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IV

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

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

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

IVa

Constructions en mer

Bauwerke im Meer

Offshore Structures

IVb

Structures des fondations pour les maisons hautes

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Foundation Structures for Tall Buildings

IVc

Structures des fondations pour les ponts de grande portée

Fundationen für weitgespannte Brücken

Foundation Structures for long span Bridges

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

J.G. BOUWKAMP
President
Offshore Development Engineering, Inc.
Berkeley, California U.S.A.

Offshore Structures

Mr. Chairman, Ladies and Gentlemen,

The contributions to the theme, Offshore Structures, as published in the Preliminary Report, covered not only general design aspects of both steel and concrete structures, but also focused on at least one of the more significant design details, namely tubular joints.

The contributions by Roret and Ciolina on steel structures, and Gerbault and Xercavins on concrete structures, give an up-to-date general review of several structural systems which have been developed for the exploration and production of hydrocarbon deposits in offshore regions. Unfortunately, space limitations prevented discussion in depth of detailed technical problems associated with the design, construction and installation of these structural systems. Most problems are related to the dynamic behavior of the structures under the effects of waves, currents, wind, ice and earthquakes, and require full consideration of the soil-foundation-structure-fluid interaction. For instance, soil conditions are a major factor in evaluating both the short- and long-term dynamic response of gravity platforms under cyclic, wave-induced loads. Equally significant is the dynamic behavior of steel tubular trusses and their connections under such load conditions. In addition to the response to extreme waves and winds, earthquakes are another important factor. One of the most significant elements in the structural design of steel tubular trussed towers is the tubular joint. It was, therefore, particularly fortunate that the contributions of Kurobane, et al, and Okumura, et al, were addressed to the performance of tubular joints. In light of my own experience in this subject, I would like to focus on this particular aspect in greater depth. However, first I would like to note the contributions by Shimada and Yamamoto, and Coulard. Although the authors focused on harbor and coastal structures, rather than offshore structures, they do draw attention to the fact that offshore developments are not complete without the necessary onshore facilities for equipment maintenance and transfer of oil and gas to shore.

The history of tubular joint fatigue research goes back almost twenty years. At that time the primary focus was directed towards developing joint design criteria which would assure the overall structural integrity of a tower structure under extreme wave and wind conditions.(1,2,3,4) These criteria reflected the environmental conditions in the relatively shallow waters of the Gulf of Mexico where extreme waves were the critical design condition. The resulting design criteria based on the "punching shear" strength concept resulted in joints which were capable of withstanding these extreme load conditions, even when applied for a limited number of loading cycles. Consequently, in the initial studies, consideration was given to the low-cycle fatigue resistance of these joints. The design was typically a jacket-type multi-legged trussed tower with tubular members. Joints normally consisted of 42 to 48 inch diameter column members and tubular brace members of approximately 16 to 24 inches in diameter. The design of these joints was typically based on extreme load conditions which would produce a nominal stress reversal in the diagonal members of 28 ksi.

These studies readily indicated that the flexibility of the column member wall was the main source of stress concentrations and consequently resulted in early fatigue failure; "hot spot" stresses were identified. In order to stiffen the column wall, one considered increasing the column wall thickness, using external ring stiffners or in-plane gusset plates, or possibly, overlapping and interwelding the web members.(5,6,7,8,9) Comparative studies clearly indicated that a joint with a thickened column wall and non-overlapping web or branch members was superior from the point of view of strength and low-cycle fatigue resistance. These initial studies successfully guided the design of tubular joints under these particular environmental conditions (extreme waves). However, these design rules became inadequate where offshore developments moved into more hostile regions. Not only the extreme sea state, but also the repeated cyclic low-forcing wave effects became significant design considerations. Hence, a more general approach to fatigue and cumulative fatigue damage became necessary.

Comparative studies of the type noted before could no longer provide sufficient input to determine the structural response resulting from the broad load spectrum which now had to be considered in design. Consequently, other theories and their applicability had to be evaluated. Miner's linear cumulative fatigue damage hypothesis was used in establishing the effect of cumulative fatigue damage. Equally important, however, was the need of determining the stress versus number-of-cycles, or "S-N", curves necessary to determine the ultimate fatigue resistance for given stress levels. Considering the complex environmental conditions and material response, different theories have been considered and their application to tubular joint design evaluated. In this respect, a linear-elastic fracture mechanics approach to determine theoretical S-N curves has been found to yield a satisfactory correlation between theoretically predicted and experimentally observed fatigue crack behavior.(10)

In principle, this type of approach is needed in order to develop design criteria for the many different systems which are designed to operate in the often hostile offshore regions now considered for development. The severity of the environmental loads requires a new approach to tubular joint design considering both joint strength, joint stiffness and fatigue reisitance. There is no doubt that solutions can be obtained. However, it will require the most advanced integrated approach of loading methodologies, computer modeling techniques, analysis, fabrication and material science. Not only material properties affecting the ultimate response are significant, but also the prediction of the highly complex dynamic response of offshore structures, both stationery and floating, steel or concrete. In that instance, the combined effects of waves, currents, winds, soil conditions and earthquake loadings should be fully considered. Only a highly advanced approach in analyzing the anticipated performance will provide the basis for a satisfactory design.

I noted in the last paragraph, concrete structures as well. While I did address myself to the fatigue aspect of tubular joints and the overall design requirements of steel platforms, concrete or pre-stressed concrete structures require an equally rigorous design approach. Consequently, the engineer's total understanding of the many environmental and structural performance aspects in every phase of design, is essential. Extending knowledge and performance data to structures similar in concept, but located in different environments, does require the utmost of engineering attention in order to prevent the potentially catastrophic outcome of extending the "present state-of-the-art" to new regions without full consideration of all factors involved. In that respect, I hope that the subsequent discussions and future contributions published by the IABSE on this subject may aid the profession in a field which , because of the short history and engineering complexity, requires our fullest attention.

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Ultimate Strength Design Formulae for Simple Tubular Joints

Formules du calcul à la résistance limite pour les noeuds simples de profilés circulaires

Formeln für die Ermittlung der Traglast von einfachen Knotenpunkten in Rohrprofilen

YOSHIAKI KUROBANE
Professor

YUJI MAKINO
Instructor
Kumamoto University
Kumamoto, Japan

YOSHIYUKI MITSUI
Associate Professor

1. INTRODUCTION

One of the Committees in the Architectural Institute of Japan is carrying out revision of the "Specification for Design of Tubular Structures in Steel" that was first published in 1962. Although the Specification is applicable to the building structure, it may provide a good deal of information for the design of offshore structures of tubular members.

This report discusses about the experimental grounds of those provisions for the design of the tubular X, T, Y and K-joints for static loadings which are presently under deliberation in the Committee. The provisions are based on the ultimate strength formulae that were selected for such use from the results of a series of regression analyses of the test data obtained in Japan and the U.S.A. since 1963*. The derivation of the formulae is described in detail in Reference [1].

All the existing ultimate strength formulae for these joints have been subjected to two questions as follows:

1. Most of the existing ultimate strength formulae tend to overemphasize the strength of the T, Y and K-joints when the diameter to thickness ratio of the chord (D/T) becomes greater than about 50.
2. The strength of the K-joint increases as the two braces intersect and then overlap with each other. This behavior is not adequately taken into account in the existing formulae.

To overcome these difficulties the reanalyses of the test data were carried out as a continuation of the past studies by the authors and their colleagues[2],[3].

The strength of the tubular joints under static loads is an influencing factor in determining the design of any tubular structure and yet it still covers some areas that are not fully understood at the present stage, which may be clear from the later discussions in this report. In this regards the authors wish to welcome any comment on the proposals presented in this report.

2. DEFINITION OF ULTIMATE STRENGTH

The ultimate strength referred herein is the maximum axial compressive force applied at the brace ends when a joint fails as a result of excessive local bending deflections of the chord walls. The strength of a joint that fails owing to

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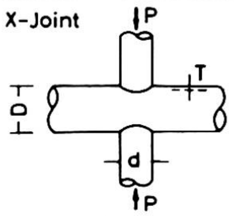
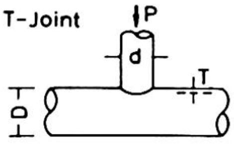
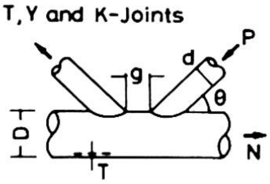
* Additional test data are now being gathered from Europe through the activity of the Subcommission XV-E of the IIW.

failures in a member, such as fracture of the tension brace or local instability of the compression brace, is outside the scope of the present definition of the ultimate strength of the joint.

The local failure of the chord walls occurs also at the points where tension braces are attached. The final rupture of these joints is controlled by cracking of the chord and/or brace walls at the toes of the brace to chord welds. The joints that fail in this manner always attain a far greater strength than that in the former case where the braces are under compression. The ultimate strength data for the joints under tension should therefore be treated separately and are excluded from the regression analyses in Referenc [1].

Table 1 Proposed Ultimate Strength Formulae

Type of Joints	Predicted Ultimate Strength, P_u
X-Joint 	$P_u = \dfrac{6.57}{1-0.810\,d/D}\,\sigma_y T^2$ (a)
T-Joint 	$P_u = 6.43\,[1+4.60(\frac{d}{D})^2]\,\sigma_y T^2$ (b)
T, Y and K-Joints  $\bar{N} = N/\sigma_y A$ $A = \pi(D-T)T$	$P_u = 2.11(1+12.1\frac{d}{D})\,f_g\,f_\theta\,f_{\bar{N}}\,\sigma_y T^2$ (c) $f_g = 1+3.88(1-20.9\frac{T}{D})(1-0.530\frac{d}{D})\cdot$ $[1+\frac{2}{\pi}\tan^{-1}(0.237-0.183\frac{g}{T})]$, but not less than 1.0 $f_\theta = (1-0.167\cos\theta+0.049\cos^2\theta)/\sin\theta$ $f_{\bar{N}} = 1+0.262\bar{N}-0.391\bar{N}^2$ ($\bar{N}$: positive for tension)

According to the past tests, most tubular joints reached the maximum load after full plastic deflections of the chord walls were produced at the local portions where the braces were attached, and then unloading took place. A typical load-deflection curve of such joints is shown by the curve 1 in Fig. 1. In some joints, however, the overall stiffness increased again after they sustained full plastic deformations of the chord walls and eventually carried a greater load than the first maximum load. The load-deflection curves of the latter type are shown by the curves 2 and 3 in Fig. 1. The ultimate strengths used for the analyses were the first maximum loads that were attained by the joints after sufficient areas of the chord walls yielded.

K-joints often fail in a combination of various failure modes depending upon the dimensions of the joints. Even though the final failure of a K-joint was governed by one of the other modes than the excessive local deflections of the chord walls, the ultimate strength of the joint was included in the data so far as the joint sustained full plastic deflections of the chord walls under the compression brace at the maximum load.

3. PROCEDURE OF REGRESSION ANALYSIS

The multiple regression analyses were carried out to develop the best-fit equations of the ultimate strengths of the joints. In the process of building a mathematical model for a prediction equation, it was first assumed that the joint was able to be replaced by a simple and fictious structure of which ultimate strength would represent the ultimate strength of the actual joint.

Such a simplified model structure is a ring with an effective width Be. The ring has the same diameter D, thickness T and yield stress σ_y as those of the actual joint and is subjected to concentrated forces acting at d distant points, where d is the outer diameter of the brace. These concentrated forces represent the axial compressive force P in the brace.

According to the simple plastic theory, the collapse load of the ring Pu is given by the equation,

$$Pu = \frac{Be}{a} f_0(\frac{d}{D}) \sigma_y T^2 \quad (1)$$

where a is the mean radius of the chord. Be and $f_0(d/D)$ are functions of geometrical parameters of the joint and vary with the type of the joint.

Therefore, the model may be written in the form,

$$Pu = f_0(\frac{d}{D}) f_1(\frac{d}{D}, \frac{T}{D}, \frac{g}{T}, \theta, \bar{N}) \sigma_y T^2 \varepsilon \quad (2)$$

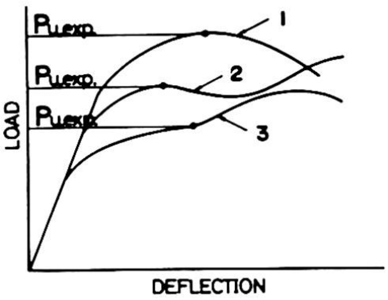


Fig. 1 Examples of Load-Deflection Curves

in which Pu is the ultimate strength of the joint, g is the clear space (gap) between the two braces, θ is the angle of intersection between the compression brace and the chord and $\bar{N}$ is the dimensionless axial stress in the chord (See Table 1). ε is the error term, which was assumed to be multiplicative rather than additive because the model (2) consists of multiplicative terms of the influencing factors each of which has a certain physical meaning [1]. It is assumed here that errors ln(ε) are independent random variables with mean zero.

Since the postulated model was nonlinear in the parameters, the linearization and iterative techniques were exercised to fit the model by the method of least squares [4]. A series of such analyses were performed with several alternative models for the functions f_0 and f_1 of Eq. 2. The resultant regression equations were compared on the basis of the "multiple correlation coefficient R^2". The final selection of an equation was made such that the selected equation would explain the variation of the ultimate strength data better (attain a larger R^2) with less predicting variables.

In Reference [1] are shown all the data used for the analyses and also the reference sources of them. An effort was made to utilize as far as possible measured values rather than nominal values for the independent variables. Although the yield stress in the circumferential direction may be more meaningful in this model, such yield stress is not usually measured in most experimental works. σ_y,s adopted herein are the longitudinal yield stresses measured on tensile coupons cut from the as-rolled chord materials.

4. RESULTS OF ANALYSES AND DISCUSSIONS

The three equations shown in Table 1 were selected as the ultimate strength prediction equations for the tubular X, T, Y and K-joints. Eq. c is applicable to any of the T, Y and K-joints, where g is infinitely large in the T and Y-joints and becomes negative when the braces overlap in the K-joint. R^2 was of 92%, 95% and 91% in Eqs. a, b and c, respectively.

The residuals provided by Eq. c are plotted overall in a form of a frequency histogram (Fig. 2). It appears that the residuals follow a normal distribution. According to a chi-square goodness of fit test this assumption of normality was found to be acceptable at the 0.05 level of significance. From this it may not be unreasonable to assume that, if the model is correct, errors ln(ε) are normally distributed.

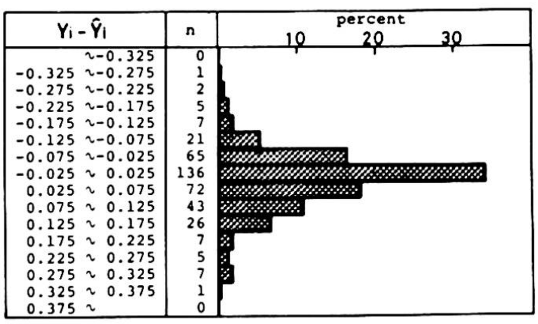


Fig. 2 Histogram of Residuals

Since the models are nonlinear, statistical tests that are true for the linear case do not apply. However, since the number of observations n is large, the 95% confidence limits for an individual predicted value are approximately given by

$$Pu \cdot e^{\pm 2s} \tag{3}$$

where s is an estimate of standard deviation and is approximately obtained by the equation,

$$s = \sqrt{\frac{\text{(residual sum of squares)}}{n - p}}$$

in which p is the number of parameters in the regression equation. The above statement is valid only when the errors $\ln(\varepsilon)$ are normally distributed.

The predicted mean value Pu and the approximate 95% confidence limits according to (3) are compared with the test results "Pu,exp"in Figs. 3 through 6. These plots indicate no strong abnormality in the residuals and the present regression analyses would not appear to be invalidated.

It is important to note that the formulae in Table 1 are applicable only within the ranges of variation of the predicting variables. Figs. 8, 9 and 10 illustrate how the predicting variables varied in the test data.

The Japanese Specification referred to earlier tentatively assumes a factor of safety of 2 on the predicted ultimate strengths of the X, T, Y and K-joints. The allowable force may be increased 50% above Pu/2 when the joint is under combined permanent and temporary loadings. This safety factor appeared to be conservative from Figs. 3 through 6. In order to calculate a probability of failure for a joint, however, it is necessary to know probability distributions of loads and the yield stress of the materials.

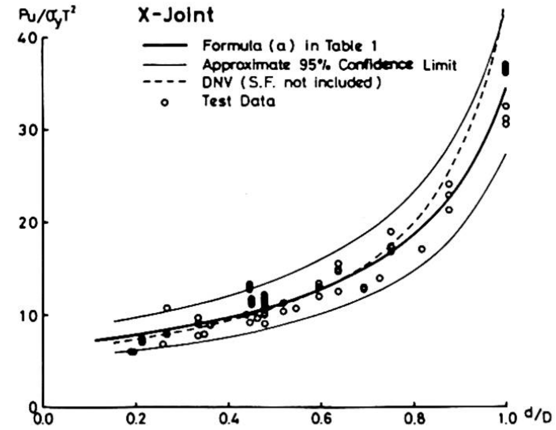


Fig. 3 Predicted Ultimate Strengths and Test Results

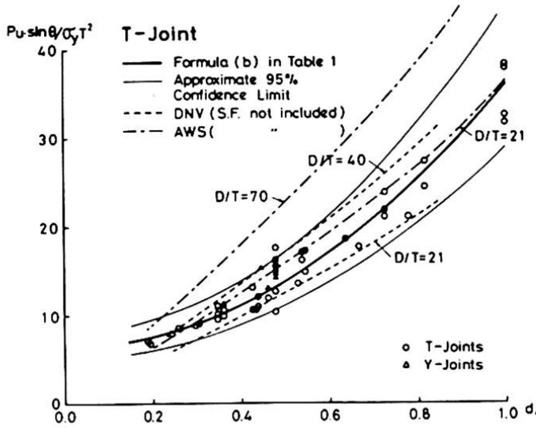


Fig. 4 Predicted Ultimate Strengths and Test results for T and Y-Joints

5. COMPARISON OF PROPOSED FORMULAE WITH EXISTING FORMULAE

The proposed formulae and the test results are compared with the formulae recommended in the AWS and DNV-Codes [5],[6] in Figs. 3 through 6 where factors of safety are not taken into account. The DNV-formula agrees well with Eq. a as well as with the test results for X-joints. However, both the AWS and DNV-formulae are not necessarily consistent with the formulae b and c nor with the test results for T, Y and

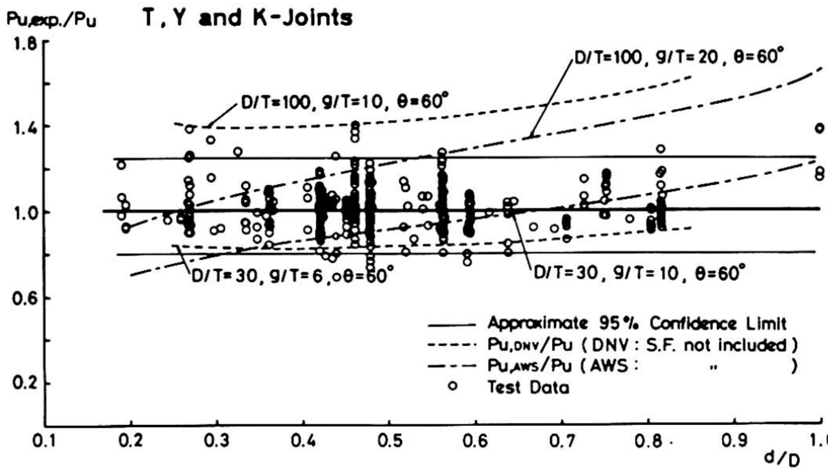


Fig. 5 Ultimate Strengths of T, Y and K-Joints: Comparisons between Formula c, AWS and DNV Formulae and Test Data

k-joints. In most cases, the AWS and DNV-formulae are risky when D/T=100 and are too conservative when g/T is less than about 8. Response of the AWS formula to a variable d/D looks to be different from what is observed in the test results for K-joints. Both the formulae are generally applicable to the T and Y-joints and the K-joints with extended braces, when D/T is less than 40.

Another comparison is made between Eq. c and the formula by Okumura et al [7] in Fig. 7. The ultimate strengths of T, Y and K-joints predicted by the Okumura's formula are scattered between the two dashed lines in the figure (when θ=60°). This formula does not overestimate the strength of these joints with large D/T ratios, but it again is too conservative for a majority of K-joints with intersecting braces.

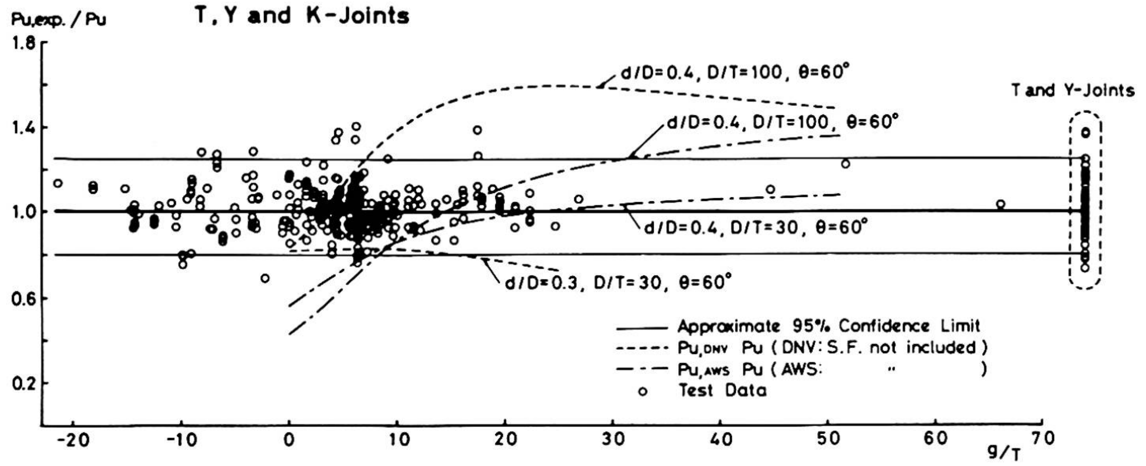


Fig. 6 Comparisons between Formula c, AWS and DNV Formulae and Test Data

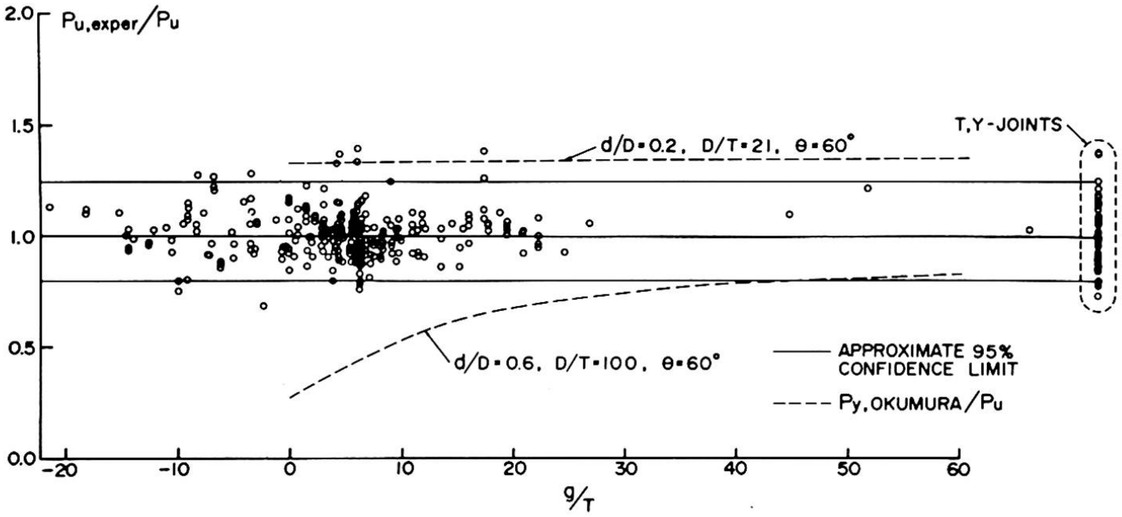
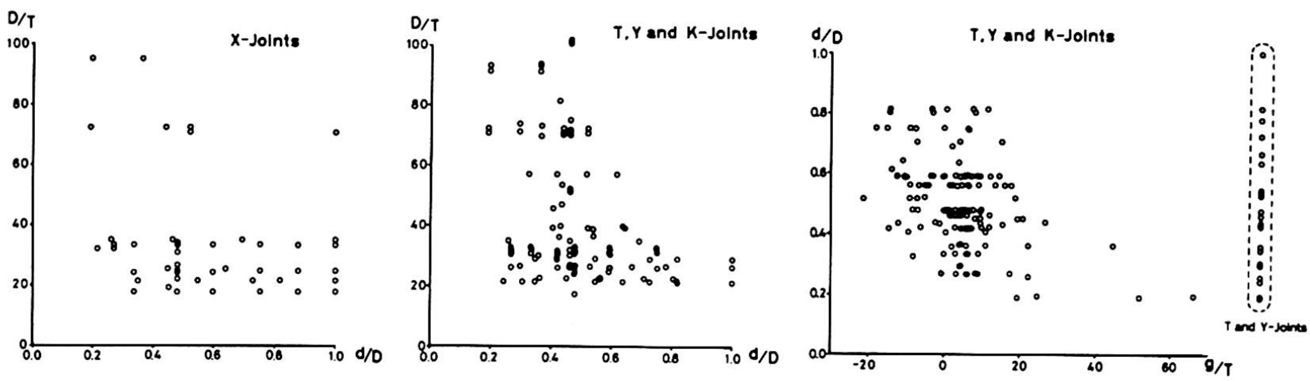


Fig. 7 Formula by Okumura et al Compared with Test Results (T, Y and K-Joints)



(from left) Figs. 8, 9, 10 Independent Variables in Test Data

6. CONCLUSIONS

The ultimate strength formulae summarized in Table 1 is applicable to the tubular X, T, Y and K-joints with a wide range of variation of each of the geometrical parameters. The formula c in the table can also applies to the K-joints with overlapping braces, which may be the first of such examples.

The test data, however, are still unavailable for some important ranges of variation of predicting variables. Examples of the areas that require further studies are:

1. The joints with a very heavy chord (D/T<20) and with a very light chord (D/T>100). The joints in these two extremities are often used in Jack-up and semi-submersible type offshore rigs, respectively.
2. The K-joint with large braces (d/D≃1).
3. Resistance of the joints under bending at the brace ends. It is to be noted that in the regression analyses an effect of secondary bending moments on the strength of the K-joint was treated merely as a factor that induces random errors.
4. The joints in high strength steels. The effects of material properties and heat treatments are still unaccountable factors that require additional work. Most of the test results were obtained through the joints made of cold formed tubes in mild steels or in low alloy medium strength steels.

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SUMMARY

This report presents the design formulae for the tubular X, T, Y and K-joints under static loads. It also discusses about the experimental grounds on which the formulae are based. The proposed formulae are compared with various existing formulae.

RESUME

Ce rapport présente les formules du calcul pour les noeuds de profilés circulaires, en forme de X, T, Y et K sous l'influence des charges statiques. Il présente aussi les bases expérimentales qui ont permis l'établissement de ces formules. Les formules proposées sont comparées avec d'autre formules existantes.

ZUSAMMENFASSUNG

Dieser Bericht enthält Bemessungsformeln für X, T, Y und K-Knoten von Hohlprofilen, unter statischen Belastungen. Die den Formeln zugrundeliegenden experimentellen Daten werden angegeben. Die Formeln werden schliesslich mit verschiedenen bereits bekannten Formeln verglichen.

Méthode de calcul à la fatigue

Methode zur Bemessung auf Ermüdung

Fatigue Design Method

F. CIOLINA
Professeur, Chef du Département Etudes
Compagnie Française d'Entreprises Métalliques
Paris, France

1 - INTRODUCTION

Deux communications du rapport préliminaire faites par nos confrères japonais donnent des résultats très intéressants sur la tenue à la fatigue des assemblages tubulaires en milieu marin.
Les développements présentés concernent des études expérimentales sur des noeuds types.
Cette communication a pour but de montrer les problèmes particuliers posés pour le dimensionnement des structures off-shore par l'utilisation des éléments précédents, à la fois sur le plan de la note de calcul et sur le plan pratique de la conception de la charpente.

2 - METHODES DE CALCUL

Le principe de l'étude à la fatigue consiste à déterminer les variations σ de contraintes élastiques en un point donné pour différents états de mer classés suivant la hauteur H des vagues mesurées de crête à creux. Il est possible de déterminer la fonction :

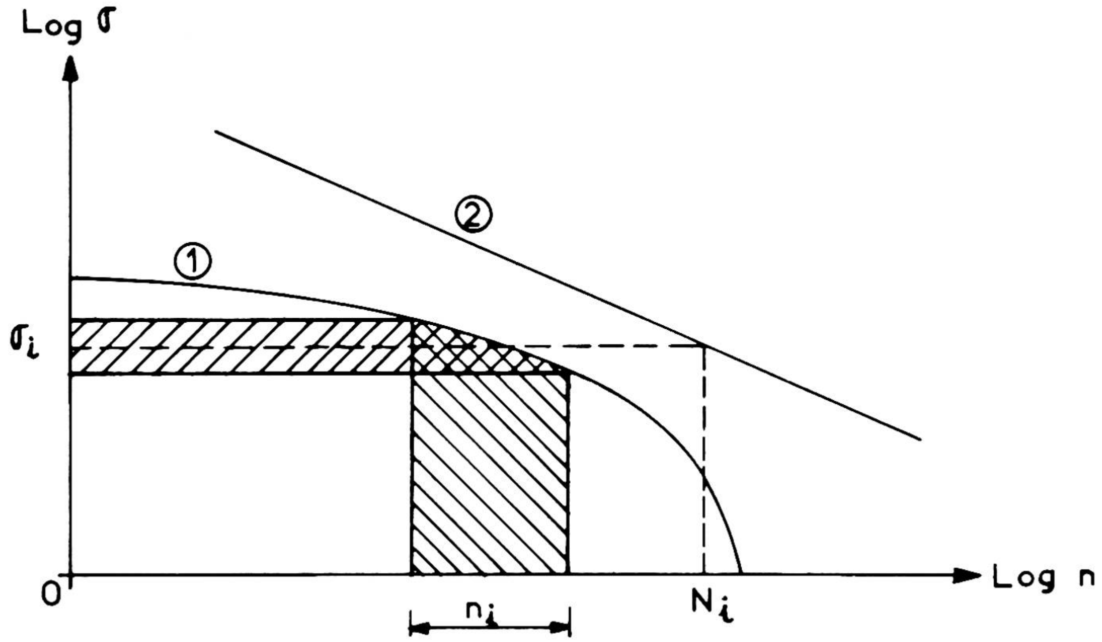
$$\sigma = f\,(N) \qquad N \text{ nombre de cycles}$$

En divisant les contraintes en q classes, de valeur moyenne σi, le critère de PALM GREEN MINER s'écrit :

$$\sum_{i=I}^{q} \frac{ni}{Ni} \leq I$$

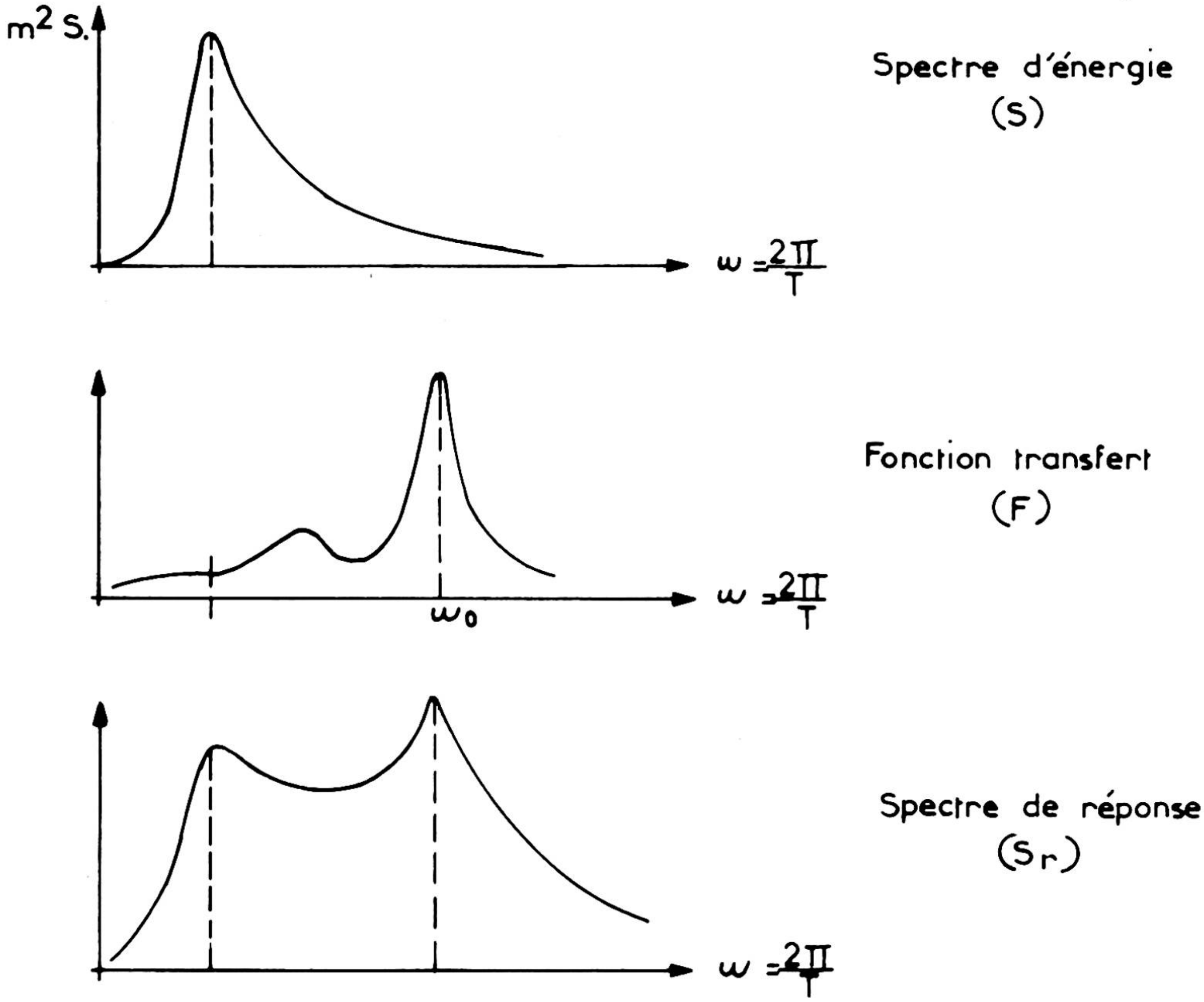
N_i correspondant au nombre de cycles maximum que peut supporter la structure pour des variations σi de contraintes.

L'utilisation du critère décrit ci-dessus pose quelques problèmes d'application pratique que nous détaillons.



① Distribution des variations de contraintes

② Courbe de fatigue



2.1 - Parmi les classes de vagues, celles correspondant aux hauteurs les plus faibles (1 à 3 mètres) contribuent pour une part très importante au dommage. En effet, la contrainte dynamique qui en résulte est très importante par rapport à l'effort statique. Ce phénomène résulte de la période qui est en général voisine du mode propre de vibration de la structure. Une telle analyse conduit à une estimation trop pessimiste du dommage.

Pour remédier à cette difficulté on peut faire une analyse spectrale des hauteurs de vagues (utilisation de spectres comme ceux de JONSAP ou de PIERSON MOSKOWITZ).
L'étude du comportement dynamique de la structure pour une direction de vague donne une fonction transfert F.
On obtient le spectre de réponse au niveau des contraintes grâce à l'équation :

$$(S_r) = (F^2)\ S$$

A chaque spectre S_i on peut associer la valeur :

$$mo_i \text{ moyenne} = \int_o^{+\infty} {}^i S_r\ (\omega)\ d\omega$$ et la probabilité P_i de dépassement de σ est égale à : $P_i = \exp. \left(- \frac{\sigma^2}{mo_i}\right)$

Il en résulte que si l'on tient compte de tous les spectres, chacun ayant une probabilité P_i, on arrive à :

$$P = \sum_i p_i\ P_i$$

Cette fonction permet la détermination de $\sigma = f\ (N)$

Une telle démarche conduit à une évaluation plus optimiste (dommage cumulé de 30 % inférieur environ).
L'effet dynamique d'amplification diminue dans la mesure où les pointes du spectre et de la fonction transfert sont très éloignées.

2.2 - Un autre problème est celui de l'analyse des différentes phases de la vie de la structure. Celle-ci peut être remorquée (voir figure 1) puis basculée et mise en position.

Deux dommages peuvent être évalués :

- celui en cours de remorquage, lié à la route maritime choisie et à l'époque de l'année (été ou hiver)
- celui pendant la vie de l'ouvrage

Le cumul de ces deux valeurs est très discutable car les schémas hydrostatiques d'étude sont très différents. Actuellement, il n'existe pas de théorie valable sur l'accumulation des dommages dans des conditions très variables de temps.

Heureusement, les zones affectées par les efforts maxima sont très différentes suivant les périodes considérées.
Dans un "Jacket" (figure 2), les efforts en cours de basculement dépendent en grande partie des zones ballastables et affectent peu les parties hautes et basses, très sensibles en phase définitive.

Figure 1 - Colonne articulée en cours de remorquage

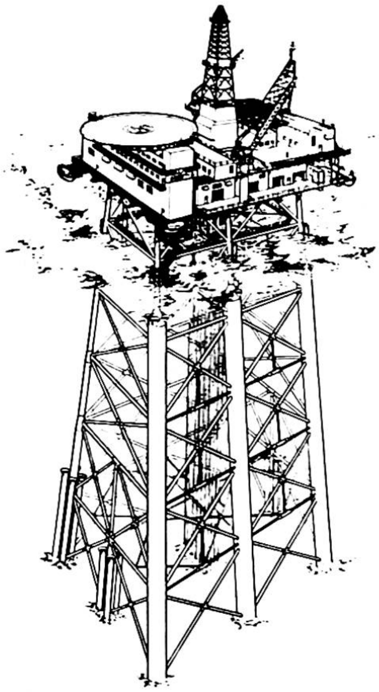


Figure 2 - Plateforme BRENT A

3 - MODIFICATIONS PERMETTANT DE REDUIRE LES DOMMAGES DUS A LA FATIGUE

Les calculs de fatigue nécessitent une bonne connaissance des conditions météorologiques et de la structure. Une telle coïncidence ne se produit qu'en fin de chantier d'assemblage. Il est donc utile de procéder à des modifications ne mettant pas en cause l'ensemble de la construction sous peine de retards inadmissibles.

Ces modifications possibles sont d'ordre très différent :

3.1 - Les efforts en cours de remorquage proviennent d'une statistique de houle liée à la période pendant laquelle se fait l'opération et au choix de l'itinéraire.

Il est possible de réduire le dommage en étudiant des itinéraires longeant les côtes avec des abris préalablement aménagés (coffres d'amarrage et signalisation, préinstallés). Un risque important réside dans les prévisions météorologiques à plus de 24 Heures en cas d'opération en pleine mer.

Une colonne articulée (voir figure 1) a pu être mise en place en Mer du nord, notamment sur le champ de BRENT grâce à une telle étude.

La position de remorquage peut constituer également un facteur important pour diminuer le dommage ; En restant dans le cas particulier des colonnes articulées, le basculement préalable en site abrité et un remorquage en position verticale, donnent une solution intéressante. Dans un tel cas, la colonne pilonne très peu même sous l'action de vagues de 3 à 4 m. Il est donc possible d'effectuer un remorquage pendant l'hiver en Mer du Nord.

Rappelons qu'une telle possibilité a été utilisée pour la mise en place de la colonne articulée sur le champ de FRIGG et pour celle installée sur le champ exploité par MOBIL.

L'étude des risques de fatigue est remplacée par une reconnaissance très soignée des hauts fonds sur l'itinéraire de remorquage.

3.2 - Les structures marines off-shore peuvent appartenir :

- soit à des ensembles composés de tubes (voir figure 3)
- soit à des ensembles composés de poutres en caisson (Pont de plate-forme fixe - voir figure 4)

Dans le premier cas, les risques de fissuration de fatigue sont concentrés dans les régions voisines des noeuds. Pour réduire les pointes de contraintes génératrices de fatigue, il est possible de mettre en place des diaphragmes internes lorsque le diamètre des pièces le permet. Ces diaphragmes doivent avoir des tracés très soignés pour obtenir une bonne égalisation des efforts.

L'étude peut être faite grâce à une modélisation convenable du noeud et à l'emploi d'un programme aux éléments finis. Les dissymétries du noeud et des barres adjacentes ainsi que les cas de charges à envisager, limitent les analyses. Bien souvent, les diagrammes obtenus permettent des conclusions qualitatives et non quantitatives.

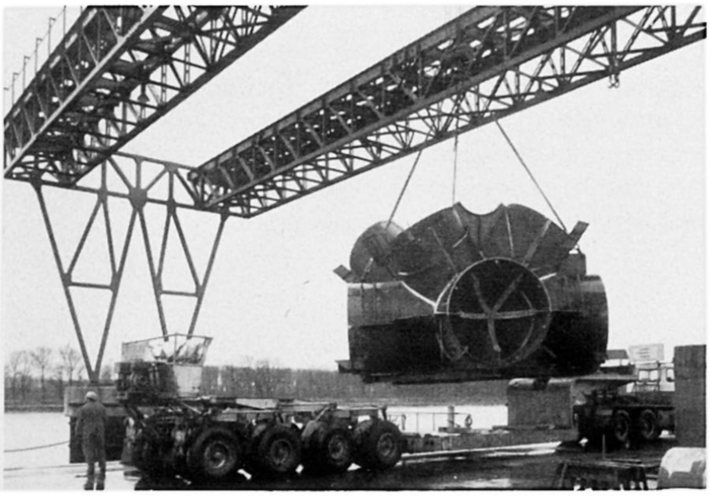


figure 3 - Noeud de plate-forme Penta 87

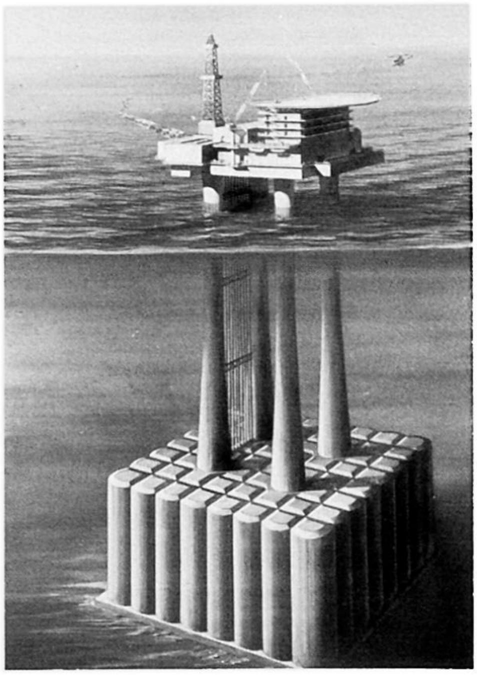


figure 4 - Cormorant Pont en acier

Pour des barres de petit diamètre (Ø < 1 mètre), les aménagements sont très limités si le noeud est complètement réalisé. Dans le cas où les barres composées de tubes sont libres aux extrêmités, on peut limiter les risques d'arrachement grâce à des recouvrements (over lapping). Sinon, la solution "gousset" ou "anneau de renfort" peut réduire légèrement les efforts longitudinaux, mais n'apportent pas de réduction à la contrainte d'arrachement (hot spot stress).

Dans le deuxième cas (poutres caissons), les efforts de fatigue sont sensibles au droit des ouvertures (diaphragmes ou passages de canalisation au droit des âmes de caissons). Celles-ci sont à border par des goussets suffisants.

La complexité des efforts transmis, nécessite une étude très poussée en éléments finis et il convient d'éviter des sections horizontales de tôles trop sollicitées dans les calculs de prédimensionnement. Enfin, les efforts dynamiques dus à la houle peuvent entraîner des renforts localisés sous les supports des modules.

RESUME

Le comportement en fatigue des structures marines nécessite une étude difficile qui doit tenir compte des spectres de houle et du comportement dynamique du système. Les risques de dommage à prendre en compte en service ou en phase de remorquage peuvent être diminués grâce à un certain nombre de dispositions constructives ou d'aménagements des itinéraires de remorquage.

ZUSAMMENFASSUNG

Das Ermüdungsverhalten von Bauwerken im Meer erfordert aufwendige Untersuchungen, welche das Wellenspektrum und das dynamische Verhalten des Systems berücksichtigen müssen. Die in Betracht zu ziehenden Risiken einer Beschädigung im Betrieb und beim Abschleppen können dank einer Anzahl von baulichen Massnahmen oder durch sachgemässe Auswahl der Schleppwege vermindert werden.

SUMMARY

Fatigue behaviour of off-shore constructions requires complex investigations which must allow for both swell spectra and overall dynamic behaviour. The damage risks to be reckoned with under service conditions and during towing can be reduced by resorting to a number of engineering solutions or by determining suitable tow routes.

A new Kind of Hybrid Construction

Une construction hybride nouvelle

Eine neue hybride Konstruktion

FRODE JOHAN HANSEN
M.Sc., F.I.C.E., Consultant
Per Hall Consultants Ltd., Hong Kong
Hong Kong

1. Introduction

In the earlier stages of the development of deep water oil production platforms for the North Sea the choice was oversimplified to one of using a pile supported steel jacket or a concrete gravity structure. Recent designs have combined both materials and are often referred to as "hybrids". (Fig. 1). In this paper the arguments for a new type of hybrid are explored and the resulting design (Tricon) briefly described. (Fig. 2).

2. The Problems and Some Solutions

An offshore platform structure in deep and exposed waters is a special case of a tall building with almost its entire payload concentrated in the "penthouse", subject to exceptionally large horizontal loadings of a cyclic nature and with uniquely difficult site access and foundation conditions. However, having accepted these rather special design and construction parameters, the design of such a structure should still be tackled and ultimately make as much sense as that of any other structure. In other words, the design concept should be governed by the "payload" and the permanent working conditions, be reasonably easy to construct and in the process possibly permitting some temporary overstressing.

The present generation of self-floating "jacket" structures does make sense in that respect, albeit with some obvious shortcomings.

The tubular joints, the node points, have become so large that the secondary stresses have reached a magnitude of prime importance, requiring a great amount of internal stiffening. They are however still not large enough to permit more than one or two welders to work inside the joint. Furthermore, the number of node points in a major structure is still so large that the prefabrication of these node points is a very critical element in the construction programme.

A tubular space frame with a minimum number of members and hence node points leads to a much more satisfactory structural concept. The tubular members become so big that they achieve a natural buoyancy even if made of reinforced or prestressed concrete, and the node point become so spacious, that a considerable work force can get inside them They also become so accessible, that the internal stiffening members can be designed and positioned so correctly that secondary stresses can be reduced to what they ought to be; i.e. of secondary importance.

Another serious shortcoming of the larger pile supported jacket structures is the great proportion of weather and crane barge dependant offshore work. This proportion will ultimately, with increasing water depths and wave heights and narrowing weather windows, reach a "point of no return", and in the North Sea it lead to the introduction of the "gravity" structures.

The present generation of concrete, steel or hybrid gravity structures has solved the offshore installation problem by avoiding it - but at a very heavy premium. For their construction they require very special facilities - such as extremely deep and sheltered water - and their design leave no room for the designer to produce an optimum solution to the prime purpose of the structure; i.e. to support the payload safely and economically above the waves. It is a fact, that the optimum solution for the permanent conditions is incompatible with the temporary conditions, and that the structure is governed by the requirements for floating stability and structural strength during the towing and sinking operation. Having met these temporary conditions as well as those imposed by the restrictions of the available construction site, the designer is then left with the rather unsatisfying task of just checking, whether the structure is adequate for the permanent conditions. Hardly the way to produce the most economic structure, and at times, when the upper soil strata of the seabed are too weak, not even a way to produce just an acceptable structure.

It is in the approach to the foundation problem that both types of existing offshore structures show their worst limitations.

The pile supported structures suffer not only because the piles are difficult and costly to drive, but also because they are not the best and most economic foundation method. Loads of the same order of magnitude are often supported under water; bridge piers in connection with river or harbour crossings. In such cases one would generally dig down to a suitable foundation level and one would never contemplate driving piles to carrying capacities of several thousand tons a piece.

Similarly the gravity structures suffer not only because they become disproportionately large and impose high stresses on the upper and weaker soil strata, but also because they impose an impossible tas on the designers. It is impossible to tailor make a foundation to suit an unprepared and practically unknown seabed or to determine the right length and strength of a penetrating skirt without exact information on the soil into which the skirt has to penetrate. Still, this is the very first design decision, which has to be made.

From the above considerations it would appear, that deep footings (assuming it is feasible to install them) are cheaper than long steel piles and also a more rational foundation method than resting directly on top of the seabed.

They are independant of the seabed topography and they eliminate the risk of sliding and the risk associated with scour of the seabed. They can furthermore be designed properly, taking full account of the prevailing soil properties, if the construction of the footings can be reversed to become the last item and not the first on the construction programme.

3. The Design

The above design considerations have resulted in the Tricon concept. Tricon consists of three basic elements: (i) on the top a short legged jack-up platform designed to the clients requirements, (ii) a main three legged space frame optimised to carry the payload and resist wave action, made up of large tubular members with few node points, and (iii) caisson footings sunk to a firm foundation.

The tubular space frame is constructed horizontally as a normal jacket structure. All the tubular members have considerable natural buoyancy and the structure as a whole floats in a very shallow water. The footings are circular hollow cylinders suitably stiffened to be able to spread the column load over the required foundation area. They can be made of steel, but even made of concrete will they possess enough buoyancy to float in shallow water.

Since the structure is being constructed horizontally, (Fig. 3) the footings can be added as the last item on the programme, and even with the diameter fixed the height can be increased until the last minute.

The structure is obviously extremely stable floating horizontally, but because of the positive buoyancy of all the tubular members it remains stable during all phases of the upending process which is achieved by controlled flooding.

Having touched down with the three footings resting on top of the seabed the tops of the columns are well above the waters surface. The caissons are now sunk into the seabed by means of excavation inside the footings. The excavation process is a self-contained and

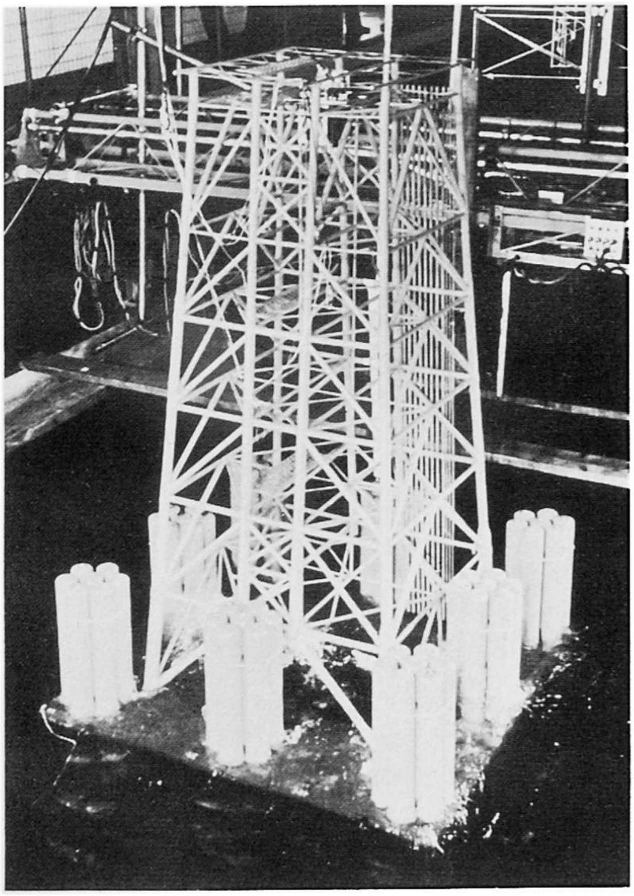


Fig. 1. RDL (NORTH SEA) HYBRID

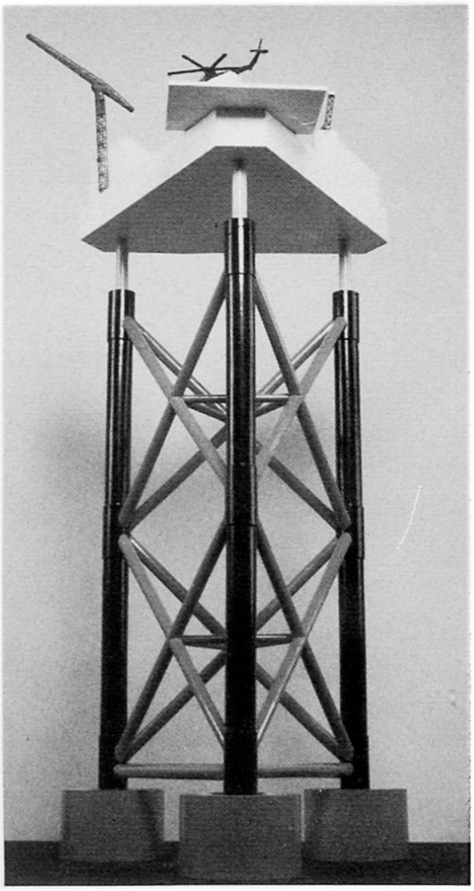
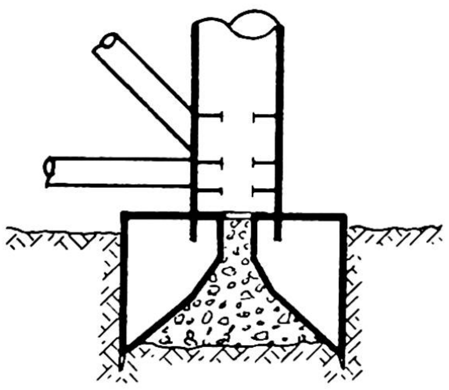


Fig 2. TRICON

Fig 3. TRICON IS FABRICATED & FLOATED HORIZONTALLY

practically weather independant operation. The necessary tools are built into the structure and there is plenty of room in the top of the columns to accommodate the working crew.

The excavation takes place under water with purpose made digging or tunnelling tools dragging the excavated material from the perimeter of the footings to the centre, where it can be removed by pumping or air lifting.



Caisson Foundation

As soon as the cutting edge of the footings has penetrated into the seabed and sealed off the interior of the footings from the outside water pressure, the structure gains instant "fixity" and it is possible to reduce the internal pressure and thus utilising the external hydrostatic pressure to assist the penetration into the seabed. In that respect the sinking of the Tricon feet is easier and more controllable than a caisson sinking in compressed air or even an open excavation for a bridge pier. Compared to the piledriving, where each hammer blow must exert a force of several thousand tons to achieve penetration, the excavation process is obviously much simpler, requiring a force of only a few tons to break down the soil into removable lumps and thus achieve penetration into the seabed.

The excavation process is continued until the tops of the columns are at a predetermined depth below the waters surface, access to the working space in the columns being gained through removable access shafts. With the footings at the right depth the space underneath the footings is backfilled with granular material and the Tricon foundation is completed.

The temporary access shafts are then removed and the underwater foundation is ready to receive the "jack-up" superstructure.

To land the "jack-up" legs on the Tricon columns requires a spell of calm weather, but only a very short spell, and no calmer than what is required to carry out one lift with a crane barge; i.e. equivalent to bringing one pile section into the pile guides or placing the hammer on top of one pile.

If required the whole process can be put in reverse for the removal and movement to a similar site. Excess hydraulic pressure can be utilised to force the raising of the caissons in a controlled manner.

If, as intended, the Tricon concept combines the structural economy of a "jacket" with the offshore installation economy of a gravity structure, it should logically lead to cheaper offshore structures.

SUMMARY

The new type of hybrid construction "TRICON" divides naturally into three distinct main sections:

the top - a conventional three-legged "jack-up" platform

the tower - a triangular tubular space frame with minimum number of members

the foundation - three deep caisson footings sunk into the seabed to a suitable foundation level.

It combines the best structural features of a "jacket" and the offshore installation advantages of "jack-ups" and "gravity" structures.

RESUME

La nouvelle construction hybride "TRICON" se divise en trois parties distinctes:

le sommet - une plate-forme conventionnelle à trois pieds sur vérin

la tour - un treillis tubulaire triangulaire avec un minimum de membrures

la fondation - trois profonds caissons de pied ancrés à un niveau de fondation convenable au fond de la mer.

Ceci combine les meilleures caractéristiques de chaque élément et les avantages d'une plate-forme auto-élévatrice et à embase.

ZUSAMMENFASSUNG

Die neue hybride Konstruktion "TRICON" lässt sich in drei Hauptabschnitte unterteilen:

oben befindet sich die übliche Plattform, die an drei Beinen auf und ab bewegt werden kann

der Turm ist eine fachwerkartige Rohrrahmenkonstruktion mit einem Minimum an Stäben

die Gründung erfolgt über drei Tiefcaissons, welche in den Seegrund bis zur tragfähigen Bodenschicht abgesenkt werden.

Damit werden gleichzeitig die Vorteile einer Hubinsel und einer Schwergewichts-Plattform realisiert.

IVa

Synthesis and Conclusions

Synthèse et conclusions

Synthese und Schlussfolgerungen

J.G. BOUWKAMP
President
Offshore Development Engineering, Inc.
Berkeley, California U.S.A.

Offshore Structures

Mr. Chairman, Ladies and Gentlemen,

We have just heard some additional highly interesting discussions on the subject of offshore structures which further emphasized the complexity of operating in offshore regions.

We would make a serious mistake if we would consider this field as an exclusively structural engineering territory. There is no question that the structural engineering input is of vital importance in developing an ultimately satisfactory structure. However, offshore engineering design requires the input of the oceanographer, who defines the sea state; the ocean engineer, who translates the ocean environment into hydrodynamic loads; the naval architect, who considers the stability and platform motions of both floating drilling and production systems and offshore towers during the towing and upending phases; the geotechnical engineer, who provides the necessary data to develop the foundation design; and the fabricator or contractor, who ultimately creates the system.

Each of these areas represents a challenge to the expert:

Variability and Environmental Influence on Wave Conditions

Most design wave spectra are based on open sea observations or at least, observations taken in areas of relatively open water. Since sea conditions can be strongly affected by bottom and shore topography, some attention should be given to ascertaining how such factors may modify wave conditions in a specific design condition, and to sensitivity studies which are aimed at finding the effect on a structure of anticipated variations about the design wave conditions.

Interaction of Wind Velocity Profile and Waves Impinging on a Structure

If a structure is exposed to storm waves of height similar to the dimensions of the above-water portion of the structure, the wind velocity profile and wind force acting on the structure vary with time. This wave influence on the design wind characteristics will be superimposed on the natural gustiness of the storm wind environment and undoubtedly have an influence on both the mean and time dependent wind forces.

Correlation and Proper Simultaneous Treatment of Waves and Current

Important questions here involve the appropriate way of representing the combined flow field and the computation on bodies exposed simultaneously to waves and currents.

Hydrodynamic Loadings

Hydrodynamic loads, acting on the emerged structural members, are a result of the fluid motion associated with waves and current, and of the body motion induced in response to these forces, as well as earthquake and other external excitation. The combined structure-fluid motion effects are referred to by the terms "fluid-structure interaction" or "relative motion effects". The prediction of these fluid forces requires the solution of the complete fluid equations of motion (Navier-Stokes equations) with complicated boundary conditions, including the free surface, moving rigid boundaries of the structure, and the fixed boundaries, e.g. the ocean bottom. Complete solutions of this problem have been obtained in only a few elementary cases, and often substantial simplifications to the problem have been made. In the case of bodies whose dimensions are of the same order as the wave lengths, viscous forces are greatly exceeded by inertia forces, and may therefore be neglected in determining both the flow kinematics and the forces on the body. For bodies of small cross-section compared to the wave length, the pressure, velocity and acceleration field associated with wave motion is assumed unaffected by the presence of the body. The viscous drag forces and inertia or pressure forces are computed, using these kinematic quantities combined with imperical drag and inertia coefficients; the so-called Morrison formula. However, there is a range of body sizes in which both diffraction and viscous effects are important. It is in this area that a thorough understanding of the relative importance of these effects is essential. Furthermore, members which pierce the mean water surface are subjected to some other forces. These are the time-varying buoyancy and impact or slamming forces. The former is determined by geometric consideration, but the latter is not well understood at present. The wave impact force on above water structural members is similar to the slam force on a ship pitching in heavy seas or the impact of a seaplane when landing. However, it is necessary to recognize the differences between the phenoma which apply on the one hand in the case of a moving body contacting a moving wave surface, and on the other hand, a moving wave impinging on an essentially fixed body.

Wind Loading

Wind action on offshore platforms can contribute significantly to design loads. To evaluate the magnitude of wind loads, it is necessary to first predict the nature of the wind produced at the site, in terms of maximum mean wind velocity and the fluctuation of the wind velocity about this maximum mean value, i.e. gust characteristics, and second to transform these velocities into drag forces. Thi transformation is difficult to achieve realistically due to the very irregular geometry of the platform and its equipment. Conceptually, the wind force transfer function can be characterized as depending on the altitude and air density, time velocity direction characteristics shape and effective area, motion response characteristics of platform, projected area, and variation of the force with velocity.

Ice Loadings

Ice loading on offshore platforms in the Arctic seas can be extremely severe. Ice islands and icebergs, which are of land origin, can easily destroy a platform. Fortunately this danger is limited since these are rare; therefore, no consideration of their effects is given in the design of offshore platforms. On the other hand, sea ice is frequently present and must be considered in the design. This ice, which forms in sheets, can be pushed into ridges, rafts, and hummocks, producing unconsolidated masses which later consolidate upon refreezing. The depth of this ice is highly variable, causing similar variations in the magnitude of force exerted on offshore platforms. This maximum force is limited by failure conditions at the edge of the ice mass; therefore, its magnitude depends on (1) the contact area, as determined by the thickness of the ice sheet and the maximum width of indentation caused by the structure during impact, (2) the aspect ratio of ice thickness to width of structure, (3) the shape of the structure, (4) the strength of the ice in unconfined compression, (5) the elastic properties of the ice, and (6) the conditions of ice as unfrozen or frozen, to the face of the structure.

Soil Loadings

Soil loadings can result from lateral and vertical loadings on foundation elements and develop rapidly, as a result of soil deformations in which the instantaneous and/or accumulative relative deformations are large. Such deformations could be developed from bottom soil movements induced by wave action, earthquakes and other similar flow slides or combination of the same. In addition, settlements, both laterally and vertically can occur during long periods, and may affect the foundation stability over the long time.

Earthquake Loadings

The earthquake loading is similar to the other principle loads acting on offshore structures in that the intensity of loading is influenced strongly by basic characteristics of the structure. However, in the case of the earthquake loading, the input forces depend on the mass and stiffness of the structure rather than on the exterior form, and by adjusting these parameters, particularly the stiffness, the designer can exert a great influence over the seismic loads a structure must resist. This fact is particularly significant in regard to the non-linear behavior of an offshore structure, because damages which occur in the structural system generally lead to reductions of stiffness which allow relaxation of the seismic input intensity. Thus, to a significant extent, earthquake damages tend to be self-limiting processes and the primary concern of the designer is to insure that the vertical load capacity of the structure is maintained as the earthquake damages occur.

Structural Modeling

With the loads defined, a mathematical model of the structural system must be established to permit the three-dimensional analysis of the internal forces and stresses. In the event of high overloads, such as produced by high magnitude earthquakes, this model must properly reflect the inelastic deformations produced. Further, in the interest of providing a suitable working environment, motion of the deck must be predicted realistically. Only through refined and advanced analytical procedures and modeling techniques is it possible to accurately predict the structural response.

Foundation Modeling

To realistically model fixed platforms for three-dimensional analysis purposes, it is necessary that the foundation be properly modeled so that the structure-foundation and direction effects will be included in the results. As in the case of structural modeling, the techniques and methodologies used for this purpose vary considerably in practice. Basically there are three basic forms of foundation modeling for spread footings or mat foundations, namely: (1) constant discrete springs and dashpots for the soil foundation with the spring constants determined from elastic-static half-space theory and the dashpot coefficients having assigned values to represent material damping in the soil, (2) discrete springs and dashpots for the soil foundation with the spring constants and dashpot coefficients being frequency dependent in accordance with the elastic or viscoelastic dynamic half-space theory, and (3) finite element representations of a body of soil at the base of the structure. Foundation modeling for fixed towers supported on piles is most difficult to accomplish realistically. The techniques and methodologies used for this purpose range widely in practice, the simplest form being that of assuming full fixity for the piles at some specified effective depth in calculating lateral stiffness and assuming a distribution of the vertical load transfer from the pile to the soil in calculating vertical and rotational stiffnesses. For firm foundation conditions, the forces applied to the pile cap will be rapidly transferred to the soil with increasing depth into the foundation. In this case, pile cap impedance functions can be approximated using the impedance functions generated for a rigid massless circular disc supported on an elastic or viscoelastic half-space. For softer foundation conditions, the forces applied to the pile cap are transferred to the soil at deeper depths; thus, a more realistic representation of the load transfer from piles to soil is needed. This problem is a topic of basic research at the present time which should yield suitable pile foundation models in the near future.

Analysis Procedures

Dynamic analyses of linear fixed towers can be carried out in either the time domain or the frequency domain. Most analyses of this type, carried out in the past, have used the time domain approach. Recently, however, the frequency domain approach has become comparative through the development of Fast Fourier Transform techniques. If frequency dependent parameters are included in the overall fluid-structure-foundation system (e.g. frequency dependent hydrodynamic drag and inertia coeffecients and/or frequency dependent foundation impedance functions), a time domain solution is not possible, but the frequency domain approach can be used without difficulty.

Performance Criteria

In addition to general loading methodologies and structural and foundation modeling techniques, other criteria related to the detailed structural response (e.g. fatigue, non-linear behavior, failure) are equally essential in the design process. Based on his experience in fields other than offshore engineering, the structural engineer is particularly qualified to establish detailed prototype response criteria for both elements, connections and material properties. Furthermore, the overall design may not only be governed by the ultimate performance aspect, but also by the behavior of the structure during the different phases of construction, tow-out and installation. In the past, these latter phases have been considered as structurally less significant than the actual design phase. However, present offshore developments require structural systems for water depths of 800 to 1,000 feet or more. The remoteness of these regions may require tows covering several thousands of miles between assembly yard and installation site. Also, upending processes at the installation site necessitate a careful assessment of the dynamic response during this stage. It is now essential not only to consider the platform stability, but also the dynamic structural response during these tow-out and upending phases. This aspect becomes particularly significant if one considers the duration of these tows under potentially most adverse weather conditions and the often hostile prevailing sea state at the site. Damages during this phase, be it permanent or of fatigue nature, are factors which need full consideration in formulating the final design.

The previous comments emphasize that offshore structural design reflects the combined effort of many engineering disciplines. To achieve an optimum design for both stationary and floating structures in a mostly hostile, highly random and often insufficiently known environment, requires an integrated approach involving the most advanced technologies.

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

YORIHIKO OHSAKI
Professor
University of Tokyo
Tokyo, Japan

Foundation Structures for Tall Buildings

This presentation in Working Session IV concerns the foundation structures for tall buildings. In the writer's Introductory Report (Ohsaki 1976), he has pointed out a number of problems associated with design of foundations for tall buildings.

The first problem is the heavy weight of a tall building which requires large bearing capacity of supporting soil stratum. Bearing capacities of sand stratum, gravel stratum, or heavily over-consolidated clayey stratum are usually in the range of 150 to 400 tons/sq.meter, as shown in Fig.1, allowing the

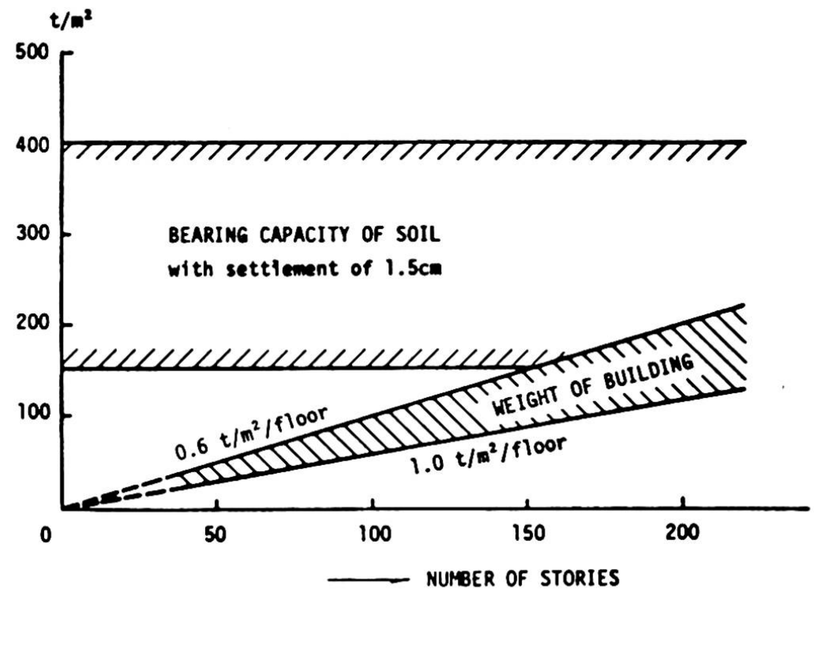

Fig.1

settlement of approximately 1.5 centimeters. On the other hand, unit weights of tall buildings of typical, steel construction are estimated to be in the range of 0.6 to 1.0 tons/sq.meter/floor. Therefore, the total weights of buildings are in the triangular range in Fig.1, which shows that there is no serious problem in foundation design, if soil stratum of such strength can be encountered within the depth of a building basement.

If such stratum is located at a certain depth below the base of a building, a pile foundation can be used and, fortunately, no fundamental problem has been met so far in performance of pile foundation. The application of floating foundation may also be possible, which balances the weight of building and the weight of excavated soil so that no additional load would be applied to the supporting soil stratum. It is possible theoretically up to the building of approximately 40 stories; however, it seems that there has been no experience of applying the idea of floating foundation actually to high-rise buildings.

The second problem is the effects of wind force. As shown in Fig.2, when a building is subjected to wind force, the overturning moment causes an increase

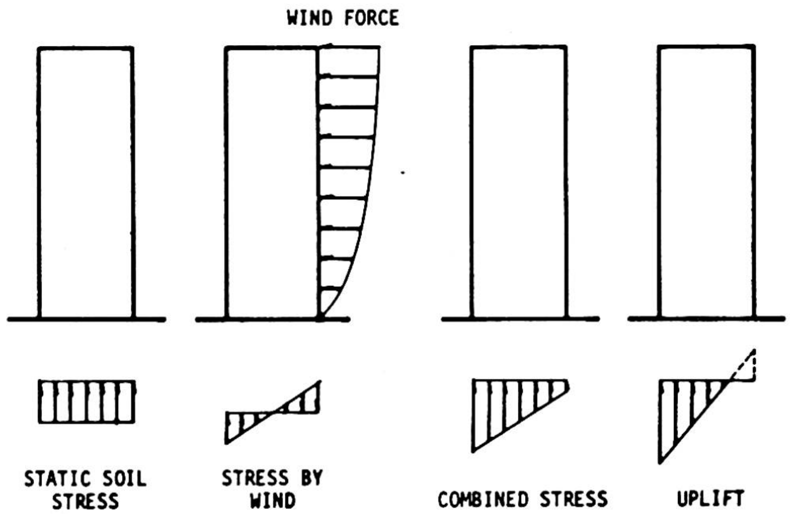

Fig.2

of soil reaction stress on one side and a decrease on the other side. Usually, as the combined stress is within twice the static stress, if Fig.1 is again considered, there would be no danger of failure of supporting soil. If overturning moment increases still more, uplift takes place as also shown in Fig.2, which indicates a tendency to make the structure unstable. However, the uplift can readily be prevented by providing setback as shown in Fig.3 and, when the setback can not be provided, anchoring shown in Fig.3 may be an effective way, but it has not been actually utilized for tall buildings up to the present time.

In a foundation slab, the outer part shaded in Fig.4 is most effective to resist the overturning moment, while the middle part is not so effective mechanically. Moreover, if permanent settlements are produced beneath the edge of a foundation slab under repeated wind loadings, the presence of the middle part is rather harmful causing the so-called riding effect, which will accelerate the rocking motion of the superstructure. In consideration of these and other facts, Schlaich and Otto suggest an idea of ring foundation for a tower structure, and describe in detail many examples of their design experiences of actual tower structures in their Preliminary Report (Schlaich & Otto 1976).

The last problem is the effects of seismic forces. In Japan of extremely high seismicity, seismic forces are usually larger than wind forces for typical buildings of less than approximately 50 stories. However, as a general shape of acceleration response spectrum in Fig.5 indicates, the earthquake input to tall and flexible buildings is rather small when compared with low and stiff

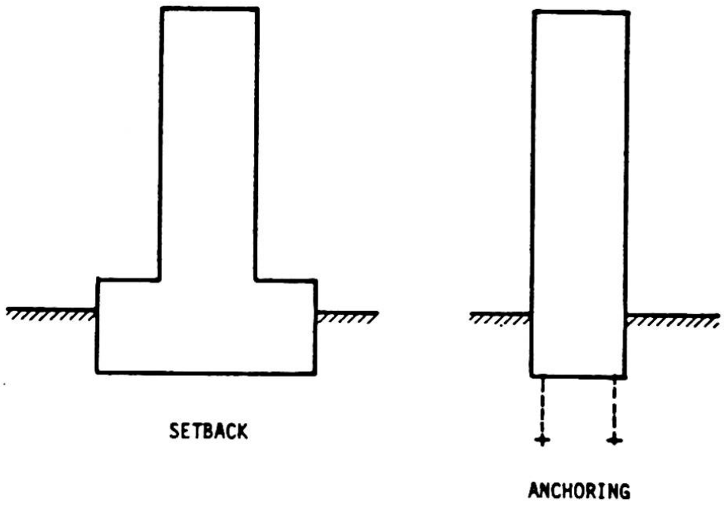

Fig.3

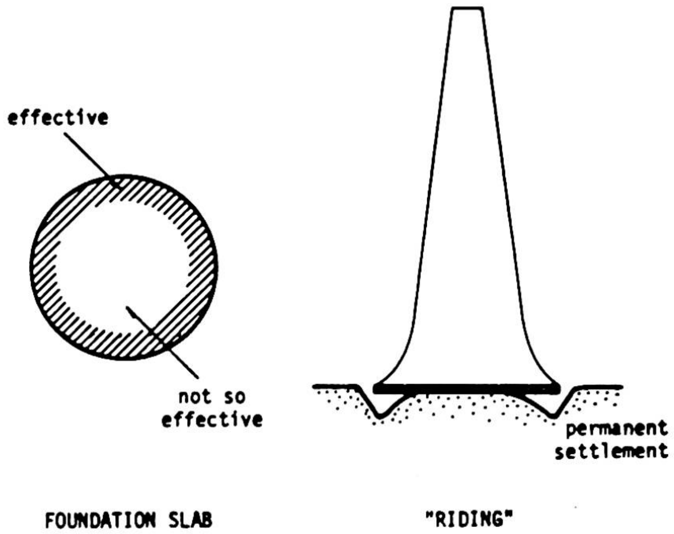

Fig.4

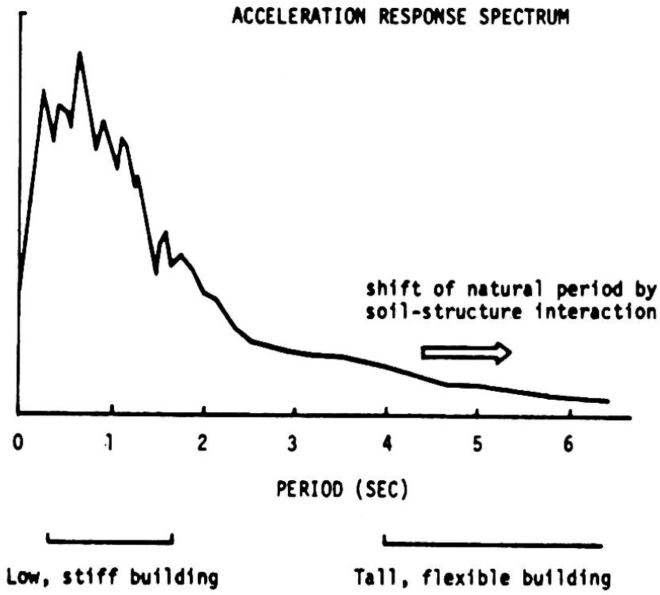

Fig.5

buildings. Moreover, the effect of soil-structure interaction tends to still reduce the input force, although the interaction effect is not important in general for flexible buildings.

In conclusion, there may be several problems in foundation design specific to tall buildings, they will not be of great difficulty to overcome unless soil conditions are extraordinarily unfavorable.

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Schlaich, J. & U. Otto (1976) : Zur Gründung hoher Stahlbetontürme, Preliminary Report, 10th Congress, Tokyo, International Association for Bridge and Structural Engineering, pp.349-355

IVb

Foundation Structure of the CN Tower (Toronto)

Fondation de la tour de télécommunication "CN Tower" (Toronto)

Fundation des Fernmeldeturms "CN Tower" (Toronto)

BRUNO THÜRLIMANN
Professor of Structural Engineering
Swiss Federal Institute of Technology
Zurich, Switzerland

1. Description of Structure

The recently completed CN Tower in Toronto (Canada) is the highest free standing structure of the world rising to a height of 550 m (Fig. 1). It consists of a tapered, Y-shaped, multi-cellular concrete shaft up to 447 m and a steel antenna mast of 103 m covered by fiber reinforced plastic cylinders (Fig. 2).

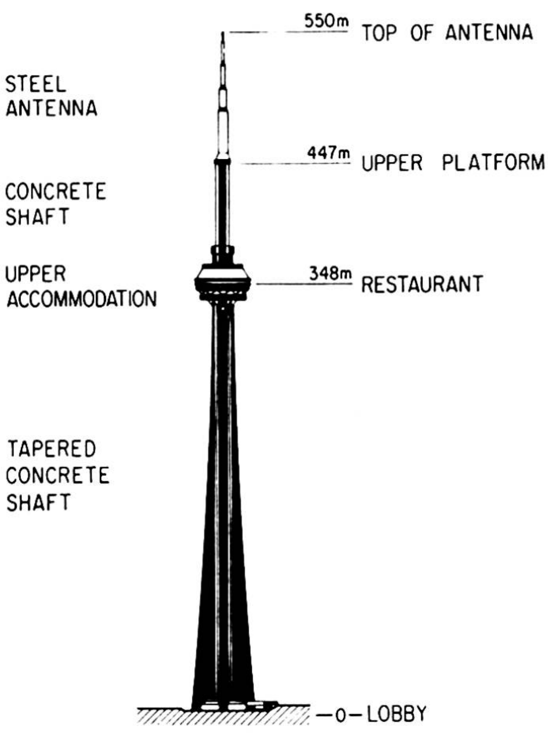


Fig.1: CN Tower, Toronto

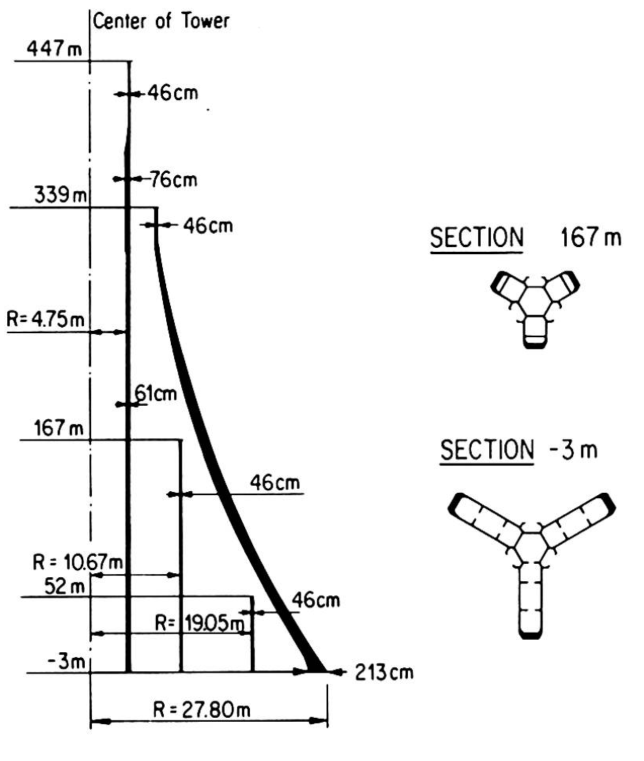


Fig.2: Typical Dimensions of Tower Shaft

The tower houses various telecommunication transmission facilities. In addition a restaurant and observation decks at the 348 m level and an upper observation platform at the 447 m level are open to the general public.

The selection of the shape and the cross section of the concrete shaft was influenced not only by statical and architectural considerations. Due to the fact that only three walls, i.e. the outer walls of the three legs are sloping the application of slip forms in casting the entire shaft became possible. Vertical prestressing of the tower resulted in a number of advantages and cost saving:

- No tensile stresses under the 50 year wind.
- About 75% less steel weight for the same strength, cost savings relative to the unit prices of prestressing and ordinary reinforcement.
- Concentrated forces, no splices, less congestions due to reinforcements.
- Great reduction in the quantity of ordinary steel to be placed during the slipping operation of the formwork.

2. Foundation Base

The foundation base consists of the Meaford-Dundas shale formation which is typical for the downtown area of Toronto. Some twenty borings and four observation test wells of 76 cm diameter were made, one reaching a depth of 40 m. Under an unconsolidated fill of about 9 m the bedrock comprises a sedimentary sequence of horizontally thinly bedded or laminated shales, calcareous shales and limestones. Some narrow zones of rubbled shale, 15 to 45 cm thick occur. Some clay seams up to 1.5 cm thick are also present. These seams together with the presence of vertical cracks in the shales lead to the use of prestressing of the foundation structure as will be explained later on. The ground water level is at a depth of about 6 m.

In the following Table 1 the unconfined compression strength of the rock samples is listed.

	Shale	Calcareous Shale	Limestone
Amount of Sequence Unconfined Compression Strength	84 kg/cm^2	190 kg/cm^2	1100 to 2000 kg/cm^2

Table 1: Compression Strength of Rock Base

3. Foundation Structure

The foundation structure is shown in Fig. 3. It follows the Y-shape of the tower shaft. The hollow core slab has a thickness of 5.50 m. The live anchors of the prestressing cables of the tower walls are located in these caverns. The prestressing operations for these cables, up to 450 m long, could hence be executed in this protected area without interference of the work on the platform of the slip form.

The outer walls of the three tower legs are inclined with respect to the vertical as illustrated in Fig. 4. The horizontal components of the forces in these walls due to dead load and wind moment produce considerable tensile forces in the foundation slab. Resisting forces can be built up by (1) the reinforcement of the slab, (2) friction forces between slab and rock and (3) resistance of the rock wall around the perimeter.

It was decided to take up these permantly acting spreading forces by posttensioned cables, Fig. 5. Frictional forces (2) could not be relied upon because of the above mentioned clay seams and high pore water pressures in response to dynamic wind forces. Resisting forces (3) along the perimeter would only be activated after substantial horizontal movements. Hence, in order to prevent vertical cracking of the foundation slab and the tower walls above as well as of the rock layers below prestressing was chosen.

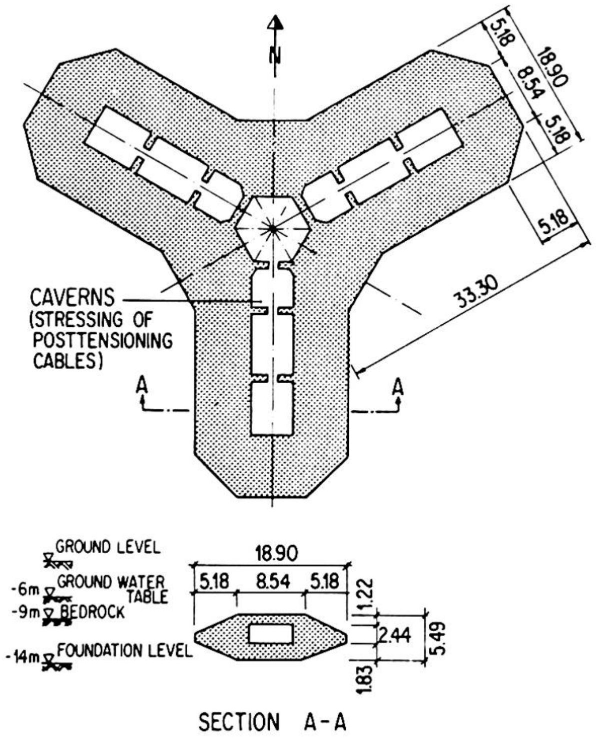


Fig.3: Foundation Slab

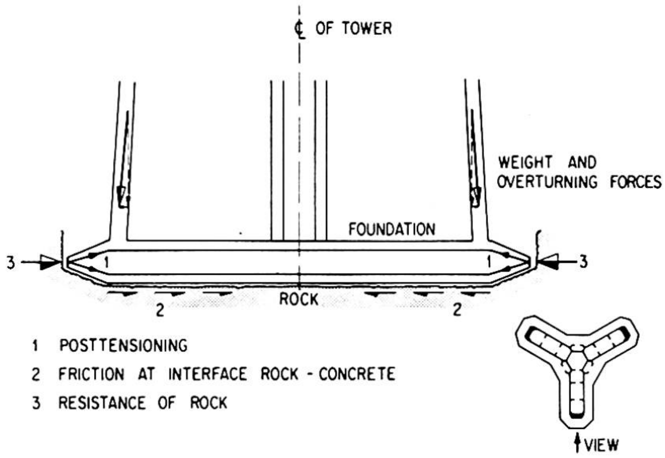


Fig.4: Horizontal Forces on Foundation

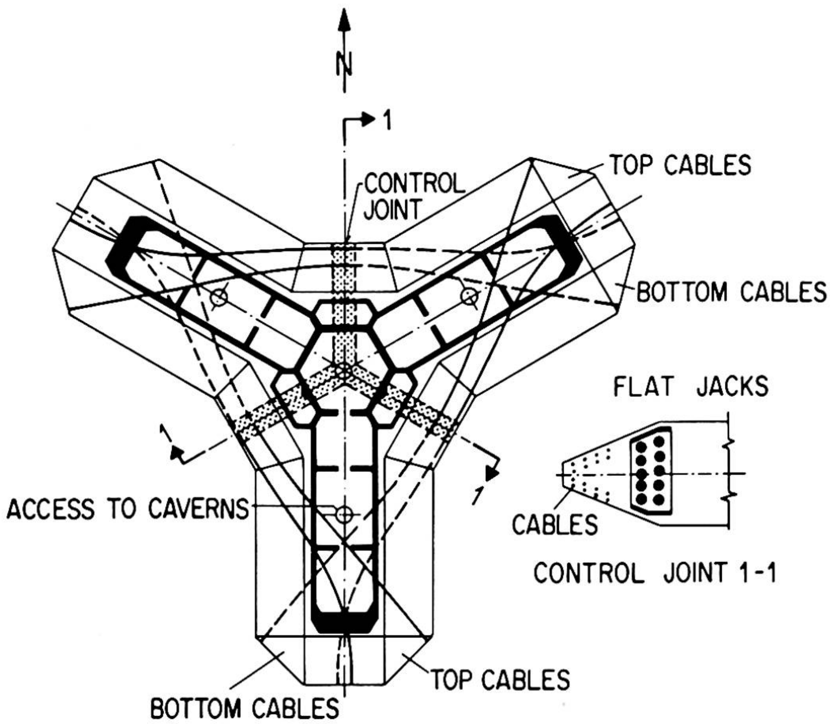


Fig.5: Prestressing of Foundation Slab

Bands of prestressing cables in the arrangement of Fig. 5 were placed running below and above the slab caverns. Control joints 1-1 separated the three legs completely. A sequence for the posttensioning operation was developed such that the outer edges of the three legs remained fixed. In this way vertical cracks in the shale at these critical points and a possible deterioration of its resistance could be avoided. By activating the active anchors at both cable ends and the flat jacks in the control joints in an appropriate sequence this condition could be met. A subsequent analysis of the jack forces showed that the rock base provided practically no frictional resistance as anticipated.

In order to reduce volume changes due to hydration heat and shrinkage a modest dosage of cement of 210 kg per 1 m^3 of concrete was used.

Tapering edges (see cross section Fig. 3) around the foundation slab were selected on the basis of a stress analysis on the interaction between slab and rock. In such a way a considerable reduction of the local shear stresses along the edges was achieved and the possibility of vertical cracking of the shale greatly reduced.

4. Summary of Design Data

The most pertinent results of the analysis of the foundation structure are as follows:

Loading

Wind: Predicted extreme mean hourly gradient wind speed, 50 years return period:

v_{50} = 167 km/h

Base Forces, 50 years period:

Moment $M_{50} = 4.42 \cdot 10^5$ m•to

Shear V_{50} = 1800 to

During the initial design period the exact dimensions and contours of the antenna mast, the upper and lower accommodation levels and other details were not yet fixed. Hence, 25% higher reference values were selected for making the design calculations. This corresponds also to about a 10% increase in the wind speed.

Reference Forces:

$M_{ref} = 1.25\ M_{50} = 5.52 \cdot 10^5$ m•to

$V_{ref} = 1.25\ V_{50} = 2250$ to

Foundation Stresses

1. Dead Load + Live Loads + Buoyancy

   Normal Force N = 94'800 to

   Average Stress σ = 5.5 kg/cm^2

2. 0.9 (Dead Load + Live Loads + Buoyancy)
   Reference Base Moment M_{ref}

   N = 85'300 to

   $M_{ref} = 5.52 \cdot 10^5$ m•to

   Max. Edge Stress σ = 10.5 kg/cm^2

Overturning Moment

In Fig. 6 the conditions are listed. A conservative estimate for the maximum edge stress σ_{max} = 84 kg/cm² equal to the unconfined compression strength of the shale (Table 1) is used. Assuming a normal force due to 0.9 (Dead Load + Live Loads + Buoyancy), N = 85'300 to, the following value results:

Overturning Moment:

$$M_{over} = 1.68 \cdot 10^6 \text{ m·to}$$

Hence, the safety margin becomes:

$$s = \frac{M_{over}}{M_{ref}} \cong 3$$

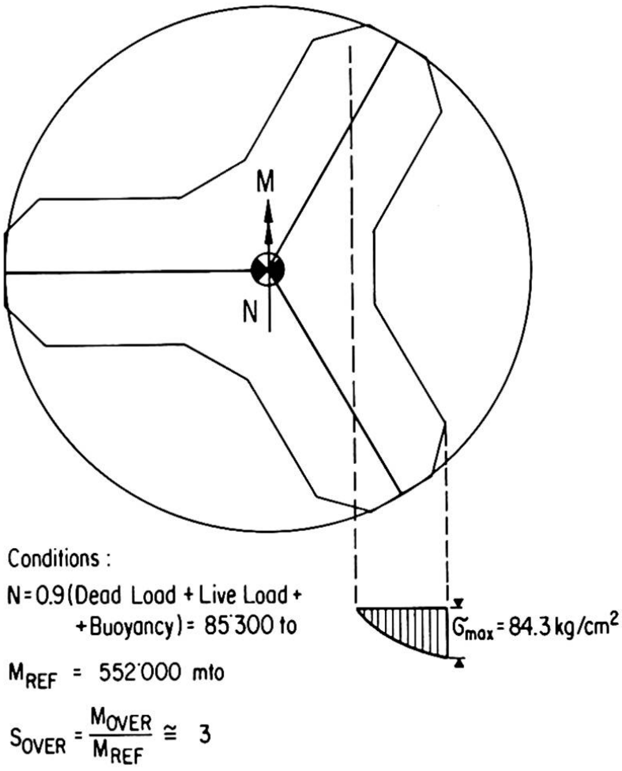


Fig.6: Overturning Moment

5. Acknowledgement

CN Tower Ltd., Toronto, are owners of the structure. The architects were John Andrews and Webb, Zerafa, Menkès, Housden, Toronto. For the structural design and the technical supervision R.R. Nicolet and Associates, Montreal, with F. Knoll in direct charge, were responsible. The Boundary Layer Wind-Tunnel Laboratory, University of Western Ontario, London, Ontario, was charged with the analysis of wind loads. The writer was consultant to CN Tower Ltd. and R.R. Nicolet and Associates and reviewed the structural design. He expresses his sincere appreciation to all parties involved for a most effective and stimulating collaboration.

6. Reference

Franz Knoll: "Structural Design Concepts for the Canadian National Tower, Toronto, Canada". Canadian Journal of Civil Engineering, Vol. 2, No. 2, 1975, pp. 123 to 137.

SUMMARY

The 550 m high CN Tower (447 m concrete shaft, 103 m steel antenna mast) is presently the highest free standing structure of the world. The Y-shaped foundation slab resting on a shale formation is prestressed. The reason for prestressing, the arrangement and stressing of the cables and some pertinent design data are presented.

RESUME

La "CN Tower", de 550 m de hauteur (la colonne en béton armé mesure 447 m et l'antenne métallique 103 m) est aujourd'hui l'ouvrage le plus haut du monde. La dalle de fondation précontrainte repose sur une couche de schistes argileux. Les raisons en faveur de la précontrainte, la disposition et la tension des câbles ainsi que différents aspects importants du dimensionnement sont présentés.

ZUSAMMENFASSUNG

Der 55o m hohe CN Tower (447 m Beton-Schaft, 103 m Antenne aus Stahl) ist heute das höchste freistehende Bauwerk der Welt. Die Fundationsplatte in Y-Form ruht auf einer Formation von Schiefertone und ist vorgespannt. Die Gründe für die Vorspannung, die Auslegung und das Spannen der Kabel sowie weitere wichtige Angaben über die Bemessung, werden aufgeführt.

IVb

Der Richtfunkturm des Fernmeldezentrums Arsenal in Wien

The Radio Link Tower of the Telecommunications Centre Arsenal in Vienna

La tour radiophare du centre des télécommunications "Arsenal", à Vienne

ROBERT KRAPFENBAUER
o. Prof. Dipl.-Ing. Dr. techn.
Ingenieur Konsulent
Wien, Österreich

Dieses bedeutende Turmbauwerk wurde im Auftrage der Post- und Telegraphendirektion Wien im Rahmen eines Fernmeldezentrums als größtes Postbauvorhaben Österreichs errichtet. Der entwerfende Architekt war Dr. Kurt Eckel, der Statiker Prof. Dr. Robert Krapfenbauer.

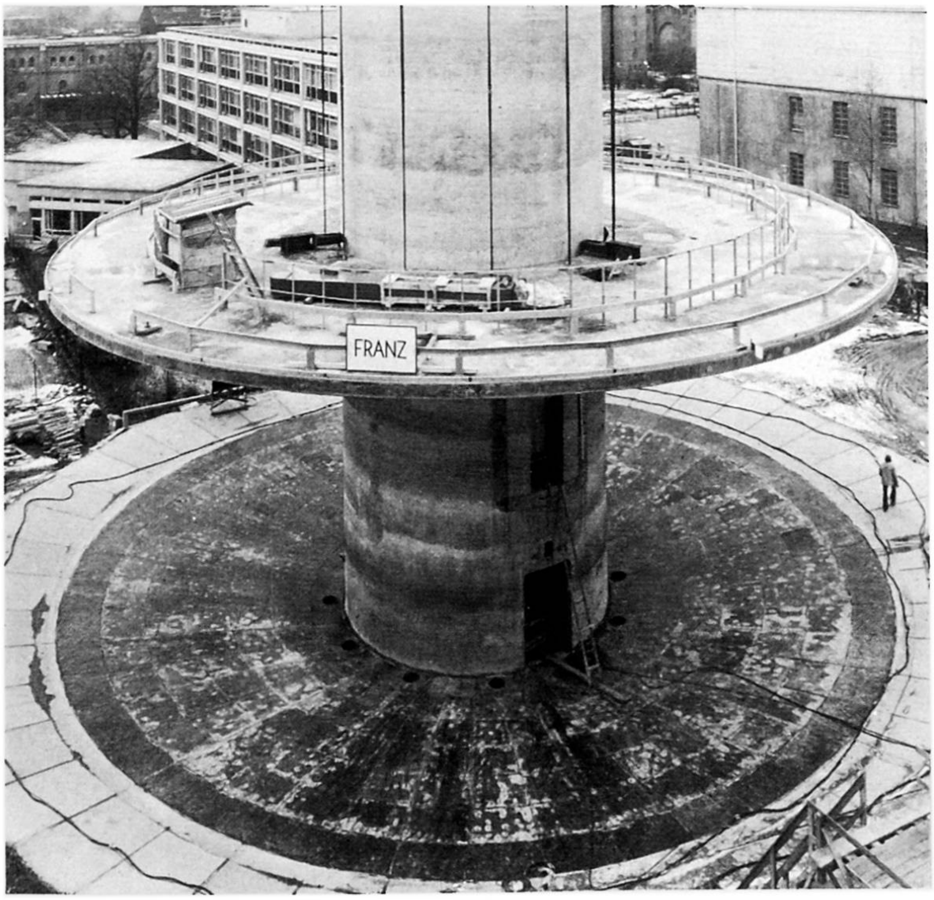
Die gesamte Anlage umfaßt zwei Betriebsgebäude von 12 bzw. 7 Geschossen, verbunden durch einen Verkehrskern, einen Bürotrakt, einen Zwischentrakt und den Richtfunkturm. Die verbaute Fläche beträgt 23.000 m^2. Der gesamte umbaute Raum beträgt 289.000 m^3. Das Turmbauwerk aus Stahlbeton hat eine Höhe von 136,6 m mit einer aufgesetzten 18,4 m hohen Stahlantenne, so daß die Gesamthöhe 155 m beträgt.

Der Richtfunkturm des Fernmeldezentralgebäudes Arsenal ist vom Postbetrieb her als hochgradig setzungsempfindliches Bauwerk aufzufassen. Für die Konstruktion des Gesamtbauwerkes war zur sicheren Gewährleistung des Fernmeldebetriebes die Forderung nach einem maximalen Endtangentenwinkel von 0,3 Altgrad ausschlaggebend. Ein nicht unerheblicher Anteil der Endtangentenneigung stammt von der Schrägstellung des Fundamentes unter der Wirkung des Einspannmomentes infolge Windwirkung auf den Turm.

Der Baugrund besteht aus einer im Mittel 4,5 m starken Anschüttung, der 8,5 m starken Arsenalschotterdecke und darunter liegenden tonigen Mittelpannon-Schichten. Aufgrund der Bodenanalyse und der Setzungsberechnungen konnte mit einer mittleren zulässigen Bodenpressung von $\sigma_m = 5,5$ kg/cm^2 gerechnet werden. Die Endsetzungen wurden mit 4 cm ermittelt.

Geophysikalische Untersuchungen ergaben relativ horizontal verlaufende Pannon-Schichten über eine Bruchstaffel des Hauptbruches des Wiener Beckens. Das Vorhandensein dieser Bruchstaffel weist auf die besonders bebengefährdete Lage des Baugrundes hin, in der Standberechnung mußte mit einer Horizontalbeschleunigung von g/40, dem fünffachen Mindestwert der österreichischen Normen, gerechnet werden.

Die Gesamtlast von 13.120 Mp ergibt bei einer Aufstandsfläche von 328 m^2 eine mittlere Bodenpressung von 4,0 kp/cm^2. Das maximal auftretende Moment aus der Überlagerung des Windmomentes mit den Momenten aus einseitiger Nutzlast, Bauimperfektionen und Momenten II. Ordnung beträgt 28.500 Mpm und bringt mit der maximalen Vertikallast eine Randpressung von 7,0 kg/cm^2. Selbst bei reinem Eigengewicht, ohne Erdauflast auf dem Fundament und maximalem Windangriff entsteht kein Klaffen in der Sohlfuge.



ANHEBEN DER PLATTFORMEN BEIM RICHTFUNKTURM ARSENAL

Der Gründungskörper des Fernmeldeturmes Arsenal besteht aus einer im Grundriß kreisförmigen Bohrpfahlwand und dem eigentlichen Turmfundament aus Stahlbeton B 300 im Inneren der kreiszylindrischen Bohrpfahlwand. Die Fundamentsohle liegt 8 m und die Pfahlspitzen 16 m unter Gelände.

Die Bohrpfahlwand mit einem Innendurchmesser von 21,30 Meter wird aus 90 cm starken, 16 m langen, überschnittenen Bohrpfählen gebildet. Sie diente einerseits als Baugrubenumschließung für die Erstellung des eigentlichen Fundamentkörpers, andererseits wird mit ihr eine 2 Meter starke Störzone in den oberflächennahen Bodenschichten durchfahren, so daß eine Lastübertragung des Turmbauwerkes auf eine 3 Meter starke, annähernd horizontal liegende Schicht des Wiener Tegels erfolgt.

DER RICHTFUNKTURM DES FERNMELDEZENTRUMS WIEN-ARSENAL

Das Fundament selbst wird aus einer 1,20 Meter starken kreisförmigen Sohlplatte, einer 70 cm starken Zylinderschale in Verlängerung des Turmschaftes, einer 65 cm starken, außenliegenden Kegelschale mit einer Neigung der Erzeugenden zu der Horizontalen von 52,15° und einer obenliegenden 80 cm starken horizontalen Abschlußscheibe gebildet.

Im Inneren des Fundamentkörpers befindet sich die Putzgrube für die Aufzugsanlage und ein Lagerplatz für Ausgleichsgewichte zur Kompensation einseitiger Nutzlasten auf den Turmplattformen. Der Hohlraum zwischen Kegel- und Zylinderschale ist überdies zu Inspektionszwecken begehbar.

Während des Bauvorganges des Turmbauwerkes und nach dessen Fertigstellung wurden laufend Messungen vorgenommen. Als Gesamtsetzung des Turmbauwerkes ergaben sich nach zwei Jahren Bestand 1,5 cm. Schwingungsmessungen ergaben genau den rechnerisch ermittelten Wert der Grundfrequenz, Messungen der Bewegungen des Turmes bei Windgeschwindigkeiten von 110 km/h ergaben Werte weit unter den vom Postbetrieb festgesetzten Grenzwerten. Es konnte somit nachträglich die Wahl der Fundierung des Fernmeldeturmes in Wien-Arsenal in technischer und wirtschaftlicher Hinsicht voll bestätigt werden.

Der Turmschaft wurde sodann auf dem Fundamentkörper in Gleitbauweise errichtet. Der Schaft hat einen konstanten Durchmesser von 8,4 m; die Wanddicke wird vom Erdniveau bis zur Spitze in drei Stufen reduziert (von 70 cm über 50 cm bis auf 35 cm). Auf dem Stahlbetonschaft liegt eine Betonplatte von einer Dicke 1,50 bis 1,85 m auf, die den Stahlmast trägt.

Auf dem Turm sind vier scheibenförmige Antennenplattformen sowie ein Betriebsgeschoß in ihrer Mitte angebracht. Zwei der Stahlbetonplattformen haben Durchmesser von je 25,4 m und wiegen je 470 Tonnen, zwei haben Durchmesser von 30,20 m und wiegen je 600 Tonnen. Alle vier Plattformen haben vorgefertigte Versteifungsrippen und einen vorgespannten Umfassungsring an der Außenkante. Der untere Teil der Plattformen besteht aus einer flachen, konischen Schale, die zusammen mit den vorgefertigten inneren Radialrippen und der oberen Deckplatte ein steifes Raumtragwerk bildet. Die Dicke der konischen Schalen liegt zwischen 8 und 9 cm. Die Anordnung der Schalen auf dem Stahlbetonschaft ist (von unten nach oben) folgende: eine kleinere, eine größere Plattform, das Betriebsgeschoß, eine größere und schließlich oben wieder eine kleinere Plattform.

Das Betriebsgeschoß, genannt Gondel, ist eine Stahlkonstruktion mit Außenverkleidung aus Aluminium-Blech und Glas. Der Durchmesser dieses Geschosses beträgt 35 m.

Sowohl die vier Antennenplattformen als das Betriebsgeschoß wurden auf Erdniveau gegossen und montiert und schließlich mittels der Lift-Slab-Methode auf ihre endgültige Höhe gebracht. Dies war erforderlich infolge der hohen Windgeschwindigkeiten von 75 bis 90 mph an der Turmspitze (100 m), die für eine Herstellung an Ort zu gefährlich gewesen wären.

Der Bauvorgang war folgender:
Für die beiden oberen Plattformen wurden Schalungen angefertigt, die Plattformen hieraufgegossen und gehoben. Als nächstes wurde das Betriebsgeschoß auf provisorischen Stützen in geringer Höhe über dem Erdboden montiert und geliftet. Die Schalungen für die verbleibenden beiden unteren Stahlbetonscheiben blieben zur Wiederverwendung erhalten. Nach dem Hochziehen wurde jede Plattform mittels zwölf stählerner Halterungen am Turm befestigt. Durch die Anwendung der Lift-Slab-Methode konnte die Konstruktionszeit auf drei Jahre beschränkt werden.

ZUSAMMENFASSUNG

Der Richtfunkturm des Fernmeldezentrums Arsenal in Wien stellt ein Turmbauwerk von 136,6 m Höhe mit einer aufgesetzten 18,4 m hohen Stahlantenne dar, sodass die Gesamthöhe 155 m beträgt. Der Gründungskörper für den Turm besteht aus einer im Grundriss kreisförmigen Bohrpfahlwand und dem eigentlichen Turmfundament aus Stahlbeton im Innern. Die Fundamentsohle liegt 8 m und die Pfahlspitzen 16 m unter Gelände. Der Turmschaft aus Stahlbeton wurde in Gleitbauweise errichtet; er trägt vier Antennenplattformen aus Stahlbeton und ein Betriebsgeschoss in Stahlkonstruktion. Diese fünf Teile wurden auf Erdniveau hergestellt und im Lift-Slab-Verfahren hochgezogen.

SUMMARY

The Radio Link Tower of the Telecommunications Centre "Arsenal" in Vienna is a tower structure of 136,6 m height with a 18,4 m high steel antenna set on it, so that the total height is 155 m. The foundation body for the tower consists of a circular wall of boring piles and the foundation itself of reinforced concrete within this wall. The sole of the foundation is 8 m and the pile ends

lie 16 m below ground-level. The tower shaft of reinforced concrete was built by the sliding consctruction method; it carries four antenna platforms of reinforced concrete and one equipment pod in steel construction. These five parts were made and assembled on ground level and lifted by means of the lift-slab-method.

RESUME

La tour radiophare du centre des télécommunications "Arsenal" à Vienne est une tour de 136,6 m de hauteur avec une antenne métallique de 18,4 m à son sommet de sorte que la hauteur totale est de 155 m. La fondation de la tour consiste en une paroi circulaire formée de pieux, et le corps propre de la fondation à l'intérieur de ce cercle est en béton armé. La base de la fondation est à une profondeur de 8 m, les pointes des pieux à 16 m de profondeur. La tour est en béton armé; elle porte 4 plate-formes en béton armé pour les antennes et un étage de service en construction métallique. Les 5 parties ont été construites au niveau du sol et levées par la méthode "Lift-Slab".

Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

FRITZ LEONHARDT
Prof. Dr.-Ing.
Leonhardt und Andrä, Consulting Engineers
Stuttgart, GFR

Foundation Structures for Long Span Bridges

1st Paper

The foundation of the Seine River Bridge Brotonne, a cable-stayed bridge of prestressed concrete with a main span of 320 m (fig. 1) (reported by J. L. Brault and J. Mathivat) is in different ways interesting. The bridge deck is about 54 m above water-level, the single pylon in the central axis of the bridge is 70 m high; pylon and pier together have a total height of 125 m (fig. 2). At the foot of the pier there act vertical loads of 17 300 tons and a transverse bending-moment due to wind forces of 31 800 tm. The French engineers chose a single circular column with a diameter of only 12.5 m and a depth of 35 m for the foundation of the tower-like piers of this very great bridge (fig. 3). The upper 9 m of the soil are soft, underlying are 8 m of alluvial sandy clay followed by 10 m of sand and gravel, resting on chalk with a compressive strength between 20 and 80 kp/cm^2. The column of the foundation goes 8 m deep into this chalk corresponding to only 2/3 of her diameter.

For carrying the load and moment, the horizontal earth resistance was taken into account in spite of the fact that the stiffness coefficient of the 8 m thick alluvial soil is only 0.12 kp/cm^3 and that of the sand-gravel only 1.5 kp/cm^3. The chalk also has a rather small stiffness coefficient for horizontal pressure of 8 kp/cm^3. With these assumptions, the soil pressure distribution was calculated as shown in fig. 4 with a maximum horizontal pressure above the chalk of 2.6 and 2.8 kp/cm^2. In the base section of the column, there is a remaining bending moment of 18 000 tm, which results in large differences of vertical soil pressure with a maximum value at the edge of 23.8 kp/cm^2. At this level, the stiffness coefficient of the chalk for vertical pressure would, therefore, cause a considerable deformation.

This is a daring foundation, however, one can expect that it will behave well, because the extreme wind loads caused by extraordinary gales in costal environment are relatively rare and their peak gusts, which develop highest wind velocities, last usually only one or two seconds. One knows that the statically calculated soil pressure will not realize by such short-time load attacks due to the mass inertia of the big structure and of the soil. Therefore the base of the foundation will mainly have to carry the vertical load and will not get to feel much of the wind loads.

But not only the small dimensions of the foundation of this large bridge are interesting but also the construction method. The construction was started with a self-supporting slurry trench wall acting as a cofferdam, built with 80 cm thick unreinforced concrete to a depth of 31 m (fig. 5) intruding 4 m deep into the chalk. The material inside this cylinder-cofferdam was dredged into the chalk and the concrete for the base could be executed in the dry. In this way it was also simple to construct the thick reinforced concrete walls of the main column.

This example demonstrates how economical such foundations of large bridges can be built, if one makes good use not only of the vertical bearing capacity of the soil, but also of the horizontal resistance. Hereto it is necessary to investigate reliably the data for soil behaviour, as it has been done for this bridge by several core drillings with 120 mm diameter to a depth of 50 m.

2nd Paper

In the second report, the Japanese engineers M. Ohashi, S. Kashima and O. Yoshida describe the foundation of the towers of the suspension bridge with a main span of 870 m, the Ohnaruto Bridge, which belongs to the system of bridges which shall connect the main island with Shikoku Island. The towers stand in shallow water of only 3 m depth on sandstone, the construction of the foundation will, however, be influenced by strong tidal currents with velocities up to 5 m/sec.

At the beginning it is reported that the Japanese turned during the last 12 years more and more to pile foundations for their large bridges. At the Ohnaruto Bridge, there are 9 big piles in steel pipes with diameters of 4.6 and 7.2 m under each tower leg (fig. 6), which intrude 15 m deep into the sandstone rock. The solid concrete pile cap has a thickness of 9 m. The caps for the two groups of piles are connected with a 6 m thick slab of reinforced concrete.

The multi-column-foundation was analysed as a space frame, the pile legs being spring-supported horizontally and vertically to imitate the response of the rock. The analysis was checked by model tests and by large scale models on the real ground.

With regard to earthquakes, dynamic model tests were also conducted not only with the pile-foundation but for comparison also with solid block-foundation.

The result was, that the multi-column-foundation shows larger deformations but also higher natural frequencies than the solid block-foundation (fig. 7). The natural frequency of the foundation is, on the other side, so different from the frequency of the steel towers, that the dynamic analysis of the tower and of the foundation can be done individually. The first mode of vibration is dominant, because the pile cap is so heavy.

The analysis as space frame gave a good agreement with model measurements. It was found also that the horizontal spring constant of the ground related to the total multi-pile-foundation is equal to that of one pile multiplied by the number of piles.

3rd Paper

Pile-foundation for long-spanned bridges are also described by the Russian engineers K.S. Silin, N.M. Glotov, V.N. Kutzenko and G.P. Solovyev (fig. 8). They often use piles made of prefabricated reinforced concrete tubes with a diameter of 3 m and a wall thickness of only 12 cm. 6 m long elements are flanged together with bolts. These large pipes are driven into the alluvial ground by vibration and simultaneous dredging. If the foundation reaches rock, then 3 to 4 m deep holes are drilled into the bed-rock by the use of a special turbo-drilling machine, which was developed for the oil industry of the USSR. After cleaning, the hole and the concrete pipe are filled with heavily reinforced concrete.

For a large railroad bridge, inclined drilled piles with steel pipes of 1.4 m diameter had been used, which reach to 40 m depth under the water level and end in dense clay (fig. 9). At the foot of the pile a conical cave is cut out of the clay with a diameter up to 3.5 m in order to increase the bearing area of the pile. This hole was filled with bentonite slurry to protect it against possible soil slides. After introducing the reinforcing cage, the concrete was filled in through pipes. The allowable capacity of such a pile is 800 tons, test loadings went up to two times this value.

4th Paper

In the fourth report, the Japanese engineers T. Okubo, K. Komada, K. Yahagi and M. Okahara describe a new type of foundation in deep water with sheet piling, using steel pipes with diameters up to 1.2 m and wall thicknesses up to 19 mm (fig. 10).

At the outer steel pipe faces, there are two small diameter steel pipes with a slot, which are used to guide each following pile during the driving process. The voids of the small pipes are filled with mortar in order to get a stiff and tight connection between the pipes.

The flexural stiffness of the pipes allows to use the structure as a cofferdam. With such pipes, ring shaped or oval or rectangular,foundation arrangements can be made in sizes suitable for bridge piers (fig. 11).

The authors describe different possibilities of application (fig. 12). The footing of a bridge pier can be placed directly on top of the steel pipes above the water level. The water can also be pumped out of the interior of the sheet piling to a certain depth and steel brackets can be welded to the pipes for carrying the footing of the pier. After concreting the pier, the sheet piling above this level can be cut off.

The authors describe how they calculate such sheet pile foundations by means of a finite strip method, using different coefficients of the sliding resistance in the connection between adjacent pipes and assuming different distribution of the soil pressure. The results of these analyses have been compared with measurements at a large scale foundation with circular pile arrangement. The analytical result is strongly influenced by the amount of sliding resistance between the steel pipes and the comparison with measurements is not satisfactory. The authors conclude that more rational design methods should be developed and good design standards should be established.

This pipe-sheet pile foundation has already been used for 12 bridges with good success und with pile lengths up to 57 m. This special foundation can undoubtedly be further developed, a stiff connection between the pipes will be desirable in order to get a good use of the possible spatial resistance of such structures.

5th Paper

In the last contribution, the Japanese engineers S. Suzuki, M. Ishimaru, F. Nemeto and Y. Nojiri give a report on model tests for the foundation of the main pier of the Hamana Ohhashi Bridge, which has with 240 m the longest span of prestressed concrete box girder bridges. The foundation has to carry 27 000 tons vertical load and must be able to resist the horizontal inertial forces caused by the heavy superstructure during earthquakes, which result in bending moments longitudinally of 181 000 tm and transversely of 188 000 tm.

A rectangular multi-cell box caisson with slide lengths of 26 / 19 m and a depth of 30 m was adopted (fig. 13). The piers on top of this caisson are 20 m high.

Two models of acrylic resin were made. At the first model surface strains were measured in the usual way. The second model was used for three-dimensional photoelastic stress analysis, it was heated up to 130° C, loaded and cooled down. Thin slices of the model are then cut out which show the interior stress distribution in polarized light. The results (fig. 14) confirm the nonlinearity of stress diagrams in thick members. The distribution of shear stresses is not reported. Also the lengths of introduction of load attacks in prismatic bodies are confirmed.

The caisson was also analysed three-dimensionally with a finite element program which, however, was developed for thin plates, therefore this analysis is questionable. It was also used to study the influence of Poisson's ratio which is 0. 39 for acrylic resin and 0. 17 for concrete. The smaller value of the concrete leads to smaller flexural stresses in the thick upper plate of the caisson.

At this investigation, data are missing about the distribution of soil pressures, which must play a considerable part in the resistance of earthquake forces. The results show, on the other side, that the forces due to earthquakes do not cause serious stresses in such caisson foundations, even not under such heavy loads.

Remarks

From the contributions to the preliminary report we can gather that big progress is made with pile foundations. The diameters of the piles and their lengths are more and more increasing. The reports deal mainly with stress analysis, mathematically or by model tests, but say nothing about the distribution of the load carrying reactions of the soil either by the pile foot or by friction along the pile's surfaces. One misses also considerations or means to secure the durability and life of the piles, for instance in salt water or in polluted harbour waters. One is further missing structural details, mainly details of the reinforcement of piles and pile caps. In my introductory report I have pointed to the splitting forces at thick reinforcing bars caused by high bond stresses as they develop in pile caps under large bridge loadings. Such splitting forces can be a considerable danger for the safety of such structures which most engineers do not yet realize.

The pile caps are usually large size solid concrete members in which the concrete stresses due to loads may be smaller than the stresses due to interior restraint caused by temperature or shrinkage differentials. For such thick concrete members prestressing to a degree to counteract tensile stresses under permanent load can lead to better behaviour and more safety than a heavy reinforcement with thick bars.

It is desirable that researchers and practising engineers being responsible for the safety of the structures, will pick up these problems and do not simply place several layers of thick reinforcing bars with small spacing into such pile caps. It can safely be predicted, that such reinforcements will not give the carrying capacity as calculated, but the concrete will split in the plane of the reinforcing bars before the structure reaches the required ultimate load.

Closing, I wish to draw your attention again to my introductory report for this Congress in which I have described a proposal for the foundation of the towers of a cable-stayed bridge with a span of 1 470 m for the crossing of the Straits of Messina which would have to be built in 95 m deep water. I proposed an economical ringshaped foundation of which the lower part would have to be built in a dry dock and then floated to the bridge site. The proposal was made in 1972 before Ekofisk in the North Sea was built. The extraordinary achievements of the engineers who have meanwhile built the oil tanks and drilling-platforms in the North Sea did prove that such a construction method will also be safely applicable for bridge piers in these days.

In the preliminary report of this Congress you find under theme IVa a paper of our French engineer colleagues M. Gerbault and P. Xercavins informing us that the Shell platform CORMORANT will be founded in the North Sea at a place where the water is 154 m deep. Compared to these gigantic structures our bridge piers are small. But the large height of the foundation tower suggests to use ring foundation similar to those which I developed for large TV towers. The proposed ring foundation will cut down the necessary amount of concrete and steel considerably and will result in great economy compared to conventional types of foundation. I would appreciate if this proposal would be considered for the big bridges crossing sea straits, which are being planned in several parts of the world.

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Preliminary Main Pier Designs for the Great Belt Bridge

Avant-projet des piles principales du pont sur le Great Belt

Vorstudien für die Pfeiler der Brücke über den Grossen Belt

NIELS J. GIMSING
Professor
Technical University of Denmark
Lyngby, Denmark

The design of the main piers for the Great Belt Bridge has during the last decade been investigated in a number of studies.

In the chosen bridge alignment the water depth reaches 55 m, and a direct foundation can be established on Danian Limestone or Paleaoic Marl few meters below.

This paper describes shortly three pier designs, here designated pier A, B and C.

PIER A

For the International Design Competition 1966 a road bridge as shown in figure 1, with a main pier design as shown in figures 2 and 3 was proposed by the author et al [1].

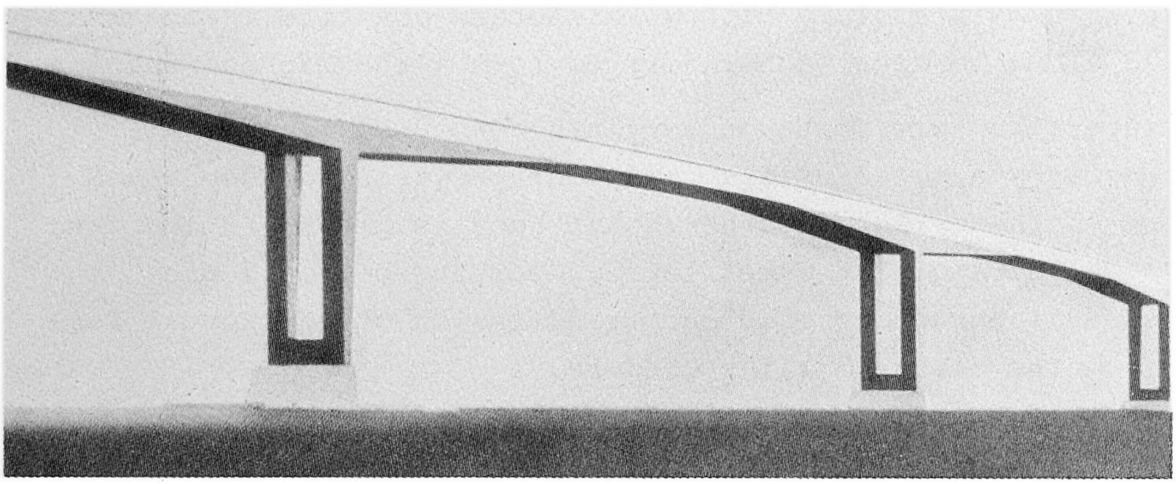

Figure 1. Road bridge designed as a frame with two 400 m wide main spans. From a first prize design by Gimsing, Madsen and Nissen in the International Design Competition 1966.

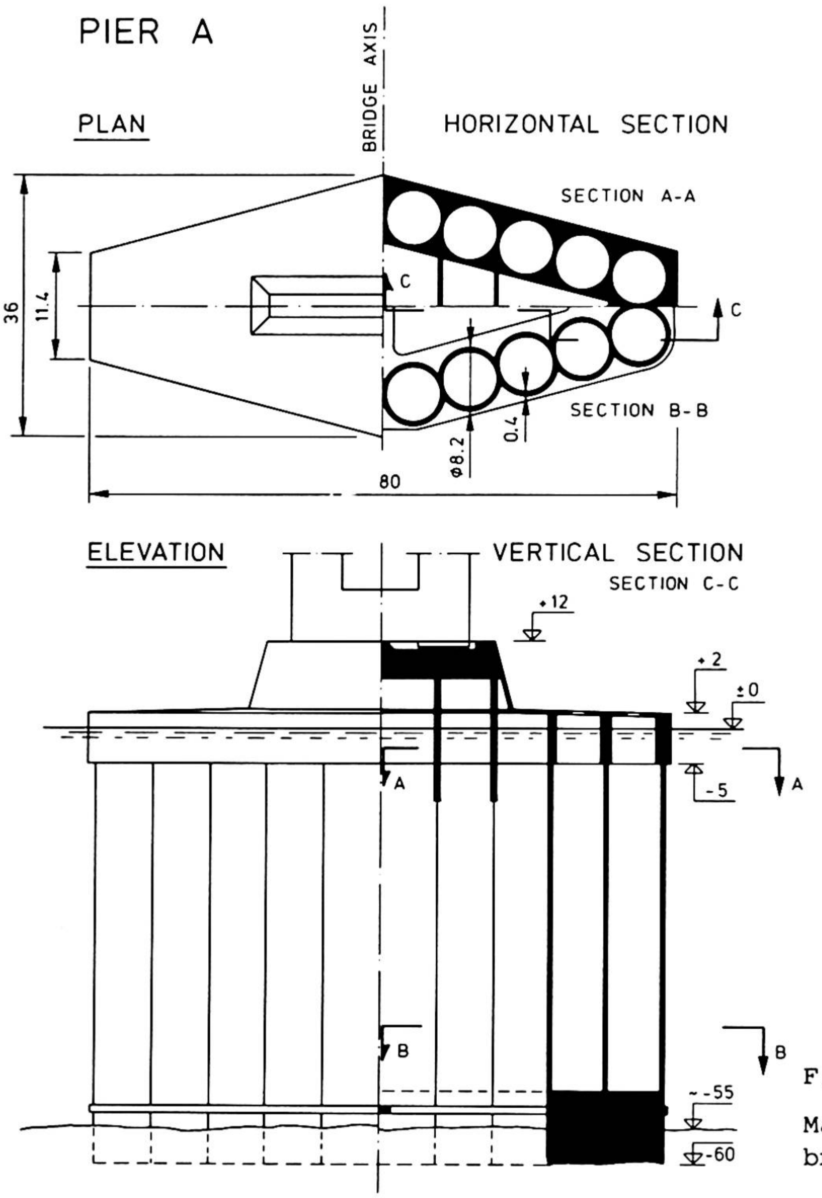

Figure 2.

Main pier design for the bridge shown in figure 1.

This pier, founded 60 m below water level, should be designed for very large forces from ice pressures and collision (equivalent static collision force: 120 MN).

The pier consists of 20 hollow reinforced concrete cylinders joined along vertical generators to form a rhombic box section.

The superstructure rests on a plinth connected to the wall of cylinders by a covering slab and four prestressed deep beams. On top of the cylinders a hexagonal hood is placed to increase the local strength at water level. Near the bottom a prestressed diaphragm assures the transmission of horizontal forces from the earth to all cylinders.

The construction of the pier should start in a dry dock by casting the lower part. After placing of temporary bulkheads

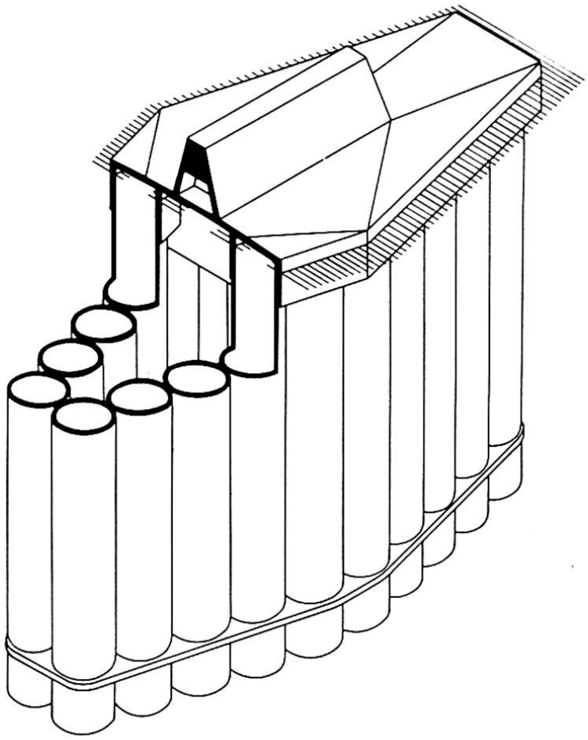

Figure 3.

Isometric projection of the pier design shown in figure 2.

the pier can be floated to deeper water and the remaining part cast in slip forming. When placed in final position the temporary bulkheads are removed and excavation inside cylinders carried out. Finally, concrete plugs are cast under water at the bottom of all cylinders.

PIER B

1970-72 a Technical Commission appointed by the Ministry of Public Works made a comparative study of different solutions for the Great Belt crossing, among these the cable-stayed bridge shown in figure 4. [2].

Here a more solid pier structure, as shown in figure 5, was proposed. The pier is composed of square wells, 42 in the lower and 20 in the upper part.

In principle this pier should be constructred as pier A, with the exception that temporary steel tubes had to be added to prolong the outer wells during floating and excavating.

A dynamic investigation of the collision case gave the following results:

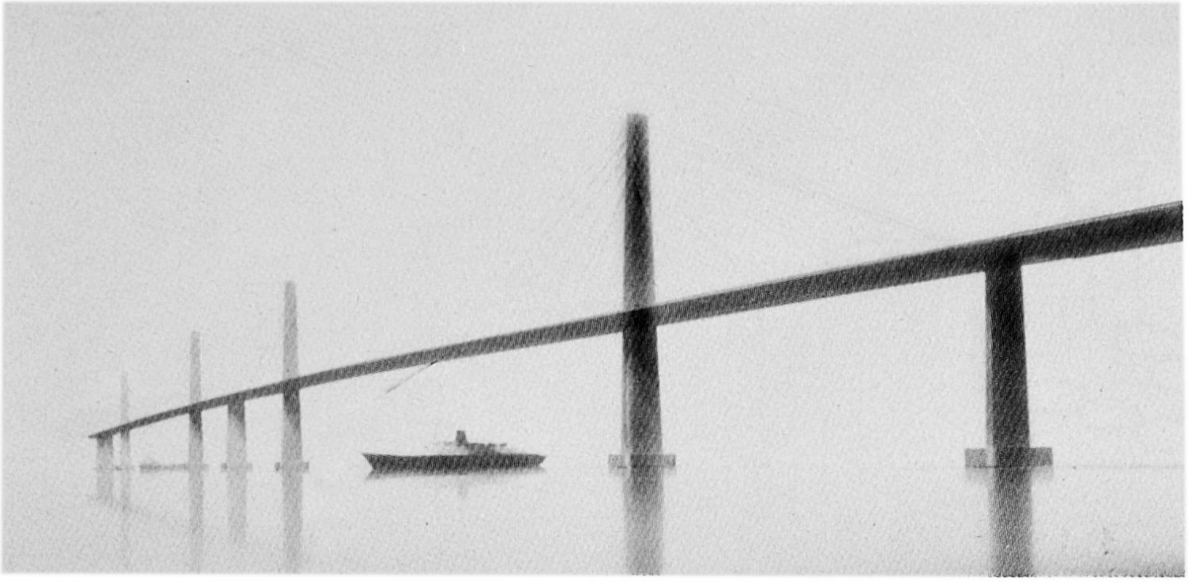

Figure 4.

Combined road and rail bridge designed as a cable-stayed bridge with two 600 m wide main spans. From the investigations made by the Technical Commission 1971.

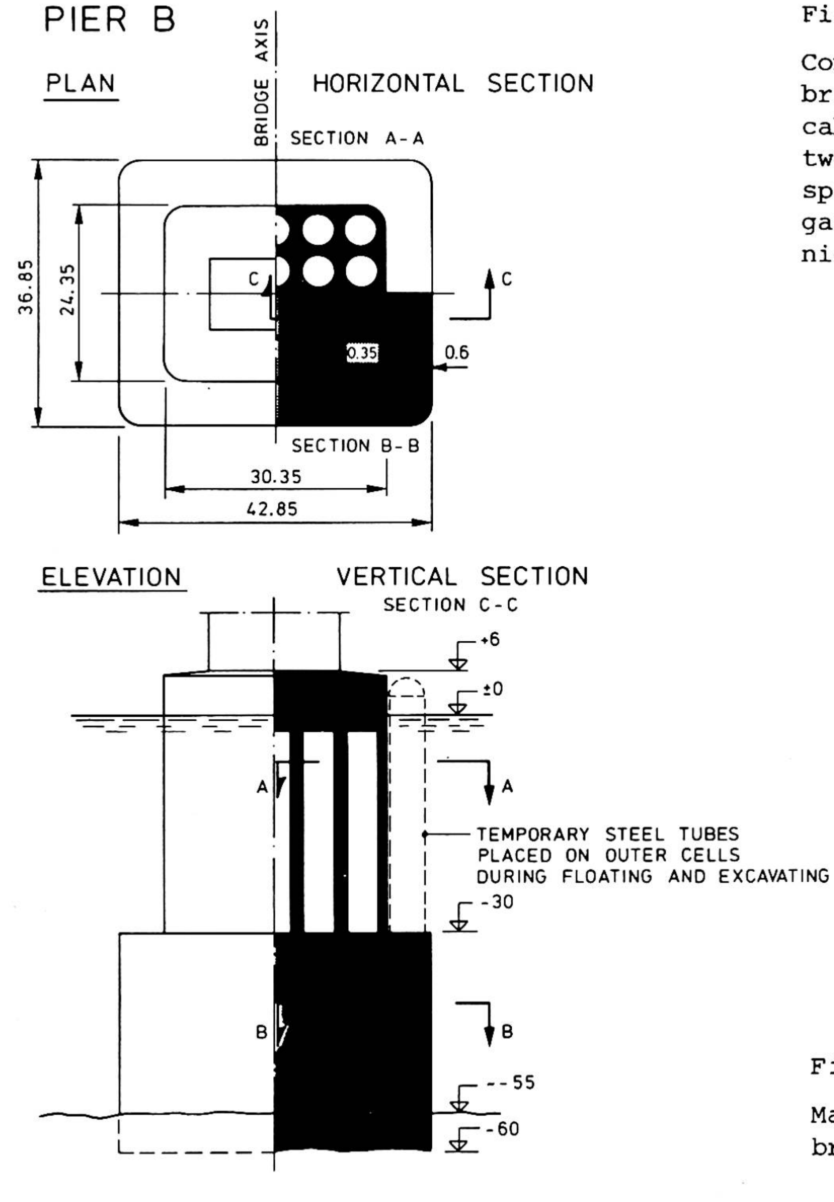

Figure 5.

Main pier design for the bridge shown i figure 4.

A collision by a ship of maximum 14.000 tdw going at full speed, or a tanker of 100.000 tdw drifting abeam at current speed would only give local damage.

A collision by a tanker of 100.000 tdw going at full speed will give a permanent horizontal displacement of 0.1 m at water level. In the superstructure this displacement only induces a flexural stress of 90 MPa and consequently no danger of collapse would exist.

PIER C

Recently, the author has made a study of a cable-stayed bridge with two main spans separated by a triangular pylon structure (figure 6).

The corresponding pier structure, shown in figure 7, contain a wall of cylinders as in pier A, but the number is increased to 26 due to the larger width required to give support for the double pier shafts.

Despite the significantly larger outer dimensions pier C requires only slightly larger quantities than pier B. This is due to the hollow design, which also gives an advantageous increase of the stability.

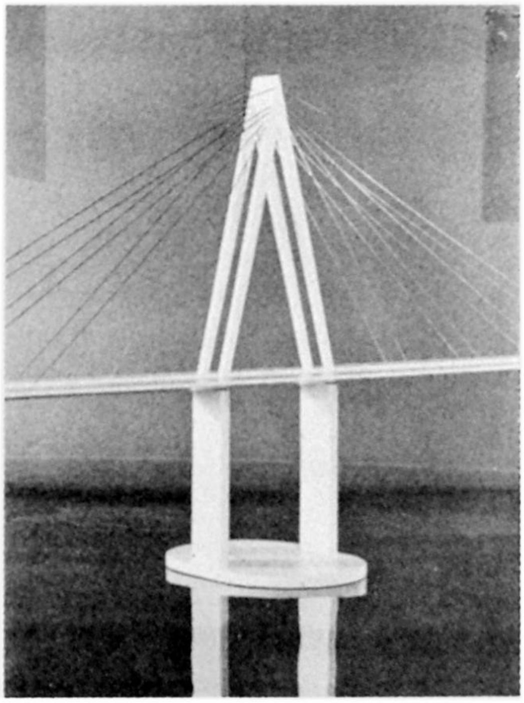

Figure 6.

Triangular pylon structure investigated by the author 1975.

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[1] Gimsing, N.J., K. Madsen and J. Nissen: Great Belt Bridge. Våg- och Vattenbyggaren. No. 5, 1967.

[2] Gimsing, N.J.: Pont sur le Grand-Belt (Danemark), Travaux 1973 no. 463.

[3] Leonhardt, F. & W. Zellner: Cable-stayed Bridges: Report on latest developments. Canadian Structural Engineering Conference, 1970.

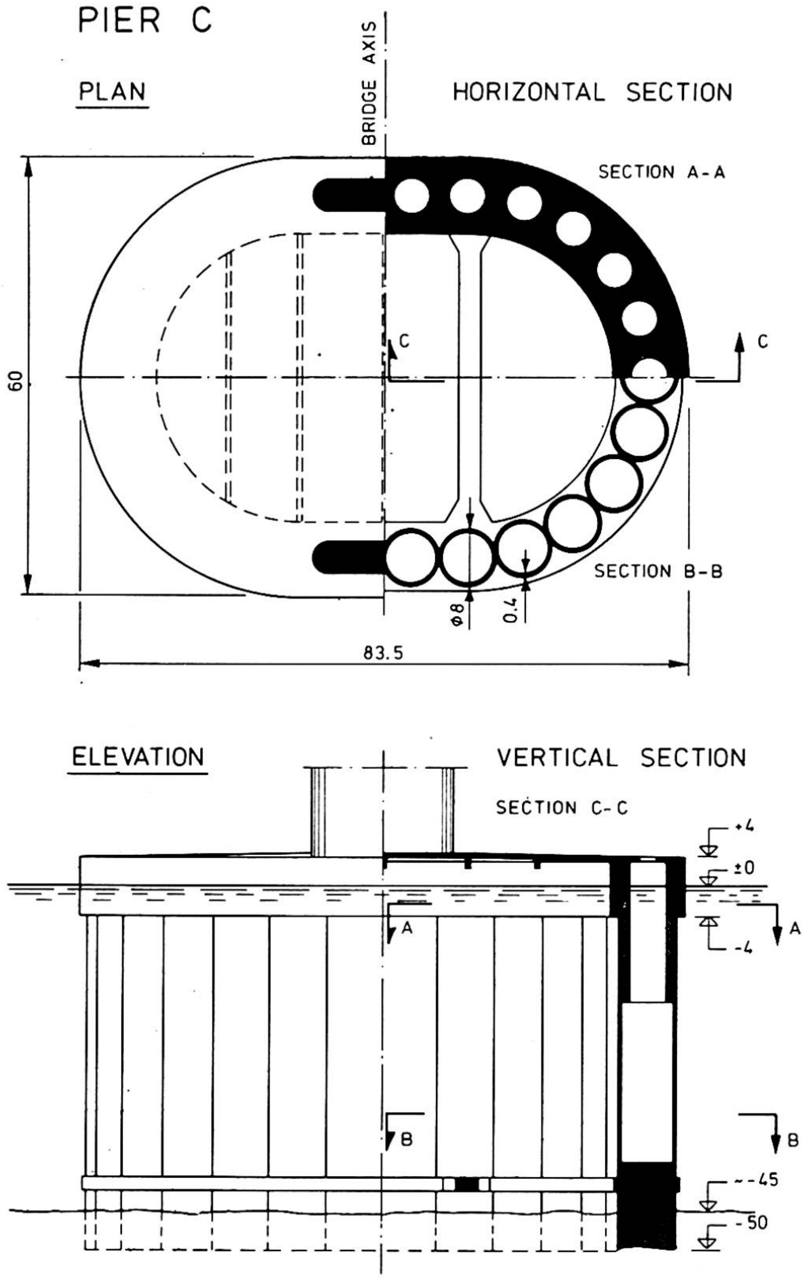

Figure 7.

Main pier design for the pylon structure of figure 6.

[4] Leonhardt, F.: Fundationen für weitgespannte Brücken, Einführungsbericht, IVBH, 10. Kongress, Tokyo 1976.

[5] Gimsing, N.J.: Multispan Stayed Girder Bridges. ASCE Journal of the Structural Division, October 1976.

SUMMARY - Preliminary investigations have indicated that the main piers for the Great Belt Bridge can advantageously be constructed by using hollow concrete caissons.

RESUME - Les études préliminaires ont montré que les piles principales du pont sur le Great Belt pouvaient être avantageusement réalisées à l'aide de caissons creux en béton.

ZUSAMMENFASSUNG - Vorstudien für die Pfeiler der Brücke über den Grossen Belt haben gezeigt, dass Caissons aus Beton Vorteile bieten.

Foundations of Zarate-Brazo Largo Bridges

Les fondations des ponts Zarate-Brazo Largo

Die Gründung der Zarate-Brazo Largo Brücken

SILVESTRO BRUSCHI
Chief of Concrete Department
TECHINT Engineering Company
Buenos Aires, Argentina

1 - Introduction

The Zárate-Brazo Largo Highway-Railway Complex, now under construction, will permit the crossing of the Paraná River at about 80 km North of Buenos Aires City. At present this crossing is made using rafts and ferry-boats, and these bridges will permit a great saving in time and cost, not only for the Argentine interprovincial traffic but also for the international traffic with Uruguay and Brasil.

At this point, the Paraná River is divided in two branches, each navigated by ocean going ships. The Complex is, then, formed by two identical bridges with a free span and clearance of 300 m x 45 m respectively. The bridges' decks support one railroad and one four lanes highway.

Each bridge is 550 m long, with a central span of 330 m and two lateral spans of 110 m each. Both are cable-stayed bridges with cables fanning out from the top of the 120 m high towers. These towers, as well as the two anchorage pier, are of reinforced concrete hollow rectangular section. The deck is of high resistance steel, and is formed by two lateral box girders and plate deck, suitably stiffened and covered with a reinforced concrete pavement.

2 - Description of Foundations

Both bridges' main piers (towers) and the two anchorage piers of one of them are on the river bed. The other two anchorage piers, and almost every access viaduct pier, are on the flood bed of the Paraná River.

The soil over the entire width of the river flood bed is formed by alluvial deposits of great depth. The upper layers show lime and soft clay. At different depths there are some half consolidated clay layers, sand layers, and finally very thick dense sand layers. These dense sand layers can take the loads transmitted by the piers. The upper part of these sand layers is at depths ranging between 20 and 50 m.

Consequently, all piers were founded on big diameter piles. Two-meter diameter cast in place piles were used, with steel casing when built in water, and without steel casing when built on land.

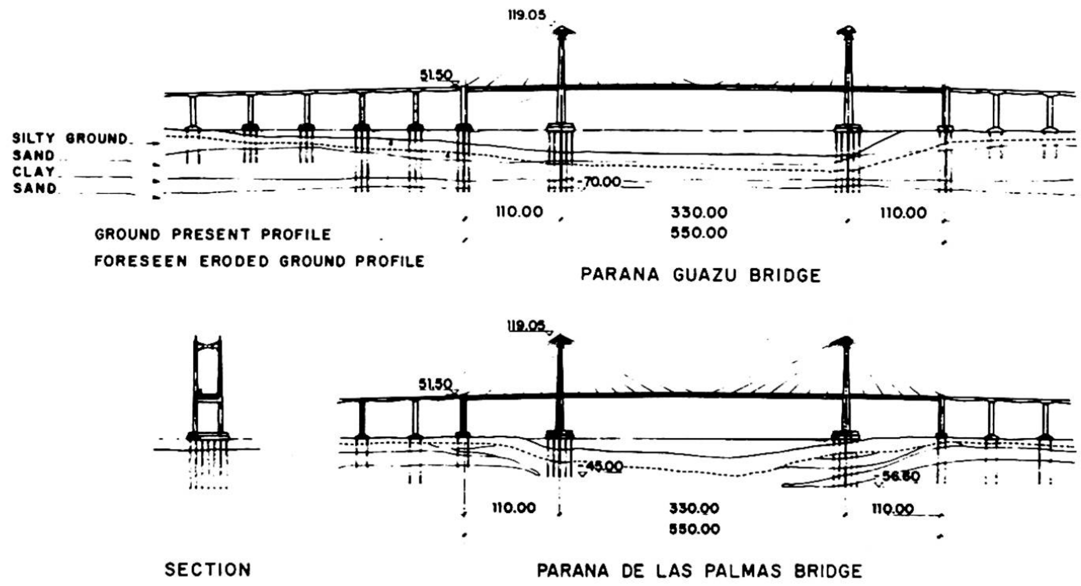

Fig. 1 - Bridges lateral views and geological profiles

The piles' length was determined by the need of entering a minimum of 5 m in dense sands. Also the eventual future modifications of the river beds due to erosion phenomena both general and around the piles, have to be taken into account in said determination. Thus, pile length varied between 25 and 73 m.

Parallelepiped pile caps, lightened by hollows, join the piles. These pile caps are built over the water level. Preformed slabs hang from their lower edge and sink some centimeters into the water for aesthetic reasons.

Based on the soil characteristics and taking into account load transmission by the tip and lateral friction, the maximum allowable vertical load for piles in water was established at 1200 tons plus the pile weight.

A simple arrangement was adopted for the plant distribution of piles. They were lined up in both directions, separated by a distance practically the same as the minimum advised (three times the pile diameter). Because of the different water depth in both rivers, now and even more in the eventual future situation of an eroded bed, a greater number of piles was needed for the Paraná Guazú than for the Paraná de las Palmas. Because of its greater slenderness and greater bending moments due to horizontal loads, the maximum allowable vertical load on the pile head had to be reduced in the Paraná Guazú piles to 1000 tons.

For the Paraná Guazú main piers, 45 piles and one parallelepiped pile cap were adopted. The pile cap has a plant of 30 m x 59 m, and a thickness of 5,50 m and is lightened by hollows in the less stressed areas. The concrete volume for this cap was 8,300 m3, reinforced with 1,100 tons of high strength steel.

For the Paraná de las Palmas, 33 piles and one pile cap, with an "I" type plant of 30 m x 45 m and a thickness of 5,50 m, also lightened, were adopted. The concrete volume was 6050 m3, reinforced with 860 tons of high strength steel.

3 Pile-caps Calculations

Pile cap as a whole is a thick slab bearing on a series of point supports. The load on this slab is applied on rectangular areas noticeably smaller than the slab's dimensions. This slab is lightened by cubic holes in areas far from the load application area. Thus, it might be considered as a grid, supported on every point of girder crossing. However, the girders are joined by very thick slabs, in their upper and lower parts, which increase the grid stiffness considerably.

Hence, we performed a double calculation following two different approaches:

The first one was to use a grid model, solving it by STRUDL. Loads on piles and values for bending moments and shear forces on the different members were thus obtained and the necessary reinforcing steel sections for the different parts of the pile cap was determined.

The second approach was to try to follow the stress paths into the mass of pile cap accepting the same loads on piles as obtained by the first procedure. Concrete stresses were verified and the necessary steel sections for tensioned members were determined. These were compared with those steel sections calculated before, and the bigger one was selected in each case.

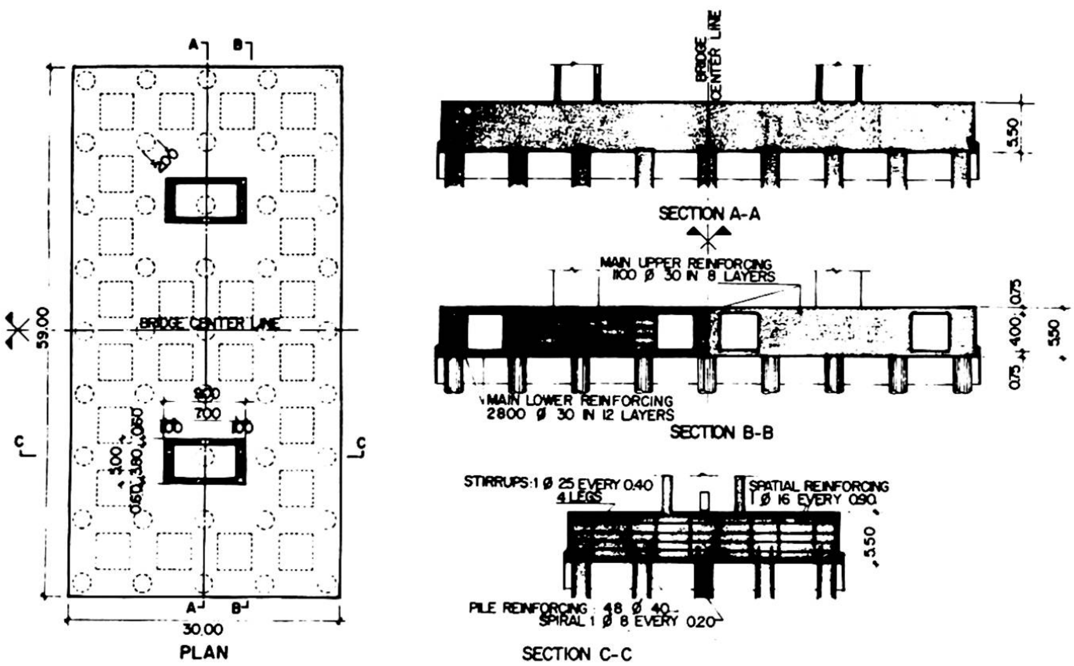

Fig. 2 - Paraná Guazú Main Piers Pile Caps

4 - Piles Calculations

Two questions arose from the beginning of the project regarding dimensioning and verification of piles in water:

a) How to determine the actual pile embedding into the soil, in order to establish buckling length and bending due to horizontal loads.

b) How to consider the pile cross section to take into account steel casing.

Because of scheduling it was impossible to perform horizontal load test in time to use its results for design. Thus, the Winkler elastic model of the pile-soil system was adopted. More sophisticated models, i.e. considering the pile as an elastic element sunk into a semi-infinite elastic medium, or taking into account the soil plasticity and viscosity were discarded due to the lack of enough experimental data.

According to the adopted model, it is acceptable to consider soil horizontal reaction P, at a certain depth x, proportional to the horizontal displacement of this point of the pile, that is: $P = Es \cdot y$

Factor Es is the soil reaction module, which generally varies with depth. Taking into account soil characteristics, it was decided to adopt a parabolic relationship, as follows: $Es = 50 \sqrt{z}$ (z expressed in cm; Es in kg/cm2).

Once the supposed elastic fixing characteristics were thus defined, the solution of the structural problem could be obtained replacing this elastic fixing by a rigid fixing at a certain depth $\Delta\ell$ below the soil level. To obtain this depth, we calculated the length of a pile rigidly fixed at both ends, with the same cross section as the actual pile, which under an horizontal load on its head will deflect the same as the pile perfectly fixed in the pile cap and elastically fixed in the soil would do. In this case, we obtained $\Delta\ell = 7.00$ m.

As for the second question, the problem was to decide if the structural member obtained by filling a steel tube with concrete can or cannot be considered equivalent to a reinforced concrete column.

Several publications mention theoretical studies and test made about this problem. However, there is still no complete treatment which allows the establishment of dimensioning rules, which could be used in our case, mainly because the large diameter.

The following criterion was finally adopted: first, calculate the maximum allowable axial load without bending, using the ACI formula for tubular steel columns filled with concrete; then calculate the maximum allowable bending moment, without axial effort, considering the pile as a reinforced concrete column using Jimenez Montoya's ultimate strength charts.

As the reinforcement for this column we computed the complementary inner reinforcing made up by common steel round bars, with a spiral tie, plus the full useful section of the common steel casing (that is, after deducting 5 mm for corrosion allowance).

Thus established the two end points of the interaction curve M - N, this curve should be determined. The linear relationship joining such points can be considered as a lower limit. Taking into account the convex shape of interaction curves for reinforced concrete columns ultimate strength calculations, we considered our curve could follow a quadratic relationship.

Nevertheless, later comparisons with the recommendations of CIDECT (Comité International pour le Développement et l'Etude de la Construction Tubulaire) indicated the convenience of introducing a correction, reducing the allowable moments in the zone of maximum axial load.

Thus, an elliptical interaction curve was adopted, the ellipse's two semiaxes being given values of N_{all} for $M = 0$ and M_{all} for $N = 0$. Where there were high axial force values, the elliptical relationship was replaced by a linear relationship obtained by plotting the line by the two points (N_{max}; $M = 0$) and ($N = 0$; $2 M_{max}$).

One detail of the design that merited special attention was the linking of the pile with the pile cap. In the static scheme adopted the piles are fixed in the pile cap. Normally, this type of fixing is materialized by lengthening the reinforcing of the entire pile and anchoring it into the pile cap. In this case, a very important part of the pile's reinforcing is represented by the casing. However, it was totally impractical to lengthen the casing in the pile cap and anchor it in such a manner that the casing's maximum tension load could be transmitted to the pile cap, because it would have produced considerable interferences with the reinforcings of the pile cap.

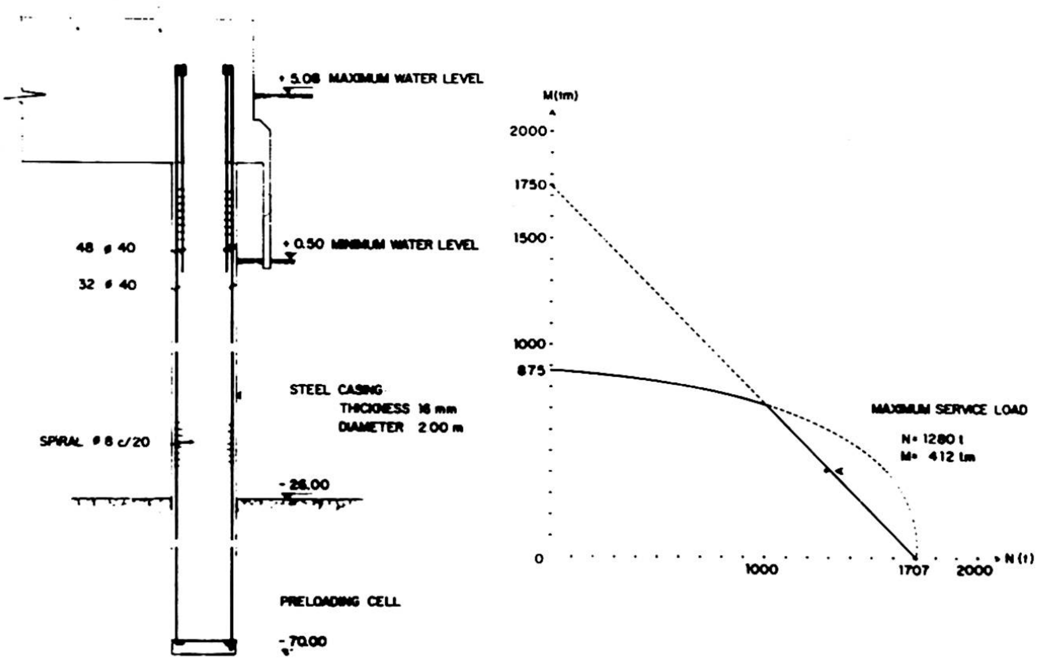

Fig. 3 - Pile Details and Interaction Curve

On the other hand, it must be taken into account that the vertical force on the pile is always compression and that tension could only happen in a very li - mited sector of the casing under exceptional load conditions, and with very low tension stresses. Thus, the idea of anchoring the casing to withstand tension forces was discarded, adopting instead the transference of the compression for ce by installing a collar of thick steel plates that provides a contact surface with the concrete ten times greater than the casing's effective section.

On the contrary, about the complementary inner reinforcing it was decided to lengthen it within the pile cap with a sufficient anchor length. In addition, to parcially compensate the discontinuity that occurs in the casing interruption, the complementary reinforcing in the section near and into the pile cap was increased 50 %.

5 - Piles Construction

The equipment used consisted of a floating barge with two cranes , one with a 75 tons capacity plus an excavating attachment, and the other, used for auxiliary service, with a 25 tons capacity. In addition, a hydraulic claw permits the handling of the steel casings.

After securing the barge in the exact position, a temporary steel casing slightly larger than the pile was lowered, penetrating some 5 meters below the river bed.

Then, the hole is excavated by means of a rotating bucket. During the ex cavation process, both perforation and temporary casing are permanently filled with bentonitic slurry.

Once the perforation reached the prescribed depth, the steel casing, 16 mm thick, is placed. The different parts of the casing are welded together , while they are driven downward, by means of x rays controlled welding. At this point, the temporary casing in the upper part was no longer necessary and it was removed by the claw. Then, the slurry was replaced by clean water and the placing of the pre-loading cell and reinforcing proceeded.

After concreting the pile under water, the job is completed by cutting the casing at the predetermined spot and welding the supporting collar. Finally, the pre-load grouting is performed.

The pre-loading cell consists of a metallic basket filled with crashed stone. After concreting the pile, cement grouting, under pressure of up to 80 kg/cm2, is inserted into the cell by means of a system of tubes and valves.

With this procedure two beneficial effects are obtained for the pile's load capacity: to fill any eventual cavity or area of loose soil that may exist close to the pile tip; and to compress the lower soil with a total load of 2500 tons. Under this load, which approximately doubles the working load, it is reasonable to expect that complete soil settlement is occured in such way that under working loads the pile cap's settlement is due only to the elastic shortening of the pile.

6 - Pile Cap Construction

The majority of the pile caps for main bridges' piers are located in the river bed; thus, the form bottom was supported by a temporary steel structure that rest on provisional brackets welded to the pile casings.

Steel forms are used for the bottom and sides of the pile cap. Concreting is done in several stages planned to simultaneously minimize the shrinkage effects and obtain a good monolithic structure. Thus, the working joints are offset in the different layers and have a serrated surface.

The concrete specified for these pile caps has characteristic strength of 210 kg/cm2, which has been tested with ample margin. The reinforcings use type III steel (cold twisted) with an allowable stress of 2.400 kg/cm2.

SUMMARY

Foundations of two identical bridges of 330 m span over the Parana River include large diameter steel-cased cast-in-place reinforced concrete piles and reinforced concrete pile caps. This paper describes design criteria and construction methods used for these foundations. Due to great diameter and slenderness of piles, special formulae to verify safety conditions were used.

RESUME

Les fondations de deux ponts identiques de 330 m de portée sur la rivière Parana sont composées de pieux de grand diamètre bétonnés sur place dans des tubes en acier sur lesquels reposent les semelles en béton armé. Ce rapport décrit la méthode de calcul et les techniques de construction utilisées. Dû au grand diamètre et à l'élancement remarquable des pieux, des formules spéciales ont été utilisées pour vérifier la sécurité.

ZUSAMMENFASSUNG

Die Fundation der zwei gleichen Schrägseilbrücken über den Parana mit einer Hauptspannweite von 330 m besteht aus Stahlbeton-Pfählen grossen Durchmessers mit einer äusseren mittragenden Stahlhülse und verstärkten Pfahlkopfplatten. Dieser Artikel beschreibt das angewandte Berechnungsverfahren sowie Baumethoden. Wegen des grossen Durchmessers und der grossen Schlankheit der Pfähle, wurden spezielle Formeln angewendet, um die Sicherheit nachzuweisen.

IVc

Humber Suspension Bridge South Tower Caisson Foundations

Fondations en caissons de la tour sud du pont suspendu sur le Humber

Die Senkkasten-Fundation des südlichen Pylons der Humber-Hängebrücke

BERNARD P. WEX
Partner
Freeman Fox & Partners
London England

The Humber Suspension Bridge, which will have the world's longest main span - 1410m - is situated about 300 km north of London, England. On the north side of the river, the bridge foundations presented relatively few problems, being located in chalk. The foundations on the south side, however, are located in Kimmeridge clay overlain by glacial and alluvial material, posing considerable design and construction problems.

Kimmeridge clay is an over-consolidated fissured silty clay. It is hard under the existing over-burden pressure but rapidly disintegrates under the action of water when this pressure is removed. The basis of the foundation design for both the south anchorage and south tower was therefore to minimise the heave of the clay and so keep the fissures closed to prevent the ingress of water.

The Consulting Engineers prepared two alternative designs for the tower pier foundations, one using large diameter bored piles, the other using caissons. Messrs John Howard, the Contractor, who constructed all the bridge foundations, submitted a lower price for the caisson scheme than for the piles.

The tower pier is located about 500m from the south bank of the river in water about 8m deep at high tide. The twin hollow circular caissons, each 24m dia, were sunk through an artificial "sand island". The level of the sand island top was about +4m and the founding level of the caisson was -36m, giving a penetration of about 7m into the Kimmeridge clay.

Caisson sinking was by excavating under water using grabs, since the founding level was too deep to permit the use of compressed air. To control verticality of sinking and prevent base heave, the lower 27m of each caisson was divided into one centre cell and six outer cells. Thus, after de-watering the caissons, bottoming and plugging could be carried out one cell at a time, if necessary, counterweighting by sand fill all but one unplugged cell.

Calculations based on shear box tests on consolidated samples of 10% bentonite suspension, indicated a skin friction of about 20kN/m^2 for sinking the caisson to -25m through sand. "Actual" skin friction values (using the results of divers' surveys to assess the end bearing of the caisson) were estimated as 25-28kN/m^2. A sinking effort of 28kN/m^2 was found to be satisfactory. Calculations estimated average skin friction to be approximately 40kN/m^2 if no bentonite were employed.

In sinking the caissons, considerable difficulty was experienced in the cohesive material. The only satisfactory way of removing the clay from beneath the haunched cutting edge was by means of high-pressure water jets, working at about 30kN/m^2, either lowered down holes in the caisson walls, or held in purpose-built frames in the cells. A sinking effort of 50kN/m^2, including kentledge proved necessary to sink in the Kimmeridge. This high figure was due to partial destruction of the bentonite annulus arising from difficulties encountered in the overlying glacial and alluvial material.

During de-watering and final bottoming, borehole extensometers were used to check the amount of heave below the foundation and give warning (although there was a time lag between the removal of load and full recovery), thus permitting the envisaged sand counterweighting procedure mentioned above to be dispensed with and saving considerable time thereby. After plugging, the caissons were ballasted with water to prevent generalised heave of the foundation area. The water was removed as the external applied load was increased by construction of the concrete tower.

SUMMARY

The foundations for the south tower of the Humber suspension bridge (1410 m main span) are in Kimmeridge clay overlain by glacial and alluvial deposits. Twin hollow caissons 24m dia divided into one centre and six outer cells sunk to a depth of 40m through a sand island in the river by use of grabs, high-pressure water jets and kentledge, the penetration into the clay being 7m. A sinking effort of 50kN/m^2 was required. After plugging the caissons were loaded with water, which was removed as the load of the tower increased during its construction.

RESUME

Les fondations de la tour sud du pont suspendu sur le Humber (portée centrale 1410m) sont fondées dans l'argile de Kimmeridge, située au-dessous des sédiments glaciaires. Des caissons jumelés creux de 24m de diamètre,formés d'une cellule centrale et de six cellules extérieures, étaient enfoncés, dans un îlot de sable construit dans la rivière, jusqu'à une profondeur de 40 m. On a lesté les caissons et excavé à l'intérieur avec des bennes et des jets d'eau à haute pression pour pénétrer de 7m dans l'argile. La pression d'enfoncement nécessaire était de 50 kN/m^2. Pour augmenter le poids, les caissons étaient d'abord bouchés et chargés d'eau. Puis, l'eau était évacuée à fur et à mesure que la construction avançait.

ZUSAMMENFASSUNG

Die Fundamente des südlichen Pylons der Humber Hängebrücke (Mittelöffnung 1410 m) stehen im sog. Kimmeridge Lehm, welcher von Gletschergeröll und alluvialen Anschwemmungen bedeckt ist. Hohle doppelte Betonsenkkästen von 24m Durchmesser, unterteilt in eine Mittel- und sechs Aussenzellen, wurden durch eine Sandinsel im Fluss mit Hilfe eines Greifbaggers, Wasserdruck und zusätzlicher Belastung 40m tief, davon 7m in den Lehm,versenkt. Eine Auflast von 50kN/m^2 war hierzu nötig. Nach Abdichtung der Senkkästen wurden sie mit Wasser gefüllt, welches während der Herstellung des Pylons sukzessive wieder entfernt wurde.

Foundation of a Reinforced Concrete Arch Bridge

Fondations d'un pont en arc, en béton armé

Fundation einer Stahlbeton-Bogenbrücke

ILIJA STOJADINOVIC
Grad.Civ. Eng., Director
Design Bureau, Mostogradnja
Beograd, Yugoslavia

FOUNDATION OF THE REINFORCED CONCRETE ARCH THE SPAN OF 390 M

The bridge connecting the Yugoslav Coast with the Island of Krk, in the Adriatic Sea, consists of two reinforced concrete arches over which a prestressed concrete superstructure is to be installed. The appearance of the bridge is shown in Fig.1.

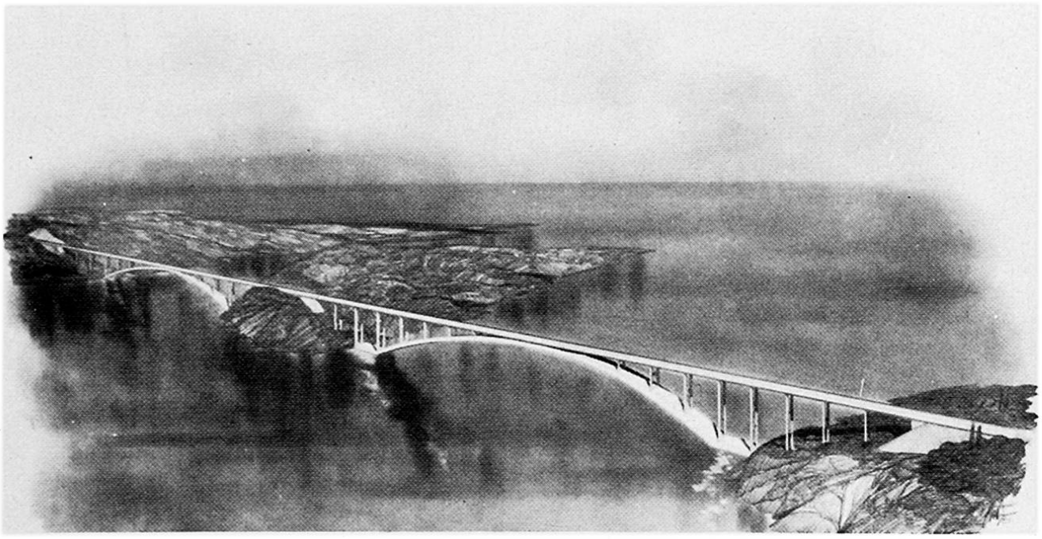

FIG. 1

The reinforced concrete arch over the smaller opening has the span of 244 m, and the arch over the larger one, has 390 m between two supports. As the distance between two shores in this opening is much longer, over 460 m, two arch supports shall be partially founded in the sea, in order of reducing a span, as shown in Fig.2 .

The arch reaction (of approximately 14.000 t) has been distributed to the filled oblique pier founded in the sea (transferring 9.000 t to the rock), and to the nearly horizontal box structure above the sea level that transfers to the coast rock mass the other component of the arch reaction (6.000 t), and the arch bending moment originated from the bridge loading.

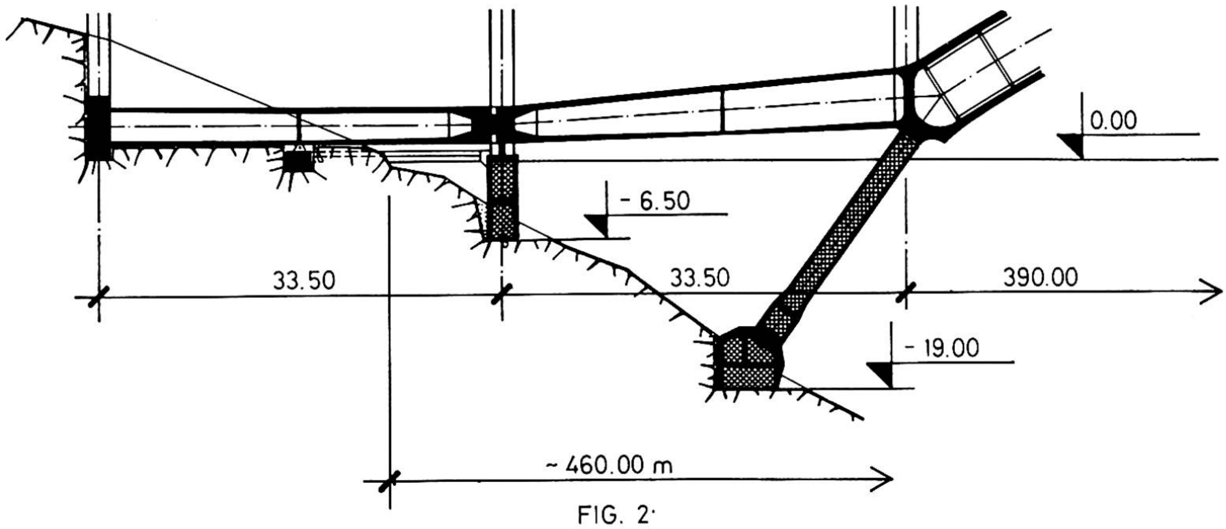

FIG. 2

The bridge construction works are being performed by "The Civil Engineering Enterprise Mostogradnja " from Beograd, Yugoslavia, according to its design.

The process of foundation works shall be revealed through the construction stages.

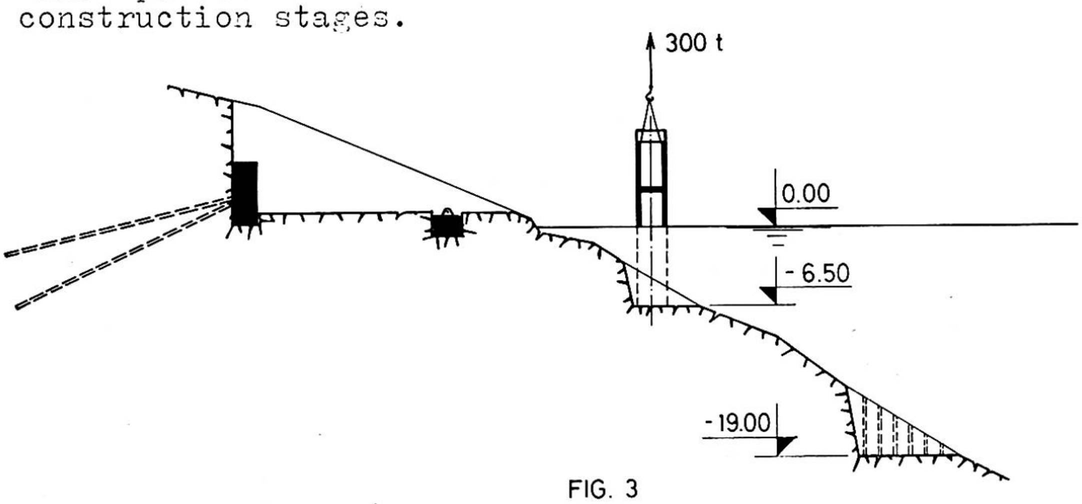

FIG. 3

To mine and excavate the limestone at the places of the arch foundation, was the first step to be executed (see Fig. 3.). The sea boreholes have been prepared operating from a floating device and the explosive installed by the divers. The excavation of material has been executed by the excavators. Finally, a team of divers has accomplished the leveling and clearing operation of the foundation bottom at the level of -6,5o, i.e. -19,oo m under the sea surface, performing corrective mining where it was necessary.

Upon completion of excavation on the shore the anchors shall be fixed into the rock, in the lenght of 2o,oo m. The cables consisting of 24 steel wire cords, ø 7 mm, shall be installed in the bores, diameter of 1o5 mm. The entire anchor force of 3800 t shall be then applied to concreted lateral footing.

Simultaneously to the excavation, affixing the anchors into the rock and cocreting of the foundation onthe shore, there shall be prepared a hollow pier structure devided into the appropriate compartements, with a caisson chamber in the bottom part. Concreting is to be executed at the site auxiliary quay where it shall be transferred from, to the sea, by a floating crane, the bearing capacity of 360 t, and installed in the bottom.

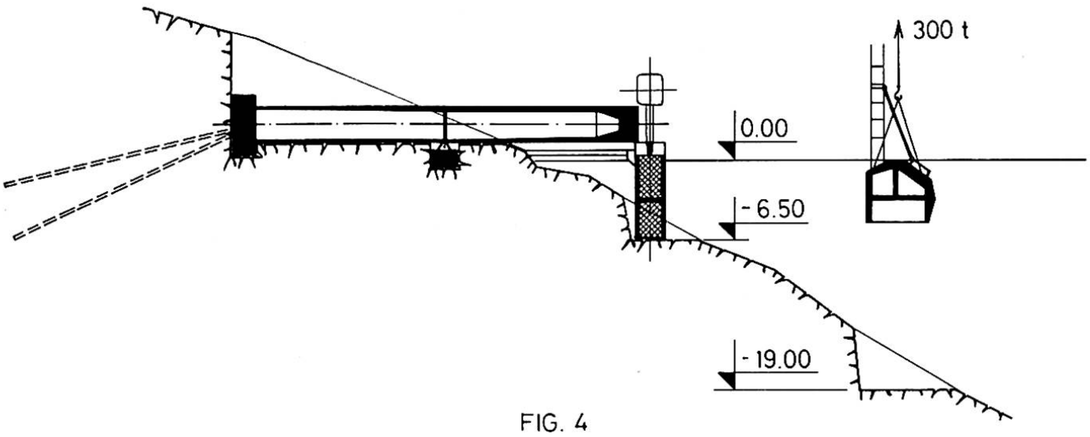

FIG. 4

When placing in its position, the pier shall be connected to the foundation on the shore by a couple of prefabricated concrete ties,thus determining its precise position. Having the protection of compressed air the level of the pier has been regulated by four hydraulic jacks installed in the corners of the caisson chamber.

Concreting of the hollow pier compartments has been planned to be performed upon completion of clearing the soil in the caisson bell (see Fig. 4.).Providing the protection of compressed air,the first step was to concrete in a caisson bell and subsequently the upper part of the pier,performing the concreting operation in waterless space having the water previously exhausted.

Concreting of the horizontal box structure, extending from the pier to the foundation block on the shore, shall be accomplished simultaneously to the above described.This structure shall be prepared, by prestressing,to accept and transfer to the anchors,which are fixed in the rock, the tensile stress occuring in it, prior to assembling of the reinforced cocrete arch.

Together with these works the caisson bells for the arch oblique pier foundation shall be concreted.This shall be performed on a floating platform provided with self-sinking and floating mechanism. The bells should be lowered until the entire body is immersed as their original weight of 300 t, shall then be considerably reduced (up to 140 t, due to existing air in the upper compartments),thus facilitating their transfer and placement by the crane to the previously prepared ground at the level of -19,oo m from the sea surface.

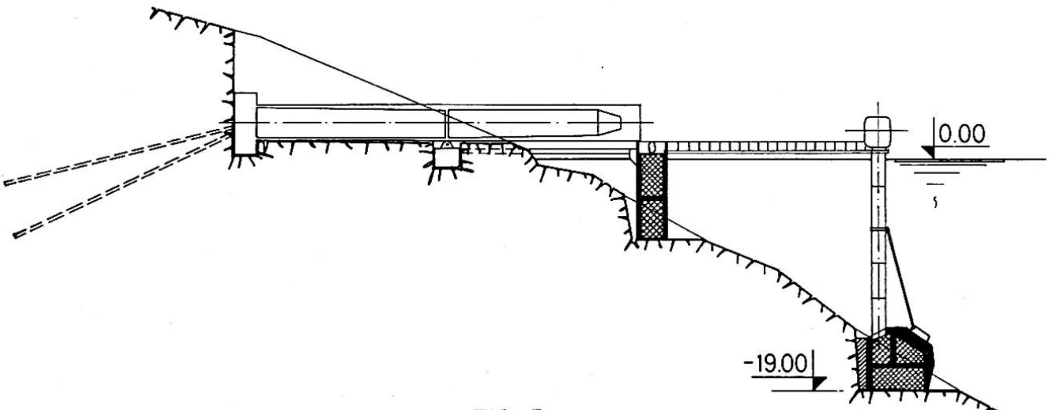

FIG. 5

The caisson bell should be placed on the bottom along with a pair of caisson tubes (see Fig. 5.). These tubes are coupled transversely and their longitudinal position affixed by the obliquely arranged steel bars and by the steel structure of the approaching bridges, thus affixing the position of the caisson bell itself. The foundation level is controlled by the hydraulic jacks installed in the corners of the caisson chamber.

Completion of excavation intended to level and clear the terrain underneath the caisson bell should be followed by concreting of the foundation cavity. The chamber is to be filled with concrete under the protection of compressed air and the upper compartments through the caisson tubes as the water has been previously pumped out from those and the hardening of the concrete in the lower chamber has progressed sufficiently.

The hollowed area next to the caisson, toward the rock mass, should be filled with concrete by the application of the contraction method, for the purpose of confronting with the horizontal component of reaction originated from the arch oblique pier.

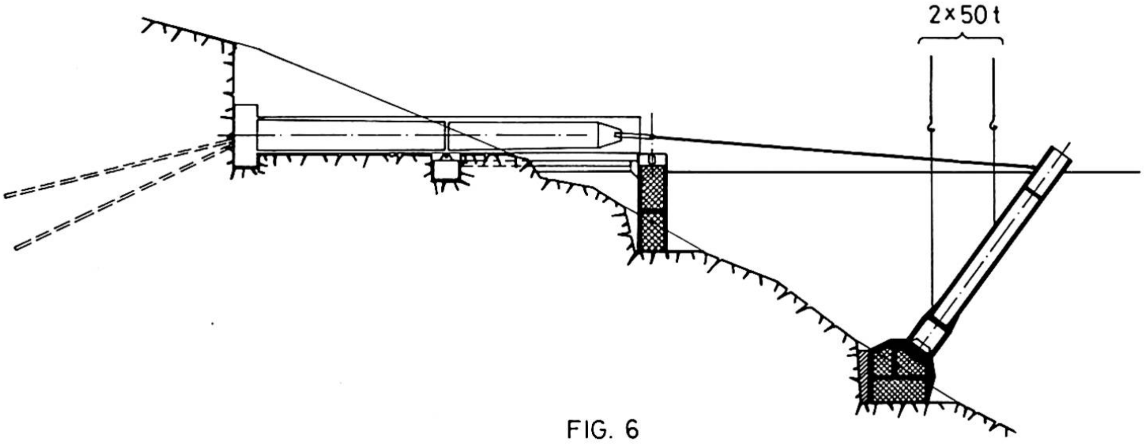

FIG. 6

At the same time the excavation in the caisson bell is being performed, followed by the pouring of concrete into the oblique pier foundation cavity, the oblique pier box structures should be as well concreted, on the same floating platform upon its floating out to the surface. By sinking the platform the oblique piers should be transferred by the floating crane to their determined positions (see Fig. 6.). The cavities in the pier penetrated by air are designed in a manner to reduce its weight, after being immersed, to the rate suitable for the installation operation. Affixing of the oblique pier into,by the design predicted position, should be accomplished attaching the temporary steel ties equiped with the device for the adjustmeht of their length.

Upon sealing the joint of the pier bottom and foundation by the application of the epoxy resin,concreting of the pier hollow body should be commenced in waterless conditions. The chamber next to the foundation is the first to be filled with concrete, and then, one by one, the other compartments after the water has been pumped out.

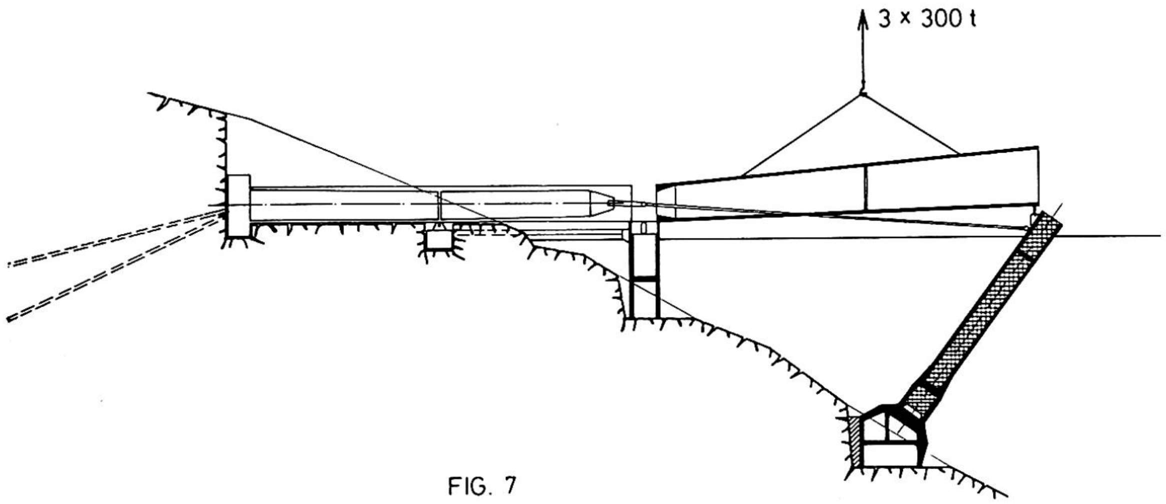

FIG. 7

After that the box structure,prefabricated on the shores in three segments (300 t each), has to be assembled above the arch oblique pier by the floating cranes (see Fig. 7.).Upon completion of assemblage, their transversal and longitudinal joints should be concreted too.

The supporting blocks are the last to be concreted, that is to say the concreting shall be performed above the pier,as well as the joint of the oblique pier and horizontal box structure at the place where the reinforced concrete arch is leaning (see Fig. 8.).

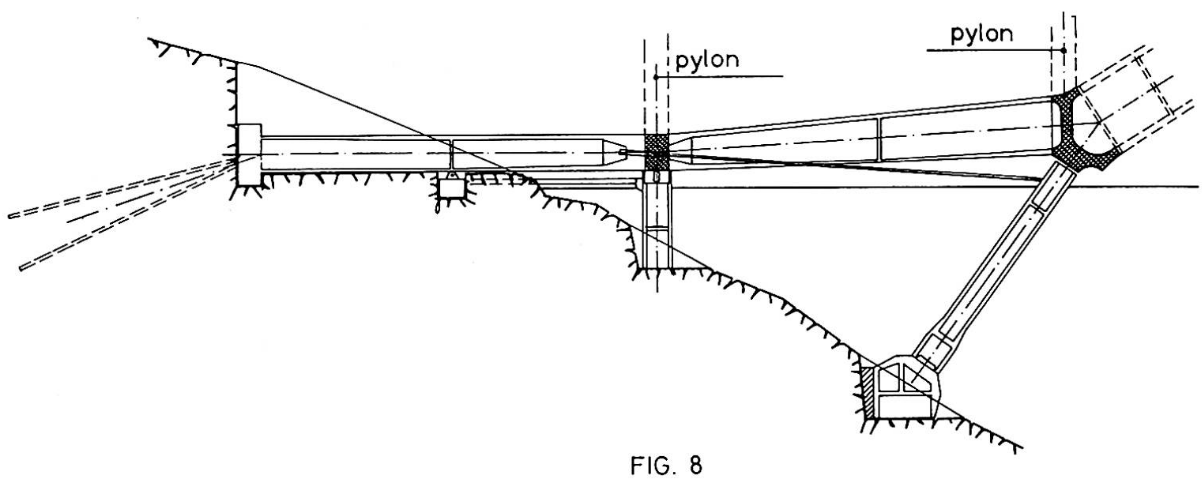

FIG. 8

Further,construction of the object should be continued upon estabilishment of this structure designed for the purpose of transfer of the arch reaction to the ground. Applying a sliding shuttering,the piers determined to sustain the superstructure in the final stage, are to be concreted as the first.In the arch construction they shall serve for suspension of the auxiliary steel ties, which, on the other hand, carry the suspendingly installed arch consisting of prefabricated reinforced concrete elements.

The installation of the arch has been predicted to begin in the early 1977.

SUMMARY

The foundation of a reinforced concrete arch bridge is described. The arch's reaction is transmitted to the ground through a horizontal box structure and an oblique pier, founded 19,00 m below the sea surface. The greatest part of this foundation structure will be composed of prefabricated reinforced concrete elements, which will be placed into the final position by a floating crane of great bearing capacity.

RESUME

Les fondations d'un pont en arc, en béton armé sont décrites. La réaction de l'arc est transmise au sol par l'entremise d'une structure en caisson horizontale et d'une pile oblique fondée à 19 m au dessous du niveau de la mer. La plus grande partie de la structure de fondation est composée d'éléments préfabriqués en béton armé, qui sont mis en place à l'aide d'une grue flottante de grande capacité.

ZUSAMMENFASSUNG

Die Fundation einer Stahlbeton-Bogenbrücke wird beschrieben. Die Bogenreaktion wird durch ein horizontales Kastentragwerk und einen Schrägpfeiler in den Baugrund geleitet. Die Fundamentkonstruktion liegt 19 m unter Meereshöhe. Der grösste Teil der Konstruktion besteht aus vorfabrizierten Stahlbetonelementen, welche mittels eines Schwimmkrans an Ort und Stelle versetzt werden.

Field Observation of Long Span Bridge Foundation Designed on the Results of Models Tests

Mesures in situ sur les fondations d'un pont de grande portée dimensionnées sur la base d'essais sur modèle

Messungen an Fundationen einer weitgespannten Brücke, welche auf Grund von Ergebnissen von Modellversuchen bemessen wurden

S. SUZUKI
Chief

M. ISHIMARU
Project Engineer

Hamana By-Pass Construction Office, Japan Highway Public Corporation
Tokyo, Japan

F. NEMOTO
Manager, Hamana Ohbashi Construction Office
Kajima Corporation
Tokyo, Japan

Y. NOJIRI
Assistant Head, Civil Eng. Dept.
Kajima Institute of Construction Technology
Tokyo, Japan

1. Introduction

The main caissons of HAMANA Bridge was designed on the results of model tests, because the method of designing such a multi-cell caisson had not been established yet, as is stated in the Preliminary Report "Model Test for Design of Long Span Bridge Foundation" written by the authors. The caissons were sunken in March 1974 and the construction of superstructure was completed in August 1976. During the construction, the stress of one of these caissons was measured and compared with the results of model tests in order to confirm an actual state of the caisson designed in this way.

This paper shows the method of measurement and consideration of the result of measurement.

2. Instruments

Strain-gages and reinforcement-stress-gages, which are all Carlson type gages, were embedded in four sections, i.e. 12 strain-gages were embedded in the lower part of the pier to check the load from superstructure; 2 strain-gages and 6 reinforcement-stress-gages, in the top and the bottom of the slab to know its bending; 8 strain-gages, in the upper part of the bulkheads to know the transmission of load. In each section, a null-stress-gage was embedded in order to cancel the effects of creep or drying shrinkage in the measured data.

3. Loads under Consideration

The measurement was conducted for 21 months from the beginning of construction of the pier to the completion of the girder. The loads in this period consisted of the vertical load and the moment load which came through the pier. The vertical load was gradually increasing by the selfweight of pier and girder. The moment load, which was caused by the unbalanced weight of the main and side girders fluctuated according to the stage of construction of the girders.

The maximum values of these loads per one pier were 12,000ton and 27,000 ton-m respectively.

4. Results and Consideration

The measured strains were compared with calculated strains which were obtained by applying the strain-load relation based on the model tests to the

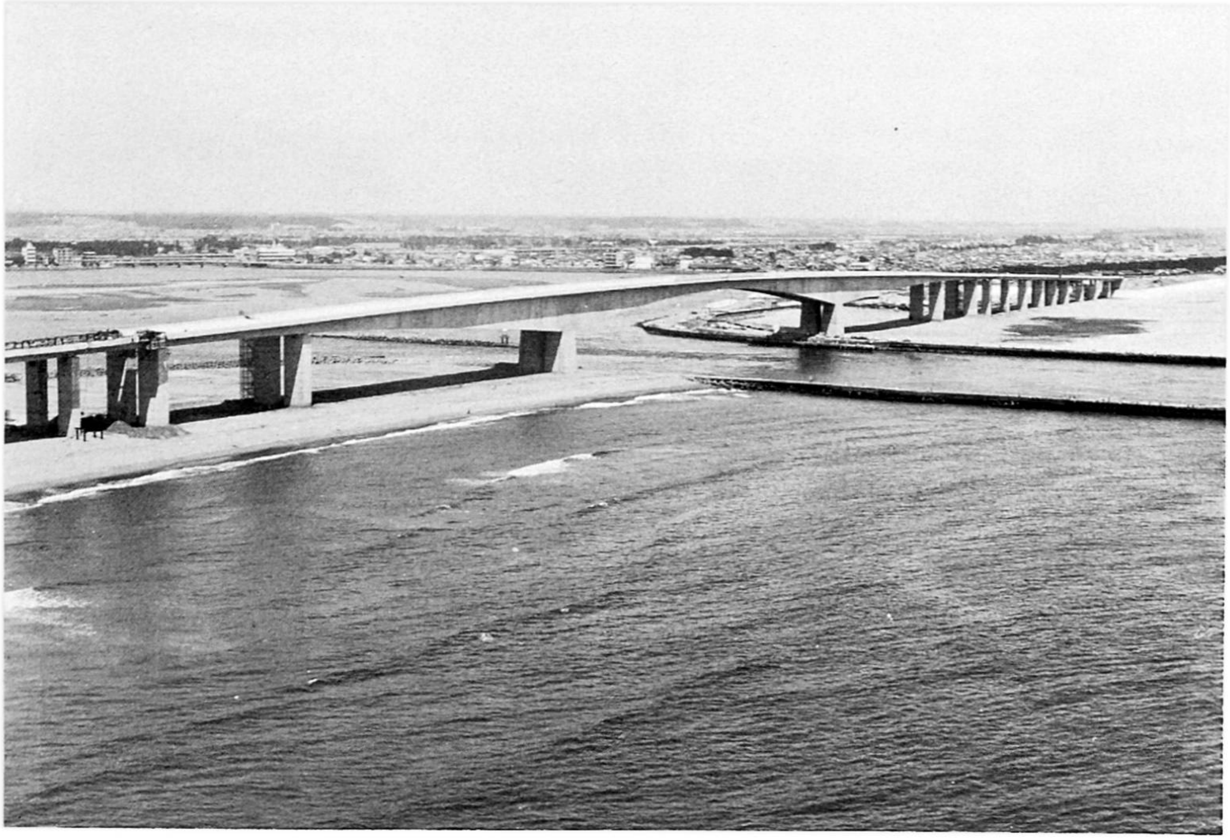

Photo. 1 Nearly Completed HAMANA Bridge

estimated loads depending on the stage of construction.

The strains detected at the pier showed good agreement with the calculated strains. This means that the slab of the casson was loaded by the just load as was estimated. The strains detected at the slab and the bulkheads also showed good agreement. This fact strongly supports the conception which was used in the design.

Considering the state of the slab, which was regarded as of great importance in the design, it can be considered that no tension cracks occured in the concrete during construction, because the maximum tension strain near the lower surface was $70{\sim}90\text{x}10^{-6}$ and less than the general cracking strain of concrete. This also means that the slab will not crack under service loads, because the loads in these two conditions are almost equal.

Through the above study it has been certified that the calculated strains agree with the measured strains unless the slab cracks. But under the design load during earthquake, the slab will crack and its bending stiffness will reduce, therefore, the stress distribution is different from the results of model tests. In order to deal with this effect of cracking, Finite Element Method(F.E.M.) mentioned in the preliminary report had been used. Depending on the result, the reduction of bending stiffness of the slab resulted in slite increase of vertical stress in the bulkheads beneath the pier while the bending moment of the slab decreased. And in the actual design this increase of vertical stress had been estimated with sufficient safety.

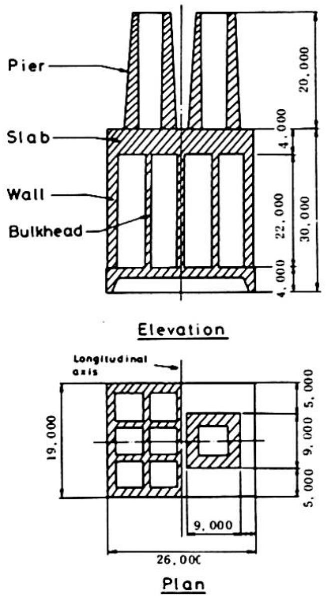

Fig.1 Caisson of HAMANA Bridge

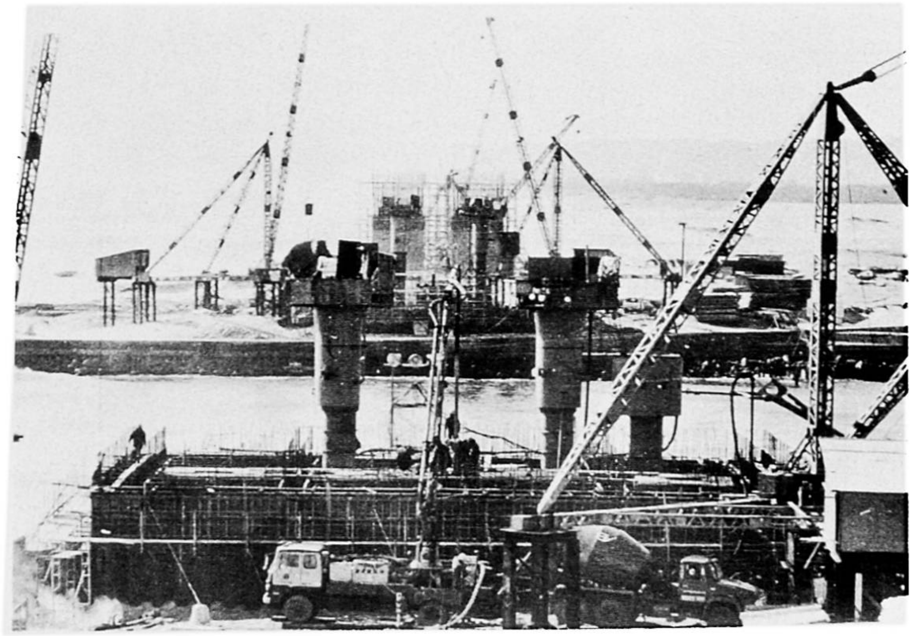

Photo. 2 Caisson under Construction

SUMMARY

After the multi-cell type huge caisson of HAMANA Bridge had been designed on the results of model tests as is stated in the Preliminary Report, the stress measurement during construction was carried out to know the actual state of these caissons. The results of measurement showed good agreement with the designed values and the validity of this design method was certified.

RESUME

Les caissons multicellulaires en béton armé des fondations du pont de HAMANA ont été dimensionnés sur la base d'essais sur modèle (voir Rapport Préliminaire). Les auteurs ont pratiqué une série de mesure de contraintes pendant la construction pour connaitre l'état réel du caisson. Les résultats montrent une bonne correspondance avec les valeurs calculées, ce qui prouve la validité de la méthode de calcul employée.

ZUSAMMENFASSUNG

Nachdem die vielzelligen Senkkästen der HAMANA Brücke auf Grund von Ergebnissen von Modellversuchen bemessen worden waren (siehe Vorbericht), wurden Baustellen-Messungen durchgeführt, um das tatsächliche Verhalten dieser Senkkästen zu untersuchen. Die Ergebnisse der Messungen zeigen gute Uebereinstimmung mit der Berechnung, wodurch die Gültigkeit dieser Bemessungsmethode nachgewiesen werden konnte.

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