Theme F

Design Assumptions and Influence on Design
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Ship Collision with the Tokyo Bay Crossing Bridge - Tunnel

Collision avec le pont-tunnel de la baie de Tokyo

Schiffsanstoß gegen den überquerenden Brücken-Tunnel in der Bucht von Tokio

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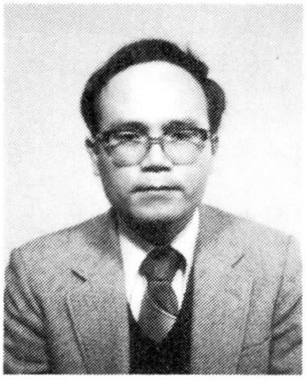
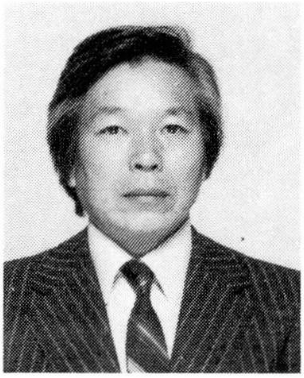


Yujiro Wasa, born 1944, got his master degree at Kobe University, Japan. He joined Nihon Doro Kodan in 1968. Since then he has been in charge of the construction and design of expressways in Japan. He is responsible for the research of the Tokyo Bay Crossing Bridge-Tunnel.



Masashi Oshitari, born 1936, got his civil engineering degree at Waseda University, Tokyo, Japan. He has been engaged in design of bridges and immersed tunnels for 21 years in a consulting engineering firm. He is responsible for the design of immersed tunnels at present.

SUMMARY
The Tokyo Bay Crossing Bridge-Tunnel, 15 km long is planned to cross the Tokyo Bay almost at the middle. A collision probability study was carried out looking into the combined effects of the actual behaviour of vessels entering into and navigating around the Bay, natural environment and the probability of sea accidents and storms. The conclusion obtained so far from studies of the existing situation revealed the need of a protection system for the bridge section of the Crossing against 200,000 devt. vessels in storms and 5,000 devt. vessels in ordinary weather .

RÉSUMÉ
Il est prévu que le Pont-Tunnel de la Baie de Tokyo, de 15 km de long, traverse la baie en son milieu. Une étude de probalitité de collision a été réalisée , en examinant les effets combinés du comportement des navires navigant dans la Baie, de l'environnement naturel, de la probabilité d'accidents en mer et de tempêtes. Les résultats déjà obtenus ont révélé le besoin d'un système de protection de la partie du pont traversant la baie pour les navires de 200.000 t de chargement en cas de tempête et de 5.000 t de chargement en temps normal.

ZUSAMMENFASSUNG
Der die Bucht von Tokio durchquerende Brücken-Tunnel mit einer Länge von 15 km wurde entworfen, um die Bucht von Tokio ungefähr in der Mitte zu durchqueren. Eine Studie über eine Kollisionswahrscheinlichkeit wurde ausgearbeitet, welche kombinierte Einwirkungen des gegenwärtigen Verhaltens von Schiffen, die in die Bucht fahren, natürliche Umgebung sowie Wahrscheinlichkeit eines Unglücks im Meer oder Sturm mit einbezog. Die Folgerung aus nun verfügbaren Daten ist, daß die Brücke mit einem Schutzsystem gegen 200.000-Tonnen Schiffe im Sturm und 5.000-Tonnen Schiffe in anderen Bedingungen versehen werden muß.

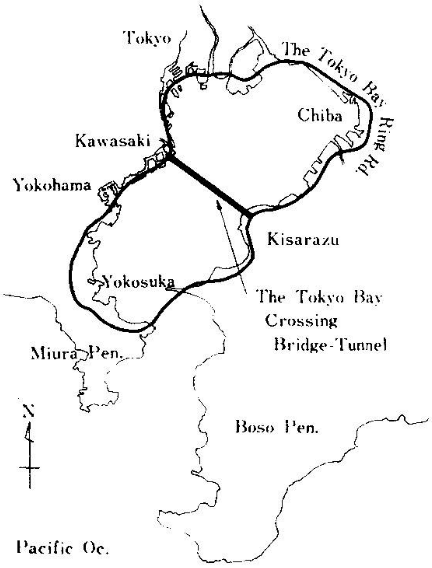
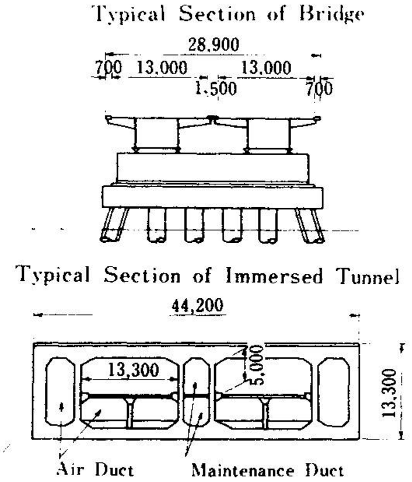


Fig.1 Site of the Crossing



1.THE TOKYO BAY CROSSING BRIDGE-TUNNEL

1.1 Profile of The Tokyo Bay

The Tokyo Bay located at about the middle of Japan facing the Pacific Ocean has a long oval shape of roughly 70Km by 20Km(see Fig.1). There are a lot of large port facilities for big cities such as Tokyo which is the center of politics, economics and industries of Japan, as well as Yokohama, Kawasaki, Chiba and others. These ports currently handle about 450,000 vessels of various sizes and 60,000 tons of cargo annually, forming the biggest industrial center in Japan.

1.2 Outline of The Crossing

A new ring road project around the Bay to connect this highly densed and developed living/producing area was planned, carried out and now more than half is opened to the public. The Tokyo Bay Crossing Bridge-Tunnel under planning is to cross the Bay at the middle, cutting short the ring road.
The ring road is 100m wide in most part and consists of a motorway, a national highway and a local trunk road. The Crossing on the otherhand consists of only a motorway. It will cross the narrowest part of the Bay by means of bridges, man-made islands and a tunnel almost straight for about 15Km between the shores.(See Fig.2)Passages will be secured for large vessels, medium ones and small ones separately. A long immersed tunnel will be constructed for securing the passsage of large vessels.

2. INVESTIGATION INTO THE ACTUAL CONDITION

2.1 Navigation of Vessels

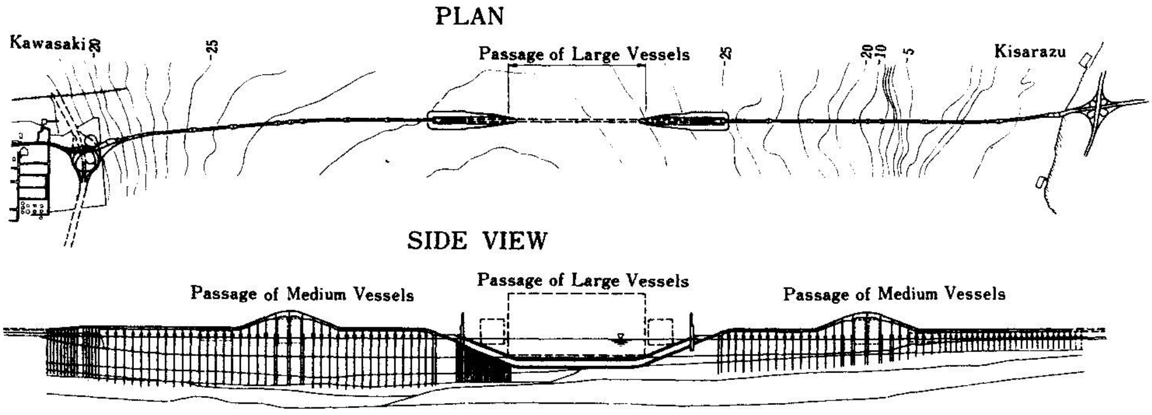


Fig.2 The Tokyo Bay Crossing Bridge-Tunnel

2.1.1 Number of Vessels

The annual number of vessels passing through the Uraga Channel at the mouth of the Bay was 300,000 in 1978 and the number of those entering the principal six ports in the Bay totalled 450,000. Out of these vessels, some 270,000 annually would be going around the proposed site of the Crossing.
The estimation for the year 1985 is 370,000 vessels at the Uraga Channel and 640,000 vessels moving around in the Bay.(See Fig.3)

2.1.2 Size and Type of Vessels

As to the distribution of the size of vessels estimated for the year 1985 shown in Fig.4, more than half is made up of ships of less than 500 gross tonnage. The main passage of the Crossing is planned for 200,000 deadweight size of vessels. The distribution of types shown in Fig.6 is mostly shared by cargo carriers.

2.1.3 Speed of Vessels

The actual average speed of vessels at the site of the Crossing, although the maximum speed recorded was 20 knots (37Km/hour), was 10 knots and less than 14

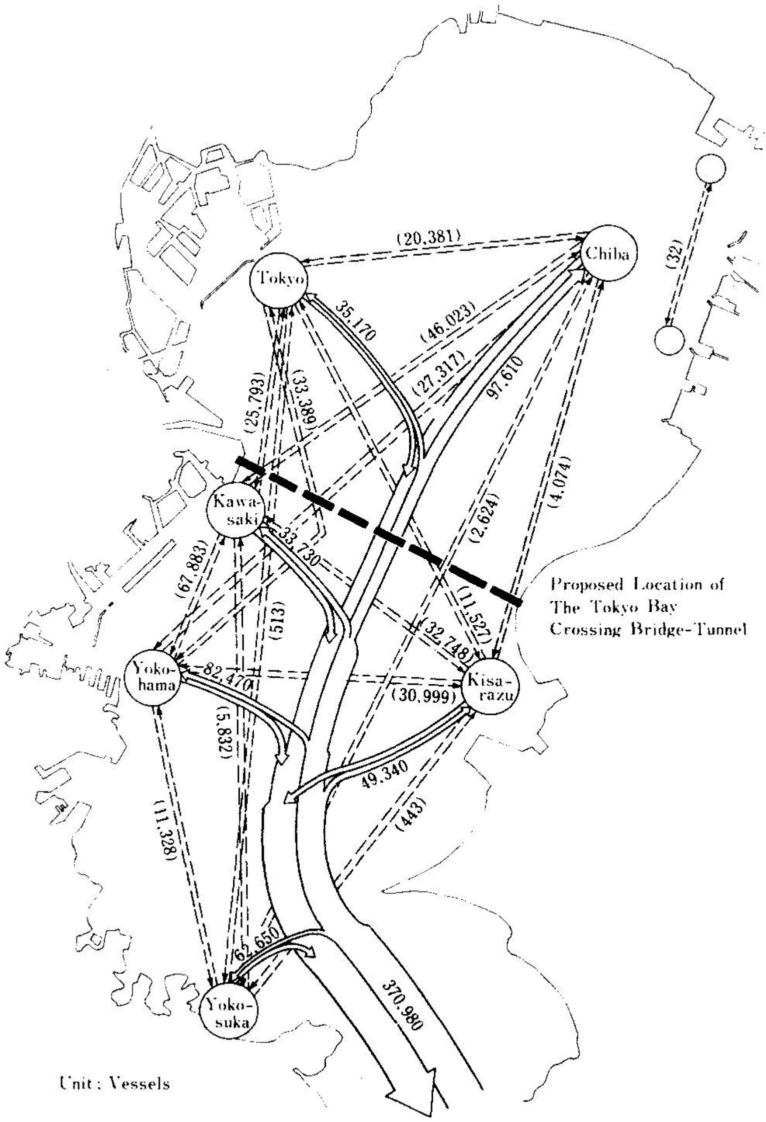


Fig.3 Number of Vessels in the Year 1985

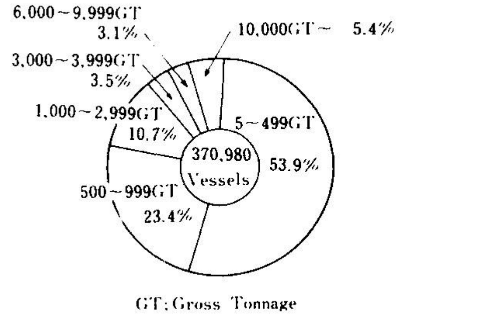


Fig.4 Size of Entering Vessels

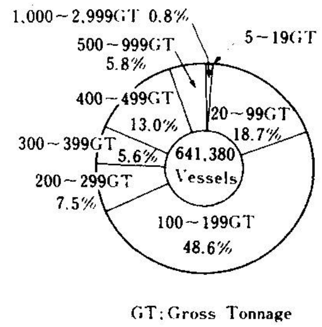


Fig.5 Size of In-Bay Vessels

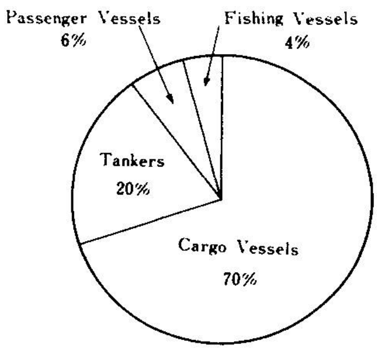


Fig.6 Type of Vessels

knots for more than 95% of the vessels.

The behaviour of navigation is controlled by several regulations. Actual observation revealed that most of the vessels navigated according to these rules and in order along the passage. However the exceptions were small fishing boats, traces of which were found to be criss-crossing all over the Bay.

2.2 Natural Environment

2.2.1 Wind

The wind records observed around the Bay revealed the wind blowing mostly toward North or South and rarely to the other direction (see Fig.7).

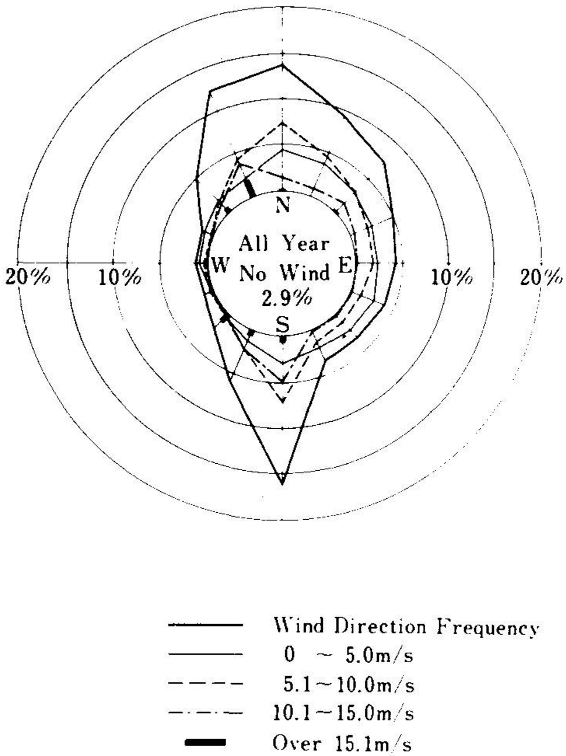


Fig.7 Wind Frequency

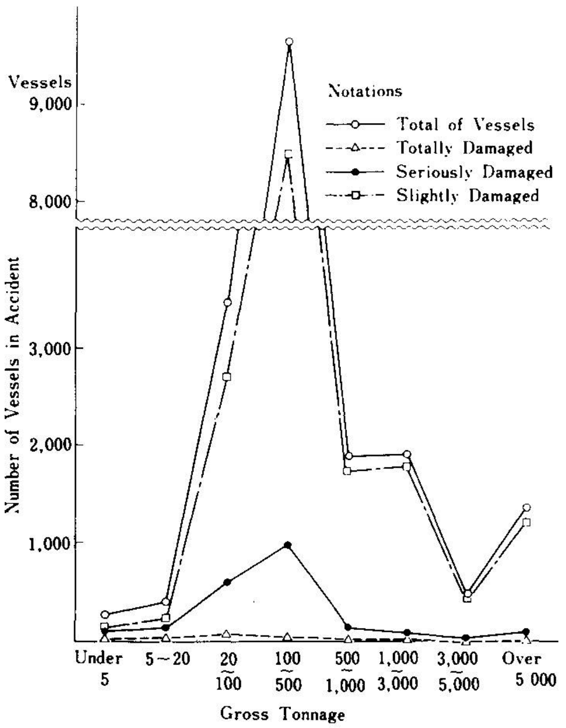


Fig.8 accident Rate

The wind velocity for designing of the Crossing was determined by analyzing those records to be basically 41m/s (as of 1979) which was the expected value for a period of 100 years.

2.2.2 Tidal Current

The direction and speed of current in the Bay keep changing every minute accordingly to the tide. However the maximum speed of current was relatively small, 0.8 knots, near the proposed site of the Crossing.

2.2.3 Typhoon

There were statistically about 28 typhoons on average spotted annually and four of them, roughly 14% of all, hit Japan. The probability of strong typhoons with a wind velocity of more than 35 m/s coming around and/or landing near the Bay was about once in ten years.

2.3 Probability of Sea Accident

The accident rate of all the vessels entering the Bay was statistically one in about 1,000 vessels in which one in about 3,800 vessels involved in collision and one in about 5,700 vessels involved in running-on. However most of the accidents occurred inside the port area, so that the previous figures of rate otherwise decrease to about 21,700 and 13,200 vessels respectively. More than half the accidents involved vessels of 100/500 gross tonnage and about 60% of all were fishing boats. The cause of accidents was in most cases related to the manner of navigation. However the damages sustained were usually minor and rarely serious or beyond recovery. (See Fig.8, 9). The collision rate of vessels entering into ports increases as the vessels got larger supposedly due to more difficulty in maneuverability.
The relation between the size of vessels and running on accidents was not clear.

3 PROBABILITY OF COLLISION

3.1 Probability in Case of Storms

The vessels striking the structures of the Crossing in storms were supposed to be the vessels anchoring in the Bay but swept-away by strong winds, the force of which exceeded the capacity of anchor (hereinafter called "Swept-away").
The probability of collision in storms was calculated by a study based on the

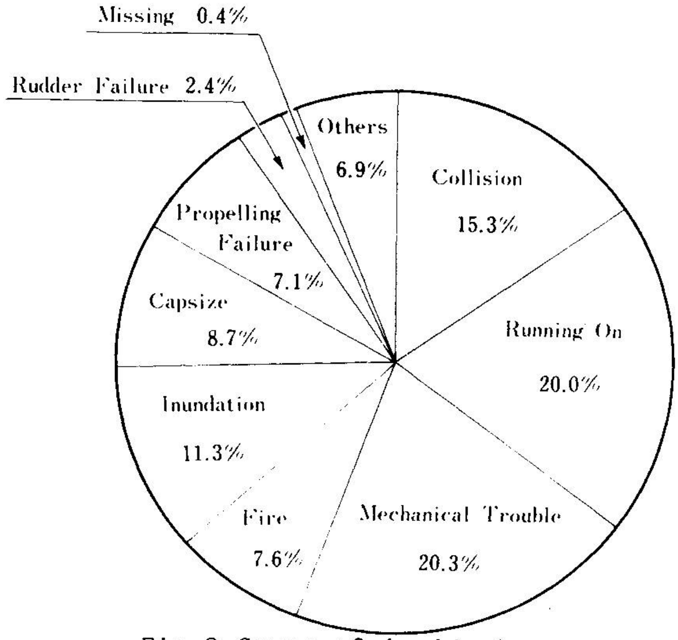


Fig.9 Cause of Accident

investigation of actual samples of vessels evacuating from ports and swept-away vessels, the assumption of conditions, demand and capacity for evacuation of vessels in the Bay, as well as the wind velocity at which vessels start being swept away, record of strong storms and the rate of swept-away of evacuating vessels.

3.1.1 Evacuation Record

The actual evacuation of vessels, mostly cargo carriers of 7,000/ 11,000 gross tonnage, during five typhoons hitting the Osaka Bay between 1965 and 1968 was as follows.
-Distance between vessels and shore-line ;
More than 80% of vessels kept a distance of about 3.7km from shore-line.
-Distance between each vessel;
The average distance between each vessel was about 1,300m but 10,000 gross tonnage class vessels stayed 1,600 ± 300m away from one another.
As an example of counter-measures for evacuation of vessels in the Tokyo Bay during a strong typhoon, many vessels evacuated to the outside of ports following the advice of authority.

Table-1 The Number of Vessels Demanding Evacuation in The Tokyo Bay

Gross Tonnage of Vessels	3,000 / 10,000	10,000 / 20,000	20,000 / 80,000	over 80,000	Total
Number of Vessels Demanding Evacuation	141	59	26	9	235

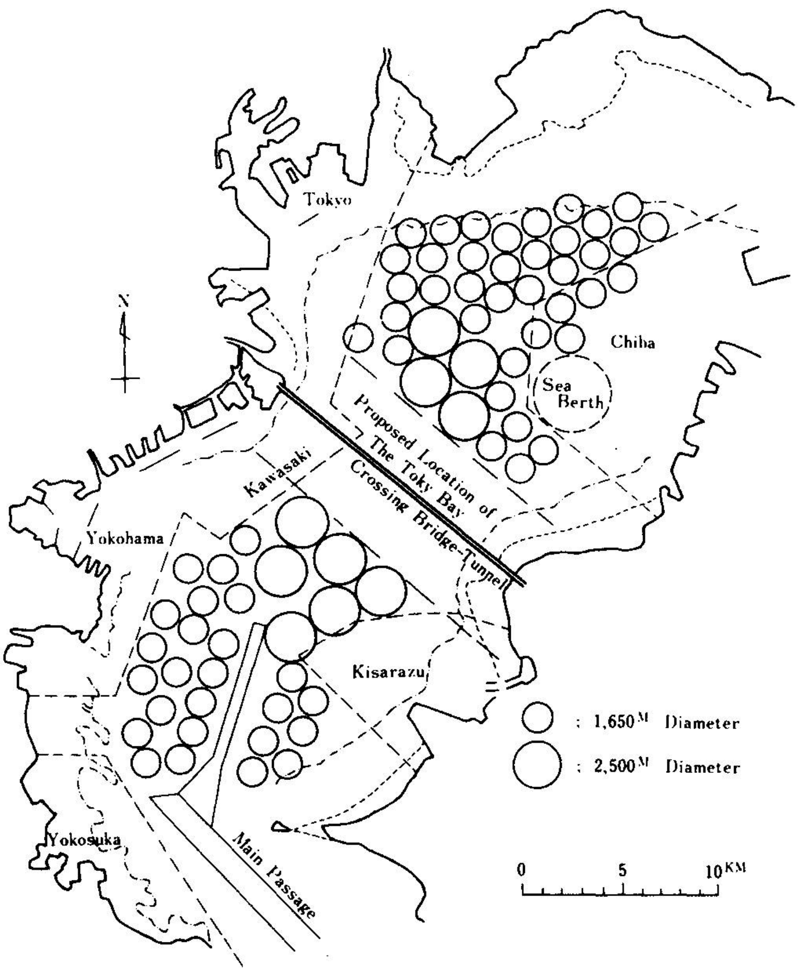


Fig.10 Evacuation Capacity.

3.1.2 Evacuation Demand and Capacity of the Tokyo Bay

The demand of vessels evacuating from ports in the Bay was assumed to be as shown in Table-1. On the other hand the capacity was assumed to be slightly more than 70 vessels taking into account conditions of evacuation, depth, nature of seabed soil and size of vessels, etc. and by the manner of drawing circles of required diameter in possible area for evacuation as shown in Fig.-10. The lack of capacity was clear.

3.1.3 Records of Swept-Aways

The investigation into the records of swept-away vessels revealed no relation with the size of vessels or the manner of anchoring. Therefore it was considered that the phenomenon of swept-away was largely affected by the technique of navigation and the state of countermeasures against severe weather according to the weather forecast and sea condition.
The speed of vessels being swept-away was statistically between 0.1 and 3.0m/s with a wind velocity of 30 to 35m/s and for a distance of up to 10km.

3.1.4 Wind Velocity Starting Swept-Away

Vessels tend to start being swept-away, as a result of a study, when the wind velocity of ten minutes on average reaches 25m/s for lightly loaded ones and 30m/s for heavily loaded ones.

3.1.5 Record of Strong Wind apart from Typhoon

A wind of more than 25m/s average velocity, supposedly 30m/s on the sea, never occurred except in case of typhoons according to the Weather Bureau of the Tokyo Area.

3.1.6 Rate of Swept-Away of Evacuating Vessels

It was clear that the vessels evacuating from ports and anchoring in the Bay during a storm would not start to be swept away all at once when the average wind velocity reached the previously mentioned value for starting.
Therefore a study was carried out to confirm the relations between the maximum wind velocity and the rate of swept-away, ratio to the number of evacuating vessels, by looking into the records of typhoons and the inquiries covering the vessels entering the Bay. The following results were obtained.

- Maximum Wind Velocity : 25m/s Rate of Swept-Away : About 1%
- Maximum Wind Velocity : 35m/s Rate of Swept-Away : About 35%
- Maximum Wind Velocity : 42m/s or over Rate of Swept-Away : About 100%

As a conclusion, the swept-away vessels will be about one percent with a maximum wind velocity of roughly 25m/s and all with over 42m/s.

3.1.7 Probability of Collision

- Occurrence Probability of Strong Wind (P_1)

The study revealed the interval of occurrence of wind velocity to start swept-away as 0.73 years to 25m/s for lightly loaded vessels and 6.4 years to 32m/s for heavily loaded ones, by taking into account the statistical interval of strong wind occurrence around the Bay.
The occurrence probability of wind velocity to start swept-away was consequently as follows.

Lightly Loaded Condition (25m/s) $p_{11}=10^{-3.81}$ times/hour
Heavily Loaded Condition (32m/s) $p_{1f}=10^{-4.75}$ times/hour

- Probability of Swept-Away (P_2)

By supposing the swept-away would occur according to the said rate among evacuating vessels, the probability of swept-away, defined as a probability of more than one vessel being swept-away among evacuating ones, was as follows using Poisson Distribution.

Lightly Loaded Condition (Rate of Swept-Away 4%) $p_{21}=10^{-0.06}$
Heavily Loaded Condition (Rate of Swept-Away 21%) $p_{2f}=10^{0.00}$

- Probability of Vessels Approaching The Bridges (P_3)

The product of probability P_1 and P_2 is the probability of more than one vessel being swept-away in the Bay. But in this instance, the probability of swept-away vessels drifting nearer to the bridges was determined as the ratio of the vulnerable part of bridges against collision to the whole length of the Crossing in the susceptible area as follows by assuming the swept-away would occur at random regardless of where the vessels anchored.

$$P_3 = \frac{\text{Length of Bridges}}{\text{Length in Susceptible Area}} = \frac{6.8\text{Km}}{11\ \text{Km}} = 10^{-0.21}$$

- Probability of Vessels Passing Between The Piers (P_4)

The probability of vessels passing between the piers of bridges was determined to be P_4 = 0 since most of those navigating around the site of the Crossing were longer than the pitch of the piers.

- Probability of Collision in Storms (P)

The probability of collision in storms was the product of probabilities of each factor, as follows.

- Lightly Loaded $P = P_{11}*P_{21}*P_3*(1-P_4) = 10^{-4.08}$ times/hour
- Heavily Loaded $P = P_{1f}*P_{2f}*P_3*(1-P_4) = 10^{-4.96}$ times/hour

Those values showed that there would be one collision every about 1.37 years or so in case the vessels were lightly loaded and every about 10.4 years or so in case the vessels were heavily loaded. But in this study the term "lightly loaded" was defined as empty, a situation which would hardly occur in a storm since the draught of every vessel would be lowered as a precaution to increase steadiness by means of more ballast. Therefore it would be more reasonable to assume the condition of loading to be in between half and heavy which meant the collision rate in storms would be once in about five to ten years.
As a conclusion of this study, the probability of collision, even if all vessels in the Bay were heavily loaded, was once in about ten years which was rather high. Further, it was clear from the investigation into actual conditions that the size of vessels had no relation with their swept-away. Therefore 200,000 deadweight vessels which were the largest ones entering the Bay, had to be considered as object of collision in storms.

3.2 Probability of Collision in Ordinary Time

The collision in ordinary time was statistically caused by navigational errors. Therefore the probability of errors was studied.

Table-2 Summary of Probability of Collision in Ordinary Time

Gross tonnage of Vessels: Passage	Damage	5 ~ 100	100 ~ 500	500 ~ 1,000	1,000 ~ 3,000	3,000 ~ 10,000	10,000 ~ 20,000	20,000 ~ 100,000	over ~ 100,000	Total
For Small Vessels	Totally	$10^{-6.94}$ (994)	—	—	—	—	—	—	—	$10^{-6.94}$ (994)
	Seriously	$10^{-5.33}$ (24)	—	—	—	—	—	—	—	$10^{-5.33}$ (24)
	Minor	$10^{-4.54}$ (4.0)	—	—	—	—	—	—	—	$10^{-4.54}$ (4.0)
	Total	$10^{-4.47}$ (3.4)	—	—	—	—	—	—	—	$10^{-4.47}$ (3.4)
For Medium Vessels	Totally	—	$10^{-7.29}$ (2230)	—	—	—	—	—	—	$10^{-7.29}$ (2230)
	Seriously	—	$10^{-5.18}$ (17)	$10^{-6.23}$ (193)	—	—	—	—	—	$10^{-5.14}$ (16)
	Minor	—	$10^{-3.91}$ (0.9)	$10^{-4.73}$ (6.1)	—	—	—	—	—	$10^{-3.85}$ (0.8)
	Total	—	$10^{-3.88}$ (0.9)	$10^{-4.71}$ (5.9)	—	—	—	—	—	$10^{-3.82}$ (0.8)
Sub Passage		—	$10^{-4.21}$ (1.9)	$10^{-4.37}$ (2.7)	—	—	—	—	—	$10^{-3.98}$ (1.1)
For Large Vessels		—	—	—	$10^{-5.05}$ (13)	$10^{-5.41}$ (29)	$10^{-5.70}$ (57)	$10^{-5.90}$ (91)	$10^{-6.90}$ (907)	$10^{-4.79}$ (7.1)

Note ; The probability of collision to be Times/Hour.
Figures in bracket to be the interval year of occurrence.

3.2.1 Probability of Collision in Main Passage of Large Vessels

The probability of collision with man-made islands located at both sides of the tunnel was studied by the following four methods (see Table-2).
- The rate of vessels running on the small islands at the mouth of the Bay to all the passing vessels.
- The study of general statistics on both sea accidents and harburs.
- The investigation of draught and traces of vessels by normal distribution.
- The investigation of draught and traces of vessels by Rayleigh distribution.

3.2.2 Probability of Collision in Sub Passage

The probability as shown in Table-2 was determined as the rate of vessels navigating in water shallower than their draught running on the man-made islands by assuming the traces of passing vessels being normal distribution.

3.2.3 Probability of Collision in Passage of Medium and Small Vessels

Assuming the probability of navigational errors to be 10^{-4}, the product of this value and the probability of collision without correcting the direction were determined as the probability of collision in the passage of medium and small vessels as shown in Table-2.

4 CONCLUSION

The study revealed the possibilities of vessels striking the bridges of Crossing in both entirely different situations which were the swept-away vessels in storms and navigational errors in ordinary time. The conditions of collision in these situations can be concluded as follows.

4.1 In Storms

Since the object of collision was the swept-away vessels, the absolute speed of striking vessels would be the added value of the swept-away vessel's speed against water and the speed of current, and could be considered to be 2.0m/s for heavily loaded vessels and 4.1m/s for lightly loaded ones.
The 200,000 deadweight vessels, which were the largest ones entering the Bay, should be considered as the object.

4.2 In Ordinary Time

Most of the causes for collisions in ordinary time were navigational errors. The correcting efforts of navigation were taken statistically when the approaching vessels were at the latest some distance twice their length before striking. Therefore the striking speed would not be the normal navigating speed but the one reduced after some operation to avoid the collision and to lower the speed by means of stopping the engines etc., and assumed to be about 12 knots (6.2m/s). The object of collision could be determined as the vessels of less than 100,000 gross tonnage which had a collision probability of less than $10^{-7.0}$ (negligible in engineering terms) for the main passage of large vessels, and 1,000 G.T. for the passages of medium vessels which were bridge sections.

4.3 Further Study

The protector would be large if it were designed fully according to the results of this study. Therefore further study including some additional specific investigations and introduction of stricter regulations on navigation and evacuation, would be needed to establish more appropriate and adequate countermeasure against ship collision and protection systems for the Crossing.

Ship Collision and the Farø Bridges

Collisions de navire et ponts de Farø

Schiffsanprall und Farö-Brücken

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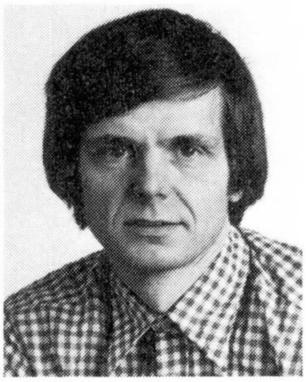
A. Ole Jensen, born 1926, graduated 1950 from the Technical University of Denmark as a civil engineer. After military service and employment in the Laboratory for Harbour Construction and Foundation he joined C & N in 1952, and has since then been occupied mainly with design of bridges, harbours and tunnels.



Egon A. Sørensen, born 1946, graduated from the Technical University of Denmark in 1970 as a civil engineer. After military service he joined C & N in 1971 and has since then worked with studies and design of bridges, design and installation of steel structures for North Sea platforms and maintenance of tunnels.

SUMMARY

The paper describes the navigational situation at the Farø Bridges, at present under construction, and further describes how the design assumptions for the ship collision loading case have been derived from recordings of the present ship traffic in the area by means of theoretical analyses and prognoses. The calculated effect of the ship collision load on the elements of the substructure is mentioned and compared with the results of independent calculations based on different principles.

RÉSUMÉ

L'article décrit la situation de la navigation sous les ponts de Farø actuellement en construction. La probabilité de collisions de navire a été estimée sur la base de la densité du trafic maritime actuel dans la région, au moyen d'analyses théoriques et de pronostics. L'effet calculé de la force de collisions de navire avec les éléments de l'infrastructure est mentionné et comparé avec les résultats de calculs indépendants basés sur différents principes.

ZUSAMMENFASSUNG

Der Artikel beschreibt die Navigationslage an den Farö-Brücken, die sich im Bau befinden. Die Lastannahmen eines Schiffsanpralls wurden aus Aufzeichnungen über den gegenwärtigen Schiffsverkehr im Gebiet sowie theoretische Analysen und Prognosen getroffen. Die berechnete Einwirkung des Schiffsanpralls auf die Elemente des Unterbaues ist erwähnt und mit den Ergebnissen von unabhängigen Berechnungen verglichen.

1. INTRODUCTION

1.1 General Briefing

The Farø bridges are two motorway bridges, one leading from Sjælland to the small island of Farø and the other leading from Farø to Falster. They will form part of the motorway connection from Copenhagen to Rødby and Germany, see Fig. 1. Since 1965 Christiani & Nielsen A/S has been acting as consultants to the Danish Road Directorate for these bridges. A great number of studies, sketch proposals and preliminary designs have been worked out, concluding in a tender project presented in April 1979. Since the construction contracts were signed in May 1980, Christiani & Nielsen A/S has worked out the detailed design for all the bridge piers, pylons and abutments including the foundations, and is also performing the supervision of the site construction. The bridges are scheduled for completion in 1985.

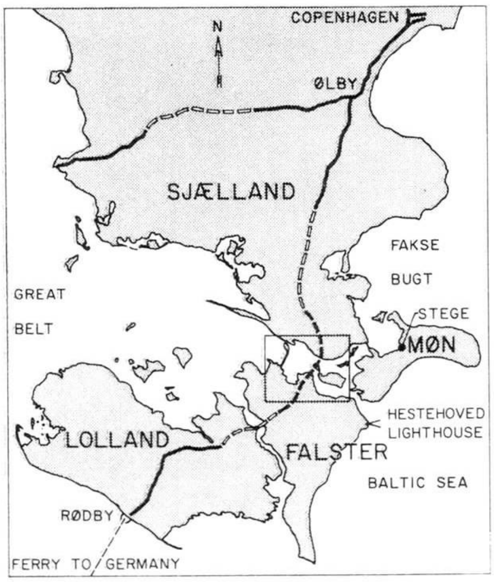


Fig. 1 Location map

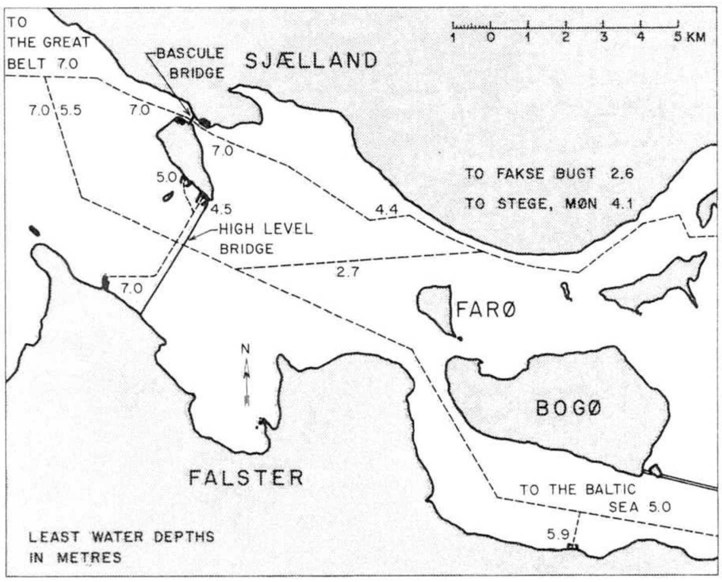


Fig. 2 Present navigation pattern

1.2 Physical Conditions for Shipping

The natural water depths in the area vary considerably, from extended shoals of about 3 m water depth to deep channels of 10 to 38 m water depth. The present navigation pattern in the area, which could possibly influence or be influenced by the linkage structures, is shown in Fig. 2, where the main fairways are indicated with dotted lines. The water depths restricting the shipping in the fairways are also given. Fig. 2 shows, moreover, the existing rail- and road connection between Sjælland and Falster, consisting of a bascule bridge with a 25 m wide navigation opening, and a high level bridge with a 26 m high and 111 m wide main navigation opening. The northern fairway towards east is passing another high level bridge with a 26 m high and 80 m wide navigation opening.

1.3 Navigational Aspects of Selected Linkage

The finally selected linkage lay-out is shown in plan and elevation on Fig. 3 and 4 respectively. From Fig. 3 it is seen, that the fairways can be straigthlined for adequate lengths before and after passing the bridges and intersect the longitudinal bridge axes close to right angles. In the bridge between Sjælland and Farø with a general span length of 80 m, two 20 m high navigation openings for one-way traffic are arranged in the two spans next to pier No. 6,

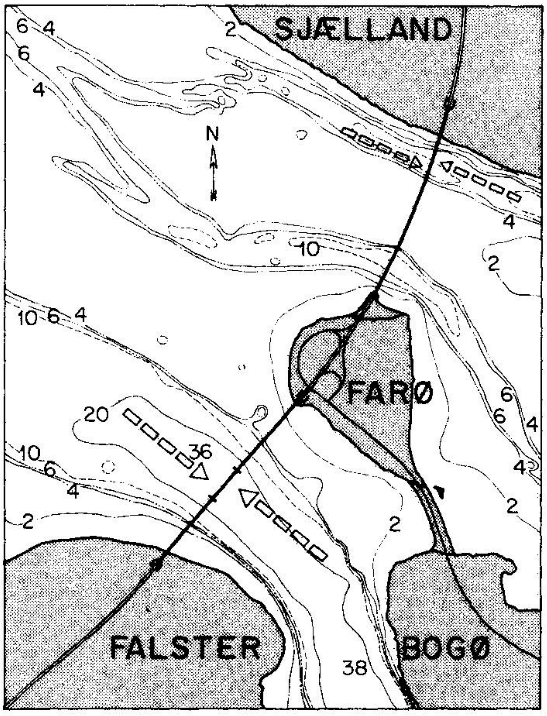


Fig. 3 The Farø Bridges, plan

whereas in the bridge between Farø and Falster an integral cable-stayed bridge part provides a main span of 290 m length between piers Nos. 9 and 10 with a 26 m high navigation opening. The deep channel just north of Farø has a blind ending towards east and is, therefore, not used for navigation.

The ordinary R/C bridge piers all have pier shafts with a uniform outer shape, hexagonal in cross section, in the full height from the foundation block to the bridge bearings, see Fig. 5. The pier shafts are solid below level +2.00 m and above that level, hollow with a wall thickness of 0.40 m to 1.00 m, depending on the loads and height of the pier. Fig. 5 further shows one of the two R/C pylon piers for the cable-stayed bridge. The pylons are also composed of a lower, solid and an upper hollow part. The R/C foundation blocks are all placed either entirely below sea bed or below the possible draught of ships.

The design philosophy adopted in respect of ship collision was on the one hand that the piers shall be the strong and unresilient part of a collision, and, on the other hand that ships, whether large or small, shall not meet any unexpected, submerged structures.

Other design philosophies were contemplated, for instance to diminish the effects of ship collisions by providing the piers with resilient fenders or to surround the piers by embankments or "islands". However, these ideas were abandoned due to high costs and adverse hydraulic effects.

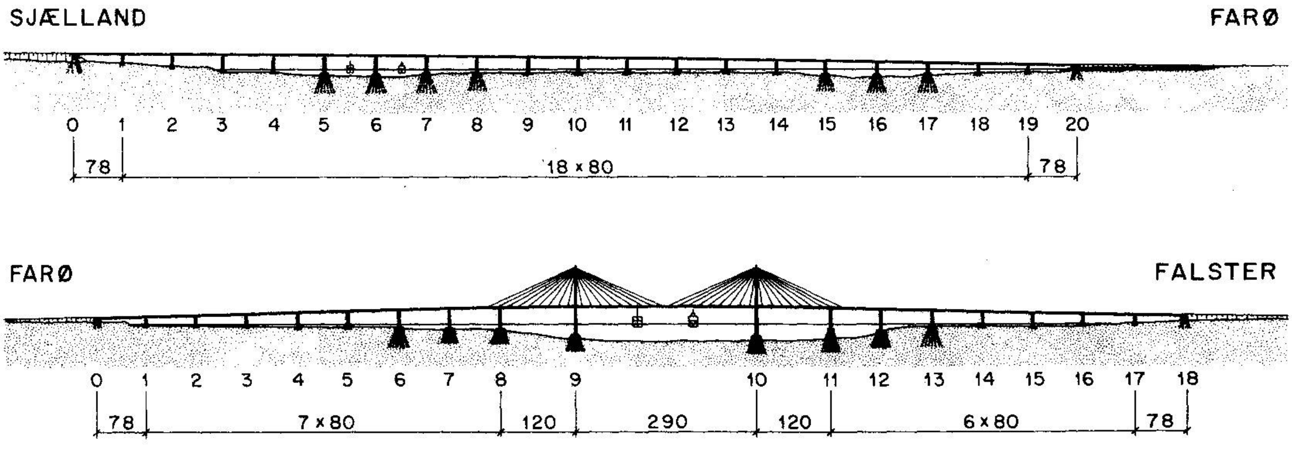


Fig. 4 The Farø Bridges, elevation

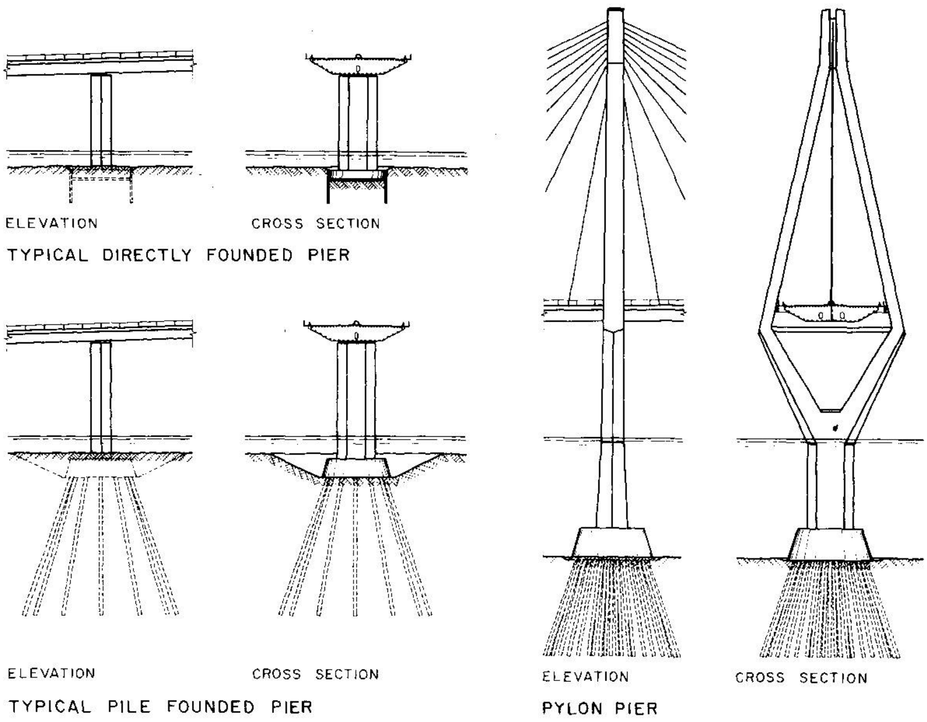


Fig. 5 Bridge piers

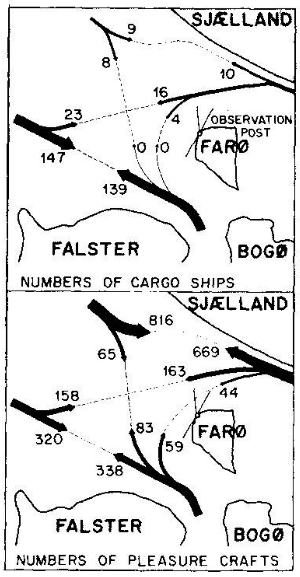


Fig. 6 Ships recorded from Farø 12.July-2.Aug.72

2. DESIGN SPECIFICATIONS FOR SHIP COLLISION

2.1 Recording of Shipping

A basic impression of the shipping was obtained in 1966 by questioning the harbour authorities of seven harbours in the area on the annual traffic in 1965 for different ship categories.

When the alignment at Farø was finally selected a more comprehensive recording was arranged in 1971-72 as follows:

- From Hestehoved lighthouse at the easternmost point of Falster all passing ships, cargo ships as well as pleasure crafts, were recorded with estimation of size. This should cover the major part of the ships passing south of Farø. Duration 18 months.
- In the harbour of Stege on Møn all berthing cargo ships were recorded and their sizes were noted. This should cover the major part of the bigger ships passing north of Farø. Duration 15 months.
- From a point on Farø, see Fig. 6, all ships were recorded by means of a specially developed instrumentation allowing determination of position, direction,speed, length and mast height. Duration 3 weeks in July-August.

All the recordings were statistically treated. Fig. 6 shows the distribution of ships recorded from Farø. It appears that the traffic with pleasure crafts is quite considerable.

For determination of the dead weight of the cargo ships recorded from Farø the following formula was developed on the basis of the compiled information:

$$DWT = 55 + \left(\frac{L}{6.2}\right)^3 \text{ tons, where } L = \text{ship's length in m}$$

The biggest ship recorded was determined at 2200 dwt, going at 5.5 m/sec.

The inherent collision force for the cargo ships recorded from Farø was calculated from the recorded data by the formula given in [1] :

$$P = \frac{v^{2/3} \times L^2}{1100} \text{ MN}$$

where v = ship's velocity in m/sec

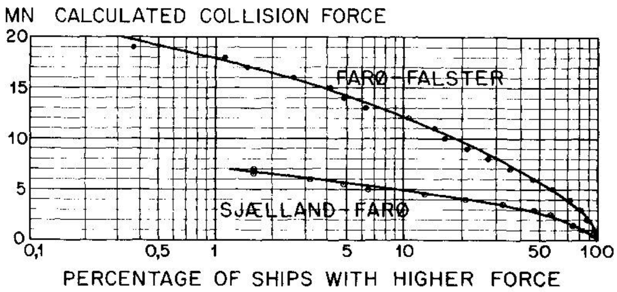


Fig. 7 Inherent collision force of cargo ships recorded from Farø

2.2 Determination of Characteristic Collision Load

Based on experience from the English Channel Macduff [2] proposes a "causation probability" of 0.0002 for ships in an area with platforms in the North Sea, meaning that there is one chance in 5,000 that a ship will be out of normal control due to such causes as poor visibility, rudder or engine failure or faulty navigation. Assuming the same causation probability for the Farø Bridges and considering that, due to the geometrical relationship between pier dimensions, span lengths and possible approach angles, the chance that an uncontrolled ship in the area will collide with a pier is about one in 7, the real probability of a ship collision on a Farø Bridge pier is one in 35,000.

Based on all the ship recordings and assuming "status quo" conditions it can be expected that, during 100 years, approx. 65,000 cargo ships will pass a bridge from Sjælland to Farø, and approx. 500,000 cargo ships will pass a bridge from Farø to Falster. Consequently, during 100 years 65,000/35,000 ~ 2 ships will collide with a bridge pier between Sjælland and Farø, whereas 500,000/35,000 ~ 14 ships will collide with a bridge pier between Farø and Falster. However, due to the relatively low standard of navigational aids at the fairway between Sjælland and Farø, and in view of the uncertain assumptions, the number for this bridge is arbitrarily increased from 2 to 10.

Defining the maximum load P_m as the force to be exceeded once per 100 years the following was found from Fig. 7, thus still assuming "status quo" conditions:

- Sjælland-Farø: One of the 10 colliding ships, i.e. 10% of them, will exert a collision force higher than 5 MN, hence P_m = 5 MN.
- Farø-Falster: One of the 14 colliding ships, i.e. 7% of them, will exert a collision force higher than 14 MN, hence P_m = 14 MN.

The effect of future development of the shipping was then evaluated by a sensitivity analysis and a prognosis for the navigation. Based on this investigation it was decided to assume for the next 100 years an unchanged number of ships, a 10% increase of speed and a 50% increase of dead weight.

Based on the statistics and the formulas mentioned in 2.1 was then found a characteristic collision load of 7 MN for Sjælland-Farø and 20 MN for Farø-Falster.

The characteristic load for Farø-Falster corresponds to collision by a "characteristic ship" of 2250 dwt with a speed of 6.25 m/sec, which data, incidentally, are very close to those for the biggest ship recorded from Farø.

2.3 Load Specifications

The ship collision load specifications finally adopted for the individual piers were based on the above theoretical considerations, but regard was, of course, also made to the fact that the actual water depth at some piers restricts the size of ships to hit them. Furthermore, for bridge piers more than 240 m away from any of the navigation openings in the Sjælland-Farø bridge, it was decided to neglect ship collision loads, as it was found that the very unlikely event of a ship collision on one of these piers would cause only local damage to the pier shaft.

Thus, the following ship collision loads P_1, acting in a direction perpendicular to the longitudinal axis of the bridge, are specified:

- Piers Nos. 4 to 9 in the Sjælland-Farø bridge and piers Nos. 4 and 5 in the Farø-Falster bridge $P_1 = 7$ MN
- Pier No. 6 in the Farø-Falster bridge, 4 m water depth $P_1 = 14$ MN
- Piers Nos. 7 to 12 in the Farø-Falster bridge $P_1 = 20$ MN
- All the remaining bridge piers $P_1 = 0$

Alternatively is specified a collision load P_2 acting parallel to the longitudinal axis of the bridge, where $P_2 = 0.5\ P_1$.

Based on compiled information regarding dimensions and ultimate strength of ships' hull and superstructure, it is specified that the loads P_1 and P_2 shall be assumed to act as uniformly distributed loads p as follows:

For P_1 = 14-20 MN and P_2 = 7-10 MN the width of the loaded area shall not exceed <u>10</u> m, and the load p shall be

- From <u>10</u> m to <u>15</u> m above sea level p = 50 kN/m^2
- From <u>5</u> m to <u>10</u> m above sea level p = 500 kN/m^2
- From <u>5</u> m below to <u>5</u> m above sea level p = 1,000 kN/m^2

For P_1 = 7 MN and P_2 = 3.5 MN the underlined dimensions are multiplied by 0.6, whereas the loads p are unchanged.

The loads shall be arranged so as to produce maximum stresses in the members investigated. However, for the calculations of the pier foundations the resultant loads P_1 and P_2 are assumed to act at sea level and as shown on Fig. 8 in plan.

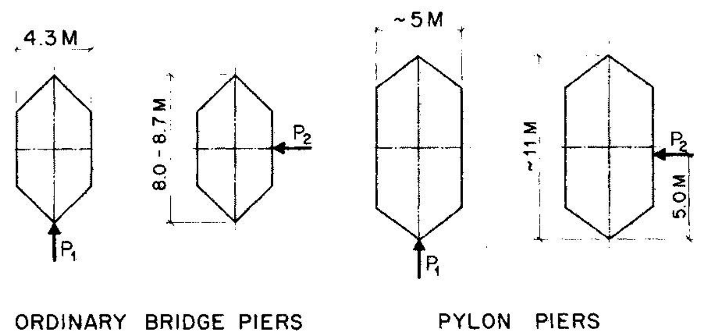


Fig. 8 Location of resultant loads for design of foundations

Referring to 2.2 all the above ship collision loads are characteristic loads. They shall be combined with dead weight of bridge structures only, and the combinations are considered extreme and random assuming a partial coefficient of safety of 1.0 to the loads.

The sea level shall be assumed to vary +/- 0.5 m from mean sea level, which covers about 99% of the time.

3. DETAILED DESIGN OF SUBSTRUCTURE

3.1 General

The following design assumptions were agreed upon with the Road Directorate:

- Ship impact to be considered as static load.
- Structural behaviour in accordance with the theory of elasticity.
- Pier shafts and pylons to be designed with as well as without lateral stiffening effect of the superstructure.
- Foundations to be designed only without lateral stiffening effect of superstructure.

3.2 Pier Shafts and Pylons

The ship collision loads have determined the thickness and horizontal reinforcement of the walls of the upper hollow part. They have also determined the vertical reinforcement of the ordinary piers designed for 14 and 20 MN ship collision load, whereas the vertical (longitudinal) reinforcement of the pylons is mainly determined by the construction phase.

3.3 Pier Foundations

The project consists of both directly founded piers and piers founded on piles. Direct foundation is used only at water depths less than 4 m and the relatively modest ship collision load (viz. zero or 7 MN) being specified here, has not been dimensioning for the foundation. The same applies to the pilefounded piers of the Sjælland-Farø bridge, whereas the ship collision load has been dimensioning for the Farø-Falster bridge piers Nos. 6-12, all pilefounded. A typical, piled foundation is shown in fig. 9.

The chosen form of the pile groups with the piles radially placed in respect to the pier centre means that part of the load on the pier will be taken up as shear and bending moment in the piles. Therefore, knowledge of the axial as well as the lateral bearing capacity in the soil is necessary. Assumptions for the soil-pile interaction in the form of load-deflection curves for both lateral and axial resistance have been established in co-operation with the Danish Geotechnical Institute, whereupon C&N has made the calculations by means of EDP-programmes capable of taking into consideration the variation of the soil properties with the depth and the non-linear course of the load-deflection curves.

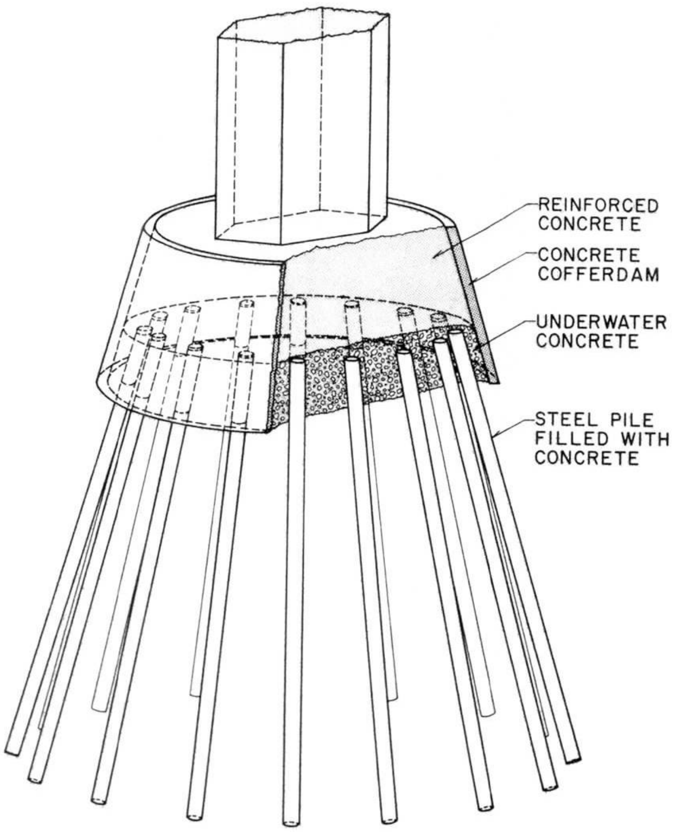


Fig. 9 Piled foundation of bridge pier

For all load combinations, including the one with ship collision load, the criterion for acceptance of the piled foundation has been that neither axial force, shear force nor bending moment in the most heavily loaded pile must exceed the design value of the bearing capacity of the soil or the design strength of the pile. Regarding the bending moment in the piles the exception has been made that plastic deformations have been accepted in some cases, as long as the subsequent loadings could be taken up without exceeding the design strength of the pile.

4. SUPPLEMENTARY INVESTIGATIONS

4.1 General

To check the validity of the results from the detailed design, the Road Directorate had supplementary investigations of ship collision on some selected piers carried out, as described in 4.2 and 4.3.

4.2 Investigations According to the Theory of Plasticity

The investigation of some selected piers according to the theory of plasticity has been made by the Danish Geotechnical Institute by means of an EDP-programme especially developed for this purpose. The collision load and the partial safety factors used were the same as those applied for the theory of elasticity. When comparing the results it was found that, because of the less strict rupture criterion, the pile foundations could withstand 20% to 50% higher ship collision loads when calculated in accordance with the theory of plasticity than when calculated in accordance with the theory of elasticity.

4.3 Ship Collision as a Dynamic Load

These investigations have been made by B. Højlund Rasmussen and we refer to B. Højlund Rasmussen's paper on this issue.

In the main series of these calculations the stiffening effect of the superstructure was taken into account, and it was found that the pile forces were somewhat lower than those calculated for the detailed design in accordance with 3.1, whereas bending moments considerably higher were found in pylons and pier shafts. Nevertheless, the sections in question, determined by other load combinations and structural criteria, proved to be sufficiently strong to withstand also these moments.

A calculation not considering the stiffening effect of the superstructure was also made for pier 11 in the Farø-Falster bridge to try to get a more comprehensive comparison between static and dynamic applications of ship collision load. This calculation showed that the dynamic collision load will give slightly higher pile forces than the static collision load, but considering the other supplementary investigations this was accepted.

5. FINAL REMARKS

In designing the Farø bridges respect was paid to the shipping, in the planning phase as well as in determining the shape and strength of the structures. Thus, strengthening had to be made of pier shafts and pylons, which were assumed to be exposed to ship collision load and of the foundations for pylons as well as for piers, which could be hit by ships with a draught of more than about 4 m.

Control calculations, based on different principles, revealed that a certain extra safety might exist in some of the structures.

This had to be utilized for one of the so-called anchor piers for the cable-stayed bridge, viz. pier No. 11, where especially poor soil conditions were found. The resulting low tension resistance of the piles would make it very expensive to obtain the full prescribed safety using the design assumptions in accordance with 3.1, so a somewhat lower factor of safety was accepted in this case.

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[2] T. Macduff: The probability of vessel collision. Ocean Industri - Sept. 1974 pp 144-148.

Conséquences de chocs de bateau sur le pont du Verdon

Konsequenzen einer Schiffskollision mit der Verdon Brücke

Consequences of a Ship Collision with the Verdon Bridge

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RÉSUMÉ
Le projet du futur pont du Verdon, long de 10 km sur la Gironde a été dirigé par le problème du risque de choc de bateaux sur ses piles. La protection a été concentrée sur les deux piles encadrant le chenal de navigation; les conséquences de la rupture de toute autre pile courante ont été limitées.

ZUSAMMENFASSUNG
Bei der geplanten, 10 km langen Brücke über die Gironde (Pont Du Verdon) stand das Problem einer eventuellen Schiffskollision mit den Zwischenpfeilern im Vordergrund. Die Schutzmaßnahmen konzentrierten sich auf die beiden Pfeiler, die die Fahrrinne begrenzen; die Folgen eines Bruches eines anderen normalen Pfeilers wurden eingeschränkt.

SUMMARY
The project of the future 10 km long Gironde bridge »Pont du Verdon« was dominated by the problem of possible ship collisions with the piers. Measures of protection were concentrated on the piers marking the fairway. The consequences of a collapse of any other normal pier were limited.

0. PRESENTATION

0.1 Le constructeur en site aquatique (ponts, ports, "offshore") doit se soucier de plus en plus des accidents provoqués par le choc de bateaux. Pour tenter de les prévenir ou d'en limiter les conséquences, l'ingénieur est actuellement mieux renseigné sur le comportement du bateau-projectile(1) que sur celui de sa structure-cible. Notamment par les études empiriques de :

- MINORSKY, sur chocs réels entre bateaux [I] (§ 1.1)
- WOISIN, par essais de percussion de modèles de navires sur écrans fixes [II] (§ IV)

0.2 Hypothèses et notations (2)

1 - Le temps t et les déplacements $D(t)$ sont comptés à partir du début du choc ($t=0$; $D=0$). L'accent désigne la dérivation par rapport à t .

2 - Nous intéressant essentiellement au sort de la structure, le bateau-projectile I sera défini par un seul degré de liberté, de translation : $\vec{D}_1$, parallèle à sa vitesse initiale $\vec{V}_1 = \vec{D}'_1\ (-0)$.

La masse $\underline{m_1}$ affectée au bateau comprend, en sus de sa masse propre, celle de l'eau qu'il entraîne dans son mouvement, soit, en pratique, de 10% (choc frontal : fig.1.1) à 40% (choc latéral : fig.1.2) de m_1 .

Fig. 1.1

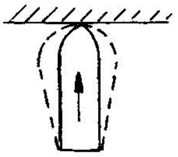
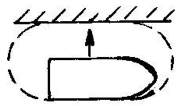


Fig. 1.2



3 - La structure frappée II est supposée élastique. Elle est, jusque là, en équilibre statique. Elle est discrétisée en noeuds K ($K \geqslant 2$, le noeud d'impact recevant le numéro 2). A chacun sont affectées la masse $\underline{m_K}$ et l'inertie massique $\underline{J_K}$ correspondant aux degrés de liberté (DDL) de translation et rotation du noeud.

4 - Le choc est supposé sans rebond : I et II restent en contact pour $t \geqslant 0$. Nous avons donc à étudier la structure globale (I + II), à $\underline{N}$ DDL, au total. Notre analyse portera sur :

- la schématisation de la structure II
- mais complétée, de surcroît, par un "élément de charge" entre noeuds 1 et 2 de façon à introduire les conditions à l'origine (+0), au noeud 1, soit, en notant :

$$\{i\} = \{1, 0, 0 \cdots 0\}^T$$

  la colonne de terme courant (structure) nul, hormis le premier (bateau 1), égal à l'unité :

$$\{D(0)\} = \{0\} \quad \text{et} \quad \{D'(0)\} = V_1\,\{i\} \qquad (1)$$

Cet élément de charge (1-2) reçoit la raideur $\underline{r_1}$ et l'amortissement relatif $\underline{\alpha_1}$. r_1 peut témoigner des capacités de déformation locale à l'impact. Supposer son amortissement "critique" ($\alpha_1 = 1$) peut permettre d'éviter tout rebond entre les 2 corps.

(1) essentiellement grâce aux compagnies d'assurances de ces bateaux, notamment quant ils sont à propulsion nucléaire. [1] [2]

(2) Symboles : $\{\ \}$ = colonne 1xN ; $[\ ]$ = matrice-carrée NxN ; $\lceil\ \rfloor_{u}$ = matrice diagonale NxN ; T = transposée

1. CAS ELEMENTAIRES : CONNAISSANCE DU BATEAU PROJECTILE (I)

1.1 Choc entre 2 bateaux : (1) et (2)

1.1.1 Si l'on admet (02.2) que les phénomènes hydrodynamiques peuvent se traduire par simple majoration des masses propres des bateaux, les forces extérieures sont toutes verticales. D'où, en projection horizontale :

$$\Sigma F_j = \Sigma m_j D''_j = 0 \qquad (j = 1,2)$$

ce qui, par intégration, conduit à la conservation de la quantité de mouvement globale des 2 bateaux. Par ex., dans le cas où le bateau (1) (masse m_1, vitesse initiale V_1) heurte le bateau (2) (m_2, $V_2 = 0$), les 2 partent après le choc à la vitesse commune D' telle que :

$$m_1 V_1 + m_2 \cdot 0 = (m_1+m_2)\, D' \qquad \text{soit : } D' = D'_1 = D'_2 = \frac{m_1}{m_1+m_2} V_1$$

Il y a donc, lors du choc, perte d'énergie cinétique :

$$\underline{\delta E} = \frac{1}{2} m_1 V_1^2 - \frac{1}{2}(m_1+m_2)\, D'^2 = \frac{1}{2} \frac{m_1 m_2}{m_1+m_2} V_1^2 \qquad (2)$$

qui se transforme en travail de déformation plastique (irréversible) des bateaux ou en chaleur.

1.1.2 Une étude statistique de chocs réels entre bateaux a permis à Minorsky de mettre en lumière une excellente corrélation linéaire entre δE (mMN) et le volume d'acier A (m^3) broyé lors du choc (fig.2) : $\delta E = 42,2\, A + 29,3$ (3)

En désignant par : $D_1 - D_2 = \underline{x}$ (m) l'enfoncement mutuel des bateaux, la force extrême F (MN) qu'ils exercent l'un sur l'autre est donc :

$$F(x) = F_1 = F_2 = \frac{d\,\delta E}{dx} = 42,2\, \frac{dA}{dx} \qquad (4)$$

Connaître l'architecture des bateaux permet de définir A (x), et donc l'effort F (x) qui provoque leur écrasement mutuel sur la longueur x. La figure 3 en donne quelques exemples [III] et [IV]. Mais il est plus simple d'étudier directement chaque bateau isolément.

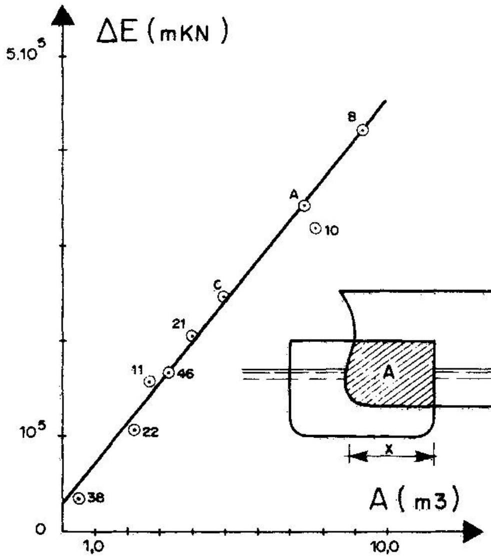


Fig. 2

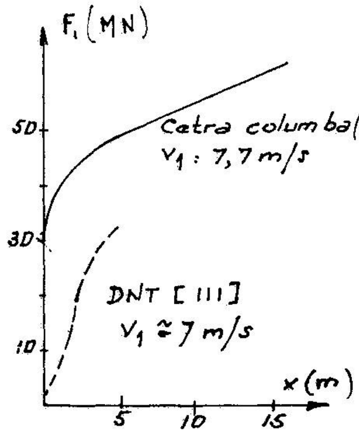


Fig. 3

1.2 Choc de bateau sur un écran rigide (essais de Woisin)

Nous négligeons ici tout amortissement (cf. néanmoins 3.3).

1.2.1 L'équation du mouvement du bateau élastique (de rigidité r_1) durant le choc est (fig.4) :

$$m_1 D''_1 + r_1 D_1 = 0$$

$$\text{soit, avec } \omega_1^2 = \frac{r_1}{m_1} \quad : \quad D_1 = \frac{V_1}{\omega_1} \sin \omega_1 t \qquad (5)$$

jusqu'au rebond (à $t = \frac{\pi}{\omega_1}$; $D_1 = 0$; $D'_1 = -V_1$; $D''_1 = \omega_1 V_1$)

Puis le mouvement est parabolique : $D_1(t) = V_1 \left(t - \frac{\pi}{\omega_1}\right)\left[\frac{\omega_1}{2}\left(t-\frac{\pi}{\omega_1}\right)-1\right]$

durant le temps $2/\omega_1$, et cela recommence (fig.5), avec une avancée, double du rebond vers l'arrière.

1.2.2 Pour un bateau réel, <u>non élastique</u>, la force ($r_1 D_1$) est à remplacer par sa loi propre F_1 (D_1)

L'équation du mouvement : $m_1 D_1'' + F_1(D_1) = 0$

donne, après multiplication par D_1', intégration, et compte tenu des conditions aux limites : $\int F_1 \, dD_1 = \frac{1}{2} m_1 \left(V_1^2 - D_1'^2\right)$

Si la loi F_1 (D_1) est monotone, le bateau subit son effort maximum F_{1M} quand D_1 est maximal, soit $D_1' = 0$. Alors (fig.6) :

$$\int F_1 \, dD_1 = \frac{1}{2} m_1 V_1^2 \qquad (6)$$

L'énergie cinétique initiale du bateau se transforme donc dans son travail de déformation, soit $\frac{1}{2} r_1 D_1^2$, pour un bateau élastique, auquel cas :

$$F_{1M} = V_1 \sqrt{m_1 r_1} \qquad (7) \qquad \text{et} \qquad D_{1M} = V_1 \sqrt{\frac{m_1}{r_1}}$$

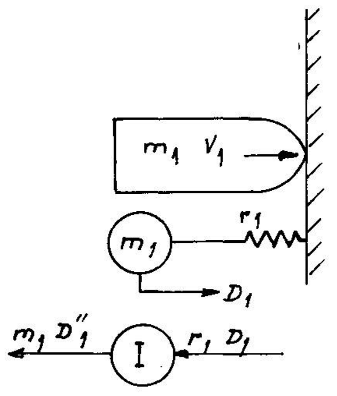


Fig. 4

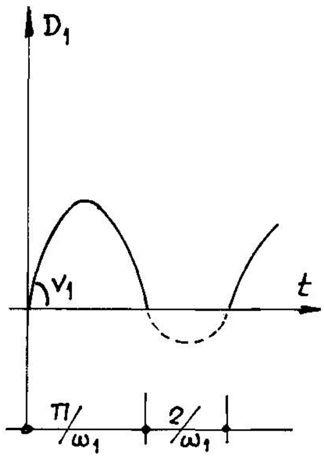


Fig. 5

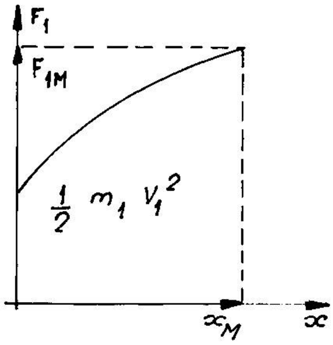


Fig. 6

2. CHOC DE BATEAU SUR STRUCTURE ELASTIQUE - ETUDE DE L'ENSEMBLE (I) + (II)

2.1 Equation du mouvement

La structure globale (bateau + structure, attelés par l'élément 12) a pour matrices (carrées symétriques NxN) : de rigidité [R], de masse [M] et d'amortissement [A]. Son comportement (supposé élastique) est régi, pour t>0, par l'équation, classique en absence de force extérieure :

$$[M]\{D''\} + [A]\{D'\} + [R]\{D\} = \{0\} \qquad (8)$$

où $\{D\}$ est le vecteur des déplacements des N DDL de la structure globale.

2.2 Vibrations propres de la structure globale

2.2.1 Ce sont les vibrations harmoniques : $\{D(t)\} = \{\delta\}\sin\omega t$, que peut subir cette structure, non amortie (A=0). Elles répondent donc à :

$$\left[[R] - \omega^2 [M]\right]\{\delta\} = \{0\} \qquad (9)$$

Ce système homogène n'a de solution $\{\delta\}$ non nulle que si le déterminant de la matrice carrée de son premier membre est nul. D'où, en l'écrivant, les N <u>pulsations propres = ωj</u>, de la structure globale, que nous classons par valeurs croissantes ($\omega_1 < \omega_2 \dots < \omega_N$).

2.2.2 Nous notons la matrice (NxN), non symétrique, des vecteurs propres :

$$[\Delta] = \left[\{\delta_1\}\{\delta_2\}\cdots\{\delta_N\}\right] = \begin{bmatrix} \delta_1^1 & \delta_2^1 \cdots & \delta_N^1 \\ \delta_1^2 & & \\ \cdots & \cdots & \cdots \\ \delta_1^N & & \delta_N^N \end{bmatrix}$$

Ces vecteurs propres sont orthogonaux à $[M]$ et $[R]$. Le système (9) étant homogène, ils ne sont connus qu'à un facteur près. Nous précisons leur définition en les normant par rapport aux masses, soit :

$$[\Delta]^T[M][\Delta] = [1]_d \qquad \text{et donc} \qquad [\Delta]^T[R][\Delta] = [\omega^2]_d$$

L'ensemble des déplacements modaux propres du projectile (1) constitue la colonne :

$$\{\delta^1\} = \{\delta_1^1 \ \delta_2^1 \cdots \delta_N^1\}^T = [\Delta]^T\{i\}$$

Quand les masses sont concentrées : $[M][\Delta]\{\delta^1\} = \{i\}$

2.3 Résolution

2.3.1 (8) se résout grâce au changement de variable : $\{D(t)\} = [\Delta]\{z(t)\}$ (10)

En prémultipliant par $[\Delta]^T$, et en supposant qu'on puisse définir dans chaque mode propre, j, un amortissement relatif αj (≤ 1) on obtient le système des N équations différentielles indépendantes :

$$z_j''(t) + 2\,\alpha_j\,\omega_j\,z_j'(t) + \omega_j^2\,z_j(t) = 0 \qquad (j = 1 \text{ à } N)$$

D'où, en posant : $\omega_j' = \omega_j\sqrt{1-\alpha_j^2}$; $S_j(t) = \frac{1}{\omega_j'}\,e^{-\alpha\omega_j t}\sin\omega_j' t$;

$$C_j(t) = 1 - e^{-\alpha\omega_j t}\left(\cos\omega_j' t + \frac{\alpha\omega_j}{\omega_j'}\sin\omega_j' t\right) \quad : \quad z_j(t) = [1 - C_j(t)]\,z_j(0) + S_j(t)\,z_j'(0) \quad (11)$$

2.3.2 Les conditions à l'origine (1) s'écrivent :

$$\{z(0)\} = [\Delta]^{-1}\{D(0)\} = \{0\}, \text{et} : \{z'(0)\} = [\Delta]^{-1}\{D'(0)\} = m_1 V_1 \{\delta^1\}$$

d'où = $$z_j(t) = m_1 V_1\,\delta_j^1\,S_j(t) \qquad (12)$$

Si l'amortissement du mode propre K devient "critique" ($\alpha_K = 1$)

$$z_K(t) = e^{-\omega_K t}\left[z(0) + t\,(z'(0) + z(0)\,\omega_K)\right] = \delta_K^1\,t\,e^{-\omega_K t}\,m_1 V_1$$

2.3.3 On repasse ensuite aux déplacements réels $\{D(t)\}$ par (10)

2.3.4 Les sollicitations et donc les contraintes dans la structure, ne dépendent que de sa déformation. Elles se calculent donc, à tout t, sous l'action des forces statiques :

$$\{F\} = [R]\{D(t)\} = [M][\Delta]\{\omega^2 z(t)\} = [M][\Delta]\{\delta^1\omega^2 S(t)\}\,m_1 V_1 \qquad (13)$$

3. EXEMPLE SIMPLE D'UNE STRUCTURE (II) A UN SEUL DDL

3.1 La structure-cible est supposée schématisable par un seul noeud : 2, où est concentrée la masse m_2, et dont le seul DDL (suivant l'axe de V_1) est : D_2 suivant lequel sa rigidité est r_2 (fig.7). Son amortissement relatif est α_2

La structure globale (aux 2 DDL : D_1 et D_2) a pour matrices :

$$[m] = \begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \qquad \text{et} \qquad [R] = \begin{bmatrix} r_1 & -r_1 \\ -r_1 & r_1 + r_2 \end{bmatrix}$$

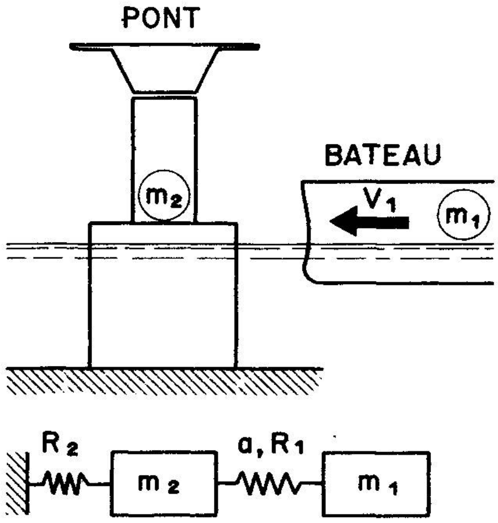


Fig. 7

Ses pulsations propres $\left(\omega_1 < \sqrt{\frac{r_1}{m_1}}\ , \sqrt{\frac{r_2}{m_2}} < \omega_2\right)$ sont racines de :

$$(r_1 - m_1\omega^2)(r_1 + r_2 - m_2\omega^2) - r_1^2 = 0$$

Nous posons : $\beta = \left(1 - \frac{m_1}{r_1}\omega_1^2\right)\sqrt{\frac{m_2}{m_1}}$

3.2 L'équation du mouvement ($t>0$) est :

$$\begin{Bmatrix} D_1(t) \\ D_2(t) \end{Bmatrix} = \frac{V_1}{1+\beta^2}\begin{Bmatrix} S_1(t) + \beta^2 S_2(t) \\ \beta\sqrt{\frac{m_1}{m_2}}\left(S_1(t) - S_2(t)\right) \end{Bmatrix}$$

D'où les actions servant à calculer les sollicitations (et les contraintes) à tout t :

$$\begin{Bmatrix} F_1(t) \\ F_2(t) \end{Bmatrix} = \frac{m_1 V_1}{1+\beta^2}\begin{Bmatrix} \omega_1^2 S_1 + \beta^2\omega_2^2 S_2 \\ \beta\sqrt{\frac{m_1}{m_2}}\left(\omega_1^2 S_1 - \omega_2^2 S_2\right) \end{Bmatrix}$$

3.3 Tant que ω_2 reste fini (r_1 et r_2 finis) : $S_j(0) = C_j(0) = 0 \quad (j = 1,2)$ on vérifie bien les conditions aux limites pour $t=0$. Il en va de même si la structure devient rigide (r_2, donc ω_2 infinis ; D_2 (t)). Le projectile prend alors le mouvement harmonique amorti :

$$D_1(t) = V_1\ S_1(t) \qquad \text{avec} \qquad \omega_1 = \sqrt{\frac{r_1}{m_1}}$$

3.4 Mais si le projectile est rigide (r_1 infini) ω_2 devient infini. D'où une brutale discontinuité des vitesses à $t = 0$, car : $e^{-a\omega_2 t}$ et $(1-C_2)$ passent de 1 à 0; et D'_1 et D'_2, respectivement de V_1 et 0, à la valeur commune : $\frac{m_1}{m_1+m_2}V_1$ (1). Il y a donc conservation de la quantité totale de mouvement.

Après le choc, bateau et cible prennent le mouvement commun :

$$D_1(t) = D_2(t) = \frac{m_1}{m_1+m_2}V_1 S_1(t)\ ; F_2(t) = \frac{m_1}{m_1+m_2}m_1 V_1\omega_1^2 S_1(t)\ ; F_1(t) = \frac{m_1}{m_2}F_2(t)$$
$$\left(\omega_1 = \sqrt{\frac{r_2}{m_1+m_2}}\right)$$

La structure II doit résister à la force totale (F_1 + F_2) de maximum, en absence d'amortissement : $m_1 V_1\sqrt{\frac{r_2}{m_1+m_2}}$

(1) Alors $\beta = \sqrt{\frac{m_1}{m_2}}$; $C_1(+0) \neq 0$

4. COMPARAISON AVEC L'EXPERIENCE

4.1 Cet essai d'analyse ne concerne que des corps restant dans leur domaine élastique et donc réversible. En réalité, ils le dépassent, comme le prouvent :

- la perte irréversible d'énergie cinétique initiale, δE, lors du choc
- la faible valeur du coefficient de Minorsky (3) et (4) = 42,2 MN/m2 (0,42 MPa, soit seulement le dixième de la résistance des aciers utilisés) : lors du choc, le comportement du bateau est donc essentiellement anélastique (voilement des tôles, flambement des longerons, déchirures) et non dirigé par l'épuisement mécanique de son acier.

4.2 La fig.8 [II, tiré de V] décrit l'effort $F_1(t)$ subi par un modèle de bateau durant un choc d'essai par Woisin. Au début, le comportement est élastique, avec un rebond, jusqu'à ce que l'étrave se plastifie, sous l'effort, désormais constant F_1*. Le bateau s'écrase sur la longueur x, assez importante pour que cette phase d'écoulement consomme la majorité de son énergie initiale, soit : $F_1*x \simeq \frac{1}{2} m_1 V_1^2$; l'effort F_1* est donc voisin de la moitié seulement de l'effort maximal F_{1M} (7) initial.

La courbe 9 [mêmes réf.] donne l'allure de F_{1M} en fonction de la masse m_1 du bateau (sans préciser la vitesse initiale V_1 (10 m/s d'après le montage d'essai) avec une variation de ± 50%, due à la différence des étraves des bateaux essayés). Cette allure est bien parabolique, comme l'indique (7).

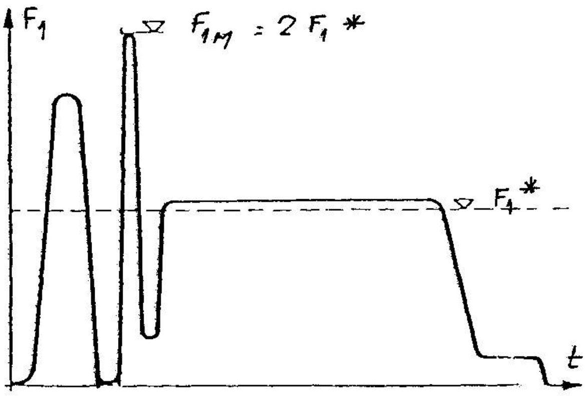


Fig. 8

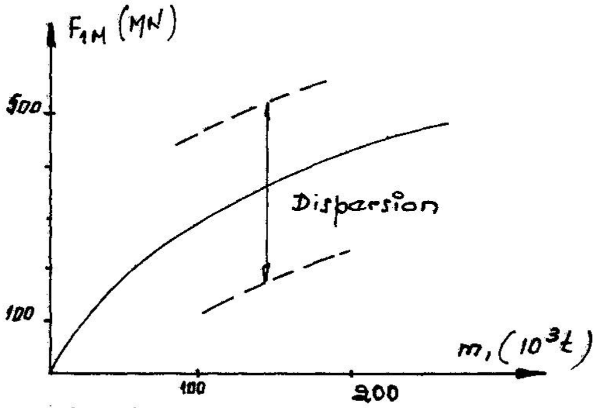


Fig. 9

4.3 Mais cela concerne le bateau. Pas ce qui nous intéresse directement : la structure frappée. Or, il faut bien prendre conscience que l'effort $F_2(t)$, qui agit sur celle-ci diffère de celui, $F_1(t)$, sollicitant le navire, car ces 2 forces connaissent leurs valeurs maximales à des temps différents. (F_2 bien avant F_1).

Les calculs précédents nous montrent que la cible (II) risque de subir son action maximale F_{2M} très peu après le début du choc, alors que son déplacement D_2 est encore très faible, et le bateau peu abimé. Dresser à ce moment un bilan énergétique est difficile, et ne nous renseigne guère sur la valeur de F_{2M} (plus proche de : $V_1\sqrt{k_2 m_1}$ que de F_{1M}, qu'elle peut fortement dépasser).

4.4 Certes, ces calculs supposent la structure élastique. Sa plastification pourrait aiderà réduire F_{2M}. Mais au prix de déplacements excessifs du tablier porté.

Par ailleurs, même en comportement strictement élastique, le calcul se heurte à un problème de schématisation de la structure. Plus, en effet, on discrétise celle-ci en de nombreux noeuds, pour affiner son comportement, et plus on réduit la masse affectée à chacun d'eux, en particulier m_2, au noeud d'impact 2. Or, l'exploitation numérique de la méthode prouve que, ce faisant, la sollicitation locale de la structure (déterminante pour son dimensionnement) augmente. Il faut donc connaître l'aire d'impact du bateau, à partir de laquelle celui-ci est supposé mobiliser la masse partielle m_2 de structure "arrosée". Il conviendrait de le vérifier par essais.

POUR CONCLURE

Même limitée à l'hypothèse élastique de comportement de ses piles, la tentative précédente d'analyse du choc d'un bateau sur une structure "offshore" s'avère délicate, notamment quant aux conditions aux limites, et à la discrétisation (en masses) de la cible. Son application au projet du Verdon (objet d'une autre communication) et au premier dégrossissage d'un pont sur Gibraltar [VI] nous a prouvé que nous manquons surtout de résultats expérimentaux pour en tester le bien-fondé. Compte tenu de l'augmentation inquiétante des accidents par chocs de bateaux et du coût qui en résulte, le temps semble bien venu de lancer un programme international d'essais, que pourrait utilement aider à définir le présent colloque de Copenhague.

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Structural Design to Resist Impact

Conception de structures résistant aux chocs

Aufprallsicherer Entwurf und Konstruktion

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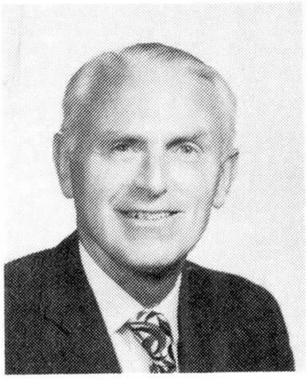

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SUMMARY
A bridge pier, under impact loading from a ship collision, exhibits a dynamic response and, in conjunction with the foundation, is the ultimate participant in the energy absorption equation. The capacity of this pier foundation system to resist the impact in a ductile fashion can be significantly enhanced at moderate cost by selecting an appropriate configuration for the pier base, proper reinforcement for pier shafts, and sand or concrete fill of hollow piling, and by providing adequate bearing support and restrainers for superstructure connection to the pier cap girder.

RÉSUMÉ
Soumise à une charge d'impact provenant de la collision d'un navire, une pile de pont produit une réaction dynamique, et, associée aux fondations, participe en dernier lieu à l'équation d'absorption d'énergie. La capacité des fondations de la pile à résister aux chocs de manière ductile peut être sensiblement améliorée à bon marché en choisissant la configuration appropriée à la base de la pile, en renforçant correctement les piliers, ainsi qu'en fournissant des supports et retenues appropriés au raccord de la superstructure joignant la partie supérieure de la pile.

ZUSAMMENFASSUNG
Ein Brückenpfeiler weist beim Aufprall durch einen Schiffskörper eine dynamische Reaktion auf und ist, in Verbindung mit dem Fundament, der elementare Bestandteil in der Gleichung der Energieabsorption. Die Fähigkeit dieses Pfeilerfundamentes, einem Aufprall in nachgiebiger Weise zu widerstehen, kann bei geringen Kosten erheblich verbessert werden, indem eine geeignete Konfiguration für die Pfeilergrundlage, eine angemessene Verstärkung der Pfeiler sowie eine Sand- oder Betonfüllung für Hohlräume ausgewählt, und außerdem geeignete Lagerungen und Verstrebungen für die Hochbauverbindung zum Pfeilerträger vorgesehen werden.

1. STRUCTURAL DESIGN TO RESIST IMPACT

1.1 Introduction

Studies of actual ship collisions with bridge piers have shown that the side and approach piers are at least as likely to be hit as are the main piers and that such collisions may be catastrophic -- in disruption of traffic, damage to the ship, and loss of life [1]. Providing adequate protection against collision may be practicable for main piers but will often be impracticable or uneconomical for these approach and side piers. Even for protected piers, the protection may not be able to fully absorb all the energy of a maximum collision and remnant forces may be delivered to the pier. Further, while it will generally prove impracticable to design a bridge pier to withstand by itself the maximum ship collision forces, which as shown in the paper by Brink-Kjaer, Broderson, and Hasle Nielson [2], can reach values of 300 to 600 MN, a high proportion of the actual collisions will involve smaller vessels and lower impact velocities.

This paper therefore addresses the design of the bridge pier itself and the practicable means which may be taken to enhance its capacity to resist impact and to minimize the consequences of ship collision.

Mr. Sven Fjeld in his introductory lecture [3] discusses indirect design measures "to obtain reasonably ductile and robust structures." In a particularly relevant section of that paper he states: "Measures to obtain ductility are:

- Connections of primary members to develop a strength in excess of the member.
- Redundancy in the structure so that alternative load distribution may be developed.
- Avoid dependence on energy absorption in slender struts with non-ductile post-buckling behavior.
- Avoid pronounced weak sections and abrupt change in strength or stiffness.
- Avoid, as far as possible, dependence on energy absorption in members acting mainly in bending.
- Utilize non-brittle members.

1.2 Ship Interaction with Bridge Piers

As has been printed out by numerous authors, the energy of the ship plus its associated hydrodynamic mass must be absorbed by such vessel-related phenomena as crushing of the ship hull and hydrodynamic damping, by elasto-plastic and crushing deformations in any protective systems, and by deformation of the pier system itself. It is this last item which will be specifically addressed in this paper since most published literature treats the pier system as a rigid structure.

In actual cases of catastrophic collision involving large ships, the ship is finally brought to rest by the deformation of the pier system, e.g., the pier is displaced laterally and crushed. In less catastrophic cases the pier has been damaged locally and displaced on its foundation but without collapse. These two illustrations show that the bridge pier system does play an important even if undesired role in the absorption of remnant energy (the Δ_3E of Woisin as quoted by Saul and Svensson [1]).

The pier system typically consists of the pier shafts and cap, supported on a large footing which may incorporate piles, plus the soil and water acting with the pier as it is accelerated by the colliding force and then brought to rest.

It is important to note that there is an added mass effect of both the surrounding water and the soil. The forces developed are resisted not only by the inertial forces involved and the deformations in the pier proper but by the soil under the footing, that around any piles, and that acting against the side of the pier in passive resistance. Soil resistances require measurable strains in order to mobilize resisting forces.

This then becomes a dynamic mode of resisting the collision forces that reach the pier, in which the natural period of the pier-foundation system determines the degree of compliance. Fortunately the duration of ship impact by large vessels, 2 to 5 seconds or more (see Brink-Kjaer, Broderson, Nielsen [2]) is of the same order as that of the bridge pier, typically 2 to 4 seconds under maximum strain. The exact interaction depends to a high degree on the foundation soils and to a lesser degree on the relative masses of the colliding ship and pier system.

So far, the discussion has assumed a massive pier under an impact from a large colliding ship that will excite the entire pier, e.g., an impact applied at the pier base or footing. If the impact is on the pier shafts, then of course these respond primarily as a member in flexure and shear and the resistance of the pier-soil system cannot be fully mobilized.

An impact produces not only lateral shear forces on the pier but also overturning moments, leading to high bearing on the far side and reduced bearing or even producing uplift on the near side. The moment developed is of course dependent on the elevation of impact. Of importance for both gravity-base bridge piers and gravity-based offshore structures is the reduced effective bearing area which arises under high lateral forces.

1.3 Enhancing the Global Resistance of the Pier

In addition to the normal energy considerations for ship-bridge pier collision, momentum aspects are also involved, since this is a dynamic response. The larger the mass of the pier, the longer the period; hence, the greater the compliance available, especially for the more severe collisions. Thus arises our intuitive belief that a large massive pier, whether founded on piles or on soil or rock, will be more effective in resisting a collision than a pier of minimal mass.

The pier is accelerated by the collision, then decelerated by the soil. This is almost never an elastic response, thus most of the stored energy is used up in damping, although the pier will typically rebound a short distance from its maximum deformation.

The more massive and presumably larger pier will therefore mobilize greater inertial forces in itself, the surrounding water, and the supporting soil.

The mass of a pier therefore should not be minimized in design. Thick footing blocks are more desirable than thin ones.

Especially for a side pier where navigational and hydraulic characteristics may not be so severe, the pier base may be carried upward either in concrete or by simply piling a mass of gravel on top of it, contained by walls.

Alternatively the pier base may be flared up into the shaft, in a gradual transition rather than the typical abrupt change. This will then have the advantage of avoiding an abrupt change in stiffness, as recommended by Fjeld. It will add mass to the pier. It can be designed to serve as a deflector to cause the ship's bow to shear off prior to hitting the pier shaft.

In any event, to the maximum extent possible, the dimensions and profile of the pier base and base-shaft transition should be such as to force the bulbous bow typical of larger ships even at light draft to engage the base before the upper flared bow hits the shaft. This may encourage the raising of the footing block and enlargement of the base in plan, all of which also adds resisting capability.

1.4 Piling

It is increasingly common to design bridge piers using tubular (cylinder) piles of either steel or concrete capped at the waterline with the footing block. Such piers are well-suited to seismic areas because of their flexibility, but this unfortunately may reduce their capacity to absorb maximum collision impact.

These tubular piles, while flexible, tend to have a non-ductile mode of ultimate failure due to compression and buckling. The compressive capacity and ultimate curvature of concrete piles can be significantly increased by increasing the hoop (confining) reinforcement. The buckling capacity and local deformation capacity of steel cylinder piles can be significantly improved by filling them with sand.

Tension ties should be provided between the pile and capping block to prevent pull-out under overturning. If any batter (raker) piles are used, adequate reinforcement must be provided in the capping block to prevent punching shear.

Finally the mass of the footing block can be increased as noted earlier, either by concrete or gravel fill.

1.5 Scour

Scour around bridge piers can significantly reduce their capacity for lateral loads such as collision. It removes the favorable passive resistance of surrounding soil and decreases the added mass of the soil participating in the dynamic response of the pier. In the case of pile-supported piers, it may lead to unacceptable displacements at the head of the shafts. Paradoxically, within the piles' capacities, it may increase the dynamic energy that is absorbed by the pier.

This, therefore, is an added reason for taking pains to provide adequate scour protection around bridge piers in a waterway.

1.6 Keying and Doweling

Piers founded on rock, hardpan, or conglomerate may have their lateral resistance significantly increased by appropriate keying. This mobilizes additional soil mass in both passive resistance and inertial resistance. The concrete key should be checked to ensure that its shear capacity is adequate.

The overturning resistance as well as the shear resistance can be increased by doweling from the pier base into the rock.

1.7 Pier Shafts

If these are impacted, as by a large barge or flared bow of a ship, they have comparatively little resisting capability. As they deform in flexure, failure in compression and shear will usually occur before the global resistance of the pier can be mobilized.

Many dual shaft piers are connected either at the top by a pier cap and sometimes by intermediate diaphragms as well, causing the two shafts to act as a rigid frame. In this case, the far shaft may fail in compression and the near shaft in tension. In the case of the Tampa Bay Bridge, the near pier failed by pull-out bond failure of the lapped splices of the vertical bars. The far pier failed in compression in a brittle fracture mode.

The ultimate capacity of these shafts can be enhanced significantly at relatively small cost.

- Lapped splices should be staggered and employ double the code length for overlap, since the code requirements are for static, not dynamic loads. In particular, the typical design in which all the main vertical bars from the pier base end one meter or so above the base, to lap with similar bars from the shaft, should be avoided. This is a point of maximum moment and shear, and splices should be staggered as far above the pier base as practicable.

As an alternative, mechanical splices, certified to develop full strength of the bars under impact load, can be employed.

- To prevent initiating compression failure due to high bearing under the ends of bars, laps should be tied at both ends.
- Compressive failure, combined with bending can be rendered much more ductile by means of confinement. Tests on rectangular cross-section members have shown that the ultimate curvature (while still carrying the design axial load) can be increased by a factor of three (to a strain of 0.008) by providing proper confining spirals or stirrups, in an amount similar to that required for seismic design of columns.

  Tails of stirrups should be turned in and anchored in compressive zone.
- Increasing the vertical steel reinforcement, especially near the juncture with the base and cap, can significantly improve ductility as well at ultimate moment capacity, especially if combined with increased confinement.
- Punching shear capacity of hollow shafts can be improved significantly by the use of through-wall stirrups, as described for the shaft walls of offshore structures by Fjeld. [3]

In some cases, twin shaft piers can be designed so that even with the rupture of one shaft, the cap is so connected to the remaining shaft that it can carry the dead load of the span in cantilever. This provision has also been mentioned by Fjeld. [3]

1.5 Superstructure Considerations

In a number of catastrophic ship-bridge collisions, the dislocation and deformation of the pier and the shaft have caused a span to fall off its bearings. This is analogous to the similar problem experienced so often in earthquakes.

Longer bearing (support) areas can be provided.

Stops can be provided at the ends of cap girders, to prevent girders falling off sideways.

Restrainer devices, similar to those used in Japan and California, should be provided to connect superstructure elements on all overwater spans.

Finally, chains have been installed which catch a span or girder even after it has moved off the support, preventing it from falling free.

This type of failure, so catastrophic is consequences, seems inexcusable in the future, since preventive action such as noted above, is so economically and easily accomplished.

Finally, although bridge authorities have been slow to adopt it, the need is being recognized to incorporate signal lights and warning devices at the ends of bridges to stop the senseless loss of life due to roadway traffic continuing to drive over the open span.

2. CONCLUSIONS

The ability of bridge piers to absorb ship collision without catastrophic collapse can be significantly enhanced by selecting appropriate configurations for the pier base.

The evaluation of the energy dissipation during collision should consider the dynamic response of the bridge pier-foundation system. The ability to exhibit "compliance" depends on the period of response of the pier foundation system under large impact forces relative to the duration of impact. For this reason, massive piers have greater energy absorbing capacity under major impact.

The capacity of pier shafts to absorb impact and their ductility can be increased by up to three times by increased splice and anchorage embedment lengths, and by increased confinement in the form of properly detailed hoop steel. Similarly, the catastrophic dislodgement of superstructure girders and spans can be inhibited by enlarged bearing support areas, and restraining devices.

Structural solutions, such as those outlined above, cannot by themselves give full protection but can, at minimal increase in cost, enhance the ductility of the overall pier system and minimize the consequences resultant from ship collision.

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Safeguard System of the Bisan-Seto-Bridge in Japan

Protection du pont Bisan-Seto, Japon

Schutz der Bisan-Seto Brücke in Japan

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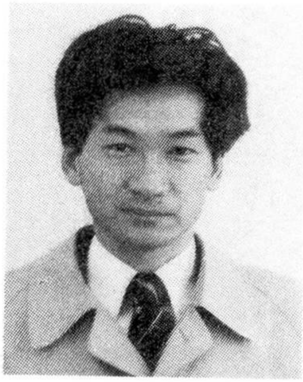


Kuniaki Shoji, born 1948, got his doctor of engineering degree at the University of Tokyo in 1975. He studies naval architecture and marine engineering including the problems of mooring dynamics and ship's collision.

SUMMARY
This paper decribes the behaviours of ships passing near a bridge pier and of the colliding ships with the pier, and also deals with the deformation characteristics of ships and protections. The performance of the protection to be installed on one of the piers of the Honshu-Shikoku Bridges is presented.

RÉSUMÉ
L'article décrit le comportement de navires près des piles de pont et lors de collisions avec celles-ci. Il traite les caractéristiques des déformations des navires et des protections. L'article présente la protection qui doit être réalisée pour une des piles du pont Honshu-Shikoku.

ZUSAMMENFASSUNG
Dieser Aufsatz behandelt das Verhalten von Schiffen in der Nähe eines Brückenpfeilers sowie die Kollision mit dem Pfeiler. Verformungseigenschaften von Schiffen und Schutzwerken werden beschrieben. Der Schutz eines Pfeilers der Honshu-Shikoku-Brücken wird dargestellt.

1. INTRODUCTION

In Japan, the Kojima - Sakaide route of the Honshu-Shikoku bridge project is now under construction as shown in Fig.1. The main bridge of this route is the Bisan-seto Bridge which spans the main traffic route of ships. The traffic of this route is more than 450ships per day. The massive piers of the bridge are builded in this ship's passage of the Bisan Straits where the water depth is over 30m and the tidal current is about 4knots. Consequently the probability of ship collision with the piers is existed.

This paper describes the fundamental investigation about the safeguard system against the ship collision with the piers of the Bisan-seto Bridge and the details of the protection already installed on one of the piers tentatively as shown in Fig.2.

2. BEHAVIORS OF SHIP COLLISION WITH PIER

In this waterway the environmental conditions affected on the ship's handling are severe considerably. Because, the tidal current is very strong and moreover westerly wind becomes rough in winter. Sometimes these severe conditions adversely affect on ship's steering. In this chapter, the behaviors of the ship collision which is caused by such strong current or wind are presented.

2.1 Flow Pattern around the Pier in Current or Wind

The flow of the tidal current or wind around the pier is curved. Fig.3 and 4 show the velocity distribution or the streamline around the pier in the tidal current or wind. In Fig.3 the result by model experiment coincides with the result of full-scale measurement. Fig.4 is the example of the model experiment in the model basin with wind tunnel. It is observed that the velocity becomes high by 15 ~ 20% on the transverse side of the pier. These current or wind velocity distributions around the pier is almost represented by potential flow for the ideal fluid [1].

2.2 Collision of Navigating Ship in Current or Wind

When a ship passes through

Fig.1 The Kojima-Sakaide Route of the Honshu-Shikoku Bridges

Fig.2 The Protection Installed on the No.5 Pier of the South Bisan-seto Bridge

near the pier, she deviates her course from the original path by the unsymmetrical force and moment. This force and moment is occured by the sheer flow near the pier and occasionally brings on the ship collision with the pier. In Fig.5 and 6, the boundary of ship course clearance to the pier side (Y_o), for keeping on safe navigation are presented. They are obtained by the simulation which is used the steering motion equations of the ship[2].

The course clearance to the pier for keeping safe navigation which is shown by the ratio of Y_o to the pier width (B_p) is depended on the velocity of current or wind to ship speed (V_c/V_s or V_a/V_s).

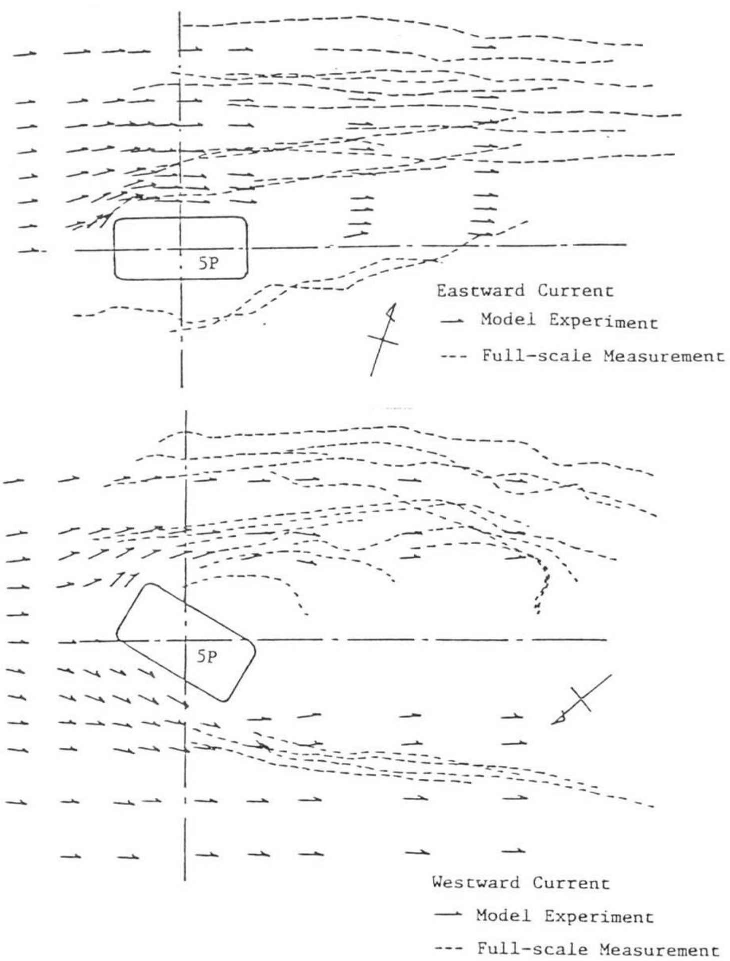


Fig.3 Flow Pattern around the No.5 Pier

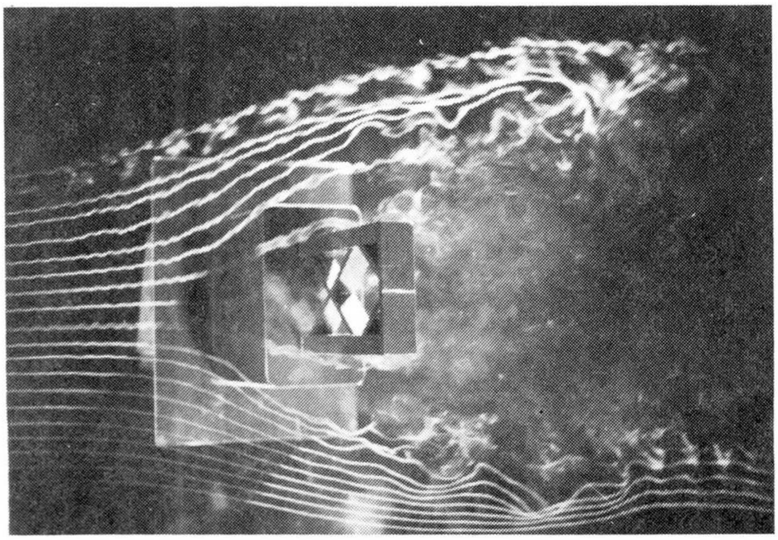


Fig.4 Streamline around the Pier in Wind

2.3 Collision of Drifting Ship in Current or Wind

When a ship is unsteerable owing to her engine or rudder trouble she is just drifted by current or wind.

2.3.1 Drifting in Current

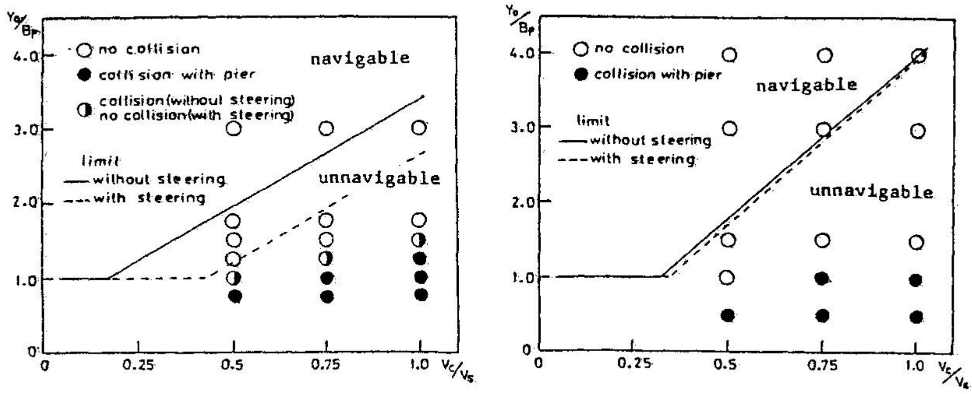
According to the model experiment the behaviors of unsteerable ship under current are as follows.

(1) drifting course
In Fig.7 the dangerous drifting course of ship to come into collision under the current is shown. It is noticeable that if the ship's heading obliques to the current direction, the ship is drifted not downstream but diagonally.

(2) colliding speed
In Fig.8, the ship's colliding speed (Vsc) with the pier under the strong current is presented with the ratio to the current velocity (Vc). The colliding speed increases as the growth of the transverse distance between the colliding position and the center of the pier (Y). The colliding speed increases by about 20% of current velocity (Vc) when the ship collides with the corner of the pier.

2.3.2 Drifting in Wind

According to the model test results the unsteerable ship is drifted by wind down abeam and the drifting speed is described as following formula,



(a) reverse current (b) fair current

Fig.5 Boundary of Navigability of Ship in Current
(Breadth of Pier / Length of Ship = 0.54)

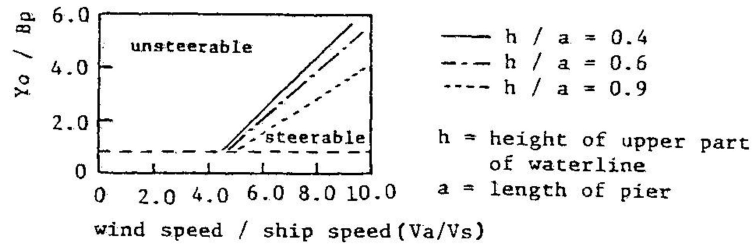


Fig.6 Boundary of Navigability of Ship in Beam Wind

$$Vs = 0.041\sqrt{\frac{S}{L\,d}}\ Va \qquad (1)$$

where Vs; drifting speed of ship in wind, S; transverse projected area of ship, L; length of ship, d; draft of ship, Va; velocity of wind.

Moreover the speed of the ship collided with the pier is increased by the confused wind around the pier as shown in Fig.9. The colliding speed increases by about 10% of the speed (Vs) obtained from the formula (1) on the case of collision with the corner of the pier [3].

3. STRENGTH CHARACTERISTICS OF SHIP AND PIER PROTECTION

3.1 Load-Deformation Characteristics of Ship

Static collapse tests were conducted to examine the load-deformation characteristics using steel bow models which simulate the transversely framed structure of cargo - type ship of 500 GT and 4000 GT. Calculated formulae to the load-deformation characteristics are as follows,

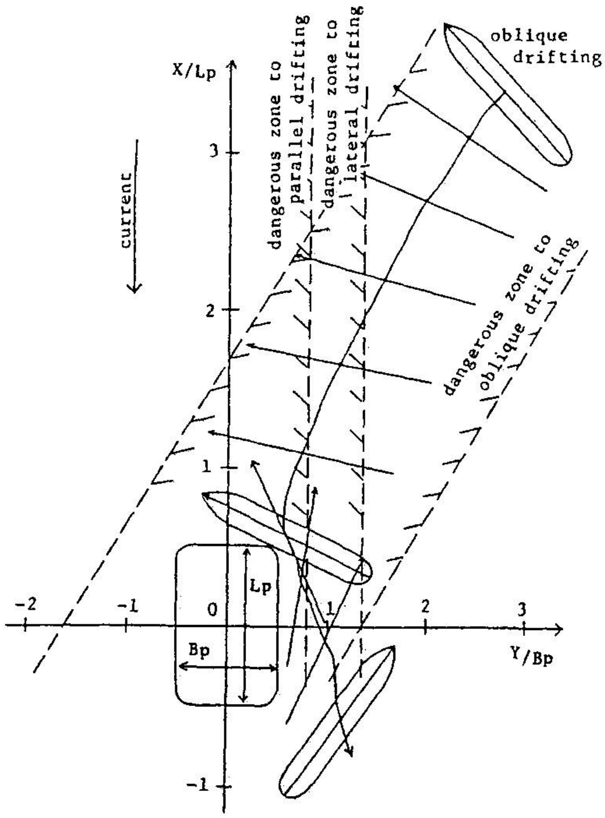


Fig.7 Dangerous Zone to the Drifting Ship in Current

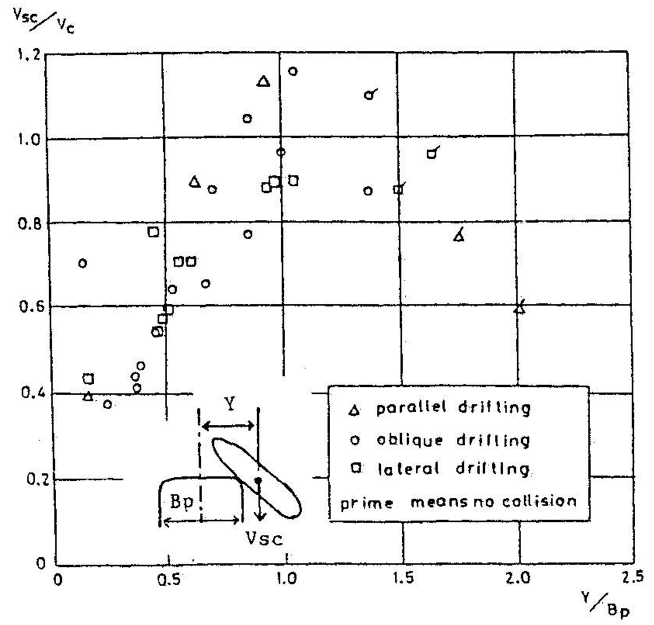


Fig.8 Colliding Speed of Drifting Ships in Current

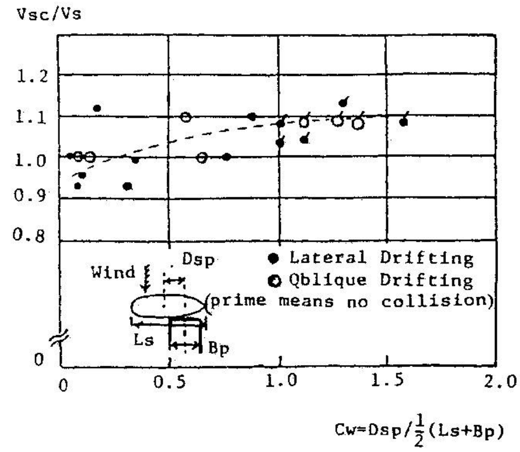


Fig.9 Colliding Speed of Drifting Ships in Wind

(1) bow collision with the straight-part of the pier

$$P = 2.72\delta_F^{-1}W^{1/3}(0.71W^{1/6}+1)^3\,X \quad \text{in } 0<X<\delta_F \qquad (2)$$

$$P = 2.72W^{1/3}(0.71W^{1/6}+1)^3 \quad \text{in } \delta_F \le X \qquad (3)$$

(2) ship-side collision with the corner of the pier

$$P = 83.1r^{-1/3}(0.95W^{1/6}+1)(0.57W^{1/3}+4r)X^{1/2} \quad \text{in } 0<X<2r/9 \qquad (4)$$

$$P = 39.2r^{1/6}(0.95W^{1/6}+1)(0.57W^{1/3}+4r) \quad \text{in } 2r/9 \le X \qquad (5)$$

where P;collapse load (ton), X;deformation (m), W;gross tonnage (GT), δ_F;raked stem length, r;corner radius of the pier.

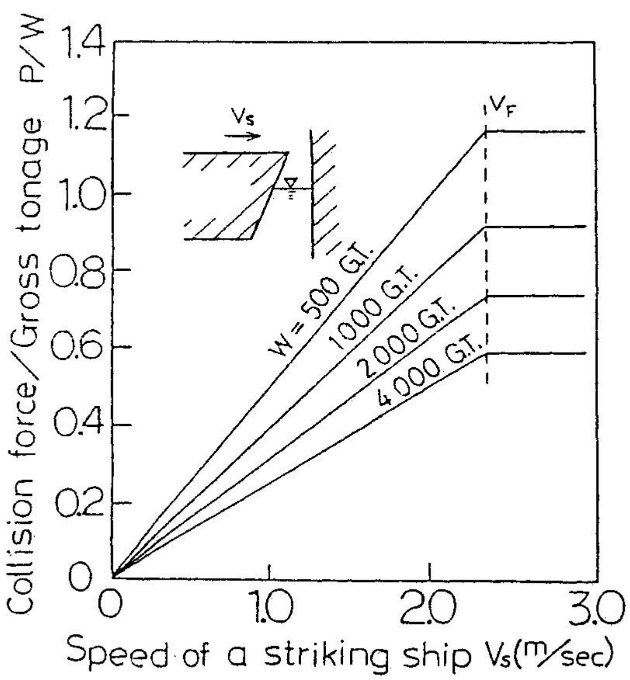


Fig.10 Estimated Impact Forces at the Bow Collision

Using the simplified load-deformation curve, ship impact forces can be estimated. In Fig.10 the estimated results are shown for the ship-bow collision with a right angle against a straight-part of the rigid bridge pier [4].

According to Fig.10, V_F which is the collided speed resulting in the full collapse of the part of the raked stem is equal to about 2.3 m/s for every ship ranging from 500 GT to 4000 GT. Maximum impact force is estimated to be about 580 tons for the 500 GT ship. Similarly load-deformation curve is estimated in the case of the ship-side collision against a corner of the rigid bridge pier.

Impact force which the drifting ship receives from the buffer is examined theoretically by one of authors [5]. Hereon, it is assumed that the ship is rigid and buffer is deformarable. Calculated formula is

$$P_M = (V_s + L_c\ \omega s\ \cos\theta)\sqrt{\frac{k}{1/M_{v\phi} + L_c^2\cos^2\theta/I_v}} \tag{6}$$

where, $M_{v\phi}$; virtual mass of ship in ϕ direction($=M_{v\xi}\cos^2\phi + M_{v\eta}\sin^2\phi$), k ; spring constant of the buffer, V_s;drifting speed of ship, L_c;the length between center of ship and colliding point (oc), ω_s;angular velocity of ship, θ;angle between oY and oc, I_v ;virtual moment of inertia around center of ship, $M_{v\xi}$;virtual mass of ship in ξ direction, $M_{v\eta}$;virtual mass of ship in η direction, ϕ;angle between oξ and V_s. The experimented data of the impact force are rather good agreement with calculated value in Fig.11.

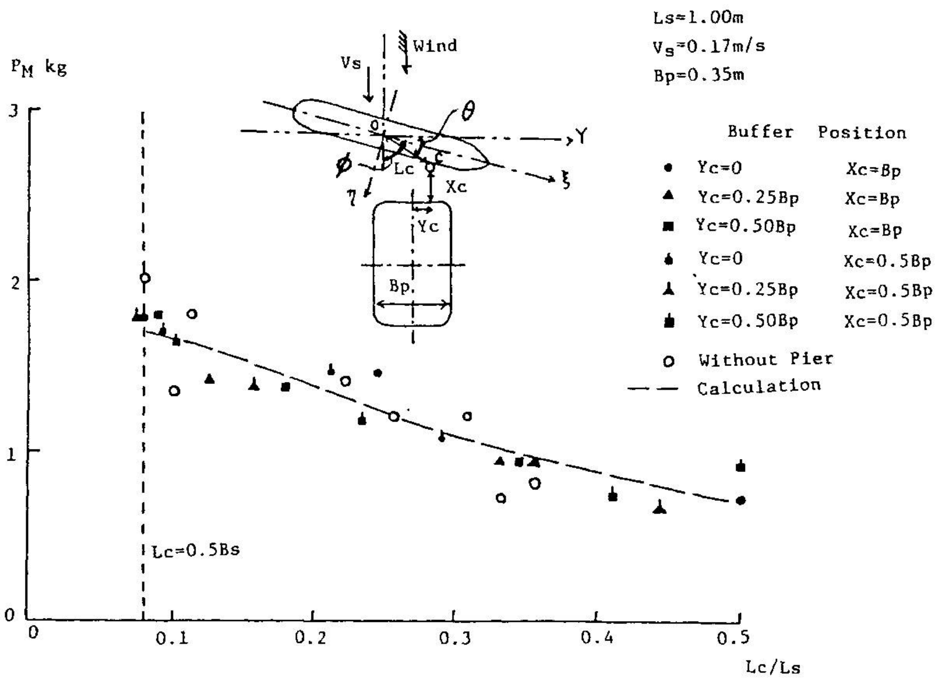


Fig.11 Impact Forces of Drifting Ship in Wind

3.2 Load-Deformation Characteristics of Protection

Judging from the viewpoint of designing the ship-pier protection, it may be said that the impact forces should be reduced to the values less than the buckling loads of the bow hull plate by means of effective buffer devices installed on the pier. The comparisons between the force - bow penetration curve for the four kinds of buffer devices are shown in Fig.12. It appears from Fig.12 that the composite type buffer device which is made from hard polyurethane foam has almost linear characteristics in the relationship between the force and the ship penetration while other three kinds of buffer devices have somewhat complicate characteristics.

It can be stated from the viewpoint of practical designing that the composite type is the most suitable buffer device among the proposed ones. The composed deformations of the bow and the respective buffer devices can be estimated from the linear combination of each load-deformation curves.

In case of the design of the protection installed on No.5 pier of the South Bisan-seto Bridge, it is based on these characteristics about the ship impact force and the bow penetration for the buffer device.

4. DETAILS OF PROTECTION INSTALLED ON THE NO.5 PIER

4.1 Collision Pattern and Size of Ship

The protection of No.5 pier of the South Bisan-seto Bridge was constracted tentatively. The behaviors of the ship collision to the pier are described in the chapter 2. Moreover, in the Bisan Straits the ship traffic route is already established according to the separation schemes by the IMO recommendation. It has the clearance of about 120m between the boundary of the traffic route and the pier.

From these situations, the conditions about the design of the protection installed on No.5 pier are set up as shown in Tables 1 and 2.

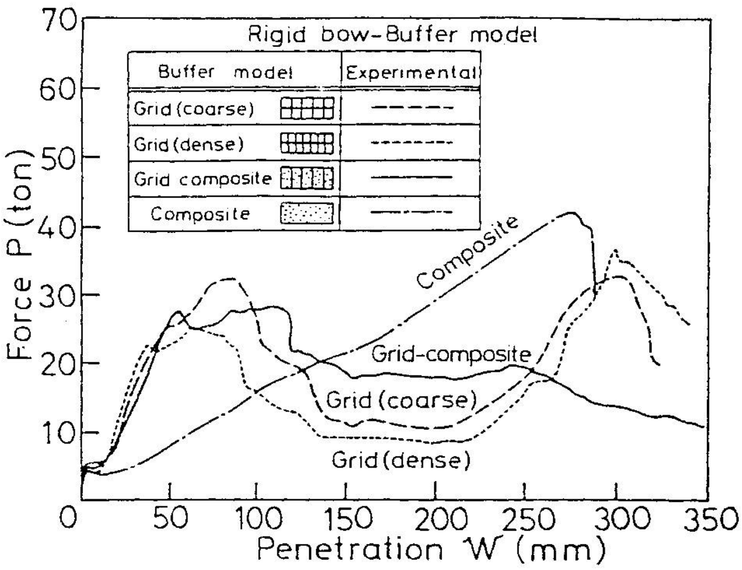


Fig.12 Comparisons between the Force-Bow Penetration Curves for Four Kinds of Buffer Devices

Size of Ship	Colliding Speed
Fishing Boat (Displacement10ton)	4 knots
Passage Crossing Ship (200 GT)	8 knots
Passage Crossing Ship (500 GT)	8 knots
Drifting Ship (500 GT)	5 knots

Table 1 Size of Ship and Colliding Speed

Kind of Ship	Colliding Forms	
	Ship	Pier
Navigating Ship	Bow	Straight-part
Drifting Ship	Ship-side	Corner

Table 2 Colliding Forms

4.2 Design Conditions of No.5 Pier Protection

In order to design the protection of No.5 pier, the strength of the ship and the allowance of collapse are estimated as shown in Tables 3 and 4. Environmental conditions is that wind velocity is 37.5m/s, significant wave height is 2.5m, significant wave period is 4.8 s, significant wavelength is 35.9 m, maximum wave height is 4.5 m and tidal current velocity is 4.5 knots. The protection of No.5 pier is composed of grid-composite type buffer and rubber fender as shown in Fig.13.

Size of Ship	Raked Stem Length	Strength of Bow	Strength of Ship-side
10 Disp.ton	———	———	7 ton/m^2
200 GT	0.83 m	186 ton	10 ton/m^2
500 GT	1.13 m	366 ton	14 ton/m^2

Table 3 Strength of Ship

Part		Critical Allowance
Ship	Bow	the collapse within 2/3 length from bow to collision bulkhead
	Ship-side	the collapse within elastic deformation
Buffer Device		the collapse of the main structure
Bridge Pier		no movement, no overturn having not a bad effect on upper structure

Table 4 Allowance of Collapse

In case of design and selection of the pier protection, the problem about the water depth, the water area, the range of tide and the maintenance is also considered.

4.3 Evaluation

It is recognized by the members of the technical committee of the Honshu -Shikoku Bridge Authority that this safeguard system is effective through the experience of about one year after installation. Moreover it is under going to study about the several problems against the environmental conditions such as current and wave.

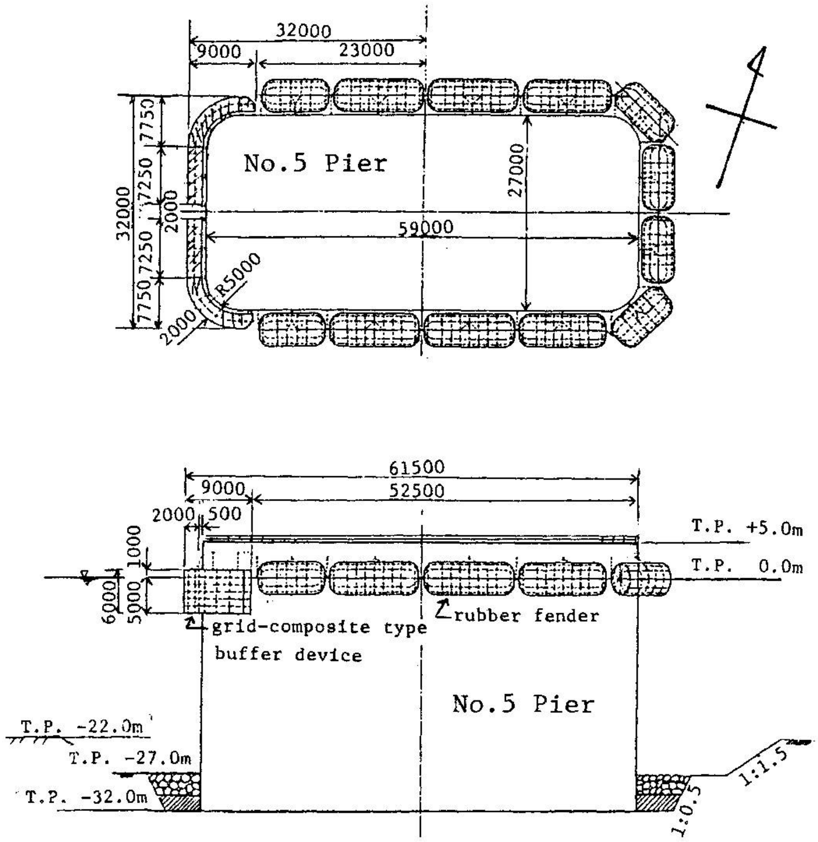


Fig.13 Details of Protection Installed on the No.5 Pier

ACKNOWLEDGEMENT

The authors wish to thank the Honshu Shikoku Bridge Authority and the members of the technical committee for their backup during the course of this investigation.

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Ship Collision Analysis for the Westerschelde Crossing

Analyse des collisions de bateaux pour la jonction sur le Westerschelde

Analyse der Kollision von Schiffen für die Verbindung über die Westerschelde

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SUMMARY

The paper describes the selection of a tunnel-bridge connection. It also explains why a suspension bridge minimizes the results of a ship-pier collision for this situation. The advantages and disadvantages of several bridge types are mentioned. To learn the risks of a collision with the stiffening truss of the bridge a risk analysis was done. Damage levels are used to judge the design.

RÉSUMÉ

L'article décrit la procédure de sélection d'un pont-tunnel. Un pont suspendu diminue les conséquences d'une collision d'un bateau contre un pilier dans cette situation. Les avantages et désavantages de plusieurs types de ponts sont donnés. Une analyse des risques a été entreprise pour le cas d'une collision contre les poutres de rigidité du pont. Des niveaux de dégâts sont utilisés pour juger le projet.

ZUSAMMENFASSUNG

Der Artikel beschreibt das Selektionierungsverfahren einer Tunnelbrücke. Eine Hängebrücke vermindert die Folgen einer Kollision eines Schiffes mit einem Pfeiler in dieser Situation. Die Vorteile und Nachteile mehrerer Brückentypen werden erwähnt. Um die Risiken einer Kollision mit dem Versteifungsträger zu schätzen, wurde eine Risikoanalyse gemacht. Schadenniveaus werden gebraucht, um das Projekt zu beurteilen.

1. GENERAL ABOUT THE PROJECT

1.1 Introduction

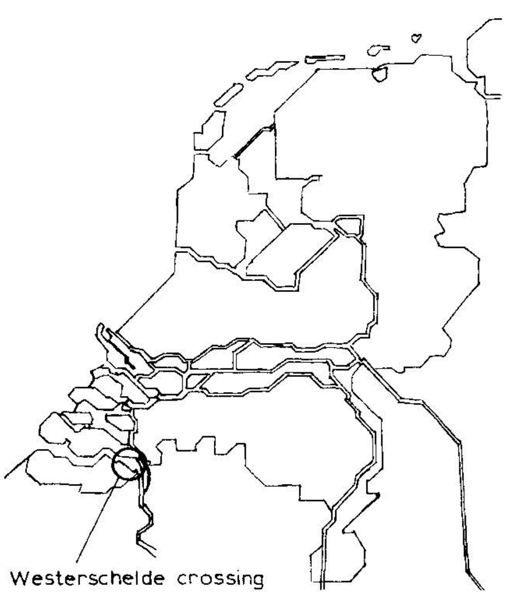
In 1978 the province Sealand decided to start with the preparations to change the present ferry connection over the Westerschelde by a fixed link under and across the river. The decision was based on promises done by the Dutch Government, that was asked to support the project.

The realisation and future control should be done by a limitid liability company. The province Sealand should hold 99% of the shares.

Income should be guaranteed by toll income. Further the Central Government promised to furnish the amount of money presently paid to cover the losses of the ferry connection in service nowadays. These losses are 70% of the operating costs. The Central Government should also furnish the amount of money necessary to realise a new ferry harbour in case no fixed crossing is realised.

Financial considerations required to have an impression of the financial risks. For this reason it was important to know the risk of a ship collision with the result no possibility to use the bridge and consequently no toll income. Together with insurance companies was looked for the costs to insure the risk, also is examined the advantages of an energy absorbing construction to reduce risks and possible insurance costs.

1.2 Location of the planned crossing



The location of the proposed bridge is in the South-West of the Netherlands across the Westerschelde estuary. The Westerschelde estuary is the only estuary which is not closed as a result of the Delta Works (These works have the purpose to defend the South-West of the Netherlands against the sea). Closing of this estuary by a dike is partly not possible and partly not allowed. Partly not possible, because the estuary is the entrance to the harbours of Antwerp, Terneuzen and Gent. Partly not allowed because the Netherlands promised Belgium an open connection with the sea in the past.

Fig. 1 Location of the bridge marked on the map of the Netherlands

1.3 Situation of the location

The location of proposed crossing has two shipping lanes. The main lane called the 'Zuidergat' and the minor lane called the 'Schaar van Ossenisse'. The minor lane is used by smaller ships to avoid busy traffic close to the locks of Hansweert, once the entrance of the bussiest canal of Europe. In the main lane big ships need relative high speed, because of the strong curvature of the lane at the location. Also for this reason the smaller ships choose for the minor lane. The plans for the crossing consist of a tunnel underneath the main channel and a suspension bridge across the subchannel. Selecting a tunnel has to do with the earlier mentioned open connection with the sea.

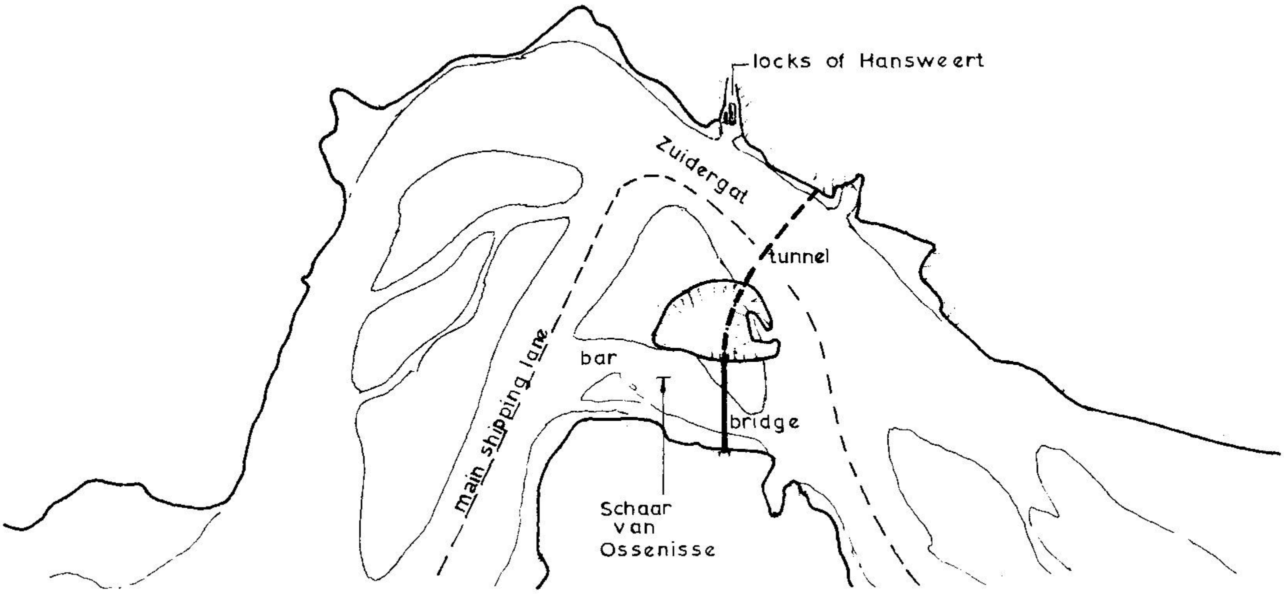


Fig. 2

1.4 General arrangement

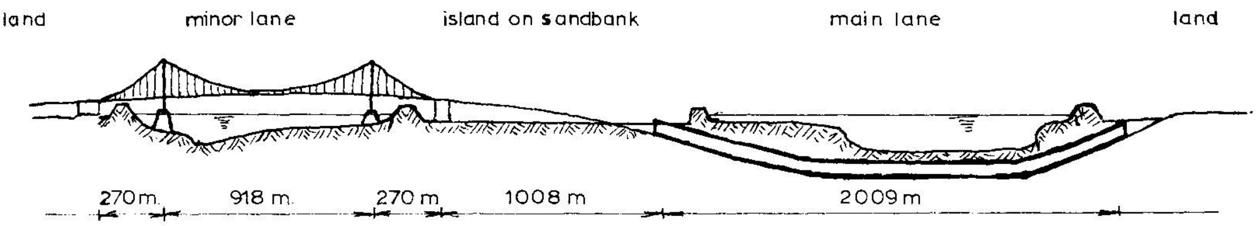


Fig. 3

To cope with the described conditions the general arrangement of figure 3 was developed.

1.5 Design

The Lock and Weir Department of the Ministry of Transport operates as the consulting engineering department for the tunnel crossing. The Bridge Department for the suspension bridge.

2. SCOPE OF THE STUDY

The Dutch Government has guaranteed in the past the Belgium Government a free connection with the sea. Free connection means also free clearance in hight. For this reason was the only possible solution a more expensive tunnel underneath of the main shipping lane. For the other less important shipping lane the link can be realised by a bridge.

With this design we got a rather unique situation. The bridge across the minor lane does not require a big clearance. Critical is the situation of a low bridge with big ships passing through the main lane very close to the bridge.

The first idea about the design was a bridge on more supports. The water depth under the bridge varies between 2 - 12 metres. A pier protection for the smaller ships was felt necessary. To the smaller ships we had also to include push barges. In the future push barges can be built together to the number of 6. Nowadays is the number 4. The weight of 6 barges can be approximately 12,000 tons. For this reason a protection is mandatory. For the protection artificial islands were selected. It became clear that because of the equilibrium of the gullies a bridge with piers with artificial islands needs bigger spans. The area is very sensitive for distrubances. The tide moves mainly through the main lane (gully) and it has to stay that way, this because it is not possible to predict what the new equilibrium is.

Bigger main spans brought two types of bridges in view, namely the stay bridge and the suspension bridge. A stay bridge in this particular situation was not in favour. This because of the big ships in the neighbourhood. A collision with the stay bridge close to the pier means the lost of a big part of the bridge. This as a result of the axial force in the deck.

As result of the mentioned considerations one choose for a suspension bridge:
- big span means fewer piers.
- fewer piers results in less artificial islands which means little hydraulic disturbance.
- with a suspension bridge the piers can be located such that they are located in shallow water.
- the deck construction is not the main construction element in regard to strength of the whole construction. Damaged areas are relatively easy to repair.

After all these considerations one question remained unanswered. What is the chance with the big ships in the neighbourhood in the main lane of a collision with the bridge deck. The study undertaken was a risk analysis of the deck construction as designed. One was not only interested in damage yes or no, but also in the change of a certain level of damage. The possible damages were differentiated in classes. Smaller damages are acceptable for the exploitation of the bridge and bigger are not. To make clear which levels were choosen, first a description of the considered deck construction. A cross section is shown in figure 4.

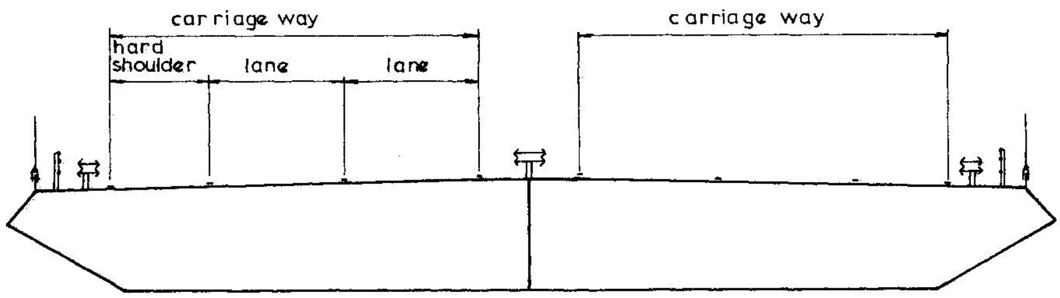


Fig. 4

The design of the crossing consists of a dual carriage way with two lanes and one hard shoulder in each direction.

The considered levels of damage are
1. scratch of dent in box girder, no consequences for the traffic
2. damage of the box edge, no consequences for the traffic
3. damage of the hard shoulder and one lane, delay in one direction
4. damage of one carriage way, delay in both directions
5. damage of total box girder, no traffic possible

3. THE STUDY

The study undertaken was concentrated on the risk values of the mentioned damage levels. To answer this it was also necessary to know what type ship or what type of collision gives what level of damage. The study is done for the bridge with the described general arrangement. Clearance in the middel of the main span is 19.935 m, near the pylons 16.067 m.

3.1 Causes

Damage of the roaddeck can only be caused by a ship which actually only can sail in the main shipping lane, because of height. The next two cases mentioned are recognized to be able to cause a collision with the deck.

a. accidently: a sea-ship of the main lane (tunnel lane) comes in the minor shipping lane (bridge lane) as result of
- a give way situation
- an accident
- a technical break down.

These situations can cause a collision if:
- it is not possible to stop in time or
- the captain thinks wrongly he can continue his trip through the minor lane.

b. wrong decision: the captain erroneously (tries a short cut) uses the minor lane 'het Schaar van Ossenisse' to reach his destination.

3.2 Institutes concerned with the study

The study is done by the Dutch Physical Laboratory TNO, the University of Delft and the Ministry of Transport (Rijkswaterstaat Bridges Department).

3.3 Method of investigation

The analysis is done by using the technique of fault tree analysis. This fault tree is built up with events which leads to the top event of a collision with the bridge. To enable the calculation of the change of the top event one must know the change of the basic events.
To know which basis events cause the top event a fault tree has to be constructed. The circumstances which have an influence on the chance of occurence of the basis event must be known.
Because certain circumstances have an influence on more events it is prefered to make a circumstance matrix of all the circumstances of influence on the fault tree.

3.4 Fault tree

3.4.1 Main fault tree

The main purpose of the study was to determine the odds of the top event e.g. a collision with the bridge. Being interested in different levels of damage there are actually more top events. In the fault tree we make also difference between a collision on the west-side and the east-side, because the circumstances are different for both sides. On the west-side the time in the tide is important. With low water a number of ships is not able to pass the bar in the sailing lane on the west-side. Also difference is made between a collision with the mast or the derricks or with the wheel house. This is done because a mast can break down before the total energy is absorbed.

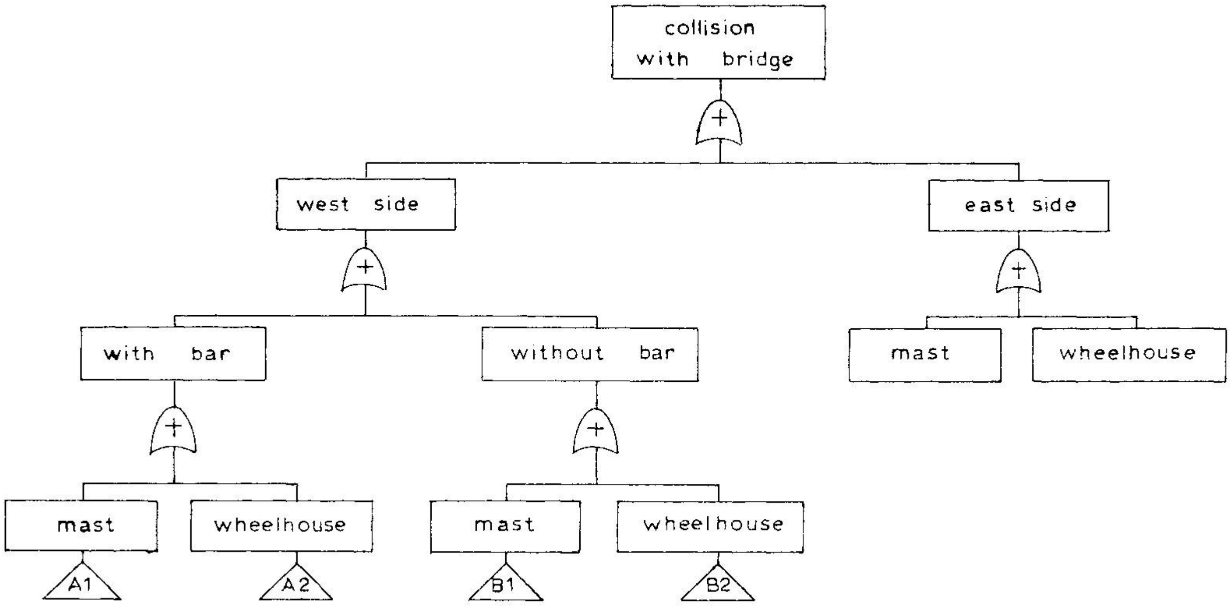


Fig. 5

3.4.2 Sub fault tree

The events, which cause the basic events of the main tree, are described with the sub trees A1, A2, B1, B2, C1 and C2. As an example fig. 6 describes a sub tree. In the subtrees A and B it is believed that a ship with a break down situation does not reach the bridge. This because the minor lane on the west-side is long and winding. The basic events are:

a. A sea going ship sailing in the main lane comes after an accident in the main lane in the minor lane as result of
   1. wrong human acting
   2. give way situation
   3. technical break down of steering equipment or engines

b. The captain thinks erroneously that he has sufficient head room to sail through the minor lane.

In a number of cases which can cause a collision it is believed that it is possible after realising the danger to make an emergency stop.

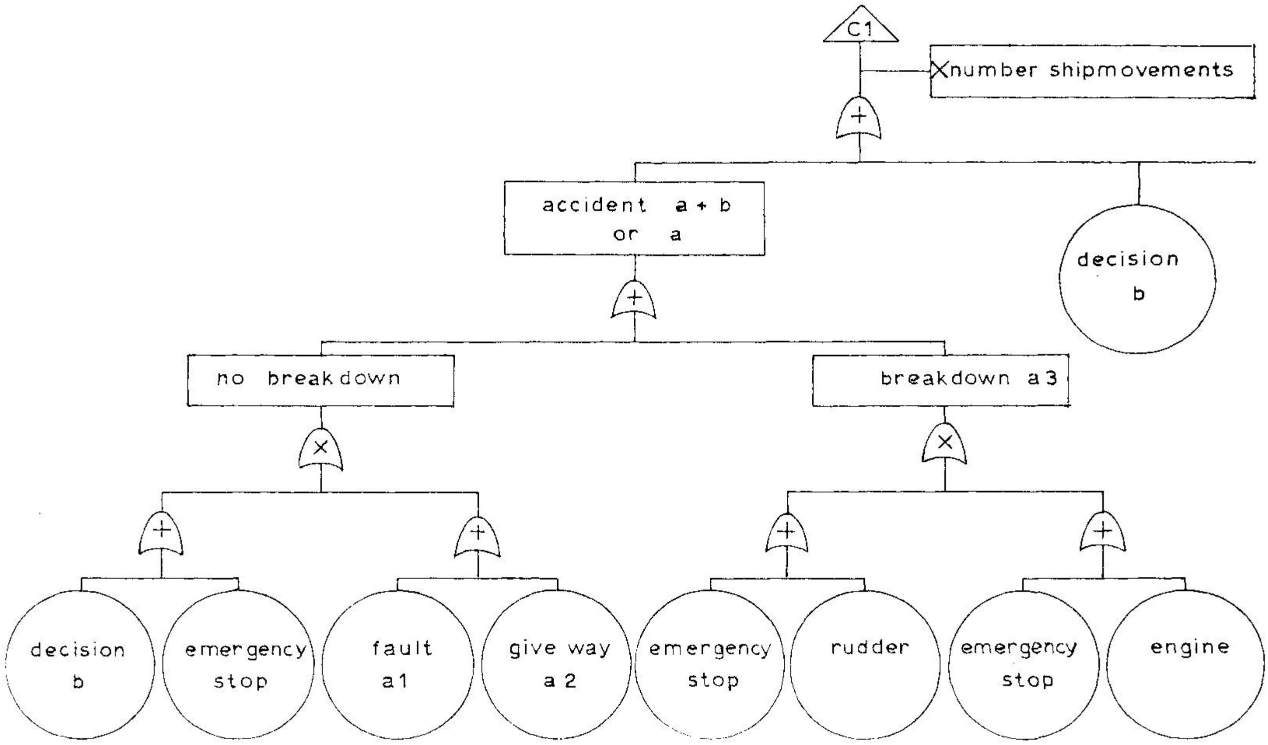


Fig. 6 Sub fault tree C1

3.5 Circumstance matrix

Circumstances of interest are:
1. type of sea going ship
2. presence of pilot
3. water depth in the lane (dependent of time)
4. day or night
5. visibility
6. weather conditions

3.6 Determination of the chance of occurence of a basic event

Chance of basic event = number of ship movements x frequence of accident

The number of ship movements is determined with the occurence matrix. The frequence of an accident is determined with information from the registration of ships which stranded. The frequence is determined by counting all the run on shore situations in the Westerschelde river and to devide them with the coast length (= 63 km). So we got the number of strandings by unit of length. The number must be multiplied by the length of the entrance of the minor lane.

3.7 Chance of top event of fault tree

The calculated chances of a collision with the bridge in the period of 10, 50 and 100 years, based on average expectation, are mentioned in tabel 1.

collision	period		
	10 years	50 years	100 years
with mast west	0.008	0.039	0.077
with mast east	0.040	0.185	0.336
with wheel house east	0.001	0.005	0.01
total	0.05	0.23	0.42

Tabel 1

3.8 Level of damage

To know the level of damage of a certain added energy we have to determine the penetration of the mast or the wheel house in the bridge deck. The penetration is calculated with the plasticity theory. The deck construction consists of a steel box girder with trough stiffeners and diaphragms. In a collision the side of the bridge acts like a membrane. The different levels of damage in which we are interested are mentioned in chapter 2. The necessary energy to cause these damages is listed below.

level 1	scratch or dent (by masts)	$E < 2$ MJ
level 2	box edge (not possible with strongest mast)	$2 \text{ MJ} \leq E < 13$ MJ
level 3	hard shoulder + one traffic lane	$13 \text{ MJ} \leq E < 53$ MJ
level 4	one carriage way	$53 \text{ MJ} \leq E < 90$ MJ
level 5	total box girder	$90 \text{ MJ} \leq E$

The change for the different levels is mentioned in tabel 2.

	total	level 1	level 2	level 3	level 4	level 5
mast west	0.077	0.031	0.046	-	-	-
mast east	0.336	0.134	0.202	-	-	-
wheel house east	0.010	-	0.008	0.002	0.0005	-

Tabel 2

The study included also an analysis of the advantages of an energie absorbing structure on the edge of the box girder.

4. CONCLUSION

On the bases of the results of this study the risks, in regard to a collision, were thought to be acceptable. For this reason the fender structure was not in favour. A fender is mostly an open structure and for this reason expensive in maintenance. The insurance companies gave no reduction on the premium in case of a fender structure.