Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 41 (1983)

Rubrik: Theme F: Design assumptions and influence on design

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Hypothèses de projet et influence sur la conception
Annahmen und Einflüße auf den Entwurf

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Design Assumptions and Influence on Design of Offshore Structures

Hypothèses de projet et influence sur la conception des constructions offshore

Entwurfs-Voraussetzungen und Einfluß auf die Projektierung der Offshore-Bauten

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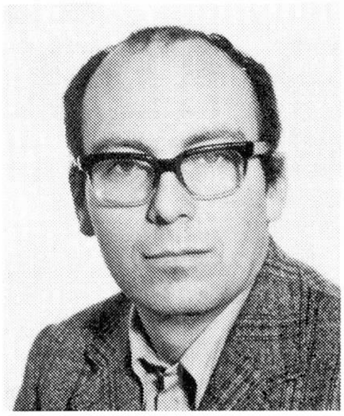

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SUMMARY

The paper reports on present design requirements as laid down by National and Certifying Authorities. Bridge pier design is relatively briefly dealt with whereas main attention is paid to design of offshore structures in steel and concrete. The basis for the requirements and assumptions is briefly outlined. The present design practice developed to meet the requirements is described with examples to illustrate the practical consequences for the design. These consequences concern all levels of design from the field development plan through platform concepts and member design to structural detailing. Platforms stabilized by buoyancy require a redundant system of water tight compartments.

RÉSUMÉ

Cette étude présente les conditions de projet couramment formulées par les autorités nationales ainsi que les organismes de contrôle. Le projet de la pile de pont est traitée brièvement; les constructions offshore, en acier et en béton, constituent l'objet principal de l'étude. Les hypothèses de base et les réglements sont brièvement décrits et les procédés et pratiques courants sont exposés avec des exemples illustrant les conséquences pratiques pour le projet des installations. Ces conséquences concernent toutes les phases du projet depuis l'étude générale jusqu'à la conception de la plateforme et l'étude détaillée des éléments de structure. Les plateformes stabilisées par flotteurs nécessitent un système surabondant de compartiments étanches.

ZUSAMMENFASSUNG

Der Artikel berichtet über die gegenwärtigen Forderungen zur Konstruktion von Seiten der inländischen Behörden und Prüfungsinstitutionen zu geben. Während der Konstruktion von Brücken begrenzte Aufmerksamkeit gegeben wird, konzentriert sich der Bericht hauptsächlich auf Offshorebauten aus Beton und Stahl. Die Grundprinzipien und Voraussetzungen der Forderungen werden kurz umrissen. Die Konstruktionspraxis, die sich zur Erfüllund der Forderungen entwickelt hat, wird anhand von Beispielen erläutert, um die praktischen Konsequenzen für die Konstruktion zu verdeutlichen. Diese Konsequenzen umfaßen alle Stufen der Konstruktion, von der Felderschließung zur Entwicklung von Plattformkonzepten und schließlich zur Dimensionerung und Festlegung einzelner Konstruktionsdetails. Die Plattform, die durch Auftrieb stabilisert werden, erfordern ein redundantes System wasserdichter Unterteilungen.

1. MAIN DESIGN PRINCIPLES

Design measures to avoid damage due to accidents such as collisions can be categorized as follows:

- event control
- indirect design
- direct design.

1.1 Event Control

The purpose is to reduce the probability and magnitude of the collisions. A majority of authoritative requirements to avoid collision damage to bridges seems to fall within this category. The main measures are beaconing, navigation restrictions and traffic monitoring. However, these measures often seem to aim at the safety of ships and efficiency of traffic rather than protection of bridges.

Offshore ship collision risk is often an important parameter for the oil field and platform layout. Even if not specified in any compulsory rules, the distance from the main platform to the offshore loading system is taken as 2400 m. The direction to the loading buoy is also chosen with a view to collision risk. The same applies to the general layout of the platform assembly at the field. Attention should be paid to confining the supply ships and possible floating work or accommodation platforms and barges in areas where they cannot jeopardize neighbouring platforms. General ship traffic is forbidden within a distance of 500 m from the platforms which are fitted with warning signals, i.e. radar, radio, light and sound to reduce the risk from such ship traffic.

Event control is considered beyond the scope of this paper and not discussed further in the following.

1.2 Indirect design

The purpose of the indirect design is to improve the general behaviour of the structure in case of unforeseen external impact without direct considerations to anticipated collision scenario. All structural codes include requirements to obtain reasonably ductile and robust structures. Bridge design specifications are often found to require divided pillars to carry the bridge even if one element is broken. Indirect design seems to play a more important role for offshore structures, such as jackets etc., than for more massive bridge piers. Flexible offshore structures which are supposed to absorb considerable energy in accident conditions must be ensured to behave in a ductile manner. Measures to obtain ductility are:

- connections of primary members to develop a strength in excess of that of the member;
- redundancy in the structure so that alternative load distributions may be developed;
- avoid dependence on energy absorption in slender struts with non-ductile post-buckling behaviour;
- 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; and
- non-brittle materials.

1.3 Direct Design

Direct design means to check the behaviour of the structure when exposed to some prescribed deterministic collision.

This paper will mainly address the direct design and associated requirements to the indirect design.

2. GENERAL STRUCTURAL CODES

Code requirements to direct design against accidental loads have been recommended in international model codes [1], [2], [3] and have eventually been introduced in the majority of new national structural codes. Ship collision against bridge piers or offshore structures is a typical example of accidental loads to be covered by these requirements.

The national design code formats for design against accidental loads are not completely uniform. Some codes require these loads to be checked in their ultimate limit state whereas others have introduced a separate limit state of progressive collapse. In the latter limit state, local structural damage is accepted provided collapse of significant parts of the structure is avoided.

Tables 1 and 2 record load and material coefficients required for design against accidental loads according to different structural codes.

In cases where local damage to the structure is accepted, rational design against ship impacts will normally encompass a check of the plastic deformations necessary to absorb the impact. In this check, restrictions to the acceptable plastic strain will be a more meaningful measure of safety than a traditional material coefficient [22] .

Load coefficients	ECCS Steel structures	CEB-FIP Concrete structures	CP 110 Concrete structures		
Dead load	1.1 – 0.9	1.1 – 0.9	1.4	0.9	1.2
Live load	1.0[1)]	1.0[1)]	1.6[1)]	0	1.2[1)]
Environmental load	1.0[1)]	1.0[1)]	0	1.4[1)]	1.2[1)]
Accidental load	1.0	1.0	1.05	1.05	1.05
Material coeficients					
Structural steel	1.0 – 1.12	–	–		
Reinforcement steel	–	1.0	1.0		
Prestressing steel	–	1.0	1.0		
Concrete	–	1.3	1.3		

[1)]Characteristic values reduced by factors taking into account the probability of simultaneous occurence

Table 1. Loads and material coefficients to be used for accidental combinations. General structural codes.

Load coefficients	Norwegian Petroleum Directorate[3]		Det norske Veritas[3]	FIP Concrete Structures		
	Accidental load included	Accidental load not included				
Dead load	1.0	1.0	1.0	1.2	1.1	0.9
Live load	1.0	1.0	1.0	1.6	1.3	0.9
Environmental load	0	1.0	1.0	1.4	1.3	1.3
Deformation load	0[1] – 1.0[2]	0 – 1.0[2]	0[1] – 1.0[2]	1.1	1.1	1.1
Accidental load	1.0	0	1.0	1.05	1.05	1.05
Material coefficients						
Structural steel	1.0	–	1.0	–		
Reinforcement steel	1.0	–	1.0	1.0		
Prestressing steel	1.0	–	1.0	1.0		
Concrete	1.1	–	1.1	1.3		

1. Indirect effects
2. Direct effects
3. Local damage accepted

Table 2. Loads and material coefficients to be used for accidental combinations. Codes for offshore structures.

3. CODES FOR BRIDGES AND OFFSHORE STRUCTURES

Bridge codes cover ship collision to a limited extent only. If at all covered, a design ship impact is seldom specified in terms of energy, impulse or force. This reluctance seems appropriate as the actual impacts will inevitably depend upon the local conditions and possible collision control measures. The design impacts required are normally expressed in terms of general philosophies, see for example the Nordic Road Federation Loading Code [11].

It has been attempted to express design collision criteria in probabilistic terms. Based on different reasoning [4] [5] and [6], all estimate an annual probability of 10^{-4} to be a feasible basis for the design impact. As the data necessary to estimate the corresponding impact are normally non-existent, this number mainly seems to include a principle to be aimed at. In practice, sailing restrictions imposed by the authorities or nature will form the basis for the choice of design impact.

Specific design rules are often given in appendices or comments to the codes. For large bridges, special design criteria will normally be prepared, in each case, based on a rational evaluation of the ship traffic at hand. Different criteria will often be implemented for the navigation span as compared to the sidespans. However, this practice might be questioned inasmuch as severe collisions have occurred a long way from the navigation span. Saul and Svensson [12] have recorded 18 major collision disasters, 13 of which concerned the sidespans and only 5 the main span.

Table 3 records the design impact forces as implemented by a selection of authorities and used for certain large bridge projects worldwide. In current design practice, the design impact is normally expressed in terms of some specific force whereas criteria for large bridges and offshore structures are expressed by ship magnitude and velocity.

Governmental authorities responsible in the North Sea and Veritas only have rules for collision resistant design of offshore structures. Their design practice is meant to cover an impact from a freely drifting supply vessel. The corresponding energy required is absorbed by deformation of the structure and the ship. An exception is the U.K. guidance [9] which specifies a very low impact velocity and requires all the impact energy to be absorbed by the structure. This requirement might lead to inferior designs of weak, flexible platforms (e.g. jackets) compared to stiff and strong platforms which will not become deformed if exposed to forces exceeding the strength of the ship.

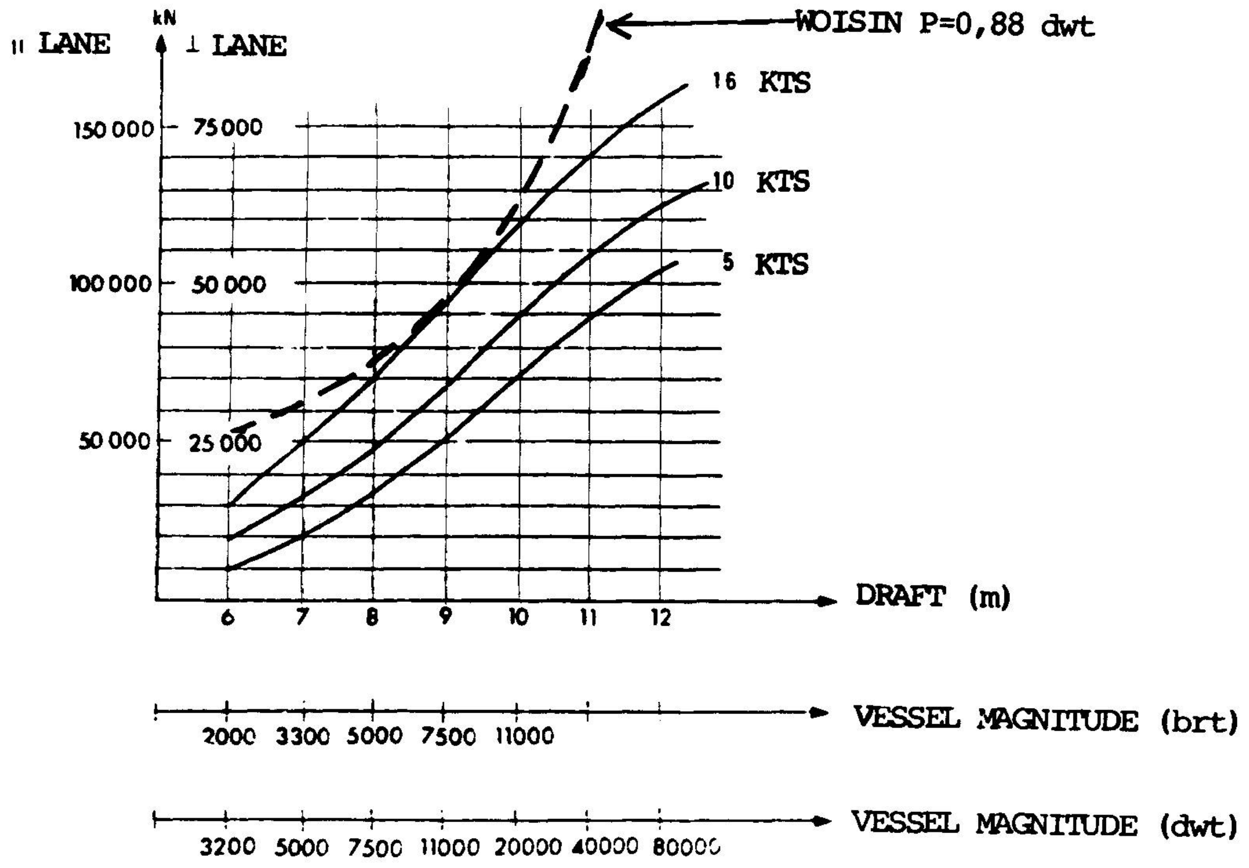

Fig. 1. Nordic Road Federation. Recommended impact force.

	Ship size Displacement if not otherwise specified	Ship Velocity	Design force	Procedure for force estimate, comment
Bridge codes and practice Nordic Road Federation Design load code (11)				see Fig. 1
Current practice small bridges:				
Norway			⊥ 3000 t ‖ 1500 t	
Sweden			1000 t	
West-Germany Roads			⊥ 2000-6000 t ‖ 1000-2000 t	
Deutsche Bundesbahn (7)	(1800 t)	(5.88 m/s)	3000 t	
Offshore structures codes and practice				
Danish shelf (8)	>2500 t	>2 m/s		Energy consideration
UK shelf (9)	2500 t	0,5 m/s		All energy to be absorbed by platform/fendering
Norwegian shelf (20)(10)(18)	5000 t 5000 t	2 m/s 0,5 m/s		PLS, energy considerations ULS
Bridge projects:				
Øland bridge, Sweden			5000 t	
Øresund Bridge, Sweden/ Danmark	50000 t	9.4 m/s	14200 t	
Great Belt Bridge Denmark				
Navigation span	250000 t (dwt)		44000 t	
Side spans	4000 t (dwt)		6000 t	
Western Bridge	1000 t (dwt)		2000 t	
Bahrein/Saudi bridge	20000 t	4.2 m/s	5600 t	
Luling Bridge USA	40000 t (dwt)	3.5 m/s	27000 t	
Okanagan Lake Floating Bridge Canada	1.135 t	2.25 m/s		
Zarata-Brazo Largo brige Argentina (12)				
Main spans	20000 t	2 m/s		
Secondary spans	10000 t	2 m/s		
Second Hobart Bridge Australia	10000 t			

Table 3. Ship impact assumptions

The combined drift and sway velocity of a freely drifting ship in waves of significant height H can be expressed as v (m/s) = $^1/_2$ H_s (m).

As weather restrictions for the operation of supply boats may be laid down in the operation manual of the platform, this formula may constitute a rational choice of design velocity.

The magnitude of supply ships in the North Sea seems to increase with time. The magnitude of supply vessels in the Veritas class is shown in Fig. 2.

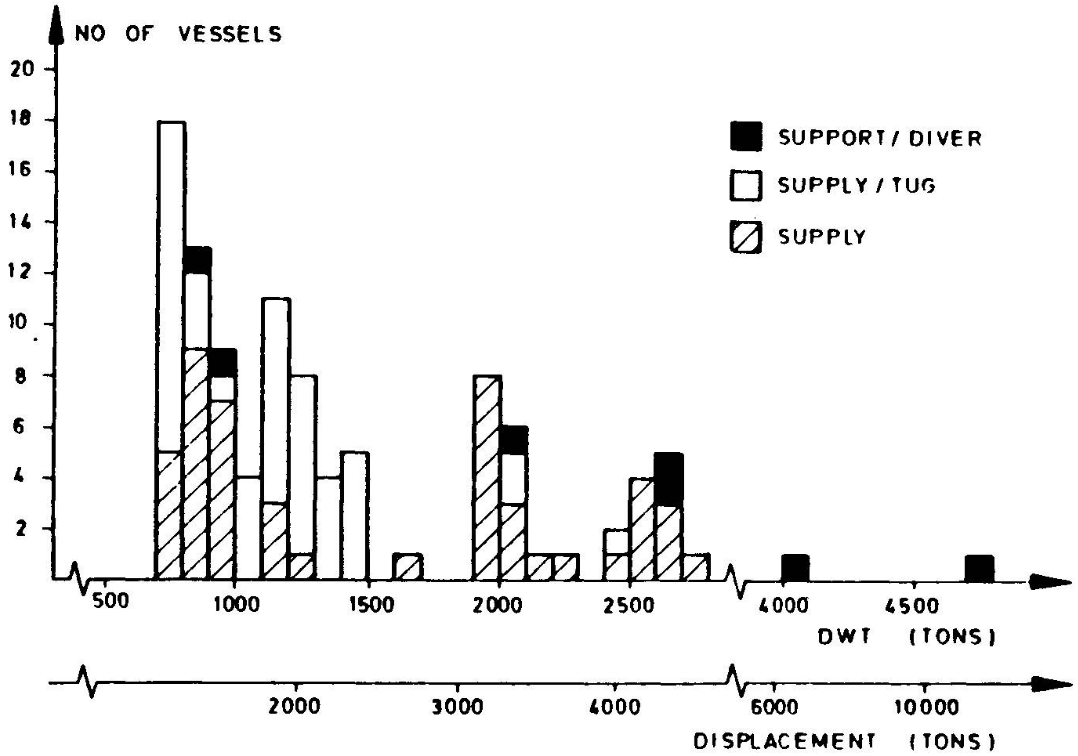

Fig 2. Supply vessels in the Veritas class - 1980.

Please note that according to Norwegian offshore rules ship impacts have to be checked at two levels. In addition to the accidental impact, the platform shall withstand an operational impact with V = 0.5 m/s. This is checked in the ultimate limit state, i.e. with normal safety factors.

Direct design control of bridges in itself usually address the resistance against the impact, and the bridge can be closed shortly after damage. Possible subsequent damage in the case of, for example, strong winds in repair periods will not normally have major consequences.

Also, offshore structures can be shut down after collision damage. However, the topside facilities will normally have a value of at least ten times the main load bearing structure. The production time lost in the period necessary to replace a completely wrecked platform will probably be several times higher than the total platform cost. Therefore, according to Norwegian rules, offshore structures are checked in two conditions [22] , [18] :

a) The ship striking the platform which experience plastic deformations, and
b) The ship will be removed but the platform will have been damaged as per a) and exposed to environmental loads corresponding to a recurrence period three times the anticipated repair time or at least one year.

In the offshore industry, the above-mentioned philosophy 10^{-4}/year being a feasible recurrence period of the design collision, would result in the conclusion that design should take severe ship impacts into consideration.

Ship traffic in the vicinity of offshore platforms may be of the following categories:

i) Authorized vessels servicing the installation
ii) Tankers for offshore loading in the area
iii) All other kinds of bypassing ships and fishing vessels.

Extensive collision risk analyses have been carried out on several platforms in the North Sea [16]. These analyses show that all categories of vessels can strike the platform with a probability of more than 10^{-4}/year unless an event control, as discussed above, is used to reduce it. A current practice is to design for the first category only, tacitly assuming such measures will be taken.

4. DETERMINATION OF IMPACT FORCES

Any rational determination of the impact load should be based on the following basic laws of physics:

a) force equilibrium
b) energy conservation.

The impact force can then be determined by the simple principle shown in Fig. 3, considering the condition.

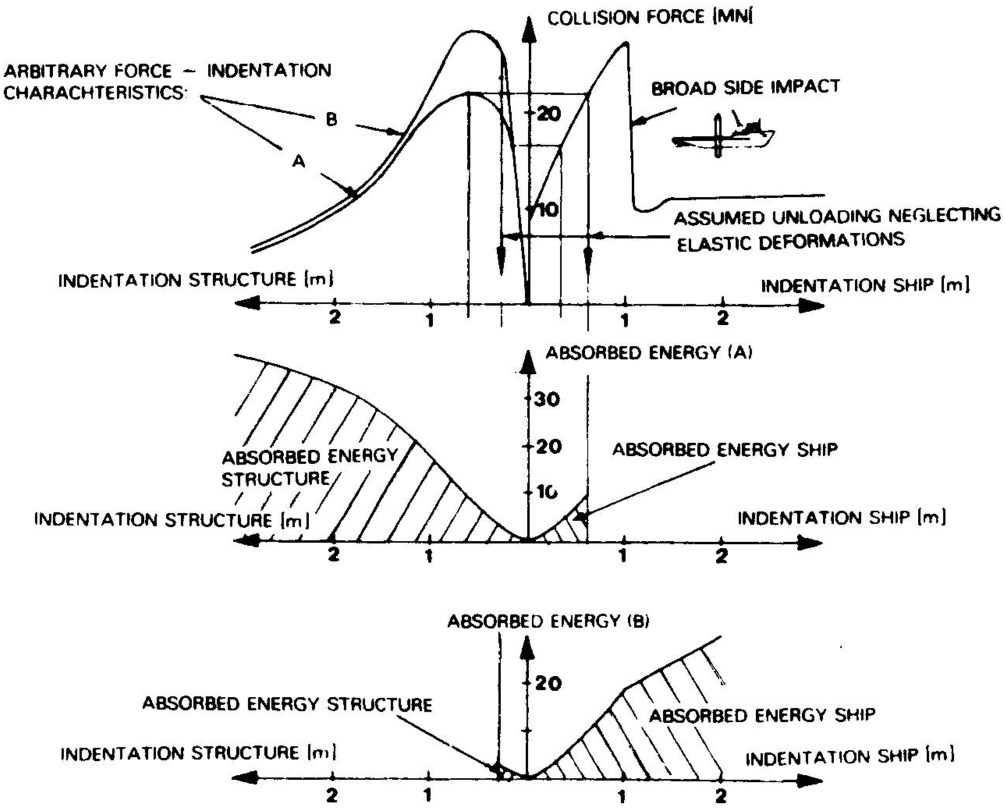

Fig. 3. Energy distribution ship/structure.

$$^1/_2\ (m + \Delta m)V^2 = A_p + A_s$$

m = ship mass
Δm = added mass
V = ship velocity
A_s = energy absorbed by ship
A_p = energy absorbed by structure

Any other basis for determination of impact forces should be rejected. If the impact is not centric, part of the energy will be lost in the rotation of the ship. However, unless specific measures have been taken to exclude the possibility of centric impacts, current practice has been to assume the full kinetic energy absorbed by ship and structure.

Depending on the dynamic properties of the structure, A_s should include the elastic energy absorption. Mainly, this contribution seems to be of importance for relatively flexible jacket structures. Rigid piers and concrete structures seem to have a small elastic energy contribution.

The water mass Δm to be added to the mass of the vessel to determine the design impact has been subject to discussion. This mass will be significantly dependent on ship configuration, water depth, impact velocity, etc. [31] .

As far as supply ships are concerned, investigations [17] show that within the range of current designs the added mass coefficient can be considered constant for a given ship in deep water. The added mass coefficients of [10] 0.4 sideways and 0.1 longitudinally seems confirmed. For an 11,000 t barge, the coefficient sideways is found to be 0.18.

Several procedures exist to evaluate the impact force on the basis of a given ship size and velocity. The classical Minorsky procedure [13] is revised by Woisin [14] and has gained widespread applications. Bridge piers are normally assumed to be infinitely stiff compared to the ship. Under this assumption, it has been found possible to express the maximum force [14] by the following rule of thumb:

$P_{max} \simeq 0.88$ dwt $\pm$ 50%

where dwt is the carrying capacity of the ship. The mean force is reported to be approximately 0.5 P_{max} (Fig. 4).

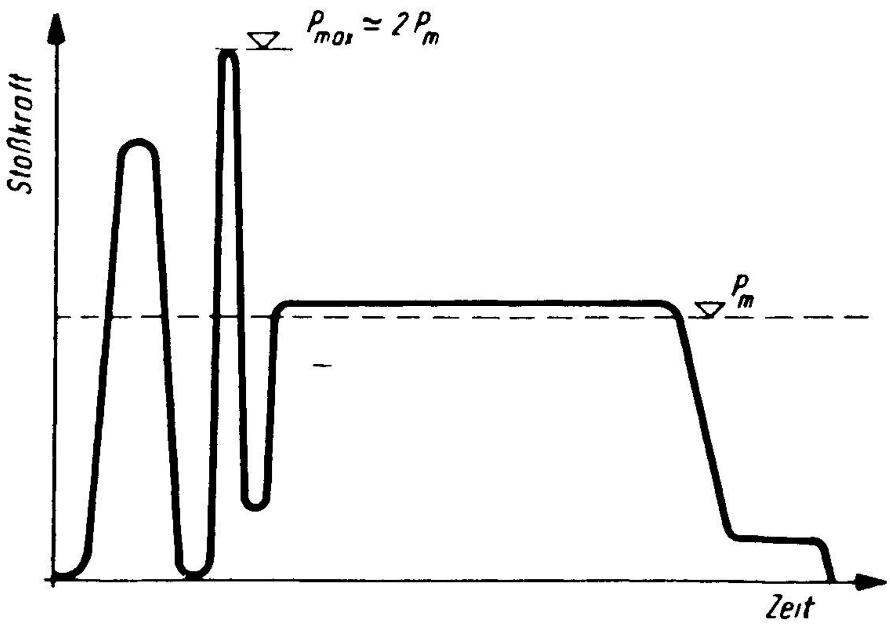

Fig. 4. Time-force relation for ship impacts [14] .

This procedure is based on model tests with passenger liners, tankers and container ships of sizes up to 195,000 t [15]. Veritas have carried out comprehensive analyses of the force indentation characteristics of current types of ships by the application of a computer programme NVINDENT [24]. The load deforming the hull is determined by accounting for the membrane force of the ship side, deck and bottom and the plastic buckling load of frames contributing during indentation.

Figs. 5 to 8 show the results for 5000 t supply ships and also a side impact on a 150,000 t tanker. These curves concur with Fig. 4 for broadside and stern impact of the supply ship. The impact of the supply ship bow and tanker broadside demonstrates different characteristics. The reason for this difference might be that Woisin apparently applies the result from impact tests where one ship rams another. Veritas have assumed an infinitely stiff obstacle. The assumption concerning constant resistance should, therefore, be used with some care for ships ramming a stiff body. Further research to achieve reliable force/indentation curves seems necessary.

In Fig. 1, the Woisin formula [14] is shown together with the Nordic Load Code recommendations [11].

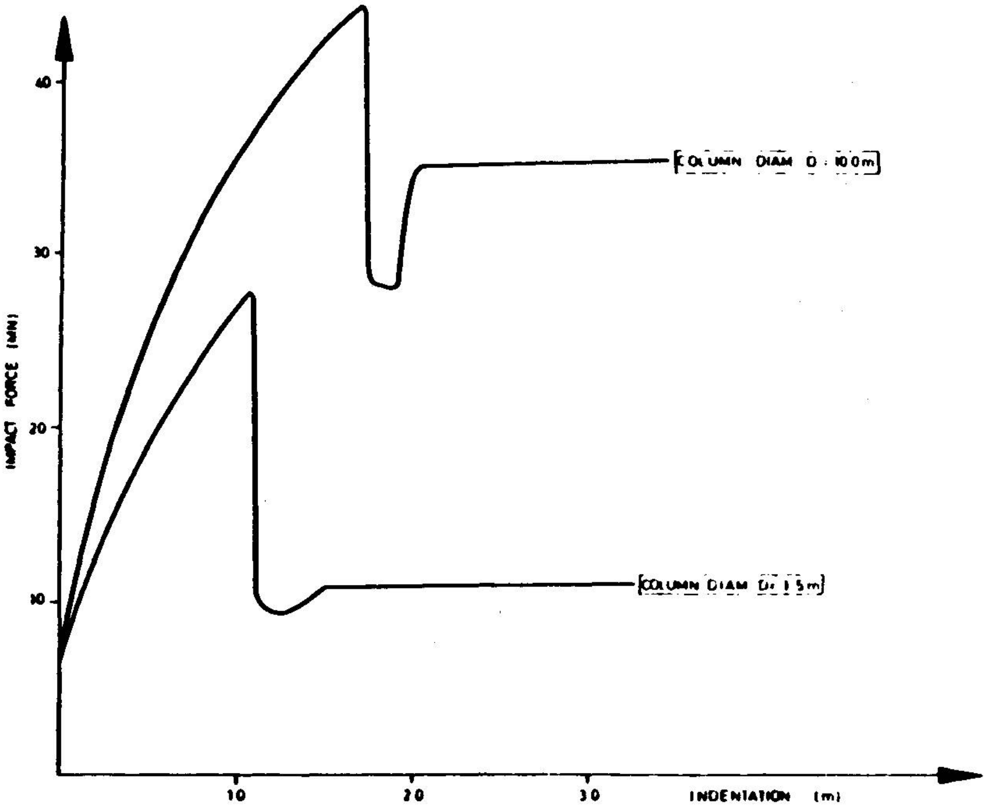

Fig. 5. Force/indentation curve for broadside impact with infinitely stiff cylinder. Boat displacement 5000 t.

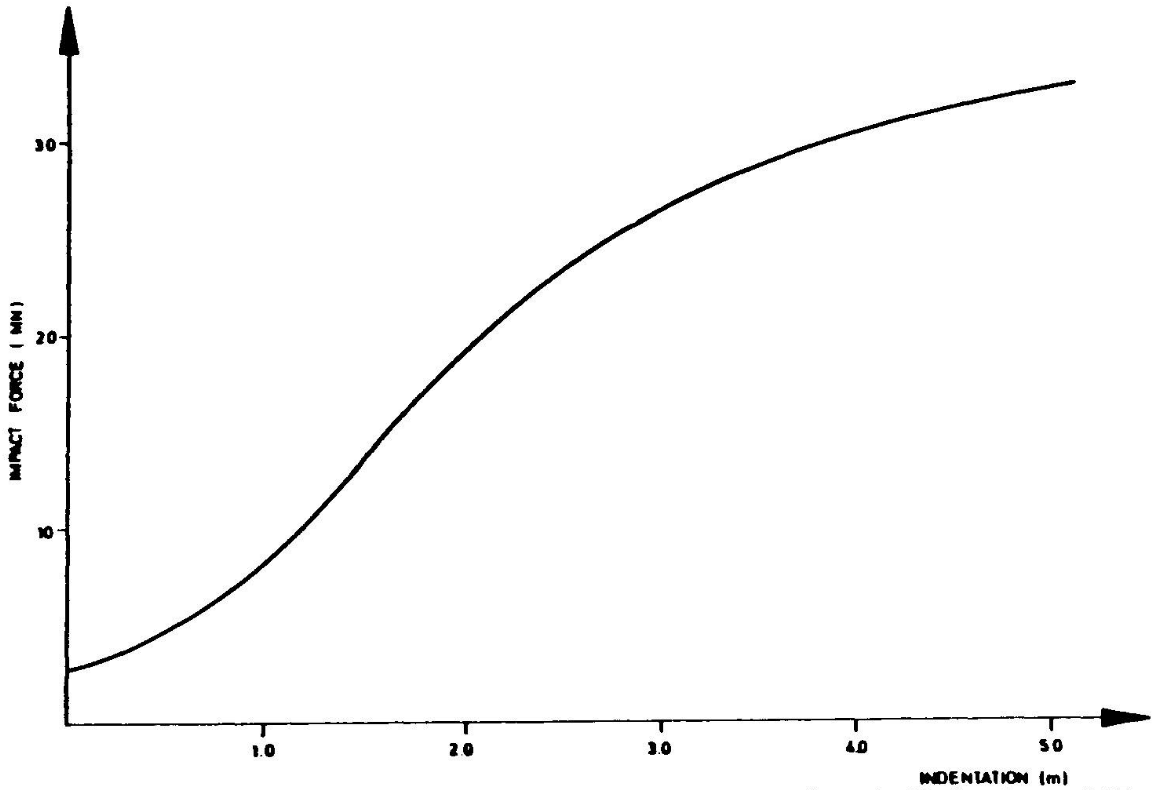

Fig. 6. Force/indentation curve for bow impact with infinitely stiff cylinder. Boat displacement 5000 t.

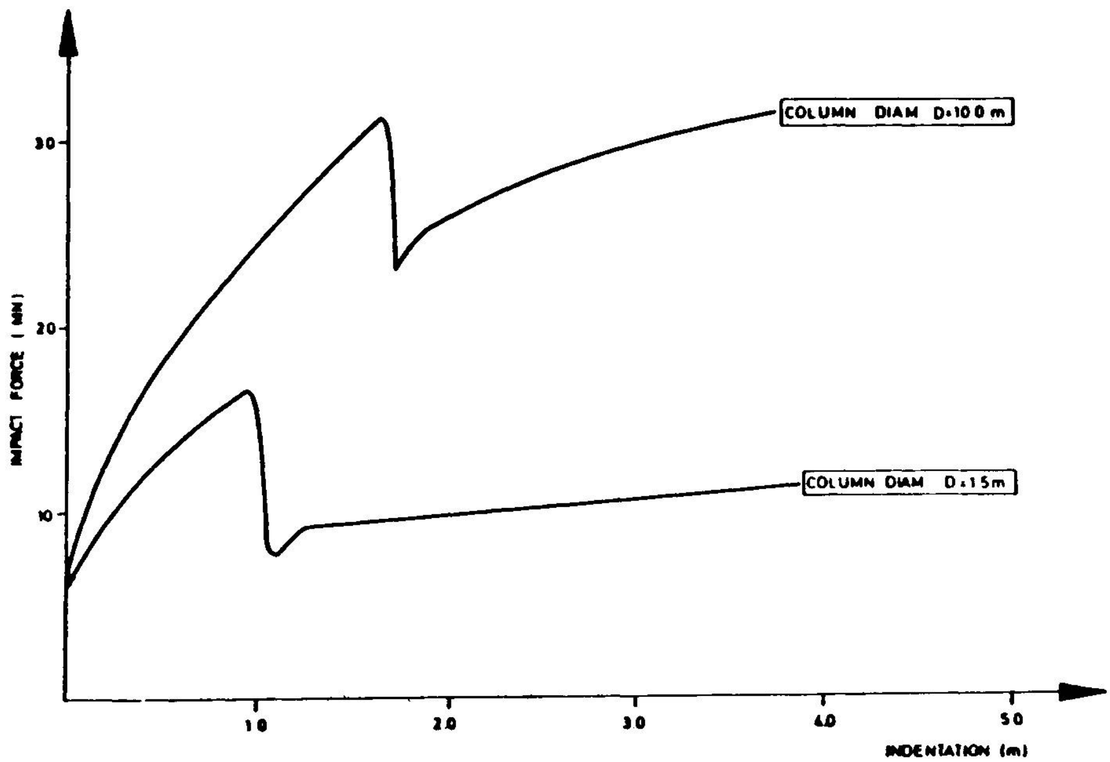

Fig. 7. Force/indentation curve for stern impact with infinitely stiff cyliner. Boat displacement 5000 t.

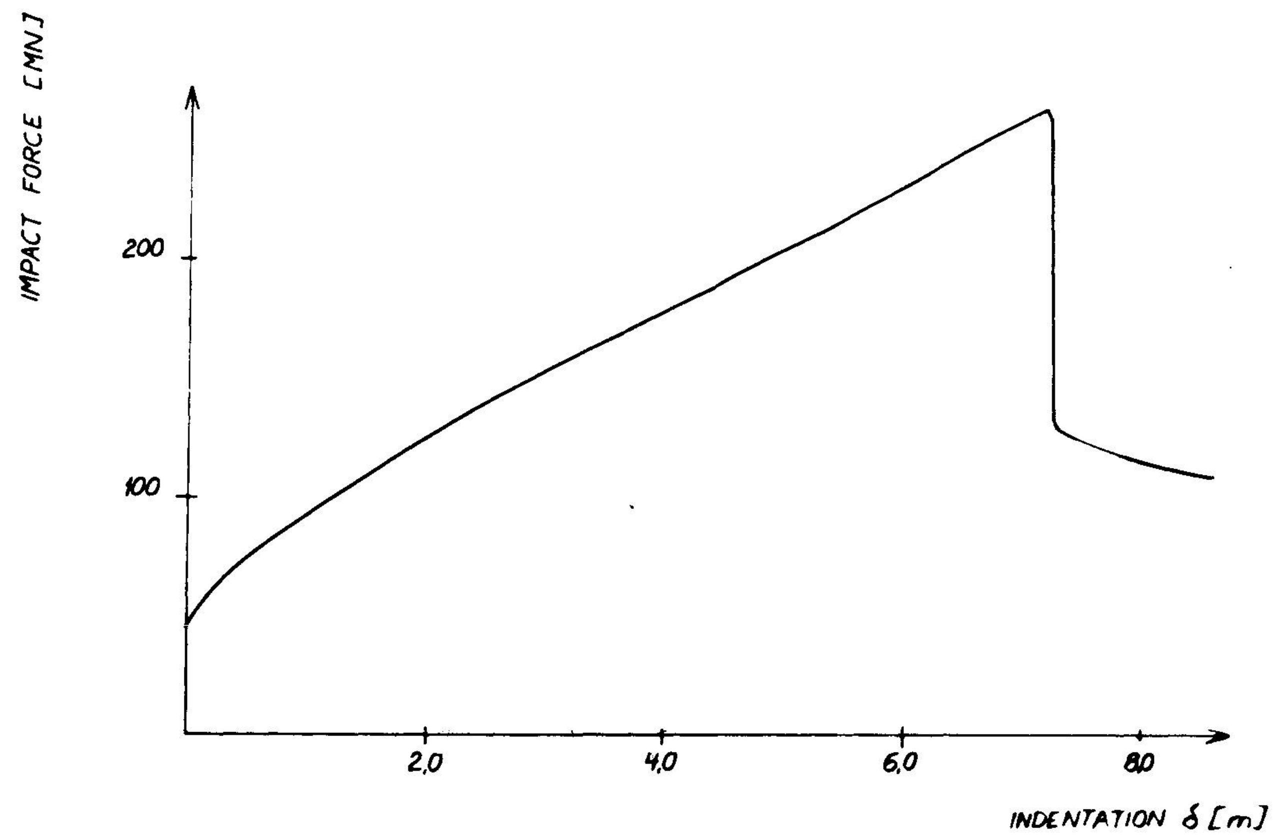

Fig 8. Force/indentation curve for broadside impact with infinitely stiff cylinder. Boat displacement 150,000 t. Diameter 13.3 m.

5. DESIGN AGAINST SHIP COLLISION

5.1 Bridges

A wide variety of pier protection methods exist, such as:

a) strengthening the pier to resist pertinent impacts,
b) protective embankments,
c) protective structures,
d) conventional fendering,
e) protective piles,
f) protection caisson,
g) floating anchored protection.

In normal cases the piers will be given sufficient strength and weight to resist pertinent impacts, often in combination with protective structures and fendering. The piers are given a configuration from which the vessel will tend to glance off without centric impact.

An excellent assembly of examples are given in [23] .

5.2 Offshore Concrete Structures

Some concrete structures are fitted with perforated breakwater walls. Provided there is sufficient height, this wall normally has a large capacity to absorb ship collision impact and further verification seems superfluous.

Otherwise, attempts to fit the offshore concrete structures with fenders have not proven successful. Fenders tend to constitute voluminous and expensive structures attracting large wave forces and are vulnerable to weather damage. For these reasons, the operational impact, as well as the accident impact are normally directly resisted by the naked concrete shell.

The following considerations address a caisson-shaft-steel deck frame with concrete shafts of diameters less than 20 m. The procedures and conclusions for large bridge piers also seems applicable.

The shaft is normally assumed to be completely stiff in the force analysis. It may be shown that the ratio of duration of impact and the natural period of global and local bending of the shaft permit the impact to be considered a quasi-static load. Its magnitude can simply be calculated considering the area under the force/indentation curve of the ship.

In the case of collision with a supertanker, the frame energy absorption and motions might require a dynamic analysis. Experience from the analysis of several platforms in the North Sea indicate that an adequate global cross-section capacity to resist impacts from a supertanker (150,000 t), is easily achieved.

Considering local response, the size of the loaded area is important. Concentrated impacts from massive, pointed objects may cause chipping or crushing at the surface. It is impossible to describe, in detail, the corresponding states of stress. Unlike a slab, the cylinder carries a portion of the load by membrane resultants depending on the size of the loaded area. In our case, punching will govern small areas only; thus the membrane effect is neglected and the transverse shear force is simply taken as the total load uniformly distributed over the perimeter. Locally, this average value may be slightly exceeded in certain cases; however, the ship side will bridge a possible concentrated failure.

Plots of local bendings and normal forces in vertical and horizontal directions corresponding to a 5000 t supply ship are shown in Fig. 9 [21] .

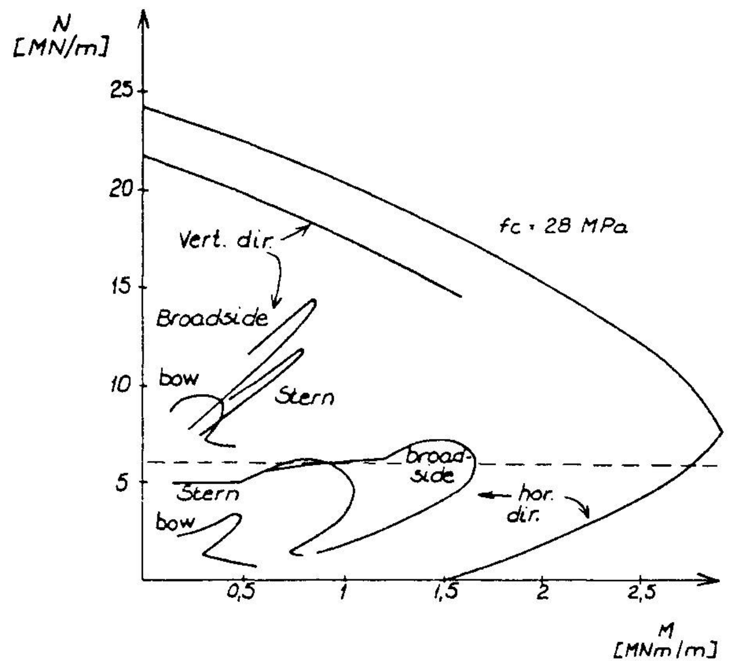

Fig. 9. Load effects and strenght of 65 cm thick concrete shaft.

In this figure, the interaction diagram corresponding to the ultimate limit state of a 65 cm thick wall is also shown. Similar large safety margins have been found for punching shear forces. The safety margins found, and the fact that the impact forces increase only slightly with the increasing impact energy, indicate that supply ships with velocity significantly more than 2 m/s can easily be taken by walls 0.6 - 1 m thick. The ship will fail completely prior to rupture of the wall. A strengthening to withstand the impact from a 150,000 t tanker requires a significantly thicker wall. A verification of this load case should be based on further studies of the post-failure behaviour of the concrete wall. As mentioned, the global frame could easily take this impact.

The punching shear strength of cylindrical shells have been subject to studies which indicate a higher strength than that for slabs. Several different formulae [25] , [26] , [27] have been developed, each on the basis of a somewhat limited number of tests. The authors have, therefore, introduced restrictions to the range of validity of their formulae.

However, the application of the formulae to a typical platform tower still shows a rather good correspondence. For loaded areas below 2 x 2 m punch strengths 20-40% above the values for a slab was calculated. The collision design consequences for the collision zone of a caisson and tower type offshore concrete structure will normally be:

- somewhat increased wall thickness,
- increased hoop and longitudinal reinforces, and
- the introduction of a significant amount of shear reinforcement.

For temporary phases during towing and construction, marine surveyors have required the structure to withstand collision with an offshore tug. In lieu of a more complete analysis, a design impact load of 2000 t over an area of 1 x 1 m is required. This load seems to be a reasonable estimate of the crushing resistance of the tugs. However, the load area appears rather small and has also resulted in an extensive amount of shear reinforcement in the lower parts of the concrete shafts.

5.3 Offshore Jacket Type Platforms

These platforms are normally fitted with boat landings for small crafts and often with barge bumpers which take the operational impact. Otherwise, fendering has not been found feasible for the same reasons as for offshore concrete structures.

A vessel can strike the jacket in several ways. Bow and stern are likely to strike both bracings and legs whereas a vessel drifting sideways is likely to strike the legs.

As discussed above, the design approach is to demonstrate the impact energy to be absorbed by strain of the ship and the platform. For this type of impact, the platform will often be the weaker part and consequently, have to absorb a significant part of the energy.

The main energy absorbing effects are:

- Elastic energy:
  In certain cases this energy can be considerable, however, it is normally a relatively modest contribution. A quasi-static analysis will normally do but should be subject to evaluation in each case. The estimate of plastic energy absorption should be based on a thorough investigation of the platform in a collision situation to identify the mode of deformation and whether a plastic behaviour can be developed as assumed. The main plastic energy contributions include:

- Local denting
  This contribution is indicated in Fig 10 [24] . In the case of thin walls the energy absorption is negligible. For thick walls, the tube tends to be stronger than the ship and will experience no plastic deformations. For intermediate thicknesses some energy will be absorbed but for bracings this contribution will still be negligible.

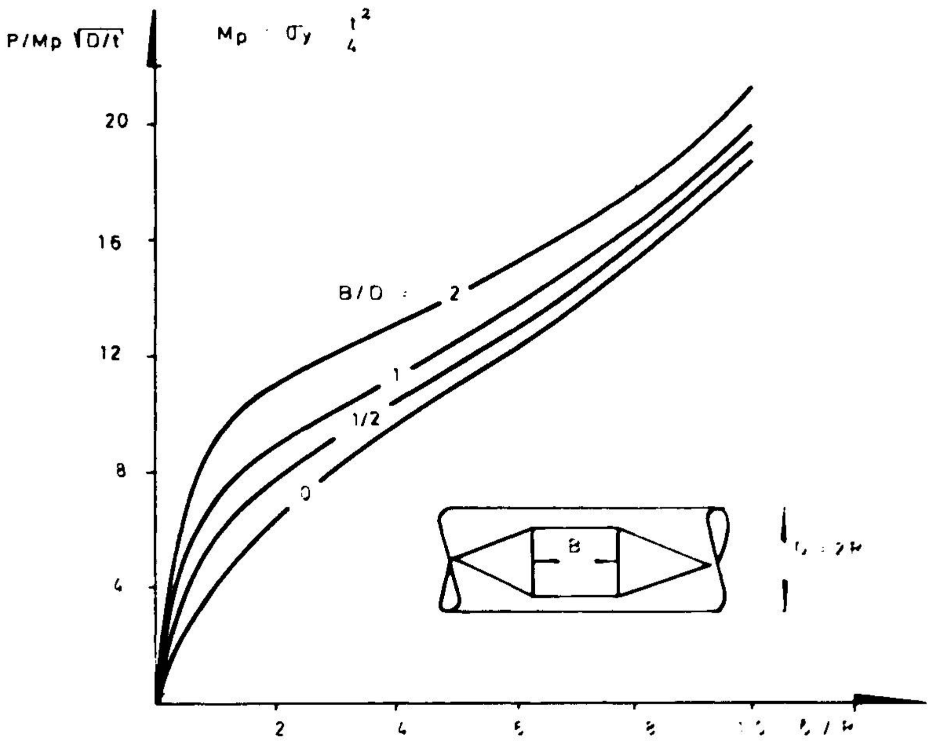

Fig. 10. Load indentation relation for steel tubes.

- Three hinge plastic mechanism
This energy absorption can only be mobilized provided several basic conditions are fulfilled. The American Petroleum Institute [19] states the following in their structural requirements.

"Tubular joints, members and piling at locations which are required to maintain their capacity through substantial concentrated, inelastic deformation should be designed to meet the compact section requirements ($D/T < 9000/f_y$). Portion of tubular members and piling which may only be moderately deformed beyond yield or column buckling need only be sized to preclude premature local buckling ($D/T < 22700/f_y$), provided their limited deformation capacity and degrading post-buckling characteristics are recognized. For tubular members with $9000/f_y < D/T < 15200/f_y$, development of full plastic load and moment capacity, but limited plastic rotation capacity, may be presumed".

(f_y in MPA)

These requirements seem to be very influenced by a paper by Sherman [23] with subsequent discussions [32], [33].

Even if these conditions are complied with, there are several factors modifying the energy absorption as presented in current textbooks:

- strain hardening
- ovalization
- local instability
- folding at ends
- incomplete restraining at ends.

These factors tend to reduce the energy absorption capacity as shown in Fig. 11 [29].

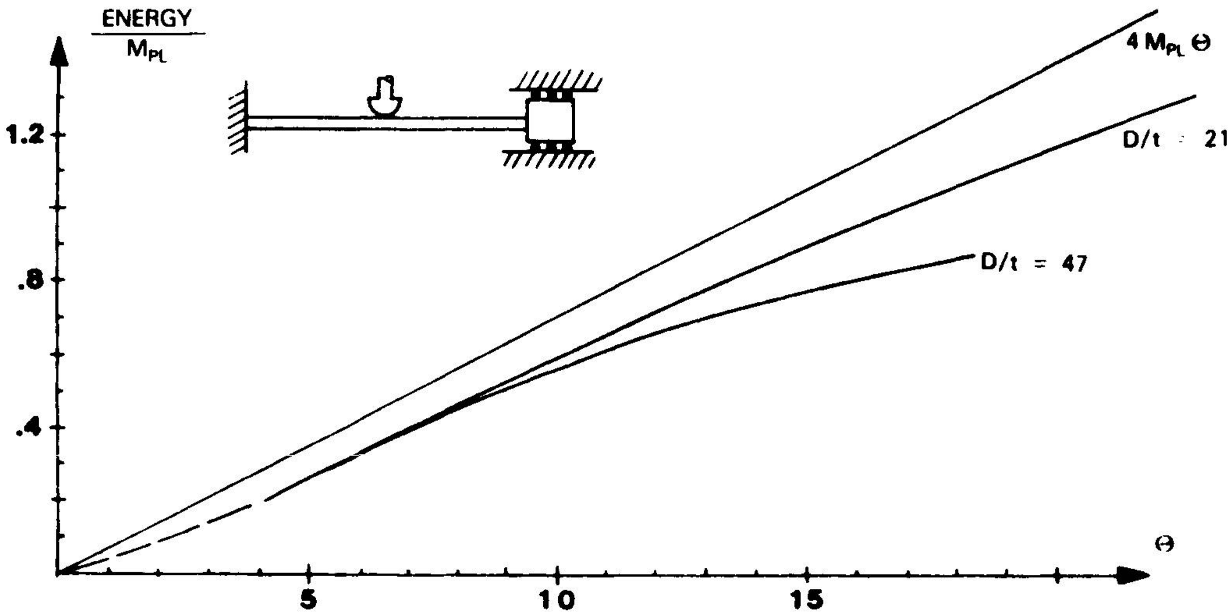

Fig. 11.- Plastic energy absorption in a three hinge mechanism.

Please note that these curves are based on a limited number of tests and should, therefore, be used with care.

If the member is exposed to a longitudinal follower load (e.g. a jacket leg) the energy contribution due to the displacement of this load has to be deducted from the three hinge mechanism energy developed to resist a lateral impact load.

Axial tension mechanism
This mechanism will normally be responsible for the majority of energy absorption. However, this also requires specific conditions to be fulfilled for its mobilization.

The whole length of the bracing has to participate. For this reason, the bracing or its joints cannot have any weak spots. Otherwise, the weaker area would yield when the main part of the bracing is still in the elastic range. Secondary bracing elements (conductor bracing, etc.) might confuse the development of the wanted mechanism. They can cause local ruptures or reduced strength at the connections with the brace member. It is to be checked that no section of the bracing member, including its joints, has a lower strength than the yield strength of the plain member. The ultimate punch strength of the joints should exceed this yield strength by at least 30%. Likewise, the adjacent structure must be demonstrated to have this strength. In practice, this is done by assuming that the impacted members are removed and forces corresponding to 1.3 times its nominal yield strength will be applied to the joints.

If these conditions are fulfilled the energy absorption given in Fig. 12 can be achieved.

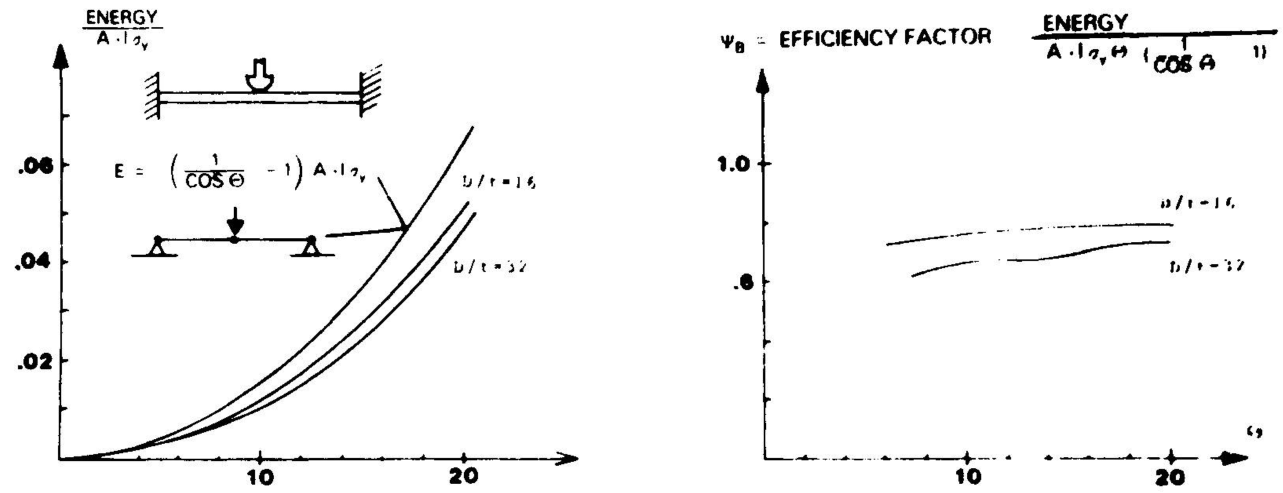

Fig. 12. Energy absorption in axially restrained members [29].

The relative contribution to the energy absorption from the different effects discussed is illustrated in Fig. 13.

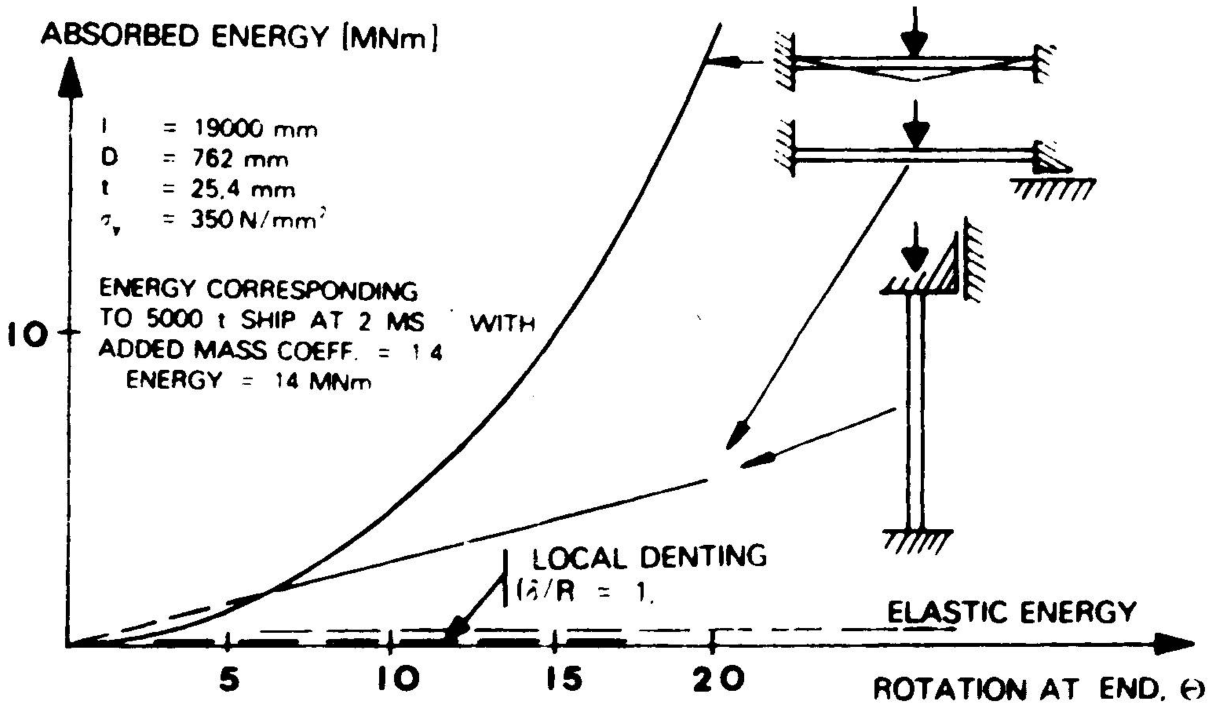

Fig. 13 Comparison of energy absorption capabilities [29].

The local denting and elastic energy seems negligible compared to the three hinge and axial tension mechanisms. Please note that the latter two energies cannot be added. The plastic moment will vanish as the member stresses turn into pure tension.

The analysis of the energy absorption capacity is the first step in collision design check. The next step is to verify the maintained bearing capacity of the structure when exposed to forces corresponding to the plastic deformations.

Finally, the deformed platform should be verified to withstand appropriate environmental conditions estimated for the repair period.

The introduction of requirements to collision resistant design of steel jackets have had some impact on the design practices and it is anticipated that more will follow.

Extreme lightweight jackets with cost savings achieved by static determinate structures were suggested in the mid-70's. So far, they have not had much success. It is assumed that future platforms will tend to be designed for structural redundancy in the impact zone. At present, K-bracings are popular, probably x-bracings will be used the most. A cleaner configuration to avoid local ruptures associated with the weak sections, appurtenances, etc. should be expected. Riser positions will tend to be changed to the inside of the structure.

The present trend seems to be increasing the strength of the jacket to exceed the strength of the ship and thus leaving all the energy absorption to ship crushing. This strategy should obviously result in increasing attention being paid to the real strength of relevant ship types.

On the other hand, the design practice outlined above might tempt the designers to design structures of low plastic resistance with large energy absorption capacity. Such designs might be vulnerable to damage even for minor impacts and should be avoided. For this reason, the requirement of the NPD and Veritas rules [10], [18] to have the platform checked in the ultimate limit state for a smaller impact (e.g. supply vessel with a velocity of 0.5 m/sec.) seems an indispensible safety requirement.

Finally, it should be mentioned that the design of current steel jackets to resist larger vessels is hardly possible. This fact accentuates the request for traffic control measures.

5.4 Buoyancy Stabilized Platforms

This heading covers a variety of different platform concepts, most of them hardly realized so far, i.e. articulated towers, tension leg platforms, catenary moored platforms, etc. These platforms are stabilized by buoyancy. Another common feature is their compliance with winds and waves, and relatively small resistance against horizontal motions. This small horizontal resistance might lead to the incorrect assumption that the platforms also comply with impacting ships and develop small impact loads only. However, as the impact duration (< 5 sec.) will be much smaller than the natural period of the platform in sway (40-150 sec.) no impact reduction is possible. On the contrary, the design should consider the possibility of a platform motion opposite to the vessel at impact which will result in an increased impact energy.

The maintained integrity for these platforms is expressed in terms of damage stability and floatability requirements [20], [30]. The damage assumptions and impact zone specified in these documents should be overruled by rational analysis of the likely damage as discussed above. On this basis, requirements to compartmentation of the buoyancy chambers and corresponding redundant buoyancy can be determined.

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Design Assumptions and Influence on Design of Bridges

Hypothèses de projet et influence sur la construction des ponts

Entwurfs-Voraussetzungen und Einfluß auf dem Brückenbau

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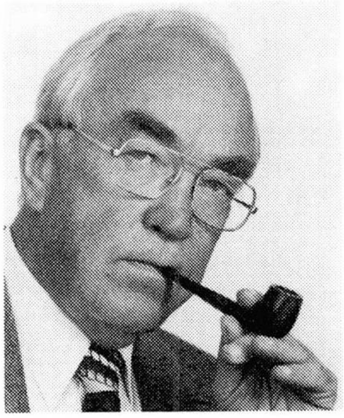

B. Højlund Rasmussen, born in 1918, trained at the Technical University of Denmark, where he obtained his civil engineering degree in 1941 and his doctoral degree in 1957. After some years on the staff of the Structural Research Laboratory of the Technical University of Denmark, he established his own consulting firm, which has now been designing bridges, mainly in Denmark for more than 25 years.

SUMMARY
The article discusses decisive factors in connection with the planning of a major bridge over navigated waters and describes the design procedure found most suitable by Danish Engineers.

RÉSUMÉ
L'article traite des facteurs décisifs dans la conception et le projet d'un pont à grande circulation enjambant une voie navigable et donne une description de processus de projet ayant été trouvé le mieux approprié par des ingénieurs danois.

ZUSAMMENFASSUNG
Dieser Artikel erläutert die in Zusammenhang mit der Planung einer Großbrücke über schiffbare Gewässer entscheidenden aufkommenden Faktoren und beschreibt das Bauverfahren, das von dänischen Ingenieuren für das am meisten geeignete,gehalten wurde.

1. DESIGN ASSUMPTIONS

Let us assume that a bridge over water can be divided into n members that are so important that a collapse of one of them would break the connection. Let us, initially, also assume that these members are the bridge piers and that we know the following characteristic quantities:

C_i: The collision force that is just sufficient to produce failure or inadmissibly big deformations.

If the pier is not rotationally symmetrical about a vertical axis, this force will depend on the angle with the bridge line at which the colliding ship hits the pier

We will define C_i as the maximum force which the pier can resist when hit centrally at right angles to the bridge line.

N_i: The number of ships passing each year which are able to exert a collision force $\geq C_i$ on the pier.

Important contributions to evaluation of the collision force which a ship can exert on a pier have been made by Minorsky [1], W. von Olnhausen [2], Woisin & Gerlach [3], Frandsen & Langsø [4], and Saul & Svensson [5].

Readers are also referred to Theme C, and need here only be reminded that this occurs as a consequence of energy exchanges during which the contact pressure between pier and ship wholly or partially stops the ship.

The maximum value and duration of the contact pressure thus depend on the weight, speed and "hardness" of the ship and the design of the bridge, and the values mentioned should, in principle, be found by means of a dynamic analysis.

Such an analysis will also provide information about the forces that will be transmitted to the superstructure during collision with a pier.

We will, however, imagine that all N ships navigating the waters crossed by the bridge can be characterized by a capacity C, which indicates the contact pressure that occurs when a ship sailing at its normal speed hits a stationary pier at its centre line.

We can then produce a curve N(C), showing how many ships with a capacity $\geq$ C pass the bridge each year (fig. 1).

On this curve we can read N_i, which naturally decreases with increasing C_i.

The curve in fig. 1 can be produced on the basis of information on the ship traffic in the years before construction of the bridge and forecasts for the development of traffic. Here we will imagine that it represents a probable situation in the middle of the anticipated lifetime of the bridge.

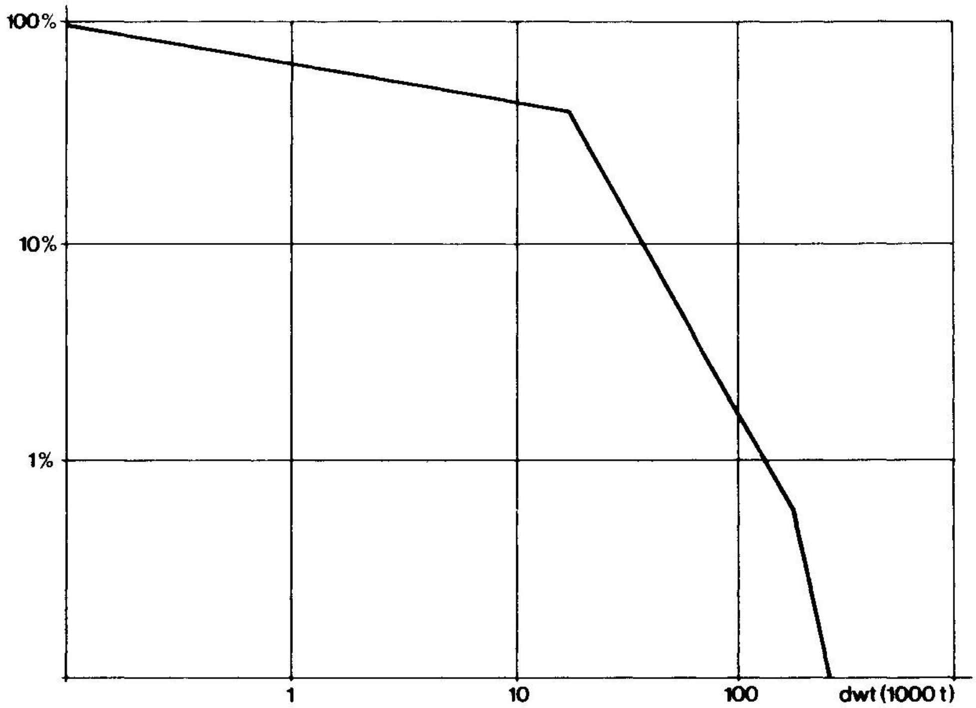

Fig. 1 Distribution of ship sizes in the Storebælt. The distribution has been forcasted to the year 1990. Example on use: 1.5% of all pasing ships are bigger than 100,000 dwt. (From [4]).

Studies of the conditions in Storebælt showed that related values of N_i and C_i lay close to a straight line when depicted on double logarithmic paper, Ↄ:

$$\log. N_i \sim \log. a_i - b_i \log. C_i \text{ or}$$

$$N_i \sim a_i C_i^{-b_i} \qquad (1)$$

The curve produced from (1) may possibly be replaced by several curve segments to approximate better the observations and expectations, but in the following we will assume that the constants a_i and b_i in (1) are known in the area in question and that they give a reasonable evaluation N_i for the pier under consideration.

The uncertain factors relating to the determination of a_i and b_i are at any rate far smaller than those involved in the evaluation of the next concept.

$\underline{p_i}$: The probability of one of the N_i ships colliding with the element and exerting a collision force $\geq C_i$.

It is obvious that $p_i = 0$ if the pier in question stands in such shallow water that the ship under consideration draws too much water to reach it.

However, if there is a theoretical possibility of contact between the ship and the pier, then there will be some probability of this occurring.

In order to find the magnitude of this probability a probability model must be established which takes into account the distance of the element from the prescribed channels, prevailing wind, current and other navigation conditions.

The model employed in the case of the Storebælt Bridge was formulated by the firm, Cap-consult A/S, Copenhagen, and assumes that a given fraction of the ships passing the bridge will be out of control. The probability of this is called the causation probability p_c and, is evaluated by T. Macduff [6] and Y. Fujii [7], at 2×10^{-4}. This causation probability covers both human and mechanical failure. In the case of the Storebælt navigation channel the causation probability was evaluated at 0.4×10^{-4} to suit the special conditions applying in this waterway.

By estimating how the ships will move after getting out of control (see fig. 2) it is possible to calculate for each pier a geometrical probability of collision with one of them.

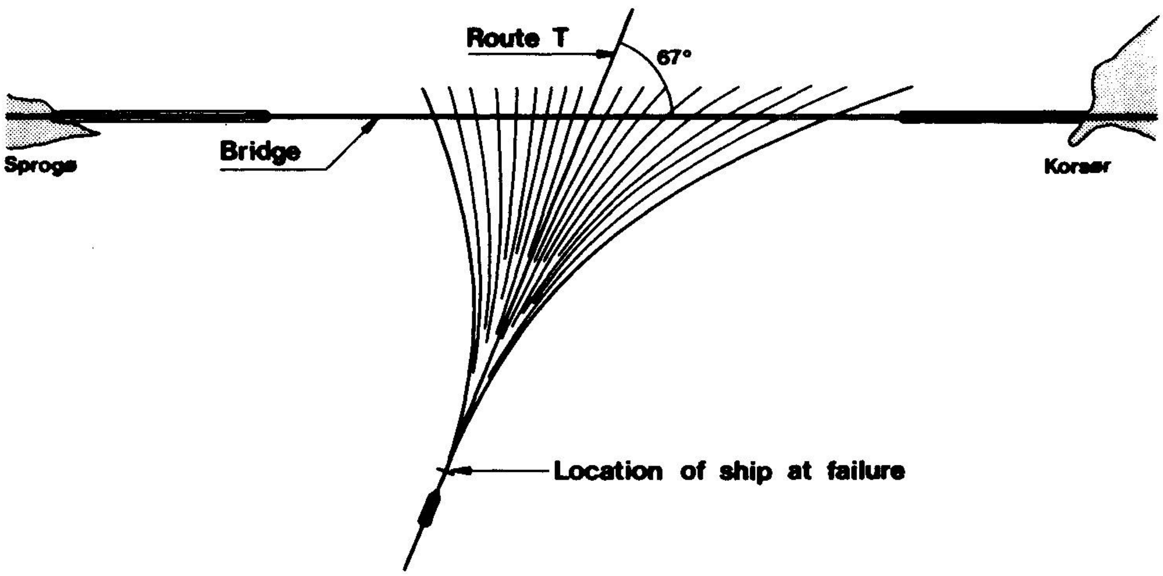

Fig. 2 Ships out of control: estimate of possible courses towards the bridge. (From [4]).

If, in our estimate of the movement of the ships having regard to wind and current, we can also incorporate an evaluation of their speed as a function of the distance from the point at which they got out of control, and if we know the reduction of the impact force that takes place when the collision is not central, we can, finally, calculate a resultant geometrical probability $p_{G,i}$ of the impact force exceeding the capacity of the pier in the direction in question.

Similarly, by evaluating the possible collision situations, we can obtain an idea of the resistance $C_{i,1}$ which the pier should have to forces $\neq$ the bridge line in relation to C_i for the probability of collapsing being equally big in all directions.

This leads to a value $p_i = p_C \cdot p_{G,i} \cdot N_i$, or, cf. (1):

$$p_i = k_i \cdot C_i^{-b_i}, \quad k_i = p_C \cdot p_{G,i} \cdot a_i \tag{2}$$

If we design the bridge so that

$$C_i \geq C_{i,max} \quad i = 1,2, \ldots, n \tag{3}$$

where $C_{i,max}$ is the biggest collision force to which pier no. i can be imagined to be subjected, the probability of an interruption of the crossing will be 0.

In many cases, this will result in prohibitive production costs.

The client will then have a natural possibility of accepting a certain risk of interruption of the connection.

By introducing two new concepts:

L: The anticipated lifetime of the bridge : the number of years after which it is estimated to be obsolete.

r: The risk of interruption of the bridge in the period L, which the client will accept.

We can formulate the following new design criterion:

$$\sum_{1}^{n} p_i \leq \frac{r}{L} \tag{4}$$

For the sake of clarity, we have so far only considered the bridge piers, but theoretically we can deal with the superstructure in the same manner provided we know the forces that are necessary to break the ships' masts and smoke stacks and to penetrate their deck superstructure or the uppermost part of their hulls, together with the height of these parts over daily water levels.

We can then include the bridge superstructure in the members considered, regarding C_i in this case as the maximum horizontal force which a bridge girder can resist.

This, however, calls for a new curve like fig. 1 for the superstructure.

The design criterion (4) was employed in the case of the Storebælt Bridge, and the result thereof for the high-level bridge over the east channel is shown in fig. 3.

However, the criterion (4) gives no direct information on the most economical distribution of the capacities C_i, although there will be an intuitive feeling that p_i should be small for those members where a failure would result in exceptionally heavy costs and inconvenience.

For a consistent economic optimization, we need to know:

P_i (C): A curve giving a price for member no. i as a function of the capacity of this member in a given interval about C_i.

SUSPENSION BRIDGE				CABLE-STAYED BRIDGE			
PIER No.	WATER DEPTH M	SHIP SIZE DWT	SHIP IMPACT MN	PIER No.	WATER DEPTH M	SHIP SIZE DWT	SHIP IMPACT MN
1,2,3	6	4.000	60	1,2,3, 4,5	6-7	4.000	60
4,5	7	10.000	100	6,7	8	10.000	100
6,7, 8,9	8-9	60.000	240	8,9	8-9	60.000	240
				10,12, 13,15	12-25	110.000	320
11,14, 20,23	15-25	250.000	430	16,17, 18,19, 21,22	15-35	250.000	430
				24	13	110.000	320
25,26, 27	8-10	60.000	240	25,26, 27	8-10	60.000	240
28,29, 30	5-6	10.000	100	28,29, 30	5-6	10.000	100
31,32, 33,34	2-5	4.000	60	31,32, 33,34	2-5	4.000	60

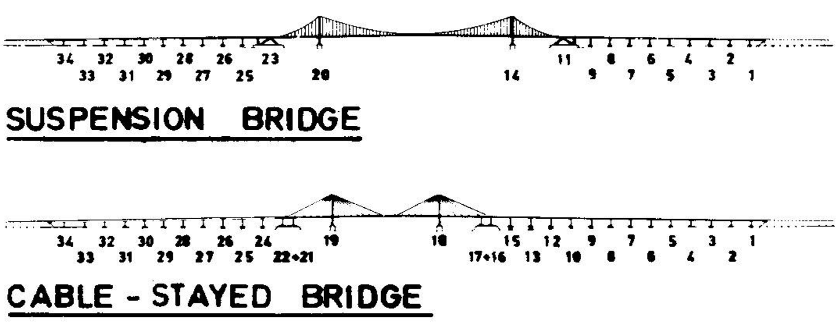

Fig. 3 Ship impact forces specified for the piers of the eastern high level part of the Storebælt Bridge.

U_i: The costs resulting from failure of member no. i. These comprise:

1) The cost of re-establishing member no. i together with the other members destroyed through the failure of member no. i.

2) The cost of establishing and operating an emergency connection during the repair period.

3) The costs resulting from loss of human life, disablement and the destruction of material assets in connection with the collapse.

4) The national economic loss through reduction of the capacity of the connection during the repair period.

The evaluation of U_i will be very uncertain and, especially as regards points 3) and 4), will be based on rather arbitrary assumptions.

We can now calculate:

R_i: The expected cost of repairing member no. i on account of ship collisions in the course of the period L.

$$R_i = p_i \cdot L \cdot U_i \tag{5}$$

and the expected gross price of the bridge in its lifetime will then be

$$\overline{P} = \sum_1^n (P_i + R_i) \tag{6}$$

In order to investigate whether maintaining the design criterion (4), variations ΔC_i in the capacity of the individual members will have a favourable influence on the effective price of the bridge, we can calculate:

$$\overline{P} + \Delta\overline{P} = \sum_1^n \left(P_i + R_i + \left(\frac{dP_i}{dC_i} + L\,U_i\,\frac{dp_i}{dC_i}\right)\Delta C_i\right) \tag{7}$$

and seek a minimum value for this subject to the condition (cf. (4)):

$$\sum_1^n (p_i + \Delta p_i) \leqq \frac{r}{L} \tag{8}$$

Under reference to (2), we have, in a certain area of C_i:

$$\frac{dp_i}{dC_i} \sim -b_i \cdot k_i \cdot C_i^{-(1+b_i)} = -\frac{p_i b_i}{C_i} \tag{9}$$

hence,

$$\Delta C_i \sim - \frac{C_i}{b_i} \frac{\Delta p_i}{p_i} \tag{10}$$

By means of (5), (9) and (10), we can rewrite (7) as

$$\bar{P} + \Delta\bar{P} = \sum_1^n (P_i + p_i\,LU_i - (\frac{dP_i}{dC_i} - \frac{p_i b_i}{C_i} LU_i) \frac{C_i}{b_i p_i} \Delta p_i)$$

$$= \sum_1^n (P_i + \frac{C_i}{b_i} \frac{dP_i}{dC_i} + (LU_i - \frac{C_i}{p_i b_i} \cdot \frac{dP_i}{dC_i})(p_i + \Delta p_i)) \tag{7a}$$

and can now find improved values $p_i + \Delta p_i$ of p_i by seeking the set that gives the least possible value of (7a) while at the same time complying with the criteria (8), supplemented by

$$p_i + \Delta p_i > 0,\ i = 1,2 \ldots\ldots,\ n \tag{11}$$

During the design of the Storebælt Bridge, consideration was given to employing (7a) in connection with (11) as design criterion, but this approach was abandoned owning to the considerable uncertainty connected with determination of the quantities U_i, the costs which would result from failure of member no. i.

One could, however, determine the relative values of the quantities U_i with considerably greater certainty, while maintaining the necessary assumptions consistently and uniformly for all members.

A minimum value of (7a) would thereby result in a reasonably good distribution of the costs between the structural members of the bridge, even with an incorrect level for the quantities U_i. This must just be set so low that the adopted design criterion (8) becomes effective.

By putting L = 0 in (7a), i.e. by disregarding the magnitude of any repair costs, one could arrive at the cheapest design that satisfies the design criterion.

It seems like that, in the planning of an offshore structure, one would have a greater possibility of calculating the consequences of a collapse and thus of employing (7a) and (11) as design criterion: however, a discussion of this falls outside the scope of this article and the author's experience.

It should, of course, be noted that the foregoing only provides information on the necessary capacities of the n members in a specific design of the bridge and that an economic optimization is therefore pointless before one is certain that a different design, for example, with other spans or a different longitudinal profile, more extensive precautions for protection of the piers etc. will not give a better solution.

1.1. Summary of design criteria

When planning a major bridge over navigated waters, certain steps must be taken as outlined below in order take account of the risk of the connection being interrupted due to collisions between the bridge and ships:

1) Procure information on the number of ships that must be expected to pass the bridge each year within a certain time horizon.

2) Arrange the ships in an order that as far as possible gives the largest force C which they can exert on the bridge during a collision, in other words, plot a curve as in fig. 1.

3) On the basis of this curve and information on navigation conditions, wind, current, etc., formulate a model that gives the probability p_i of a ship hitting an important structural member (no. i) in the bridge during a year, thereby imposing a load $\geq C_i$ on the member, where C_i is the force that just causes the member to fail.

4) By means of the model, determine a value $C_{i,1}$ of the components' resistance to forces ≠ the bridge line that the probability of failure is equally great in all directions.

5) Decide on the risk r that one is prepared to run of a breakdown of the bridge in its expected lifetime L.

6) Design the bridge so that Σp_i, extended over all members, is smaller than $\frac{r}{L}$

With this approach, and taking account of the costs resulting from an increase in C_i and the costs resulting from failure of the member, one can seek to achieve the desired result as economically as possible.

By designing on the basis of the procedure outlined above, one will have done one's best to achieve a safety level adopted in advance, although it must be admitted that the precision with which this level is reached is hardly likely to be very great.

On the other hand, precise determination of the safety level considered to be desirable is also an extremely difficult matter for the client, who, while wanting this to be as high as possible, has limited means to invest in the construction of the bridge because of necessary considerations to other national tasks.

In a manner of speaking, the concept "the risk of breakdown of the connection within a certain time horizon", puts the client and the technicians working for him on speaking terms, allowing them to negotiate and reach the best possible decision guided by the knowledge existing at any time - which must naturally be constantly widened and deepened.

The most difficult task of all is undoubtedly to judge the probability that one ship out of a number of ships that are theoretically able to collide with the bridge with fatal consequences, is actually doing so, and then to find the means to reduce this probability.

It is to be hoped that the contributions to Theme C will create the possibility of a more reliable solution of these problems.

On the other hand, with the knowledge we already possess, we can determine with reasonable accuracy the consequences of collision with a ship of known size, speed and type.

In the view of the author, the greatest advantage offered by the design method described lies in the fact that one ensures a structure without isolated, particularly weak points, and that the materials and other resources made available for construction of the bridge are distributed in the most appropriate manner. In other words, an additional investment to take account of the risk of ship collision is utilized as effectively as possible.

1.2 National design rules

Such thorough treatment of ship collision problems as described above would normally be reserved for big and really important bridges, and it is obviously reasonable to establish simplified design rules for, say, small bridges within a national area with uniform wind and weather conditions, and especially, uniform requirements to safety level.

The following section on collision force from the Joint Nordic Load Specifications [8] is an example of such national design rules:

> "Where there is a risk of a ship colliding with a bridge pier, the pier shall be designed for collision. The forces occurring during a collision will depend on the design and size of the vessel, its load and its speed, the collision point and direction of impact, together with the mass and elasticity of the bridge structure. The collision forces shall be assumed to act centrally on the pier level with the water surface, either in the longitudinal or in the transverse direction of the pier.
>
> As design vessel, use can be made of a vessel whose size must be expected to be exceeded in a specific number of passages per annum (e.g. 100 passages/year in an easily navigable channel). When determining the design vessel, account must be taken of the prevailing navigation conditions (wind, current, vision, compulsory pilotage, etc.), and of the risk which it will be reasonable to accept having regard to the design of the bridge, the width of the channel and the intensity of the traffic.

In the comments guidelines are given for evaluating the magnitude of the collision forces assuming that the bridge pier can be regarded as non-yielding and that the whole of the collision energy is absorbed through deformations (damage) in the vessel. This assumption normally applies where the pier is designed to take account of the risk of collision by big ships.

In the case of an elastic bridge pier, which can occur in the case of low collision energy, the collision force can be determined on the basis of the deformation properties of the structure and the ship.

Comments:

- On the basis of the size (tonnage or draught) of the
- design vessel and the permitted speed in the channel,
- the magnitude of the collision force can be estimated by
- means of the following diagram (fig. 4)."

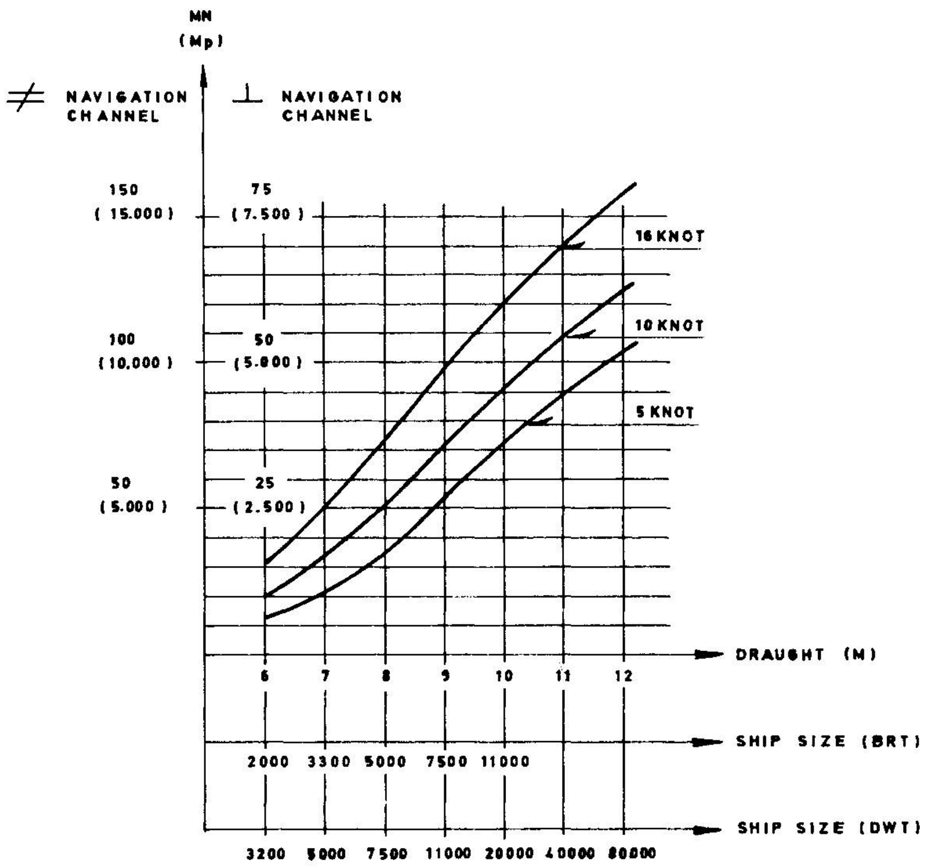

Fig. 4 Magnitude of ship collision force as a function of ship size and speed. (From [8]).

It will be seen that these rules are very similar to those proposed earlier.

We put $N_i \sim 100$ and thereby arrive at a ship of a certain size. From fig.4 we then read C_i, for which the piers are designed.

Since $P_i = p_C \cdot p_{G,i} \cdot N_i$, we accept, cf. (4), a risk $100L \cdot p_C \cdot p_{G,i} \sim 0.4 \cdot p_{G,i}$ of collision with pier no. i disrupting the bridge during its lifetime.

Contributions to Theme F containing examples of national design rules will be of great interest for the preliminary report.

2. INFLUENCE ON DESIGN

In the foregoing attention has been concentrated on establishing rules that will, with reasonable certainty, prevent a bridge over navigated waters from being interrupted on account of ship collision.

The principle effect of these rules is to make the piers often appreciably more expensive.

They must be designed as strong, solid structures without abutments or other slender members that can result in secondary, but catastrophic failure.

They must have ample resistance to loads in all directions, including torsion, which can occur in the event of eccentric impact, and it should be ensured by means of a dynamic analysis that any bearings between piers and superstructure can transmit the forces occurring during a collision.

Simply to be able to resist collisions, the piers get such large dimensions that their carrying capacity in respect of deadload, traffic load, wind, etc., cannot be fully utilized unless suitable bigger spans are introduced than have hitherto been used.

The development can be illustrated by a brief account of the proposals put forward over the years for a bridge crossing the east channel in Storebælt:

A proposal in 1936 from the Danish engineering firms, Christiani & Nielsen, Højgård & Schultz and Kampsax, resulted in the first official project from the Bridge Office of the Danish State Railways which was at that time responsible for all major bridges in Denmark.

In 1948, a broadly composed commission was appointed to investigate the conditions for and the effects of a permanent crossing. In December 1959, this commission presented its report including a proposal, which was an obvious development of the project of the Danish State Railways, envisaging a 2-level lattice girder for road and railway with navigation spans of 300 + 350 + 300 m and approach spans of 135 m.

In 1965-67, an international competition for sketch proposals was held, and following this, the working committee appointed presented a proposal with similar spanning as the 1959 proposal.

In 1970, a Technical Committee was appointed which, in its report from 1972, presented two proposals:

1. A continuation of the lattice girder solution with 5 spans, 280 + 400 + 325 + 400 + 280 m over the navigation channel.

2. A solution with two cable-stayed bridges, 210 + 600 + 210 m in direct extention of each other, forming two separate navigation spans of 600 m.

In 1973, the Board for the State Bridge Storebælt was appointed. In 1978, the Board, assisted by its consultants, prepared two tender projects:

1. A cable-stayed bridge with a navigation span of 780 m, two side spans of 300 m and approach spans of 144 m.

2. A suspension bridge with a navigation span of 1416 m, two side spans of 360 m and approach spans of 144 m.

In all cases, but especially in view of the constantly increasing requirements to resistance collision forces, the proposals in question were optimized with regard to spans, taking due regard to water depths and foundation conditions.

The big spans have the added advantage of reducing the direct risk of a ship colliding with a pier, because there are fewer piers.

In other words, the risk is reduced of environmental damages occurring through a ship with a hazardous cargo springing a leak through a collision.

We have not earlier concerned ourselves with this aspect, concentrating on whether the bridge would be damaged in a collision, and not thinking about the ship.

In this connection, I would finally like to make a few remarks regarding special protective measures for the piers, for example protective islands. With the conditions applying at many of the piers in Storebælt, protective islands proved to be an effective, low-cost method of increasing the capacity C_i, while at the same time reducing the damage to the ship.

However, with the exception of the anchor piers of the east bridge, the idea of using protective islands had to be abandoned for fear that their consistent use would reduce the passage so much that it would have damaged the environment in the Baltic.

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