IV

Constructions spéciales
(acier, béton, mixtes; études comparatives)

Spezielle Bauwerke
(Stahl, Beton, Verbund; vergleichende Studien)

Special Structures
(Steel, Concrete, Composite; comparative Studies)

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IVa

Offshore Structures

Constructions en mer

Bauwerke im Meer

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INTRODUCTION

With an ever-expanding world population the demands for space, food and energy are increasing annually. This development has caused world-wide concern since the world's land area only covers about three tenths of the earth. Therefore, with the on-shore resources gradually being depleted, growing attention has been focused on the part of the world covered by the oceans. Although studies have been carried out regarding the use of the sea for habitation, so far the main efforts have been directed to exploring the mineral resources beneath the sea. In man's quest to explore these riches, the Continental shelf - the portion of the sea floor less than 200 m below sea level - has become the initial proving ground of the structural engineer, thereby moving gradually from shallow to deeper waters. However, the development of these regions has been rather slow because of the high cost of exploration and production, and the complexity of the associated engineering problems. By early 1974, of the 27,876,000 square kilometers of ocean with depths of 30 m or less, about 60% had shown sedimentary basins potentially holding oil and gas deposits. Of this portion only 25% had been leased for exploration and actually only 15% of that has been explored.

During the sixties the offshore industry has been rapidly expanding to meet world energy demands. In 1960 there were only three of four countries and about five companies with offshore petroleum interests. Fourteen years later several hundred companies are exploring the Continental shelves of 80 countries and already 30 nations are producing, or about to produce, subsea oil and gas. Over 10,000 wells have been drilled offshore and oil and gas are being piped from as far as 400 kilometers offshore and in water depths of up to 150 m. Figure 1 shows the potential offshore oil areas around the world and the locations with current production. With the 1973 increases in the posted price of oil, the economic feasibility of offshore oil and gas exploration and production has improved drastically, unfortunately thereby contributing to an almost rampant world-wide inflation. As a result the expected production rates for the next decade will increase exponentially, as shown in Table I.

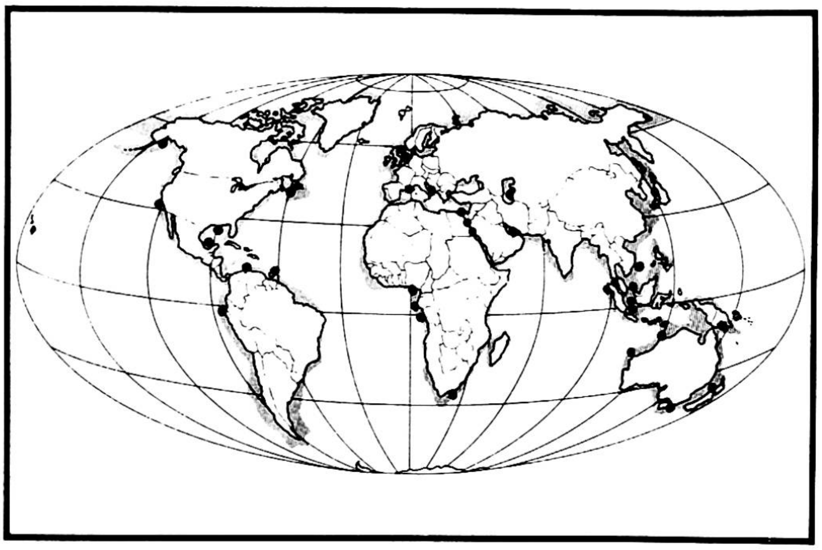

Fig. 1. Regions with offshore oil and gas

In the search for hydrocarbons the objectives of oil companies are the location of promising geological structures, the testing of these structures and the development of successful finds of oil and natural gas. It is realized that deepwater terminals and other near-shore structures may qualify as offshore structures. However, this paper will deal specifically with a discussion of structures which are used in the search and development, or in the exploration and production of hydrocarbons. Associated structures, like offshore storage facilities and underwater pipelines will also be reviewed briefly. Because of the nature of the environment the role of the naval architect in the conceptual design of certain structures is significant. However, because of his particular training, the final formulation of the design configuration is almost exclusively the task of the structural engineer.

Since this paper is a review paper the author decided first to acquaint the reader who is unfamiliar with this field of engineering with the basic types of structures operating in the offshore environment. Subsequently, a discussion of the problems associated with the analysis and design of these structures will be presented.

TABLE I - WORLD OFFSHORE PRODUCTION

YEAR	OIL million bbl/day	GAS billion cu ft/day
1974	8	28
1979	15	59
1984	30	114

OFFSHORE STRUCTURES

Based on the sequence of offshore developments, offshore structures can be grouped in two major categories, namely mobile drilling rigs and production platforms. A third group involving structural engineering could be identified as oil transportation equipment like offshore oil storage facilities, pipe-laying barges and pipelines.

MOBILE DRILLING RIGS

The offshore industry recognizes three types of mobile rigs, namely drillships, jack-ups and semi-submersibles. These units are normally used in the exploration of specific regions of the continental shelf. The jack-up platforms have an operational depth limit of maximum 90 m. For exploration at greater depths the industry uses drillships and semi-submersible drilling rigs. While the advanced units of these latter types have been designed to operate in water depths of 300 m, some of the latest designs have even specified operating depths as large as 600 m. These units are equally suited for operation in calm seas. However, in heavy seas the semi-submersible rigs, because of the partial submersion, provide a more stable drilling platform than the drillships. The roll and pitch of the drillship in heavy weather may force halting the drilling operation.

The cost of these mobile drilling units has lately increased considerably, due to both inflation and more stringent design criteria. Jack-up rigs are now costing between $14 and $18 million and drillships may vary in cost between $20 and $30 million. Finally, the advanced semi-submersibles may run as high as $35 to $40 million. The world-wide inflationary trend has also affected the day rates for these units. A couple of years ago a mobile rig could be contracted at a daily rate of about one thousandth of the initial cost of the equipment. Recently this rate has increased to $1,500 per one million dollar initial cost.

The design of the drillship is of course entirely the domain of the naval architect. However, the design of the jack-ups and semi-submersibles is a joint effort. While the naval architect is responsible for developing the seagoing characteristics of these units, the structural engineer is normally called upon to carry out the structural design of these rigs. Therefore, in the following sections only the jack-up and semi-submersible drilling units will be discussed.

Jack-up Drilling Platforms

The jack-up, or self-elevating platform, typically consists of a floating cellular hull with retractile legs. While enroute the legs are raised as high as safely possible to limit the drag. On site, the legs are lowered onto the seabed and allowed to penetrate while the hull is being raised out of the water. The major structural feature of these rigs are the legs. These units are mostly three or four legged and are presently designed to operate in water depths of up to about 110 m. Conceptual designs have been developed whereby a two-level jack-up and hull system would allow operation in water depths of about 175 m.

One of the larger typical jack-up rigs is shown in Fig. 2. This unit with a hull measuring approximately 70 m x 60 m x 8 m has three legs with a maximum length of 135 m. The maximum operating depth is 90 m with a 9 m leg penetration and an air gap between the water and the bottom of the hull or platform of 15 m. The legs of this rig are oriented vertically and are constructed as welded steel tubular trusses with a square cross-section. While most platforms have vertical legs, some rigs have slanted legs. Others again have tubular trussed legs triangular in cross-section. In some instances the legs are single thick-walled steel columns, either square or circular in shape.

While most of the jack-ups have to be towed, Fig. 3 shows a self-propelled jack-up, the first of its kind. The vessel-shaped hull measures 85 m in length and 40 m in beam. The four tubular trussed legs of triangular cross-section have a length of 108 m and allow operation in water depth of up to 76 m.

Fig. 2. Jackup Drilling Platform

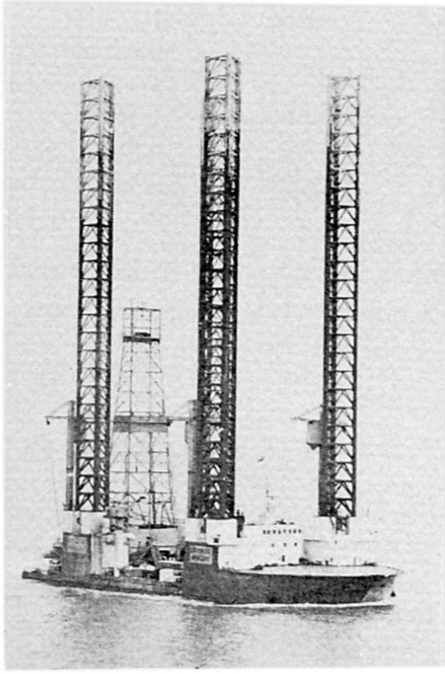

Fig. 3. Self-propelled Jackup

In general, the legs of these jack-ups reach their full length by adding on leg extension sections. These truss extensions are not used when the rig operates in shallower waters. The operating depth for these units, as listed by the owners, entirely depends on the prevailing sea conditions used in the design (maximum design wave, wave spectrum and current). Operating these rigs in a more hostile sea environment requires a reduction of the effective operating depth, not only to achieve an adequate air space but also to maintain the original life-expectancy of the unit. For instance, the deepest rated jack-up rig built to date was specifically designed to meet the stringent requirements of the Norwegian North Sea and can drill in 91.5 m of water during the summer and 84 m during the winter. Operating in other areas, with less severe environmental conditions, additional leg sections can be added to increase the operational water depth to 108 m. The three almost 135 m long slanted legs supporting this unit are square in overall cross-section and have pointed spud cans designed to obtain sufficient penetration and reduce scouring effects on the North Sea floor.

Semi-Submersible Drilling Rigs

The development of semi-submersibles during the last 10 years has shown an evolution from the pontoon-supported, multiple-column stabilizing units to the present-day twin hull rigs. The semi-submersible platform gains its main source of buoyancy from the pontoons or hulls which are submersed below the surface where wave action is less severe. Stability is provided by the vertical columns which pierce the water plane.

Some of the earlier units (1966) were pontoon supported and had three stabilizing columns as shown in Fig. 4. These units were designed to operate in water depths of up to 180 m. More advanced units having the same basic geometry have been designed to operate in depths of 245 m. These rigs have hull dimensions of about 100 m x 100 m. In addition to these units with triangular

Fig. 4. Pontoon-supported Semi-submersible

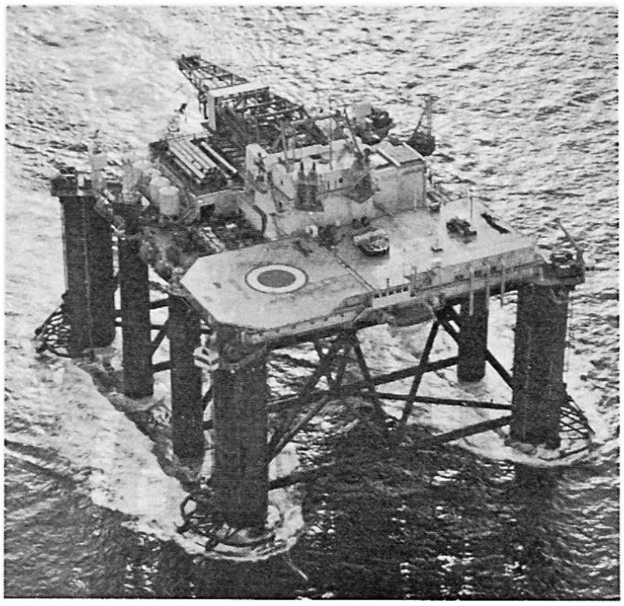

Fig. 5. Twin-hull supported Semi-submersible

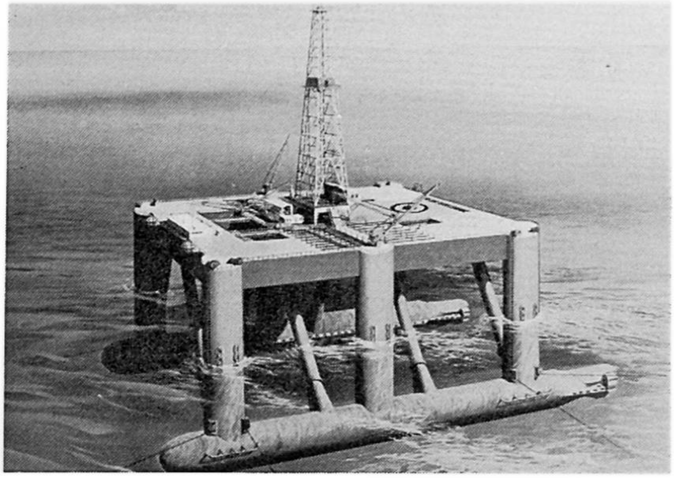

Fig. 6. Twin-hull Self-propelled Semi-submersible

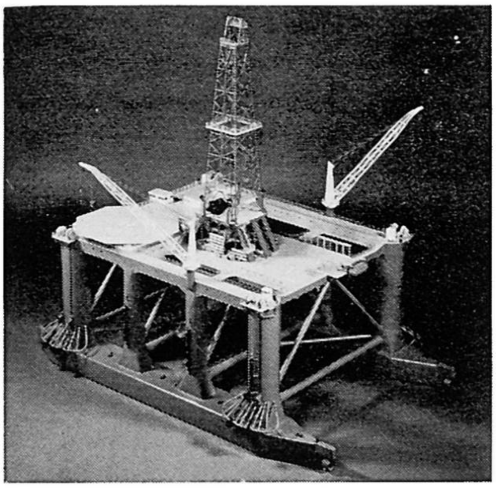

Fig. 7. Twin-hull Self-propelled Semi-submersible

column layouts, other recently built platforms have pentagonal pontoon-supported column arrangements, capable of operating in water depths of 200 m.

Most of the semi-submersibles recently delivered or presently under construction fashion a twin-hull, column stabilized platform as illustrated in Figures 5 through 8. The deck areas are virtually square, with the overall width of the twin hulls about double the height of the structure. The hull length may very depending on the self-propulsion system. These propulsion assisted units allow a reduction of the towing time and thus become more effective. Figure 9 shows one of these units in operation in the North Sea.

The operating depth of these newer twin-hulled rigs is almost invariably 300 m, thus allowing exploration of the Continental slopes. Actually, one of the most advanced twin-hull units to be placed in service in 1975 is designed to drill in a maximum water depth of about 600 m. Like all other semi-submersible

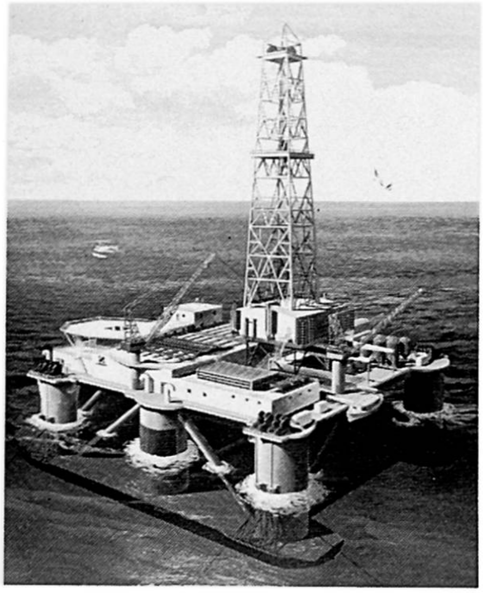

Fig. 8. Twin-hull supported Semi-submersible

Fig. 9. Semi-submersible in operation

Fig. 10. Self-propelled multiple-hull Semi-submersible

rigs this unit also uses a standard anchoring system to maintain location. However, because of the tremendous anchor forces of these deepwater units, future designs will probably use the principle of dynamic positioning.

A basically different rig design with an octagonal column layout develops its buoyancy through an orthogonal arrangement of multiple hulls as shown in Figure 10. This unit is self-propelled and can drill in water depths of 200 m. While en route only the longitudinal hulls are submerged, thus limiting the drag forces.

The steel framed semi-submersible units require that special attention be paid to the design of the truss system and the welded tubular connections. Hence rigorous analyses and model tests to determine the sea-going characteristics of these units and the associated member forces are of utmost importance. Great care should be exercised in developing appropriate joint design details, thus limiting stress concentrations wherever possible.

While semi-submersible drilling rigs have been built so far exclusively in steel, recently a design for a concrete drilling rig has been developed. This Condrill platform - as shown in Figures 11 and 12 - can be used for both exploratory drilling and as floating storage and production platform. The caisson type of structure consists of fourteen vertical cylindrical shells with external diameters of 8.25 and 15 m. These cells are poured in a single operation to form a monolithic unit. Six of the cells are capped, while the remaining eight extend above the waterline to support the double-level deck. The total concrete

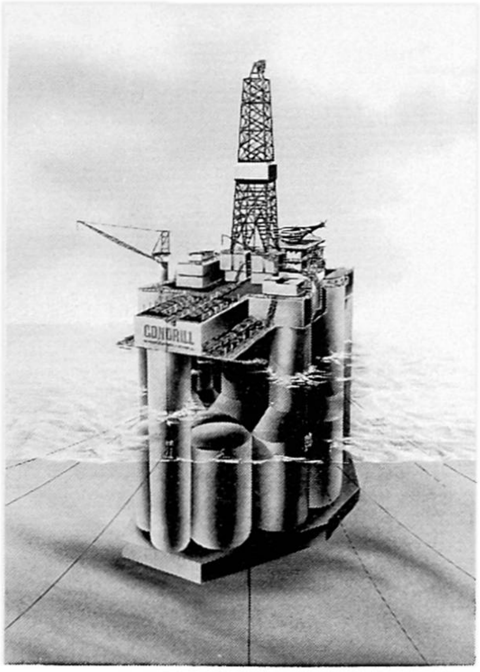

Fig. 11. Concrete Semi-submersible

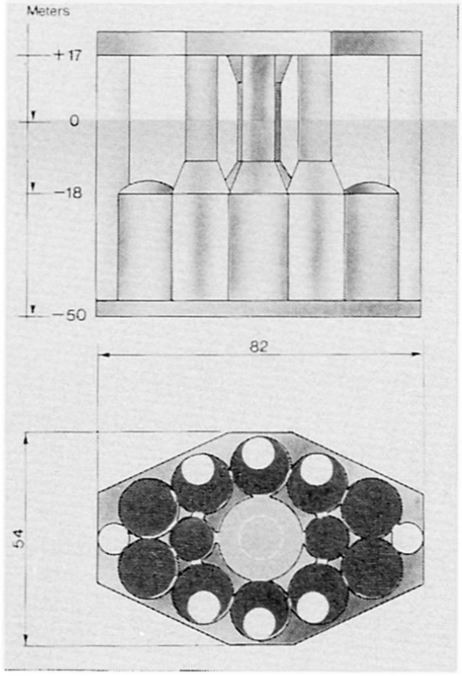

Fig. 12. Dimensions unit shown Fig.11

weight of the structure is about 50,000 ton, sixty percent of which is concentrated in the bottom slab. The storage capacity of this unit is listed as 2,000,000 barrels of oil. Drilling will take place through a 21 m diameter cell extending through the center of the platform. At the deck level this cell reduces to about 10 m in diameter. The unit is designed to operate in a water depth of 300 m and has a drilling draft of 50 m. Under tow this draft is reduced to 30 m.

DRILLING AND PRODUCTION PLATFORMS

Following the successful completion of the exploratory drilling phase, it is necessary to install a platform for the drilling of the production wells and subsequent production. These platforms have typically been designed as steel welded tubular space frames, called jackets. The vertical jacket legs, or columns, support the deck sections, while the diagonal or K-braces together with the horizontal web members provide the primary resistance against the lateral loads due to waves, currents, ice flow, wind and possibly earthquakes.

The smaller jackets - for water depths of up to 100 m - are invariably brought to the site on a barge and either lifted in position or launched off a barge. These platforms are subsequently anchored to the sea floor by driving steel piles through the inside of the jacket legs. The space between the pile and the inside of the leg is subsequently cement grouted to create an integral, well anchored truss structure. Next the deck units and operating equipment will be installed. The jackets are typically fabricated while in a horizontal position as illustrated by the North Sea Ekofisk jacket designed for a water depth of 75 m, (Fig. 13). A completed multiple platform unit located at the Leman Bank field (North Sea) is shown in Fig. 14. For most jackets the wells are normally placed outside the column legs, thus requiring conductor guide frames as shown in Fig. 13.

In case a platform is to be installed in waters with ice field movements, it is necessary to protect the well pipes by placing them inside the column legs. The absence of outside conductor pipes is illustrated by the three-legged platform located in Cook Inlet, Alaska (see Fig. 15). Furthermore, since it is necessary under those conditions that the web members do not pierce the waterline they should be restricted to the under-water portion of the tower. Because of

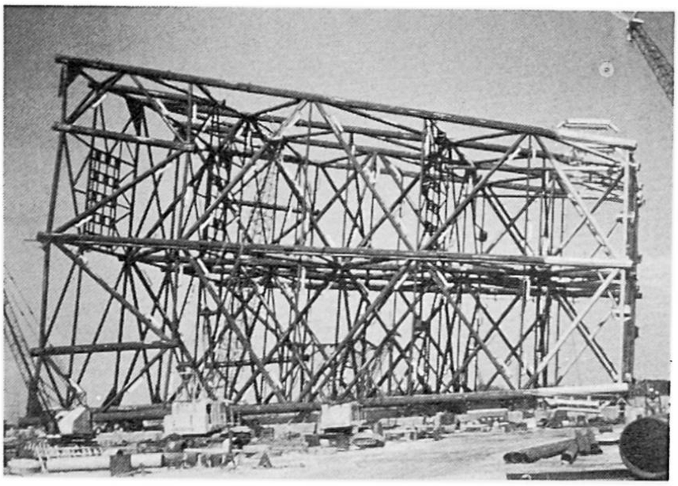

Fig. 13. Jacket at fabricating yard

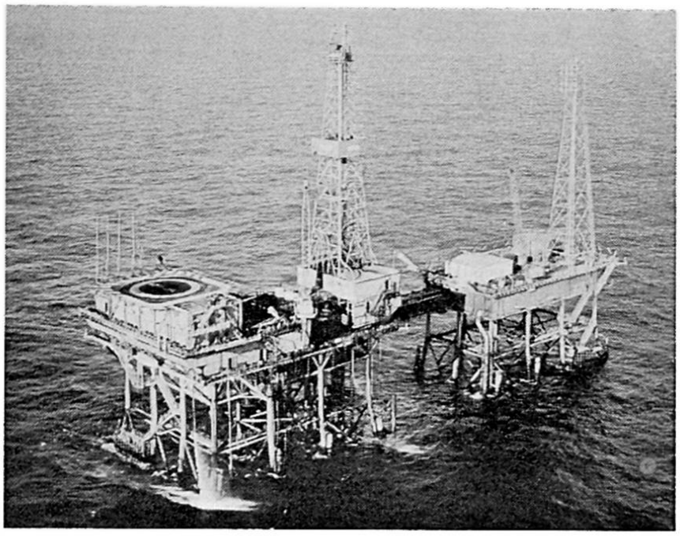

Fig. 14. Offshore Tower

the larger space requirements to locate piles and well pipes inside the jacket legs these sections are substantially larger in diameter as compared to the more standard units (4 to 5 m versus 1.00 to 2.00 m). The increased column sizes and large column surface loads require a substantial internal stiffening by either radial and longitudinal stiffners or by cement grouting the void spaces after installation, or both. These larger column dimensions provide sufficient buoyancy to float the jacket structure on its side. At location the jacket is upended by flooding the column legs as shown sequentially in Fig. 16 for a 32 conductor, four-legged Cook Inlet platform. Piles driven inside the legs secured the structure to the sea floor.

The largest steel jacket installed to date is the Highland One, a 145 m high structure standing in 127 m of water. This structure, which is one of the four jacket-type towers to be installed for production of the North Sea Forties field, was fabricated in

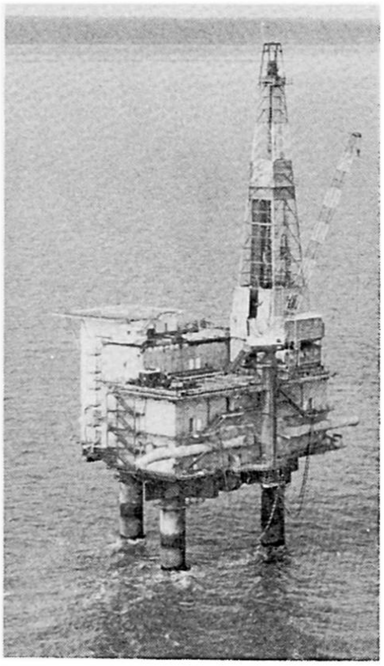

Fig. 15. Offshore Tower

a dry dock and floated out on its side while supported by a specially designed re-usable steel flotation unit (see Fig. 17). The jacket structure has a weight of 21,000 metric tons, while the flotation unit weighed 9700 tons. After the structure was tipped and sunk into place, the flotation structure was retrieved. Forty four 137-cm diameter steel piles, 73 m long and placed on the outside of the corner legs - note pile guides in Fig. 17 - secure the jacket to the sea floor. After placing the two-level deck modules together with the drill towers on top of the jacket, the total height of the structure will be about 220 m. The deck sections and auxiliary equipment, including piles and well conductors will weigh about 13700 tons. Hence, the total weight of the structure will be about

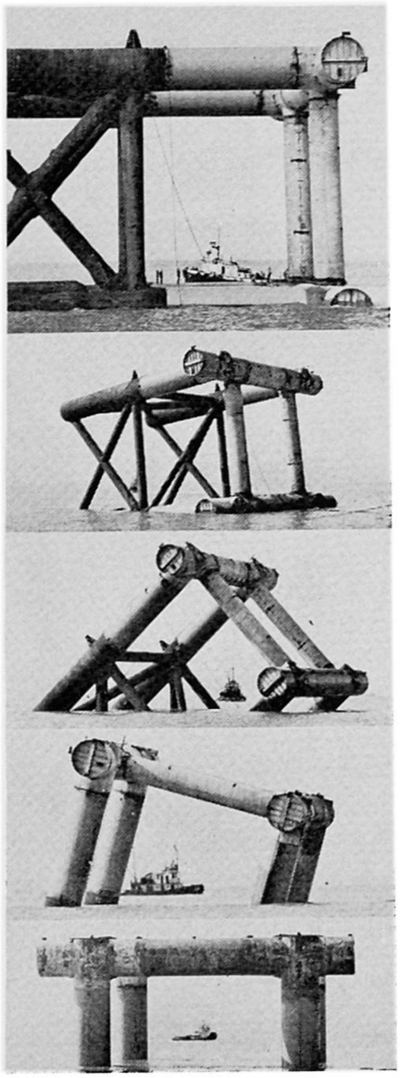

Fig. 16 Launching sequence

35,000 tons. The cost of this unit when completed is expected to be about $165 million.

The largest steel jacket platform presently under design will stand in the 162 m deep water of the North Sea Thistle field. The total height of this unit to the top of the flare stack is to be 280 m and the weight about 29,000 tons. This jacket will derive its flotation capability from two 9 m diameter legs and two additional cylindrical tanks, 82 m long and about 9 m in diameter, attached permanently to the two flotation legs. These supplemental flotation units provide a 70,000-barrel oil storage capacity when in operation. The daily production from the 60 wells to be drilled from this platform is estimated at 200,000 bbl.

The basic concept of oversized column legs on one side, in order to float the jacket out on its own buoyancy, is not new and has been used successfully before in platform designs offshore California. One of the critical aspects of the steel jacket-type platforms in a North Sea environment is the risk of upending the structure and the time and costs involved to drive the piles in order to tie the platform down to the sea bed. The latter time element reflects the risk that the jacket might be subjected to heavy weather before being properly anchored down. To reduce this risk, concrete gravity structures, serving as drilling, production and storage facilities, have been introduced in the offshore industry for the first time last year.

The concrete gravity units have the advantage that they do not need to be anchored to the sea floor because of their enormous dead weight - about fifteen times the weight of a comparable steel platform. Hence, the installation costs of concrete gravity structures - about 10% of the total cost - is considerably smaller than for the very large steel jacket platforms. The overturning movement under the most extreme sea conditions is completely counteracted by the structure's gravity. Under those circumstances it is imperative that the surface and near-surface soil conditions of the sea floor should assure the stability of the structure and soil. Hence, the soil layers should be horizontal in order to prevent sliding and to assure uniform consolidation.

Fig. 17 Jacket on flotation unit

Since weaker layers may underlie stronger but shallower surface layers, deep skirts which are to penetrate the stronger upper layers and to develop the strength of the lower soils are commonly proposed.

The first concrete gravity structure is the 1,000,000 bbl storage tank which was installed at the North Sea Ekofisk field in 1973. Fig. 18 shows the structure while under construction and Fig. 19 gives a view of the tank as installed at the 75 m deep Ekofisk site. The tank is used for both production and storage. The inner tank complex, as illustrated in Fig. 18, is protected from the direct wave impact by an almost circular perforated breakwater wall.

The concrete gravity structures under construction at this time combine storage facilities, housed in a multi cellular system at the base of the structure, with the typical drilling and production facilities. The 1,000,000 bbl storage capacity of the Condeep design is provided by the nineteen vertical cylindrical tanks each with a 20 m outer diameter and a wall thickness of about 75 cm. The tanks are arranged in a pentagonal array as shown in Fig. 20. Sixteen of the tanks are capped at a height of 50 m, while the remaining cylinders form the base of the three post-tensioned concrete columns which will rise to about 20 m above still water and will carry the steel deck structure. (See Fig. 21). Of the five Condeep platforms presently under construction around the North Sea the first one ordered will be installed in 1975 in the Beryl field at a water depth of 110 m. Two other units are destined for the Brent field, where they

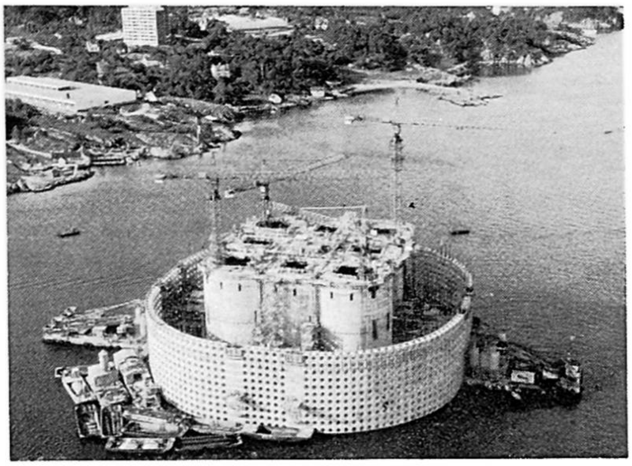

Fig. 18 Ekofisk tank under construction

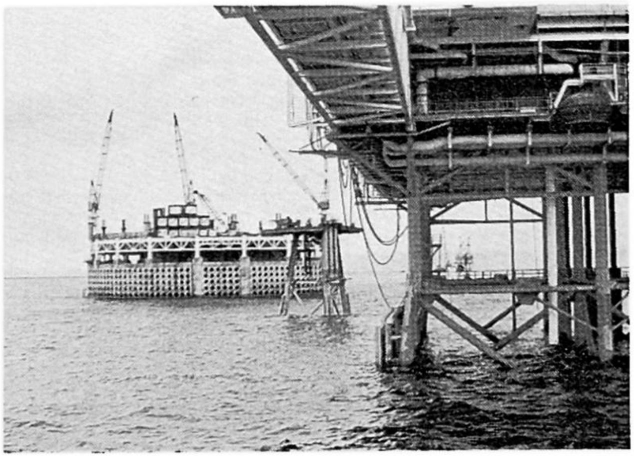

Fig. 19 Ekofisk tank installed

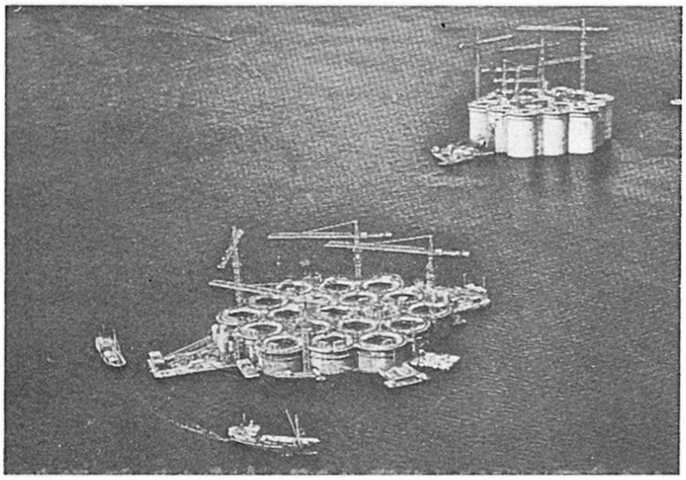

Fig. 20 Condeep Platform under construction

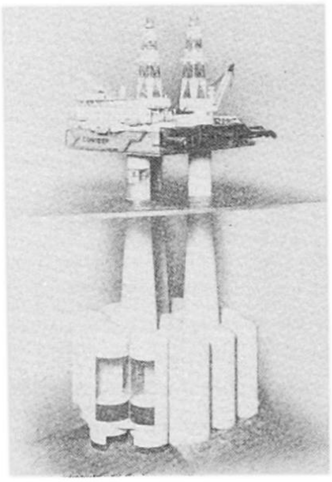

Fig. 21 Condeep Platform

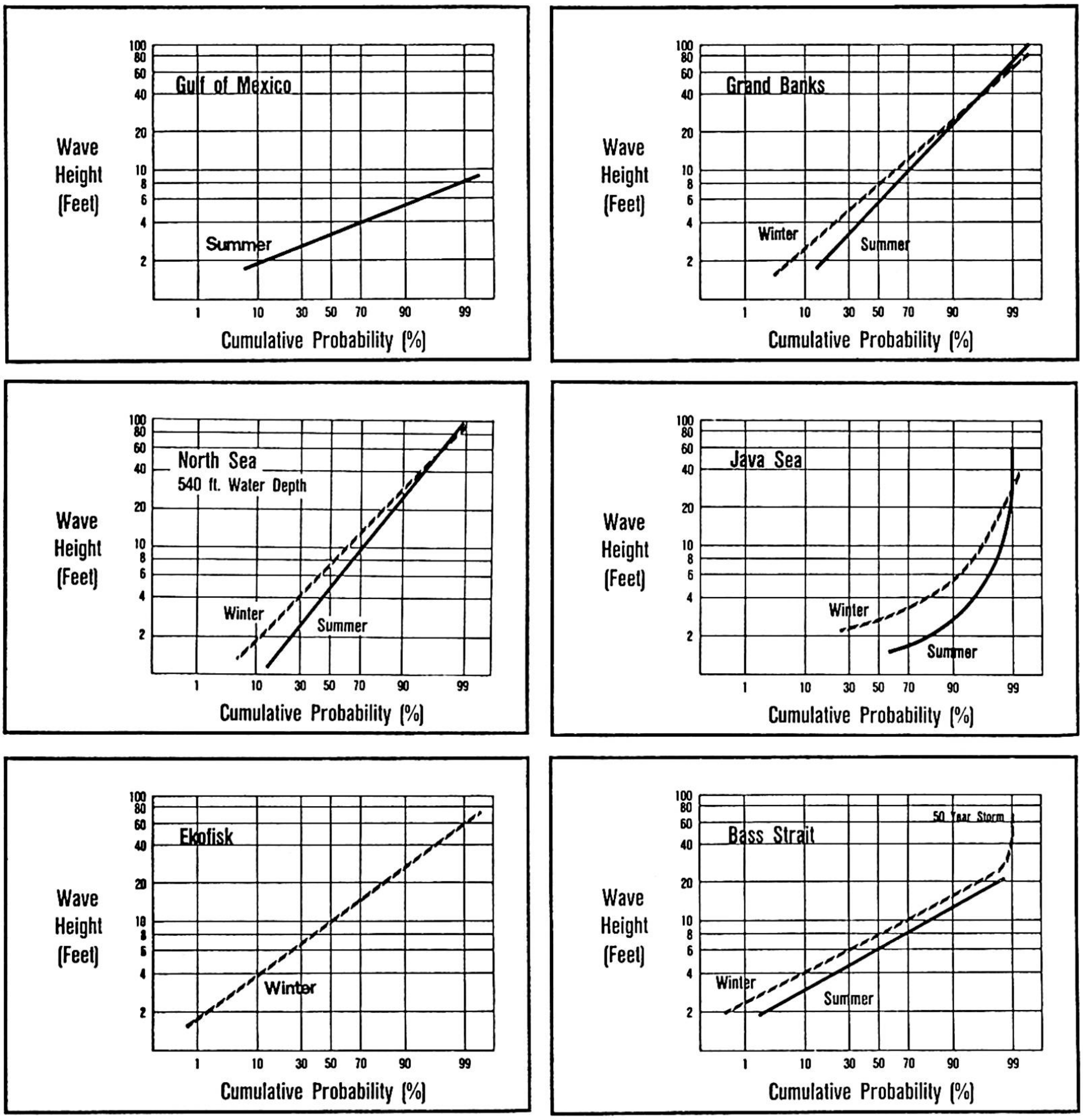

Fig. 22 Wave Data[1]

a steady operation of the barge and to prevent serious overstress in the line. Hence, larger more stable lay barges have been developed. Following the experience gained from operating the more stable semi-submersible drilling rigs, the modern pipelaying barges are also designed using the principle of semi-submergence. These units have lengths of up to 180 m and widths of 60 m. The greater deck lengths result from the requirement to handle double jointed pipes, allowing the barge to advance in larger increments. In order to design pipelines for these highly complex dynamic conditions, advanced programs of structural analyses, considering realistically both the elastic and potentially inelastic response

[1] Y. Goren,"Functional Design of Drilling and Construction Platforms," preprint 1973 Offshore Conference, University of California, Berkeley, California, USA.

of the pipeline during these pipelaying procedures are essential. Under those circumstances the associated structural design of the launching mechanism and the lay barge itself constitute an integral part of the overall design of these units.

DESIGN CONSIDERATIONS FOR OFFSHORE STRUCTURES

For the design of offshore structures the structural engineer has been and will be continually challenged to design for an environment which presents engineering complexities uncommon to the typical design considerations for land structures. In principle, structural design is determined by three major considerations, namely the environmental conditions of loading, the service requirements and the material properties. However, the final structural formulation will depend on the accuracy of specific analytical procedures which permit the evaluation of the structural response to any given set of loads. A design will be considered safe and functionally acceptable when the analytically-derived values do not surpass the limits set by the behavior criteria. The factor of safety to be reflected in these criteria, depends primarily on reliability of material properties, knowledge of design loads, accuracy of analytical procedures used to evaluate the structural response, serviceability, maintenance, and repair and replacement costs. The factor of safety can be reduced when engineering aspects as material properties and design loads are well defined and analytical procedures are highly reliable. On the other hand, economic factors associated with serviceability, maintenance, repair and replacement might well require an increase of the factor of safety. In the following sections, certain of the more important considerations in offshore design will be reviewed briefly.

ENVIRONMENTAL LOADINGS

The predominant environmental design loads are a direct reflection of the extreme sea state (waves, ice-flow, surface and tidal currents). The maximum design waves and wave spectra used in design differ significantly for different locations in the world. (See Fig. 22 and Table II.) Unfortunately, information of this nature is far from complete and often insufficiently accurate data has to be used to design drilling and stationary structures. The lack of this information is particularly critical in the design of drilling rigs which, during their design lifetimes, may have to operate in several different locations. Depending on the location, potential earthquake forces can play a predominant role in the design of stationary offshore structures. The study of offshore tower structures subjected to random earthquake excitations poses problems which are not encountered in similar land structures. Firstly, in the trussed ocean structures, the hydrodynamic forces on the structure introduce a non-linearity in the governing equations of motion, even when the material non-linearities are absent. Secondly, such structures have very high fundamental periods - from 2 seconds for towers in depths of about 120 m to over 5 seconds for those in depths of 300 m. This phenomenum prolongs the time to reach a stationary process. The response of offshore tower structures to earthquakes is, therefore, essentially a transient response. For the relatively more rigid gravity structures the non-linearity will be less severe. However, on the other hand the earthquake loads will be far greater and the capacity to absorb energy in a ductile fashion will be significantly less or non-existent.

MATERIAL CONSIDERATIONS

The previous aspects as applied to the design of relatively simple structures under well defined service conditions will normally pose little problem. However, for structures, which, because of the environmental circumstances will

TABLE II WAVE AND CURRENT DATA [1]

Location	100 Year Wave		50 Year Wave		Current Maximum Speed	Average Wave Height–Feet		Wave Height Above 10 Ft. Percentage
	Height Ft.	Length Ft.	Height Ft.	Length Ft.		Winter	Summer	
North Sea								
240 Ft. Water Depth	76	1093	74	1051		6.75	4.0	11.9
540 Ft. Water Depth	94	1394	89	1330	4.3 Ft/Sec.	8.33	5.65	22.8
Java Sea								
150 Ft. Water Depth	28		26.5		1.5 Knots.	2.62	2.25	Less than 1%
"Tsunami"–Max.	(100-135)							
Bass Strait			69 Highest 10%		2.2 Knots.	8.28	7.14	20.4
Gulf Of Mexico								
Eugene Island (88 Ft.)	53.5	949				4.87	3.83	Less than 2%
300 Ft. Water Depth	58	1232	48.5 (25 yrs)	986				
Grand Banks								
600 Ft. Water Depth	79		70 (25 yrs)		3 Ft./sec. 5 Ft./sec. (100 yrs)	7.77	5.65	20.9

be subjected to severe and variable loads, advanced engineering judgment regarding the behavior and material properties is of decisive importance. Offshore structures, or ocean structures in general, fall undoubtedly in the latter category. The complexity of the design of such structures is not only affected by the environmental and material considerations but also by economic aspects. The need of advanced engineering principles is of utmost necessity to asses the economic feasibility of developing offshore energy resources.

The design of offshore structures normally considers load conditions under both towing and in-service conditions. One of the main loadings to be considered is undoubtedly the survival load which reflects an extreme storm or earthquake condition and depends on the projected life of the structure. This condition will normally be the governing factor in the design of fixed structures in shallow waters of relatively calm seas. However, when fixed and mobile offshore structures are to be exposed to hostile seas and are placed in deep water environments the typical survival design loads may well cease to be the decisive load condition. Particularly,if based on an operational service capability, the life expectancy of the structure has to be well defined. In that instance, the repeated loads causing relatively low cyclic stresses may well become a predominant factor in the design of the structure. In that instance, an optimum design approach to develop a fatigue-resistant structure has to be considered as well. This aspect may be particularly critical for steel structures and the design of tubular steel connections. A similar concern has also been voiced regarding the fatigue strength of reinforced and prestressed concrete in sea water. Corrosion and corrosion fatigue are important factors which affect the design of offshore steel structures. Cathodic protection has proven to be effective to improve the life expectancy of offshore structures. In case of concrete offshore structures, limiting crack widths will be necessary to prevent corrosion of the reinforcing steel and possible fatigue. Also cyclic temperature effects due to the storage of hot oil and cold sea water may pose certain problems in thick-walled concrete storage facilities. Information regarding the generally complex material problems in both steel and concrete is necessary to enable the structural engineering to develop an optimum offshore design.

will stand in water 140 m deep. These two structures should be installed by 1976. The base of these gravity structures are typically constructed in a graving dock. After the walls of the lower cylinders are built high enough so that the entire unit can float, the dock is flooded and the structure is towed to a deep-water site to complete the construction, including installation of the deck sections. A limiting factor in the construction of these units is the lack of sufficient deep water facilities.

In addition to the five Condeep structures under contract, six other concrete gravity structures are presently under construction. Three of these units are of the Sea Tank design while a fourth structure is designed by Andoc. The two designs are in concept similar to the Condeep design, except that the oil-storage base is square rather than pentagonal and the decks are supported by four columns rather than three. The two remaining structures are designed by C.G. Doris and are similar in concept to the original Ekofisk storage tank, using the perforated breakwater concrete wall.

Recently, also steel gravity structures have been introduced. Four structures designed by Technomare and intended for the 85 m deep Loango field near the mouth of the Congo River are presently being fabricated. These structures consist of a steel tubular trussed tower, with the six columns arranged in a pentagonal array. This tower structure is supported by a trinagular base truss with a flotation cylinder located at each corner.

With future exploration and production moving to ever increasing depths neither the steel piled jacket nor the gravity structure seem to be a feasible solution, particularly when the sea environment is very hostile. Hence, recent studies have been focused on the development of tension-leg platforms. One design would have a steel tubular trussed frame - in principle similar to the triangularly based semi-submersible drilling rigs as shown in Fig. 4 - held down by vertical pre-tensioned cables from each of the three corner flotation columns. Such a system would require deep-water sea bed anchors drilled into the ocean floor.

SUPPLEMENTAL OFFSHORE FACILITIES

In addition to the production platforms the development of offshore resources requires equipment necessary to bring oil and gas to shore. Foremost in this category are underwater pipelines and pipelaying barges. Pipeline platforms and offshore storage tanks play an integrated role in the oil and gas transportation system. However, from a structural viewpoint their design criteria are similar to the stationary structures discussed earlier.

Laying pipelines in calm shallow waters has been extremely easy as compared to the complexities of laying lines in deeper waters and under adverse weather conditions. The larger water depths (up to 200 m),the increasing distance to shore, the higher operating pressures and the larger forces require large-diameter thick-walled line pipes. The launching of these deepwater pipelines from conventional lay barges has become virtually impossible. Therefore new very large barges have been developed and are presently in operation. In order to guide the pipe from the barge deck into the water the so-called stinger, which provides a predetermined curvature to the line has become a standard feature of these modern lay barges. These stingers can be structurally articulated (multiply hinged) or be built in a few sections with adjustable roller supports. Under all circumstances it is essential that the line remains under tension to prevent collapse. While the stinger configuration and pipe tension permit a control of the pipe deformation, the sea state will be the ultimate limiting factor in the design of the line. The roll, heave and pitch of the lay barge should be minimized to allow

SUMMARY

The search for offshore oil and gas and the subsequent development has opened up an almost entirely new field of structural engineering. Several types of structures, often very large, have been developed for both exploration and production. The design of these units is highly complex and requires detailed information regarding environmental and service conditions as well as material properties.

RESUME

Les forages marins à la recherche de pétrole et de gaz, et les suites qu'ils comportent, ont ouvert un champ presqu'entièrement nouveau aux charpentes. Plusieurs types de structures, parfois gigantesques, ont été réalisées tant pour l'exploration que pour la production. Le dimensionnement de ces ensembles est extrêmement complexe et nécessite une information détaillée concernant les conditions d'environnement et d'exploitation, et les qualités des matériaux utilisés.

ZUSAMMENFASSUNG

Die Suche nach Oel und Gas im küstennahen Meer und die zugehörigen Entwicklungen haben ein praktisch neues Gebiet im Bauwesen eröffnet. Verschiedene Typen von oft gewaltigen Bauwerken wurden für Suche und Förderung entwickelt. Entwurf und Berechnung derselben ist in hohem Masse komplex und erfordert eingehende Information über Umwelts- und Betriebsbedingungen sowie über Materialeigenschaften.

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IVb

Foundation Structures for Tall Buildings

Structures des fondations pour les maisons hautes

Fundationen für Hochhäuser

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1. FOREWORD

A foundation must transmit the load of a structure to the underlying soil or rock safely and without excessive settlement. Decisions in design of all foundations must always be made with this object in view. To achieve the object the following design principles are stipulated in the Structural Standards for Building Foundations of the Architectural Institute of Japan, which is a nation-wide organization of building engineers and architects:

i) The foundation should be supported by a strong, stable soil stratum or rock; the support by incompetent soils should be avoided.

ii) The foundation of a building should not be supported by different soil strata with noticeably different characteristics.

iii) A building should not be supported by foundations of different types.

iv) The stresses induced in the soil at the foundation base should be distributed as uniformly as possible throughout the plan of a building, and should afford a sufficient margin for safety against failure of the supporting soil strata as well as development of excessive settlement.

v) The bases of columns should be tied with foundation beams of sufficient stiffness so that the entire foundation forms a rigid grid to act as a unit.

Even though this paper is concerned with the foundation design for tall buildings, one needs hardly add anything to these general principles. However, because of characteristics peculiar to the structural systems and loading conditions of tall buildings, a number of problems may be pointed out which require special considerations. In relation to the foundation of a tall building, such problems will briefly be summarized and discussed in the following sections from three major standpoints:

1. Static weights
2. Wind forces
3. Seismic forces

2. STATIC WEIGHTS

Tall buildings are characterized, first of all, by their large weights far heavier than usual, low to medium-rise buildings. The fact frequently imposes rigorous problems on the design of foundations in various aspects.

AVERAGE WEIGHT - In Fig.1, average weights (dead plus design live loads excluding weight of foundation) are shown in metric tons per

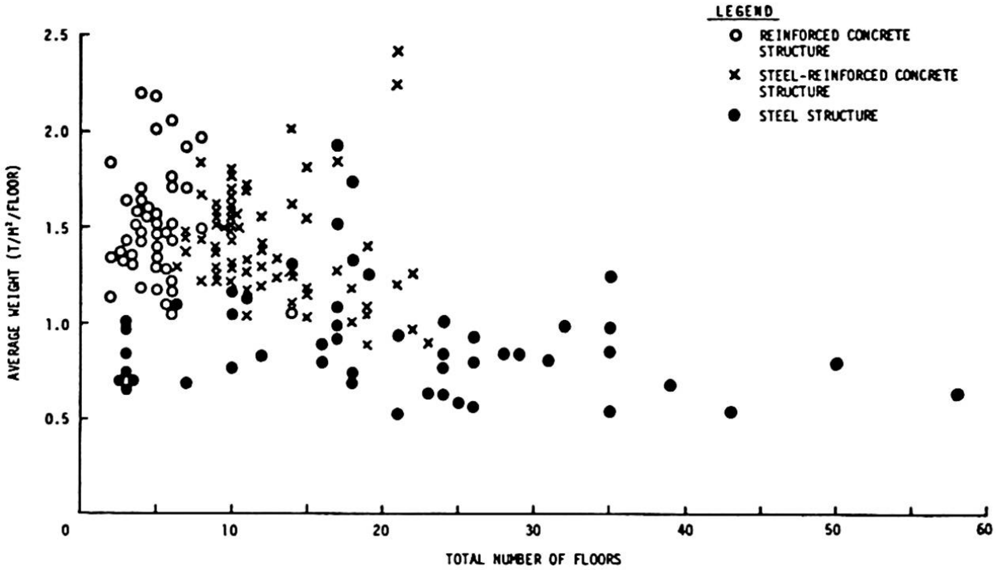

Fig.1

unit area per one floor with respect to typical Japanese buildings of different structural types. In spite of the wide range of values for low to medium-rise buildings, it may be seen for tall buildings that the values nearly tend to converge and may likely be between 0.5 and 0.8 tons/sq.meter/floor. In this connection, it is a common practice to accommodate a tall building with the basement of reinforced concrete or steel-reinforced concrete whatever the structural material of the superstructure may be. Unit weight of the basement alone is approximately equal to 1.75 tons/sq.meter/basement floor on the average.

In Fig.2, average weights of foundations (footing, mat, foundation beam, excluding weight of piles or piers) are also shown in terms of the total number of floors of a building, indicating that the values are in the range from 0.12 to 0.17 tons/sq.meter/floor for typical, tall buildings.

Hence, the estimate of the total average weight of a tall building including foundation may probably be in the range of 0.6 to 1.0 tons/sq.meter/floor. Although little information is available to the author concerning the weights of tall buildings in foreign countries, they may likely be of no significant difference from the above rough estimate.

BEARING CAPACITY - Fortunately, almost all major cities in this country are underlain by firm and stiff sandy or gravelly strata of sufficient thickness, which geologically belong to Diluvial deposits and can be encountered from the ground surface within a depth from 10 to 30 meters. Therefore, all of tall buildings designed so far are supported by those strata using either a spread foundation or a pier foundation of rather short length, being able to comply with Item i) in the aforementioned design principles. Up to the present time, no tall building has been constructed with long, flexible pile foundation.

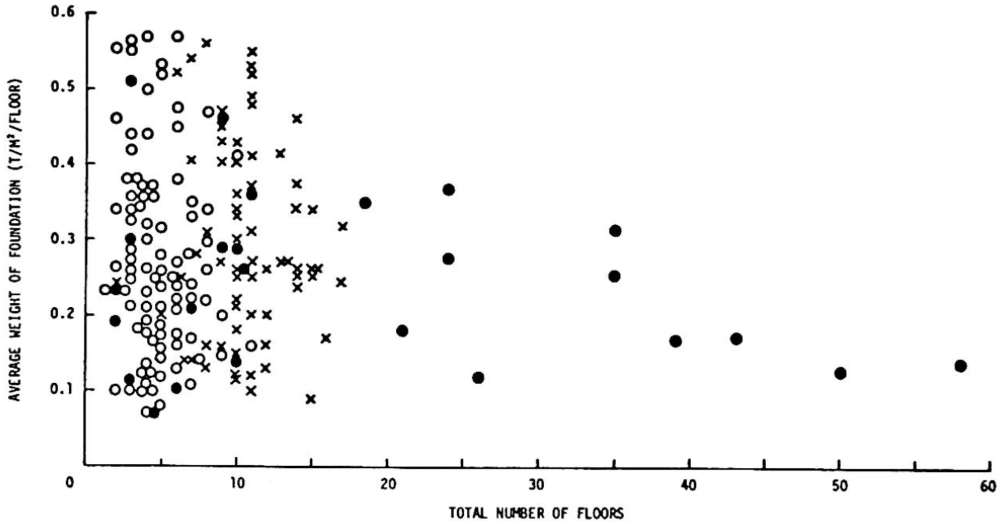

Fig.2

In practice, the ultimate bearing capacity of soil is estimated on the basis of Terzaghi's bearing capacity formula (Terzaghi 1951)

$$q_{ult} = \alpha c N_c + \beta \gamma B N_\gamma + \gamma D_f N_q \qquad (1)$$

where

- q_{ult} : ultimate bearing capacity
- c : cohesion of soil
- γ : unit weight of soil
- α, β : shape factor
- N_c, N_q, N_γ : bearing capacity factor
- D_f : depth of embedment
- B : width of foundation

The ultimate bearing capacity thus calculated at the design stage is usually verified by performing a field plate-loading test before or during construction. In Fig.3, a hatched zone is shown which represents the range of bearing pressure vs. settlement curves in the field plate-loading tests for sand and gravel strata supporting tall buildings. These tests are ordinarily performed by using a small loading plate 30 by 30 or 45 by 45 centimeters square. The majority of the loading tests give results in reasonable accord with those estimated by eq.(1). Furthermore, in cases where actual tall buildings provided with mat foundations of large dimensions and the basement of considerable depth are involved, real bearing capacity may probably be much larger than are shown in Fig.3 because of the effects of large width

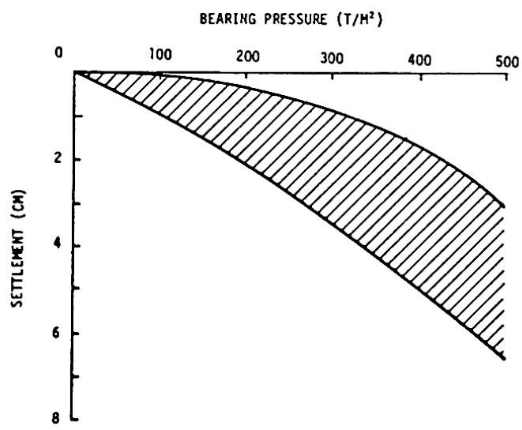

Fig.3

and deep embedment, which are accounted for by parameters B and D_f in the second and third terms of eq.(1), respectively.

When a pier foundation is used beneath the base level of a tall building, the bearing pressure at the bottom of a pier sometimes approaches to about 400 tons/sq.meter in our recent experience. In such a case, the full-size loading test of a pier is usually required and, so far, the safety of the bearing soil strata has been verified to be still sufficient. In some cases, for the purpose of estimating the ultimate bearing capacity at the design stage, Meyerhof's formula (Meyerhof 1950) is utilized, which is known to be pertinent for a deep foundation allowing larger bearing capacity factors than those in eq.(1). However, the compatibility of this analytical approach with the field loading test results does not seem to be completely established yet.

If the bearing pressure comes to an extremely high magnitude, the safety of foundation must be ensured not only against the sliding failure, which is primarily governed by shearing strength of the soil and analytically represented by Terzaghi's or Meyerhof's formula, but also against the so-called crushing failure which is related to crushing strength of individual soil grains themselves. Judging from a few test results performed so far, the ultimate bearing capacity resulting from crushing failure seems to be approximately

1800	tons/sq.meter	for sandy gravel
2000	"	for cemented sand.

In consideration of the aforementioned average weight of tall buildings, these knowledge and experience may provide a basis for stating that there is no practical limitation of the height of a building from the geotechnical standpoint, provided that competent materials with bearing characteristics not inferior to those shown in Fig.3 can be encountered within a reasonable depth from the ground surface.

SETTLEMENT - Any building undergoes settlement during construction and, under adverse conditions, may suffer a long-continuing, post-construction settlement caused by consolidation of the underlying cohesive soil deposits.

The former is usually referred to as immediate settlement. The immediate settlement results primarily from elastic compression of the soil mass beneath the loading area and, in addition, is associated with the recompression of rebound or heave taking place as a consequence of stress relief by excavation if the construction of a deep basement is involved.

The elastic settlement may be evaluated on the basis of theoretical analysis of an elastic solid. In the aforementioned A.I.J. Structural Standards, the following formula (Steinbrenner 1934, Fox 1948) is recommended for the evaluation:

$$S_o = \mu_H \mu_D q \sqrt{A}/E \tag{2}$$

where

S_o : elastic settlement
q : average loading intensity
A : contact area of foundation
E : modulus of elasticity of soil mass
μ_H, μ_D : settlement factors

With respect to a few tall buildings in the city of Tokyo, the results of computation on the basis of eq.(2) are compared in Table 1 with actually observed elastic settlements, indicating fairly reasonable agreement.

Table 1 : Computed and Observed Elastic Settlements of Tall Buildings

Building	Total Number of Stories	Settlement(cm)	
		Computed	Observed
A	39	2.2	0.8
B	43	1.5	1.5
C	50	1.3	1.5

For the computation in Table 1, moduli of elasticity were estimated on the basis of the load-settlement curves obtained by field plate-loading tests. In addition, in the case of Building C, the average modulus of the supporting soil mass was also measured dynamically by seismic exploration. It is interesting to note that the results of seismic exploration can provide another reasonable determination of static soil modulus if the observed value is pertinently modified by taking into account the strain levels within the stressed soil mass (Seed 1969).

Under the present situation, the analytical procedure to evaluate the settlement of a pier foundation seems not yet successful even as a crude approximation for practical purposes. Therefore, the evaluation is usually made by performing a full-size loading test or on the basis of available, previous data under appropriate conditions similar to the new site.

Differential settlements have entailed no critical consequences so far for tall buildings of a simple shape, probably because of the physical and structural reasons: (1) Since the buildings are designed so as to rest on a firm soil stratum in compliance with Item i) of the design principles, the maximum settlement is already limited to such an extent that no significant differential settlements occur, and (2) the tall buildings essentially possess high, structural stiffness to withstand vertical distortions and it functions, in turn, to minimize the differential settlements by redistribution of the column loads. Particularly, the presence of the basement having thick walls and a rigid foundation grid makes an important contribution in this respect.

An exception is the case shown in Fig.4, i.e., a tall building with structurally united low-rise annex. Large difference in weights of the high-rise and low-rise portions of the building shown in Fig.4(a) will probably result in significant differential settlements. A preventive measure is usually taken in practice in such a way that both portions are first constructed separately and, immediately prior to the completion, they are connected and finished. For a tall building with partial pier foundation as shown in Fig.4(b), which

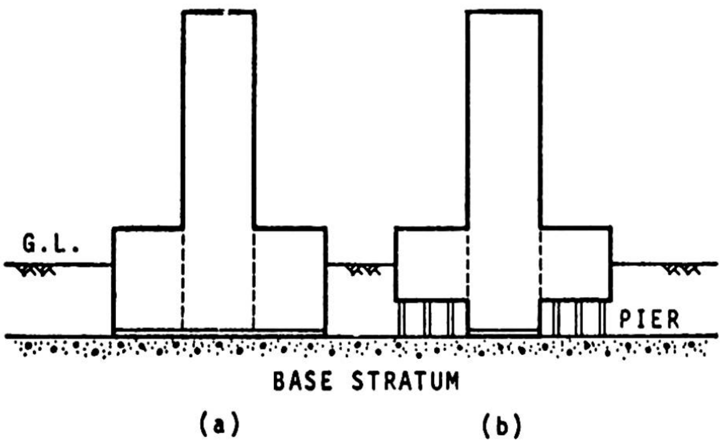

Fig.4

inevitably violates Itam iii) of the design principles, the same process of construction is frequently adopted since, as pointed out previously, much uncertainties are involved in estimating the settlement of a pier foundation.

The rebound at the bottom surface of excavation frequently presents a troublesome problem. Reports from a few building sites (Endo et al 1969) disclosed that the observed rebounds amounted to

32 mm for a 16 meters deep excavation
60 mm for a 25 meters "

The rebound taking place at the bottom of excavation can not be noticed unless special measurements are made. It is required for a precise observation of the rebound to install reference points immediately below the proposed bottom level of excavation and to carefully protect them against damage by excavation works. On the other hand, it appears possible to a certain extent to calculate the approximate amount of rebound by referring to the slope of unloading branch of load-settlement curve obtained by a field plate-loading test, provided that the stratification of underlying soil deposits is not too much complicated.

When there is no noticeable non-uniformity in the distribution of column loads, no special measures are taken in practice even if appreciably large rebound is anticipated. However, for a building with extreme difference in weight distribution, the problem of different recompression is usually solved by providing construction joints as described previously.

The second type of settlement results from consolidation of underlying cohesive soil deposits, which increases continuously even after the completion of a building. In fact, in the majority of cities in our country, the firm bearing stratum is usually underlain by clayey soil deposits. Fortunately, however, they are the sediments in the geological era of old Diluvium and in most cases highly over-consolidated, resulting in no critical consequence from the standpoint of consolidation settlement.

FLOATING FOUNDATION - Where a firm stratum of sufficient load supporting capacity can not be encountered within a reasonable depth from the ground surface as is frequently seen in a city underlain by marine or lacustrine sediments of low strength and high compressibility, an effective way of constructing a building of low to medium height is to use the so-called floating foundation.

The basic concept of floating foundation is that the weight of a building is compensated by the weight of the excavated soil so as to impose no additional loads upon underlying soil deposits as a result of constructing the building. The term of floating foundation may be rather misleading and, in a strict sense, it should be referred to as a compensated foundation (Zeevaert 1972).

Now, referring again to the average weights of tall buildings described previously, assume approximately

average weight of superstructure = 0.65 t/m^2/floor
average weight of basement = 1.75 t/m^2/floor
average weight of foundation = 0.15 t/m^2/floor
unit weight of soil mass = 1.50 t/m^3
average story height of the basement = 4.00 m

and let

N : total number of floors
N_B : total number of floors in the basement

W : total weight of the building per unit area
W_e : total weight of the excavated soil mass per unit area,
then

$$W = 0.65 \times (N - N_B) + 1.75 \times N_B + 0.15 \times N \quad (3)$$

$$W_e = 1.50 \times 4.00 \times N_B \quad (4)$$

To establish full compensation, eqs.(3) and (4) must be equated:

$$W = W_e \quad (5)$$

From eqs.(3), (4), and (5), one obtains

$$N_B \simeq 0.16N \quad (6)$$

and Table 2 shows a few numerical solutions of eq.(6). The above computation is merely a very crude arithmetic; nevertheless, eq. (6) or Table 2 implies that the application of the concept of fully compensated foundation may practically be precluded for a tall building of more than approximately 40 stories.

Table 2 : Required Depth of Basement for Fully Compensated Foundations

Total Number of Floors	20	30	40	100
Required Number of Basement Floors	3	5	6	16

3. WIND FORCES

Winds acting on tall buildings develop large temporary loading which must be delivered ultimately to the soil through the foundation or the basement walls. Shear force and overturning moment resulting from wind loads at the foundation level of a tall building are both characterized by their extremely large magnitude. Judging from experience of designing tall buildings up to the present time, wind forces and wind moment at the foundation level become larger for a typical building of more than 50 to 60 stories than those resulting from earthquakes even in this country of extremely high seismicity.

As a tall building is usually accommodated with the basement of a considerable depth, the large shear force is resisted by the difference of earth pressures acting on leeward and windward faces of the basement as well as the frictional forces of surrounding soils along its side and bottom faces. If a building is supported on pile or pier foundation, the lateral resistance at the top of the piles or piers contributes as well to withstanding the shear force. If this is the case, however, little benefit of frictional resistance along the bottom face of foundation may be expected because of loose contact of the soil or separation resulting from ground subsidence.

In practical analyses, the resistances by Rankine's pressure and shearing strength of the soil, acting on each corresponding face of the basement, are usually taken into consideration. As to the lateral resistance of piles or piers, the beam-on-elastic-foundation method or its extension to plastic range are frequently referred to. As is well known, however, the deformations required for full mobilization of these resistances are not necessarily the same and, moreover, may likely exceed the acceptable limit of movement. It is an important but difficult question at the present time to calculate the contribution of each resisting component compatible with the tolerable displacement, because so many complex factors are involved (DeSimone 1972).

The overturning moment causes an increase of bearing pressure or pile load on the leeward side and a decrease on the other side of the foundation; the former must be withstood by bearing capacity of the soil with an adequate margin for safety. If the overturning moment becomes still larger and the decrease of bearing pressure at the windward edge of the foundation exceeds the static pressure, the problem of uplift is encountered, which is an important matter peculiar to a tall building. The occurrence of uplift may be interpreted as the commencement of a transient motion from stable to unstable state of a structure, it can not be overlooked and should be avoided if possible.

To illustrate the possibility of uplift by wind loading, now assume a simple, prismatic model of a building as shown in Fig.5. The model is assumed to be directly placed on the ground surface. The total height is assumed equal to 3.5N meters, where N is the total number of stories and the average story height of 3.5 meters may be used without introducing much error. If 0.65 tons/sq.meter /floor is again assumed as the average unit weight of tall buildings, it is apparent that the bearing pressure of the supporting soil is equal to

$$\sigma_{static} = 0.65N \quad t/m^2 \qquad (7)$$

under static, permanent loading.

Then, consider the building is subjected to wind pressure; the wind pressure coefficient C = 1.2 and the wind pressure distribution $q = 0.12x^{1/4}$ (t/m^2) at the height of x meters are assumed, which may probably be an acceptable assumption for the purpose of approximate computation. Under these loading conditions, the overturning moment becomes

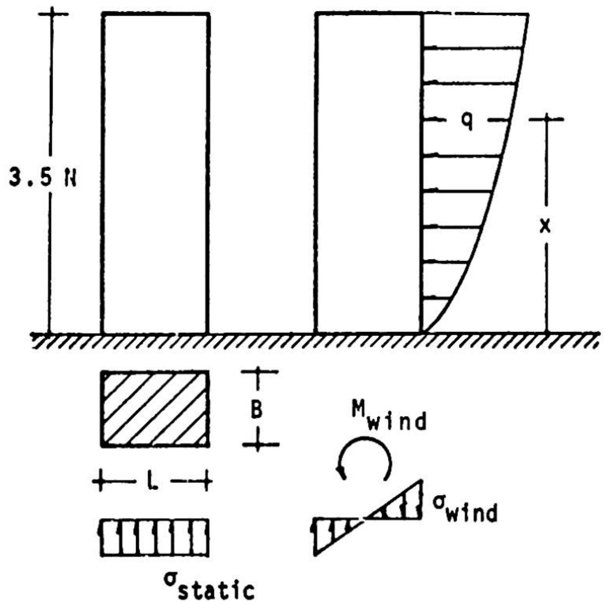

Fig.5

$$M_{wind} = \int_0^{3.5N} Cq\cdot Bdx = 1.072BN^{9/4} \quad t\cdot m \qquad (8)$$

and the maximum bearing pressure is represented by

$$\sigma_{wind} = (6/BL^2)\cdot M_{wind} \qquad (9)$$

Obviously, the condition to cause no uplift is $\sigma_{wind} \leqq \sigma_{static}$. From these equations, the minimum side length required for preventing the initiation of uplift may approximately be expressed by

$$L_{min} = 3N^{5/8} \qquad (10)$$

or as shown numerically in Table 3.

Table 3 : Minimum Side Length for Preventing Uplift

Total Number of Stories, N	20	40	60	90	120	200
Minimum Length, L, in Meters	20	30	40	50	60	80

In this connection, it may not be useless to pay attention to the behavior of foundation after the uplift has once taken place. Now, consider a loading plate as shown in Fig.6(a). The plate is also assumed resting on linear springs, which represent the subgrade reaction of the supporting soil and can not develop any tensile reaction. Then, corresponding to the combined effect of P and M, three different stress patterns can be distinguished; (b) no uplift, (c) on the verge of uplift, and (d) partial uplift. Obviously, under the action of constant, vertical load, the stress state transfers from (b) to (d) through transition point (c) as the moment gradually increases. The relationship between the moment and the angle of rotation can be expressed by

$$\left.\begin{array}{lll} M^* = \theta^* & \theta^* \leqq 1 & \text{for state (b)} \\ M^* = 3 - 2/\sqrt{\theta^*} & \theta^* \geqq 1 & \text{for state (d)} \end{array}\right\} \qquad (11)$$

where

$$M^* = M/(PL/6)\,,\ \theta^* = \theta/(2P/kL^2)\,.$$

The rotational characteristics of the loading plate expressed by eq.(11) are shown in Fig.7. It will be seen in Fig.7 that the behavior of foundation develops non-linearity if the moment exceeds a limit

$$M_{limit} = PL/6 \qquad (12)$$

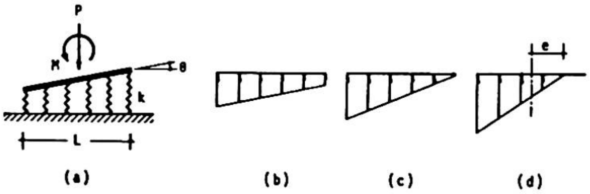

Fig.6

which is a counterpart of eq.(10) for a tall building subjected to wind loading.

Such non-linearity should be referred to as geometrical non-linearity to distinguish it from the one developed by plastic properties of the soil itself.

No serious problem of uplift has been actually encountered so far in the design of tall buildings, since it can readily be overcome by extending the lower portion and setting back the upper portion of the building or by providing adequately deep basement. However, careful attention should be paid to such a building of urban location where little space is allowed between the building and the property lines.

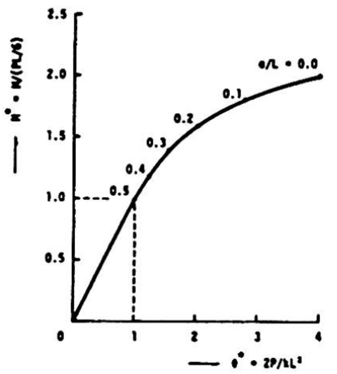

Fig.7

The behavior of a basement in delivering shear force and overturning moment to the surrounding soil is another difficult problem to be dealt with analytically. A proposal (Ohsaki 1973) is presented on the basis of the theory of elastic halfspace, but it is primarily of theoretical interest and difficult to apply to design purposes. Three-dimensional finite element approach to the problem appears to be useful and, in fact, is utilized frequently in practical design. However, it requires considerable judgment and experience for selecting representative values of soil parameters which should be taken into the analyses and, in addition, the agreement between calculated and actual behaviors has not yet satisfactorily been verified.

4. SEISMIC FORCES

As has been pointed out previously, the major concern in designing the foundation of tall buildings lies in the effects of wind forces rather than seismic forces under the majority of situations; nevertheless, the dynamic effects of an earthquake must still be of great interest to the building engineers, since they might affect the design of not only the foundation but the overall structure to a considerable extent.

Usually, the influences of the foundation and the underlying soil deposit upon a building are discussed by dividing them into three categories:

(1) soil amplification, which implies that the stiffness and thickness of entire soil strata affect the motion at the surface of the soil,

(2) dynamic soil-structure interaction, which represents the combined effects on the ground motion of the presence of a building and the deformation and energy-dissipation characteristics of the soil immediately beneath the building, and

(3) resonance, which may take place between the building and the ground motion thus developed, resulting in high stresses and large distortions in the building.

The foundations of tall buildings are carried down through soft soils to a stiff soil stratum or rock from necessity for bearing heavy weight. First, this fact may likely minimize disadvantageous effects of soil amplification, whether the building is directly rested on the bearing stratum by the spread foundation or it is supported on the pier foundation. A number of reports are available indicating that the difference between the response spectra of the earthquake motions observed at the base of buildings and those observed at the level of the bearing strata is hardly noticeable with buildings supported on piers of high stiffness (Ohsaki 1969). A case where a tall building is still associated with the large earthquake motion as a result of soil amplification is that the support of long, flexible piles is involved. It is also reported frequently that the characteristics of response spectra of the earthquake motions at the base of pile-supported buildings exhibit a tendency to resemble those which would be observed at the ground surface of the same site (Ohsaki 1969). It is extremely probable that piles of large flexibility develop the same movement with the amplified motion of the surrounding soil deposit.

Secondly, for a building with the foundation carried down to a stiff bearing stratum, the effects of soil-structure interaction are of minor significance from the practical viewpoint. The interaction induces rocking and swaying motions to a building and, as a result, shifts the fundamental period of the building toward the longer side. Numerically, however, its effect on buildings up to 40 stories high is not likely to exceed 4 percent if the shear wave velocity for the underlying soil is approximately 500 meters/sec (Whitman 1972). Furthermore, the response spectrum of input acceleration has in general a downward slope in the range of fundamental periods of tall buildings and, consequently, interaction always acts beneficially to reduce the stresses in structural elements in a tall building.

Thus, the dynamic design of a tall building subjected to seismic forces is almost solely related to characteristics of the motion of the stiff bearing material itself, which have been considered to rarely involve harmful components to tall, flexible buildings.

In recent years, however, a new finding is being frequently pointed out that even the seismic motions of rock or rock-like hard stratum involve the wave components of extremely long periods, which might have considerable damage-potential to a tall building on account of the resonance. Fig.8 represents two examples of velocity spectra for such rock motions during earthquakes of considerably short, epicentral distance. This fact of long-period inclusion is also observable in a large number of microtremor records obtained at the outcrops of rock or at the deep-seated hard strata, while the true character of such wave components has not yet been unmasked from the seismological standpoint. It might possibly be of

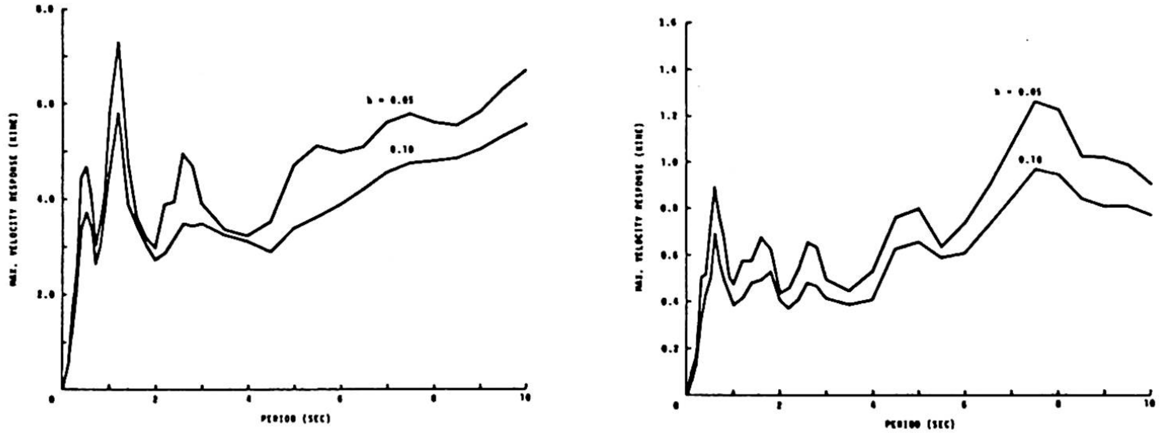

Fig.8

no little significance since, so far, such nature of input earthquake motions has seldom been taken into consideration as a basis of designing tall buildings.

5. CONCLUSION

A foundation in general must transmit any load of a structure eventually to the underlying soil or rock safely and without excessive settlement. Decision must always be made with this object in view for the foundation of a tall building as well.

However, because of characteristics peculiar to the structural systems and loading conditions of tall buildings, a number of problems are encountered which require special considerations.

Tall buildings are characterized, first of all, by their extremely heavy weights, and this fact frequently imposes rigorous problems on the design of foundations in various aspects. The average weight of tall buildings is estimated statistically to be in the range of 0.6 to 1.0 tons/sq.meter/floor.

Where a strong, stable stratum can be encountered within a reasonable depth from the ground surface, the ultimate bearing capacity and differential settlements usually give no critical consequence in spite of the heavy weight of tall buildings. To the differential settlements resulting from the large difference in distribution of loading intensity and the rebound of the bottom of excavation, attention should be paid however. Where the site is underlain by deep sediments of soft soils, the concept of floating foundation may hardly be applicable to a tall building, although it is quite effective for a building of lower height.

The foundation of a tall building is subjected to large lateral force and overturning moment during high winds. Behaviors of the foundation and the basement walls in transmitting these loads to the surrounding soils are considerably difficult to deal with analytically, being sometimes associated with another problem of uplift.

Seismic forces affect the design of foundations in a number of ways such as soil amplification, dynamic soil-structure interaction and resonance. If, however, the foundation is carried down through soft soils to a stiff bearing stratum, the disadvantageous effects of amplification and interaction are of minor significance, except in the case where a long, flexible pile foundation is involved. A finding that even the seismic motions of rock involve the wave components of extremely long period may be of no little significance for the design of tall buildings, requiring further studies.

In this Introductory Report, presentations are mostly made in general terms and it is not intended to discuss any specific problem in detail. A few simple, numerical examples are presented, but they are only for illustrative purposes.

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SUMMARY

In this introductory Report, a number of problems related to the design of foundations of tall buildings are briefly summarized and discussed primarily from the standpoints of static weights, wind forces and seismic forces with a few illustrative, numerical examples.

RESUME

Ce rapport introductif présente un résumé sur les problèmes concernant le dimensionnement des fondations pour maisons hautes; ils sont examinés essentiellement en vue des charges statiques, du vent et des effets sismiques. Quelques exemples sont présentés.

ZUSAMMENFASSUNG

Der vorliegende Einführungsbericht behandelt eine Anzahl von Problemen, welche sich beim Entwurf von Hochhaus-Fundamenten stellen. Die Probleme werden kurz zusammengefasst und vor allem im Hinblick auf statischen Lasten, Windkräfte und Erdbebeneinwirkung an Zahlenbeispielen diskutiert.

Fundationen für weitgespannte Brücken

Foundation Structures for long span Bridges

Structures des fondations pour les ponts de grande portée

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In den Berichten über weitgespannte Brücken finden wir meist nur knappe Angaben über die Gründungen, obwohl gerade sie oft mehr Können und Wagemut der Ingenieure erfordern als die Überbauten. Die Ingenieure widmen sich offensichtlich dem sichtbaren Teil der weitgespannten Brücken weit mehr als den unsichtbaren Gründungen mit dem Ergebnis, daß die Überbauten einen höheren Reifegrad der Entwicklung sowohl in technischer als auch in wirtschaftlicher Hinsicht erreicht haben als die Gründungen. Die Arbeitskommissionen der IVBH beschlossen daher, in den nächsten Jahren den Gründungen mehr Aufmerksamkeit zu schenken. Es besteht kein Zweifel, daß aus Erfahrungen und Beobachtungen bei ausgeführten Gründungen manche Erkenntnis gewonnen werden kann, die dazu beitragen wird, künftig solche Gründungen einfacher und wirtschaftlicher zu bauen. Der Generalberichter zu diesem Thema hofft daher sehr, daß zum Tokio-Kongress 1976 wertvolle und nützliche Berichte über interessante Gründungen für große Brücken eingehen werden. - In diesem Vorbericht sollen nur einige Probleme angesprochen werden.

Mehr Baugrunduntersuchungen nötig

Die wichtigste Vorarbeit für Gründungen ist zweifellos die Baugrunduntersuchung, die in groben Zügen schon der Wahl der Trasse des Verkehrsweges vorausgehen muß, damit unnötige Schwierigkeiten umgangen werden. Neben einer gründlichen geologischen Analyse sind nach wie vor Bohrungen möglichst mit ungestört entnommenen Kernproben das beste Mittel, die Baugrundverhältnisse zu erkunden. Für schwer belastete Fundamente sollte man dabei weder an der Zahl noch an der Tiefe der Bohrungen sparen, auch wenn der Geologe einen gleichmäßigen Baugrund erwartet oder über die Art der tieferen Schichten sicher zu sein glaubt. Jeder erfahrene Brückenbauer weiß, daß der Baugrund immer wieder Überraschungen bietet, sei es im Fels oder in Sedimentschichten. Für jede größere Gründung sind daher innerhalb der geplanten Gründungsfläche mindestens 4 bis 6 Bohrungen bis in eine Tiefe durchzuführen, die etwa $1,5\sqrt{F_g}$ (F_g = Gründungsfläche) entspricht. Die Kosten solch tiefer Bohrungen lohnen sich im Durchschnitt, weil nicht rechtzeitig erkannte Unregelmäßigkeiten im Baugrund während der Bauausführung stets zu äußerst unangenehmen Änderungen und Mehrkosten führen. Es gibt zahlreiche Beispiele in der Geschichte des Großbrückenbaues für solchen Kummer.

Trifft man Fels an, so ist der Brückeningenieur in der Regel glücklich, doch sollten gerade auch Schichtung, Schichtneigung, Klüftigkeit und dergleichen des anstehenden Felsens gründlich überprüft werden.

Dem Verfasser passierte es einmal bei dem Verankerungswiderlager einer Hängebrücke, daß der Fels zwischen zwei nur rund 20 m voneinander entfernten Bohrungen eine über 30 m tiefe Kluft aufwies, die anzeigte, daß der dem Fluß zu gelegene Felsteil sich schon talwärts geneigt hatte und daher zur Aufnahme großer Horizontalkräfte ungeeignet war. Es genügte nicht, den Spalt auszuheben, zu reinigen und mit Beton zu füllen, sondern die Verankerung mußte durch vorgespannte Bodenanker zusätzlich gesichert werden.

Felsgründungen

Wenn man gut gelagerten Fels antrifft, dann ist die Gründung in der Regel einfach. Die hohe Tragfähigkeit sollte jedoch in Zukunft mehr ausgenützt werden, indem man Pressungen von 20 bis etwa 60 kg/cm^2 je nach Güte des Felsen zuläßt. Es ist heute auch nicht schwierig, Biegemomente hoher Brückenpfeiler oder Verankerungskräfte von Hängebrücken oder Schrägkabelbrücken mit gebohrten, vorgespannten Felsankern aufzunehmen. Schon vor rund 15 Jahren hat ein Vergleich zwischen einer durch Betongewicht gesicherten Verankerung einer Hängebrücke und einer weitgehend aus gebohrten Felsankern bestehenden Lösung deutlich gezeigt, daß die Felsanker wesentlich billiger werden. Inzwischen wurde sowohl die Bohrtechnik als auch die Technik der Felsanker verbessert, so daß die Überlegenheit heute noch größer sein müßte. Dies gilt vor allem, wenn man gesundes Urgestein antrifft.

Caisson-Gründungen

Hat man angeschwemmten Boden, sei es Sand, Schluff, Mergel oder Ton, so sollte man die Fundamentgröße und damit die Bodenpressung hauptsächlich im Hinblick auf die für den Überbau erträglichen Setzungen wählen. Dabei spielen eigentlich nur die Setzungsdifferenzen oder zu Schrägstellung führende ungleiche Setzungen hauptsächlich bei statisch unbestimmten Hauptträgersystemen eine Rolle. Meist sind heute die Überbauten weitgespannter Brücken so schlank und gegen ungleiche Setzungen so unempfindlich, daß man beachtliche Setzungen ohne Nachteile in Kauf nehmen kann. Bei den heute weit verbreiteten Gummitopflagern, ob fest oder auf Teflon gleitend, kann man zudem ohne hohe Kosten den Überbau mit hydraulischen Pressen nachstellen, so daß Setzungen nachträglich ausgeglichen werden können. Die zulässige Bodenpressung nimmt mit der Gründungstiefe zu, weil die Grundbruchsicherheit durch die auflagernde Bodenschicht zunimmt. In der Regel ist es günstiger, Gründungen großer Brücken mit höherer Bodenpressung tiefer zu machen als die Lasten weiter oben mit niedriger Bodenpressung auf eine großflächige Fundamentplatte abzutragen, die dann große Biegemomente erleidet und entsprechend viel Stahl braucht. Es wäre erwünscht, daß gerade über diese Frage künftig mehr gearbeitet wird. Aus Probebelastungen großflächiger Pfähle weiß man, daß der Spitzendruck auf sehr hohe Pressungen gesteigert werden kann, ohne daß Grundbruchgefahr besteht, und so kann man zweifellos auch bei Caissons mit größerem Durchmesser ziemlich hohe Bodenpressungen ausnützen, ohne damit das Maß der Setzung viel zu vergrößern oder die Sicherheit in unzulässiger Weise zu verringern. Natürlich gibt es hier Ausnahmefälle bei Böden mit verhältnismäßig hohem Porenwassergehalt, wo die Setzungen dann über sehr lange Zeit anhalten und mehrmaliges Nachstellen oder dergleichen bedingen würden. In der Tendenz sollte man jedoch die höhere Tragfähigkeit der Böden in größeren Tiefen mehr als bisher ausnützen.

Tiefe Gründungen können einerseits mit Caissons, andererseits mit Pfählen ausgeführt werden. Caissons, die mit Druckluft abgesenkt werden, sind zwar sehr zuverlässig, aber wegen der erschwerten Arbeitsbedingungen wenig beliebt und auch nur bis ~ 30 m Tiefe geeignet. Die offenen Caissons können sehr tief gegründet werden und werden gern dort gewählt, wo die Gründung im Bett von Flüssen mit stark wechselndem Wasserstand, d. h. mit starken Hochwassern und hohen Strömungsgeschwindigkeiten, ausgeführt werden muß. In solchen Flüssen muß man mit erheblichen Verlagerungen der Flußsohle (Kolk, scour) rechnen, wobei in Flüßen wie Indus oder Ganges 20 bis 30 m tiefe Auskolkungen nicht ungewöhnlich sind. Sie bedingen Gründungstiefen von 40 bis 60 m, um die Pfeiler für Hochwasserkräfte standfest zu machen. Für solche Verhältnisse ist zweifellos der kreisrunde Caisson aus Stahlbeton immer noch die beste Lösung, wobei die Wanddicke reichlich gewählt werden muß, damit der Caisson der Ausbaggerung folgend absinkt. Sinkhilfen mit tixotropem Betonit oder mit Injektionsspülung sind merkwürdigerweise in diesen Ländern noch nicht verbreitet. Der Kreiszylinder ist günstig, weil die Bodenpressung vorwiegend Druck und nur bei ungleicher Verteilung geringfügig Biegemomente erzeugt, die in der Regel nicht gefährlich werden können. Eine schwache Ringbewehrung genügt, weil Biegemomente durch eine entsprechende Biegeverformung die Druckverteilung günstig beeinflußt. Kommt man mit einem Gründungszylinder nicht aus, so ist es in der Regel besser, z. B. zwei zylindrische Brunnen nebeneinander zu stellen und sie über dem Niedrigwasser kräftig miteinander zu verbinden, als zu rechteckigen Caissons überzugehen, die zur Aussteifung Zwischenwände brauchen und selbst dann noch verhältnismäßig viel Biegebewehrung erfordern (Bild 1).

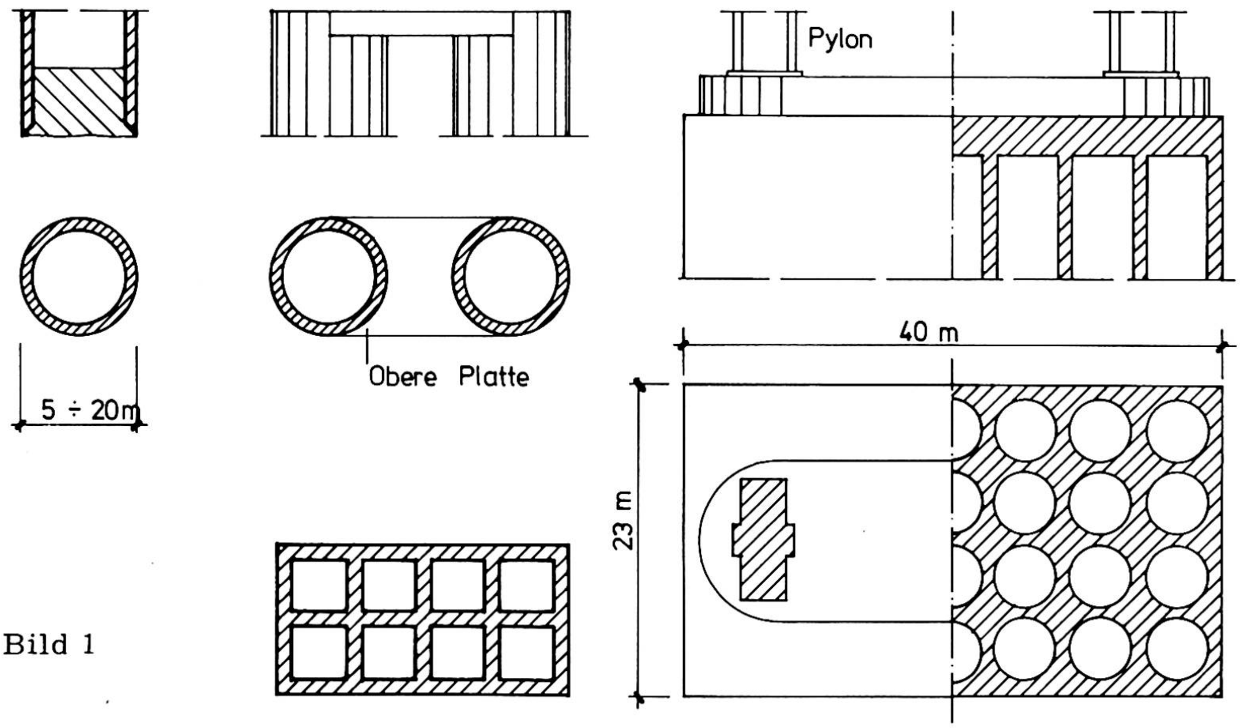

Bild 1

Die Caissons können bei Niedrigwasser auf Grund oder auf einer mit einer Spundwand geschützten Inselanschüttung hergestellt werden. Hat man die Gründung in schiffbarem tieferem Wasser auszuführen, so wird man heute den unteren Teil der Caissons in der Regel einschwimmen und zwischen wenigen Führungspfählen absenken oder - wie bei japanischen Brücken ausgeführt - mit großen Schwimmkranen genau am gewünschten Ort versetzen. Die japanischen Brückeningenieure haben für eine ihrer neuen Brücken mit einem Mammut-Schwimmkran von 2000 t Hubkraft einen vorgefertigten Caisson von rund 12 m Durchmesser und 30 m Länge versetzt - eine beachtliche Leistung.

Die Amerikaner verwendeten mehrfach Caissons, die aus einer ganzen Batterie von Kreiszylindern aus Stahlblech zusammengesetzt waren, die zum Einschwimmen oben luftdicht verschlossen wurden. Zum Absinken auf den Flußgrund werden zunächst die Zwischenräume zwischen den Stahlzylindern soweit wie nötig ausbetoniert. Der Boden wird dann in den Stahlzylindern ausgebaggert, bis der Caisson auf den tragfähigen Grund abgesenkt ist. Mit dieser Methode wurden die Pylonen der Tejo-Brücke Lissabon auf der Südseite bis auf 83 m unter dem Mittelwasserspiegel gegründet (Bild 1, rechts). Das Verfahren ist zwar sicher, aber wegen des hohen Stahlverbrauches für die meisten Länder zu teuer.

Pfahlgründungen

Wenn keine großen Kolktiefen und starken Strömungen zu berücksichtigen sind, dann beherrscht heute die Pfahlgründung selbst für sehr große Brückenlasten das Feld. Ein grosser Fortschritt war 1958 bis 1960 bei der Gründung der Brücke über den Maracaibo See in Venezuela erzielt worden, wo einerseits Rammpfähle mit Durchmessern bis zu 100 cm und Längen von über 50 m, und andererseits Bohrpfähle mit Durchmessern von 135 cm und Längen bis über 60 m erfolgreich verwendet wurden. Bei Probebelastungen wurden die Bohrpfähle mit rund 2000 t belastet, ohne die Grenztragfähigkeit zu erreichen. Erstmalig wurde dort ein erhöhter Spitzendruck durch Zementinjektionen an der flachen Pfahlspitze erzielt.

Wenige Jahre danach wurde bei der zweiten Brücke in Abidjan (Westafrika) ein Verfahren angewandt, um die Mantelreibung durch Injektionen unter Druck wesentlich zu vergrößern. Auch bei der größten Brücke Südamerikas von Rio de Janeiro nach Niteroi herrschen Pfahlgründungen vor. Bei den 1973 begonnenen beiden großen Schrägkabelbrücken über den Rio Parana bei Zarate-Brazo Largo wurden Bohrpfähle mit 2,0 m Durchmesser auf rund 70 m Tiefe unter dem Wasserspiegel in über 30 m tiefem Wasser gegründet (Bild 2), wobei die

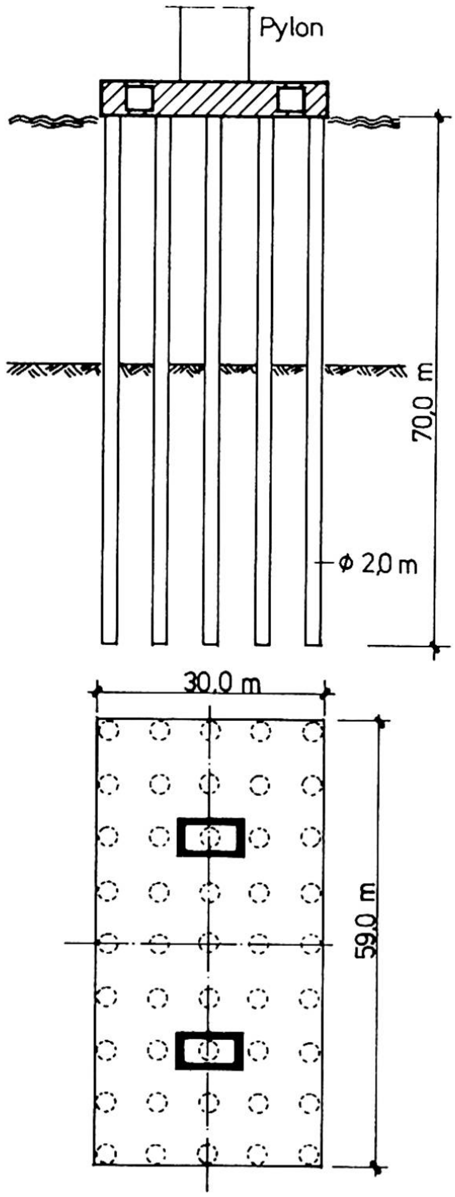

Bild 2: Pfahlgründung für Paraná-Brücken, Argentinien

Tragfähigkeit der Pfahlspitze wieder durch Zementinjektionen verbessert wurde. 35 Pfähle genügen so, um rund 36 000 t Last unter den Pylonen einer 330 m weit gespannten Schrägkabelbrücke für Eisenbahn und Straße zu tragen. Es besteht kein Zweifel, daß hier die Entwicklung noch nicht am Ende ist, sowohl die Durchmesser als auch die Längen der Bohrpfähle können weiter gesteigert werden.

In den dicken Kopfplatten, mit denen große Pfahlgruppen oben miteinander verbunden werden, treten je nach den Größenverhältnissen zwischen der Grundfläche der Brückenpfeiler und der Pfahlgruppe erhebliche Querkräfte und Biegemomente auf, die bei der Größe der Brückenlasten von vielen Tausend Mp sehr starke Bewehrungen bedingen. Man findet häufig, daß diese Bewehrungen aus sehr dicken Stäben (ø 40 bis 50 mm) in 4 bis 6 Lagen einfach übereinander angeordnet werden. Viele Ingenieure haben noch nicht erkannt, daß damit eine große Gefahr verbunden ist, weil die hohen Querkräfte extrem hohe Verbundspannungen zur Folge haben, die zum Spalten des Betons in der Ebene der Bewehrungslagen führen können. Diese Gefahr wird natürlich verstärkt, wenn mehrere Lagen dicht übereinander liegen. Solche Bewehrungen muß man in einem gegenseitigen Abstand von mindestens 4- bis 6fachem Stabdurchmesser verlegen, wobei die gesamte Bewehrung auf eine Höhe von 0,1 d bis 0,15 d verteilt werden sollte. Diese mehrlagigen Zuggurte müssen außerdem lotrecht verbügelt und auf die 1,2fache Breite der Pfahlköpfe beschränkt werden, wenn man Aufhängebewehrung zwischen den Pfählen vermeiden will. Man vergleiche hierzu die Berichte von W. Taylor über Versuche zu Pfahlkopfplatten der Lower Yarra Bridge, Melbourne, Australien, (vgl. CaCA-Technical Reports, London).

In der weiteren Entwicklung wird es zweifellos richtiger sein, diese großen Pfahlkopfplatten horizontal vorzuspannen, sie damit rissefrei zu halten und die Korrosionsgefahr zu bannen.

Gründungen in tiefem Wasser

In der Welt sind einige Großbrücken geplant, die Meeresstraßen mit verhältnismäßig tiefem Wasser überqueren und für die Brückenpfeiler in 60 bis 100 m tiefem Wasser gegründet werden müssen. In der Regel sind hierfür mit offenen Caissons hergestellte, massive Betonblöcke von riesenhaften Abmessungen vorgesehen, die einen erheblichen Aufwand erfordern und die Brücken

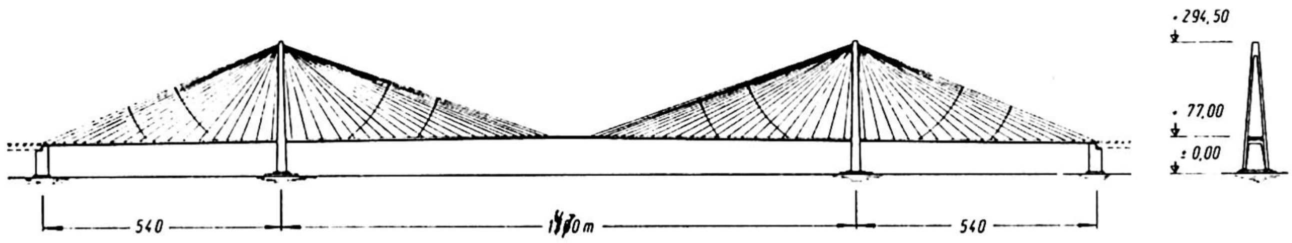

Bild 3: Entwurf Gruppo Lambertino für Brücke über die Straße von Messina, 1972

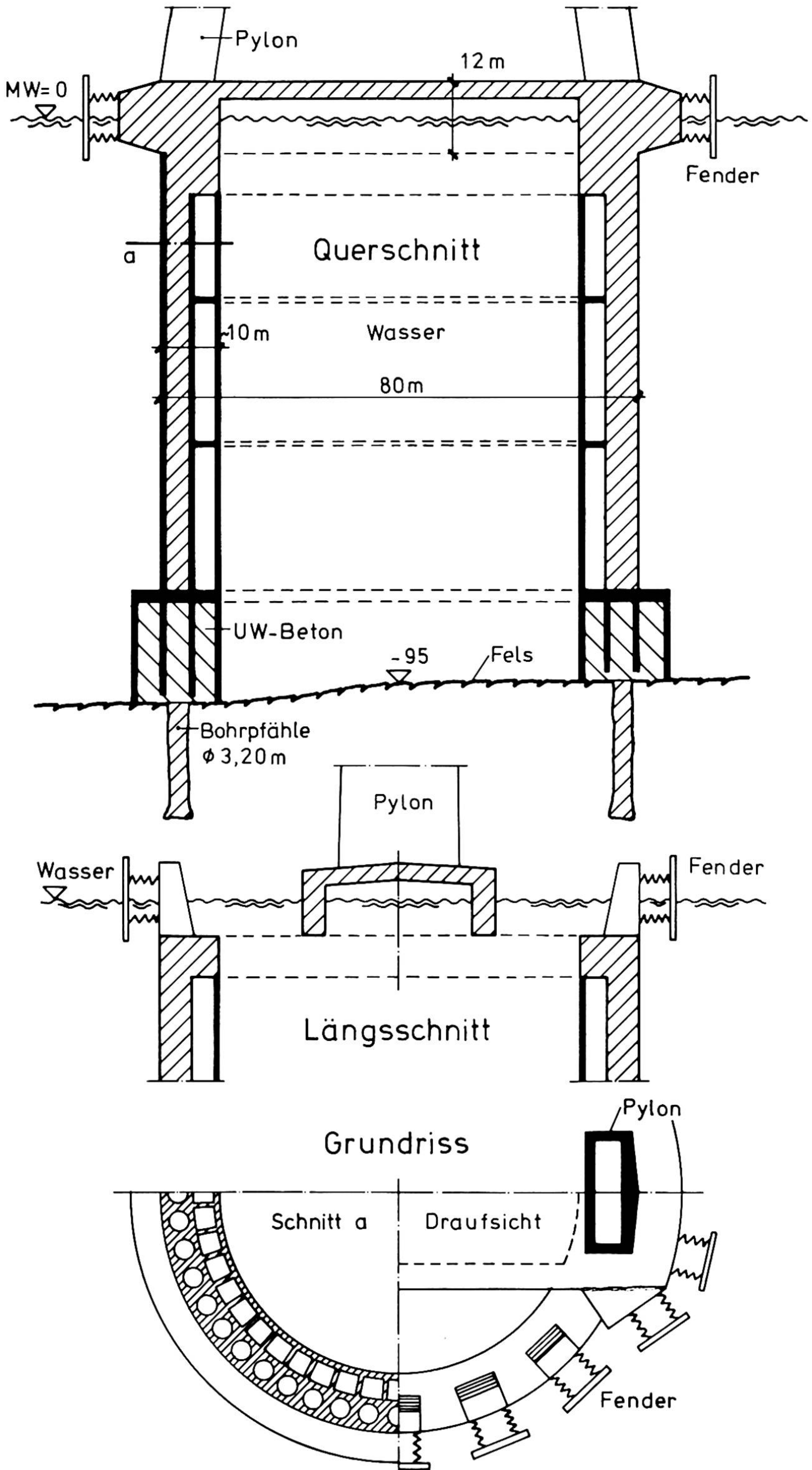

Bild 4: Vorschlag für die Gründung der Pylonen der Schrägkabelbrücke des Bildes 3

unnötig verteuern. Der Verfasser hat für einen Entwurf einer Brücke über die Straße von Messina als Schrägkabelbrücke von 1470 m Spannweite die Gründung der großen Pylone mit einem ringförmigen Zylinder von 80 m Durchmesser vorgeschlagen, in dessen Innenraum das Meereswasser verbleibt (Bild 3 und Bild 4) [1]. Der untere Ring wird dabei in einem Trockendock hergestellt, dessen Boden die Form der Felsoberfläche an der Gründungsstelle hat. Der Ring wird geflutet und in tieferes Wasser gebracht. Daraufhin wird die Zylinderwand mit Gleitschalung weitergebaut, wobei außen eine 5 m dicke, mit kreisförmigen Röhren durchsetzte Ringwand angeordnet ist, an die sich eine weitere rund 6 m dicke Zellenkonstruktion für den Auftrieb anschließt. Ähnlich wie beim Bau des großen Nordsee-Ölbehälters Ekofisk wird der Pfeiler schwimmend in genügend tiefem Wasser, jedoch in Ufernähe weitergebaut. Der fast fertige Pfeiler wird dann an seinen Bestimmungsort geschleppt, an versenkten schweren Ankerblöcken gegen die Tideströmungen festgehalten und durch Fluten der inneren Ringkammern auf die gesäuberte Felsoberfläche abgesenkt. Der Fels besteht dort aus einem Konglomerat. Die Fuge zwischen dem Felsboden und den Wänden der großen Ringkammer kann mit vorweg eingesetzten, mit Stahlgitter bewehrten Gummischläuchen mit Druckwasser abgedichtet werden. Danach werden die Ringkammern über Zuleitungen ausbetoniert.

Durch die kreisförmigen Röhren in der äußeren Zylinderwand hindurch werden nun Bohrpfähle in den Fels vorgetrieben, deren Tiefe von der Felsqualität abhängig zu wählen ist. In diesen Bohrpfählen werden Spannglieder verankert, mit denen die vom Eigengewicht der Brücke her ohnehin schon große lotrechte Druckspannung im Zylinder noch vergrößert wird, um ihn gegen Erdbeben- bzw. Meeresbeben-Kräfte sicher zu machen. Vor diesem Vorspannen werden die Zylinderröhren natürlich ausbetoniert, so daß der äußere 5 m dicke Mantel des Zylinders massiv wird. Falls erforderlich, können auch die Zellen des unmittelbar anschliessenden inneren Ringes vollbetoniert werden.

Die Pylonenbeine der Brücke stehen unmittelbar auf dem Ring und werden durch einen 12 m hohen und rund 32 m breiten U-förmigen Riegel miteinander verbunden. Nur dieser Querriegel ragt aus dem Wasser heraus. An den Zylinderwandungen sind rundherum kräftige Fender an aus dem Wasser herausragenden Pfeilern befestigt, die einen eventuellen Schiffsstoß mit großem Verformungsweg von etwa 3 bis 4 m abfangen.

Nach dem erfolgreichen Bau von Ekofisk in der rauhen Nordsee, die bis zu 20 m Wellenhöhe aufweist, sollten keine Zweifel mehr daran bestehen, daß auch Brückenpfeiler solcher Abmessungen in Wassertiefen von 80 bis 100 m gegründet werden können.

Dieser Vorschlag für die Gründung eines Pfeilers in der Straße von Messina wurde hier nur als Anregung für solche Projekte beschrieben.

Zum Kongress werden zweifellos manche interessante Berichte über Gründungen weitgespannter Brücken vorliegen, zudem im gastgebenden Land Japan die zur Zeit größten Brücken der Welt gebaut werden.

[1] F. Leonhardt und W. Zellner: Vergleiche zwischen Hängebrücken und Schrägkabelbrücken für Spannweiten über 600 m IVBH-Abhandlungen Band 32-I, Zürich 1972

ZUSAMMENFASSUNG

Gründlichere Baugrunduntersuchungen und höhere Pressungen werden empfohlen. Grosse Caissongründungen sollten bevorzugt mit zylinderförmigen Caissons gebaut werden. Bei den Pfahlgründungen werden Bohrpfähle bereits bis 2,5 m Durchmesser und bis 70 m Tiefe eingesetzt. Einige Hinweise für die Bewehrungen von Pfahlkopfplatten werden gegeben. Für Pfahlgründungen in sehr tiefem Wasser, 60 bis 100 m, wird ein Vorschlag mit einem offenen Zylinder beschrieben, der für eine 1470 m weit gespannte Brücke über die Strasse von Messina entworfen wurde.

SUMMARY

More, better and deeper soil investigations and higher allowable soil pressures under large bridge foundations are recommended. Caissons should preferably be designed as circular cylinders. For pile foundations, drilled piles with diameters up to 2,5 m and dephts of 70 m have been built. Some advice for pile cap reinforcement is given. For foundations in very deep water (60 to 100 m) a proposal with an open cylinder is described, as it was designed for a cable stayed bridge with 1470 m span across the Straits of Messina.

RESUME

Il est recommandé de prévoir de meilleures analyses des sols de fondation et de considérer des pressions admissibles supérieures pour les fondations des ponts de grande portée. Il est préférable d'envisager des caissons cylindriques. En ce qui concerne les fondations en pieux, il faut savoir que des pieux d'un diamètre de 2,50 m et d'une hauteur de 70 m ont déjà été réalisés. Quelques indications sont données pour la protection de la tête des pieux. Pour le cas de fondations en eau très profonde (60 à 100 m), la proposition d'un cylindre ouvert est faite, comme c'est le cas pour le projet de pont suspendu, de 1470 m de portée, sur le détroit de Messine.