Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 42 (1983)

Rubrik: Theme D2: Means of reduction of consequences of collisions

Theme D2

Means of Reduction of Consequences of Collisions

Dispositifs diminuant les conséquences de collisions

Vorbeugemaßnahmen zur Verringerung der Folgen eines Zusammenstoßes

Leere Seite
Blank page
Page vide

Bridge Dolphins Subjected to Impact

Protection de pont soumis à des chocs

Aufprall gegen Brückenabweiser

C. P. HEINS †
Professor
University of Maryland
College Park,
Maryland, USA

Professor Heins received his BSCE from Drexel University, M.S. from Lehigh University, and PH.D. from the University of Maryland. He has been a Visiting Professor to Japan, the People's Republic of China, Syria, Saudi Arabia, Turkey and Korea. Professor Heins has done research on bridge structures and pier protection, and has written a book on the subject. Professor Heins was killed on December 24, 1982, in an airplane accident in China.

This paper was submitted by Professor Heins in October 1982, following a special invitation from members of the Scientific Committee to contribute to this colloquium. When the news of his untimely death reached us during the editorial process, the University of Maryland and his family agreed to our proposal to publish his paper in the Preliminary Report as a posthumous contribution.

SUMMARY
The dynamic response of a circular mooring dolphin is predicted considering soil-structure interaction. A study of the response has resulted in a series of simplified design equations for direct application.

RÉSUMÉ
La réaction dynamique d'une protection circulaire est fonction de l'interaction sol- structure. L'étude aboutit à une série d'équations simplifiées permettant des applications directes.

ZUSAMMENFASSUNG
Die dynamische Reaktion eines runden Anlegeabweisers wird unter Berücksichtigung der Wechselwirkung zwischen Boden und Bauteil vorhergesagt. Eine Untersuchung der Reaktion hat zu einer Reihe vereinfachter Konstruktionsgleichungen zur direkte Anwendung geführt.

1. THEORY

1.1 GENERAL

The dynamic response of a dolphin, supported by a soil medium, requires consideration of the following conditions:

(1) Structure - Soil Interaction

(2) Structure - Fender (shock absorber) Interaction

(3) Failure Criteria

(4) Energy Requirements

All of these conditions were considered in developing the computer mode. The solution of such a system results in the evalution of the dynamic forces applied to the dolphin at any time interval. This force is then applied to the actual dolphin model, which is represented as a cantilever beam on an elastic foundation. The analysis of this beam gives the resulting deformations, shears, moments, and stresses in the dolphin.

1.2 MODEL

The dynamic response of a single degree of freedom (SDOF) system or linear spring system can readily be determined (3). The maximum effects imposed on this spring mass system, when subjected to an initial velocity v , is:

$$a_{max} = -v_0 \lambda \qquad (1)$$

$$y_{man} = v_0 / \lambda \qquad (2)$$

$$\rho_{max} = v_0 \{km\}^{\frac{1}{2}} \qquad (3)$$

where:

a = acceleration

y = displacement

ρ = force

m = mass

k = spring constant

$\lambda = \{k/m\}^{\frac{1}{2}}$

In evaluating the dolphin-vessel impact response, it will be assumed that the SDOF system can simulate such interaction. The spring mass (m) will represent the ship weight, and the spring constant (k), will represent the dolphin stiffness. The initial velocity (v_0) of the spring mass respresents the impact velocity of the ship, which has a mass (m). The spring constant (k) represents the stiffness of the dolphin-soil system.

In order however to determine the initial acceleration and deformation of such a system, a preditor-corrector scheme is utilized (1). Utilization of such a method permits rapid determination of the initial deformations and accelerations during initial impact.

The spring constant is determined by applying a unit load to the dolphin-soil system, which gives:

$$k = 1/\Delta$$

This constant is then used to determine λ, and then the resulting acceleration, displacement and force can be determined. The process is then repeated for subsequent time intervals.

Finally the end force ρ max is applied to the end of the elastically supported dolphin. Resulting internal moments and shears are then determined.

The entire scheme has been computerized, in order to expedite the analysis. This computer program was then used to determine the general response of various dolphins. The parameters of these dolphins will now be described.

2. PARAMETRIC STUDY

2.1 TYPE OF DOLPHIN

The type of dolphin to be considered in this study consists of a single steel cell, filled with gravel and earth, as shown in Figure 1. The range in parameters that were considered are as follows:

2.2 PARAMETERS

Single Cell Dolphin

D - Diameter (m); 6.1, 9.1, 12.2, 15.2
t - Thickness (cm); 2.54, 5.08
+M.L. - Length of dolphin above mud line (m); 6.1, 12.2, 18.3
-M.L. - Length of dolphin below mud line (m); 6.1, 12.2, 18.3
R = (-M.L.)/(+M.L.); 1.0, 1.5, 2.0
k_s = Soil Modulus (N/cm^3); 27.1, 54.3, 135.7. 271.5

The soil modulus (k_s) has been selected (4) such that the soil type represents soft clay (k_s = 27.1 N/cm^3) to dense sandy gravel (k_s = 271.5 N/cm^3), as given in Table 1.

The ship parameters consisted of velocity v_0 and weight (w). The velocity was held constant at 1.0 knots. The ship weight (tons) was then increased accordingly until failure was instituted in the system, in order to find the maximum strength of the system.

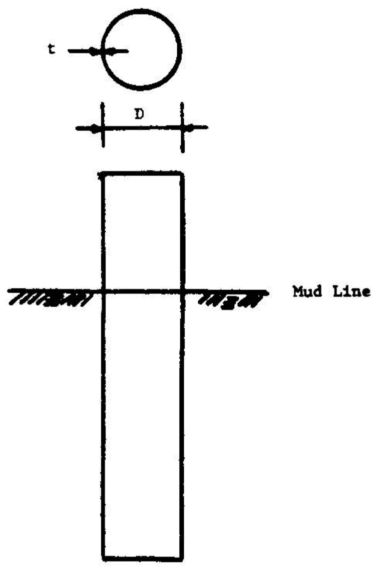


Fig. 1 Single steel cell dolphin

Soil	k_s
Sense sandy gravel	219.9 - 393.6
Medium dense coarse sand	157.5 - 314.9
Medium sand	111.3 - 282.3
Fine or silty, fine sand	78.7 - 187.3
Stiff clay (wet)	54.3 - 219.9
Stiff clay (saturated)	27.1 - 111.3
Medium clay (wet)	38.0 - 141.2
Medium clay (saturated)	10.9 - 81.4
Soft clay	1.6 - 38.0

Table 1 Soil modulus k_s (N/cm^3)

2.3 RESULTS

Typical results for the dolphin response are given in Figures 2 through 7. These results are all for a constant ship velocity of v_0 = 1 knot. Figures 2 and 3 show the resulting dolphin spring constant k_d as a function of dolphin parameters. These figures are useful in design, as they indicate the magnitude of k_d, which is required in the simplified design approach.

Figures 4 through 7 show the relationships between the ship weight (tons) and the soil modulus k_s. The parameter k_f represents the stiffness of a fender or shock absorber which may be attached to the dolphin. The unsafe region of these curves indicates that soil failure will occur. These failure curves are valid for all the cell parameters given previously.

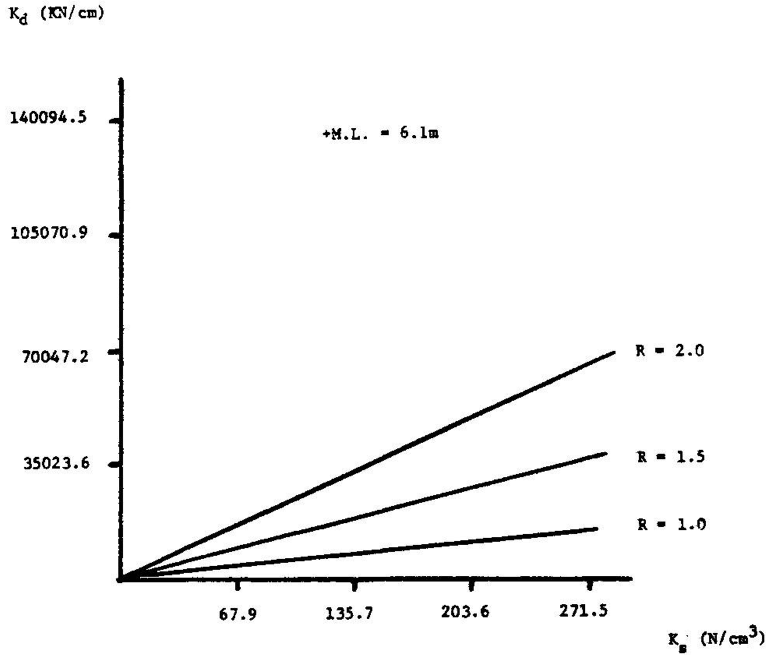


Fig. 2

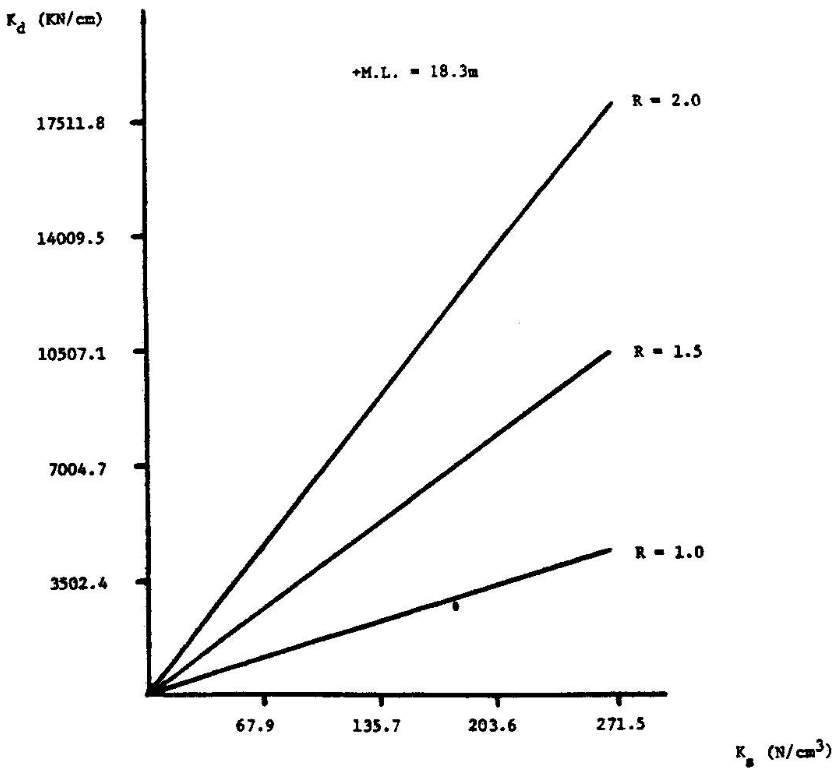


Fig. 3

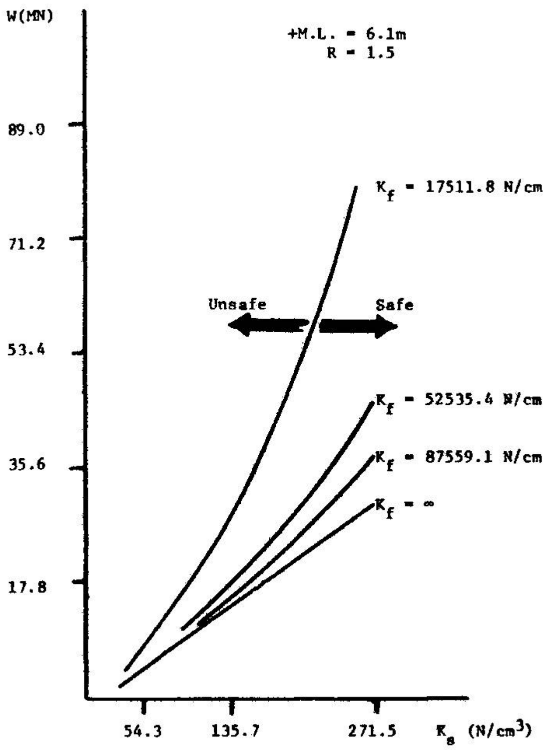


Fig. 4

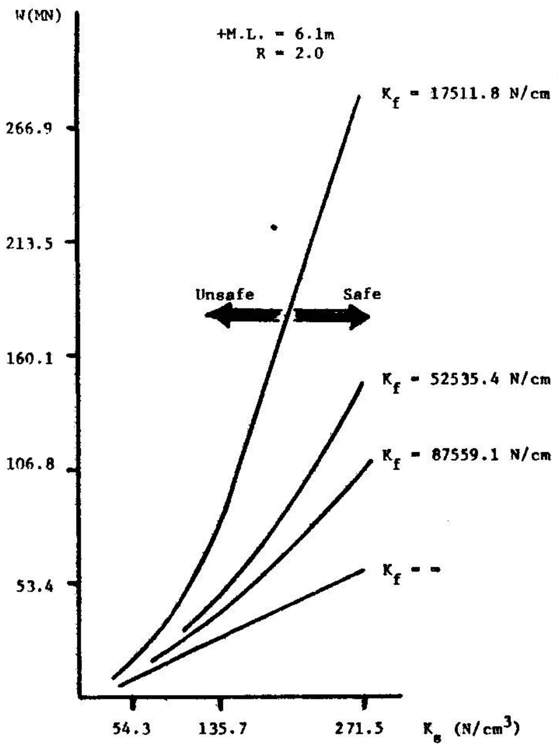


Fig. 5

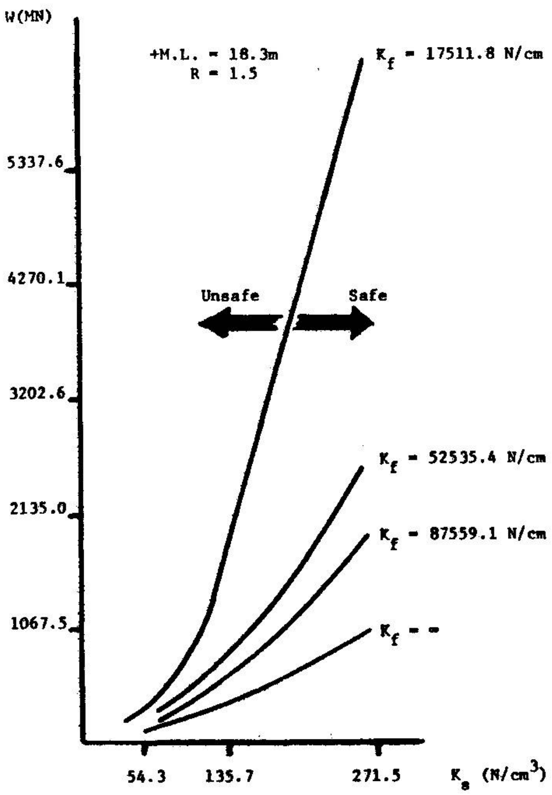


Fig. 6

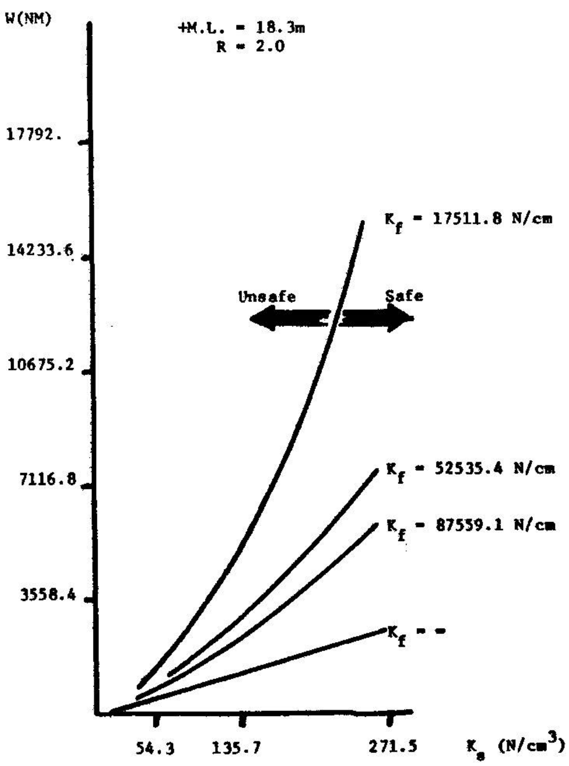


Fig. 7

3. DESIGN CRITERIA

The maximum load or ship weight that can be applied to the dolphin is governed by the strength of the soil or strength of the cell. The soil will fail when the strain of the soil reaches a maximum. The cell will fail either due to development of a maximum bending or shear stress or a combination. Examination of the failure mode of either the cell dolphins or clusters, indicates that the bending mode will not govern and therefore only soil or cell shear failure will be examined (1).

3.1 CELL DOLPHIN

As given previously by Eqn. (3); $\rho = v_0 \, (km)^{1/2}$, however the maximum shear V developed in the cell is equal to ρ therefore;

$$V = v_0 \, (km)^{1/2} \tag{5}$$

however the shear stress is governed by

$$\tau = VQ/It \tag{6}$$

where:

$I = D^3t$, $Q = D^2t$. Therefore,

$$\tau = V/Dt \tag{7}$$

and substituting in Equation (5) gives;

$$\tau = C \frac{v_0 (km)^{1/2}}{Dt} \tag{8}$$

where C is a constant, which was determined from the computer analysis. Examination of these analyses has resulted in a value of C = 0.732. Therefore, the final design equation for the cellular dolphin is;

$$\tau = 0.732 \, v_0 \frac{(km)^{1/2}}{Dt} \tag{9}$$

if soil failure does not govern.

4. CONCLUSIONS

Using a computer oriented model, the dynamic response of a series of cellular dolphins have been examined. These results have permitted the development of a series of design equations and charts, which will permit rapid design/analysis of such dolphins when subjected to vessel impact. A complete set of design curves are available in Ref (1).

REFERENCES

1. Chiu, L.Y.B., "Analysis of Dolphins Subjected to Vessel Impact," M.S. thesis, University of Maryland, College Park, MD, December 1980.

2. Derucher, K.N., Heins, C.P., "Design of Bridge Protective Systems and Devices," M. Decker, New York, NY, 1978.

3. McCallion, H., "Vibrations of Linear Mechanical Systems," J. Wiley & Sons, New York, NY, 1973.

4. Bowles, J.E., "Foundation Analysis and Design", McGraw Hill Book Co., New York, NY, 1972.

NOTATIONS

A	Cross Sectional Area of Dolphin
D	Diameter of Dolphins
I	Moment of Inertia
k	Spring Constant
k_d	Dolphin Stiffness
k_f	Fender Stiffness
k_s	Subgrade Modulus of Soil
R	-M.L./+M.L.
V	Shear
w	Vessel Weight
+M.L.	Pile Length above Mud Line
-M.L.	Pile Length below Mud Line
a	Acceleration
M	Mass
t	Thickness of Cell
v_0	Initial Velocity of Vessel
y	Displacement
λ	Natural Frequency
τ	Shear Stress

Leere Seite
Blank page
Page vide

Ecran de protection des ouvrages en mer contre les collisions

Kollisionsschutz für »Offshore«-Bauten

Anticollision Screen for the Protection of Offshore Structures

André VITALIS
Ing. Génie Civil Marine
Paris, France

André Vitalis, né en 1917. Ingénieur A. et M. Paris. Ingénieur Ecole Navale Brest. Etude et réalisation d'ouvrages maritimes et portuaires (15 ans). Engineering offshore pour plates-formes béton en mer du Nord (5 ans). Actuellement Ingénieur-Conseil pour l'engineering et la pose des rejets d'eau des centrales nucléaires côtières.

RÉSUMÉ
Le but de l'équipement étudié est de protéger les ouvrages en mer, fixes ou flottants, contre les collisions de corps dérivants. Le principe de cet écran est d'agir comme un amortisseur qui freine la course du corps flottant en absorbant graduellement l'énergie cinétique accumulée par celui-ci. Il peut être utilisé pour la protection de structures en mer ou côtières contre l'impact de corps flottants de taille importante, dérivant à vitesse appréciable.

ZUSAMMENFASSUNG
Die betreffende Ausrüstung dient dazu, die unbeweglichen oder treibenden Bauwerke auf offener See gegen den Zusammenstoß mit treibenden Körpern zu schützen. Diese Schutzwand wirkt hauptsächlich als ein Stoßdämpfer, der den Lauf des treibenden Körpers bremsen soll, indem er die kinetische Energie, die dieser aufgespeichert hat, nach und nach aufnimmt. Sie kann dazu dienen, die Meeres- oder Küstenanlagen gegen den Zusammenstoß mit größten, mit in berechenbarer Geschwindigkeit treibenden Körpern zu schützen.

SUMMARY
The purpose of the shock absorber screen is to protect offshore structures, fixed or anchored, against collision with drifting bodies. The principle of this device is to act as an energy absorber and to stop the drifting vessel by developing a damping energy sufficient to cancel the kinetic energy of the ship. It can be utilized for protection of any kind of offshore or coastal structure against collision with largest size of drifting bodies at appreciable speed.

1. GENERALITES

Le système de protection des structures en mer dont il est question vient tout naturellement en réponse aux préoccupations des experts dont les études font apparaître que les possibilités de désastres par collision sont loin d'être négligeables et que d'autre part, la faculté d'absorption d'énergie des plates-formes offshore est très faible.

Ces plates-formes ne peuvent en effet supporter, sans dommage trop sérieux, que l'impact de collision d'un petit navire de 2.000t à 2,5 noeuds (ce qui correspond à une énergie de 2.350KJ, compte tenu des masses d'eau entrainées).

Il est donc hors de question d'envisager de concevoir des structures qui soient capables d'amortir l'énergie cinétique développée par un navire de 400.000t dérivant à 3 noeuds (soit une énergie de 715.000KJ).

Si l'on veut se prémunir contre l'impact d'un corps flottant à la dérive dont l'énergie dépasse 2.000KJ, il faut donc prévoir un système extérieur à l'ouvrage à protéger et qui lui soit indépendant. C'est l'objet de la présente communication.

En fait, le principe est connu, c'est celui qui est appliqué à l'apontage des avions sur les porte-avions. L'énergie étant dissipée par l'effet conjugué d'un effort et d'un déplacement. L'effort est repris par un câble d'acier très peu élastique et le déplacement est assuré par l'action de vérins hydrauliques. L'énergie absorbée est d'environ 60.000KJ, ce qui, si le même système pouvait être appliqué en mer, correspondrait à l'impact d'un navire de 36.000t lancé à 3 noeuds.

2. DESCRIPTION

Le dispositif présenté repose sur le même principe mais ses constituants sont conçus de manière à ce qu'il puisse être opérationnel en mer tout en ne nécessitant qu'un minimum d'entretien.

2.1 Principe du système

Pendant son action le système permet de développer une puissance d'amortissement suffisante pour annuler l'énergie cinétique du corps flottant en mouvement, stoppant ainsi sa course à une distance raisonnable de la zone à protéger.

Cette fonction est assurée grâce aux propriétés élastiques des aussières en nylon, qui peuvent supporter de grandes charges, tout en conservant un bon coefficient d'élasticité. Ainsi, en cas de collision, l'énergie développée par la déformation des lignes d'ancrage en nylon permet de stopper le navire.

2.2 Arrangement général

Les principaux constituants du système de protection (voir figures 1 et 2) sont :

- un écran flottant de forme polygonale, entourant partiellement ou entièrement le site à protéger. Chaque côté de l'écran comporte plusieurs câbles, tendus entre 2 flotteurs principaux situés à chaque extrémité, et maintenus au niveau de l'eau par quelques flotteurs intermédiaires;
- des lignes d'ancrage, constituées de câbles acier et nylon,connectées aux flotteurs principaux. L'ancrage proprement dit est effectué par

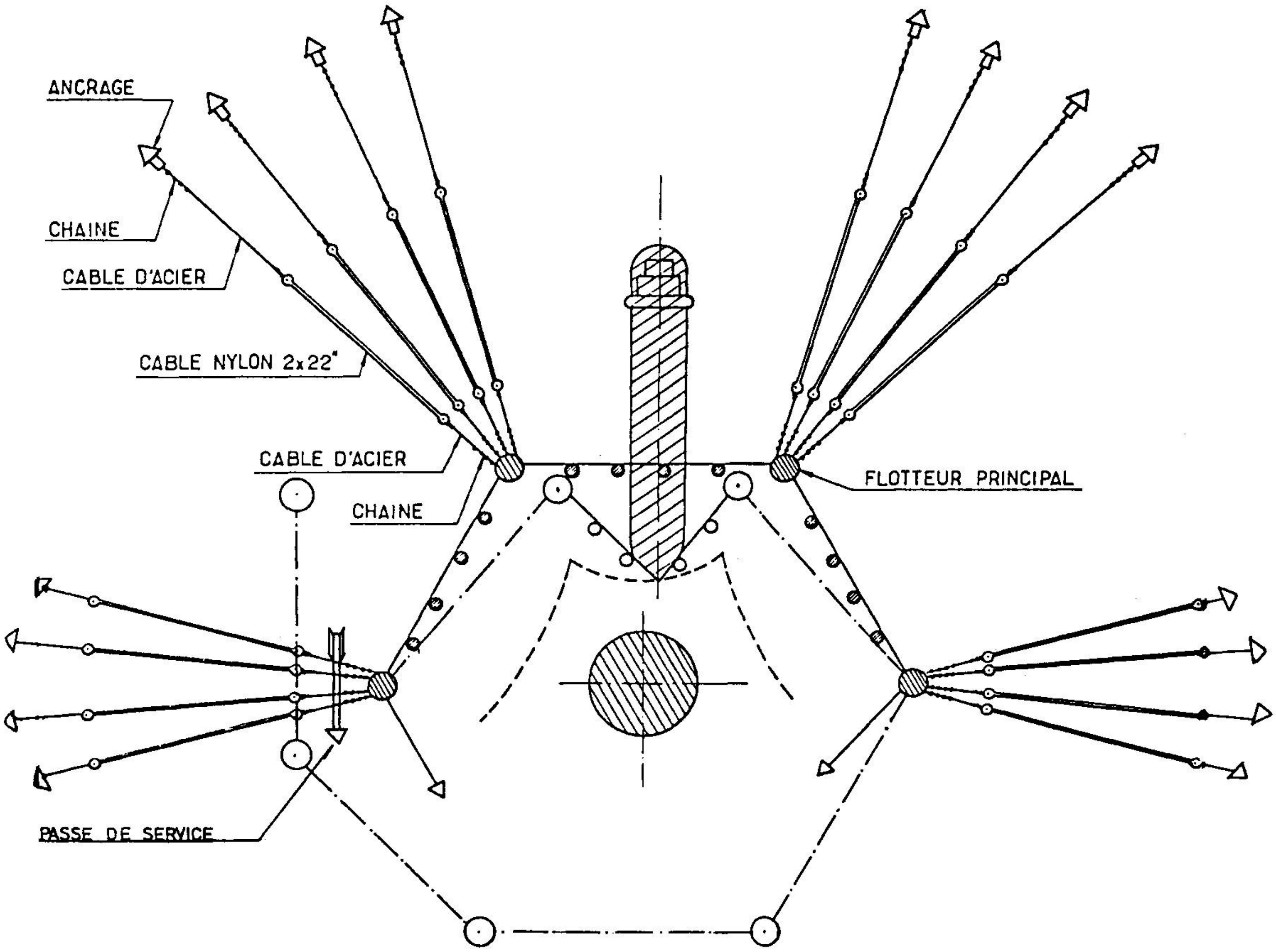


Fig. 1 Ecran de protection (vue en plan).

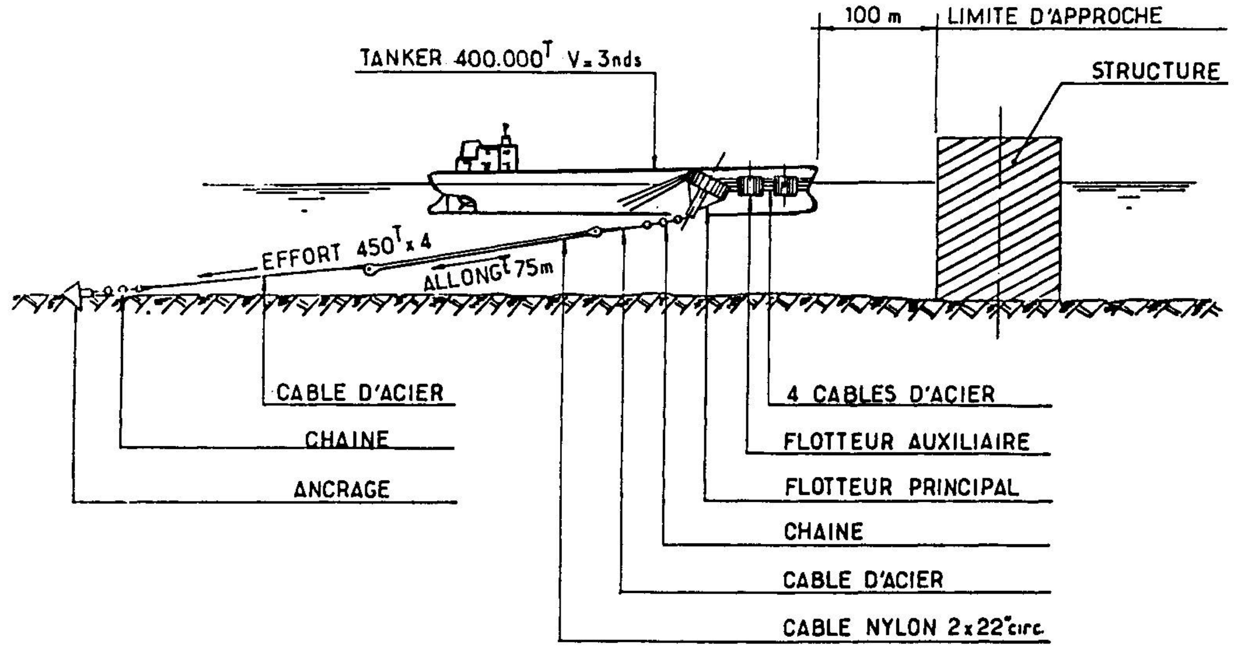


Fig. 2 Ecran de protection (élévation).

ancres ou par pieux battus, suivant la nature du terrain et l'effort de retenue demandé. Ces lignes d'ancrage sont mises en tension, de manière à maintenir le système polygonal en équilibre.

2.3 Action de l'écran

Quand un navire dérive sur le système de protection, il déforme l'écran polygonal, entraînant avec lui les flotteurs principaux, ce qui tend les lignes d'ancrage en nylon. Le navire est ainsi stoppé par l'énergie emmagasinée, puis repoussé jusqu'à ce que l'écran de protection retrouve un nouvel état d'équilibre. Le système retient ainsi le navire en difficulté, l'empêchant de récidiver, jusqu'à ce qu'il se soit dégagé de lui-même, ou aidé par un navire de secours.

2.4 Capacité du système

Les études entreprises sur le sujet ont permis de déterminer les dimensionnements respectifs des composants. Pour permettre une action efficace et une réutilisation immédiate de l'écran, un facteur de sécurité de 2, par rapport aux taux de travail maximal, a été pris en compte, qui s'applique :

- sur le dimensionnement de tous les constituants, spécialement les lignes d'ancrage nylon et les câbles de l'écran horizontal ;
- sur la réserve de flottabilité :
  - des flotteurs principaux, soumis aux réactions verticales de lignes d'ancrage en nylon ;
  - des flotteurs intermédiaires, sur les quels le navire à la dérive peut exercer une action verticale et qui doivent donc toujours maintenir l'écran horizontal en surface ;
- sur la zone effective à protéger, c'est-à-dire que la déformation maximale de l'écran polygonal ne doit pas excéder la moitié de la distance initiale entre le système de protection et la structure à préserver.

Les études effectuées à ce jour ont porté principalement sur un écran de protection de gros gabarit, le but visé étant de stopper un navire de 400.000 DWT dérivant à 3 noeuds. Le dimensionnement d'un tel système peut paraître énorme à priori, et le coût assez élevé, mais il est évident que pour des protections moins ambitieuses, les structures finales seront beaucoup plus légères, et de moindre coût.

Le comportement du système de protection soumis à un choc a été étudié sur ordinateur grâce à un programme de simulation qui détermine la déformation maximale de l'écran polygonal. Ce programme calcule dans un premier temps la loi de comportement des lignes d'ancrage en nylon, et applique ces résultats à une série de configurations de l'écran, chacune représentant un choc possible. On obtient ainsi les déformations maximales du système de protection pour des collisions venant de toute incidence, l'enveloppe de celles-ci délimitant la zone effectivement protégée. [1]

Outre la représentation de l'écran de protection soumis à un choc, le programme donne, pour chaque cas considéré, les taux de travail de câbles en nylon et des côtés horizontaux, ainsi que l'énergie emmagasinée par chaque ligne d'ancrage. Cela a permis d'affiner le dimensionnement de chaque composant, pour arriver à la capacité requise, en respectant partout le coefficient de sécurité 2.

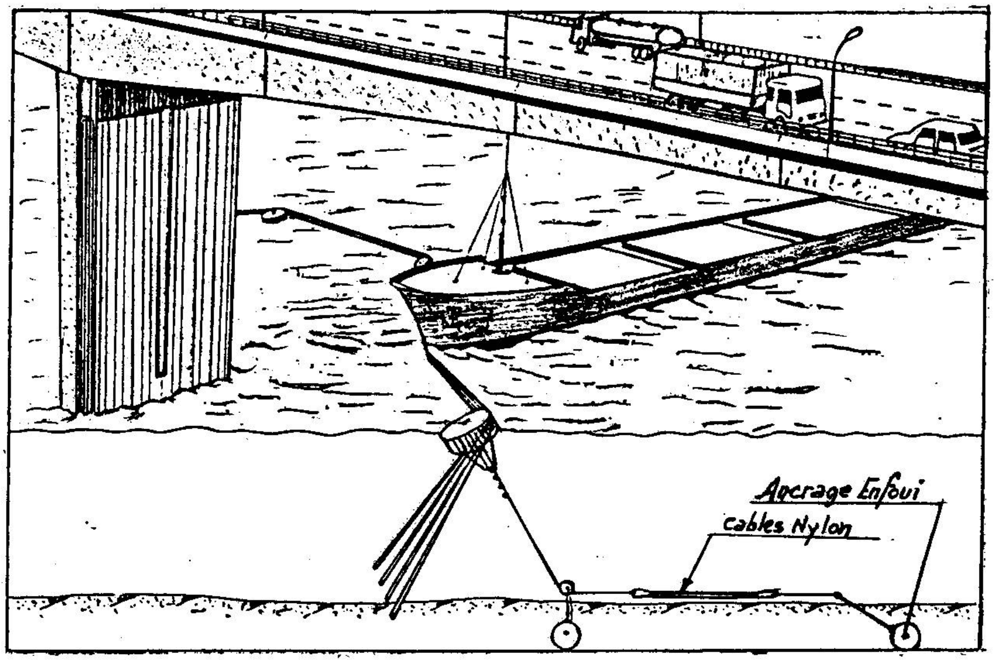


Fig. 3 Protection de piles de ponts.

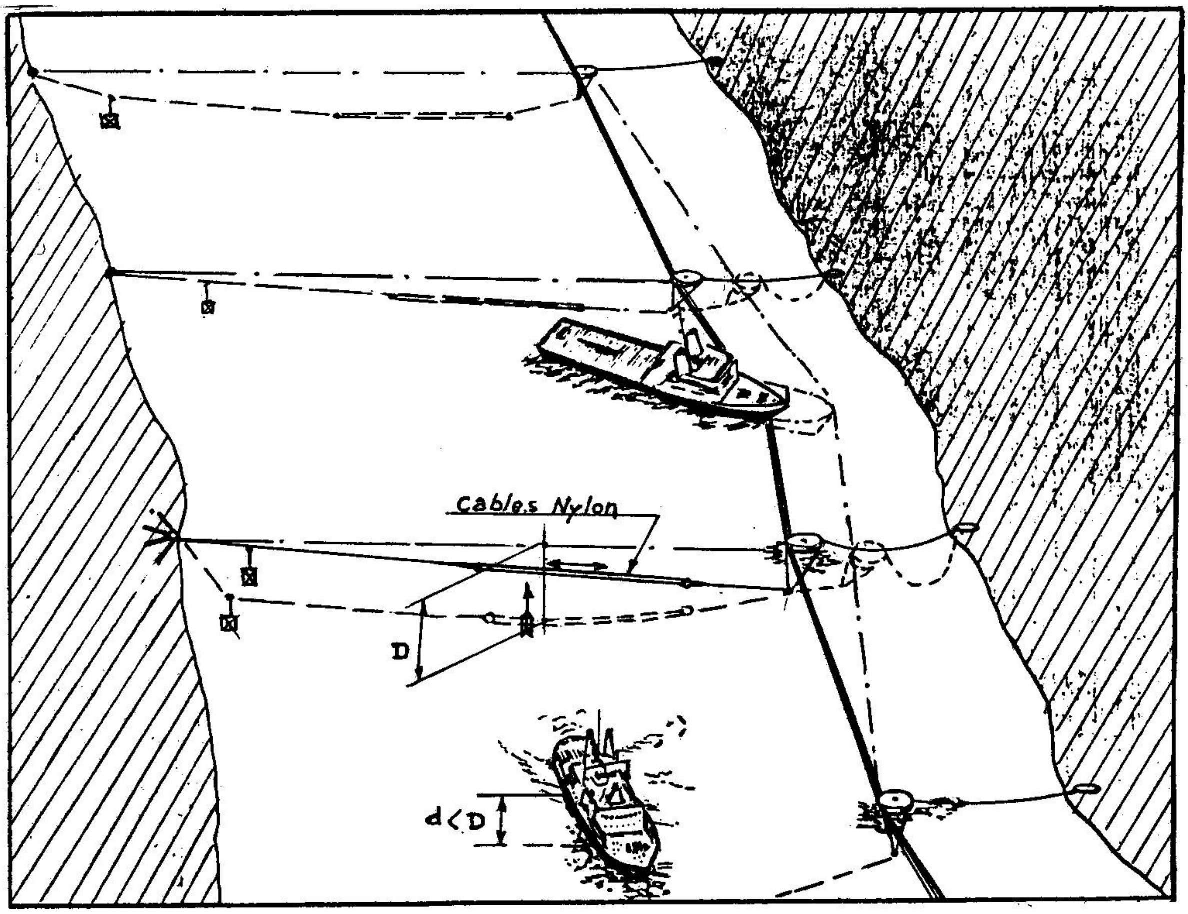


Fig. 4 Navigation dans les passes étroites (Fjords)

C'est l'effort de traction admissible sur le câble de nylon (qui peut être important) ainsi que son allongement sous cet effort (qui est de 25%) qui déterminent la capacité du système.

A titre d'exemple, un câble de nylon de 21" de circonférence possède un effort de rupture de 5.500 KN.

Quatre câbles, en double, de 300m de longueur, disposés sur l'ancrage d'un flotteur principal sont capables d'absorber une énergie de 870.000KJ avec un coefficient de sécurité de 2.

On voit donc bien qu'un tel système est apte à retenir le navire de 400.000t lancé à 3 noeuds dont nous avons parlé puisque l'énergie qu'il développe n'est que de 715.000KJ.

Il va de soi, par ailleurs, que le système polygonal est maintenu en équilibre au repos par 2 mouillages auxiliaires munis d'un système de tensionnement.

3. CONCLUSIONS

En conclusion, ce système dont la forme et la conception peuvent varier en fonction des applications envisagées, peut parfaitement convenir :

- à la protection de sites côtiers ou de structures offshore situés en bordure de voies maritimes très fréquentées.
- à la protection de musoirs de ports et de toute avancée artificielle en mer.
- à la délimitation des chenaux d'accès et des zones d'évitage de navires.
- à la protection contre les icebergs à la dérive de structures offshore, opérant en zone arctique.
- à la protection de piles de pont dans un estuaire ou une rivière.

Il faut noter en outre que l'originalité de ce système réside en ce que :

- il peut stopper un navire sur une courte distance sans l'endommager.
- il reste opérationnel immédiatement après une intervention sans remise en état préalable.
- il est adaptable à toute profondeur d'eau du site.

Deux exemples de variante du projet sont illustrés par des croquis d'artiste : (fig. 3 et 4)

A. Navigation dans les passes étroites (fjords).
B. Protection de piles de pont.

REFERENCES

1. VITALIS A., Ecran de protection anti-collision - Proceeding of the International Symposium on offshore structures held at COPPE, Federal University of RIO DE JANEIRO, BRASIL - October 1979 p. 4.129 -

Analytical Technique for Ship-Fender Interaction
Méthode d'analyse de l'interaction navire-défense
Analytisches Verfahren zur Untersuchung der Wechselwirkung von Schiffsstoßfängersystemen

Chen-Wen JIANG
Senior Ocean Engineer
Giannotti & Assoc., Inc.
Arlington, VA, USA

Chen-Wen Jiang, born i 1946, received his PhD from MIT, Cambridge, MA, He has been involved in R&D in the areas of dynamics and hydrodynamics of ocean vehicles.

Richard C. JANAVA
Department Head,
Giannotti & Assoc. Inc.
Arlington, VA, USA

Richard Janava, born in 1942, received his MS from Brooklyn Polytechnic Institute, NY. He has been involved in the areas of static and dynamic analyses of ship structures, shock and vibration, survivability and fender performance.

SUMMARY
The design and selection of appropriate fender systems must consider the dynamic interaction between the ship and port or pier structure. An analytical technique is developed to study the dynamic interaction of marine fender systems expressed by generic algorithms with ship's hull structure. The frequency dependence of the hydrodynamic coefficients is considered in the form of a simplified convolution integral. The calculated values presented are compared with existing experimental and theoretical results.

RÉSUMÉ
La conception et le choix d'infrastructures appropriées de défense doivent prendre en considération l'interaction dynamique entre le navire et la structure du port ou des piliers. Une méthode d'analyse est ainsi mise au point pour étudier l'interaction dynamique de systèmes de défenses maritimes, exprimés à l'aide d'algorithmes génériques, avec la carène d'un navire. La dépendance de fréquence des coefficients hydrodynamiques est envisagée sous la forme d'une intégrale simplifiée d'enroulement. Les valeurs calculées présentées ici sont comparées aux conclusions d'expériences et aux résultats théoriques déjà connus.

ZUSAMMENFASSUNG
Bei der Konstruktion und der Auswahl geeigneter Stoßfängersysteme muß die dynamische Wechselwirkung zwischen Schiffen und Hafen- oder Pfeilerstrukturen berücksichtigt werden. Es wird ein analytisches Verfahren zur Untersuchung der dynamischen Wechselwirkung von Stoßfängersystemen in der Schiffahrt entwickelt,das sich durch artmäßige Algorithmen der Schiffsaußenwände ausdrückt. Die Frequenzabhängigkeit von den hydrodynamischen Koeffizienten wird in Form eines vereinfachten Faltungsintegrals berücksichtigt. Die dargestellten, errechneten Werte werden mit vorhandenen experimentellen und theoretischen Ergebnissen verglichen.

1. INTRODUCTION

In order to achieve more appropriately designed marine fender systems, an understanding of the vessel-fender dynamic interaction is essential. The dynamic analysis can describe more accurately the fender's energy absorption characteristics and operational performance requirements.

There are several approaches which consider the total energy of a berthing ship to be absorbed by the fender/pier system in addition to the magnitude of the fender reaction force generated. A simple and commonly used approach is the energy method; e.g., Lee [8], Brolsma et.al. [1]. Accurate predictions using this method involve knowledge of hydrodynamic coefficients and system stiffnesses for the total system. Empirical values are commonly used in this method. System damping has been ignored in this approach. Alternate statistical approaches consider information obtained from model measurements to determine the design energy value of the berthing impact and fender energy absorption; e.g., Svendsen [10]. With the design risk selected, the design value of the energy to be absorbed by the fender system can be determined. The disadvantage of this method is in obtaining the necessary statistical information for problem solution. Kim [5] proposes an approximate and simpler method. The idea is based on the use of time average of the kinetic energy of the berthing ship during the time interval of fender compression. This method is applicable only to the ship in calm water and berthing broadside.

The time domain solutions of forces and motions have been developed by van Oortmerssen [12] and Fontijn [3]. Both of these methods use Impulse Response Function techniques. It is easy to adopt different external forces and some other factors in the time domain method. The procedure presented herein is based upon a simplified convolution integral for added mass and damping calculations, which eliminates the disadvantage of the expensive calculation in the above time domain solutions. Although the mathematical problem is formulated herein in sway, yaw, surge and roll motions, only the lateral motion results are presented here. The hydrodynamic coefficients, as functions of frequency, can be determined theoretically using two-dimensional strip theory or three-dimensional source distribution method. In order to verify the current technique with Fontijn's results, the same hydrodynamic coefficients are used herein.

2. DYNAMIC RESPONSE OF THE SHIP AND THE BERTHING STRUCTURE

During the berthing, the ship will undergo dynamic motion which has six degrees of freedom; namely, swaying, yawing, rolling, surge, heave and pitch. The heave and pitch motions are of little consequence in energy dissipation and may be neglected. In the following analysis, it is assumed that there is no sliding contact along the fender's surface. Also, the coupling effects between each mode have been neglected.

Consider the dynamic equilibrium of the center of gravity of the ship as shown in Figure 1. The equations of motion are:

$$(m_k+a_k)\,\ddot{x}_k + b_k\,\dot{x}_k = f_k \qquad k = 1, 2, 4, 6 \qquad (1)$$

where x_1, x_2, x_4 and x_6 are the surge motion (x, positive forward), sway motion (y, positive to port), roll motion (ϕ), and yaw motion (θ), respectively. The m_k, a_k and b_k are inertia mass, hydrodynamic mass and hydrodynamic damping in the corresponding directions. f_k represents the external fender reaction force on the ship in the k direction.

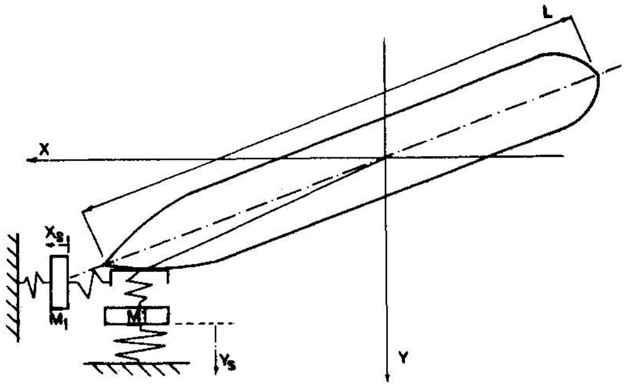


Fig. 1 Definition Sketch of Plan Section

For the berthing structure with the effective mass, M_s, the dynamic equations are:

$$M_s \ddot{x}_s + \mu_x \dot{x}_s + k_x x_s = -f_1 \tag{2}$$

$$M_s \ddot{y}_s + \mu_y \dot{y}_s + k_y y_s = -f_2 \tag{3}$$

where $K_{x,y}$ and $\mu_{x,y}$ are the structure stiffness and damping, respectively. The displacement of the point of contact can be calculated after the dynamic equations (1) to (3) are solved.

If the elastic deformation of the ship hull is under consideration, let K_h, and S_h and μ_h define the stiffness, deflection and the damping coefficients of the hull at the point of contact respectively. The equations of motion of the ship hull are:

$$M_h \ddot{S}_{hx} + \mu_{hx} \dot{S}_{hx} + K_{hx} S_{hx} = f_1 \tag{4}$$

$$M_h \ddot{S}_{hy} + \mu_{hy} \dot{S}_{hy} + K_{hy} S_{hy} = f_2 \tag{5}$$

In the case of a very rigid berth, the deflection of berthing structure can be neglected due to the high stiffness of the structure. The effects of the ship's local stiffness can be investigated with this method.

The solution of dynamic equations is carried out by the numerical integration method with appropriate choice of problem parameters.

3. HYDRODYNAMIC MASS AND DAMPING

The hydrodynamic mass and damping are important parameters to be considered in the determination of the berthing forces. The hydrodynamic mass is governed by the following factors:

- Ship characteristics (beam, draft, size)
- Under-keel clearance
- Water depth

- Type of berthing structures (open, semi-open, solid)
- Fender characteristics
- Berthing modes
- Berthing velocity.

Most of the methods to determine the hydrodynamic characteristics do not take into account all of the above-mentioned factors. A recent review by Kray [7] has presented a good summary of the state-of-the-art on hydrodynamic mass determination. The most common practice is to have a constant added mass as presented by Vasco Costa [13], Komatsu and Salman [6]. Hayashi and Shirai [4] presented a theoretical formulation of ship added mass in shallow water as a function of the ratio of draft to water depth, the Froude number of the ship and the coefficient of head loss of the counter flow under the hull. Oda [9] conducted experiments to verify their theory. In order to have better agreement in very shallow water berthing, Oda modified the theory by neglecting the shear stress acting on the hull and also used a friction parameter as a function of sway force coefficient, contraction coefficient and water depth to draft ratio. The theory modified by Oda will be used to compare with the existing experiments and present method.

The constant added mass coefficient is not appropriate for berthing of ships in shallow water with different berthing speeds. van Oortmerssen [12] and Endo [2] calculated the hydrodynamic characteristics of ship as function of frequencies in the shallow water based on a three-dimensional approach. Comparisons have been made with the results for cases available in the literature. Generally speaking, the agreement is good. In the study by Fontijn [3], the two-dimensional hydrodynamic characteristics at low frequencies were modified to fit the experimental data. Time histories of fender force in Fontijn's paper compared well with the present method.

4. MATHEMATICAL METHOD

As mentioned earlier, a basic difficulty arises in the time domain solution dealing with the frequency-dependent hydrodynamic coefficients. The force derived from drag and added inertia is a complex function of ship movement at each given frequency. The time history of a force denoted by f(t) can be described by means of the convolution integral as:

$$f(t) = \int_{-\infty}^{\tau} K(t-\tau)\, y(\tau)\, d\tau \qquad (6)$$

where $y(\tau)$ is the time history of the ship motion, e.g., sway. The Kernal function K(t) is the Fourier transformation of the complex transfer function of $F(\omega)$ with respect to the ship motion, which is given by:

$$K(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} F(\omega) e^{i\omega t}\, d\omega \qquad (7)$$

This procedure has been used in a number of investigations to obtain the histories of forces, e.g., Fontijn [3] and van Oortmerssen [12].

In the present application, the convolution operation is implemented by digital computation in a different manner. Tou [11] has shown that, for a single input sinusoidal frequency of amplitude a, the instantaneous value of the output time sequence, $f(n\Delta t)$, can be predicted by a weighted sequence of n previous time history steps as follows:

$$f(n\Delta t) = a \sum_{m=1}^{n} w_m \sin [\omega(n-m)\Delta t] \tag{8}$$

where:

w_m = weighting coefficients

Δt = time interval

However, the instantaneous values of this function can also be given in terms of an input sinusoid as:

$$f(n\Delta t) = |F(\omega)| \; a \sin (\omega n\Delta t+\alpha) \tag{9}$$

Where α is the phase angle of the output relative to the input. Equations (8) and (9) set equal to each other give a single equation relating the transfer function, F, to the weighting coefficients, w_m, for a given frequency ω. When dealing with inputs containing a number of frequencies, this method leads to a set of simultaneous equations for the value of w_m in terms of the values of the transfer function at each frequency. These equations can be solved by using a least squares approach for a set of weighting coefficients, w_m. When applying this method, the operation is over the past time history of the ship motion. The convolution integral is then replaced by a truncated summation:

$$f(n\Delta t) = \sum_{m=1}^{n} w_m \, y[(m-n)\Delta t] \tag{10}$$

After the evaluation of the added inertia force and drag force, the differential equations are solved by Runge-Kutta step-by-step integration procedure.

5. RESULTS

The analytical technique described herein has been applied to the case in Fontijn's paper [3]. The main dimensions of the ship used in the model are: waterline length = 2.438 m, beam = 0.375 m, draft = 0.15 m, block coefficient = 1.0, ship mass = 137.24 kg, water depth = 0.2 m. The added mass and damping coefficients were determined by Fontijn using ship theory three-dimensional effects on the low frequency range. The same hydrodynamic coefficients are used in order to have a fair comparison. Two types of fenders are considered: a linear fender of constant stiffness, k_1, equal to 140 kg/m, and a nonlinear fender force represented by $k_1 \Delta y_f + k_2(\Delta y_f - d_f)$, in which k_1 = 64 kg/m, k_2 = 113 kg/m, d_f = 0.00664 m and Δy_f is the deflection of fender.

Figure 2 shows the comparison between present method with Fontijn's calculated and measured fender forces as a function of time. This analytical method gives good agreement with the experiment. Hydrodynamic coefficients have great influence on the fender response. The use of the existing results is the first step in the development of this method. The nonlinear fender stiffness is calculated for the same ship in Figure 3. Theoretically, the program has been formulated to accept characterization of any fender system for the calculation of energy absorption. Various generic fender system algorithms have been developed which will be part of further program development.

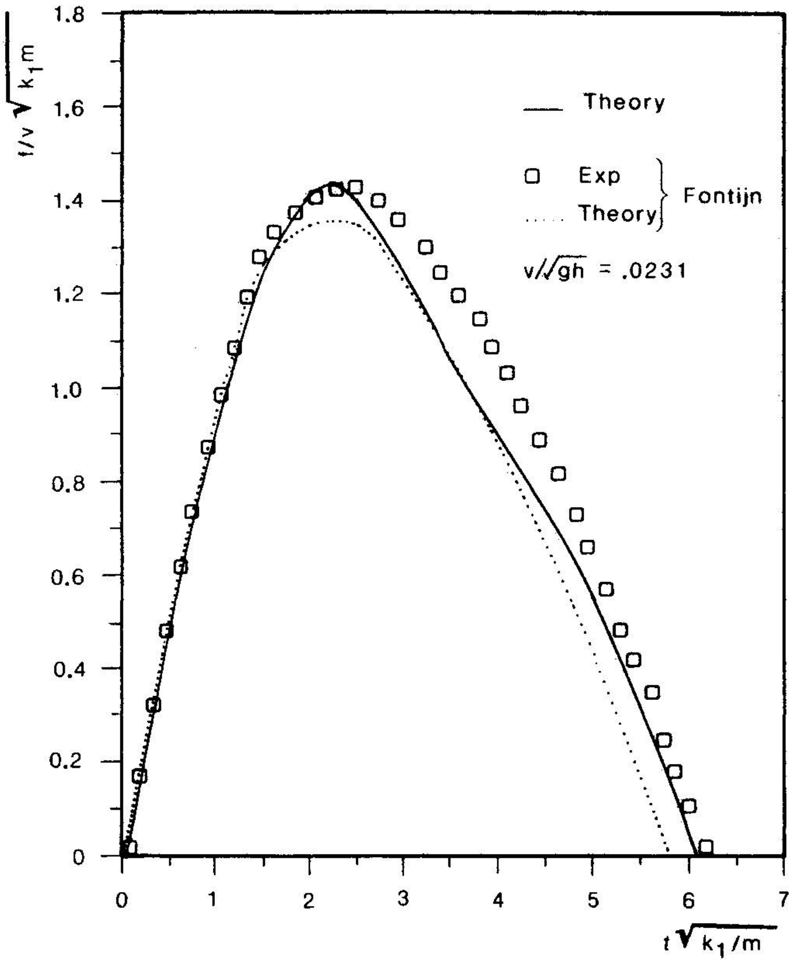


Fig. 2 Time History of Fender Force: Linear Fender

Hayashi and Shirai's method [4] is considered a simple way to calculate the added mass coefficient. They took into account the shallow water, ship speed and characteristics. That method was modified by Oda [9] to have a better agreement with experiments. Based on the modified theory, fender force on the same model used in this paper is presented in Figure 4. The approaching phase shows better agreement than the detaching phase; Oda shows the similar behavior in his results.

6. CONCLUSIONS

The frequency dependence of the hydrodynamic coefficients can be easily represented by a simplified convolution integral. This method has good agreement with existing theory and experiment. The hydrodynamic coefficients significantly affect the vessel-fender interaction. More experimental and theoretical studies are needed in order to include additional environmental effects. This method provides a means of analyzing the dynamic ship/fender problem in a simplified cost-effective manner.

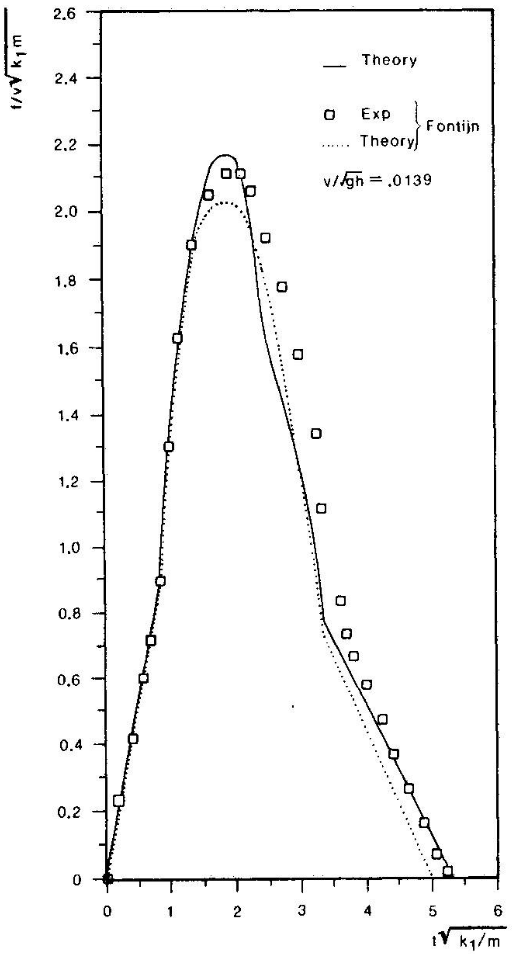


Fig. 3 Time History of Fender Force: Nonlinear Fender

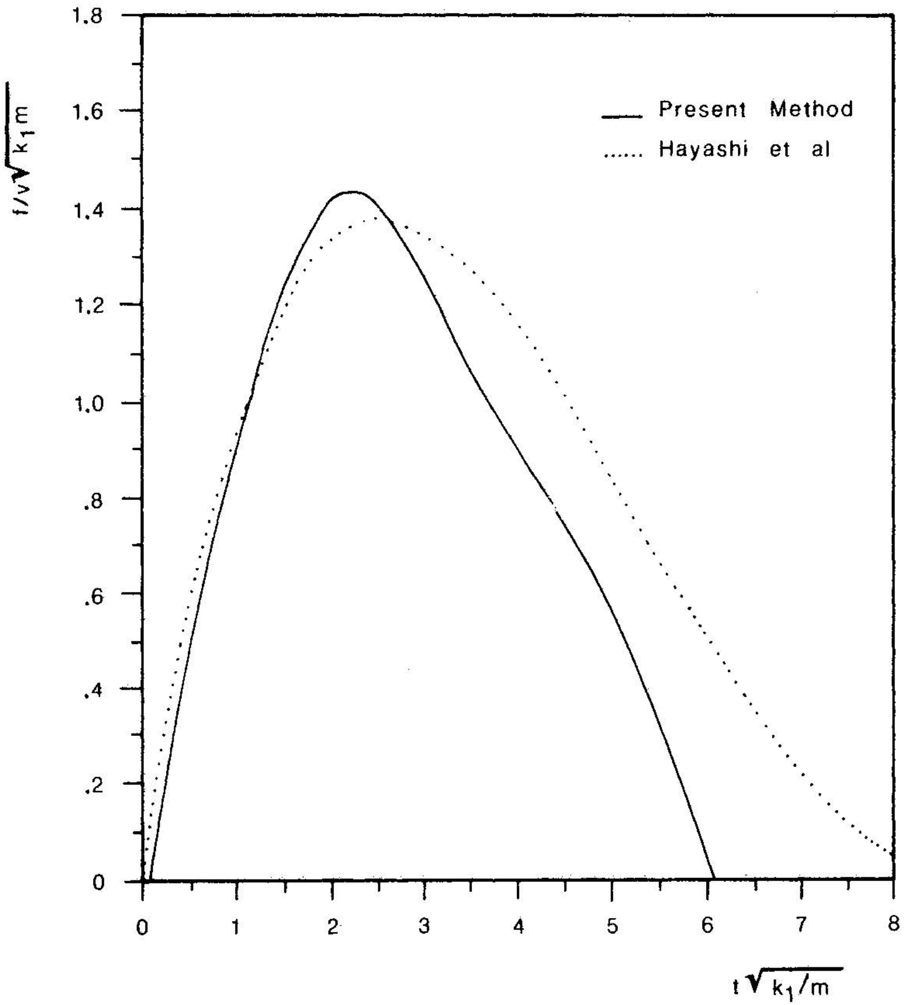


Fig. 4 Time History of Fender Force: Comparison

7. ACKNOWLEDGEMENTS

This work has been supported by the Office of Naval Research, Contract No. N00014-82-C-0018, under the direction of Dr. Eugene A. Silva. The authors also wish to express their appreciation to Drs. T. T. Lee and J. G. Giannotti for their many valuable comments and continuous encouragement.

8. REFERENCES

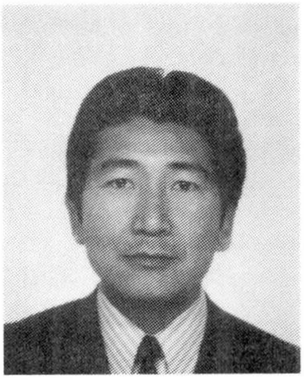
1. BROLSMA, J. V., HIRS, J. A., et.al, On the Fender Design and Berthing Velocities. 24th International Navigation Congress, 1977.
2. ENDO, H., Motions of a Vessel Moored Near Vertical Walls and with Limited Under-Keel Clearance. Thesis, Univeristy of Hawaii, 1981.
3. FONTIJN, H. L., The Berthing of a Ship to a Jetty. Journal of the Waterway Port Coastal and Ocean Division. ASCE, Vol. 106, No. WW2, May 1980.
4. HAYASHI, T. and SHIRAI, M., Added Masses of Large Tankers Berthing to Dolphins. ASCE, 15th Coastal Engineering Conference, Honolulu, Hawaii, Vol. III, 1976.
5. KIM, C. H., Sway Added Mass and Impact Load of a Large Tank to a Jetty in Shallow Water. The Arctic-Offshore-Deepsea Systems Symposium, ASME, 1982.
6. KOMATSU, S. and SALMAN, A. H., Dynamic Response of the Ship and the Berthing Fender System After Impact. Proceedings of JSCE, No. 222, April 1972.
7. KRAY, C. J., Safety of Ships and Structures During Berthing and Mooring. Journal of Marine Technology Society, Vol. 16, No. 1, 1982.
8. LEE, T. T., A Design Criteria Recommended for Marine Fender Systems. Proceedings of the 11th International Conference on Coastal Engineering, London, ASCE, 1968.
9. ODA, K., Virtual Masses of Supertankers Berthing to Dolphins. Proceedings of the 28th Japanese Conference on Coastal Engineering, 1981.
10. SVENDSEN, I. A., The Statistical Approach. PIANC International Study Commission for Improving Fender Design Systems, Edinburgh, 1981.
11. TOU, J. T., Digital and Sampled-Data Control Systems. McGraw-Hill Book Company, 1959.
12. van OORTMERSSEN, G., The Motions of a Moored Ship in Waves. Thesis, the Delft University of Technology, The Netherlands, 1976.
13. VASCO COSTA, F., The Berthing Ship, the Effect of Impact on the Design of Fenders and Other Structures. The Dock and Harbor Authority, 1964.

Modèles réduits de protection de pile de pont

Modellversuch zum Schutz von Brückenpfeilern

Small-Scale Models of Bridge Pier Protection

Minh Phong LUONG
Maître de Recherche
C.N.R.S.
Paris, France



Minh Phong Luong, né en 1937, Ingénieur Civil des Ponts et Chaussées en 1961, Docteur ès Sciences en 1964, Chercheur au L.M.S. depuis 1961, Enseigne la Mécanique des Sols à l'Université P. et M. Curie à Paris.

RÉSUMÉ

L'étude de la protection d'une pile de pont contre d'éventuels chocs de navires a été réalisée en laboratoire par des essais sur modèles réduits. Les résultats expérimentaux obtenus ont permis une évaluation à la fois qualitative et quantitative de l'influence des différents paramètres caractérisant le navire et le massif de protection de la pile de pont.

ZUSAMMENFASSUNG

Der Schutz eines Brückenpfeilers gegen eventuelle Zusammenstöße mit Schiffen wurde im Labor in Modellversuchen durchgeführt. Die Versuchsergebnisse ermöglichten sowohl eine qualitative als auch quantitative Einschätzung des Einflusses verschiedener für das Schiff und den Brückenpfeilersockel typischer Parameter.

SUMMARY

A study on how to protect bridge piers against impacts from vessels has been carried out in a laboratory through experiments on small-scale models. The experimental results obtained have made it possible to put forward a both qualitative and quantitative evaluation of the different parameters characterising the vessel and the protective block on the bridge pier.

1 - INTRODUCTION

La construction d'une pile de pont dans une zone du fleuve accessible à la navigation à marée haute nécessite l'étude d'une protection efficace contre d'éventuels chocs de navires de 80.000 tonnes de déplacement en cas de fausses manoeuvres.

Il a été envisagé d'entourer la pile d'un talus de sable suffisamment dimensionné pour résister et encaisser les chocs d'un navire qui avancerait horizontalement à une vitesse V constante pouvant atteindre 15 noeuds et dirigé parallèlement à l'axe du chenal de navigation, les navires étant affectés d'une panne de barre.

2 - DONNEES CARACTERISTIQUES

Les hypothèses simplificatrices suivantes ont été admises :

2.1- Le navire est supposé avoir tellement d'inertie qu'il ne déjauge pas. Il pénètre alors horizontalement dans le talus du massif de sable de protection. La dissipation d'énergie cinétique par soulèvement d'ensemble du navire est négligeable.

2.2- Le navire est considéré comme indéformable. C'est une hypothèse pessimiste pour les efforts subis par le massif.

2.3- Les dimensions moyennes des navires considérés sont de l'ordre de :

Longueur	:	$L = 220$ m
largeur	:	$\ell = 36$ m
hauteur	:	$h = 12$ m

2.4- La similitude hydraulique du modèle est celle de Froude, faisant correspondre les masses et les inerties de sorte que le modèle représentera en similitude la déformation que subirait un massif de protection à cohésion négligeable sous les chocs des navires.

3 - MODE OPERATOIRE DES ESSAIS

3.1- Modèle du navire

Une étude paramètrique préliminaire a permis d'étudier l'influence des différentes caractéristiques géométriques du navire.

- La longueur ne semble pas intervenir de façon significative car la résistance à l'avancement du modèle dans le talus de sable n'intéresse que sa partie frontale ;
- la forme de l'avant : c'est un dièdre représentant l'étrave, caractérisé par l'angle 2θ au sommet variant de 30° à 180° (figure 1) ;
- la largeur ℓ varie de 6cm à 36cm ;
- la hauteur h de 6cm à 12cm.

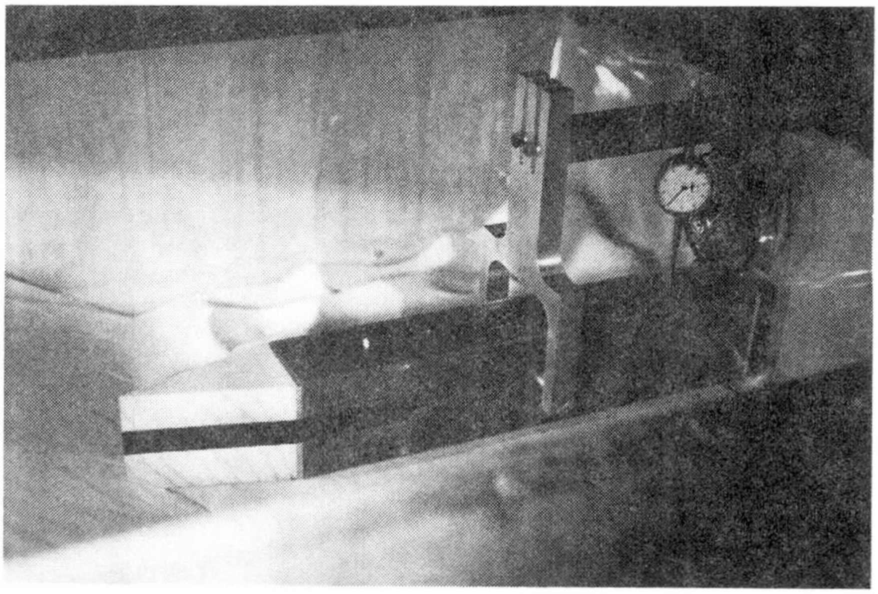


FIGURE 1 : Forme frontale du navire.

La navire avance horizontalement à une vitesse constante V vers le talus de sable (figure 2) poussé par une fourche qui s'appuie sur un axe à pivotement libre. L'angle d'incidence i entre la direction de la course du bateau et la normale à l'arête du talus dans le plan horizontal a varié de 0 à 45°.

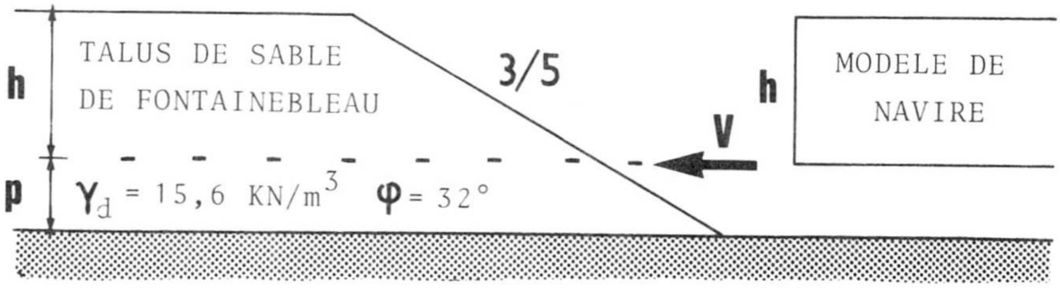


FIGURE 2 : Talus de protection contre les chocs des navires.

3.2- Massif de sable

Le matériau utilisé pour constituer le talus est du sable blanc de Fontainebleau. C'est un sable à grains siliceux très résistant et homogène, de granulométrie uniforme. Le coefficient de Hazen est de l'ordre de :

$$\chi = \frac{d_{60}}{d_{10}} = \frac{0,29}{0,17} \sim 1,7$$

Le poids volumique correspondant à notre mise en place pour les essais est 15,6 kN/m^3.

Le talus naturel observé est de l'ordre de 32°.

Différentes pentes de talus ont été réalisées : 3/5 , 1/2 , 1/2,5 , 1/3 sur un horizon supposé rigide situé à une distance par rapport au fond du modèle de navire (figure 2).

3.3- Dispositif de mesure

Lors des essais sur modèles réduits, nous avons cherché à évaluer avec précision :

- la résistance à l'avancement due au frottement latéral sur le fond et les deux côtés du navire ;
- la résistance à l'avancement due à la butée sur la partie frontale du navire.

Nous avons mesuré en fonction du déplacement horizontal D :

- la force totale horizontale F_H, nécessaire pour faire avancer le modèle ;
- la force horizontale F_F due au frottement sur le fond et les deux côtés à l'aide des lames flexométriques étalonnées ;
- la force verticale F_V nécessaire pour maintenir le modèle horizontale. Ce dernier est supposé avoir une très grande inertie pour ne pas déjauger.

Le dispositif de mesure permet de tracer simultanément pour chaque essai les courbes F_H, F_V et F_F en fonction du déplacement horizontal d (figure 3).

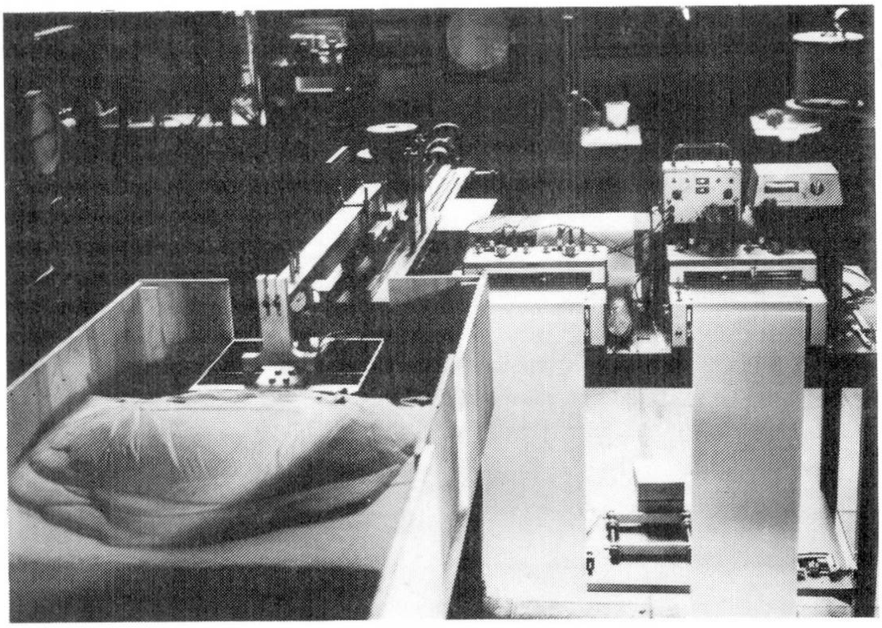


FIGURE 3 : Dispositif expérimental.

4 - RESULTATS EXPERIMENTAUX

A chaque instant au cours de l'essai, les valeurs des forces horizontale F_H et verticale F_V donnent l'obliquité de la réaction par rapport à l'avancement. Nous avons pu vérifier grossièrement que cette obliquité est de l'ordre de grandeur de l'angle de frottement interne φ.

4.1- Ordre de grandeur de la butée

La théorie de la butée d'un écran vertical sur un massif de sable nous donne les valeurs suivantes du coefficient de butée, [1] :

$$\varphi = 30° \quad K_p\ (\lambda = o\ ,\ \beta = o\ ,\ \delta = -\ \varphi) = 6{,}42$$
$$\varphi = 35° \quad K_p\ (\lambda = o\ ,\ \beta = o\ ,\ \delta = -\ \varphi) = 10{,}2 \quad .$$

Soit, pour un modèle de dimensions h = 10cm, ℓ = 8cm, une force de butée égale à :

$$F = \frac{1}{2}\ \gamma\ h^2\ \ell\ K_p \quad .$$

D'où les valeurs calculées des composantes F_H et F_V :

$$\varphi = 30° \quad F_H = 34{,}7\ N \quad F_V = 20\ N$$
$$\varphi = 35° \quad F_H = 52{,}1\ N \quad F_V = 36{,}5\ N$$

4.2- Ordre de grandeur du frottement sur le fond

La force de frottement sur le fond a pour valeur :

$$T = \int_0^{30cm} \tau\ .\ \ell\ .\ dx$$

Or, $\tau = \sigma\ tg\ \varphi = \gamma y\ tg\ \varphi = \gamma\ \alpha\ x\ tg\ \varphi$.

Donc : $T = \frac{1}{2}\ .\ \gamma\ .\ \alpha\ .\ tg\ \varphi\ .\ 30^2\ .\ \ell$.

D'où les valeurs calculées du frottement T (en N) pour chaque pente α du talus :

φ \ α	3/5	1/2	1/2,5	1/3
30°	19,5	16,2	13,0	10,8
35°	23,6	19,7	15,7	13,1

4.3- Comparaison des valeurs expérimentales et calculées

L'angle de frottement interne φ du sable est estimé voisin de 32°. Les essais ont donné des valeurs qui concordent assez bien avec le calcul (cf. Tableau) :

Valeurs de F_H en N avec h = 10cm, ℓ = 8cm.

φ \ α	3/5	1/2	1/2,5	1/3
30°	54,2	50,9	47,7	45,5
Essais	75,0	64,0	60,0	58,0
35°	75,7	71,8	67,8	65,2

Les valeurs de F_V aussi sont assez bien encadrées par les valeurs calculées au paragraphe 4.1.

4.4- Interprétation des résultats

L'exploitation des résultats s'est avérée plus commode en examinant la variation du travail W dissipé, c'est-à-dire l'aire comprise entre l'axe des abscisses et la courbe F_H pour un déplacement total de 30cm avec les différents paramètres.

i - Vitesse d'avancement :

La courbe W(V) indique une valeur constante : la résistance à l'avancement est indépendante de la vitesse : c'est un problème quasi-statique (figure 4).

ii - Incidence de la trajectoire par rapport au talus :

La courbe W(i) indique une décroissance presque linéaire lorsque l'incidence augmente de 0° à 45°, (figure 5).

iii- Poids volumique du sable :

Des essais dans du sable sec et du sable immergé indiquent que le travail W augmente avec le poids volumique γ, (figure 6).

iv - Forme dièdre de l'avant du bateau :

Les résultats expérimentaux font apparaître un léger accroissement de W avec l'angle du dièdre, (figure 7).

v - Largeur ℓ du bateau :

Nous avons obtenu une croissance linéaire de W avec la largeur ℓ, (figure 8).

vi - Hauteur h du bateau :

L'allure des courbes W(h) est parabolique de concavité dirigée vers l'axe des W, (figure 9).

vii- Pente du talus :

Nous avons constaté une croissance de W avec la pente α. Le travail W tend vers une valeur limite correspondant au cas du mur vertical, (figure10).

viii- Frottement sur le fond et sur les parois latérales :

La mesure directe de F_F a montré que ce frottement n'apporte pas une résistance aussi importante qu'on aurait pû l'espérer : elle est de l'ordre de 2% de la résistance totale à l'avancement dans le talus de sable, ce qui est tout à fait insignifiant.

ix - Profondeur de l'assise rigide :

Les valeurs expérimentales de la résistance totale à l'avancement semblent peu dépendre de l'épaisseur p de la couche comprise entre le fond du modèle et l'assise rigide, (figure 11).

x - Rugosité des parois du modèle :

Un essai a été réalisé en tapissant le modèle de papier émeri afin de pouvoir mobiliser rapidement le frottement maximum. Nous n'avons pas constaté une différence significative, ce qui montre que le frottement sur les autres modèles est tel qu'il faut admettre que les surfaces sont rugueuses.

xi - Utilisation de la terre armée :

L'étude du comportement du matériau Terre Armée à l'appareil triaxial [4] a montré qu'il présente une cohésion anisotrope, proportionnelle à la résistance à la traction des armatures.

Deux essais avec un modèle en sable de Fontainebleau armé par des bandes de papier et dans une similitude très rudimentaire ont montré une grande augmentation de la résistance à l'avancement et la courbe F_H s'est incurvée vers l'axe des déplacements dès que se sont manifestés les défauts d'adhérence des armatures, c'est-à-dire le glissement des bandes par rapport au sol.

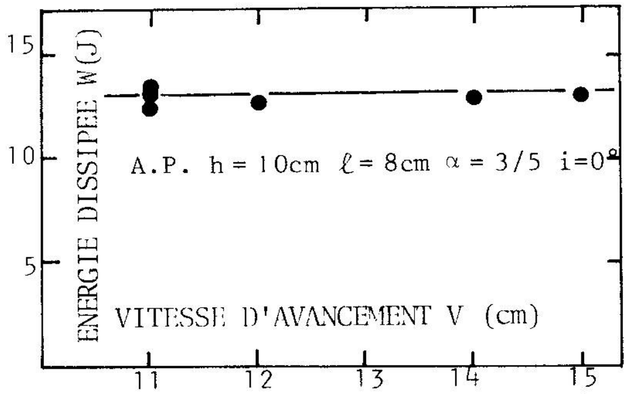


Figure 4 : Travail dissipé en fonction de la vitesse d'avancement vers le talus

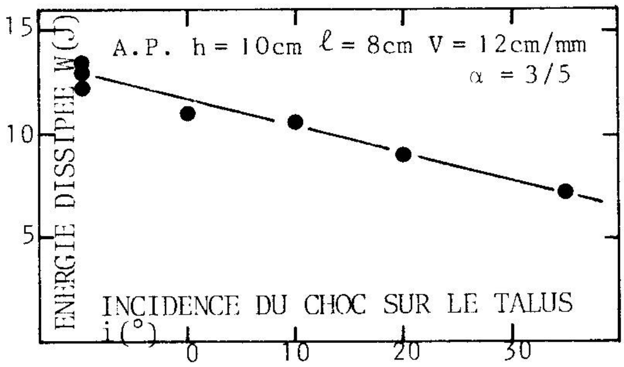


Figure 5 : Influence de l'incidence du choc.

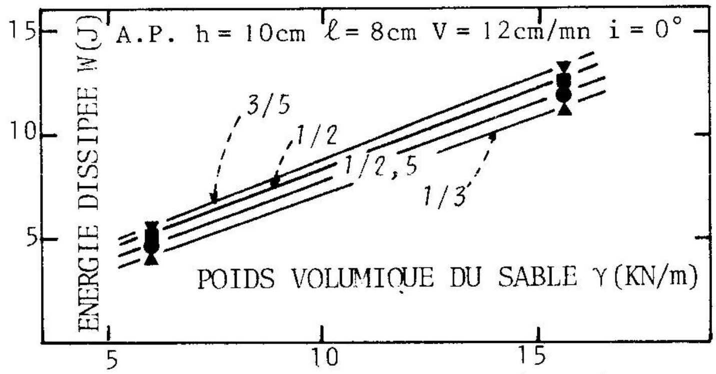


Figure 6 : Effet du poids volumique du sable.

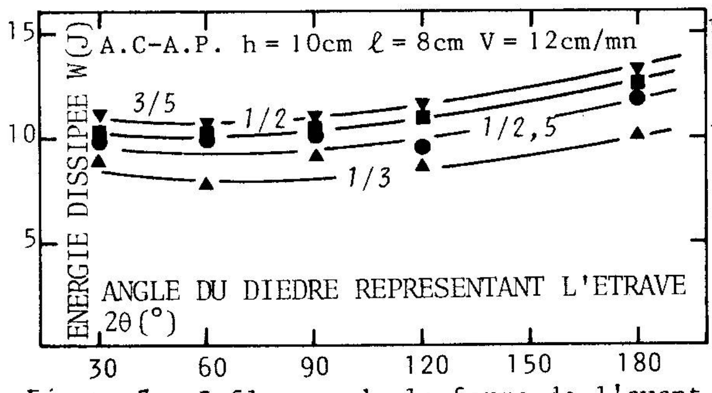


Figure 7 : Influence de la forme de l'avant du bateau.

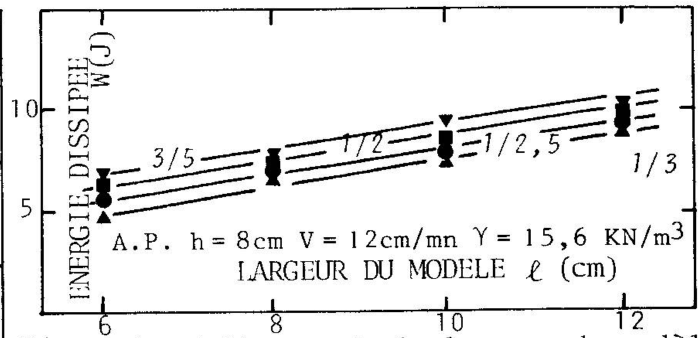


Figure 8 : Influence de la largeur du modèle bateau.

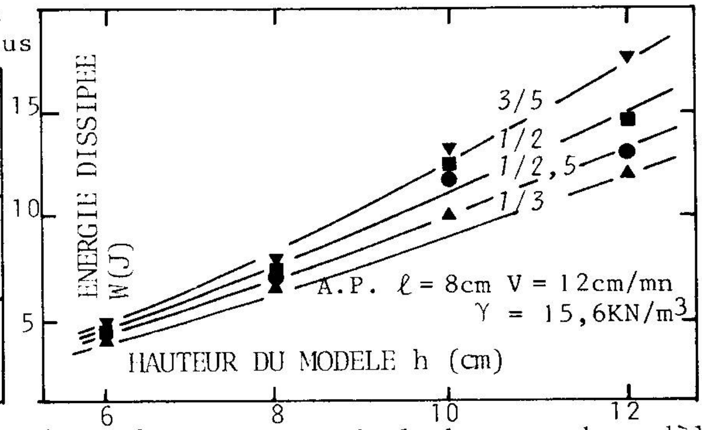


Figure 9 : Influence de la hauteur du modèle bateau.

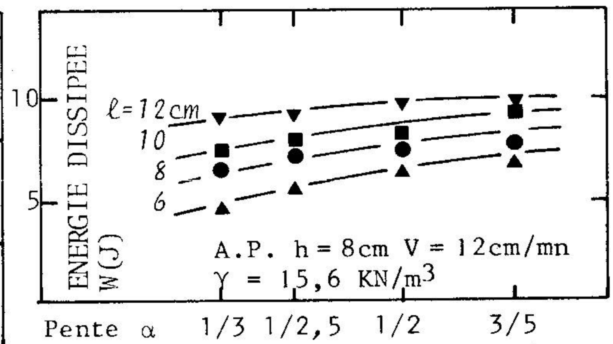


Fig. 10 : Influence de la pente du talus.

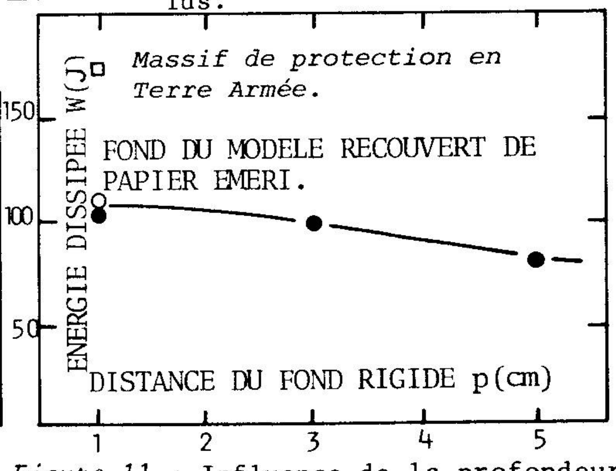


Figure 11 : Influence de la profondeur de l'assise rigide.

xii- Energie dissipée :

L'énergie dissipée au cours de l'avancement du modèle dans le talus de sable est donnée par l'aire comprise entre l'axe des abscisses et la courbe F_H pour un déplacement horizontal total de 30cm.

Les essais correspondant au modèle de dimensions h = 12cm, ℓ = 36cm ont donné un travail dissipé moyen de l'ordre de 1000 daN.cm, soit 100 J pour un déplacement horizontal de 30cm.

En utilisant la similitude du choc des navires, adoptée par SOGREAH qui conduit à prendre une échelle des longueurs égale à $\overset{*}{\ell} = 10^{-2}$ et une échelle des forces $\overset{*}{F} = \overset{*}{\ell}{}^3 = 10^{-6}$, soit une échelle des travaux $\overset{*}{W} = \overset{*}{F}\,\overset{*}{\ell} = 10^{-8}$, nous obtenons un travail dissipé dans l'ouvrage réel de l'ordre de 10^8 Joules pour un déplacement horizontal total de 30m.

Or, l'énergie cinétique d'un navire de 80.000 tonnes animé d'une vitesse de 15 noeuds est :

$$\frac{1}{2} M V^2 = \frac{1}{2} 8.10^7 \ . \ \overline{7,72}^2 = 2,38.10^9 \text{ Joules} .$$

Il apparaît ainsi que le massif de sable de protection est capable de dissiper l'énergie cinétique du navire et que l'utilisation d'un matériau terre armée pour l'ouvrage de protection de la pile devrait pouvoir facilement réduire la distance de pénétration du navire dans le talus.

5 - CONCLUSION

L'étude expérimentale réalisée a permis de mettre en évidence l'influence des différents paramètres sur la résistance à l'avancement d'un corps rigide à grande inertie dans un talus de sable.

- croissance linéaire du travail dissipé en fonction du poids volumique du sable, de la largeur du bateau ;
- croissance parabolique avec la hauteur du bateau, comme le confirme le calcul des forces de butée ;
- indépendance de la vitesse d'avancement.

Le travail dissipé au cours de l'avancement du navire dans le talus de sable semble être suffisamment important pour qu'il soit possible d'absorber l'énergie cinétique du navire lors du choc.

Etant données les dimensions relatives des navires et de la pile de pont, il est souhaitable que des essais sur modèles réduits plus élaborés soient réalisés avec simulation du rapport des masses respectives.

REFERENCES

[1] CAQUOT A., et KERISEL J., "Traité de Mécanique des Sols", Gauthier-Villars, Paris, 1966.

[2] LUONG M.P., "Etude de la Résistance à l'avancement d'un navire dans un talus de sable", L.M.S., Ecole Polytechnique, Mars 1973, Rapport d'étude pour la Chambre de Commerce du Havre.

[3] LUONG M.P., "Essais sur modèle réduits pour l'étude de la protection d'une pile de pont", L.M.S., Ecole Polytechnique, Juillet 1973, Rapport d'étude pour la Chambre de Commerce du Havre.

[4] SCHLOSSER F., et NGUYEN T.L., "Etude du comportement du matériau Terre Armée", Annales I.T.B.T.P., Avril 1973.

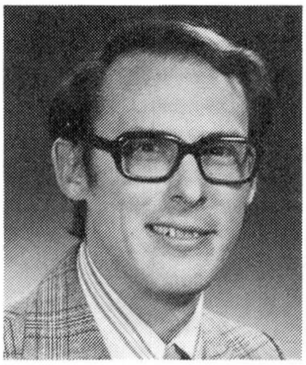
Hydrostatically Supported Sand Structures as Ship Collision Barrier

Structures hydrostatiques, en sable, contre les collisions de navires

Hydrostatisch unterstützte Sandstrukturen als Schiffsaufprallabfänger

John B. DAVIS
Principal
Golder Associates
Mississauga, ON, Canada

Masutoshi YUDASAKA
General Manager
Taisei Corporation
Tokyo, Japan



John B. Davis, born i 1941, received his master of engineering science degree in geotechnical engineering from the University of Western Ontario in 1965. Throughout his career, he has been involved in the geotechnical aspects of major civil and offshore structures in both Canada and abroad.

Masutoshi Yudasaka, born in 1931, received his civil engineering degree from the University of Tokyo in 1954. Upon graduation, he joined Taisei Corp., an engineering and construction firm, and has been involved in the design and construction of marine structures and prestressed concrete bridges.

SUMMARY

This paper describes the application of hydrostatically supported sand structures to ship collision barriers. The hydrostatically supported sand structures basically consist of sand and rubber walls. Their use in offshore engineering presents a highly competitive alternative to steel and concrete structures. Various model tests and prototype experiments have proven that hydrostatically supported sand structures are highly stable and that bearing capacity against horizontal forces is also high. A new type of ship collision barrier having high ship collision energy absorbability is proposed for low construction cost and expeditious execution of work.

RÉSUMÉ

Le présent article décrit l'emploi de structures hydrostatiques, en sable, comme barrières contre les collisions de navires. Ces structures sont essentiellement constituées de parois de sable et de caoutchouc. Leur mise en oeuvre dans la technologie offshore fait d'elles un concurrent très sérieux des structures en acier ou en béton. Plusieurs essais sur modèles et diverses expériences de prototypes ont prouvé que ces structures soutenues de manière hydrostatique étaient particulièrement stables et que leur capacité de charge vis-à-vis de forces horizontales était également importante. Une nouvelle forme de barrière contre les collisions de navires, dotée d'une grande puissance d'absorption de l'énergie produite par les collisions de ces navires, est ainsi proposée, mettant de ce fait en relief les propriétés de ces structures de sable pour un coût modique de construction et une rapide exécution des travaux.

ZUSAMMENFASSUNG

Dieses Dokument beschreibt die Anwendung hydrostatisch unterstützter Sandstrukturen als Stoßfänger bei Schiffszusammenstößen. Die hydrostatisch unterstützten Sandstrukturen bestehen im Grunde aus Sand- und Gummiwänden. Ihre Anwendung in der küstenfernen Technik stellt eine äußerst wettbewerbsfähige Alternative zu Stahl- und Betonkonstruktionen dar. Verschiedene Modell- und Prototypversuche haben ergeben, daß hydrostatisch unterstützte Sandstrukturen hochstabil und ihre Lagerungseigenschaften gegen horizontale Kräfte auch hoch sind. Hier wird ein neuer Typ von Stoßfängern bei Schiffskollisionen vorgeschlagen, wobei die Eigenschaften hydrostatisch unterstützter Sandstrukturen bei geringen Baukosten und schneller Arbeitsausführung verfügbar sind.

1. INTRODUCTION

The concept of hydrostatically supported sand structure was first conceived in 1974. Since then the feasibility of applying this concept to various offshore structures has been confirmed through model tests in laboratories and prototype experiments.

As shown in Fig. 1, the hydrostatically supported sand structures basically consist of sand and impervious rubber walls. The principle is based on its ability to dewater the sand during construction thus reducing the internal porewater pressures and providing stability for the sand mass. During and after construction, the membrane acts as a diaphragm by which the hydrostatic pressure is converted to a horizontal confining force on the sand mass. Theoretically, the membrane can be non-load bearing, its sole function being to act as an impervious wall. However, for efficiency in handling and to provide an additional safety margin during construction, membranes having nominal tensile strength will be used.

To finalize the construction technique and to prove the structure's stability, a 17 m high prototype structure "Sandisle Ann" was installed in Christchurch Bay in 1975. Christchurch Bay, Hampshire, off the southern coast of England was chosen as the location for the prototype because of the suitable water depth quite close to the shore, its seabed conditions provided a suitable foundation, and a fetch of over 300 km in the southwesterly direction of the prevailing winds was ideal as a severe marine testing environment. This prototype structure consisted of a steel deck unit within a bag fabricated of nylon-reinforced neoprene membrane. The prototype experiment demonstrated that the method of constructing hydrostatically supported sand structures was sound.

Since the construction of "Sandisle Ann", theoretical and experimental research on hydrostatically supported sand structures has continued with the development of various specific applications including ship collision barrier discussed in this paper.

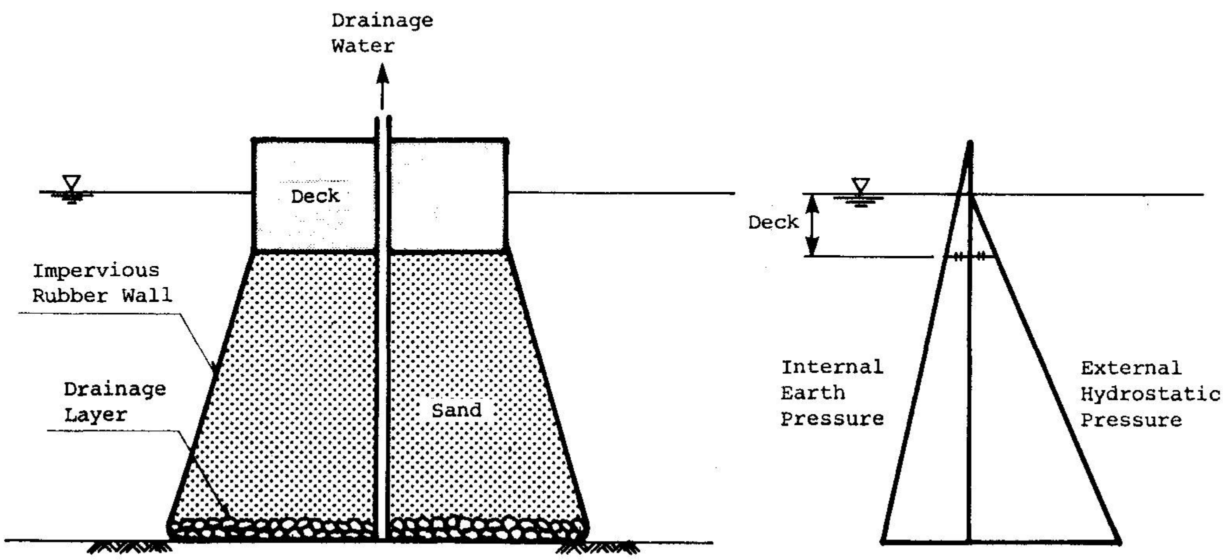


Fig. 1 Concept of Sandisle structures

2. CONSTRUCTION PROCEDURE

2-1 Stability during construction

The hydrostatically supported sand structures are not constructed within a strong, rigid container but within a relatively weak flexible bag. It is of fundamental interest to determine the stresses this bag must tolerate during the filling process. As previously stated, once the structure is completed, the membrane is only required to be impervious and carries no hoop stresses.

Sand is added into the water at the same rate as the water draining out through the base of the cell. The actual stresses in the sand are considered in terms of effective stress. In Fig. 2, an element of sand Z at a distance of z above the drainage layer is examined.

Horizontal effective stress $\sigma_H' = (x - Z)\ \dfrac{\gamma_w D}{x}$

Vertical effective stress $\sigma_V' = (x - Z)\ (\gamma' + \dfrac{D\gamma_w}{x})$

Internal shear stress $\dfrac{\sigma_V'}{\sigma_H'} = \dfrac{x\gamma_w' + D\gamma_w}{D\gamma_w}$

$= 1.7$ (when $x = D$)

$= 1.0$ (when $x = 0$)

The mobilized angles of friction are calculated as follows.

$x = D$	$x = 0$
$\dfrac{\sigma_V'}{\sigma_H'} = \dfrac{1 + \sin\phi}{1 - \sin\phi} = 1.7$	$\dfrac{\sigma_V'}{\sigma_H} = \dfrac{1 + \sin\phi}{1 - \sin\phi} = 1.0$
Mobilized $\phi = 15°$	Mobilized $\phi = 0°$

The interesting conclusion drawn from this calculation is that the mobilized angle of internal friction is constant throughout the sand mass for any individual height of fill and increases from 0° at the start of filling to about 15° at the end of filling. With the ultimate angle of internal friction at least 35° in the majority of sands, the level of shear strength mobilization is quite low. Therefore no hoops stress will act in the membrane.

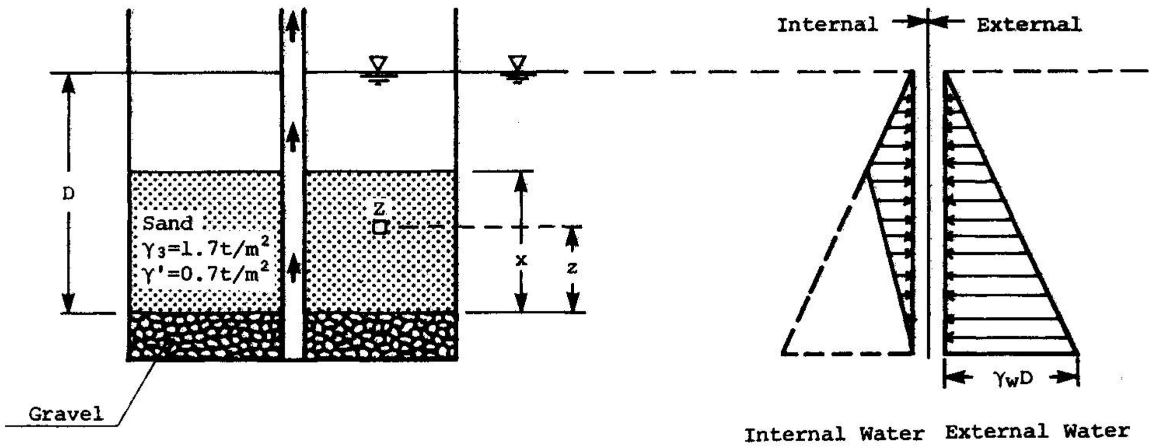


Fig. 2 Effective stresses in Sandisle construction

2-2 Construction of prototype "Sandisle Ann"

The prototype structure consisted of a steel deck unit within a bag fabricated of nylon-reinforced neoprene member. The deck was fitted with the membrane in Southampton and the prototype module was then towed from Southampton and moored alongside the attendant control vessel. After a day of preparation, construction started at 2 a.m. on Wednesday, September 15, 1976. The bag was first deployed by filling it with water. After the divers confirmed that it was properly extended and in contact with the seabed, the first fill was dropped from a central hopper within the deck unit. The wells and instrumentation were lowered into place, the buoyance of the deck trimmed, and the sand filling process began.

The following is an abbreviated construction schedule. Actual construction took only two days.

September 15	
0200	Deployment of bag.
0300	Placement of initial ballast in bag.
0300 to 0700	Lowering and preparation of main wellscreens.
0700 to 1100	Completion of gravel base layer.
1100 to 1900	Placement of pressure relief wells and connection of piezometers.
1900 to 1000 (Sept.16)	Balancing of pumping and filling system.
September 16	
1000 to 1700	Main sand filling and dewatering. Intensive monitoring of instruments.
1700 to 1900	Wait for slack water.
2100 to 2400	Filling of final meter of sand.
2400	Touchdown.

The actual filling and dewatering of the sand proceeded precisely in the textbook manner predicted from the extensive laboratory testing program. The short life of the prototype structure demonstrated the soundness of hydrostatically supported sand structure construction method.

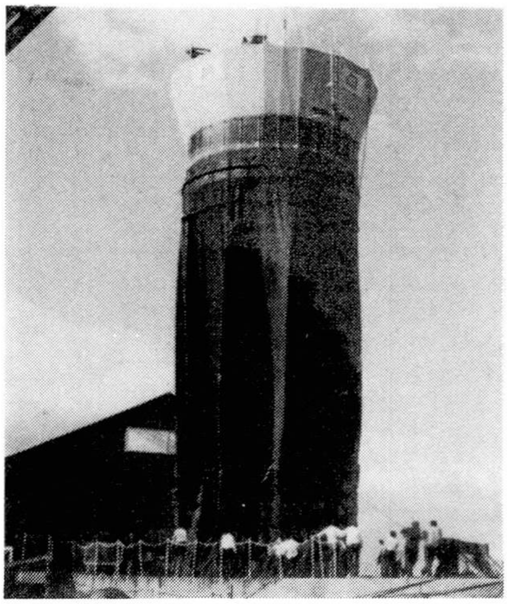


Fig. 3 Sandisle Ann being assembled at dockside

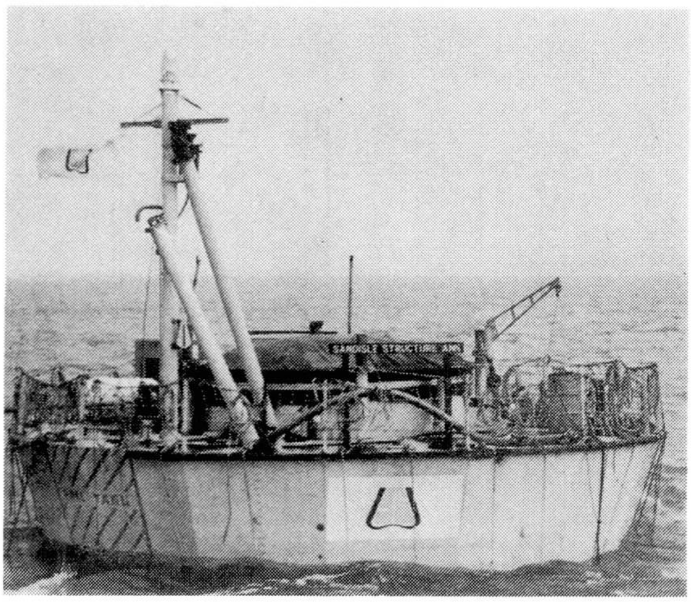


Fig. 4 Completed Sandisle Ann in Christchurch Bay

3. APPLICATION OF CONCEPT TO SHIP COLLISION BARRIER

There have been many reports submitted on accidents caused by the collision of ships with piers or offshore structures which have caused serious damage. So when constructing piers or offshore structures, sufficient protection from ship collision damages is necessary. From the structural and functional considerations, a ship collision protection should:

- Have high absorptivity of collision energy produced by oncoming ships.
- Take up the least amount of space so as not to interfere with the navigation of ships.
- Be easy to construct and at low cost.
- Be easy and inexpensive to maintain.

Needless to say, the actual design of a ship collision protection greatly depends on ship size, collision speed, water depth, condition of foundation ground, etc. Currently, there are two methods in use as ship collision protection. The most widely used method is where rubber fenders or buffers are attached directly onto piers or offshore structures, but this method is effective only when the colliding ship is of a small scale and if a large scale ship collides against it, piers or offshore structures may be seriously damaged. In the other method, independent ship collision protection structures are provided around piers or offshore structures and is effective against collision of relatively large scale ships, although its drawback is the generally high construction cost.

The hydrostatically supported sand structure, on the other hand, has none of the drawbacks of the above two methods. It is highly durable, due to the large sand mass, against great collision impact and advantageous because of its low construction cost and its short construction period.

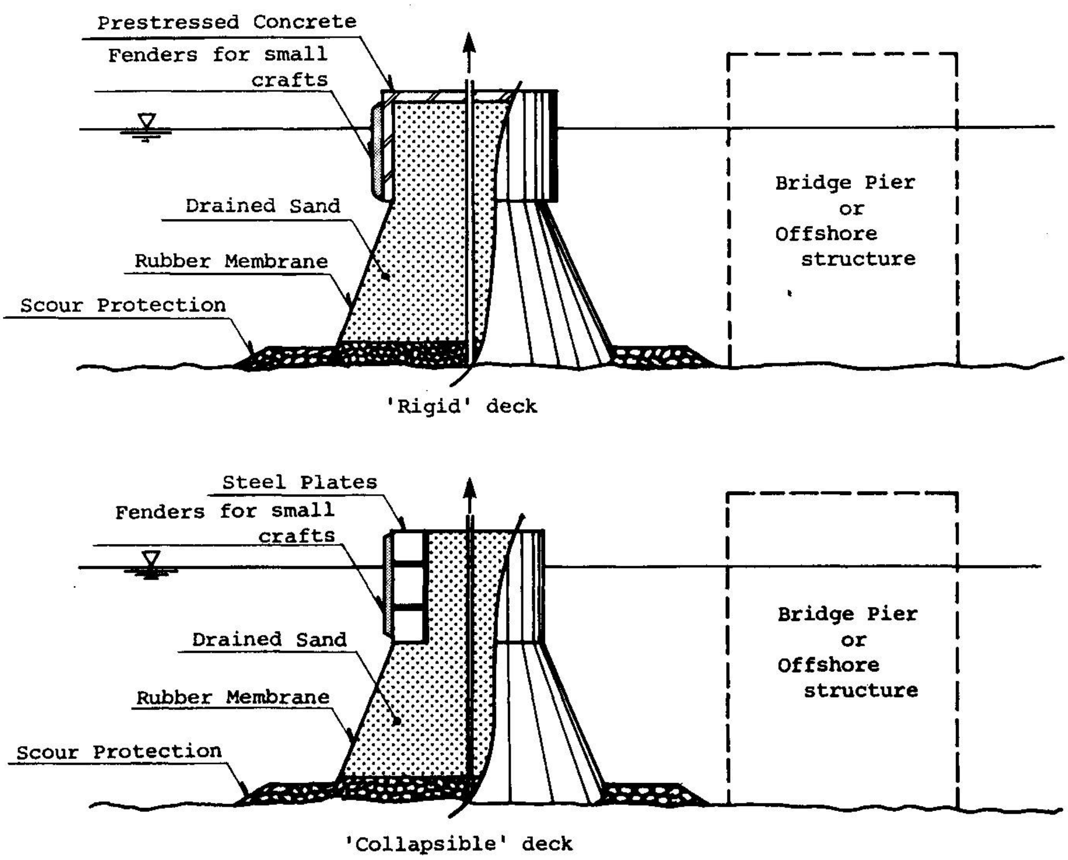


Fig. 5 Two types of ship collision barrier

Two basic types of hydrostatically supported sand structure for use as a collision protection or fendering structure are illustrated in Fig. 5. The first type of structure has a very "rigid" deck unit --- the collision energy of the oncoming ship is dissipated by crushing of the ship's bow. This results in severe damage to the ship but little damage to the Sandisle unit. The second type has a "collapsible" deck unit --- the ship's momentus is largely absorbed by crushing of the deck unit and the penetration of the ship's bow into the drained sand mass. Damage to the ship is minimized but a major portion of the Sandisle would probably be destroyed and would have to be replaced.

In designing the aforesaid hydrostatically supported sand structures, other considerations must be given such as collision of small ships and scouring at seabed around the structure. The possibility of small ship's colliding is high, but its impact force is low. To minimize the damage to both the structure and small ships, attaching rubber or timber fenders around the deck unit of the structure is recommended. Scour protection should be designed by taking into consideration the maximum tidal current velocity and the wave height at the site.

4. MODEL TEST AND ANALYSIS

When hydrostatically supported sand structures are used as collision protection or fendering structure, the most important point in designing is to grasp the behavior of such structure against horizontal impact force. Laboratory tests and analyses of the finite element method were carried out to observe and determine the maximum resistance and failure modes of the structures.

4.1 Summary of tests

A horizontal load test was carried out under two different conditions. The first condition was when the water in the model was well drained. The horizontal load was gradually increased until the model failed. The deformation and the maximum horizontal load were measured during the test. The other condition was when the water was flowing into the model, where 90% of the maximum horizontal load was applied.

The model consisted of a soft vinyl bag filled with sand and was reinforced with acryl frame at the upper half of the model. The acryl frame was provided with an opening of 6 cm x 6 cm so that the water can flow into the model. Table 1 shows the profiles of the three types of models. All models were rectangular and 60 cm high.

Fig. 6 shows the instruments used in the tests. The load was applied horizontally with a jack at a position 50 cm from the bottom of the models. Deformations of the models were measured with the four dial gauges shown in Fig. 6. The water level in the water tanks was 50 cm from the bottom of the models. The friction coefficient between the bottom of the models and the bottom of the water tank was measured to be 0.58.

4.2 Consideration

Fig. 7 shows the relationship between the applied horizontal loads and displacement at the model crest. The maximum resistance of each model was about 20 kg, 95 kg, and 210 kg, respectively. Each load-displacement curve shows that the structure undergoes a serious non-linear deformation under the horizontal force. The fact that resistance remains at a certain level despite deformation increase after yielding shows the large energy-absorbing capacity of the structure against horizontal force.

As clearly seen in Figs. 8 and 9, the failure mode against horizontal force is a sort of shear failure. Failure starts from the model's compressive side and propagates to the whole structure with an increase in horizontal force. Fig. 10 shows the distribution of principal stresses of model A obtained in an FEM analysis and the failure zone when Mohr-Coulomb's Failure Criteria is used. Failure occurs on the model's compressive side, which coincides with the model test results.

Table 2 shows the elapsed time before the models failed while the water was flowing into the model. As shown in Table 2, it took a considerably long time before the models failed. This shows that even though the deck unit is damaged at the moment of ship collision, there is no negative effect on the stability of the structure for a short period.

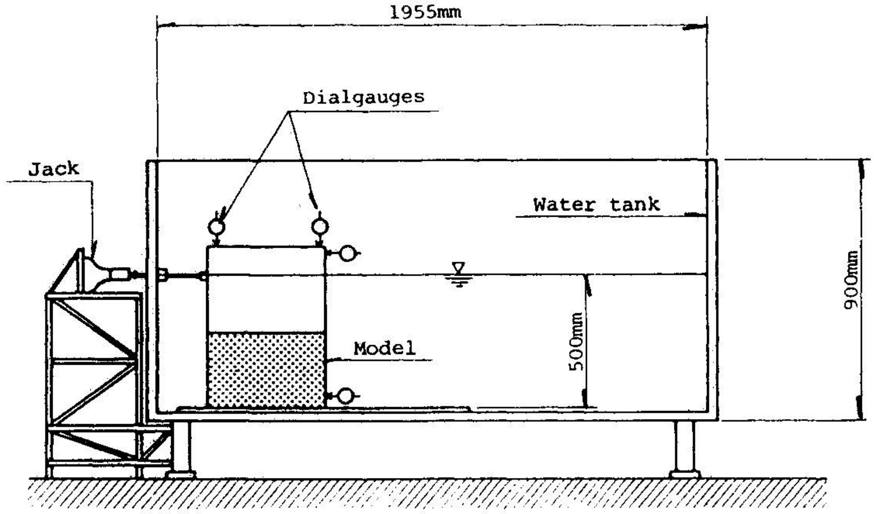


Fig. 6 Side view of test apparatus

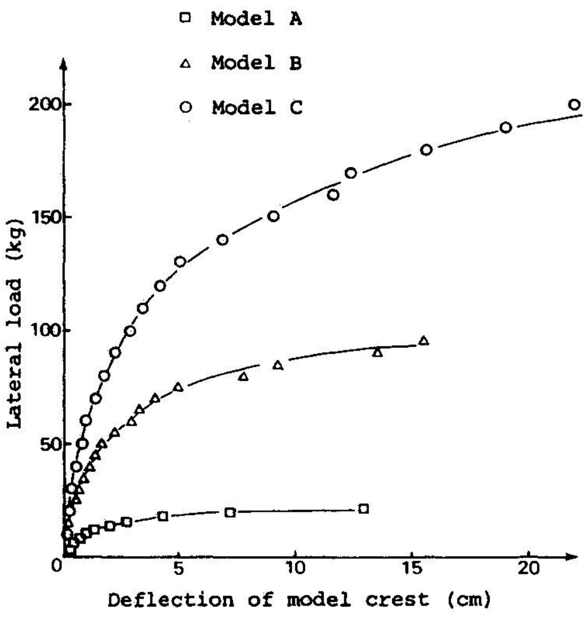


Fig. 7 Model crest deflection versus lateral load

Model	A	B	C
Height (cm)	60	60	60
Width (cm)	40	60	80

Table 1 Profiles of models

Model	A	B	C
Time (min)	7	21	42

Table 2 Elapsed time before model fail

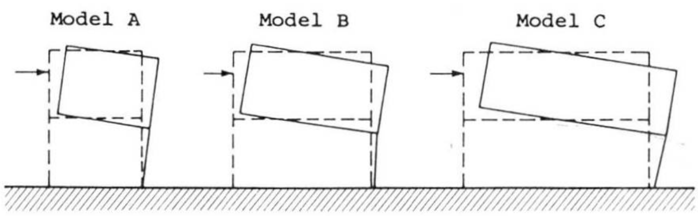


Fig. 8 Failure mode of models

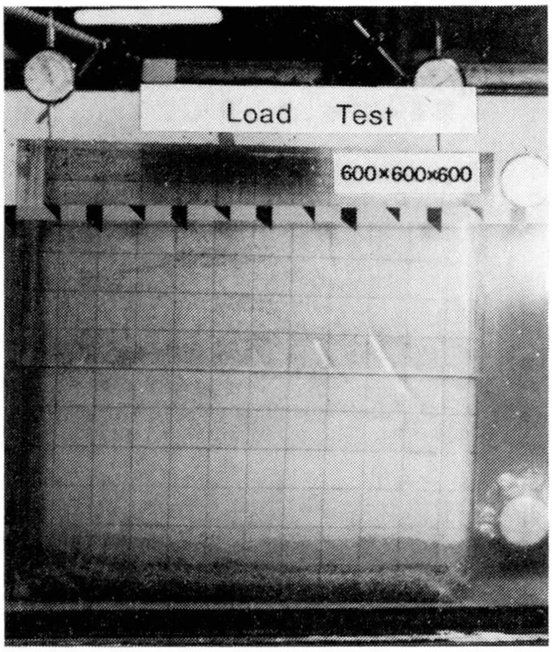
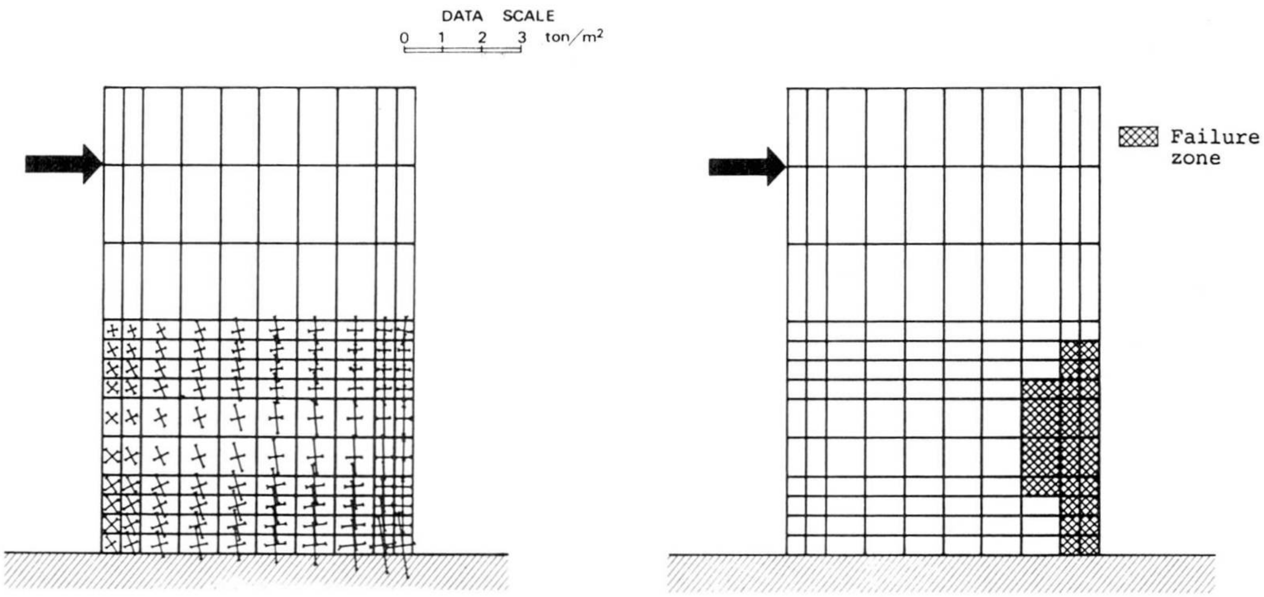


Fig. 9 Photo of failure mode



(a) Principal stress (b) Distribution of failure zone

Fig. 10 Result of analysis

5 CONCLUSION

Two types of hydrostatically supported sand structures for use as a collision protection or fendering structure are proposed in this paper. The prototype experiments demonstrated that the structure is highly stable and the construction method is sound. The load displacement curve obtained from the model test showed that the structure has large capacity of energy absorption against horizontal forces. The failure mode of the structure is a sort of shear failure and the failure starts at the compression side of the structure.

The results obtained from an FEM analysis sufficiently explains the behavior of the structures used in the laboratory test.

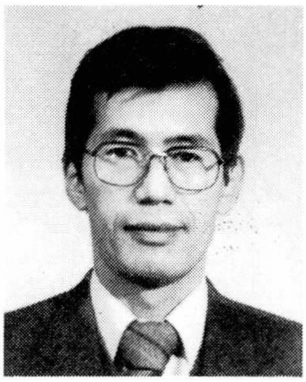
Analysis of Framed Buffer Structure around Bridge Pier

Analyse de la charpente de pare-choc autour de la pile du pont

Analyse von Pufferbau um den Brückenpfeiler

Yosho NAMITA
Dr. Eng. Chief Res. Eng.
Kobe Steel,
Kobe, Japan

Hiroshi NAKANISHI
Dr. Eng. Res. Eng.
Kobe Steel,
Kobe, Japan



Yoshio Namita, born 1934, got his doctorate at Osaka University, Japan. After making researches of bridge engineering in the Osaka University for thirteen years, he started working in the Structural Engineering Laboratory of Kobe Steel, Ltd, where he is now responsible for the research works on structural analyses.

Hiroshi Nakanishi, born 1946, got his doctorate at Osaka University, Japan. For thirteen years he has been making researches on structural analysis in the Structural Engineering Laboratory of Kobe Steel, Ltd. He is now responsible for developing the computer softwares of dynamic behaviour of offshore structures.

SUMMARY

A framed structural system is proposed for a buffer which prevents the damage to a bridge pier by ship's collision. Investigation is made into the load-deformation relation and the energy-absorbing capacity of the structure by means of inelastic large deformation analysis. Numerical results show that, from the viewpoint of energy absorption, the structural system of which truss layers collapse one after another with the headway of ship is more effective than that which suffers the local collapse of structural panels.

RÉSUMÉ

Un système de charpentes est proposé comme structure de pare-choc d'une pile de pont lors de la collision de navires. Des calculs sont effectués à l'aide de la méthode d'analyse de déformation inélastique et de la capacité de l'absorption d'énergie. Le résultat numérique montre que, du point de vue de l'absorption d'énergie, la structure, dont les couches de charpentes cèdent au fur et à mesure que le navire s'enfonce dans la charpente, est plus efficace que celle qui éprouve cumulativement la destruction locale de panneaux structuraux.

ZUSAMMENFASSUNG

Ein Fachwerk als Pufferbau, der gegen den Schaden des Brückenpfeilers durch Zusammenstoß eines Schiffs eingesetzt wird, wird vorgeschlagen. Die Belastungs-Verformungslinie des Pufferbaus und dessen Aufsaugvermögen der Verformungsenergie sind mittels der Analyse von unelastischer, großer Verformung studiert. Numerische Ergebnisse zeigen, daß, vom Standpunkt aus Energieaufsaugen, das Fachwerksystem, dessen Schichten eine nach der andern mit dem Eingreifen eines Schiffs nachgeben, wirkungsvoller, ist als dasjenige, das die Anhäufung des Lokalzerbrechens vom einzelnen Fachwerkfeld erleidet.

1. INTRODUCTION

When a bridge pier is constructed in a sea area, it is possible that accidents such as collision of a ship with the pier occur in stormy or foggy weather, especially under the severe condition such as heavy traffic and rapid tidal current.

As one of the measures to cope with the situation an idea of surrounding the bridge pier with a kind of buffer structure which would minimize the collisional damage to ship's hull as well as to the bridge pier is considered.

In this paper a framed structural system is proposed to be used as a buffer structure which prevents a ship from colliding directly with the bridge pier, and its buffering effect is investigated from the viewpoint of its energy-absorbing capacity.

2. DESIGN CRITERIA OF FRAMED BUFFER STRUCTURE

In case a ship, by accident, collides with a buffer structure surrounding a bridge pier, the following two points with regard to the buffering process should be noted in order to minimize the damage of both ship and pier:
1) The force caused by collision does not exceed the critical value for the collapse of either the ship's hull or the bridge pier,
2) The distance of ship's headway is limited within the value of the depth (from the front to the back) of the buffer structure (See Fig. 1 right).

In other words, the item 1) requires the buffer structure to have a cushioning effect that makes collisional impact small enough not to cause damage to either ship or pier, and the item 2) is for preventing the ship from coming into direct collision with the bridge pier.

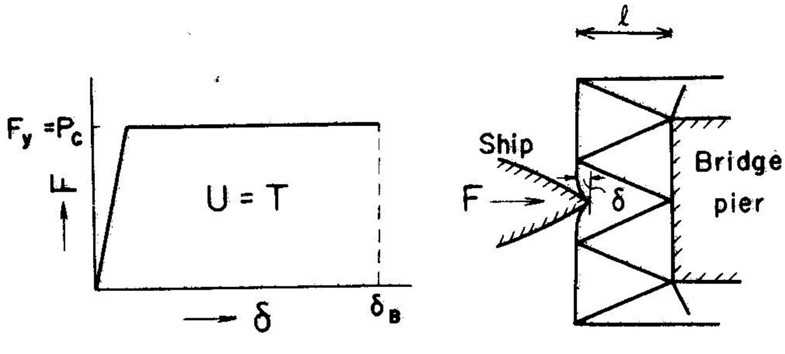


Fig. 1 Load-deflection relation of buffer structure

In general, it is considerably difficult to set up the design criteria for this kind of buffer structure because of the versatility of surrounding conditions such as size, weight and speed of ship, direction of collision, etc. In this paper, however, for the purpose of simplification and thus of making it possible to formulate the design process of the buffer structure, it is assumed that the bridge pier is surrounded by a three dimensional truss-typed framed structure, which is supposed to absorb the kinetic energy of a ship coming into collision as statical strain energy stored in the structural system during deformation.

Fig. 1 right shows a ship coming into collision with a buffer structure, which is gradually deformed and collapsed as the ship makes headway toward the pier. In this process the kinetic energy of the ship is transformed into the strain energy of the buffer structure suffering large deformation. Fig. 1 left shows a schematic load-deflection relation of buffer structure, where F is a force caused by collision and δ is the displacement of the point at which the force F acts. The structure behaves elastically so far as F is smaller than F_y and inelastic deformation occurs when F becomes F_y. The displacement δ increases until it reaches δ_B, the ultimate displacement.

During this process the work done by the external force F is stored in the buffer structure as strain energy U.

In the present analysis the buffering effect of the structure is calculated under the following assumptions:

1) Ship's speed just before its collision with buffer is small and its dynamic effect can be neglected,

2) Mass of the buffer structure is small in comparison with that of the ship and is neglected, and

3) Ship has rigid hull.

Referring to Fig. 1 the necessary condition that the buffer structure functions in full effect is that $F_y \leqq P_c$ and $U \geqq T$. P_c is the allowable maximum impact force which is to be prescribed from the safety of the bridge pier proper and T is the kinetic energy of the ship just before its collision with the buffer. And, at the same time, the length by which the ship's bow gets stuck in the buffer structure at the end of ship's movement should be less than the horizontal depth of the buffer, i.e. $\delta_B \leqq \ell$ (See Fig. 1).

In this paper, a three dimensional framed system, as shown in Fig. 2, is proposed as the buffer structure which is made to satisfy these conditions by the numerical method of analysis calculating the strain energy of the structure.

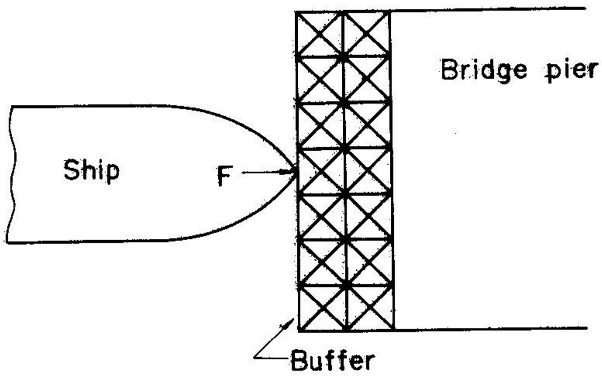


Fig.2 Framed structure for a buffer

3. METHOD OF ANALYSIS

Supposing the process of the deformation of buffer structure is perfectly traced from the beginning of ship's collision to the end of the ship's movement and the load-displacement (F-δ) relation is obtained as shown in Fig. 1, the strain energy U stored in the buffer structure can be calculated by the formula.

$$U = \int_0^{\delta_B} F \, d\delta$$

Therefore, the main purpose of analysis is to obtain the F-δ relationship of the structure at every loaded point.

As is clearly seen in Fig. 1 left, the work done by external force during elastic deformation is very small compared with that of inelastic range, therefore the strain energy stored in the buffer structure is composed, for the large part, of that due to large deformation in inelastic range. Hence, in order to analyze the present problem rigorously, the method of inelastic large deformation analysis is required, which takes fully into consideration the non-linearity of geometrical deformation and of inelastic mechanical properties of material used as well.

The method of analysis adopted in this paper for the purpose of obtaining the load-deformation relationship is the above-mentioned one based on the concept of energy principle.

As mentioned before, the object of the present analysis is a three dimensional truss-typed framed structure and the joints of its members are considered frictionless hinges. But, concerning the numerical examples shown in the following chapter, in addition to such truss-analysis, investigation was made into the behavior of the structures as rigid-jointed frameworks.

The result revealed that the bending stress in structural members made only a little contribution to the total amount of strain energy stored in the structure during its deformation up to collapse, and that a large portion of the total strain energy stored was brought about by the deformation of the structure after the formation of plastic hinges. Taking this result into account, the present analysis is limited to the one for truss-typed structure.

The method of analysis is as follows.

The total potential energy of the structure is generally shown in the form

$$W = \sum_{m=1}^{M} U_m - F^T x \tag{1}$$

where U_m is the strain energy of m-th member, F is the vector of external forces, x is the vector of joint displacements and M is the total number of members. The structure is in a equilibrium state when the total potential energy is minimized.

Fig. 3 shows a schematic illustration of the solution of member force-elongation relation which is expected to be obtained on the basis of the stress-strain relation. In Fig. 3 P_0 and e_0 are a member force and a elongation at the initial state.

Supposing the numerical computation is now under way in its i-th step (See Fig. 3), the strain energy stored in the m-th member up to the present state is expressed as

$$U = U_V + U_C \tag{2}$$

$$U_V = \left(P_i - \frac{E_i A}{L} e_i\right) e + \frac{E_i A}{2L} e^2 \tag{3}$$

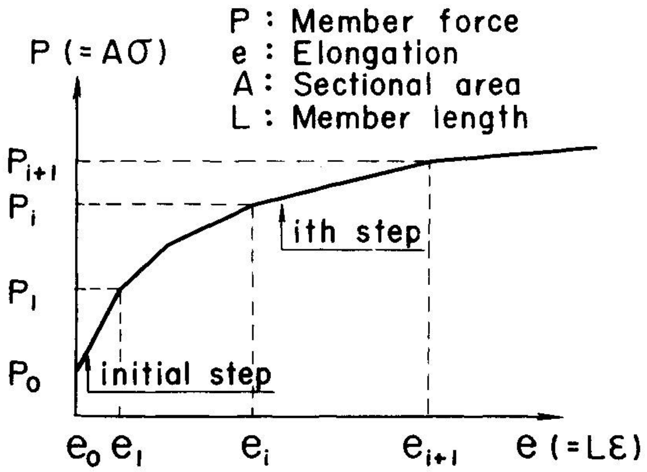


Fig. 3 Load-elongation relation of pin-jointed member

$$U_C = \frac{1}{2}(P_0+P_1)e_1 + \frac{1}{2}(P_1+P_2)(e_2-e_1) + \cdots\cdots + \frac{1}{2}(P_{i-1}+P_i)(e_i-e_{i-1}) + \frac{E_i A}{2L} e_i^2 - P_i e_i \tag{4}$$

where e is the total elongation of the member at an arbitrary state in the i-th step, and A and L are the sectional area and the length of the member respectively. In the initial step Eq. (2) becomes

$$U = P_0 e + \frac{E_0 A}{2L} e^2 \tag{5}$$

(In the above equations suffix m in each term is omitted for simplicity).

The total potential energy W in Eq. (1) is minimized by the conjugate gradient method which is often used in the optimization problems [1], [2].

Now, W is calculated from Eqs. (1) and (2). U_C in Eq. (2), however, can be omitted in the minimizing process, for it does not include any terms of unknown joint displacements. Differentiation of Eq. (1) with respect to displacement x yields

$$R_{xj} = \sum_{m=1}^{N} \{ (P_{mj} - \frac{E_{mi}A_m}{L_m} e_{mi}) + \frac{E_{mi}A_m}{L_m} e_m \} \frac{\partial e_m}{\partial x_j} - F_{xj} \tag{6}$$

where N is the total number of members meeting at joint j. Eq. (6) gives the unbalanced force at the i-th iteration in X-direction of joint j. Similar forms are obtained with regard to ones in Y- and Z-directions.

4. SIMPLIFICATION OF THE PROBLEM

In the analysis of a buffer structure, the movement of the ship which gradually splits up the structural panel should be successively traced. For this purpose it is first necessary to make clear the loading condition, and the following assumptions are made:

1) Ship's bow is a wedge-shaped rigid body,
2) Buffer structure is a truss-typed framework and every load acts only at its joints,
3) The magnitude and the direction of load is dependent on the angle of bow Θ. Friction between the bow and structure is neglected and load F acts in direction normal to the bow (See Fig. 4),
4) Structural panel unit loses its load-carrying capacity when its diagonal members are broken by bow's headway. After the collapse of a panel unit the point of load application shifts its position,
5) Fig. 5 shows the shifting of load-application points as the bow advances. The structural panel unit directly suffering the bow's touching is broken up and immediately disappears. External forces caused by the bow act on the buffer structure in the form of Fig. 5 (b). The diagonal members of panel units are broken up with further headway of the bow (Fig. 5 (c)), and these broken panel units vanish and the structural system will be as shown in Fig. 5 (d).
6) A diagonal member does not resist a compression force at all.

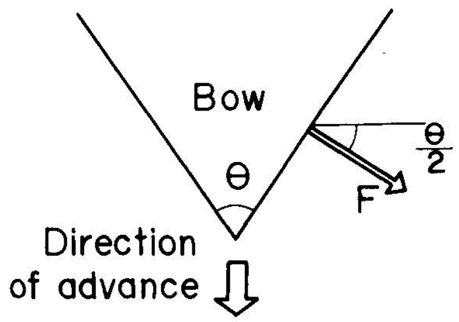


Fig. 4 Load acting on buffer

By the above-mentioned assumption computation becomes simple, and it can safely be said that the assumption causes no problem from the viewpoint of practical design calculation, for the majority of total energy absorbed by the structure is due to the deformation after the very break-up of diagonal compression members.

5. NUMERICAL INVESTIGATION

5.1 Collapse Pattern

Fig. 6 shows a collapse pattern of truss-typed buffer structure. This pattern means that panel units in one layer of the framework are simultaneously broken up and the structure is collapsed layer by layer as ship goes on (Fig. 6).

From the viewpoint of energy absorption, it can be said that this pattern is more effective than that which absorbs energy as the cumulation of the local collapse of each panel unit. Numerical investigations were made into finding out the condition under which the layer-collapse pattern occurs.

The governing factors which determine the collapse pattern of the trusses are the following:
1) Stress-strain relationship of material used,
2) Ultimate strain of material at its breaking point, and
3) Ratio of the sectional area of main (lateral or longitudinal) member to that of diagonal member.

A considerable number of numerical investigations were conducted under the assuption of Fig. 7 which provides the above-mentioned items 1) and 2). From these numerical results the item 3) was investigated.

Generally speaking, the bigger the ratio in item 3) becomes, the more likely the layer-collapse pattern is to occur; the more the number of panel units in one layer of truss-framework is, the more slender the diagonal members should be in order to cause this pattern.

An empirical conclusion was derived from the numerical results: Pattern of layer-collapse occurs when the ratio in item 3) above is larger than n+1, where n means the number of panel units in one layer of the truss.

Fig. 8 shows an example of the numerical result of a single-layer model having five panels.

Furthermore, the results with regard to multi-layer models which satisfy the above-mentioned empirical rule showed that, even when the first layer which suffers direct loading

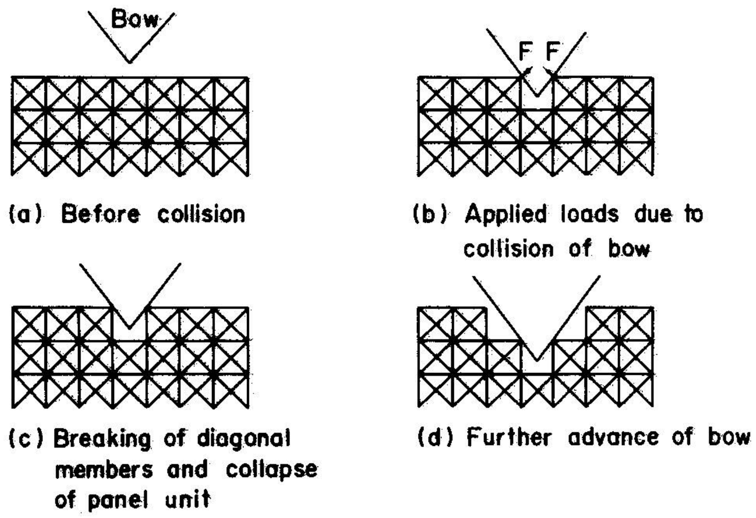


Fig. 5 Headway of ship and collapse of structural panel

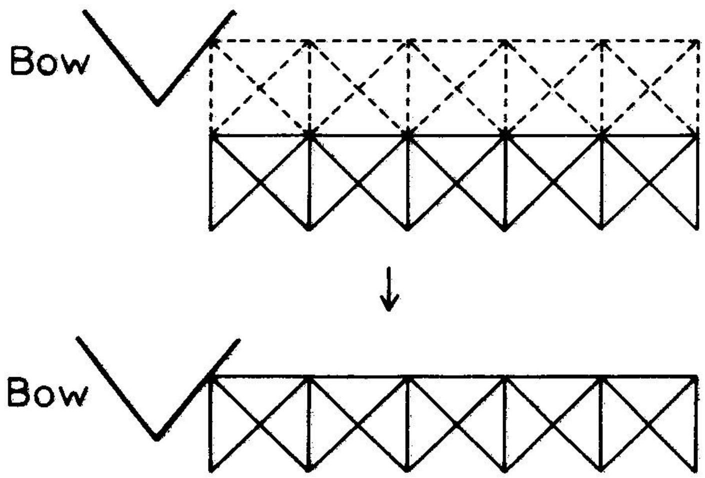


Fig. 6 Collapse pattern

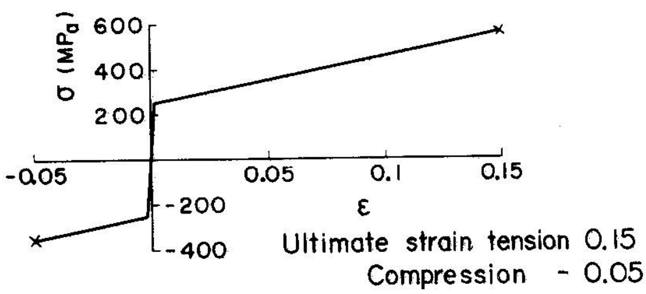
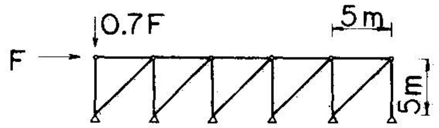


Fig. 7 Stress-strain relation

was mown down, member forces in remainder structural layers still remained in elastic range.

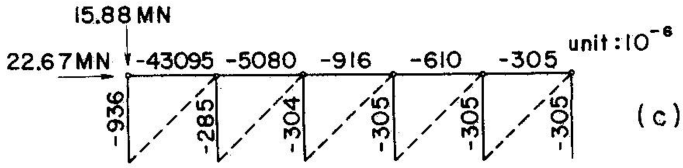


(a)

Structural model

	Sectional area
Chord member Vertical member	900cm²
Diagonal member	150cm²
Ratio of sectional area	6

(b)



(c)

Loads and strains when diagonal members are broken

Fig. 8 Example of collapse mode of a truss layer

5.2 Numerical example on multi-layer truss system

Here, a multi-layer structural model in Fig. 9 (a) is taken for numerical example and the behavior of its layer-by-layer collapse by ship's headway is examined. When the first layer has been overall collapsed, the structural system becomes a different one as shown in Fig. 9 (b) and at the same time a load application point shifts its position. This process goes on successively layer by layer and a load-deflection curve is drawn corresponding to each layer's process up to collapse. Fig. 10 shows the load-deflection curves of the structure in Fig. 9 (a). In this figure are drawn four curves one over another each of which shows the collapsing process of corresponding layer of the truss. The strain energy absorbed by each layer is obtained through integration of the load-deflection curve and is shown in the figure. As is seen in Fig. 10 four load-deflection curves resemble one another in their shape and the amount of energy absorbed by each layer is nearly equal to that of the other. In this example it can also be shown that, when a certain layer is in process of collapsing, the members of the remaining layers are still in the elastic range.

From the result of this numerical example, it can be said that the total amount of absorbed energy in multi-layer truss-typed structure is estimated by superposing the ones in each layer, so far as the structural system shows a layer-by-layer collapsing pattern.

The buffering effect of the above-mentioned structure, or the safety of bridge pier from ship's collision, is to be checked through confirming that the maximum reaction caused in pier is smaller than the allowable force and that the kinetic energy of the ship just before its collision is not larger than the energy absorbed in buffer structure during its collapsing process.

6. CONCLUSION

The results obtained in this paper are summarized as follows:

1) The computer program of inelastic large deformation analysis for trussed framework based on energy method was developed and applied to the analysis of buffer structure. The buffering effect of the structure is evaluated with the amount of strain energy which is absorbed in the structure during the process of collapse,

2) The possibility of finding out a multi-layered framework which showed the pattern of collapsing layer by layer with the penetration of ship was discussed. By employing such a system as a buffer structure, the buffering effect is considerably enhanced, and at the same time, design calculation is simplified on account of the availability for the superposition of energy absorbed in each structural layer.

In this paper the scope of the research is limited within the above-mentioned item. In practical construction, however, there remain many problems unsolved: for example, the method of installation at bridge pier, dynamic effect of wave forces, fatigue of structural members, corrosion prevention etc. Further investigations are expected to cope with these problems.

REFERENCES

1. NAMITA, Y., T. SHINKE and K. HIRONAKA, Computer analysis and model experiment of cable structures, Preliminary Report of Ninth Congress of IABSE, May 1972

2. KOWALIK, J. and M.R. OSBORNE, Methods for Unconstrained Optimization Problems, Elsevier, New York, 1968

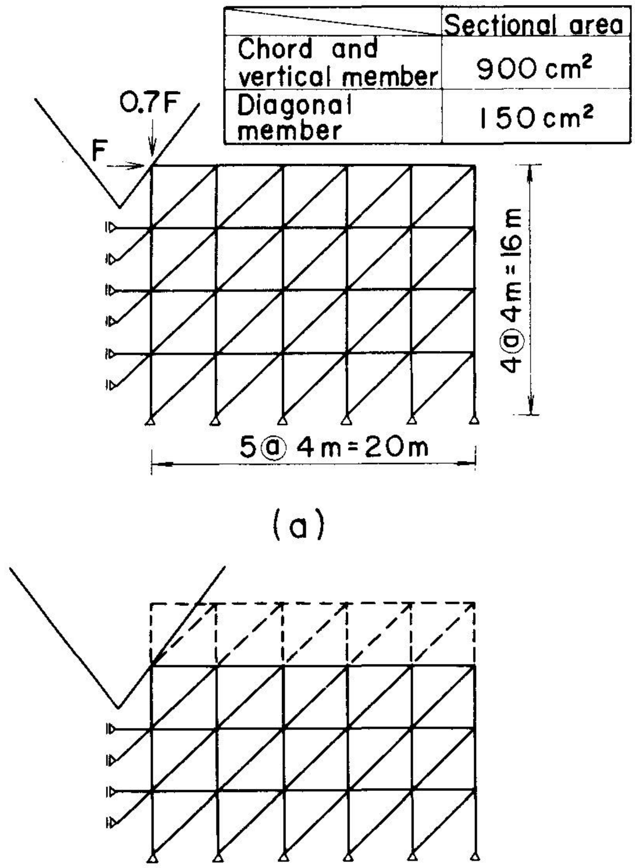


Fig. 9 Example of multi-layer truss

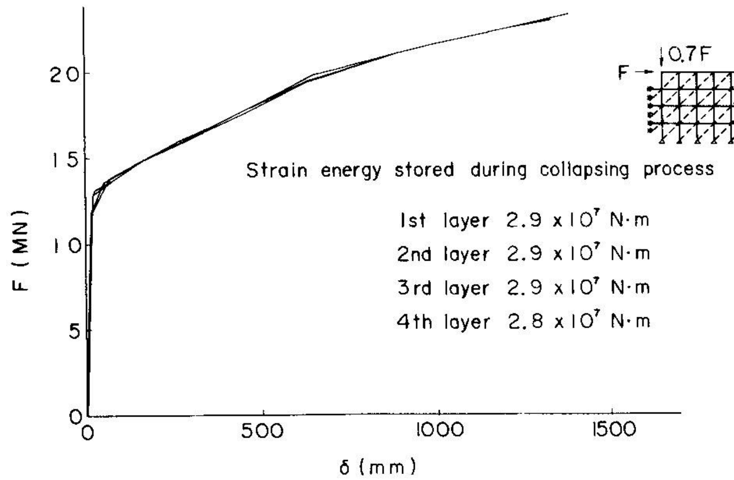


Fig. 10 Load-deflection diagram of multi-layer truss

Pier Protection by Man-Made Islands for Orwell Bridge, U.K.

Protection des piles du pont de l'Orwell (GB) à l'aide d'îles artificielles

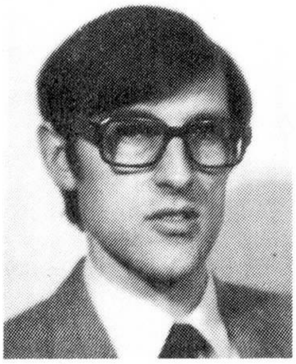
Künstliche Inseln zum Schutz der Pfeiler der Orwell Bridge (GB)

M. S. FLETCHER
Sir William Halcrow & Partners
Swindon, Wilts., U.K.

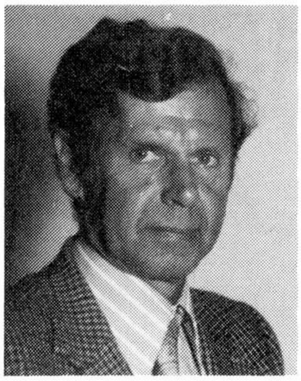
R. W. P. MAY
Hydraulics Research Station Ltd.
Wallingford, Berksh., U.K.

J. A. PERKINS
Hydraulics Research Station Ltd.
Wallingford, Berksh., U.K.

Malcolm Fletcher, an Associate of Sir William Halcrow & Partners, Consulting Engineers, has been Project Engineer for Orwell Bridge from inception to completion. He has 20 years experience of the design and construction of major bridges in UK, Pakistan and Libya.



Richard May is a Senior Scientific Officer at Hydraulics Research Station Ltd, and joined in 1974 after four years with a firm of Consulting Engineers. He has worked on projects involving urban drainage, fluvial hydraulics and offshore engineering.



Allen Perkins worked on a variety of civil engineering projects in UK and New Zealand, before joining Hydraulics Research Station in 1965. His work there has been on urban drainage, hydraulic structures and rivers.

SUMMARY
The paper describes the investigation, design and model testing of man-made islands for protecting bridge piers against impact from ships. Particular reference is made to the procedure adopted for Orwell Bridge, England, where islands of this type have been built to provide protection against ships of up to 11.000 tons loaded displacement.

RÉSUMÉ
L'article décrit la recherche, la conception et les essais sur modèle réduit d'îles artificielles pour la protection de piles de ponts contre l'impact des bateaux. Les principes adoptés pour le pont sur l'Orwell, en Angleterre, sont décrits où de telles îles ont été construites pour protéger le pont contre des bateaux de 11.000 t.

ZUSAMMENFASSUNG
Der Beitrag behandelt die Untersuchung, Modellversuche und den Entwurf künstlich hergestellter Inseln, die als Schutzeinrichtung für Brückenpfeiler gegen Schiffsanprall dienen. Insbesondere werden die Schutzmaßnahmen für die Orwell-Brücke in England beschrieben, wo Inseln dieser Art als Schutzeinrichtung gegen 11.000 Tonnen Schiffe gebaut wurden.

1. INTRODUCTION

1.1 There are now sufficient examples of the consequences of a ship striking a bridge pier to justify making protection of the piers a fundamental design requirement for any new bridge over a navigational channel.

1.2 The type of protection adopted depends upon the size and speed of the ships passing the bridge site, the profile of the river or sea bed, the arrangement of the piers within the water and the cost of the pier protection system in relation to the cost of the bridge.

1.3 Protective islands form a cheap and relatively maintenance free method of protecting piers, particularly where material to build the islands is readily available and where the profile of the river bed is favourable. These two factors determined the adoption of islands around the piers of the recently completed Orwell Bridge.(1)

2. DEGREE OF PROTECTION ADOPTED

2.1 In UK the most recent significant accident was in 1960 when two barges demolished a pier and two spans of the 80 year old Severn Railway Bridge. In the discussions on this accident Boyd (2) stated 'If there were water at a danger spot, ships would go there when they were out of control; they were like that!'.

2.2 For Orwell Bridge, which has eight piers in the approach channel to the port of Ipswich, it was decided that if it were possible for a vessel to approach a pier then that pier should be protected against such a risk, however small the statistical probability of an accident might be.

3. THE DESIGN SHIP

3.1 In order to design a protection scheme for a bridge crossing it is first necessary to define the type and size of ship that presents the greatest potential danger to the bridge : this is termed the Design ship.

3.2 Limitations on the dimensions of the Design ship may be created by the depth of water in the navigational channel; the space required upstream of the crossing to turn the ship around; the vertical clearance after the bridge has been built.

3.3 All states of the tide should be considered for both loaded and in-ballast conditions. For a given draught, a ship in ballast may have a greater displacement than a smaller ship which is laden; thus ballast conditions may provide the most critical design case for piers in shallow water.

3.4 For bridges over estuaries or navigable inland waterways the likely speed of ships passing the bridge can be obtained from pilots or regular users of the waterway.

3.5 For the eight river piers of Orwell Bridge it was necessary to consider a range of Design ships. The heaviest ships which could approach a particular pier were tabulated and then rationalised into three types:

(1) 11,000 ton displacement with 6 m draught.
(2) 9,000 ton displacement with 5 m draught (corresponding to vessel (1) in ballast).
(3) 1,000 ton displacement with 2 m draught.

These were compared with a survey of the records of ships using the port upstream of the bridge.

3.6 The maximum speed thought by users to be reasonable for the reach of river in which the bridge is located is 8 knots (4.1 m/sec). It was assumed that a ship out of control was travelling at this speed when it approached a pier.

3.7 An analysis of photographs of various accidents confirmed that, for the purpose of design, the ship should be assumed to approach any bridge pier at 90^{o} to the line of the bridge.

4. PROTECTION SYSTEMS

4.1 Small fendering systems used in ports are designed to avoid damage to the ship and to themselves. The designer of a major bridge must ensure that no significant damage occurs to the bridge; he accepts that both the protection system and the ship may be substantially damaged in an accident.

4.2 Amongst the various options for protection are:-

(1) designing the pier to withstand the impact by providing sufficient mass or structural strength;
(2) providing independent fendering systems for all angles of approach;
(3) providing large independent sheet piled dolphins at the upstream and downstream ends of each pier;
(4) supporting nets or hawsers by means of independent piles;
(5) forming man-made islands around each pier using granular materials.

4.3 The relatively low cost of solution (5), together with its ease of construction, ease of repair and freedom from maintenance, make it an attractive answer for many shallow water bridge piers.

5. PRELIMINARY DESIGN OF PROTECTIVE ISLANDS

5.1 The materials available for construction were investigated, and it was decided that a well graded granular material with less than 10% of material passing a B.S. 200 sieve (0.07 mm) would be appropriate. Protection against wave damage is required, and may be provided by rock or by armouring units backed by a graded stone filter. As there was no rock in the region of the bridge, precast concrete tripods were adopted at Orwell.

5.2 A protective island needs to be large enough to bring a ship to rest before its bow strikes the pier of the bridge. The design calculations for the islands were based on a consideration of the energy changes that occur during an impact. As a result of the impact the initial kinetic energy of the ship is dissipated or redistributed in some or all of the following ways:-

SHIP (1) change in potential energy of the ship due to change in the vertical position of its centre of gravity.
(2) crushing of the hull of the ship.
WATER (3) change in potential energy of the water displaced by the ship.
(4) generation of water waves and turbulence.
ISLAND (5) change in potential energy of island material.
(6) displacement, shear and compaction of the island material.
(7) friction between the ship and the island.
(8) generation of shock waves within the island.
(9) crushing of particles of beach material.

5.3 The inclusion of many of these factors in hand calculations proved difficult and so some simplifying assumptions were made. Bouvet's (3) analysis of tanker collisions and groundings indicates that much less damage occurs when a ship grounds than when it collides. In 69% of the groundings studied the plates of the ship were damaged to a depth of less than 0.5 m. It was therefore decided that the crushing of the hull of the ship (item (2) above), which depends upon the type of construction, would be ignored in the design calculations.

5.4 When a ship decelerates the inertial force due to the added mass of the water tends to oppose the slowing down of the ship. However in a sudden impact only a small amount of kinetic energy will be transferred from the water to the ship, the remainder being dissipated by turbulence and waves; therefore item (4) above was not considered in the calculations.

5.5 Neglecting these two items the energy balance becomes

$$KE_s = PE_w + PE_s + IE$$

where KE_s is the kinetic energy of the ship

PE_w is the change of potential energy of the water

PE_s is the change of potential energy of the ship

IE is the impact energy, equal to the total work which the ship does as it penetrates the beach, the sum of items (5) to (9) above.

6. GEOMETRY AND DESIGN CALCULATIONS FOR ISLANDS OF ORWELL BRIDGE

6.1 The islands were assumed to have side slopes of 1 vertical to 3 horizontal and flat tops coinciding with the level of High Water Spring Tide (+2.0m AOD). The three design vessels given in 3.5 were assumed to be travelling at 4 m/s when they struck the island.

6.2 Water levels above +2.0 m AOD have occurred in the tidal river during storm surges, and three water levels, +3.5 metres, +2.0 metres and +0.5 metres were chosen.

6.3 Two limiting cases were studied. In the first it was assumed that the island material was so rigid that the ship would be brought to rest by rotation about its centre of gravity and by friction between the hull of the ship and the beach material. The coefficients of friction adopted were 0.6 for steel hulls on dry granular material and 0.4 for steel hulls on wet granular material.

6.4 The second limiting case assumed that no rotation of the ship would take place and that all the energy would be dissipated by the ship ploughing into the material of the island. No account was taken of the resistance of the armoured layer on the face of the island.

6.5 The required size of the protective islands depends upon how far the ship can penetrete before coming to rest. For the limiting cases considered above, it was calculated that the bows should not penetrate more than 10 m into the horizontal section of the island. The prow of the ship was assumed to be 5 m forward of the point in the beach to which the bows had penetrated. The required horizontal distance between the top of the 1 in 3 slope and the bridge pier was therefore chosen to be 15 m.

6.6 A literature survey in 1976 did not provide sufficient data against which the various assumptions in the design calculations could be checked, so it was decided to commission a model investigation.

7. OBJECTIVES OF MODEL INVESTIGATION

7.1 The purpose of the study was to determine the size of the islands required to protect the piers of Orwell Bridge by:-

(1) modelling the proposed design of beach described in 6.1 and the three types of Design vessel described in 3.5.
(2) carrying out a series of tests at water levels of +0.5 m, +2.0 m and +3.5 m AOD.
(3) recording and analysing the movement of the ship in each test, and measuring the final position of the ship together with the shape of the impact hole it produced.

(4) determining from these results the maximum distance that a vessel could penetrate into one of the islands.

7.2 The model investigation was carried out by the Hydraulics Research Station, UK, early in 1978 and the results are published in the Study Report (4).

8. CHOICE OF MODEL SCALES

8.1 The relevant scaling laws for the model tests were obtained by considering the forces acting on the ship during its impact with the protective island. Analysis (4) indicated that the relative magnitudes of the inertial, gravitational and buoyancy forces would be reproduced correctly by a Froudian scale model in which the size of the beach material was determined by the linear scale of the model. It is also important to scale the resistance of the beach material correctly, because the path that a ship follows during an impact depends upon the magnitude of this resistance relative to the difference between the gravitational and buoyancy forces. The beach resistance can be divided into a static component and a dynamic component.

8.2 The static component is the force which the island would exert on a ship during a very slow impact, and depends upon the static shear strength of the material and the coefficient of sliding friction between the ship and the material. From Coulomb's law it can be shown that the ratio of the static resistance to the inertial force of the ship will be given correctly by a Froudian scale model provided the beach material is non-cohesive and the particles are geometrically similar to those in the prototype.

8.3 The dynamic component depends upon the relative incompressibility of the beach and becomes more important as the speed of the impact increases. The requirements for similarity of the dynamic resistance tends to conflict with the requirements for the other forces considered previously.

8.4 In the present study the tests were carried out according to a Froudian scale using a model cargo ship having an overall length of 1.66 m and a beam of 0.21 m. This model was able to represent the 11,000 and 9,000 ton Design ships at a scale of 1:100, and the 1,000 ton Design ship at a scale of 1:50. Fine sand was used for the model material in the protective islands, and the required gradings were obtained by scaling the grading of the prototype material according to the appropriate linear scale. However in both the 1:100 and 1:50 scale models it was necessary to make the materials somewhat too coarse at the fine ends of their ranges in order to ensure that they would act non-cohesively.

9. EXPERIMENTAL PROCEDURE

9.1 The tests were carried out in still water in a flume measuring 20 m long x 2.4 m wide. The model ship was driven by twin propellors powered by an electric motor, and was guided along the flume by twin wires to which it was attached at bow and stern. The protective island was formed in the dry by compacting the material in thin layers so as to obtain the voids ratio expected in the prototype islands.

9.2 The impact of the ship with the beach was recorded by means of a video camera viewing through a transparent window in the side of the flume. Replaying the video recordings frame-by-frame provided information, at intervals of 1/50 second, about the movement of the ship during the impact. The position of the boat at any instant was determined from the position of two pointers on the boat relative to a grid scale in front of which the boat was arranged to pass.

9.3 A separate series of tests was also made to estimate the static resistance of the beach material during a very slow impact. A horizontal wire was attached to the bow of the ship, and used to keep it just in contact with the beach whilst floating freely. A force was then applied to the wire causing the bow of the ship to penetrate slowly into the beach. The force was increased in steps and a video recording made of the position of the ship when it had come to rest after each increase in load.

10. TEST RESULTS

10.1 Analysis of the video recordings of each test enabled measurements to be made of the speed of the model ship prior to impact, and of the horizontal, vertical and angular positions of the ship during and after the impact. The primary result from each test was the horizontal distance that the ship penetraded into the protective island, and some typical results are shown in Fig. 1.

10.2 The tests showed that the distance penetrated by a given vessel increases as its speed is increased and as the water level relative to the top of the beach is increased. It was also found that the shallower draught of the 1,000 ton ship enabled it to penetrate further than the 11,000 and 9,000 ton ships under similar conditions. As a result the crest level of the prototype beaches was increased by means of a sloping section with a gradient of 1:29; the final design of the beaches is shown in Fig. 2.

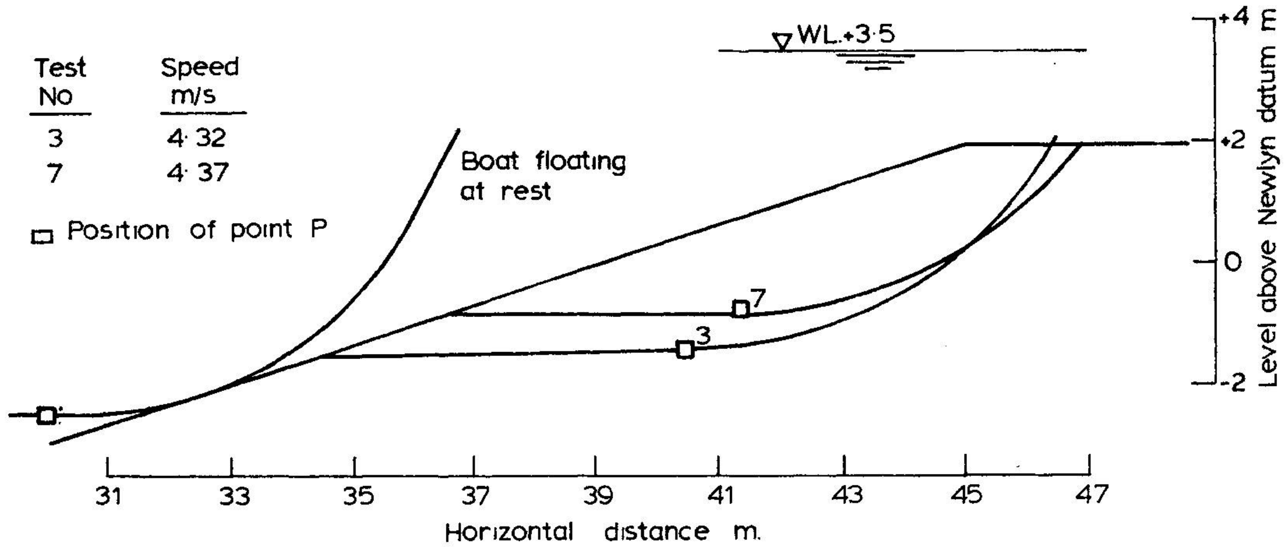


Fig. 1 Position of boat after impact 11,000 tons

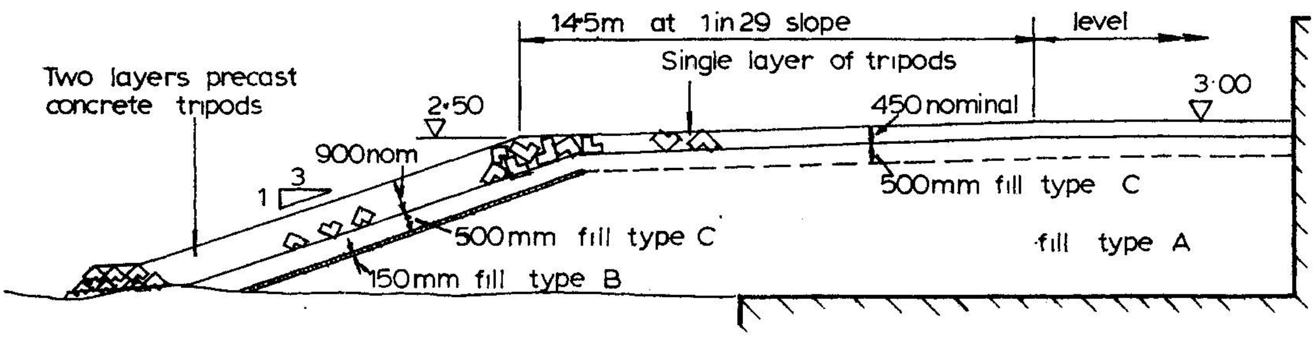


Fig. 2 Construction details of islands

10.3 The measurements were also analysed in terms of the approximate energy balance described by Eqn (1). Data from the video recordings were used to calculate the amount by which the ship was lifted and rotated by its impact with the island; knowing the cross-sectional shape of the ship then enabled $PE_S + PE_W$, the change in potential energy of the ship and the surrounding water, to be calculated (4). The results showed that, depending upon the test conditions, $PE_S + PE_W$ accounted for between about 3% and 14% of the initial kinetic energy KE_S of the ship, and that therefore the majority of the energy dissipated during the impact was absorbed by the island.

10.4 As described in 9.3 separate tests were carried out to measure the static resistance which the protective island provided to the penetration of the ship. Calculations of the work done in overcoming this resistance showed that it was equal to about 75% of the energy IE (calculated from Eqn (1)) required to produce the same penetration in an impact test. This suggests that the static resistance of the island was considerably more important than the dynamic resistance in bringing the ship to rest.

10.5 The results of the tests may be subject to some scale effects, because in the model the hull of the ship was too strong while the dynamic resistance of the material in the island was probably too high. However these two sources of error will tend to balance each other in terms of the distance that the ship penetrates into the beach.

11. CONSTRUCTION COST

11.1 The eight protective islands have been built to the arrangement shown in Fig. 2 for a total cost of £950,000. This cost includes the 45,000 precast concrete tripods used as armouring units.

12. CONCLUSIONS

12.1 The model tests confirmed that artificial islands can provide an effective means of preventing collisions between ships and the piers of a shallow water bridge.

12.2 The construction of the islands, using granular material protected by precast concrete armouring units, has been a relatively straightforward and cheap process. The future maintenance of the islands should be minimal.

13. ACKNOWLEDGEMENTS

Orwell Bridge was constructed for the Department of Transport. Consulting engineers and designers of the bridge are Sir William Halcrow & Partners, London. Model testing was carried out by Hydraulics Research Station Ltd, Wallingford, England.

REFERENCES

1. FLETCHER M.S., Orwell Bridge. IABSE Structures C-16/81, February 1981.
2. BOYD G.M., Discussion on paper by P. Mason. The Structural Engineer, No 10 vol 41, October 1963.
3. BOUVET D.M., Preliminary analysis of tanker collisions and groundings. U.S. Coast Guard, January 1973.
4. HYDRAULICS RESEARCH STATION, Orwell Bridge: Hydraulic Model Study of protective beaches to piers. Report Ex 837, November 1978.

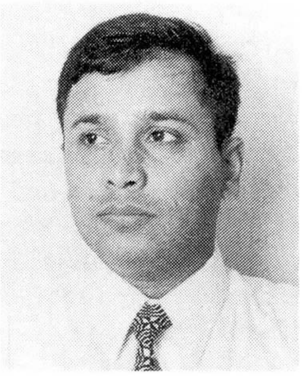
Leere Seite
Blank page
Page vide

Fenders for the Zuari Bridge in Goa

Défenses pour le pont de la Zuari à Goa

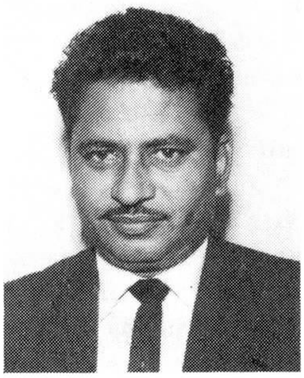
Stoßfänger für die Zuari-Brücke in Goa

G. R. HARIDAS
Technical Manager
Gammon India Limited
Bombay, India



Haridas, born 1940, got his Civil Engineering degree at the University of Bombay. He has been actively associated with the design of civil engineering structures such as bridges, hydraulic structures and industrial structures. He has been responsible for the design of prestressed concrete fertilizer silos, long-span cantilever bridges including Zuari Bridge.

R. C. HIRANANDANI
Project Manager
Gammon India Limited
Goa, India



Hiranandani, born 1927, got his Diploma in Mechanical Engineering from Karachi. For the first 14 years he worked with a Partnership firm of Hochtief, Germany and was working at Kandla Project, dealing mainly with foundation work. Joined Gammons in 1963 and has been working on various prestigious projects and is at present posted in Goa.

SUMMARY
This papers describes the design and construction of the bridge across the river Zuari in Goa, a Union Territory of India. This prestressed concrete long-span bridge was an urgent need of the area as there was no through road communication to Panjim - the capital and Margao - the commercial city of the territory. The only available mode of communication, viz. ferry crossing of the heavy road traffic with loaded trucks was dangerous as it had to negotiate the river plying with heavily loaded ore - barges. The paper highlights the fendering system provided for the bridge, and which is a novelty of its kind and has been adopted for the first time in India.

RÉSUMÉ
L'article décrit la conception et la construction du pont sur la rivière Zuari dans le territoire de Goa, en Inde. Ce pont, en béton précontraint et d'une grande portée, était vital car il n'existait aucun axe routier de pénétration allant jusqu'à Panjim, la capitale, et jusqu'à Margao, le centre commercial du territoire. Le seul moyen de communication possible, c'est-à-dire le passage en bac de l'importante circulation routière constituée de poids lourds, présentait des dangers car il empruntait la rivière encombrée de péniches. L'étude met en lumière le système de défenses conçu pour le pont, système d'un type original adopté pour la première fois en Inde.

ZUSAMMENFASSUNG
Der Artikel beschreibt den Entwurf und die Konstruktion der Straßenhochbrücke über den Zuari-Fluß in Goa, einer Provinz in Indien. Diese vorgespannte Betonbrücke mit langer Spannweite wurde dringend benötigt, da es keine durchgehende Straßenverbindung nach Panjim, der Haupstadt, und nach Margao, dem Handelszentrum der Provinz, gab. Die einzige Verbindung, eine Fähre für den schweren Straßenverkehr mit beladenen LKWs, mußte den Fluß mit schwer beladenen Erzkähnen teilen. Der Artikel stellt das Stoßfängersystem für die Brücke dar, das eine Neuheit auf diesem Gebiet darstellt und erstmals in Indien eingesetzt wird.

1. THE TERMS OF REFERENCE

1.1 Location

Goa, a Union Territory on the West Coastal of India is a picturesque land strewn with silvery beaches, palmfringed coastal line, green paddy fields and a virdant land of natural resources, famous churches and ancient temples and populated by simple joe de vivre people with harmoneous blend of Hindu and Western cultural. The area has become a great tourist attraction with people coming from all over the world to enjoy sunbathing and eating savory, hot Goan sea foods. This land is cut across by two rivers viz. Mandovi and Zuari whose hinderland is famous for iron ore. The river Zuari separates Panjim its capital with Margao the commercial city of Goa, leaving no through road communication thereby forcing ferry services for moving the traffic across the river. Exporting iron ore demands barge traffic for transportation upto Maramgao Port as easiest and economically more viable proposition. The river has, therefore, a very heavy navigational traffic both along and across the river plying with heavily ore - laiden barges upto 1000 tonne displacement as also the ferry traffic carrying loaded trucks and transport vehicles. With the industrial boom in the area and rising tourist traffic for the users of the road, ferry crossing has become increasingly hazardous in a river where loaded barges are plying day in and day out all throughout the year. A high level road bridge which could allow the navigational traffic of ore - carrying barges to pass underneath was therefore the pressing need of the day.

1.2 Planning

The bridge presently under construction caters for two numbers of navigational traffic lanes each of 55.0 M with 13.70 M clear head room above the highest tide level of R.L + 2.80 M, one for upstream and one for downstream traffic. The bed level of the river in the navigational channel is around R.L. - 6.70 M at the deepest portion. Though the river flow is affected by tidal variations because of sandy strata met at the location, a scour upto R.L. - 20.00 M has been estimated based on the velocity of current during the monsoon floods, once the bridge foundations are executed.

The navigational traffic lanes demanded a long span prestressed concrete bridge, with caisson foundations to sustain the effect of impact due to accidental collision of barges during the adverse monsoon conditions and night traffic under poor visibility. To protect the foundations and thus the bridge, a positive system of fendering arrangement was a necessity.

2. GENERAL ARRANGEMENT

2.1 Layout

The final proposal accepted for the bridge has an overall length of 807 M consisting of 627 M length of main bridge over the river portion and a viaduct of 180 M on the land portion of the Agacium bank. The main bridge has been provided with four intermediate spans of 122 M each and two end spans of 69.5 M, and the viaduct portion has five spans of 36.00 M each. The bridge caters for a 7.5 M clear roadway and 1.5 M wide footpath on either side with arrange ments to carry 150 mm dia. water pipe lines, telephone lines and electrical cables for lighting the bridge.

The central two spans ensure main traffic while adjacent spans light river traffic. Though the span arrangement and the deck structure (see figure 1) provides sufficient clearances for such navigational traffic, the central 488 M portion of the main bridge deck has a vertical curvature with end spans provided with a 2.5 percent gradient which continues over the viaduct portion as well.

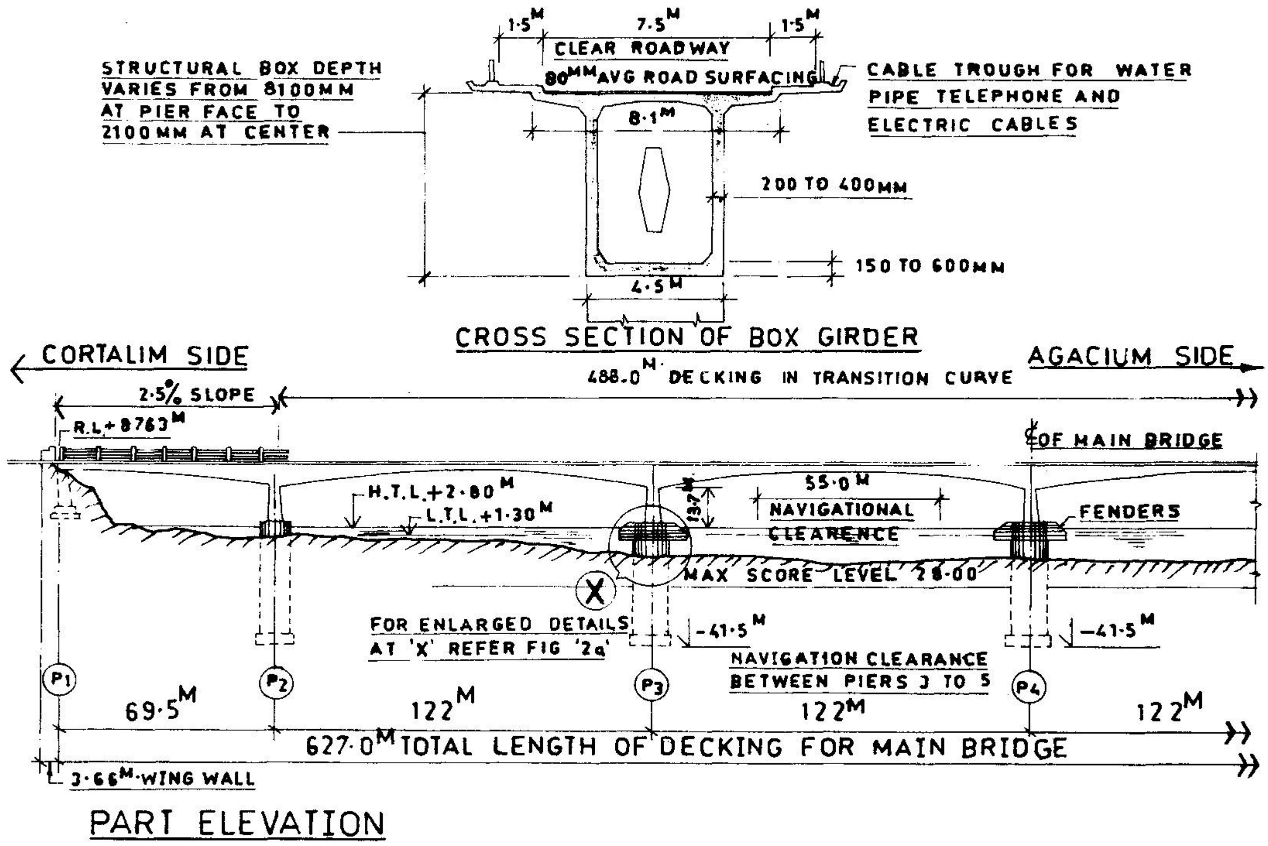


Fig. 1 Zuari Bridge in Goa. General Arrangement

2.2 Superstructure

The superstructure of the main bridge consists of a single cellular prestressed concrete box of overall width of 4.5 M over the webs and 8.1 M at the deck level and has a variable depth ranging from 8.1 M at the pier face to 2.15 M at the mid span, where the mating cantilever arms have been connected through cast steel pendulum bearings. The end cantilever arms support reinforced concrete twin girder decking of 19.5 M each. The box deck is cast-in-situ in segments by cantilever construction system using travelling gantries. The viaduct portion has a box deck with an uniform depth of 2.15 M. The prestressing force is generated by Freyssinet 2 -12 ø 7 mm H.T. cables stressed simultaneously from either side.

The foundations of the main bridge having fenders, consist of single cellular reinforced concrete caissons of about 39 M depth, with a minimum grip length of 21.5 M below the worst anticipated scour level of R.L. - 20.0 M. The caissons have an external diameterof 9.45 M and a thickness of 1.2 M at the top which is increased to 1.425 M below the scour level. The cellular reinforced concrete piers are monolithic with the superstructure and have rectangular shape at top widening to octagonal at the base so as to sit squarely on the caisson shell below.

2.3 Fenders

The ore laiden barges are likely to collide head on with the pier in adverse weather. This called for a system which besides withstanding the impact would protect the bridge foundations and still be available for rectification for subsequent use. In the scourable river, sheet pile fendering was found to be unsuitable; besides, if provided would demand heavy maintainance expenditure due to aggressive corrosive atmosphere prevailing. Reinforced concrete fenders were therefore accepted as a better proposition.

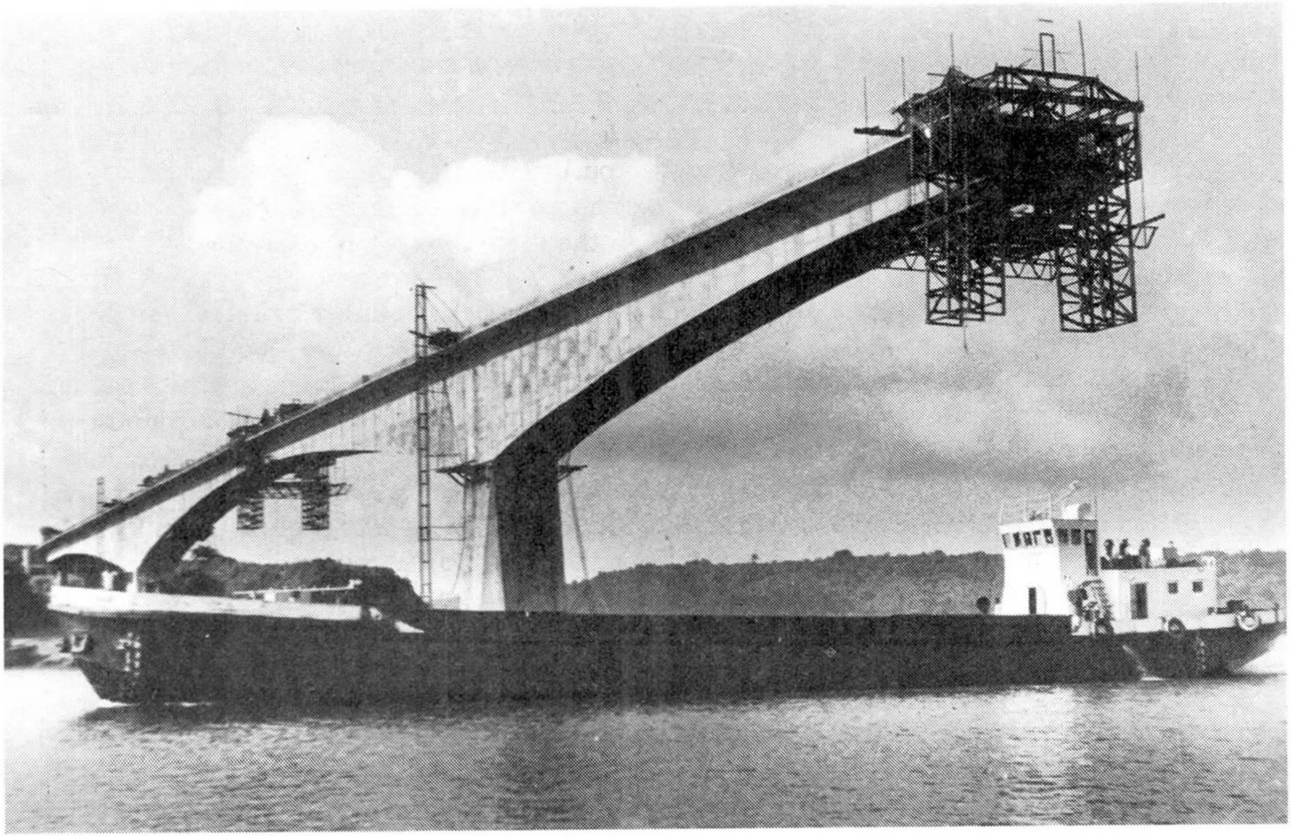
3. DESIGN CONSIDERATION

3.1 Design Data

The barges upto 1000 tonnes displacement (see photograph 1) are expected to ply downstream at 6 knots (3.2 M/sec. absolute) in the river flowing at a velocity of 1.8 M/sec. The fenders are, therefore, required to sustain a static force due to impact of these barges when they collide head on at H.T.L.

The particulars of barges plying are as under :

Displacement weight	:	1000 tonne
Length of the barge	:	44.2 M
Breadth of the barge	:	10.0 M
Depth of the barge	:	2.5 M
Draft	:	2.1 M
Thickness of plates of the barge	:	12 MM



Photograph 1: Showing empty barge plying upstream of the river with superstructure behind under construction.

3.2 General Scheme

It was felt desirable to plan the fendering system on the basis of two considerations :

(a) Psychologically - by devicing a system which would deter the captain of the barge from head on collision for the fear of destroying of the barge itself.

(b) Technically - by designing the fender in such a way that it would transfer least amount of force to the foundations and be capable of rectification if disturbed.

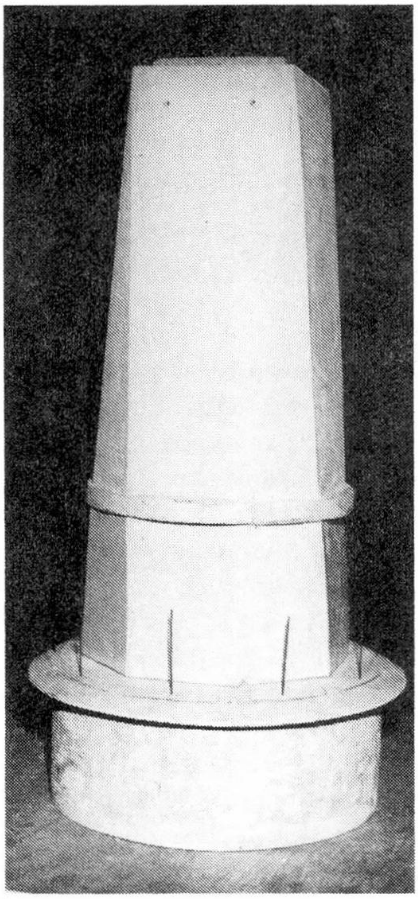
The scheme envisaged provision of ten vertical fins stiffened by four horizontal diaphragms projecting from cellular cofferdam type protective fender wall (see figure 2). The outline of the fins is so shaped and the diaphragms so placed that hull of a normal barge would collide on a larger area of fins or on two or more diaphragms simultaneous. It is envisaged that these concrete fins covered with steel armour plate at the tips would penetrate a colliding barge and tear its plate thus bring the barge to rest by dissipation of energy. To ensure only a reduced impact force is transferred to the foundations, the fender was also made discontinuous from the caisson foundation, so that, in the event of the barge collision the fender as well as the barge would move a certain distance, before the projecting portion of the fin(s) below R.L. - 1.30 M would abut against the caisson (or well) cap and stop further movement. The friction force generated between the sliding surfaces is expected to absorb most of the energy. The fender walls have been provided with 8 - 100 mm dia. holes for lifting the entire fender work from the pier head above for repairs in case of major damage. The pins provided along the inner periphery are expected to help guide down the fender during lowering once the rectification is over (see photographs 2 to 4) to ensure even seating over the match-cast surface of the cap. These pins project only 150 mm above the collar just enough for handling (the pins seen in the photograph Nos. 2 and 3 are only for identification) and would also the sharing some quantum of horizontal force incidentally.

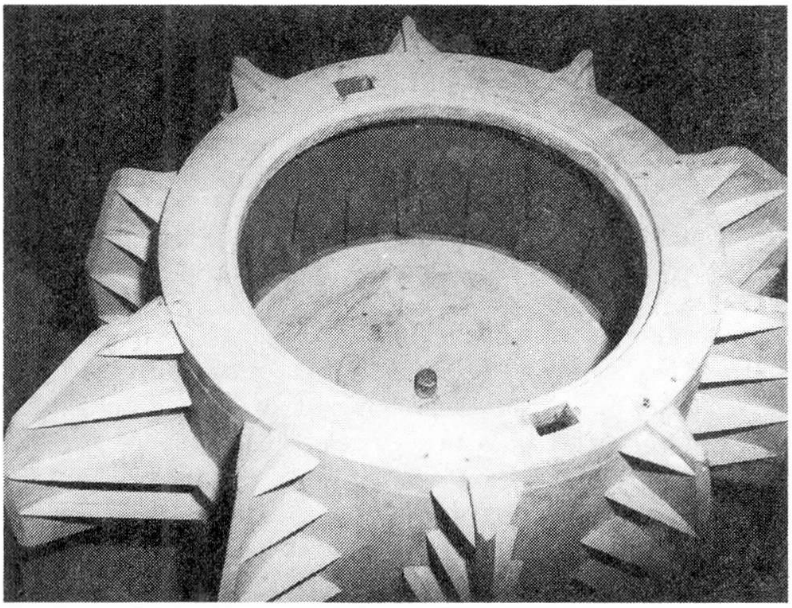
3.3 Final Proposal

Original scheme envisaged supporting the fenders on the sloping portion of the well cap (see photograph 2) which projects beyond the outer face of caisson. This was subsequently made horizontal as the construction scheme involved casting of the fender wall before final sinking of the caisson. Besides, the construction of sloping cap to line and level would have posed difficulty. The fender wall (see figure 2) was cast-in-situ on two layers of tarfelt to ensure water tightness for permitting casting of pier in dry condition later on, once the caisson is sunk to final depth and plugged. The reinforced concrete diaphragms are adequately stiffened by embedding structural steel channels, and 12 mm thick mild steel armour plates outside along the bevelled edges.

3.4 Model Studies

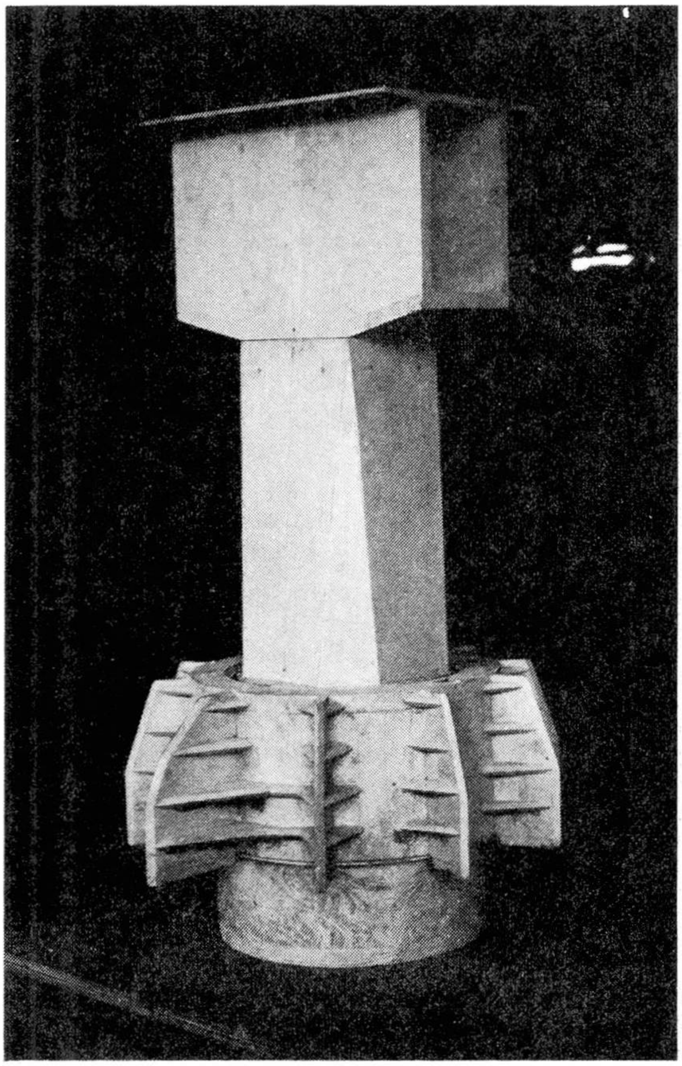
As the fendering system of this kind was being adopted for the first time, the accepting authorities viz. The Public Works Department of Government Goa and Ministry of Shipping & Transport, New Delhi, referred the case to the Central Water & Power Research Station, Khadakwasala, for a model study. The findings of the Research Station corroborated theoretical calculations. The 500 tonne static force allowed in our design was found to be adequate, though head on collision anticipated was realised to be a unlikely possibility. It was suggested a reduced speed limit be imposed on the barge traffic since barges of 1500 tonne displacement are expected to ply shortly.





Photograph 3: Model showing fender resting on well cap without pier, guide pins shown inside periphery of model are not to scale and would be of very much smaller length (refer text)

Photograph 2: Model showing caisson, well cap and pier with guide pins in the well cap.



Photograph 4: Model showing section of the bridge deck, pier, fender and caisson.

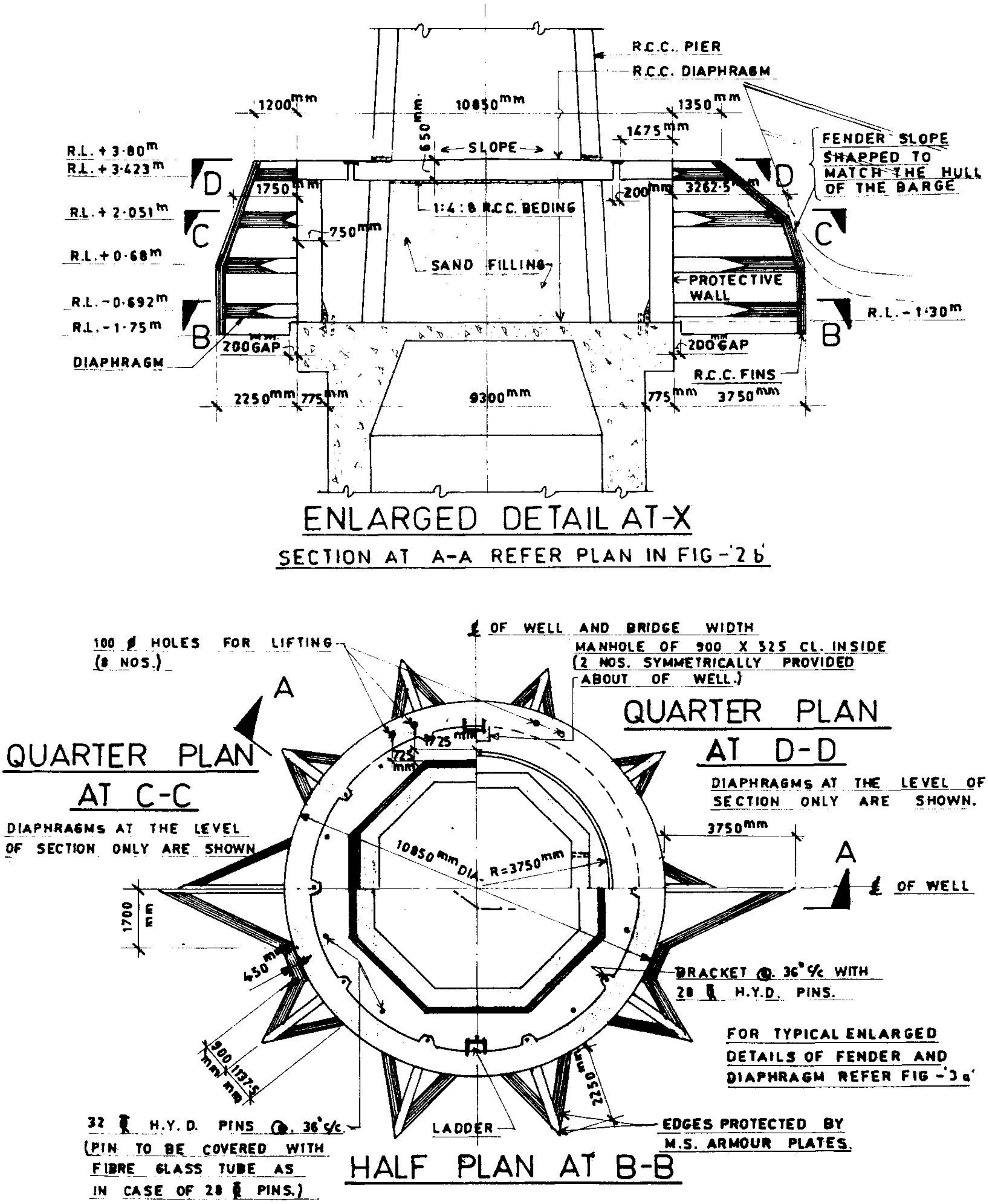


Fig. 2. Fenders for Zuari Bridge

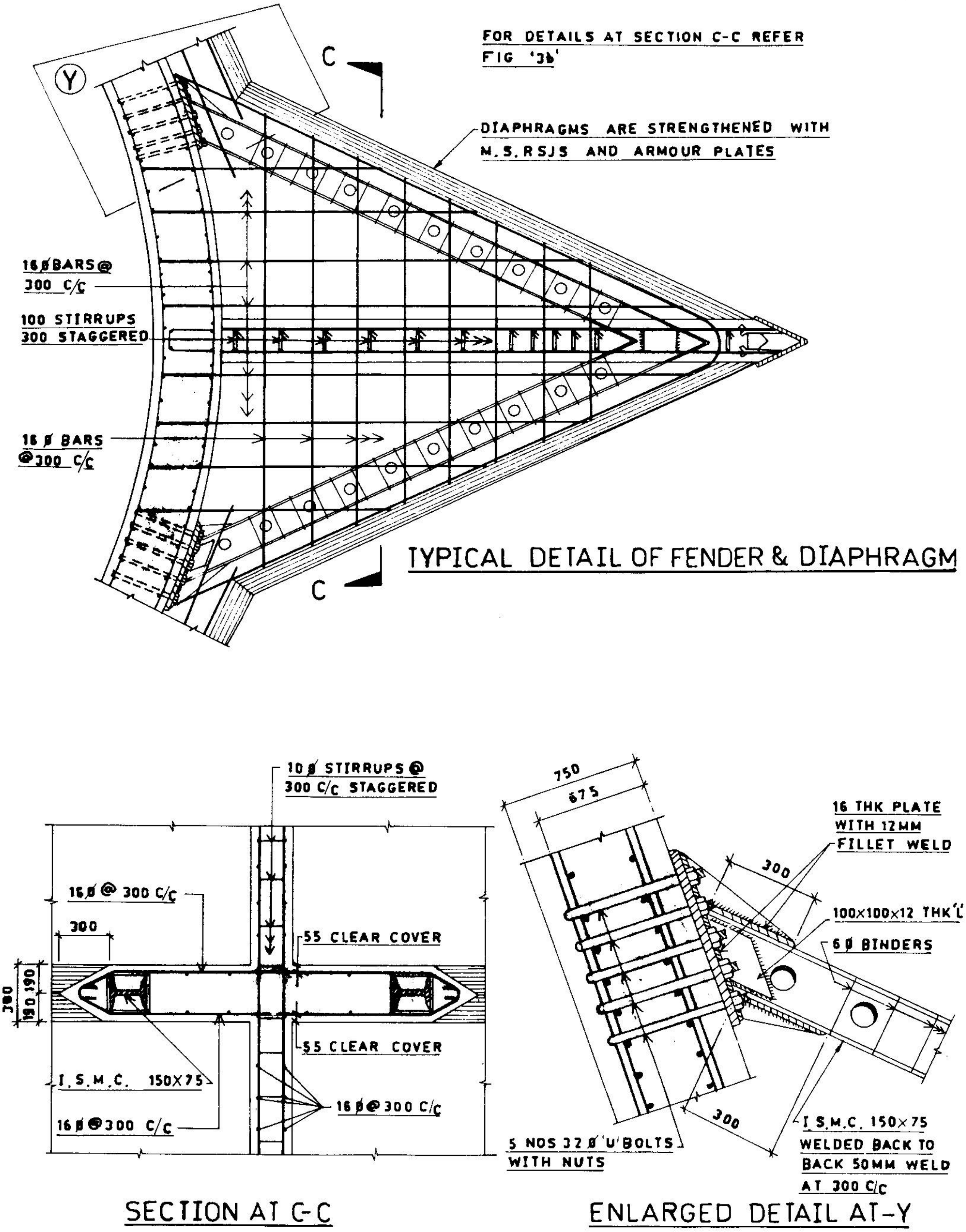


Fig. 3 Details of fenders for Zuari Bridge

4. CONSTRUCTION

4.1 Construction Sequence

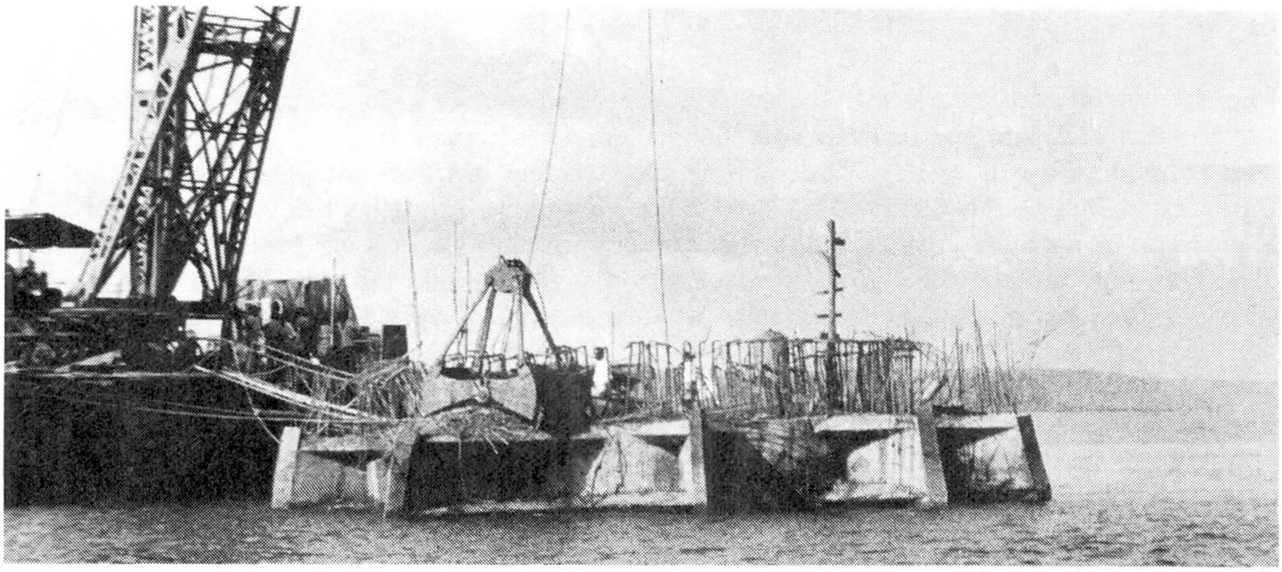
The construction sequence adopted was as follows :

First ten brackets fabricated out of mild steel twin angles and twin channels were fixed on the outer periphery of the caisson exactly below the location of the fins. The channel brackets were under the upstream and downstream fins and at the other fin locations angle iron brackets were used. Over these brackets working platform was provid: The concreting was carried out in stages. The first lift of protective fender wall and fins were cast from the bottom most level upto the bottom of the first diaphragm. The second lift consisted of the diaphragm and the protective wall for the height of the diaphragm. The next life consisted of the fins and protective wall from the top of the first diaphragm to the bottom of the second diaphragm. Similar sequence was continued till top diaphragm was reached. Thus one complete fendering system involved eight stages of concreting and each stage of concreting involved four to five days of working.

4.2 Problems faced during execution

The major problem encountered was casting of the fender wall itself. The base of the fender wall having been fixed at 1.30 M below L.T.L., it was necessary to cast the same before final sinking of the well. Since the depth of water as also the marine slush found at location demanded floating of structural steel caissons, towing them to the position and sinking, precision sinking of such floating caissons plumb at the exact location was found to be not practicable. Further the alternate hard and soft layers of bed strata dipping fairly steeply, resulted in tilting of the caissons. This necessitated casting of the fender wall carefully so that after final sinking the wall remained plumb.

The casting of fins together with the fender wall restricted sinking operation by cranes, as the projecting horizontal diaphragms and vertical fins would keep the floating crane away from the normal position (see photograph 5). The project being the first of its kind in India, precasting the fins and diaphragms was considered to be a technically advanced and risky proposition. However, with the experience gained, it should be possible to precast the system in future.

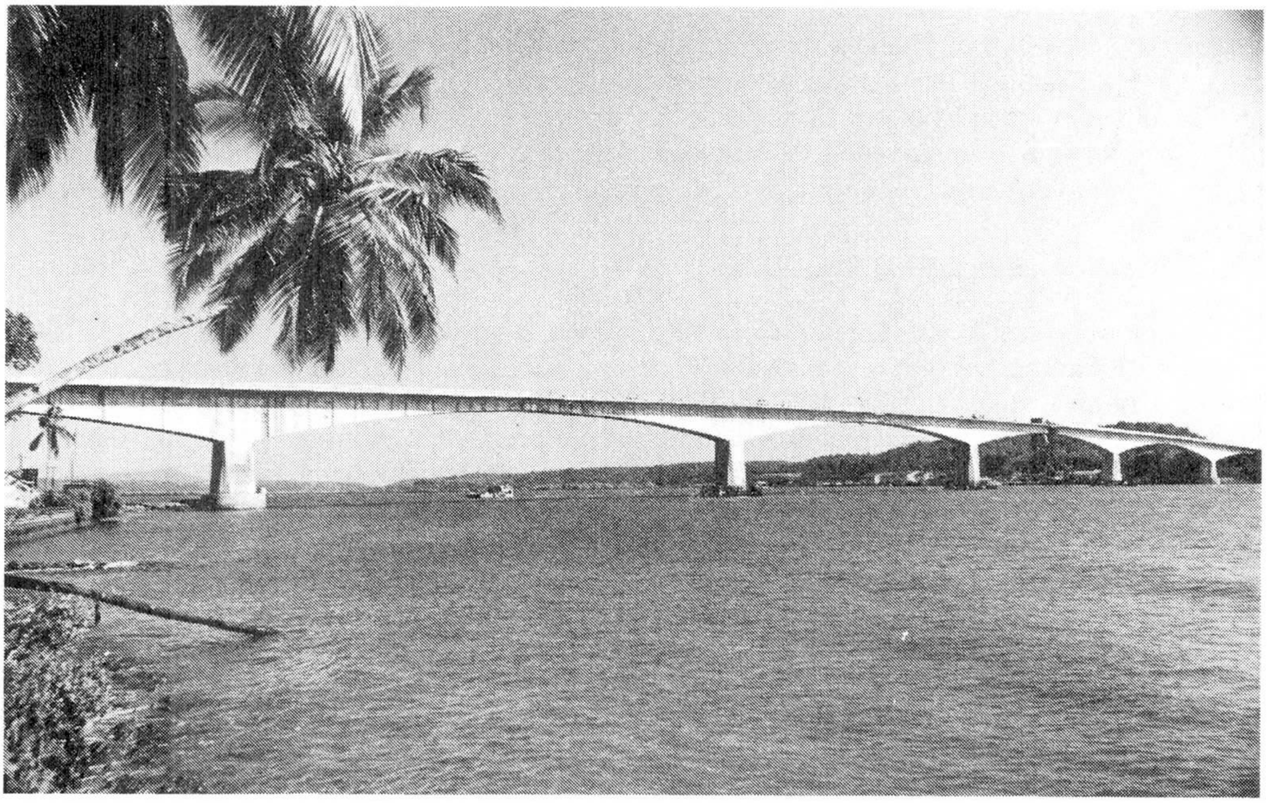


Photograph 5: Showing fender under construction.

5. MATERIAL CONSUMPTION

The quantity used in each fender :

Concrete of grade 35 N/sq.mm	:	240	Cu.m.
M.S. reinforcement of grade 240 N/sq.mm	:	80	KN
H.Y.S.D. reinforcement of grade 415 N/sq. mm	:	300	KN
M.S. structurals embedded of grade 240 N/sq.mm	:	150	KN



Photograph 6: Panoramic view of nearly completed bridge.

ACKNOWLEDGEMENT

The successful construction of the bridge with the novel type fendering system provided for the first time in our country has been largely due to the alround co-operation and encouragement given by the Public Works Department of Government of Goa as also the Ministry of Shipping & Transport, New Delhi, under whose aegis the bridge was designed and constructed. The authors also acknowledge the encouragement given by Mr. T.N. Subba Rao, Managing Director of their firm, under whose constant analytical guidance the design and implementation of this novel feature was carried out.

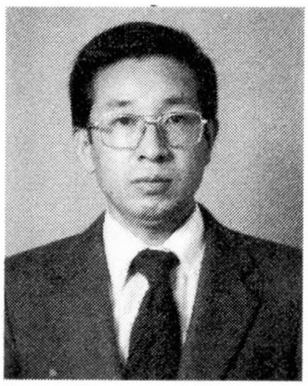
Design Specification of Buffer Structure

Spécification du projet pour la structure des butoirs

Entwurfs-Daten des Puffertragwerkes

Yoshimaro MATSUZAKI
Vice-President
Honshu-Shikoku Bridge Authority
Tokyo, Japan

Hiroo JIN
Deputy Chief of the Design Division
Honshu-Shikoku Bridge Authority
Tokyo, Japan



Yoshimaro Matsuzaki, born 1924, got his civil engineering degree at the Kyoto University, Kyoto, Japan. He has been director general of Chugoku Construction Bureau of Ministry of Construction, and has now led every project in the Authority for ten years.

Hiroo Jin, born 1942, got his civil engineering degree at the Kyushu University, Fukuoka, Japan. For 9 years he has been involved in the design and construction works and is responsible for the design of the substructure of the bridges.

SUMMARY
This paper introduces the outline and principle of design of the multi-cell type buffer attached to the marine pier of the long-spanned suspension bridges under construction between Honshu and Shikoku of Japan. The multi-cell type buffer described in this paper is a box typed steel structure with stiffners, for the purpose of absorbing the collision energy by crushing itself when a ship collided with the pier, which will minimize the damage of the ship and protect the pier.

RÉSUMÉ
Ce document présente les principes du butoir de type multi-cellulaire fixé aux piles des ponts suspendus de grande portée en cours de construction entre Honshu et Shikoku (Japon). Le butoir de type multi-cellulaire décrit dans ce document est une structure d'acier en caissons avec des raidisseurs dans le but d'absorber le choc de la collision en se déformant lorsqu'un bateau entre en collision avec une pile, ce qui minimise l'endommagement du bateau et protège la pile.

ZUSAMMENFASSUNG
Die Entwurfsgrundlagen der Multizellen-Puffer an den Wasserstrompfeilern der langen Hängebrücken, die im Augenblick zwischen den japanischen Inseln Honshu und Hokkaido gebaut werden, werden dargestellt. Der Multizellen-Puffer ist ein Stahlkastentragwerk mit Versteifern, der die Kollisionsenergie beim Aufprall eines Schiffes auf den Strompfeiler durch Verformung absorbieren soll: dadurch wird der Schaden am Schiff verkleinert und der Strompfeiler geschützt.

1. PREFACE

At present, long spanned and large suspension bridges crossing the Seto-Inland sea between Honshu and Shikoku, are under construction by Honshu-Shikoku Bridge Authority. The clearance under the beam and the pier points of those bridges have been decided for the safety of navigation. Also, such navigation aid system as navigational signals, light equipments or radio beacons are installed, therefore the provability of a ship collision with the pier is very small. However, Seto-Inland sea is congested with many ships, therefore it is strongly required to prevent such accidents as the sinking of ships, run-off of oil, etc. even in the case of an unexpected ship collision with the pier e.g. due to mis-operation of the ship under bad weather. To comply with this requirements, the Authority has endeavoured for the various investigation for the buffer with the purpose of minimizing the damage of the ship and the pier when a ship collided with the pier. From these investigations, multi-cell type and composit material type buffers have been resulted and their design standards have been obtained. In this paper, we will mainly describe the outline and the principle of design of multi-cell type buffer.

2. OUTLINE OF THE MULTI-CELL TYPE BUFFER

Multi-cell type buffer is a steel structure composed with many cells with stiffeners, cross frames, cross beams, etc. and is installed around the rigid marine pier of bridges. It absorbs collision energy by crushing itself when fairly large ship collided with it and minimizes the damage of the ship before avoiding the damage to the pier.

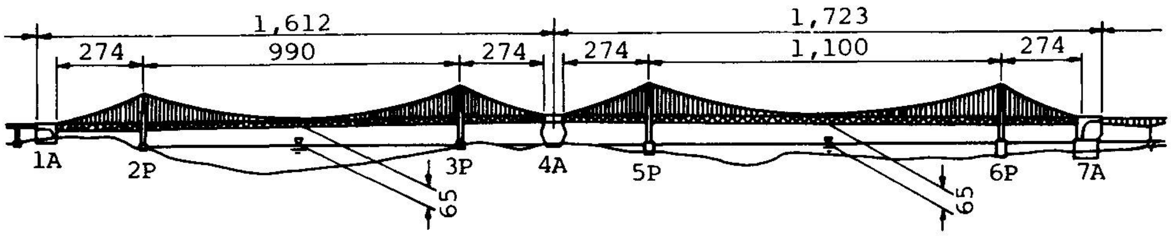


Fig. 1 General View of North-South Bisan-Seto Bridge

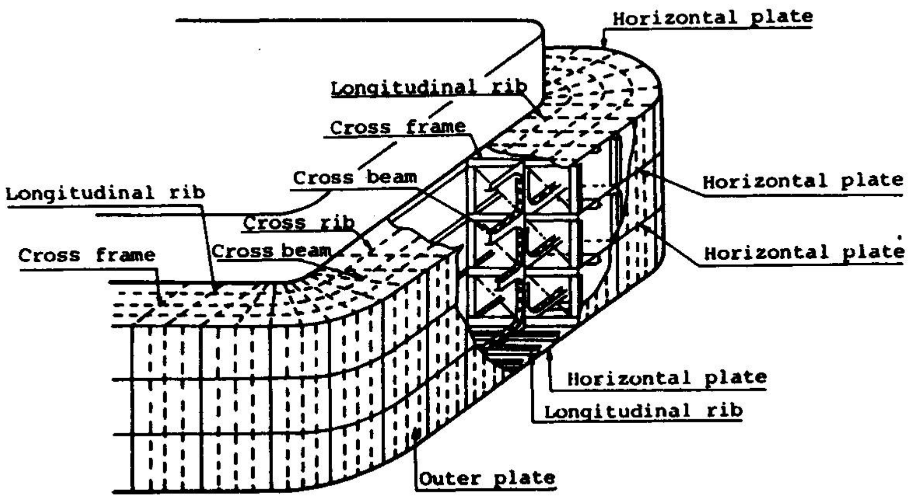


Fig. 2 Multi-cell Type Buffer

On the other hand, the composit material type buffer is the structure similar to the multi-cell type buffer, but inside is filled with such brittle materials as hard urethane foam. The principles of the both designs are almost same and only different with the multi-cell type buffer in the point of absorbing collison energy with the crush of the brittle filled material.

3. DESIGN

3.1 Conditions in Ship Collision

Generally, a ship has a bulkhead to the bow part and if the damage of collision is limited to the front side of the bulk head, it will not bring a serious accident. And in the case of the collision at the side of the ship, only plastic deformation of the shell structure of the vessel will be allowable, because crushing of the ship's side will result in such serious disasters as water immersion inside, sinking of the ship, etc.
Therefore, for the design of the buffer, allowable conditions for the deformation or partial crushing to the vessel and buffer at the time of collision are taken as shown in the Table-1.

		Allowable conditions
Ship hull	Bow	Crushing to the bulkhead of bow is to be allowed.
	Side	Conditions up to the occurrence of plastic deformation are to be allowed.
Buffer	Buffer body	Complete crushing is to be allowed.
	Attaching portion	Elastic design method is to be used as a rule.

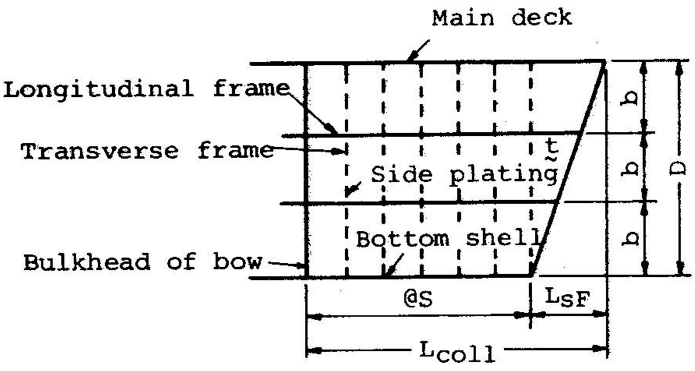
Table 1 Allowable Conditions at the Time of Collision

The methods of calculating the collision energy and the strength of the vessel are different depending on the conditions of the collision and the modes of collision necessary to consider for the design are assumed in accordance with the place of the installation as shown in the Table-2.

Item	Contents
Kinds of collided ships	Travelling ship, drifting ship
Collided portions of ships	Bow, side, stern
Collided portions of under-water foundation	Surface perpendicular to waterway (surfaces parallel to bridge axis), surface parallel to waterway (surfaces perpendicular to bridge axis), corner
Conditions of collision	Head-on collision, diagonal collision, side collision, centripetal collision, eccentric collision

Table 2 Combination of Modes of Collision

Then, it is necessary to estimate the strength of the bow and the side parts of the vessel for the design of the buffer.
It is desirable to calculate the strength of the vessel at the bow and the side parts based on the structure and type of the ships in the actual navigation records, etc. Generally, the strength of the side part can easily be calculated, however, the structures and types of the bow part vary widely and it is necessary to standardize them.



As a ship length L in common cases can conventionally be obtained depending on the ship size and a shape and dimension of the bow can be presumed using L as a parameter, the structure of the bow have been standardized as shown in Fig. 3 and Table-3, based on the investigation concerning to the actual navigation status in Seto-Inland sea.

Fig. 3 Standard Shape of Bow

Structural dimension	Symbol	Standard dimension	Unit
Depth	D	0.08L	m
Side plate thickness	t	$0.82\sqrt{L}+2.5$	mm
Spacing between frames	S	610	mm
Location of bulkhead of bow	Lcoll	0.1L	m
Spacing between longitudinals	b	3·S	mm
Width of slanted portion of bow	L_{SF}	0.25D	m
Angle of tip of bow	2θ	35° ∿ 70°	
Extreme breadth	B	L/10 + 3.81	m

Table 3 Standard Dimensions of Ship's Bow

3.2 Crushing in the Moment of Collision

When a ship collides with a buffer, the energy will be absorbed only with the crushing of the buffer because the bow or the side of the vessel could be considered as a rigid body.

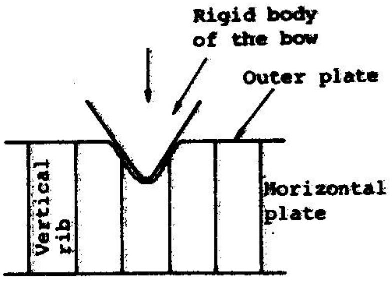


Fig. 4 Modeled Status of Crushing

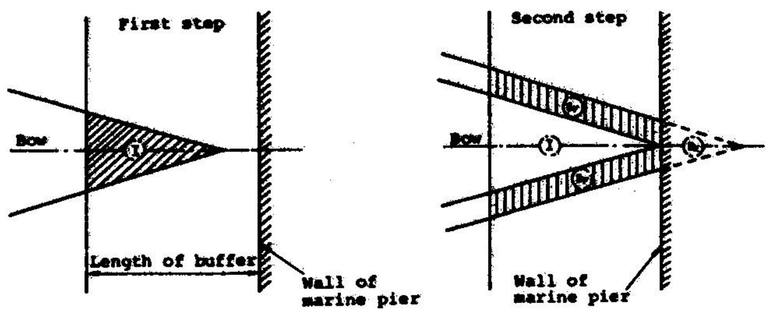


Fig. 5 Relation between Buffer and Bow in Collision

However, after the buffer is crushed in the collision with the bow and the bow has reached to the marine peir (rigid body), the collision energy will be absorbed by the crushing of the bow portion. (refer to Fig. 5)
Therefore the buffer should be designed with horizontal and vertical stiffners to be easily crushed by a vessel as a colliding rigid body. As breakage of the bow, after the crushing of the buffer, will be taken place by the buckling of the structural members of the bow, the maximum load will depend on the buckling load of the shell plate at the bow.

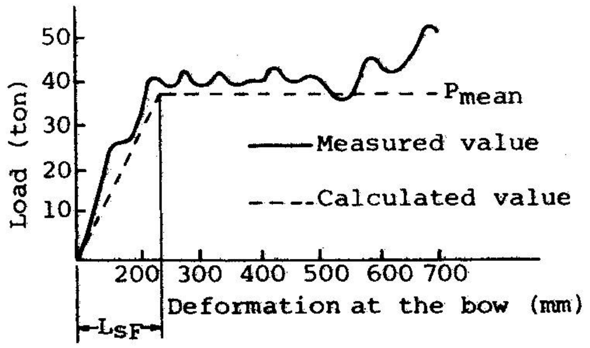


Fig. 6 Load-Deformation Curve

The crushed shapes and the maximum loads of the bow analyzed, have been checked with the results of statical crushing experiment by the models of 1/4 size of the 500 G.T ship and 1/8 size of the 4,000 G.T ship. Fig. 6 indicates the measured and calculated load-deformation curves of the experiment on the 1/4 model of the inclined bow of 500 G.T ship.

3.3 Design for Normal Time

The buffer shall be designed to keep its shape and have the durability against such loads as dead load, tidal current, wave pressure, static pressure except collision, same as the ordinary structures. Therefore, it shall be so designed that the stress of each members due to the combination of each load will be within the elastic limit.
For this reason, horizontal plate of the buffer shall be structured to have the function of buffer material in the case of collision, as well as to bear the loads to act to the buffer in normal time.

3.4 Detail of Structure

In the design of the horizontal plate of the buffer, relation between the acting load in plane and deformation of the horizontal plate in collision can be given as following:

$$F = 2 \cdot P_{cr} \tan \theta \cdot \delta \quad \text{.................................} \quad (1)$$

Where, F ; Load acting on the horizontal plate (collision load) (t)

P_{cr}; Ultimate load of the horizontal plate (t/m)

δ ; Deformation of horizontal plate (quantity of bow intrusion) (m)

θ ; 1/2 of bow angle

Further, for the load acting outside the plane of the plate in normal time, it is assumed that: (1) the plate is designed as fixed along its perimeter, (2) stiffening rib shall be a continuous beam having its span between transverse frames.

Ultimate load of the horizontal plate P_{cr} is given as following:

$$P_{cr} = \sigma_{crp}(\lambda \cdot t_d + A_R) + (\sigma_{crR} - \sigma_{crp})(A_R + b_e \cdot t_d)$$

Where, σ_{crp}; Buckling stress of the plate simply fixed along its perimeter

σ_{crR}; Buckling stress of the vertical rib as a simple column. (considering effective width of the horizontal plate)

λ ; Distance between the vertical rib

b_e ; Effective width of the horizontal plate

t_d ; Thickness of the horizontal plate

A_R ; Sectional area of the vertical rib

Furthermore, chain type, wire type, sliding type, etc. have been proposed as a structure of attachment of the buffer to the rigid foundation.

4. APPLICATION EXAMPLE OF BUFFER

Design standard of the multi-cell and composit material type buffers have been obtained on the basis of the results of years investigation and study. And the buffer based on this standard has been designed for the South and North Bisan-Seto Bridge pier 5P, fabricated and installed in 1981, and is still used practically.

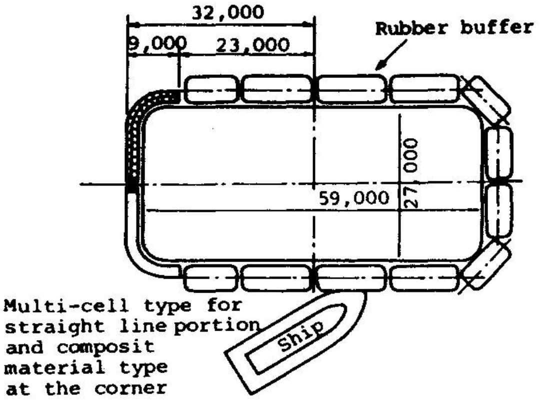


Fig. 7 General View of Buffer of Pier 5P of South Bisan-Seto Bridge

Fig. 7 indicates the general view of the buffer, and Table-4, -5, -6 indicate the ships under consideration, strength of the ship and the result of verification of the buffering function in collision.
Further, design procedure of the multi-cell type buffering is shown in Fig. 8.

Ships considered	Velocity in collision
200 G.T (Ship crossing Route)	8 knott
500 G.T (Ship crossing Route)	8 knott
500 G.T (Drifting ship)	5 knott

Table 4 Design Conditions of 5P Multi-cell Buffer

Ships considered	Inclination of bow	Strength of bow	Strength of side wall
200 G.T	0.83 m	186 tf	10 tf/m^2
500 G.T	1.13 m	366 tf	10 tf/m^2

Table 5 Calculation of the Strength of Bow and Side of Vessel

Item \ Col. status / Gross ton		Collision of bow		Collision of side
		200	500	500
Collision energy E(tf/m)		450	1040	530
Absorbed energy (tf/m)	Buffer (E_f)	136	210	530
	Ship (E_s)	314	830	-
Displacement (m)	Buffer (δ_f)	Crushed	Crushed	1.86
	Ship (δ_s)	2.10	2.65	-
Allowance of crushed length (m)		2.18	3.1	-
Strength of side wall of vessel		-	-	σ_{cr} = 0.96 kg.f/cm^2 < 1.0 kg.f/cm^2

Table 6 Result of the Verification of Multi-cell Type Buffer

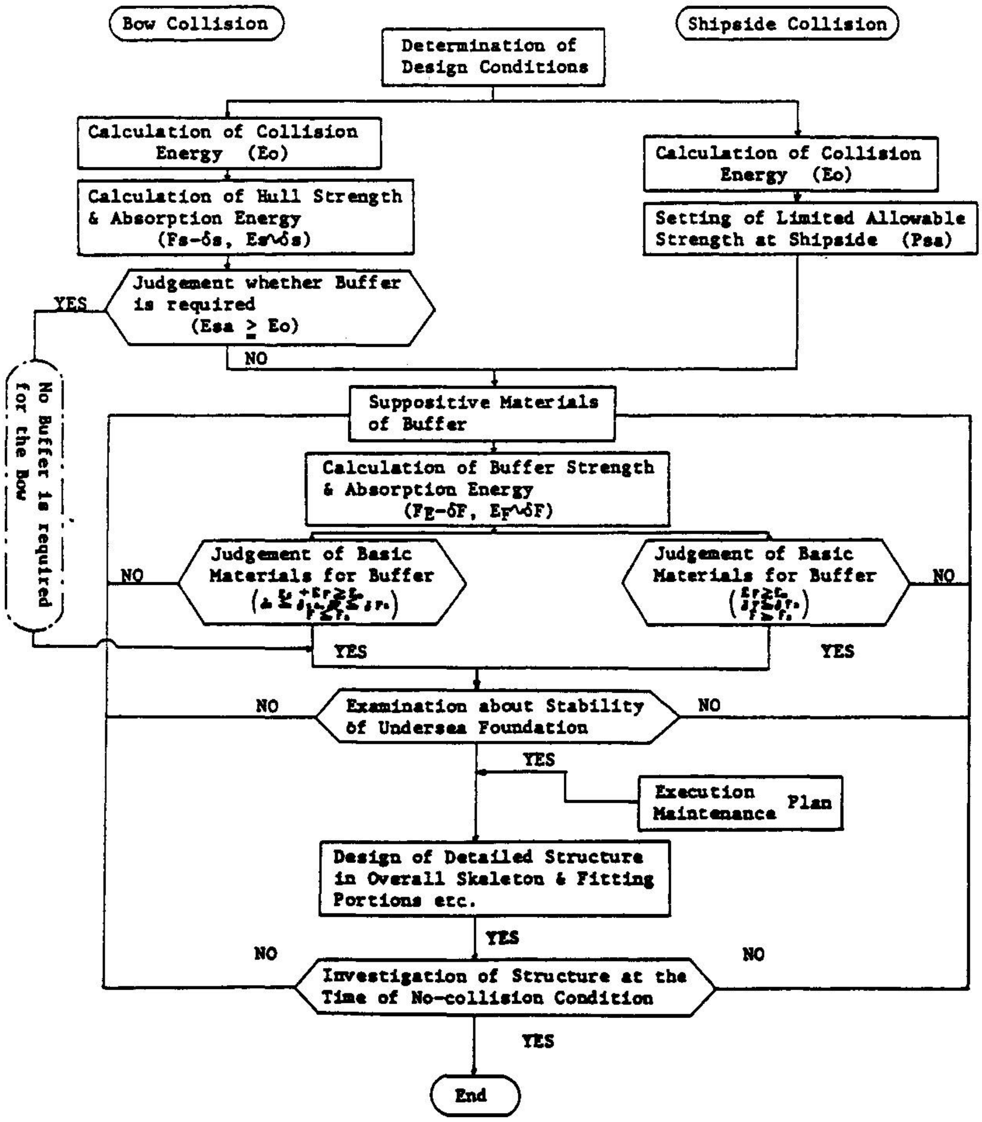


Fig. 8 Flow Chart for Design of Multi-cell Buffer

5. POSTSCRIPT

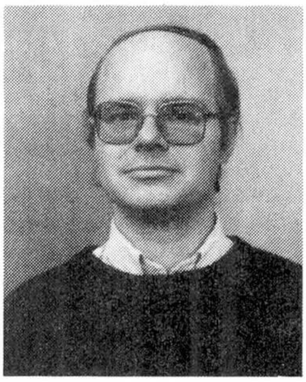
For the construction of bridges in the sea area occupied with navigation, it is necessary to prevent collision by navigational aid systems, in addition to consideration for clearance height, point of piers, etc. However, as far as collision might be assured, it is also necessary that the preventive facilities, based on the assumption of collision, shall be prepared. From such point of view, the Authority has developed multi-cell type and composit material type buffers as the facility to reduce the damage to ship and pier in the minimum and also to prevent oil pollution when a ship collided with the pier.
There still remain a few points to be studied further on the buffer in the future, such as durability, method of repairing after collision. But further investigation and study concerning to those points will be continued and it is believed that the buffer will play an important role for the safety of navigation and the structures.
Authors would expressed the highest gratitude the members of the committee "Investigation of Buffer" (the chairman: Prof. Iwai) in THE JAPAN ASSOCIATION FOR PREVENTING MARINE ACCIDENTS.

Geotechnical Model Tests for the Design of Protective Islands

Iles de protection et essais sur géotechniques modèle

Bemessung von Schutzinseln und geotechnische Modellversuche

Hans DENVER
Research Engineer
Danish Geotechnical Institute
Lyngby, Denmark



Hans Denver, born 1943, holds M.Sc. and Ph.D. degrees from the Technical University of Denmark. After short employment in a private consulting engineering firm he joined the Research Department at the Danish Geotechnical Institute in 1968. He has been responsible for numerous geotechnical test series both in situ and in the laboratory.

SUMMARY
In connection with the design of the Great Belt Bridge (Denmark) the Danish Geotechnical Institute performed quasi-static model tests with models of ship stems in order to determine the dimensions of artificial islands placed around the piers to protect the bridge against damage from a possible ship collision. The paper describes the test programme performed and offers a review of the main results. Furthermore, some of the results are explained in geotechnical terms by means of a simple earth pressure theory.

RÉSUMÉ
L'Institut Danois de Géotechnique a exécuté des essais quasi-statiques sur modèle d'une coque de navire afin de déterminer les dimensions nécessaires d'une île artificielle autour des piles pour les protéger contre les effets d'une collision. L'article décrit le programme des essais et donne les résultats les plus importants. De plus, une explication partielle est offerte en termes géotechniques avec une théorie de butée simple.

ZUSAMMENFASSUNG
Das Geotechnische Institut Dänemark hat für die Großen Belt-Brücke quasi-statische Modellversuche mit einem Schiffskörper ausgeführt. Dies war, um die notwendigen Abmessungen einer künstlichen Insel zu bestimmen, die als Schutz gegen Schiffskollisionen dienen sollte. Das Versuchsprogramm wird beschrieben und die wichtigsten Ergebnisse aufgerechnet. Gewisse Ergebnisse können mit bodenmechanischer Terminologie und einer einfachen Theorie von passivem Erddruck erläutert werden.

1. INTRODUCTION

The general problem of the risk of a ship collision into a bridge was investigated in detail during the design of the now abandoned Great Belt Bridge (Denmark), and a general reference is made to a report describing this problem and the research work carried out in this connection [1]. The presentation in this paper is confined to the description and interpretation of a test series where a model of a ship stem is forced in horizontal direction against a model of a section of a protective island built of a cohesionless material. The tests are carried out in the Danish Geotechnical Institute's laboratory under quasi-static conditions in a dry test pit. The six components of the passive earth pressure on the ship are measured during the penetration into the island, and the advantage of such tests over conventional hydraulic model tests is the possibility to create and calibrate a generally applicable earth pressure theory covering the problem.

2. TEST PROGRAMME

2.1 Model Sand

In most of the tests, the Institute's model sand (Lund-1) is used to model the island. This sand is a pure quartz sand and the unit weight of the grains is $\gamma_s = 26.5$ kN/m³. The void ratios in the loosest and densest states are $e_{max} = 0.88$ and $e_{min} = 0.59$. The mean grain diameter is $d_{50} = 0.60$ mm and the uniformity coefficient $U (= d_{60}/d_{10}) = 2.00$.

Unfortunately, the Lund-1 sand was not suited to the hydraulic tests, so in order to compare the results from the two test types, a few tests were carried out with a more coarse grained material. This gravel (called "Material 2") has the following specifications: $\gamma_s = 27.3$ kN/m³; $e_{max} = 1.02$; $e_{min} = 0.69$; $d_{50} = 6.00$ mm; $U = 1.33$.

Four dry triaxial tests with cylindrical samples (diameter $d = 200$ mm; height $h = 200$ mm) are carried out to reveal the strength of the material. The confining pressure is obtained by maintaining a constant vacuum ($\sigma_3 = 20$ kPa) inside the specimen during the test. The density for the samples is approximately equivalent to the density used in the model tests.

The results of the tests are described as the secant angle of internal friction φ_s defined as $\varphi_s = \sin^{-1}(\sigma_1 - \sigma_3)/(\sigma_1 + \sigma_3)$ as usual. For the Lund-1 sand the mean value of the friction angle is $m(\varphi_s) = 48.7^\circ$ for a void ratio $e = 0.565$.

2.2 Model Ship Stem

Two wooden models of ship bows have been applied in this test series. Model No.1 is shown in Figure 1 and is a simplified model of the bow of a tanker. Figure 2 shows model No.2 which is the bow of a container ship with bulb-stem.

2.3 Test Set-up

The shape of the proposed protective island is shown in Figure 3. A model of a section of this island is established in the laboratory pit, consisting of one or two slopes (1:1.5) and a horizontal chest.

To get reproducible test results it is of course necessary to use sand deposits of homogeneous and reproducible densities. This was obtained for the Lund-1 sand by means of a special sand-laying machinery which can provide controllable densities in a wide range by varying the intensity and the height of fall of the sand stream. Unfortunately, this method could not be used for the gravel (Material 2) - this was placed by carefully shovelling. Extensive control of the density and homogeneity verified that a usable bed could be provided in this manner.

The exact form of the desired section of the island is obtained by means of re-

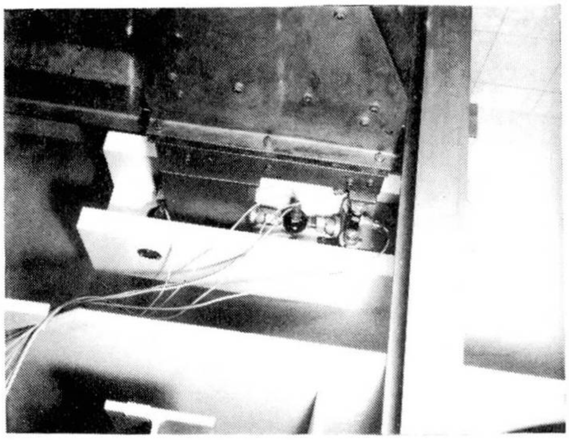


Fig.1 Model of a tanker

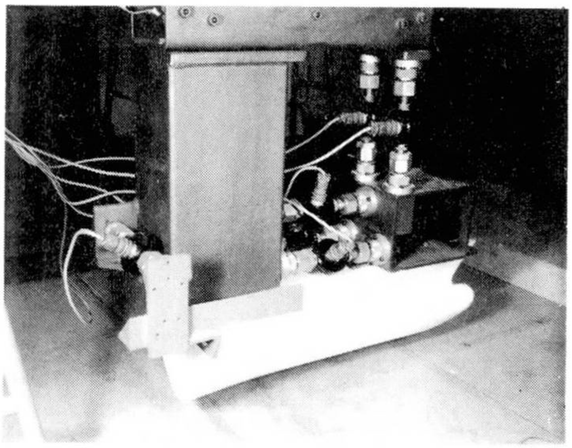


Fig.2 Model of the bow of a container ship

moving excessive material by means of a scraper resting on guiding rails fastened to the pit walls before the material was laid. The guiding rails can be seen in Figure 5.

The ship model is pressed against the island in a horizontal movement with a constant velocity of app.5 mm/min, ref. Figure 4, and the reactions of the material against the ship model are measured by six force transducers (seen in Figure 2).

The forces are logged by a data acquisition system and plotted by a computer programme.

Sand barriers created around the stem are registered by photos and motion pictures taken at regular intervals during the test.

In some of the tests a model of a pier is embedded in the island (see Figure 5). This model is provided with three force transducers to measure the horizontal component of the force on the pier caused by the penetration of the ship stem.

After a test was finished, the island was rebuilt completely (a new sand bed is established etc) to ensure a perfectly homogeneous island.

2.4 Model Tests

The total number of tests is 35. The test material was mainly Lund-1 sand - only in seven tests Material-2 has been used.

The distance between the bottom of the ship model and horizontal berm of the island has been varied from 66 to 120 mm in the tests, but is of course constant for the single test.

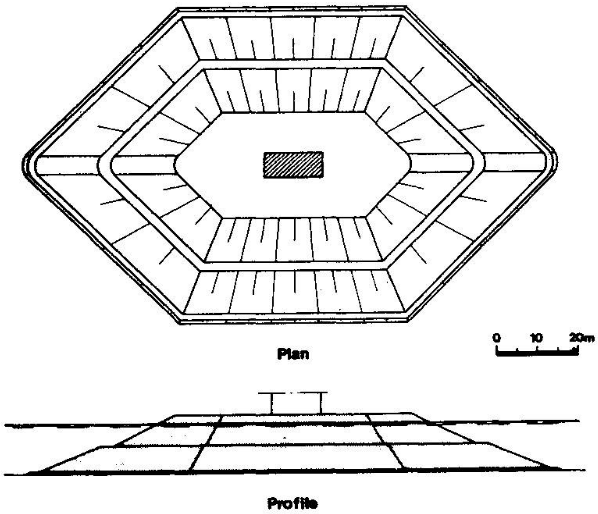
The vessel's angle of approach (θ-defined as $\theta = 0$ for an approach perpendicular to the edge of the berm) has been varied from 0 - 60° in the test series.

Different geometries of the section have been used:

(i) One slope and a horizontal berm

(ii) Two slopes and a horizontal berm - where the angle between the edges is 80° measured on the berm. Two slopes illustrate the point of the island (ref. Figure 3), and approach at different distances from the points have been tested.

Ship model No.1 has been used for all but six tests where ship model No.2 was applied.



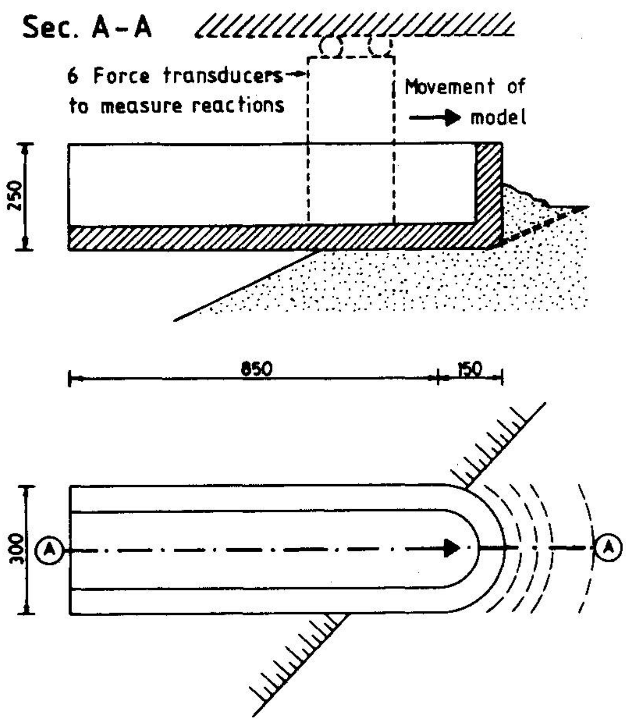


Fig.3 Proposed design of a protective island

Fig.4 Principle for a test with the tanker model (dimensions in mm)

3. TEST RESULTS

3.1 Numerical Results

The numerical results from all the tests are presented in the original reports in tables where the three components of forces and three components of moments in a coordinate system fixed with respect to the vessel are logged together with the observed penetration of the model.

Furthermore, the results are plotted - both the calculated components of forces (as shown in Figure 6) and the components of moments.

3.2 Qualitative Observations

The forces measured during the translation of the model are plotted in Figure 6 for a test with the tanker bow. Traces of rupture lines appear on the sand surface as shown in Figure 4. These traces are observed on the photographs, and the positions where new rupture lines are identified around the stem of the model during the test are also marked in Figure 6. The fluctuations in the measured force correspond very well to these marked positions. It is understood that a rupture plane consists of a weak zone compared to the surrounding soil. And just when an existing rupture plane is nearly "impossible", a new plane is created.

This effect is only observed for the Lund-1 sand. The corresponding plots for the Material-2 consisted of completely smooth curves. The reason is probably that (1) the smaller relative density used for this material yielded less dilatancy when the gravel is sheared and (2) the coarser material cannot establish the rupture plane (or narrow rupture zone) observed for the Lund-1 sand.

4. EARTH PRESSURE CALCULATION FOR A TANKER

4.1 Empirical Model

An empirical model to calculate the earth pressure on the vertical surfaces of a tanker bow is derived by the Author on the bases of the performed tests.

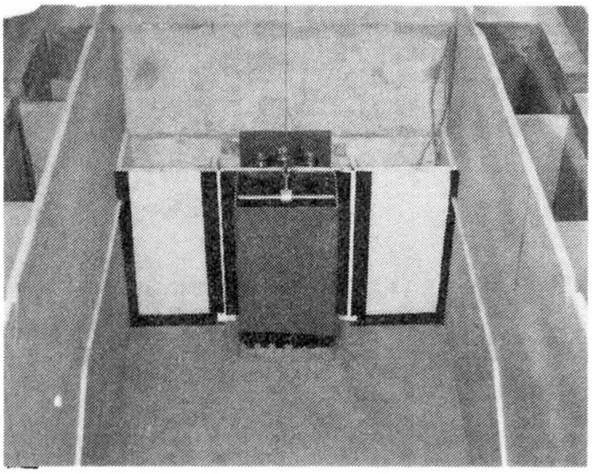


Fig.5 Pier model in the empty test pit

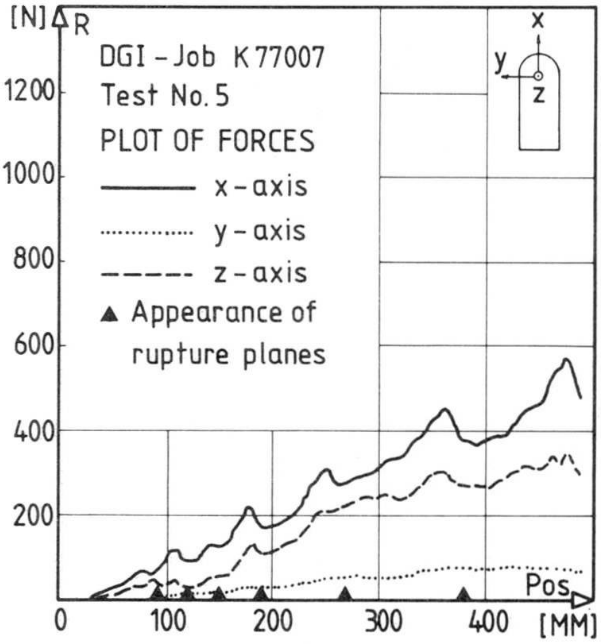


Fig.6 Components of forces plotted against the observed position (penetration) of the model

This model was controlled by the Danish Hydraulic Institute by comparing computed penetrations for a model tanker with results from hydraulic model tests performed by DHI.

Good agreement was found and an example is shown in Figure 7.

The model is presented in [2] and it includes all the parameters varied in the tests (ref. sec.2.3).

4.2 Geotechnical Model

To illustrate the assumptions in the empirical model a tentative expression is presented based on the experience from the test work.

Using the notation in Figure 8 the resultant earth pressure is calculated as

$$F_x = \int_{s_1}^{s_3} (\cos v + \mu \sin^2 v)\, dE^f - \int_{s_0}^{s_1} dF^s + \int_{s_3}^{s_4} dF^s \qquad (1a)$$

$$F_y = \int_{s_1}^{s_3} (\sin v - \mu \cos v \sin v)\, dE^f - \int_{s_0}^{s_1} dE^s + \int_{s_3}^{s_4} dE^s \qquad (1b)$$

$$F_z = -\int_{s_1}^{s_3} dF_v = -\int_{s_1}^{s_3} \mu \cos v \, dE^f \qquad (1c)$$

The coefficient of friction between the soil and the ship is denoted $\mu = \tan\delta$. In Eqs.(1) we have applied the following assumption:

$$dF = \mu \cos v \; dE^f$$

$$dF_t = \mu \sin v \; dE^f$$

where $dF^f = \sqrt{dF_v^2 + dF_t^2} = \mu \; dE^f$ for the front and $dF^s = \mu \; dE^s$ for the sides.

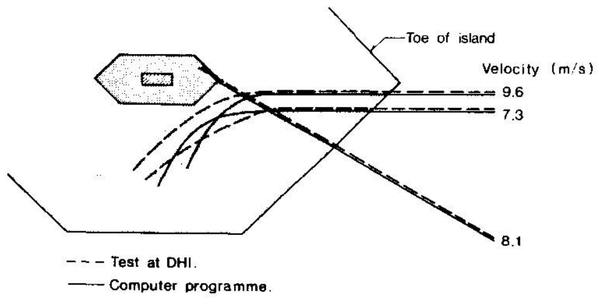


Fig.7 Examples of measured and calculated traces of a model tanker (from [1])

The total weight dG of sections 1 plus 2 is:

$$\mathrm{d}G = \frac{1}{2r}\,[l_1 h_1 (r + l_1/3)\gamma_1 + l_2 h_2 (r + l_2/3)\gamma_3]\ \mathrm{d}b = c_1\ \mathrm{d}b$$

(γ_1 and γ_2 are the unit weights of the soil in section 1 and 2 respectively).
The earth pressure dE^f on a vertical strip on the front db is found by projecting dE^f and dG on the line a which is perpendicular to the resulting force dT on the rupture line:

$$\mathrm{d}E^f = \frac{\sin(\varphi + w)\ \cos\,\delta}{\cos\,(\varphi + w + \delta)}\ \mathrm{d}G = c_2\ \mathrm{d}G$$

where $w = \tan^{-1}(h_2/l_2)$.

A reasonable calculation of E^s along the sides is based on the earth pressure at rest K_0 which for a sloping surface can be estimated to (ref.[3]):

$$K_0 = (1 - \sin\,\varphi)(1 + \lambda\,\sin\,\beta)$$

where $\lambda = -0.5$ for $\beta(= \tan^{-1}\, h_1/l_1) > 0$

The earth pressure is then calculated as:

$$\mathrm{d}E^s = \frac{1}{2}\,\gamma(h_1 + h_2)^2\,K_0\ \mathrm{d}s = c_3\ \mathrm{d}s$$

- ds is the width of an incremental vertical strip and γ can be approximated to $\gamma = (\gamma_1 h_1 + \gamma_2 h_2)/(h_1 + h_2)$.

For a situation where the vessel penetrates the island with $\theta = 0$ the resultant earth pressure is calculated as:

$$F_x = 2r(1 + \mu\,\pi/4)\ c_1 c_2 + 2a\,\mu\,c_3 \qquad (2a)$$

$$F_y = 0 \qquad (2b)$$

$$F_z = -2r\,\mu\,c_1 c_2 \qquad (2c)$$

- a is the horizontal distance between s_1 and the edge of the berm. The values of c_1, c_2 and c_3 are assumed to be constant in the calculation of Eqs.(2).

The rupture plane along the bow is actually supposed to be a narrow rupture zone where the sand after a primary shearing is in a state with a critical void ratio e_k and where the direction of the rupture zone actually coincides with the direction of the strain characteristics.

The angle of internal friction measured in the triaxial tests φ_s should therefore be corrected twofold:

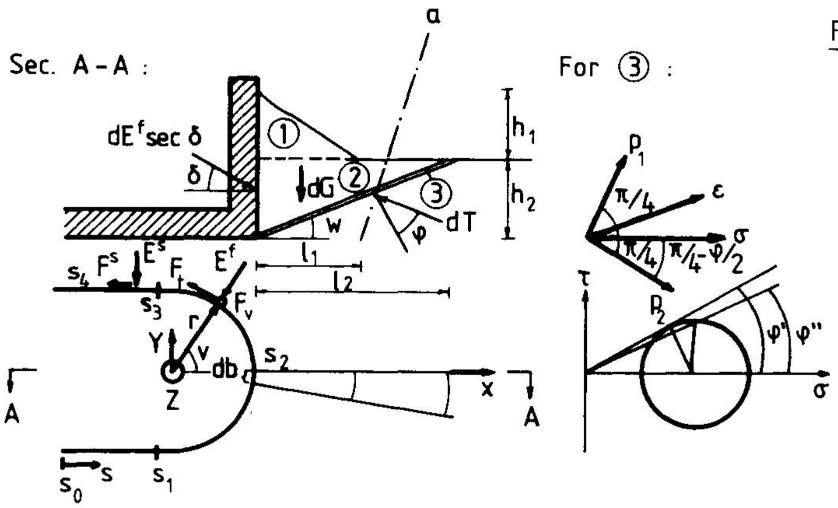


Fig.8 Notation for the earth pressure calculation

(i) The variation of φ with e follows closely the relation $e \tan\varphi = k$, and the critical void ratio is known to be approximately equal to e_{max} for the Lund-1 sand - yielding:

$$\varphi' = \tan^{-1}(k/e_{max}) = \tan^{-1}(0.643/0.88) = 36.^{\circ}2$$

(ii) The sand dilates of course no further when e_k is reached, i.e. the angles between the strain characteristics (ε) (lines with no elongation) and the principal directions (p_1, p_2) which coincide for stresses and strains are $\pi/4$. In Figure 8 it is shown that the angle of internal friction should be reduced to φ'':

$$\varphi'' = \tan^{-1}(\sin\varphi') = \tan^{-1}(\sin 36.^{\circ}2) = 30.^{\circ}6$$

It is emphazised that this correction, which is explained more thoroughly in [4], has proved useful in many cases.

Table 1 shows a comparison between calculated and measured values of F where the following values have been used: $\gamma_1 = 14.7$ kN/m³, $\gamma_2 = 17.1$ kN/m³, $r = 0.15$ m, $\mu = 0.44$.

Test No.	l_1 (m)	l_2 (m)	h_1 (m)	h_2 (m)	a (m)	F_x cal. (N)	F_x mea. (N)	F_y cal. (N)	F_y mea. (N)	F_z cal. (N)	F_z mea. (N)
3	0.20	0.40	0.10	0.095	0.20	583	458	0	-7	-184	-149
20	0.20	0.30	0.10	0.065	0.25	330	297	0	-15	-102	-79

Table 1 Calculated and measured values of F

5. CONCLUSIONS

A model test series is described and there is referred to an empirical mathematical model to calculate the earth pressure for a vessel when it penetrates an island.

Furthermore, the main results are derived in geotechnical terms in this paper, taking relevant soil parameters into account. With an overestimation of 10 - 30 % the calculation method agrees fairly well with the observed values.

6. ACKNOWLEDGEMENTS

The Author wish to thank Statsbroen Store Bælt for permission to publish the results of this investigation.

Acknowledgements are also made to N.H.Christensen and Professor Bent Hansen for valuable advice and for reading the manuscript to the paper.

Finally the Author will express his gratitude to the Danish Geotechnical Institute who has sponsored the preparation of the paper.

REFERENCES

[1] STATE BRIDGE GREAT BELT. The Great Belt Bridge, Investigation into the Ship Collision Problem.
The Minister of Public Works, Copenhagen, 1979.

[2] DANISH GEOTECHNICAL INSTITUTE. Geotechnical Model Tests, Report No.1 - 4, Job. No.K77007, 1978.

[3] NISSEN, P. (1963). Earth pressure at rest in non-cohesive soil - effect of sloping surface. Bygningsstatiske Meddelelser, Vol.34, No.3, Copenhagen.

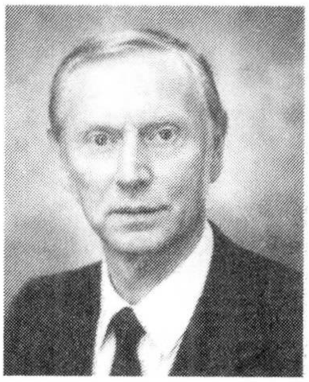
[4] HANSEN, B. (1979). Definition and use of friction angles. Proc. 7th Europ. Conf. on Soil Mech. and Found. Eng., Vol. 1, Brighton.

Floating Pier Protections Anchored by Prestressing Tendons

Protections flottantes de piles de ponts ancrées par câbles

Kabelverankerte schwimmende Schutzsysteme für Brückenpfeiler

P. E. MONDORF
Chief Engineer M.Sc.
Freyssinet International
Paris, France



Paul Mondorf, graduated 1953 from DTH, Technical University of Denmark. For several years he was with Cowiconsult, Consulting Engineers, mainly in bridge design and supervision, for five years with DTH, Structural Research Laboratory, now with Freyssinet, has been involved in off-shore and nuclear works, at present in Research and Development.

SUMMARY
The paper deals with the concept of floating systems for protection of bridge piers against ship impact. The lay-out of such systems and their basic components are analysed with particular attention to the cables, for which high durability is essential. The kinetic energy of the off-course vessel is absorbed mainly through extension of cables and heaving of counterweights. The application of such systems to a particular case is shown.

RÉSUMÉ
L'article traite la conception de systèmes flottants pour protéger des piles de ponts contre l'impact de navires. L'arrangement d'une telle protection et ses éléments constitutifs sont analysés, spécialement les câbles pour lesquels une solution à haute durabilité est proposée. L'énergie cinétique du navire est absorbée principalement par l'allongement de câbles et par le levage de poids. L'application d'un tel système à un cas concret est esquissé.

ZUSAMMENFASSUNG
Der Artikel behandelt das Konzept schwimmender Schutzeinrichtungen von Brückenpfeilern gegen Schiffsanprall. Die Auslage und die Hauptkomponenten eines solchen Schutzsystems werden untersucht, besonders die Kabel, für welche ein Typ von zuverlässiger Dauerfestigkeit vorgeschlagen wird. Die kinetische Energie des Schiffes wird hauptsächlich durch Kabelverlängerung und Schwerkörperhebung umgesetzt. Die Anwendung eines solchen Systemes an einem konkreten Fall wird gezeigt.

1. INTRODUCTION

The increasing tendency to build bridges over navigable waters combined with a trend towards larger ship sizes has focussed the attention of bridge owners and designers, shippers and navigators on the risk of collision between ships and bridge structures. A number of major events has clearly shown that the risk is a substantial one which may lead to severe damage when accidents occur.

A comprehensive survey carried out in the mid-sixties listed a considerable number of accidents [1] which has been steadily growing since [2] . The seriousness of the problem is illustrated by such recent exemples as the TJORN bridge (S), which was hit on 1980.01.18 by a 15,000 t freighter, causing the loss of 8 lives and the collapse of the 278 m steel arch main spain, and the SUNSHINE SKYWAY bridge over TAMPA BAY (USA), hit on 1980.02.09 by a 20,000 t freighter, killing 33 persons and causing the loss of the three central span steel lattice girder. The most frequent reasons for the collisions are human error, mechanical failure and bad weather.

An important lesson from actual collisions is that the risk concerns not only the piers adjacent to the navigation spans but all the piers in sufficiently deep waters, as off-course vessels may hit anywhere.

An international enquiry undertaken in the late seventies [2] showed that in several countries, the concerned parties were seeking means to reduce the potential collision risk.

The traditional ways of reducing the risk have been to increase span lengths and/or to introduce navigational restrictions, both of which are of limited value.

In many cases, the design criteria have prescribed that the piers should be designed to sustain collision load, generally from smaller and medium size vessels drifting at moderate speed, whereas more violent collisions are left unconsidered as to costly to be covered.

Other approaches have consisted in protecting the piers by means of fenders dolphins, cofferdams or artificial islands.

Pier attached fenders, dolphins etc will in many cases be found completely out of scale with the energies to be handled.

Cofferdam cells consisting of circular sheet pilings filled with gravel and braced by a top slab may form efficient and relatively inexpensive protection, provided firm bottom is available at reasonable depth.

Artificial islands may protect even against large vessels but their dimensions and cost increase rapidly with the water depth and the subsequent reduction in water section may not be acceptable.

In 1979 tender was called for the protection of the piers of the ZARATE-BRAZO LARGO bridges over the PARANA river (RA) against impact from oceangoing vessels. The two bridges were built 1971-78 for combined road/rail traffic over two arms of the river. Each bridge comprises three cable stayed main spans, 110-330-110 m, with piers placed in deep water in the silty movable riverbed on high piling bearing on sand, 56 and 70 m respectively under MWL.

The majority of the tenderers offered floating protections, one of which was accepted for execution. Other tenderers proposed fixed protections, but in the present case these came out extremely costly due to the unfavourable foundation conditions.

A floating protection consists of pontoons, buoys or suchlike, anchored to the bottom of the water and interconnected by chains or tendons, supposed to intercept off-course vessels. The system may include special devices for energy absorption.

At several occasions floating systems have been proposed, but they are often regarded with certain scepticism as not sufficiently reliable or requiring a too intensive surveillance. One of the few systems actually put into service protects the TARANTO bridge over the MARE PICCOLO (I). It is designed for vessels up to 15,000 t displacement coming at a speed of 3.1 m/sec, it consists of chains spanning between buoys and anchored to concrete blocks by other chains, equipped with energy absorbers based on pistons sliding in lead filled steel cylinders.

In recent publications SAUL and SVENSSON have summarized the theory of ship collision against bridge piers [3] and given a survey of known measures for pier protection, analysing their suitability to the ZARATE-BRAZO LARGO case (ZBL) and comparing costs and efficiencies of the dozen proposals received as an answer to the abovementioned tender [4] .

The tender has clearly demonstrated the inherent possibilities of floating protections, but also shown the necessity of further development to render such systems fully reliable.

The purpose of the present paper is to examine the possible lay-out and the basic components of such systems in order to help to ensure them the credit they deserve.

2. ARRANGEMENT OF A FLOATING PROTECTION

In the lay-out of the system, two zones have to be distinguished, one covering the main piers, the other the remaining piers in waters sufficiently deep to be reached by vessels.

For the piers adjacent to the navigation channel on-course vessels are allowed to come fairly near, hence the margin left to stop or deviate an off-course vessel will be narrow, of the order of some 5-25 m and the protective system has to be relatively stiff. The solution may consist in the provision of duly anchored buffers covering the required angle and placed sufficiently ahead of the piers to avoid all risk of being thrown against these ones.

Such buffers tend to demand considerable dimensions and may advantageously be of great mass. They must be designed to receive the impact of the vessels, either directly or through fender tendons. A part of the kinetic energy will be absorbed in the choc damaging the vessel whereas the rest will be absorbed through the proper response of the system.

For the remaining piers, the required protection can be placed further in advance of these and a relatively flexible system will suffice. No buffers need to be provided but just duly anchored buoys carrying between them a fence consisting of parallel tendons, situated a few meters above and below the water level, and designed to capture and stop the vessels over a distance which may be of the order of say 50-100 m.

In order to reduce the risk of being oversailed, the fence shall be duly braced by e.g. nylon wires and fendered by neoprene cylinders or similar.

Buffers and buoys will be anchored to the sea bed by relatively small size anchor lines designed to ensure the stand-by position of the floating elements. For the buffers at least three raking anchor lines will be required in order to closely maintain its location independently of the variations in water level.

The floating elements are further retained by large size cables of considerable length connected to fixed anchors in the river bed and weighted with one or several loads in predetermined positions along the cable. During stand-by, the loads will rest on the river bed but when the system is activated, the cable will be stretched and the loads lifted. The geometry of the system is such that the desired ratio force-displacement is achieved.

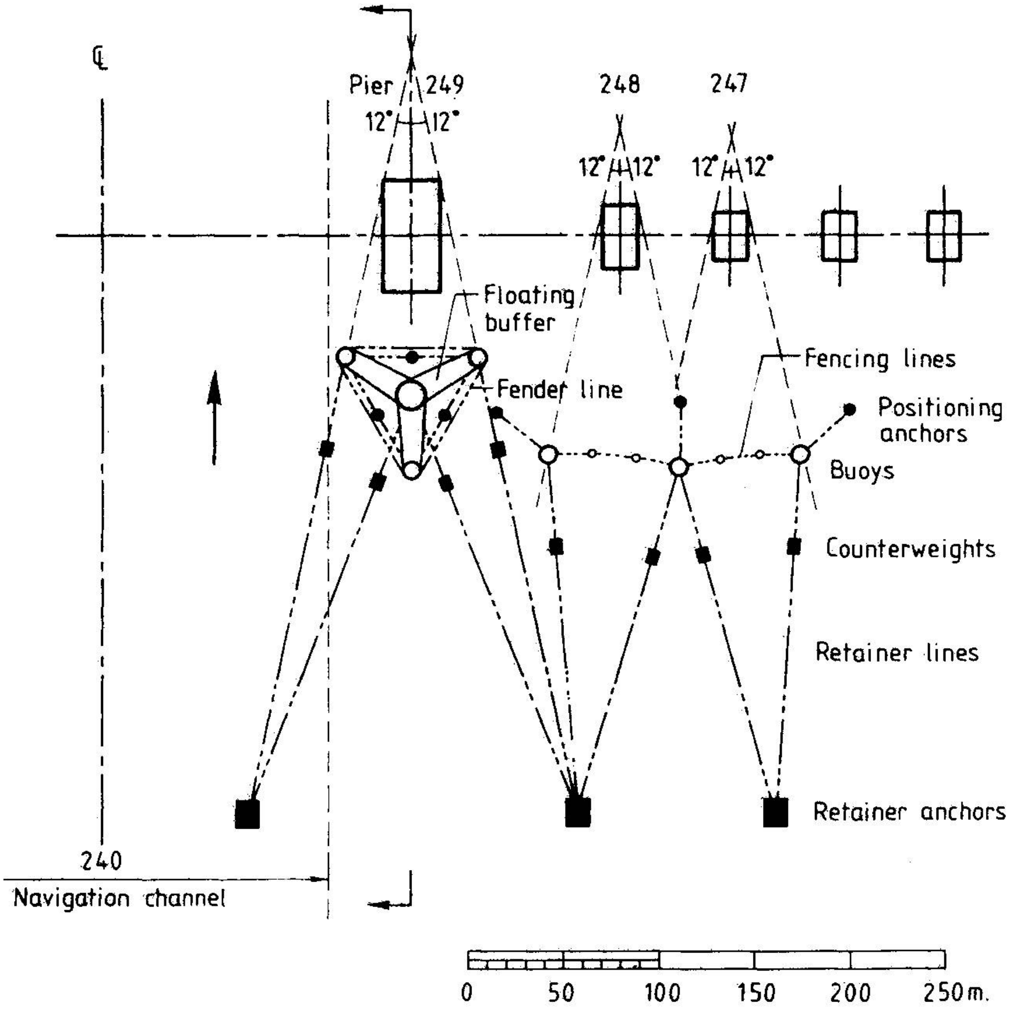


Fig.1 Floating protection. Typical lay-out. (Here e.g. shown applied to the Zarate Brazo Largo bridge over the Paraná-Guazú)

3. MARINE TENDONS

The floating protection as described requires cables for fender lines, fencing lines, anchor lines and retainer lines.

The common feature for all these applications is a demand for high breaking load, good fatigue performance, durability in dry and immerged condition, flexibility and resistance to wear, abrasion and rough treatment ; in most cases, the cables will stand under low permanent stresses but may be subjected to violent shocks.

In some cases, chains might be used, but only flexible cables constitute a multipurpose tool covering all the needs encountered here. In this paper, only tendons built up from parallel prestressing strands are dealt with. Such strands are favourable due to an excellent price/performance ratio.

A type of tendon has been developed which is specially fitted for this type of application [5] . It is constituted by parallel strands, either dry, greased or galvanized, each one covered at the mill with a tight fitting polyethylene duct. The bundle of elementary fairly parallel strands is enclosed by a watertight outer duct, generally a high density polyethylene pipe. The space left between the elementary duct-covered strands is filled with a high-viscosity pretroleum base compound of lubricant and corrosion protective capacity (e.g. VISCONORUST 2090 P-4, which possesses substantial record from nuclear prestressing works).

The outer duct may be further protected against local pressure or abrasion by a spirally wound wire or strand of steel or glass fibre covered by plastic or neoprene.

The tendons are anchored by anchorages of the same types as applied for cable stays, the fatigue performance of which has been proven by laboratory tests [6] . The individual strands may be held by swaged grips or by wedges. The anchor blocks may, depending on the size of the tendon, be single or multilevel blocks in order to reduce their diameter and thereby the strand deviations.

The front block is screwed into a socket, the type of which may be selected for the particular application of the tendon and its method of erection f.i. a hammerhead, an eye bolt or a cylinder with a collar bearing against a plate embedded in the structure and blocked against pull-out.

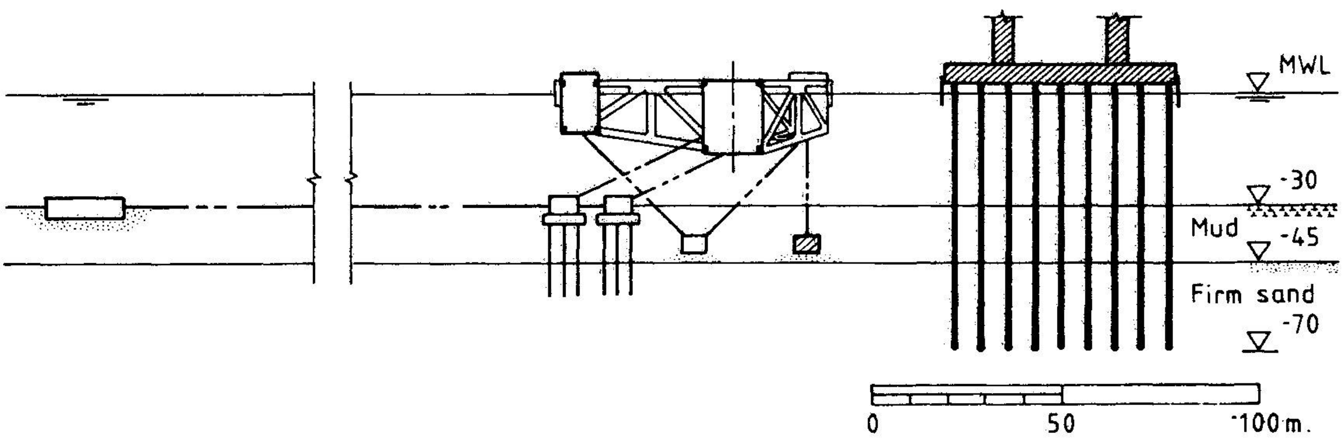


Fig.2 Vertical section through buffer. Ex :(Z B L.)

In the immediate vicinity of the anchorage, the outer duct may be replaced by a steel transition pipe in order to cater for bending stresses which otherwise might reduce the fatigue strength in that zone.

The anchorage itself is fully closed and filled with an epoxy tar compound which covers also the adjacent part of the tendon, overlapping the ducts protecting the individual strands. The outside of the anchorage is duly protected.

Tendons of this type are available in sizes up to 91 strands of 15 mm nominal diameter, i.e. up to breaking loads of 24 MN, service loads about half thereof, fatigue life 2 mill cycles with stress ranges of the order of 180 to 250 MPa at a stress level corresponding to the service load.

The tendons must be fabricated in shop near the construction site in order to ensure high quality and exclude transport and storage which might require coiling on small diameter drums that could be harmful to the preformed tendon.

It will be seen that the tendon is provided with a multibarrier protection against corrosion, namely outer PE-duct,tendon filler compound, inner PE-duct and possibly grease or galvanization. However, such a multibarrier will be efficient only if all singular points along the tendon are correctly treated.

This concerns primarily the bending radii adopted for the finished tendon. In order to keep the bending stress in the PE-duct below yield, the ratio D/d between the diameters of the bend and the duct should for long duration not be less than 50 and for short duration not less than 25. The adoption of such bending diameters will also prevent damage of the protective enclosure from the strands when these are tightened and slide over the support.

Where the tendon leaves or enters a structural element, its position cannot be predetermined, and even if a hinge were provided, it might not work properly. Therefore, in order to avoid kinks <u>either</u> the tendon will have to be protected over a certain length by a special transition pipe <u>or</u> the outlet will have to be funnel shaped with the right curvature. The latter solution offers the advantage of permitting the same tubing to apply all along the current length of the tendon. On the other hand, it may be preferable to keep the tendon sizes fairly moderate in order to prevent the bending diameters from becoming too bulky.

37 HC 15 MARINE TENDON

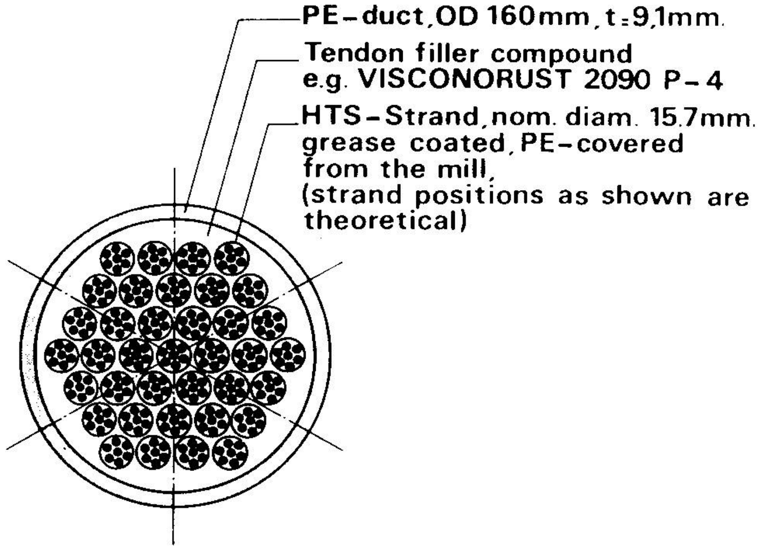


Fig.3 Marine tendon.
Typical cross section.
Ex : 37 HC 15.

The anchor lines and retainer lines may generally be arranged so that they pass through the blocks placed on the river bed but have both of their anchorages situated near the water surface for easy access.

4. FLOATING ELEMENTS

The floating elements may be constructed of steel or concrete. Steel may be preferable for smaller buoys whereas concrete due to its mass will be advantageous for the major elements. In some cases, the consideration of floatability may limit the weight and thereby favour the use of light weight concrete.

The buffer needs to cover a considerable area but should not oppose too large a section to the water flow. Therefore, it may be constituted from several caissons, rigidly connected to a central one. The openings between the outer caissons may be closed off by fender tendons hanging at about water level.

The outer caissons, but not the central one, will be exposed to vessel impact. Therefore, their punching shear stress has to be checked for a collision force determined with due regard to the displacement provoked by the shock.

For the evaluation of collision forces reference is often made to MINORSKY's formula, which is based on empirical data collected from actual ship collisions and which establishes a linear relationship between energy and deformed volume of steel, covering energies up to 5,000 MNm.

Collision tests between model pairs of ships carried out in Germany have permitted WOISIN to conclude that the collision force is fairly constant during a collision but attains for a short duration (0.1 to 0.2 sec) its maximum value which is about twice its average and depends, in first line, on the ship size and the shape of the striking parts, and only to a lesser extent on the kinetic energy involved. A simple empirical formula relating max. collision force to the ship size (DWT) has been given. The validity of the formula is extended by its author to cover the case of a ship striking a stiff body. The floating buffer is considered as such, but its capacity to withdraw under the blow will reduce the damage caused to the striking vessel and probably the collision force. Shaping of the buffer as an isosceles triangle may favour the deviation of the vessels for all cases except a frontal shock.

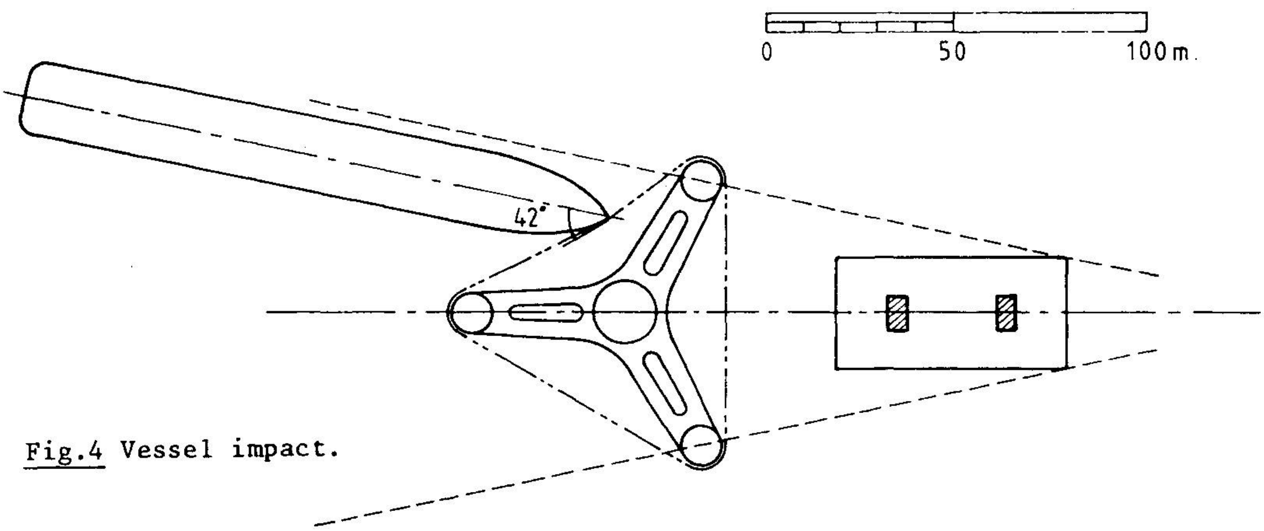


Fig.4 Vessel impact.

5. ANCHORAGE OF FLOATING PROTECTIONS

The floating elements will be anchored to concrete blocks placed on the river bottom, generally concrete caissons sunk and ballasted, through which the marine tendons are passed. The detailed design of the blocks and their supports depends on the soil conditions.

Two different types have to be considered, namely the positioning anchors and the retainer anchors.

The first ones are relatively small ; during stand-by, they should remain in position with a tolerance for the buffer anchors of say two meters horizontally and vertically, more for the anchors of the flexible system. They are allowed to move when the system is activated, provided they will not thereby cause damage to the floating elements or to the bridge.

Generally, these anchor blocks will have to be placed on a gravel coffin prepared in a carefully dredged area. In extreme cases, they may require piling and special precautions to prevent them from dropping into cavities caused by erosion of the river bed.

The retainer anchors are relatively large, their position should remain fixed but their level is of minor importance. Generally, they will have to be placed on a gravel bed, in a dredged area and protected by stone filling in order to ensure friction ; if necessary, some vertical prestressing tendons used as rock or soil anchors could be added, or a steel skirt which will force the rupture lines to pass into the supporting soil.

The technique of prestressing tendons applied as rock or soil anchors is well known ; anchors for permanent use, provided with an uninterrupted reliable barrier which fully isolates the steel from the surrounding medium have been developped, tested and frequently used under the most variable circumstances. Generally, such anchors are applied on-shore, they are less frequent in submarine condition due to high cost of installation, but technically, the case is not fundamentally different.

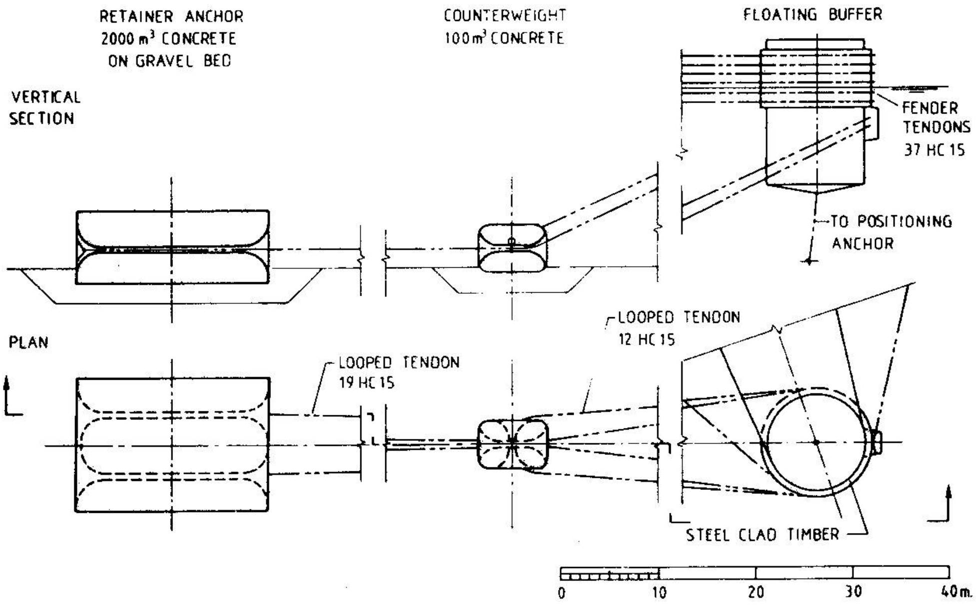


Fig.5 Floating protection. Details of tendon arrangement, counterweight and retainer anchor. Ex : (Z B L.)

6. APPLICATION

To illustrate the concept of floating pier protection its application to a bridge as the ZARATE BRAZO LARGO is shown schematically fig. 1-6.

The design criterias are as indicated in [4] , especially for the main piers, considering collision from a vessel of 20.000 t displacement, moving downstream at a ground speed of 2 m/sec under an angle with the pier axis of up to 12° to either side. The water velocity is 1 m/sec.

The kinetic energy of the vessel including that of 5% supplementary hydrodynamic mass amounts to 42 MNm. For comparison, the maximum impact force of the vessel against a stiff pier has in [4] as a first approximation been found from WOISIN's formula to Pmax. = 108 MN ± 50% with a damage length of 0.8 m.

For a frontal blow of the vessel against the buffer, a rough estimate gives a ratio of struck to striking mass of $m_2/m_1 = 1/3$, so the fraction of energy which is absorbed in the immediate plastic deformation may be assumed to $m_2/m_1 + m_2 = 1/4$ (10 MNm). The rest of the energy will be transformed by the extension of retainer tendons, working at stresses below 0.5 GUTS (7 MNm) and by the lifting of the counterweights (28 MNm). Passed the first instants of the shock, the force exerted on the vessel will not exceed 15 MN (see diagram fig. 6). Beyond the design shock the system still possesses ample margin before attaining its ULS determined by the yield of tendons or the sliding of retainer anchors. In this balance supplementary hydrodynamic energy dissipation has been neglected.

A lateral shock will demand less energy to be transformed and will probably in most cases produce a relatively soft deviation of the vessel. The post-collision behaviour of the vessel may need computer simulation or model testing.

For the system as designed the variation in water level result in a tolerance on the position of the buffer of an order of 5 m. The movable character of the river bed may require comprehensive works in order to ensure the level of the positionning anchors and the counterweights placed in the river, piling or the constitution of stable gravel coffins may be required.

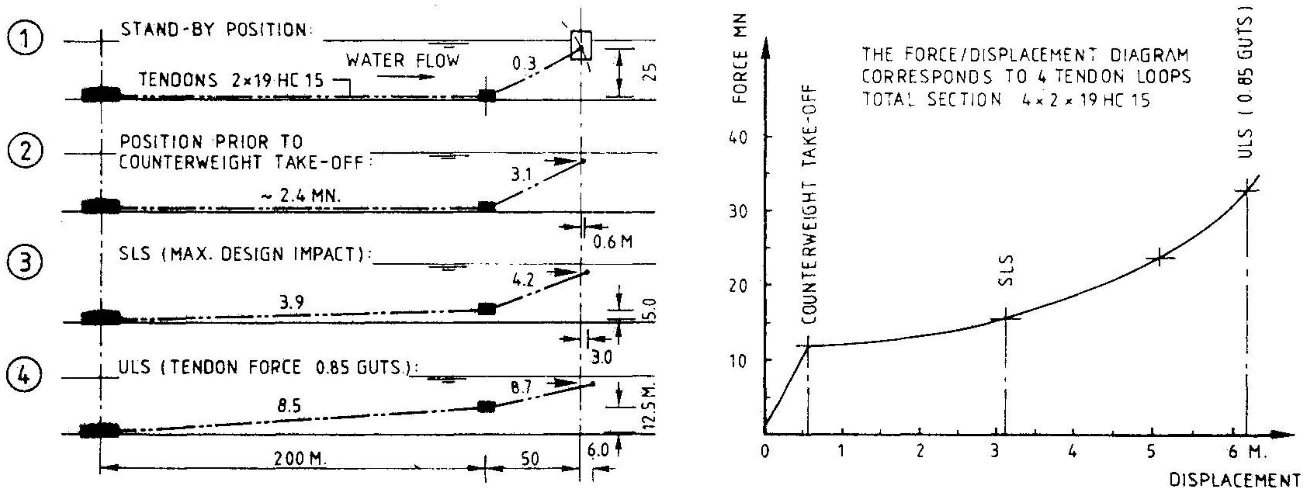


Fig.6 Performance of floating protection. Ex : (Z B L.)
Left : tendon positions and forces for one loop.
right : force/displacement diagram for buffer retained by 4 loops.

7. CONCLUSIONS

It is possible to conceive floating systems able to give bridge piers an acceptable degree of protection against ship impact.

The technology involved in the construction of such systems does not exceed what is known from marine works. The systems require a surveillance of about the same intensity as other installations in navigable waterways and the same as many bridge structrures.

In order to become efficient the systems will be expensive compared to the cost of the piers, but seem to be competitive especially in deep waters and they may often constitute the only means for the protection of the whole length of a bridge at reasonable cost. The application of such systems to waters with ice problems has not been considered by this paper.

Generally, it seems preferable to consider ship impact and the protection it might require already in the original design of a bridge structure in order to ensure correct judgment of span lengths and realistic evaluation of different foundation alternatives.

Floating protections may constitute a nuisance to other users of the waterway and its surroundings. Aesthetically, even well designed protections will probably be found to be of unaccustomed appearance.

A great number of existing bridges require protection, a floating protection will, for many of these, constitute the only realistic approach.

REFERENCE

1 OSTENFELD, Chr., Ship collisions against bridge piers, AIPC memoires 1965.

2 FRANDSEN, A.G. and LANGSO, H., Ship collision problems.
I. Great Belt bridge. II International Enquiry. IABSE proceedings P 51/80 May 1980.

3 SAUL,R. und SVENSSON, H., On the theory of ship collision against bridge piers. IABSE proceedings P51/82 May 1982.

4 SAUL, R. und SVENSSON, H., Zum Schutz von Brückenpfeilern gegen Schiffsanprall, dargestellt am Beispiel der Brücken Zarate-Brazo Largo über den Parana (Argentinien). Die Bautechnik 10/1981.

5 Brevet français P.V 773 57 52. Cable d'amarrage et dispositif d'ancrage utilisant un tel câble. Déposé le 28 nov. 1977.

6 MONDORF, P.E. et DE LA FUENTE, C., Developpement des haubans pour grands ponts. La Technique Française de Précontrainte, 9. congrès FIP, Stockholm 1982.

Pier Protection for the Sunshine Skyway Bridge

Protection des piliers du pont Sunshine Skyway

Schutz der Brückenpfeiler der Sunshine Skyway- Brücke

Michael KNOTT
Project Engineer
Greiner Eng. Sciences
Tampa, FL, USA

Michael FLANAGAN
Skyway Project Mgr.
Florida DOT
Tallahassee, FL, USA

Michael Knott, born in 1952, received his M.S. degree in Structural Engineering from Virginia Polytechnic Institute in 1976. He is a Registered Professional Engineer and for the last six years he has been responsible for the design and technical direction of numerous marine and deep foundation projects.

Michael Flanagan, born in 1940, studied Mechanical Engineering at the university of Florida and Management at Florida State University. Employed by the Florida Department of Transportation in 1962. For the last six years he has managed major highway/bridge construction programs.

SUMMARY

This paper summarizes the preliminary engineering and environmental studies performed for the pier protection of the replacement structure for the Sunshine Skyway bridge which collapsed in 1980.

RÉSUMÉ

L'article présente les études préliminaires de génie civil et d'environnement pour la protection de la structure destinée à remplacer le pont Sunshine Skyway, pont détruit en 1980 à la suite d'une collision.

ZUSAMMENFASSUNG

Dieser Aufsatz faßt die vorbereitenden Ingenieur- und Umwelt-Studien zusammen, die zum Schutz der Pfeiler der Neukonstruktion der 1980 gestürzten Sunshine Skyway-Brücke unternommen wurden.

1. GENERAL

On May 9, 1980 during an intense early-morning thunderstorm, the empty 40,000 dwt bulk carrier M/V Summit Venture struck one of the anchor piers of the Sunshine Skyway Bridge across Tampa Bay, Florida. The anchor pier was located 241.4 m from the centerline of the channel. A 396 m section of the southbound main span collapsed, and 35 lives were lost in vehicles which fell into the bay (see Appendix). Since that accident, southbound automobile traffic has been diverted to the parallel bridge structure which will operate under a two-lane, two-way traffic condition until the replacement bridge system is constructed.

The proposed 6.705 Km-long replacement structure (Fig. 1) presently under construction consists of a segmental concrete, single-plane, cable-stayed main span design. Figure 2 depicts 2400 m of the high level approaches and main span portion of the bridge. Estimated construction cost for the replacement bridge is approximately 115 million dollars. The new structure will increase the main span from 256.0 m to 365.8 m and the vertical navigational clearance from 45.4 m to 53.3 m.

Without some form of positive protection, however, the new bridge piers would still be vulnerable to ship collision. To investigate this vulnerability and to decide what types of protection alternatives to implement, the Florida Department of Transportation established a two-phased design process. Phase I, the Preliminary Engineering Phase, consisted of a broad-based evaluation of numerous types of pier protection alternatives, including the environmental impacts. Based on the Phase I study, a decision was made on which alternatives to implement for the project. Phase II services consist of the final design and construction document preparation of those pier protection alternatives which were selected. Because the Final Design Services are on-going at this writing, only the results and recommendations of the Preliminary Engineering Phase will be discussed below.

2. SHIP OPERATION REQUIREMENTS

Regulatory and developmental items relating to the operation of merchant vessels in Tampa Bay were studied. The existing Vessel Traffic Service (VTS) in the bay operates on a level of L_0, the lowest level of VTS under the U.S. Coast Guard's established VTS activity levels. Pilotage is required for all ships, but is not required for all barges. The study recommended that the following vessel operation procedures be implemented.

2.1 Vessel Speed Limits

Establish a vessel speed limit in the vicinity of the bridge. Since a vessel's speed exponentially affects ship impact energy, a limitation on speed will serve to limit the design collision energies. Based on a review of tidal flow velocities, ship operating characteristices, and discussions with the local bay pilots, a limit of 10 knots was recommended.

2.2 Ballast Requirements

Establish a requirement that all outbound light vessels must meet minimum ballast criteria before transiting the channel in the vicinity of the bridge. This requirement will enhance the stability and control of the

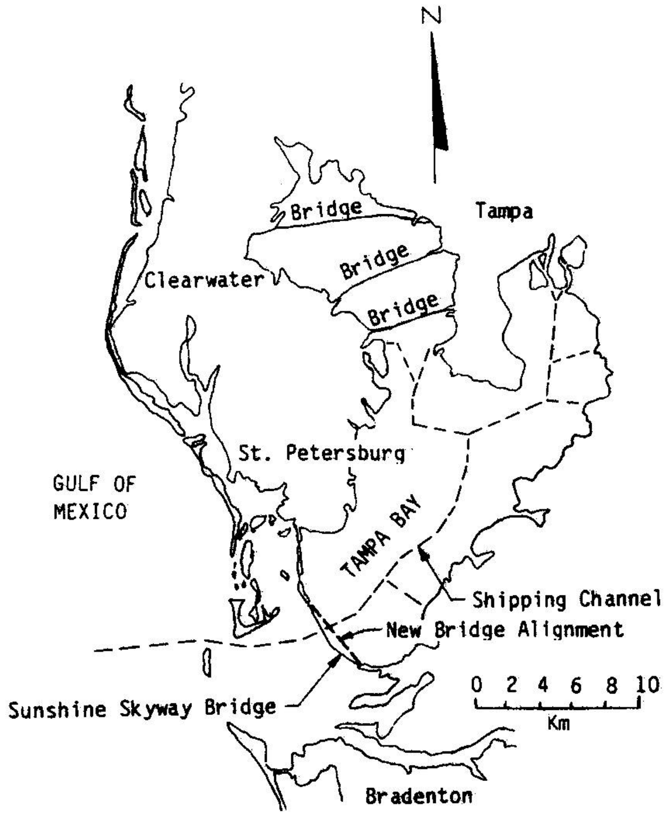


Figure 1. Project Location Map

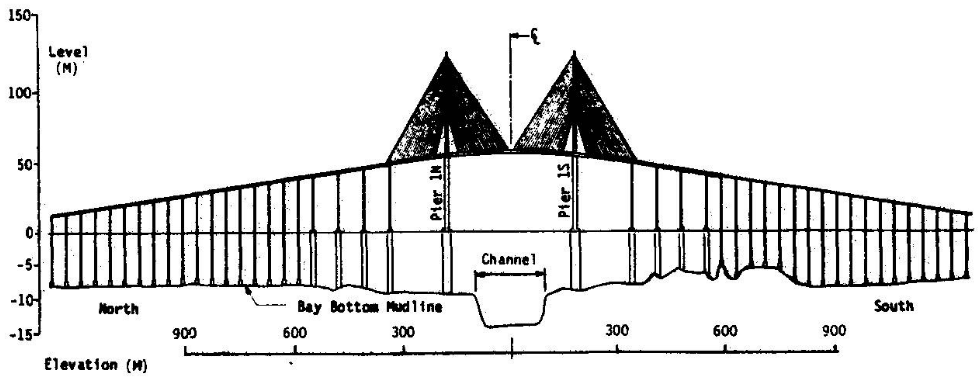


Figure 2. High Level (Main Span) Section of the new Sunshine Skyway Bridge

vessel, particularly during high wind and rough sea conditions. It is of interest to note that all historical collisions with the existing bridge involved vessels (3 ships and 7 barges) which were traveling light and without ballast. The requirement for inbound vessels to ballast is precluded because of potential adverse environmental impacts involved in deballasting polluted water within the sensitive bay environment.

2.3 Weather Requirements

Establish minimum weather conditions under which vessels can transit in the vicinity of the bridge. As a minimum, a predicted 3.2 Km visibility requirement for one hour before and after the vessel is to pass under the bridge would be established before a vessel would be allowed to leave her moorings.

3. STRUCTURAL PIER PROTECTION DEVICES

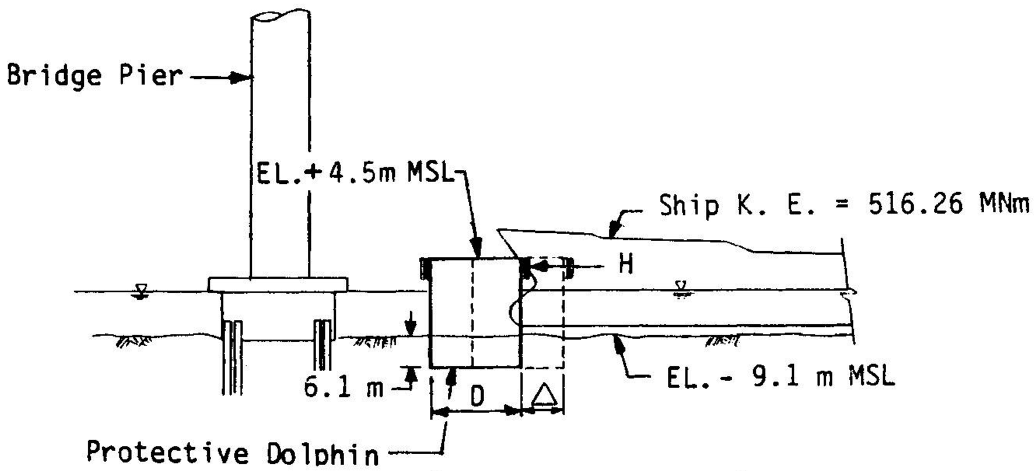
A number of pier protection systems which have been developed for other applications worldwide were considered for use on the Sunshine Skyway. Based on criteria of effectiveness, expected damage to the ship, constructibility, cost-effectiveness, maintenance, safety, and environmental impact as applied to this specific project, many of the systems were deemed unsuitable. These included cable systems, anchored ships and pontoons, sliding caissons, pile group systems, fender systems, and submerged islands. Systems which were shown to be the most desirable for the project consisted of dolphin systems and artificial islands, or a combination of both.

3.1 Dolphins

Dolphins, large-diameter circular sheet pile cells filled with material such as sand or concrete, have been successfully utilized to protect bridge piers in conditions similar to those existing at the Sunshine Skyway. Because of the existing soil conditions, the mode of dolphin failure and subsequent energy dissipation would be by sliding. The ship collision energy would be absorbed through the passive failure of the soil behind the dolphin, and through friction on the bottom of the sliding cell. The approximate length that the dolphin would slide and the duration of the impact were then calculated for the design loading conditions and the specific soils data at the project site (Fig. 3). One of the desirable characteristics of the dolphin circular shape is its tendency to redirect the vessel away from the pier under glancing-blow situations. The preliminary analysis indicated that a cluster of three 18.3 m diameter dolphins should be placed on each side of the principal bridge piers requiring protection (Fig. 4). Due to the presence of a corrosive marine environment, the steel sheet piling would require coating and cathodic protection.

3.2 Artificial Islands

Protection of bridge piers can be accomplished by constructing armored artificial islands around the piers. Several bridges in the world currently have such protection. The islands consist of a sand core which is protected against wave and current action by armored slope protection. Ship impact energy is absorbed by deformation of the island material, the rising up of the ship's weight as it slides up the island slope, and by the friction of the hull sliding against the island. The length that the vessel would penetrate into the island is primarily based on the ship geometry, island geometry, island materials, and the collision energy of the ship



Diameter (D) (M.)	Approx. Failure Load (H) (KIPS)	Approx. Stopping Distance (△) - M.
12.2	6,500	18.3
18.3	12,300	9.5
24.4	19,900	6.1
30.5	29,100	4.0

Figure 3. Ship Impact on Protective Dolphin

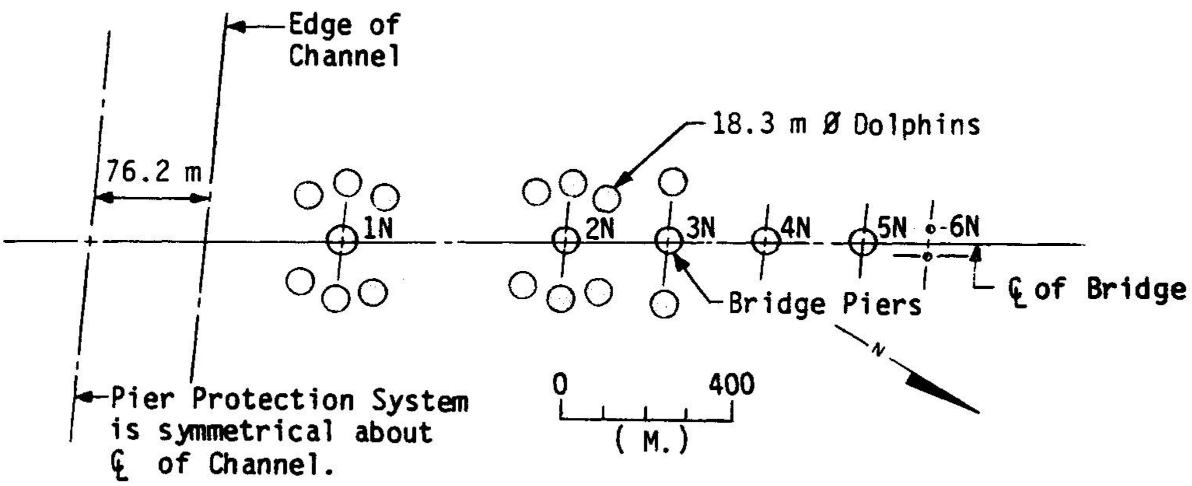
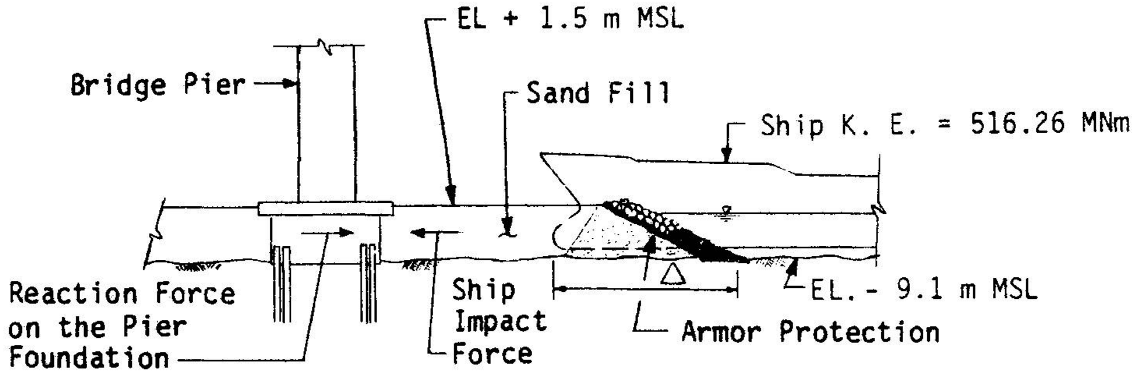


Figure 4. Plan of 6-Pier Dolphin Protection System (Piers 1-3, N & S)



Design Ships		Draft (M.)	Stopping Distance (Δ) - M. (Min.)	(Max.)
21,000	DWT (Loaded)	9.1	21.0	38.7
85,000	DWT (Ballasted)	4.7	28.4	53.3

Figure 5. Ship Impact on Protective Island

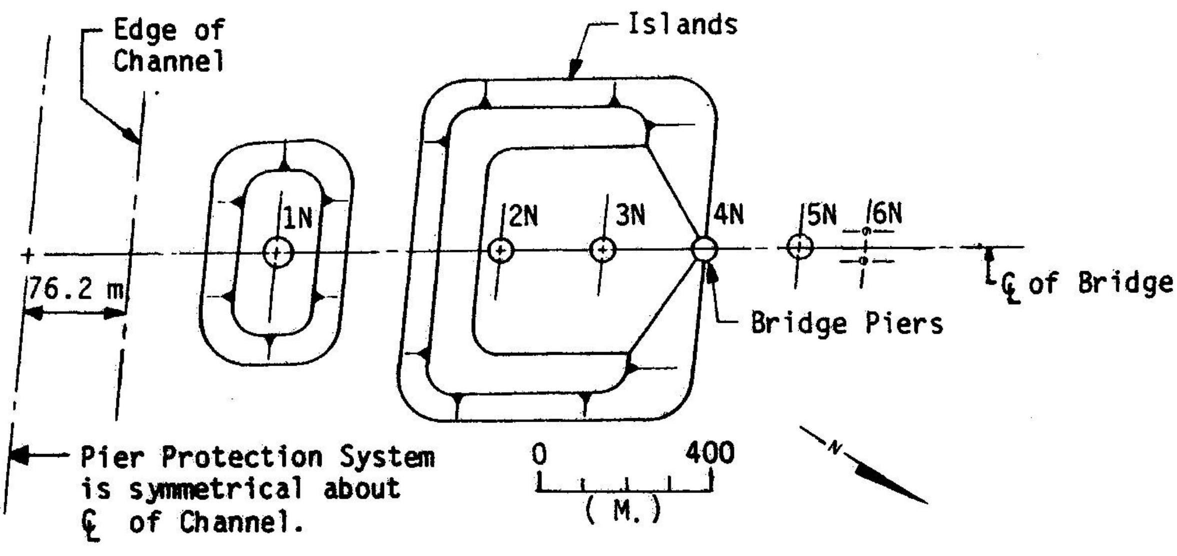


Figure 6. Plan of 6-Pier Island Protection System
(Piers 1-3, N & S)

(Fig. 5). The islands provide a high degree of ship collision protection from any direction, redirect vessels away from the piers, and stop a ship slowly, thus preventing major damage to the ship's hull. In addition, islands have a long life expectancy, are relatively maintenance free, and require only minor repairs after a ship collision. The proposed island configuration for the Skyway (Fig. 6) includes a horseshoe-shaped island, which was designed to create a positive environmental habitat in which marine life can flourish.

4. ENVIRONMENTAL IMPACTS

A preliminary environmental impact analysis was performed on the various dolphin and island alternatives under consideration. The primary focus of the study was the effect of the artifical islands on bay hydrologic patterns and on the ecology. A mathematical model was used to simulate the bay hydrologic activity and to assess the impacts of the various structural pier protection alternatives (Fig. 7). The analysis indicated that no significant changes would occur due to the dolphin alternate, and that acceptable impacts would occur due to the island alternate, providing the length of island coverage was restricted to protecting no more than six piers on each side of the channel. Extending the islands beyond this would cause excessive tidal flow velocities in the shipping channel which would adversly affect vessel operations in that area.

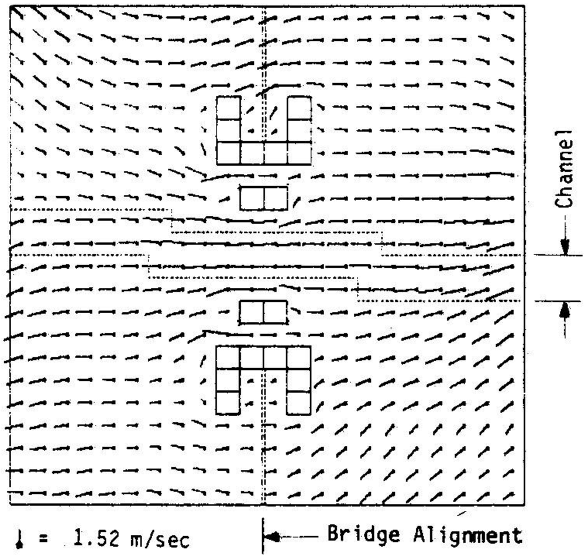


Figure 7. Typical Tidal Velocity Vectors For The 6-Pier Island Protection

5. AIDS TO NAVIGATION IMPROVEMENTS

The study recommended measures to improve the existing system of buoys and range-markers in the vicinity of the bridge. While the existing system met the minimum requirements established by the U.S. Coast Guard, it was believed that an improved system in the vicinity of the Sunshine Skyway would be of substantial benefit in providing mariners and pilots with as much navigation data as possible. The analysis indicated that 60 to 85 percent of all vessel accidents in the Bay are caused by piloting errors, with barge accidents occurring at twice the rate of ship accidents. The latter is probably a result of the lack of a requirement for all barges to utilize professional pilots.

6. ELECTRONIC NAVIGATION AID SYSTEM

In addition to assessment of the traditional aids to navigation, the study examined a variety of all-weather electronic navigation aid systems to assist a pilot in determining accurate (within 6 m) ownship position relative to the channel and the bridge. The systems studied included various types of microwave and Loran positioning systems. The system found most desirable for possible use in the Tampa Bay Harbor is based on the Loran-C signal network presently maintained by the U.S. Coast Guard. Loran-C provides ownship position information through computer processing of time-difference signals broadcast from several remote locations. Historically, the accuracy of Loran-C (usually 60 m to 800 m) has been unacceptable for precision channel navigation in restricted harbor areas. However, recent developments in the accuracy of Loran-C receivers and the development of the Loran-C survey technique have potentially alleviated this problem.

The Loran survey consists of recording time-differences at numerous positions along a shipping channel, and simultaneously computing exact ownship position referenced to a second survey system (usually a microwave system). The difference in location between the two sets of signals is the error (or the distortions) in the Loran-C grid due to the surrounding land mass. These distortions are then programmed into a small computer coupled to the Loran receiver which will then automatically compensate for the grid distortions as a vessel navigates through the channel and thus provide a pilot with accurate ownship position. To calibrate the device for daily and seasonal changes in the Loran grid, a fixed land reference station is established in the harbor area.

As part of the Skyway pier protection system, a portable, battery-powered, lightweight (less than 9.Kg) Loran-C unit as described above is being developed and tested. The unit will be carried on board a vessel by a pilot for use during the vessel's transit. The proposed unit will provide not only digital information regarding ship position and operation, but also a visual display of the vessel maneuvering within the harbor area.

7. MOTORIST WARNING SYSTEM

To protect motorists from an incident similar to the M/V Summit Venture accident, the development of a bridge warning system for both the existing and the proposed structure was undertaken. In general, the system includes vibration detectors to detect ship collisions, bridge continuity circuits to warn of a superstructure failure, weather instrumentation, closed-circuit television to monitor both ship and motorist traffic, and variable-message signs to warn motorists of bridge conditions. This warning system is recommended for the new bridge since there remains the possibility that unprotected approach piers could be struck.

In addition, the study recommended that direct VHF radio communications be established between the bay pilots and the bridge operator controlling the motorist warning system. This would enable a pilot to warn the bridge operator in the event his vessel was out of control and on a collision path with the bridge. This warning would allow the bridge operator to clear the bridge of motorist traffic prior to the potential accident.

8. THREAT ANALYSIS

The threat analysis accomplished early in the study [1] indicated that all the alternatives recommended for consideration were cost-effective. The analysis modeled the risk of vessel impacts to various bridge elements, the cost of repairing or replacing those elements damaged, the cost to the port from channel closure due to a fallen span, the cost of rerouting vehicular traffic resulting from bridge closure, and the cost of avoiding the damage by implementation of the various protection systems. The study concluded that as a minimum, the first three piers on each side of the shipping channel should have structural pier protection devices. Table 1 summarizes the cost effectiveness of the various alternatives studied.

Pier Protection Alternative	Initial Cost	Annual Maintenance	Expected Lifetime (Years)	Benefit/Cost Ratio (5% Discount)
Dolphins - 4 Piers	$17,230,000	$23,000	35	3.48
Dolphins - 6 Piers	20,022,000	26,880	35	3.32
Dolphins - 12 Piers	28,603,000	38,400	35	2.26
Islands - 4 Piers	20,440,000	7,000	50	4.59
Islands - 6 Piers	24,080,000	14,000	50	4.33
Islands - 12 Piers	34,240,000	28,000	50	3.54
Standard Navigation Improvements	1,000,000	6,000	20	17.33
Electronic Navigation System	600,000	8,000	10	6.49
Motorist Warning System	220,000	5,000	10	4.26

Table 1. Benefit/Cost Ratios for Pier Protection Alternatives

9. SUMMARY

The preliminary investigation revealed that if the new Sunshine Skyway bridge were to be unprotected, it would still be relatively vulnerable to possible ship collisions and, therefore, the implementation of adequate pier protection devices would be required. As a percentage of the overall bridge construction, the implementation of a 6-pier protection structural system, navigation improvements, electronic navigation device, and the motorist warning system represents approximately 23 percent (26 million dollars) of the total bridge cost, and 34 percent of the high level approaches and main span cost. The high cost associated with adequate protection is a result of the increased probability of catastrophic ship collision as larger and more frequent ships and barges utilize the Tampa Bay channel system.

REFERENCE

1. Knott, M. and Bonyun, D.: Threat Analysis for Ship Collisions against the Sunshine Skyway Bridge. IABSE Colloquium on Ship Collision with Bridges and Offshore Structures. Publications IABSE, 1983.

APPENDIX

Photograph of the M/V Summit Venture accident with the existing Sunshine Skyway Bridge on May 9, 1982.

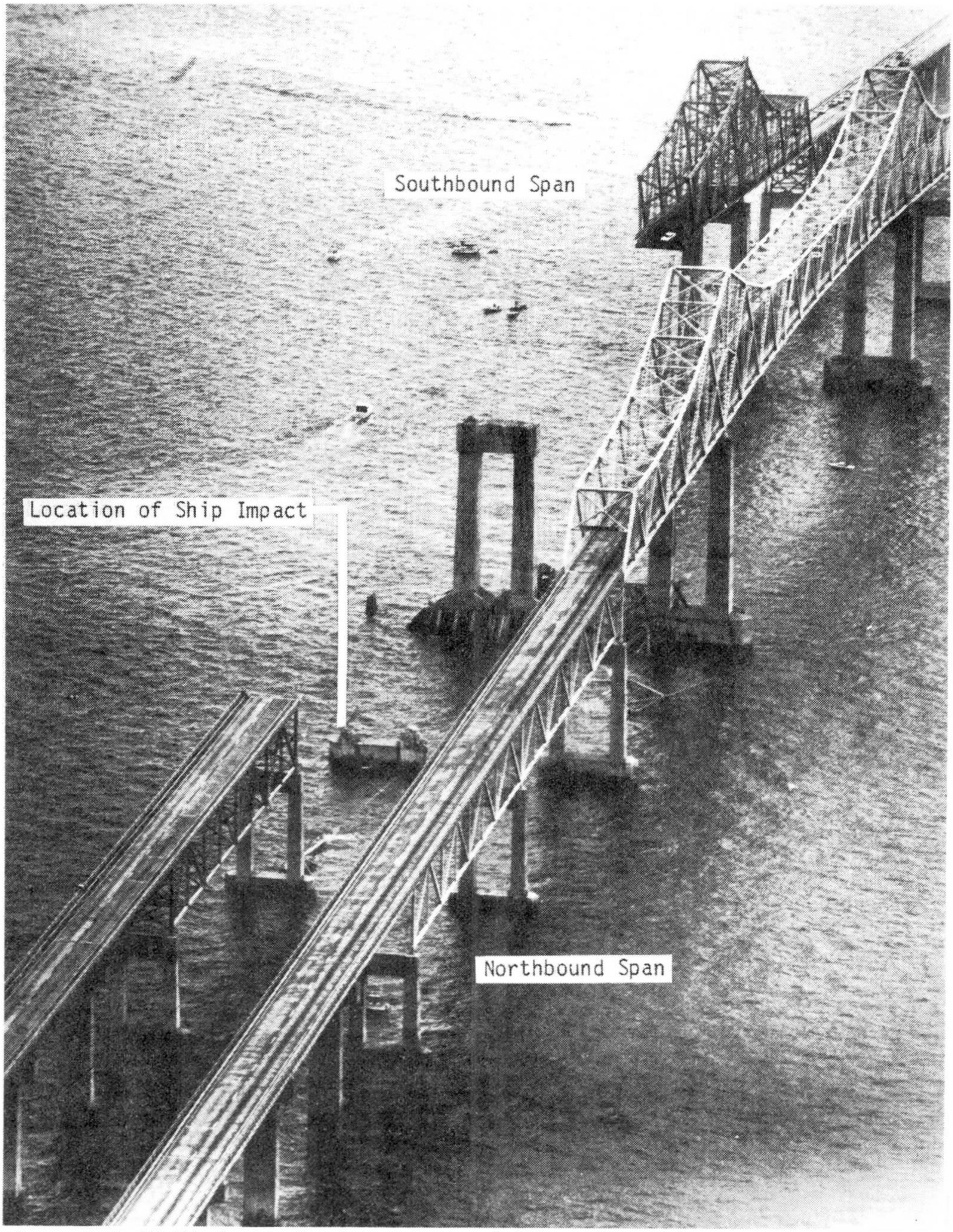


Photo: T. P. O'Neill, Courtesy Shackleford, Farrior, Stallings & Evans, P.A.

Sunshine Skyway Bridge, May 9, 1980 after being struck by the M/V <u>Summit Venture</u>

Protection des ouvrages en mer contre le choc des bateaux

Schutz von »Offshore«-Bauten gegen Schiffskollisionen

Protection of Offshore Structures against Ship Collisions

Roger LACROIX
Ingénieur Consultant
Ecole Polytechnique
Rungis, France

Roger Lacroix, né en 1928, est diplômé de l'Ecole Polytechnique, et Ingénieur des Ponts et Chaussées. Il est responsable du Département Construction à l'Ecole Nationale des Ponts et Chaussées.
Roger Lacroix est Président Honoraire de la Fédération Internationale de la Précontrainte, et Président de l'Association Française du Béton.

RÉSUMÉ
Les câbles élastiques constituent la seule solution envisageable pour protéger efficacement une structure offshore contre le choc de bateaux, dès que la profondeur d'eau dépasse quelques dizaines de mètres. L'article montre l'exemple d'une protection éloignée indépendante de la structure, pour l'arrêt des gros navires, et d'une protection rapprochée, pour les bateaux de faible et moyen tonnage.

ZUSAMMENFASSUNG
Um die Bauwerke im Meer gegen die Schiffsstöße zu schützen, gibt es nur eine Lösung, nämlich die aus elastischen Seilen besteht, sofern die Wassertiefe größer als einige zehn Meter ist. Der Artikel beschreibt einen entfernten, von dem Werke unabhängigen Schutz, für große Schiffe, und einen nahen Schutz, für kleine oder mittelgroße Schiffe.

SUMMARY
The only possible solution to protect efficiently an offshore structure against ship collisions consists of elastic cables if the water is deeper than some dozens of metres. The article gives an example of a distant protection, independant of the structure, for the protection against large tankers, and of a close protection, against small or middle ships.

1. INTRODUCTION

Qu'il s'agisse d'appuis de ponts, ou de plates-formes d'exploitation pétrolière, les ouvrages en mer sont de plus en plus nombreux aujourd'hui. Les piles des ouvrages de franchissement sont particulièrement vulnérables au choc des bateaux, et plusieurs accidents spectaculaires ont défrayé la chronique au cours des dernières années, provoquant, outre des dégâts matériels importants, la perte d'un nombre élevé de vies humaines.

La réalisation d'une protection efficace est extrêmement difficile : les masses des navires sont souvent énormes, très supérieures à celles des ouvrages qu'ils peuvent heurter, et il est donc tout à fait exclu de concevoir une structure susceptible de résister par elle-même à un choc de plein fouet.

2. PROTECTION ELOIGNEE

Lorsque la nature et le profil du fond le permettent, la meilleure protection consiste à disposer autour de l'ouvrage des massifs d'enrochements sur lesquels les navires peuvent s'échouer sans trop de dommages ; mais une telle solution ne convient que pour autant que la profondeur n'est pas trop grande, car, pour réserver un chenal de navigation de largeur suffisante, il faudrait envisager des travées de très grande longueur, en raison de l'encombrement des talus nécessaires à la stabilité des massifs. De plus, le coût d'extraction, d'approvisionnement et de mise en place des matériaux nécessaires à la constitution d'un massif croît sensiblement proportionnellement au cube de la profondeur, et devient prohibitif lorsque celle-ci dépasse quelques dizaines de mètres.

Or, pour certains grands franchissements étudiés aujourd'hui, tels que celui du Détroit de GIBRALTAR, par exemple, et pour proportion notable des plates-formes offshore, la hauteur d'eau à prendre en compte peut atteindre 200 m ou davantage. Des projets sont proposés pour des fonds dépassant largement ces valeurs, et il convenait donc de prévoir un dispositif permettant de s'affranchir, dans toute la mesure du possible, de la nécessité d'un appui au sol.

L'écran de protection Vitalis [1] répond à cette condition. Nous en rappellerons brièvement le principe, en laissant au lecteur le soin de se reporter à l'article cité en référence [2] pour de plus amples détails.

Il s'agit de câbles horizontaux maintenus à la surface de l'eau par un système de flotteurs, et tendus grâce à des lignes ancrées sur le fond.

Le principe de l'écran est d'offrir une résistance d'élasticité variable, susceptible d'annuler une énergie cinétique suffisante. Si $\vec{F}(s)$ est la résistance opposée au navire qui a parcouru la distance s à partir de son impact sur l'écran, l'arrêt a lieu lorsque s atteint une valeur x telle que

$$W = \int_0^x \vec{F} \cdot \vec{ds}$$

W désignant l'énergie cinétique du navire après le choc sur l'écran.

La fonction $\vec{F}(s)$ dépend à la fois de la configuration géométrique de l'écran, des caractères mécaniques de ses éléments, de la position du point d'impact, et de la direction de l'impact.

Pour un écran et une énergie cinétique donnés, on peut, à l'aide d'un programme de calcul électronique simple, déterminer le lieu des points correspondant à l'arrêt du navire (fig. 1), et vérifier ainsi que la structure dispose d'une protection suffisante.

Les éléments concourant à l'élasticité du système sont au nombre de tois :
1 : l'élasticité propre des câbles principaux,
2 : le changement de configuration géométrique après le choc,
3 : l'élasticité des lignes d'ancrage.

Pour les câbles principaux, des cordages tressés en nylon présentent à la fois l'avantage d'être insensibles à la corrosion, et de présenter un allongement de rupture important. La fig. 2 donne un exemple de la courbe contrainte-allongement d'un tel câble ; l'on y voit que pour une contrainte de traction égale à la moitié de la résistance ultime, l'allongement atteint 25 %, d'où la possibilité d'emmagasiner une quantité importante d'énergie.

Les lignes d'ancrage, dont la forme est celle d'une chaînette, comportent une élasticité décroissant avec la tention, dont l'intensité peut, elle aussi, faire l'objet d'un réglage.

Ces lignes peuvent être constituées par des chaînes de type classique, analogues à celles qui sont utilisées, par exemple, pour les ancrages des plates-formes flottantes semi-submersibles de l'industrie pétrolière. Les ancrages au sol peuvent être réalisés par des pieux battus, ou éventuellement par des corps morts (ceux-ci ont fait l'objet d'une étude détaillée effectuée par l'Institut Français du Pétrole, et les Ateliers et Chantiers de Bretagne).

L'étude de détail de l'écran doit comporter une vérification de la stabilité des flotteurs principaux et secondaires, ainsi que de leur mode de construction et de mise en place [2].

Enfin, comme tout ouvrage à la mer, l'écran de protection devra faire l'objet d'inspections périodiques ; les câbles en nylon ne sont pas sujets à la corrosion, mais ils sont particulièrement sensibles à l'abrasion par frottement ; il conviendra donc de revêtir les parties coisines des flotteurs, qui peuvent venir au contact de pièces fixes sous l'action de la houle. L'entretien pourra consister dans le remplacement de certains composants, en cas d'usure anormale. Les flotteurs, s'ils sont construits en béton (et notamment en béton léger, suivant une formule mise au point par l'ARBEM [3]),ne nécessitent en principe aucun entretien particulier.

3. PROTECTION RAPPROCHEE

Pour les ouvrages des champs pétroliers en mer, et aussi, dans une moindre mesure, pour les piles de pont, il est nécessaire de prévoir l'accès de bateaux de faible ou moyen tonnage, utilisés pour l'avitaillement, ou pour certains travaux d'entretien. Dès que la mer est quelque peu agitée, et même pour une faible houle, il est nécessaire de prévoir un système de défense, permettant d'éviter un choc direct du bateau sur la structure.

Un tel choc, provenant d'un bateau de service, a d'ailleurs récemment provoqué un léger enfoncement de la partie supérieure d'une colonne de plate-forme offshore, sur une surface de l'ordre de 1,5 à 2 m^2. Ce désordre a pu être réparé en dégageant les armatures, et en reconstituant l'intégrité du béton fissuré, mais il constitue un avertissement sérieux, qui montre clairement la nécessité d'une protection.

Une solution classique consiste à disposer des défenses, qui absorbent l'énergie cinétique du bateau par distorsion de blocs de néoprène, compression ou extension de ressorts, ascension de masses, etc... Ces systèmes, qui sont d'usage courant pour les quais et appontements, peuvent être très efficaces, mais ils présentent deux inconvénients importants lorsqu'ils s'appliquent aux ouvrages offshore :

- Le premier concerne la pression exercée sur l'ouvrage, pression qui, bien qu'amoindrie par la défense élastique interposée, peut être excessive. En effet, afin d'accroître la stabilité des structures offshore, il importe de conférer un poids minimal aux parties les plus élevées, c'est-à-dire celles qui sont au-dessus du niveau de la mer en situation définitive. Les concepteurs de ces structures sont donc amenés à concevoir des colonnes ou piles

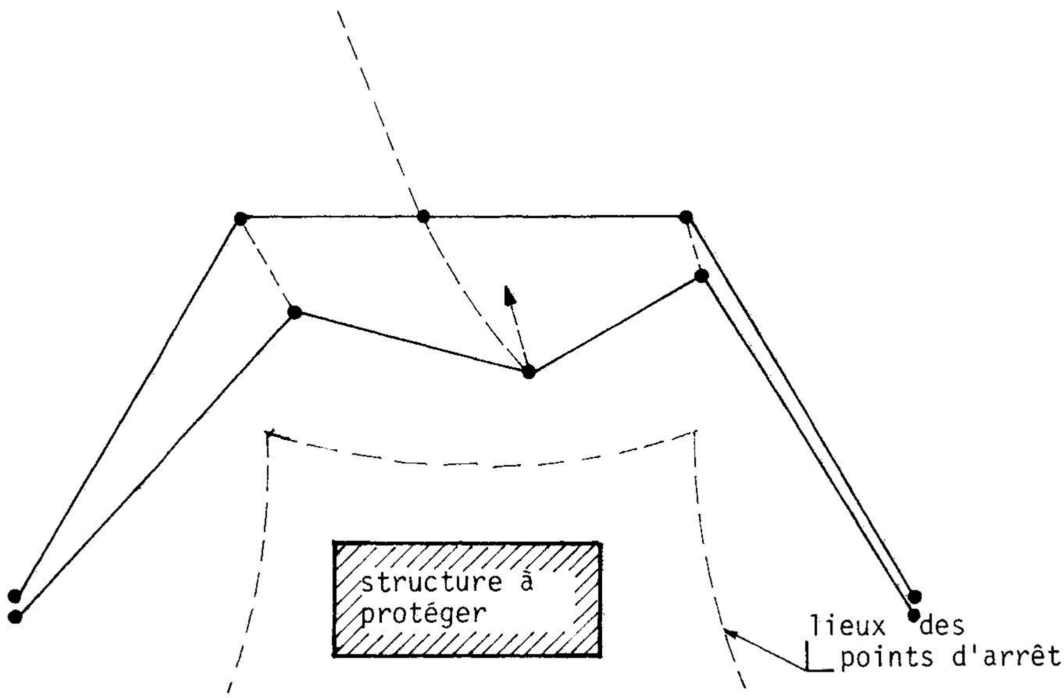


Fig. 1 - Déformation de l'écran

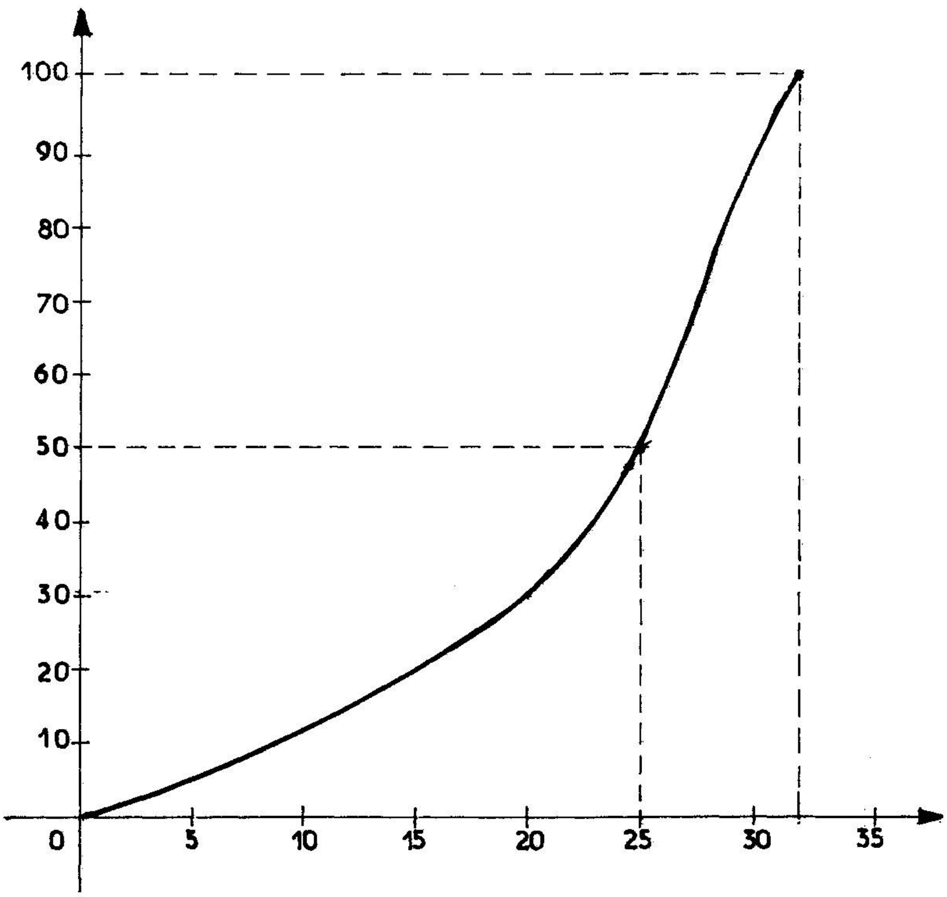


Fig. 2 - Courbe contrainte-déformation d'un cordage en nylon tressé

creuses, dont les parois sont d'épaisseur assez faible, de l'ordre de 40 à 50 cm, par exemple, pour un diamètre de l'ordre d'une dizaine de mètres. Or, si ferraillées soient-elles, de telles parois résistent fort mal à des forces localisées.

- Le deuxième inconvénient des défenses classiques réside dans leur encombrement : ces dispositifs, fixés sur les colonnes, augmentent la surface offerte aux vagues, et, comme c'est précisément au voisinage de la surface que les forces de houle sont les plus grandes, il en résulte un accroissement non négligeable des sollicitations correspondantes.

Le dispositif décrit ci-après, inventé par Monsieur Jean SOLOMOVICI [4] répond à ces objections.

Le système de base consiste en un câble horizontal entourant la colonne à protéger, et de forme polygonale en plan (fig. 3). Le câble est maintenu à chaque sommet du polygone par un support dans lequel il peut coulisser, et qui est lui-même fixé au sommet d'une charpente légère tubulaire, qui forme un tétraèdre ; le tétraèdre est appliqué sur la colonne par sa face opposée au sommet qui porte le câble.

Les tétraèdres sont articulés autour d'un axe horizontal qui passe par la base du triangle d'appui sur la colonne. Leur sommet le plus haut, situé également contre la colonne, porte également un support, dans lequel peut coulisser un câble de retenue, soumis à une tension permanente, de façon à appliquer les sommets des tétraèdres contre la colonne.

Lorsqu'une force d'impact horizontale est appliquée en un point quelconque du câble de protection inférieur, celui-ci s'allonge en coulissant dans ses supports, et il exerce sur les sommets inférieurs des deux tétraèdres les plus voisins des forces dirigées vers la colonne. Ces forces tendent à faire pivoter les tétraèdres autour de leur axe horizontal, et donc à décoller de la colonne les sommets supérieurs ; mais ce décollement ne peut se produire qu'au prix d'un allongement du câble de retenue supérieur, qui, lui aussi, peut coulisser dans ses supports.

L'allongement élastique nécessaire pour freiner le bateau sur la distance qui sépare le point d'impact sur le câble de la paroi extérieure de la structure, est donc fourni par l'allongement des deux câbles, sur une grande partie de leur longueur.

Les tétraèdres supports des câbles peuvent être, soit fixés à la colonne par les deux sommets de leur arête horizontale, soit plus simplement suspendus par des câbles verticaux, ce qui permet de les déposer aisément pour leur entretien, ou leur remplacement éventuel en cas d'incident.

A titre d'exemple, un dispositif de défense susceptible de protéger une colonne de 9 m de diamètre contre le choc d'un bateau de 2 500 t animé d'une vitesse de 1,5 m/sec pourrait être muni de câbles d'acier, d'une résistance ultime de 2 MN, capables d'un allongement ultime de 3 %.

Bien entendu, un tel système doit être protégé contre la corrosion, soit au prix d'un entretien périodique, soit, de façon plus simple, grâce à l'emploi d'acier inoxydable.

4. CONCLUSION

Contrairement à ce qu'on a pu affirmer, il est exclu de prévenir le choc des bateaux sur les structures en mer en éditant simplement une règlementation sur la navigation ; pour les piles de pont encadrant un chenal, notamment, un incident de gouvernail ou une négligence suffisent pour créer un risque très important d'accident. Il est donc indispensable de prévoir une protection.

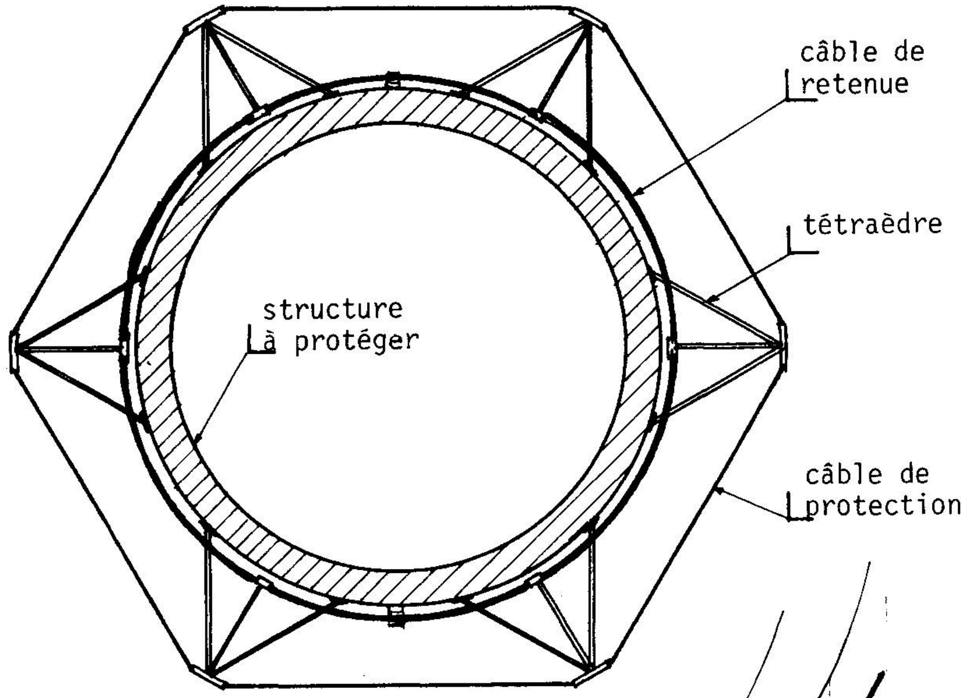


Fig. 3 - Dispositif de protection rapprochée - Vue en plan

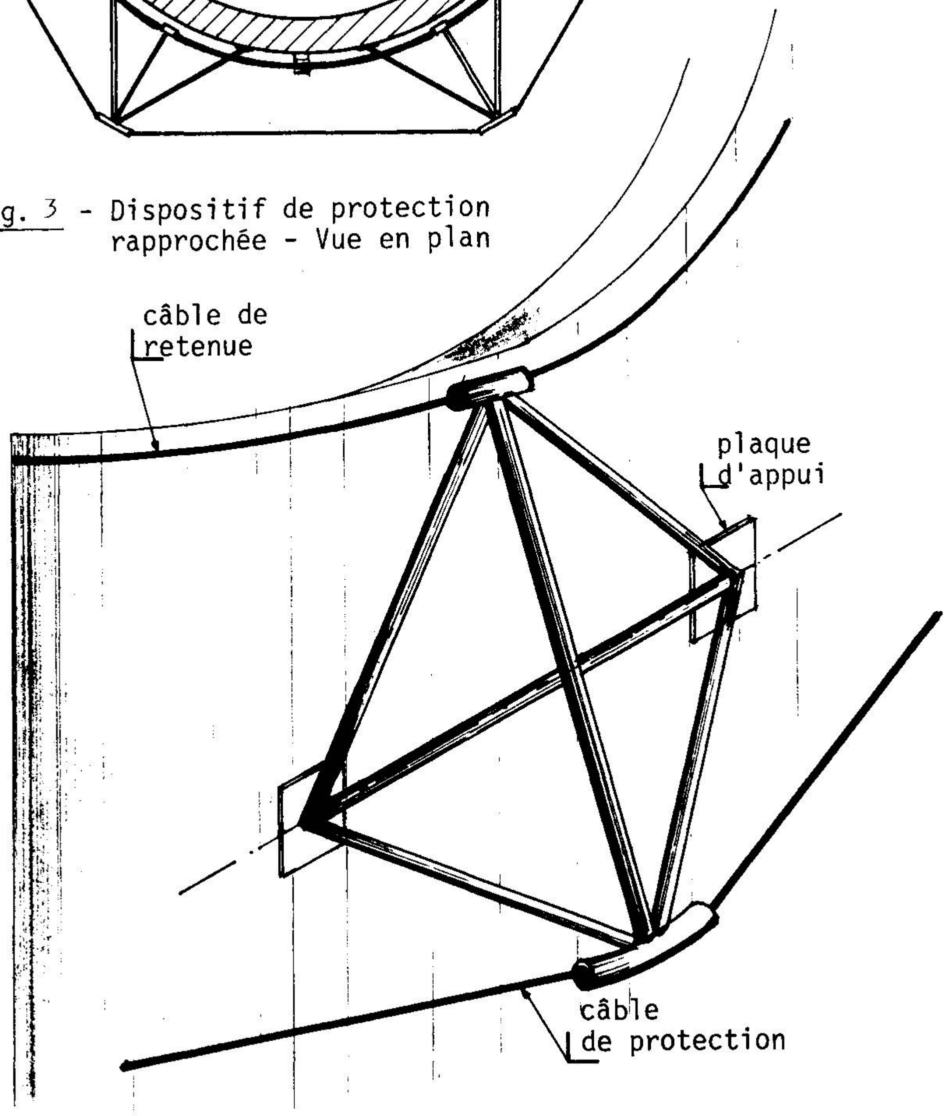


Fig. 4 - Détail d'un tétraèdre support

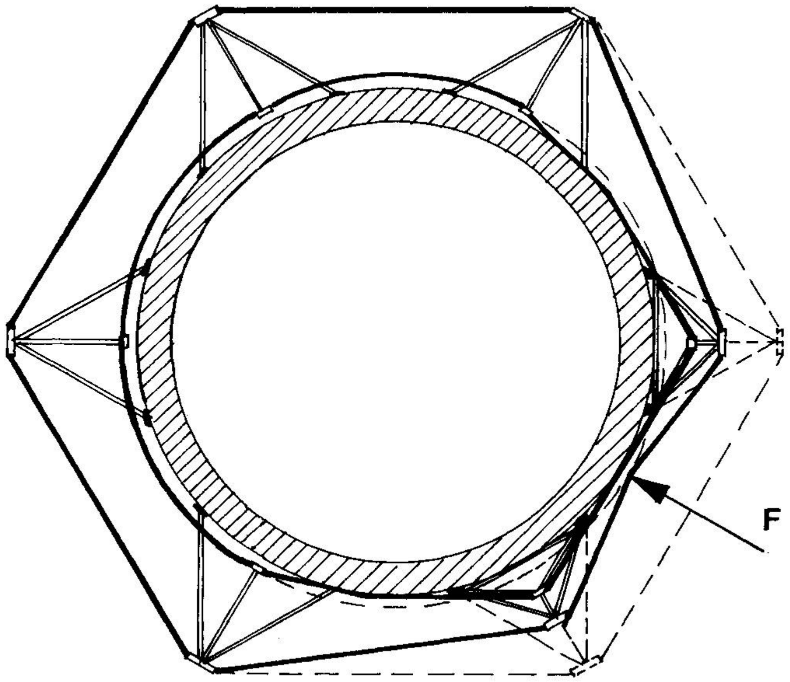


Fig.5 - Configuration du dispositif de protection après le choc

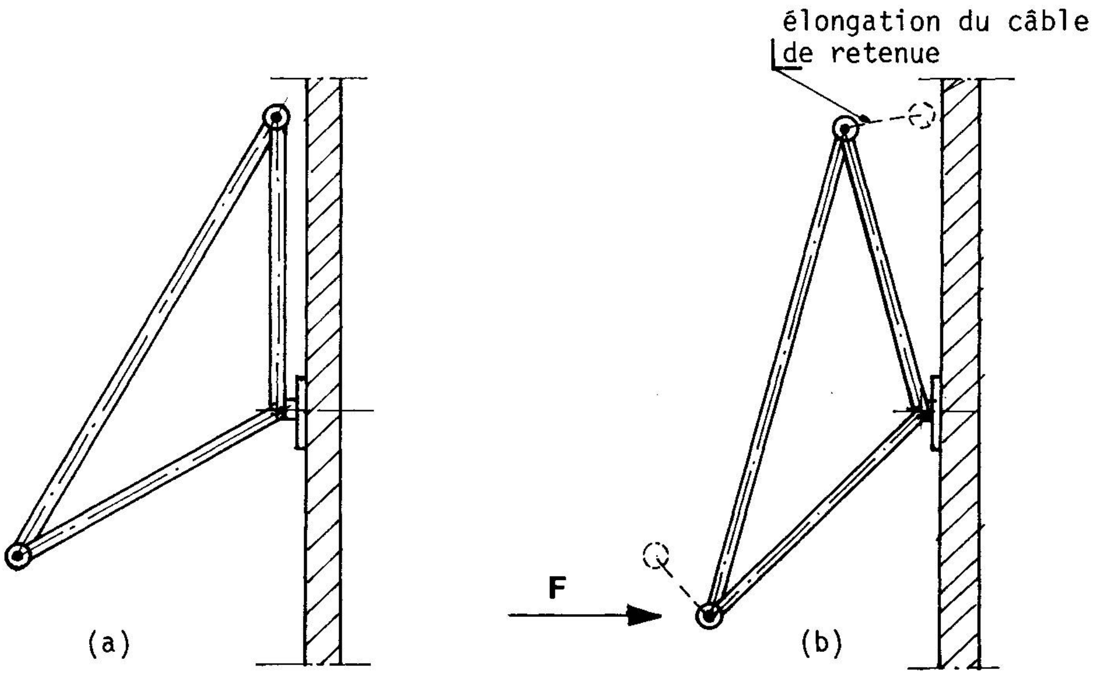


Fig.6 - Coupe au droit d'un tétraèdre
(a) au repos (b) après choc

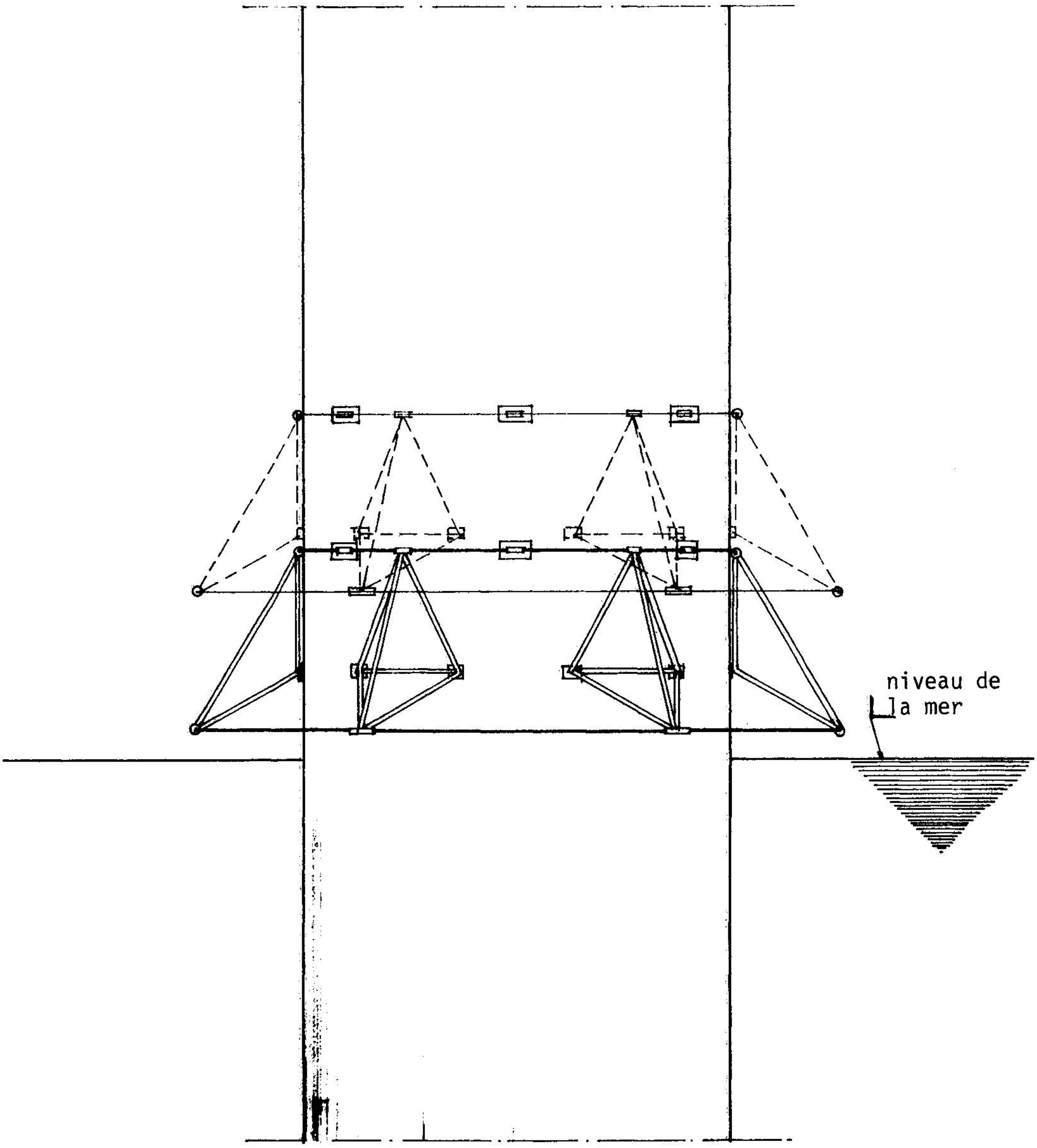


Fig. 7 - Vue en élévation d'une double rangée de câbles de protection

D'autre part, les lois de la Mécanique sont inéluctables : l'énergie cinétique d'un navire, augmentée de celle de la masse d'eau entraînée, ne peut être annulée, ou tout au moins réduite à une valeur assez faible pour être inoffensive, qu'au prix d'une longueur de freinage suffisante, si l'on veut limiter la force de retenue. Or, dès que la masse du navire croît, la force de retenue et la longueur de freinage deviennent très importantes ; pour obtenir une élasticité suffisante, il faut donc mobiliser une longueur importante de câble. Les dispositifs décrits plus haut, grâce à leur configuration, permettent ainsi de soustraire les structures aux chocs sans pour autant endommager les bateaux. D'autres configurations peuvent être imaginées ; mais de véritables progrès ne peuvent être obtenus que par une amélioration des matériaux ; compte-tenu du coût énorme d'un accident, en termes de réparation, ou de pertes d'exploitation, un effort de recherche s'impose, afin d'améliorer les caractéristiques des câbles nécessaires en toute hypothèse pour obtenir un freinage élastique convenable.

REFERENCES

1. Brevet Français n° 78 19 124 du 27 juin 1978

2. VITALIS A., LACROIX R., BONNEMAIRE B., "Ecran de Protection Anti-Collision" Proceedings of the International Symposium on offshore structures held at COPPE, RIO DE JANEIRO - Pentech Press LONDRES.

3. LESAGE J., MARTIN M., "Recherche d'amélioration des performances des bétons légers". 2ème colloque intern. sur la tenue des ouvrages en béton en mer CNEXO - BREST 1982.

4. Brevet Français n° 81 19 218 déposé le 13 octobre 1981.

Leere Seite
Blank page
Page vide

Collision Prevention Device of Floating Guide-Line Type

Ecran de protection anti-collision de type ligne d'ancrage flottante

Stoßdämpfer mit dem flotten Führungstau

Kazuki ODA
Prof. of Hydraulic Eng.
Osaka City Univ.
Osaka, Japan

Sunao KUBO
Assist Prof.
Osaka City Univ.
Osaka, Japan

azuki Oda, born 1941, received his M.E. and D.E. egrees from the Osaka City University in 1965 and 1976, ɜspectively. Most of his subsequent researches have een concerned with studies of mooring dynamics of uoys and tankers, wave forces on structures and safety avigation problems relating to the Honshu-Shikoku ɪridges.

Sunao Kubo, born 1934, graduated from Osaka Institute of Technology in 1957 with a bachelor degree in civil engineering. Since then, he has been mainly engaged in studies of wave forces on breakwaters and hydraulic model experiments on planning and layout of ports.

SUMMARY
This paper deals with the theoretical and experimental analyses on the collision prevention device of floating guide-line type, and describes the outline of the device put into practical use for preventing ship collisions with a floating platform for geological survey of the seabed.

RÉSUMÉ
L'étude traite de l'analyse théorique et expérimentale d'un écran de protection anti-collision, type ligne d'ancrage flottante; il décrit les écrans utilisés de façon préventive contre les collisions des navires avec des plates-formes flottantes utilisées pour le relevé géologique du fond de la mer.

ZUSAMMENFASSUNG
Der Aufsatz behandelt die theoretische und experimentelle Analyse von Kollisionsschutzmaßnahmen, insbesondere von schwimmenden Schutzmitteln. Es wird über die Anwendung für eine schwimmende Plattform berichtet, welche für geologische Aufnahmen des Meeresbodens verwendet wird.

1. INTRODUCTION

Since December in 1975, the construction works of the long bridges, which aim to link the Shikoku Island to the Main Land of Japan, have been carried out at different three routes by the Honshu-Shikoku Bridge Authority.

Since most of their piers have been constructed and are to be constructed in narrow straits under severe circumstances for navigation: strong currents, fogging, and extremly frequent marine traffic, a great possibility of ship-bridge pier collisions can be anticipated.

Therefore, since 1970, various kinds of appropriate measures to prevent the ship-bridge pier collision and to protect those bridge piers against ship impacts have been investigated under the Safety Navigation Committee for the Construction of Honshu-Shikoku Bridges.

The collision-prevention device presented here is one trial of those measures and of a floating guide-line type capable of turning a ship, which is rushing to the pier, at a relatively small angular velocity away from her original traveling course. The device was put to practical use in 1973 for preventing ship collisions with a floating platform for geological survey of the seabed at the projected construction location of the piers of Honshu-Shikoku Bridges.

2. STRUCTURAL OUTLINE OF DEVICE

The device is structured by two floating guide-lines stretched in V-shape on the water surface and three large buoys maintaining both ends of each guide-line which are anchored to the sea bottom as shown in Fig.1. Each guide-line is composed of pneumatic rubber fenders tied in a row, tightened with restoring forces of anchored buoy systems.

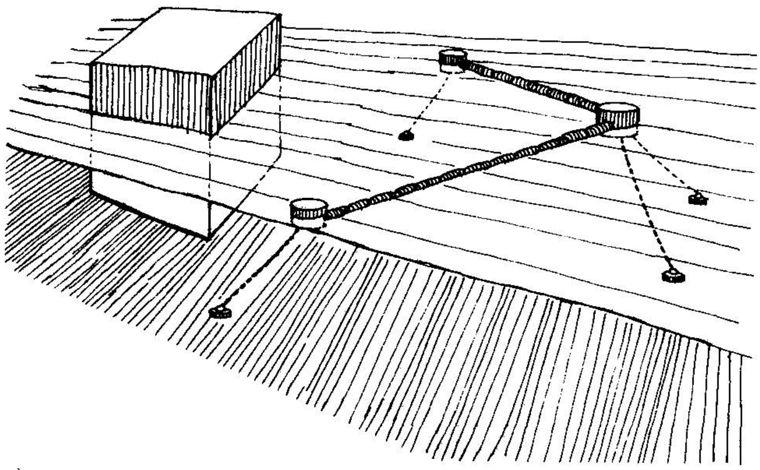


Fig. 1 Structure of the Collision-Prevention Device

3. THEORETICAL AND EXPERIMENTAL ANALYSES

3.1 Theory

Using a set of co-oridinate axes x_G and y_G with the origin fixed at the center of ship as shown in Fig.2, we can write the basic equations of ship motions in horizontal plane as

$$\left.\begin{aligned} m(\dot{u}_G - \omega v_G) &= X \quad \text{(sway)} \\ m(\dot{v}_G + \omega u_G) &= Y \quad \text{(surge)} \\ I_{zz}\,\dot{\omega} &= N \quad \text{(yaw)} \end{aligned}\right\} \qquad (1)$$

in which m = ship mass; I_{zz} = moment of inretia about the vertical axis; u_G, v_G, $\dot{u}_G$ and $\dot{v}_G$ = components of ship velocity and acceleration in the x_G- and y_G-direction, respectvely;ω and $\dot{\omega}$ = angular velocity and acceleration about the vertical axis; and X, Y and N = external forces and moments including hydrodynamic and non-hydrodynamic ones.

For analyzing the motions of ship while contacting with the floating guide-line, take the co-ordinate system and the symbol definition as shown in Fig.2, and introduce the following assumptions to simplify the subsequent, theoretical analyses:

(a) While the ship contact with the guide-line AB, the movements of the main buoy A, another guide-line AC and the sub-buoy C can be ignored.
(b) The hydrodynamic forces on the sub-buoy B and the guide-line AB, and the frictinal force of the guide-line on the ship hull can be also ignored.
(c) The configulation of mooring line can be regarded as a straight, and the elongations of mooring lines and the guide-line can be ignored.
(d) The propulsive force of the ship is constant, and the rudder angle of the ship is stationarily zero.
(e) The ship holds on contacting with the guide-line at the bow shoulder.

Then, the external forces X and Y exerted on the ship, and the external moments N in Eq.(1) are approximately expressed by (1)

$$\left.\begin{aligned} X &= -m_x\dot{u}_G + f_x \\ Y &= -m_y\dot{v}_G - cv_G^2 + f_p + f_y \\ N &= -J_{zz}\dot{\omega} + f_\phi \end{aligned}\right\} \qquad (2)$$

in which m_x and m_y = added masses in the x_G- and y_G- direction; J_{zz} = added moment of inertia about the vertical axis; c = dimensional coefficient of total ship resistances; f_p = propulsive force of ship; f_x, f_y and f_ϕ = components of reactive forces and moments on the ship hull exerted by the deflection of guide-line AB in the x_G- and y_G-direction given by

$$\left.\begin{aligned} f_x = -T_{AB}\{ & \left(\frac{x-b_x}{\sqrt{(x-b_x)^2+(y-b_y)^2}} + \frac{x}{\sqrt{x^2+y^2}}\right)\cos\phi \\ & + \left(\frac{y-b_y}{\sqrt{(x-b_x)^2+(y-b_y)^2}} + \frac{y}{\sqrt{x^2+y^2}}\right)\sin\phi\} \end{aligned}\right\} \qquad (3)$$

$$\left.\begin{aligned} f_y = -T_{AB}\{ & \left(\frac{y-b_y}{\sqrt{(x-b_x)^2+(y-b_y)^2}} + \frac{y}{\sqrt{x^2+Y^2}}\right)\cos\phi \\ & + \left(\frac{x-b_x}{\sqrt{(x-b_x)^2+(y-b_y)^2}} + \frac{x}{\sqrt{x^2+y^2}}\right)\sin\phi\} \end{aligned}\right\} \qquad (4)$$

and $$f_\phi = -(af_x - bf_y) \qquad (5)$$

in which x,y = co-ordinates of the point of contact P with ship; b_x,b_y = co-ordinates of anchor point B_B; T_{AB} = tention of guide-line AB; ϕ = heading angle of ship to the Y-axis; and a,b = lengths of moment arm of f_x,f_y around the gravity center of ship.

Forming the equilibrium equation of forces acting on the sub-buoy B, we obtain the equation for T_{AB} as

$$T_{AB} = \{w_0 A_B(h_B - \sqrt{S_B^2 - l_B^2} - W_B\} \frac{l_B}{\sqrt{S_B^2 - l_B^2}} \qquad (6)$$

with $$l_B = \sqrt{x^2 + y^2} + \sqrt{(x - b_x)^2 + (y - b_y)^2} - l_{AB} \qquad (7)$$

in which w_0 = the specific weight of water; A_B = the waterplane area of the sub-buoy B; h_B = the water depth at anchor point B_B; S_B = the length of the mooring line of sub-buoy B; W_B = the weight of sub-buoy B; l_B = the horizontal distance between the sub-buoy B and the anchor point B_B; and l_{AB} = the original distance between the main buoy A and the sub-buoy B.

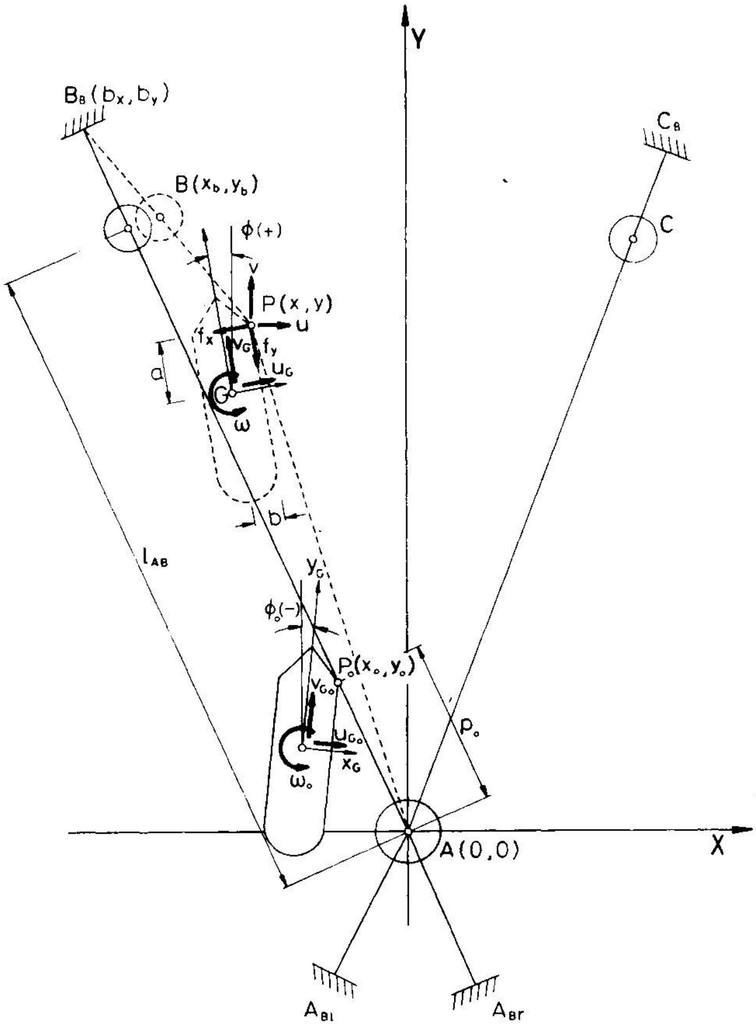


Fig.2 Co-ordinate system and symbol defenition of the collision-prevention device

On the other hand, reffering to Fig.2, we can express the relationship between the velocity components of ship parallel to the x_G- and y_G-axes fixed on the ship at her center of gravity and parallel to the x- and y-axes fixed on the earth at her point of contact P with the guid-line as

$$\left.\begin{aligned} u_G &= u\cos\phi + v\sin\phi + a\,\omega \\ v_G &= v\cos\phi - u\sin\phi - b\,\omega \end{aligned}\right\} \qquad (8)$$

with $$u = \dot{x},\ v = \dot{y} \text{ and } \omega = \dot{\phi} \qquad (9)$$

Finally, Eq.(1) can be rewritten into six simultaneous non-liner differential equations in terms of x and y, co-ordinates of the point of contact P and can be solved by means of numerical analysis using the Runge-Kutta-Gill method.

3.2 Numerical Analysis

Figs.3 and 4 show examples of neumerically calculated results of the maximum tension of guide-line $(T_{AB})_{max}$ and the maximum displacement of sub-buoy in the x-direction $(\delta_B)_{max}$ in the case that the device is symmetrically arranged with respect to the y-axis in the water with a uniform depth of 25 m; the ship of 1000G.T. is approaching at a velocity of 10 knots; l_{AB} = 80 m; A_B = 28.3 m^2; $2\beta_0$= 35^0 in which $2\beta_0$ is the included angle between two guide-lines AB and AC; and $(T_{AB})_0$ = 50 tf (490 kN) in which $(T_{AB})_0$ is the initial tension of guide-line.

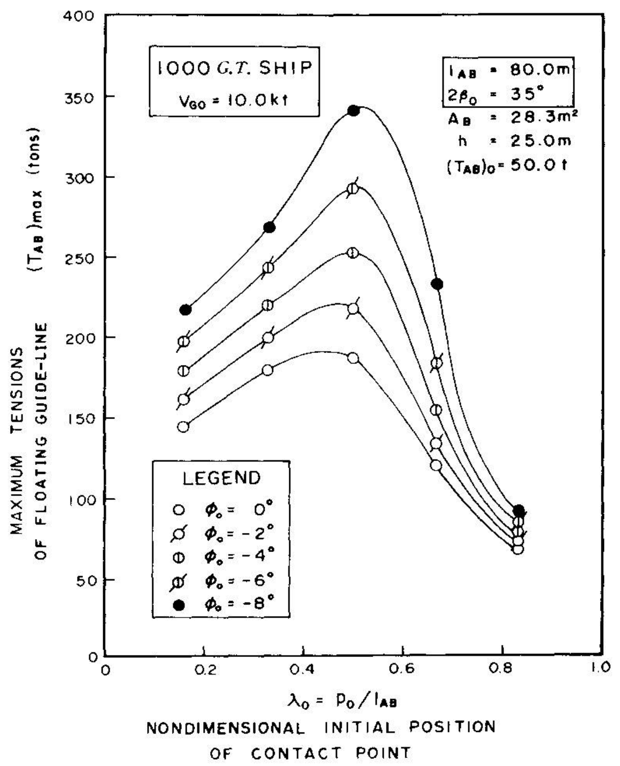


Fig.3 Example of calculated results of $(T_{AB})_{max}$

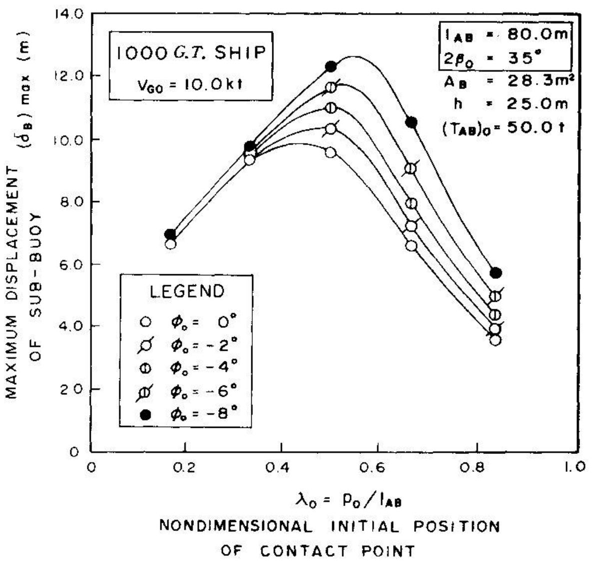


Fig.4 Example of calculated results of $(\delta_B)_{max}$

It can be seen in Figs.3 and 4 that both $(T_{AB})_{max}$ and $(\delta_B)_{max}$ are sharply affected by nondimensional initial contact position $\lambda_0 = p_0/l_{AB}$ in which p_0 is the distance between the initial contact point of ship with guide-line and the origin of co-ordinates A, and also show the peak values at $p_0 \simeq 0.5$ which increse with the initial heading angle of ship agaist the guide-line ϕ_0.

3.3 Model Tests

Model tests were conducted on a scale of 1:20 in a water basin 42 m long, 12 m wide and 0.75 m deep. A radio-controlled model ship corresponding to 880 G.T. cargo ship in prototype was used.

The quantities measured in the model tests are the approaching velocity $v_{G\theta}$ and the initial heading angle ϕ_0 of model ship; the position of initial contact point with guide-line p_0; the displacement of sub-buoy in the x-direction δ_B; the tension of guide-line T_{AB}; the tension of the mooring line of sub-buoy T_B; the tension of the mooring line of main buoy on the right side T_{Ar}.

The tensions were measured with sensitive ring-gauges on which strain gauges are mounted. Custom-made mini-turnbuckle were used to facilitate the adjustment of the initial tension of guide-line.

The ship motions and displacement of sub-buoy in horizontal plane were measured by means of analyzing 16 mm movies taken from the top of tower 6 m high above the still water level. Fig.5 shows an example of frame-photographs printed from the 16 mm movie films.

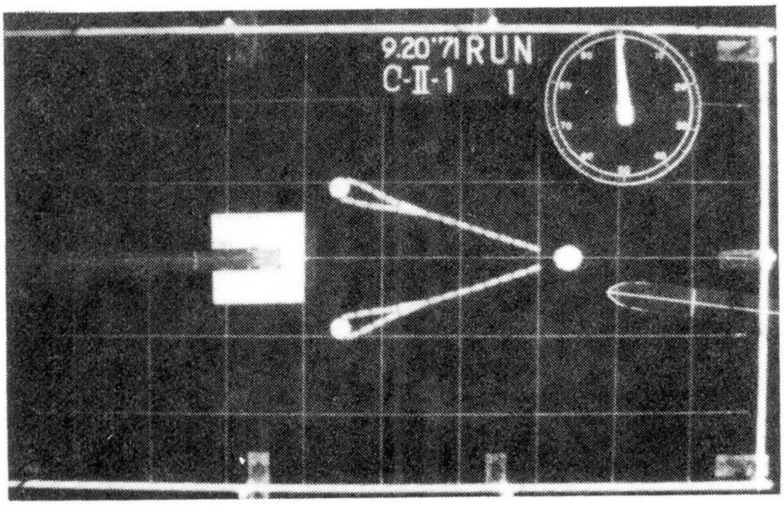


Fig.5 Frame-photograph of 16 mm movie films

3.4 Experimental Verification of the Theory

The theoretically calculated values of the maximum tentions of guide-line $(T_{AB})_{max}$ and moorig lines of buoys $(T_B)_{max}$ and $(T_{Ar})_{max}$ and also the maximum displacement of sub-buoy $(\delta_B)_{max}$ in the x-direction were compared with their experimental ones respectively.

Figs.6 and 7 show the comparisons between the calculated and experimental values for $(T_{AB})_{max}$ and $(\delta_B)_{max}$. The theoretical values are found, from these figures, to be in reasonably good agreement with the experimental ones in the case that l_{AB} = 160 cm and $2\beta_0$ = 37^0, but poor in other two cases that $2\beta_0$ is relatively large. The same trends were found for the tensions of mooring lines of buoys $(T_B)_{max}$ and $(T_{Ar})_{max}$.

The major reasons for much deviation of the theoretical values from the experimental ones in the case that $2\beta_0$ is relatively large, may be considered to be in discrepancy of the assumptions (b) and (e) discribed in sub-chapter 3.1.

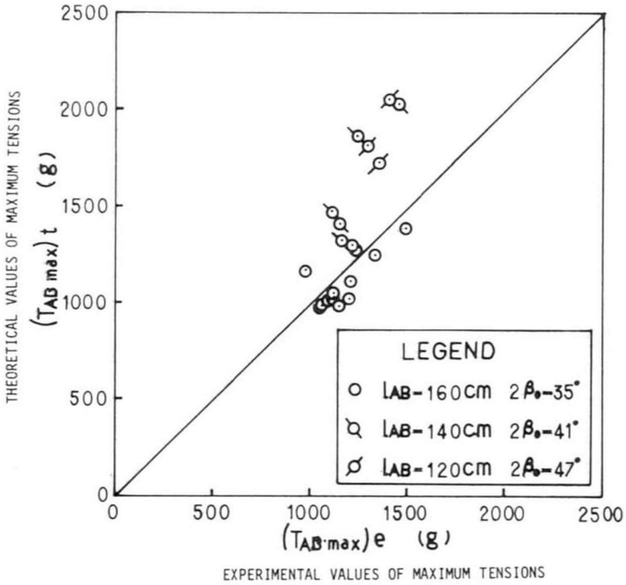


Fig.6. Comparison of the theoretical values of $(T_{AB})_{max}$ versus the experimental ones

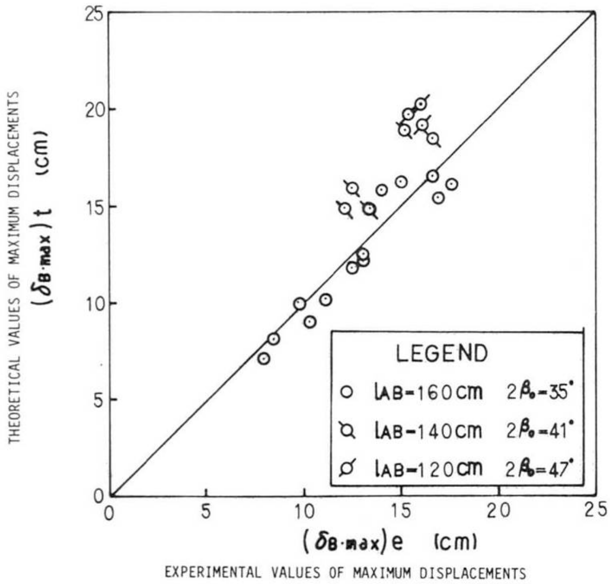


Fig.7. Comparison of the theoretical values of $(\delta_B)_{max}$ versus the experimental ones

Hense, it may be said that the present approximate theory should be applied to the case that $l_{AB}/L_{pp} \geqq 1.3$ and $\phi_0 \leq 25^0$ in which L_{pp} is the length between perpendiculars of ship and ψ_0 is the initial heading angle of ship relative to the guide-line, $\psi_0 = \beta_0 + \phi_0$.

4. TRIAL FOR PRACTICAL USE

A trial use of the present collision-prevention device was made for preventing ship collisions with a floating platform for geological survey of the seabed at the projected construction site of the piers of Honshu-Shikoku Bridges. Fig.8 shows the schematic diagram of the practical device designed as a trial use, and Fig.9 shows one of photographs of the device taken from the floating platform placed in the Akashi Straits which is between the Main Land and the Awaji Island of Japan.

The device was designed for preventing the collision of 1000 G.T.-ship with a velocity of 10 knots. Each floating guide-line is composed of fourteen pneumatic rubber fenders (produced by the Yokohama Rubber Co.,LTD of Japan) 2 m in diameter and 5 m long, tied in a low with a steel anchor chain 95 mm in diameter having a break load of 920 t_f (9016 kN). The same anchor chains were used for the mooring lines of the main buoy, which is 10.2 m in diameter and 4.8 m high, and the sub-buoys which are 8.5 m in diameter and 4.8 m high, respectively. Specially-moulded cast iron sinkers 300 t_f (2940 kN) in weight were used for the buoy anchors.

The initial tensions of the guide-lines can be arbitrarily adjusted by changing a buoy draft by means of pulling up the anchor chains into the buoys using an

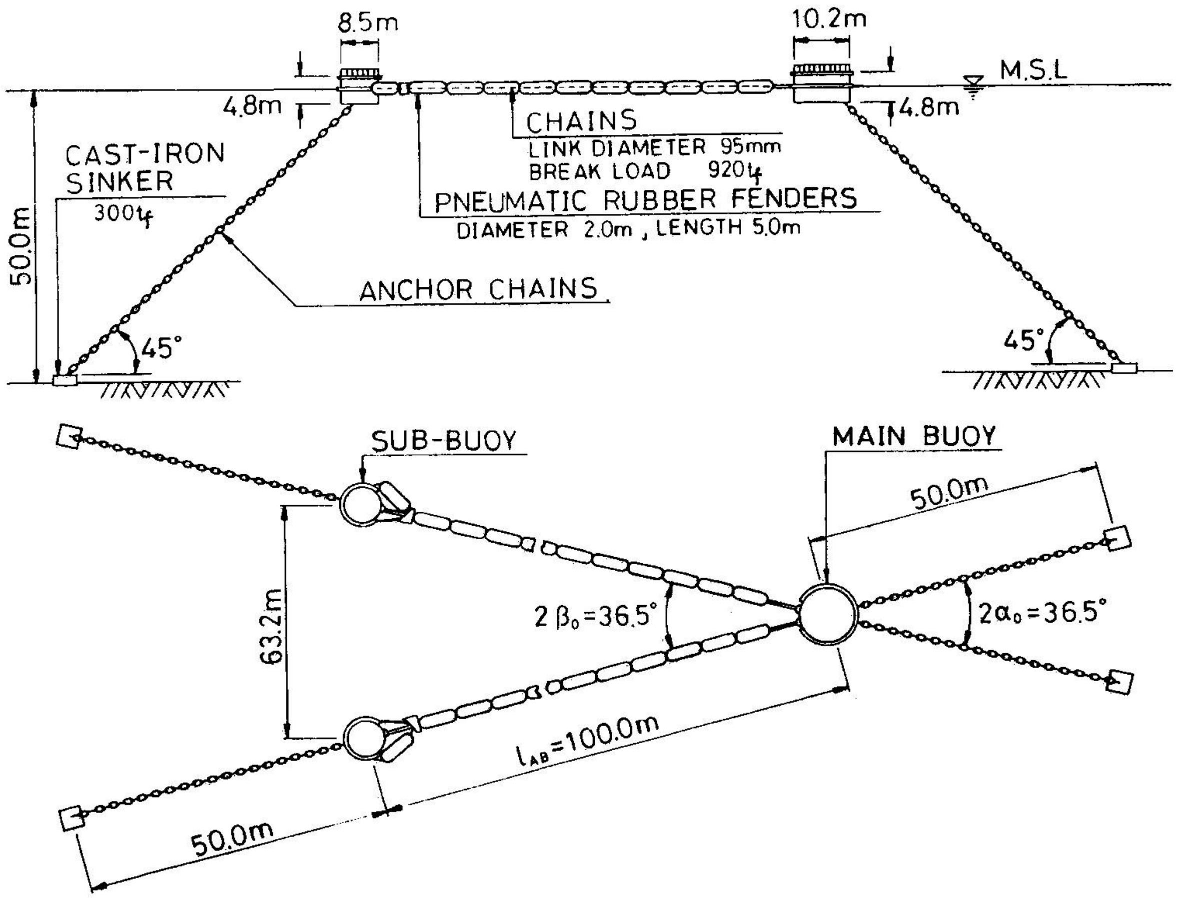


Fig.8. Schematic diagram of the trial collision-prevention device

oil pressure jack equipped on the respective buoy deck.

During the placing of the collision-prevention device in the Akashi Straits 1973 to 1975, it experienced several times the contacts of ships, which are relatively small in size ranging 200 G.T. in maximum, with itself, and showed to be very useful and effective for preventing ship collisions with bridge piers and/or offshore structures.

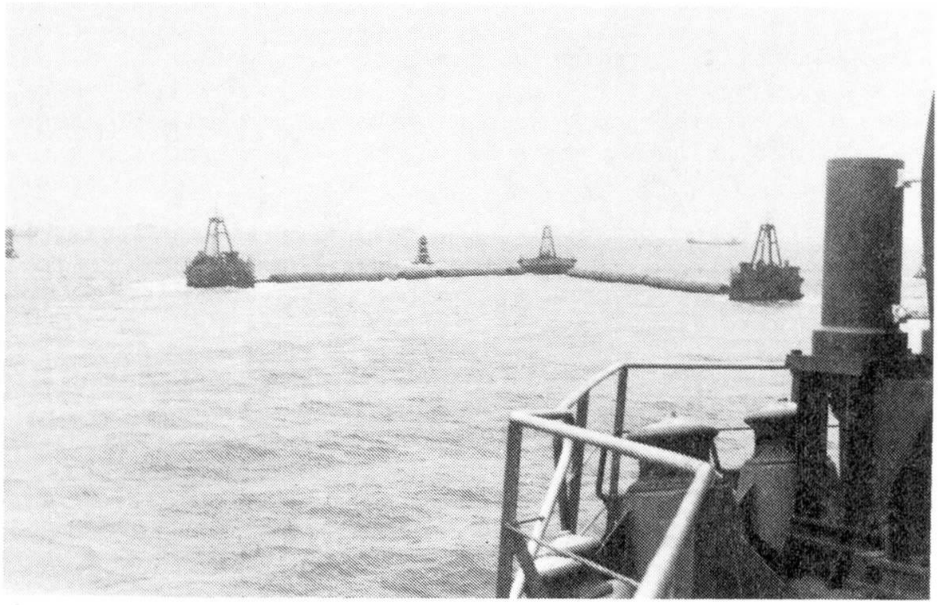


Fig.9. View of the trial collision-prevention device

5. ACKNOWLEDGEMENTS

Partial suport of the Honshu-Shikoku Bridge Authority and the Yokohama Rubber Co.,LTD is gratefully acknowledged.

REFFERENCES

1. ODA K.and NAGAI S., Protection of Maritime Structures against Ship Collisions Proc. of the 15th International Coastal Engineering Conference ASCE, pp.2810 -2829/1976.