure's behavior under wind loading. By utilizing computer graphics, the actual deformed shape of the structure was studied and where deviations from expected behavior were observed, member sizes were changed and the analysis redone so that an acceptable design could be finally achieved.

2. <u>The United States Steel Realty, Dravo Building, Pittsburgh, Pa.</u>

The structural system of the 1,700,000 sq. ft. 54-story Dravo Tower utilizes an exterior framed steel tube with a unique exposed steel stressed skin as both a structural bracing system and the facade. This approach eliminates the need for a separate non-structural exterior wall, reduces the steel in the primary lateral load resisting frame, and maximizes interior space by reducing the size of the central core structure. This stressed skin tube structural system both reduces cost while increasing the ratio of net to gross floor area, which is extremely important to the real estate developer.

The architect for the project is Welton Becket Associates of New York and the structural engineer is Lev Zetlin Associates, Inc. of New York. The owner is United States Steel Realty Corp., the major tenant is the Dravo Corporation and the general contractor is Turner Construction Corporation of Pittsburgh.

The exterior framed steel tube with columns spaced 10 feet on center, together with moment connected spandrel beams, provides all necessary lateral resistance to maintain stresses within code allowables. However, designing a highrise building with members sized to resist lateral loads based only on allowable stresses results, predictably, in a building with an unacceptable amount of sway - in this case, a sway equal to building height divided by 300 or .0033H where the height of the building is 729 feet.

The building is shown in exterior elevation in Figure 9. The typical floor plan is shown both architecturally and structurally in Figure 10.

In a conventional framed steel tube structure, steel is added to the spandrels and columns, and often to the core, to provide the additional lateral stiffness required to develop an acceptable sway in the order of the height divided by 500 or .002H. This lateral stiffness is accomplished in the Dravo Tower with the stressed skin steel facade which interacts with the primary tube structure to minimize sway.

The 5/16" steel skin, with precut openings for windows, is applied in three-story high units. Its dual function of acting as facade and a structural element is the key to the cost savings achieved with this design. Figure 11 shows the approach to the exterior facade structure.

By establishing the stressed skin exterior tube as the structural component that will resist all lateral loads, the core is designed to transmit only gravity loads. This significantly reduces the size of the core walls, adds approximately 18" of additional rentable floor space around the core's perimeter on all floors, and results in further cost effectiveness of the design. Figure 12 shows the deformed shape of a typical exterior panel as drawn by a computer aided device.

Since the degree of a building's sway is not related to structural integrity, there are no code requirements dictating its amount. With the Dravo Tower's core resisting some of the gravity loads and the primary tube structure resisting the rest of the gravity loads and all lateral loads within code allowables, the building is structurally sound without the stressed skin. The steel skin adds the significant additional structural stiffness necessary to eliminate an unacceptable amount of sway.

Therefore, by maintaining this distinction of function between the steel skin and the rest of the structural framing in both the design and analysis, the steel skin can be exposed, without fire protection, thus adding to still further cost savings.

This approach requires an in-depth analysis. The entire structure must first be analyzed as a tube system without the skin to be sure that all stress levels are within code allowables. The tube system generated by computer aided device is shown in Figure 13. The tube structure must be analyzed again with the addition of the skin as a shear membrane to be sure that an acceptable level of sway is obtained. Also, stresses must be re-analyzed to assure that in all areas of the structure they are indeed further reduced by the application of the skin and there are no local areas where increases have been induced.

In addition, the skin must be designed for a combination of in-plane shear forces, wind forces perpendicular to the skin, some compressive forces transferred from the columns due to elastic shortening, residual stresses due to the window openings, thermal forces, as well as overall and local shear buckling.

This complex analysis is required only for design; fabrication and erection, however, is executed in a conventional manner.

Between the core and the exterior tube, 47 ft. clear spans are achieved with 24 in. wide flange beams which are penetrated for mechanical distribution systems to minimize floor to floor heights. The floor typically consists of 2 in. composite electrified deck spanning 10 feet between the beams with 2½ in. stone concrete fill. The total weight of primary structural steel is limited to only 23 psf. The added stiffness due to the exterior skin is achieved within a cost of exterior skin less than that expected for a conventional curtain wall.

The 17-story low-rise bustle appended to the tower is not tall enough to act as a tube. However, it utilizes a similar structural system to provide lateral resistance, supplemented by some internal bracing that is required due to its unsymmetrical relationship to the tower. The structural system of the bustle is "tuned" to behave in a manner compatible with the movements of the tower. This permits the elimination of expensive expansion joints between the bustle and the tower.

The net result of this integrated and efficient architectural and structural design is to provide cost effective column free interior space with a high ratio of net to gross floor area. The heart of the cost effectiveness lies in the dual function of the stressed skin exterior wall which provides a facade as well as the stiffness required to minimize sway.

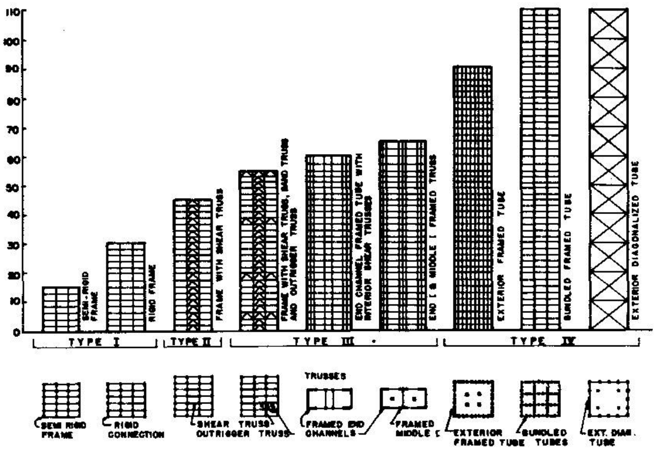

FIGURE 1 - VARIOUS STRUCTURAL SYSTEMS

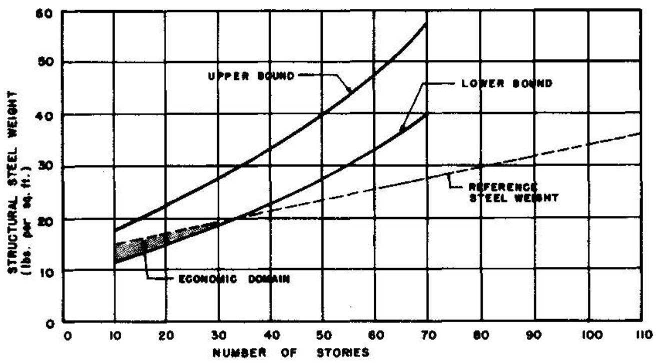

FIGURE 2A - TYPE I

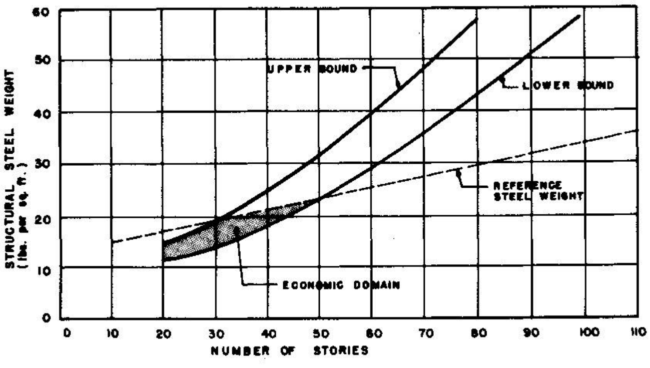

FIGURE 2B - TYPE II

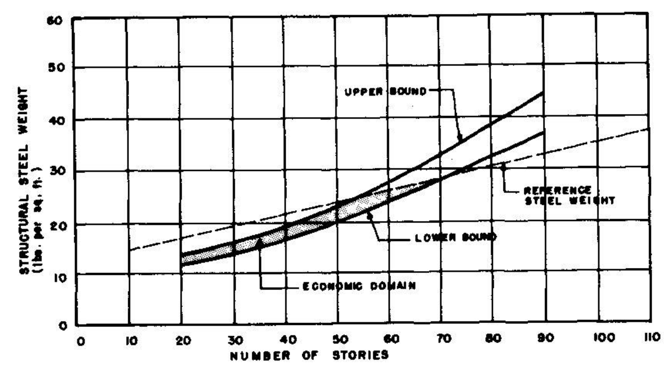

FIGURE 3A - TYPE III

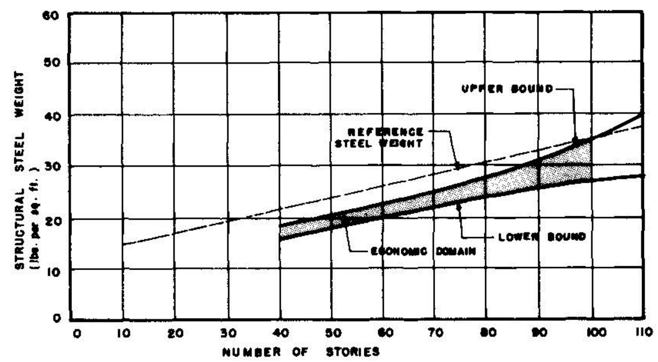

FIGURE 3B - TYPE IV

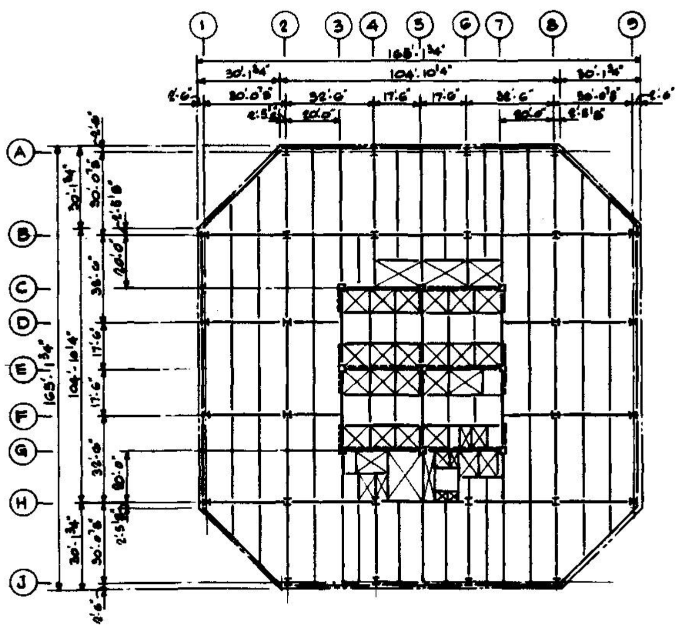

FIGURE 4 - CONTINENTAL CENTER TYPICAL FLOOR

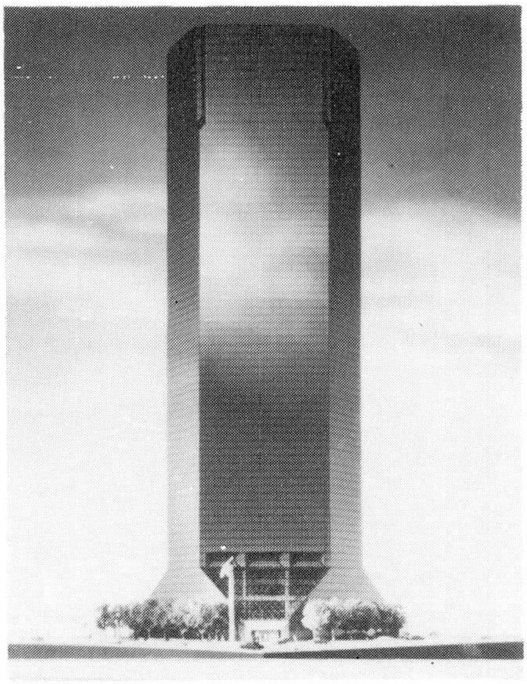

FIGURE 5 - CONTINENTAL CENTER MODEL

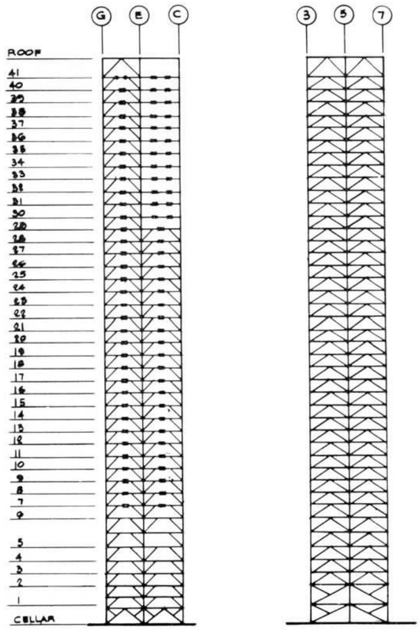

FIGURE 7 - WIND BRACING
CONTINENTAL CENTER

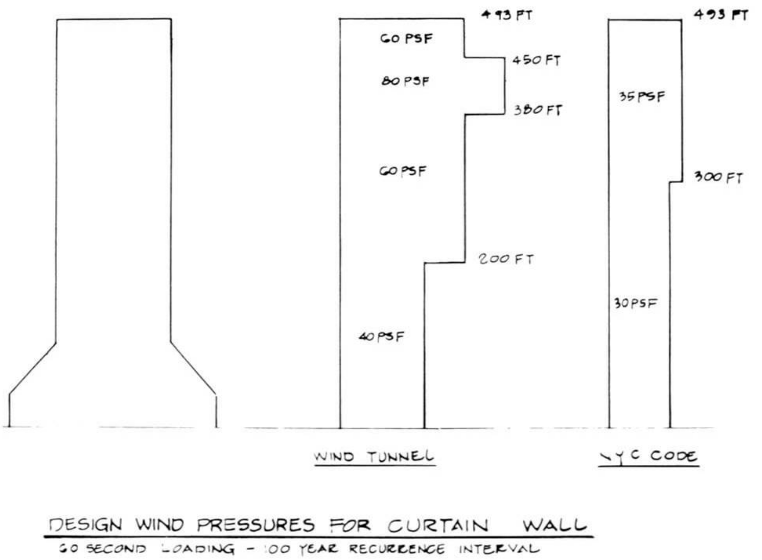

FIGURE 6 - WIND PRESSURES - CONTINENTAL CENTER

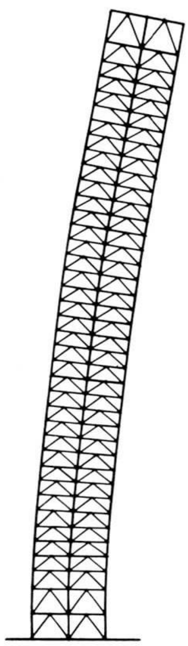

FIGURE 8 - DEFLECTED SHAPE - CONTINENTAL CENTER

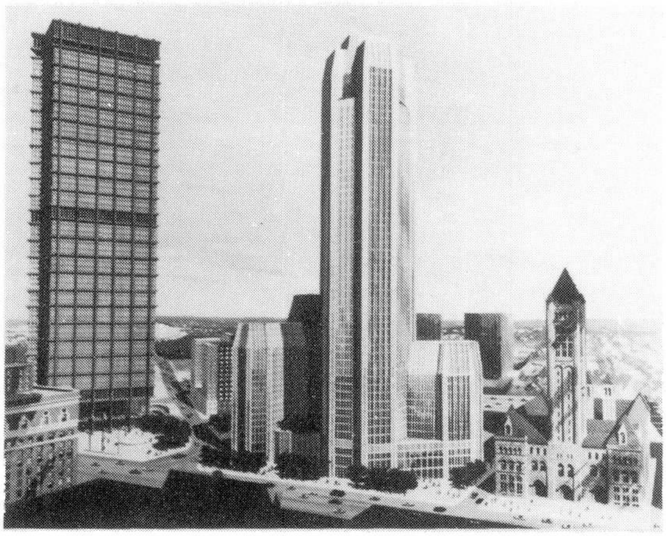

FIGURE 9 - DRAVO BUILDING

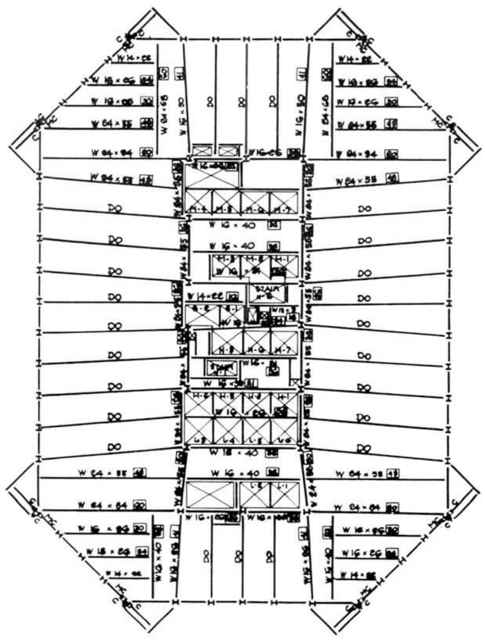

FIGURE 10 - DRAVO BUILDING TYPICAL FLOOR

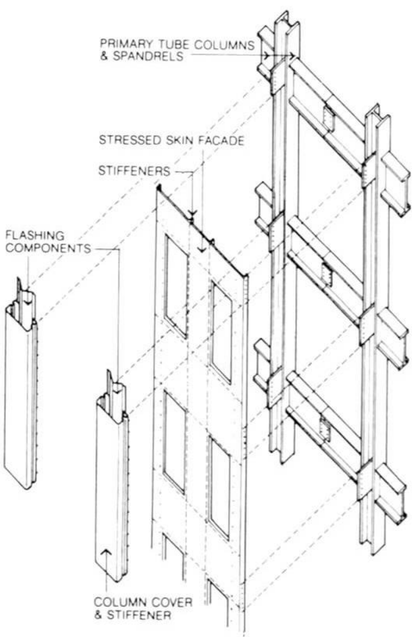

FIGURE 11 - EXTERIOR WALL SYSTEM

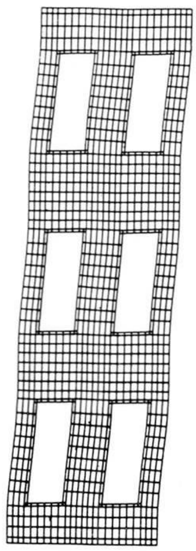

FIGURE 12 - DEFORMED SHAPE OF EXTERIOR WALL

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