

Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 41 (1983)

Artikel: Design assumptions and influence on design of offshore structures
Autor: Fjeld, Svein
DOI: <https://doi.org/10.5169/seals-31664>

Nutzungsbedingungen

Die ETH-Bibliothek ist die Anbieterin der digitalisierten Zeitschriften. Sie besitzt keine Urheberrechte an den Zeitschriften und ist nicht verantwortlich für deren Inhalte. Die Rechte liegen in der Regel bei den Herausgebern beziehungsweise den externen Rechteinhabern. [Siehe Rechtliche Hinweise.](#)

Conditions d'utilisation

L'ETH Library est le fournisseur des revues numérisées. Elle ne détient aucun droit d'auteur sur les revues et n'est pas responsable de leur contenu. En règle générale, les droits sont détenus par les éditeurs ou les détenteurs de droits externes. [Voir Informations légales.](#)

Terms of use

The ETH Library is the provider of the digitised journals. It does not own any copyrights to the journals and is not responsible for their content. The rights usually lie with the publishers or the external rights holders. [See Legal notice.](#)

Download PDF: 11.05.2025

ETH-Bibliothek Zürich, E-Periodica, <https://www.e-periodica.ch>

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

Svein FJELD
Senior Principal Surveyor
Det norske Veritas
Oslo, Norway



Svein Fjeld born in 1939, Dr.ing. degree in structural engineering Technical University of Norway 1968. 10 years design work mainly in the fields of foundations, harbours and offshore structures. Present position as assistant head of Industrial and Offshore Division of Det norske Veritas.

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 règlements 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 Erfüllung 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 Dimensionierung und Festlegung einzelner Konstruktionsdetails. Die Plattform, die durch Auftrieb stabilisiert 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 coefficients					
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 occurrence

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



Load coefficients	Norwegian Petroleum Directorate ¹⁾		Det norske Veritas ¹⁾	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.

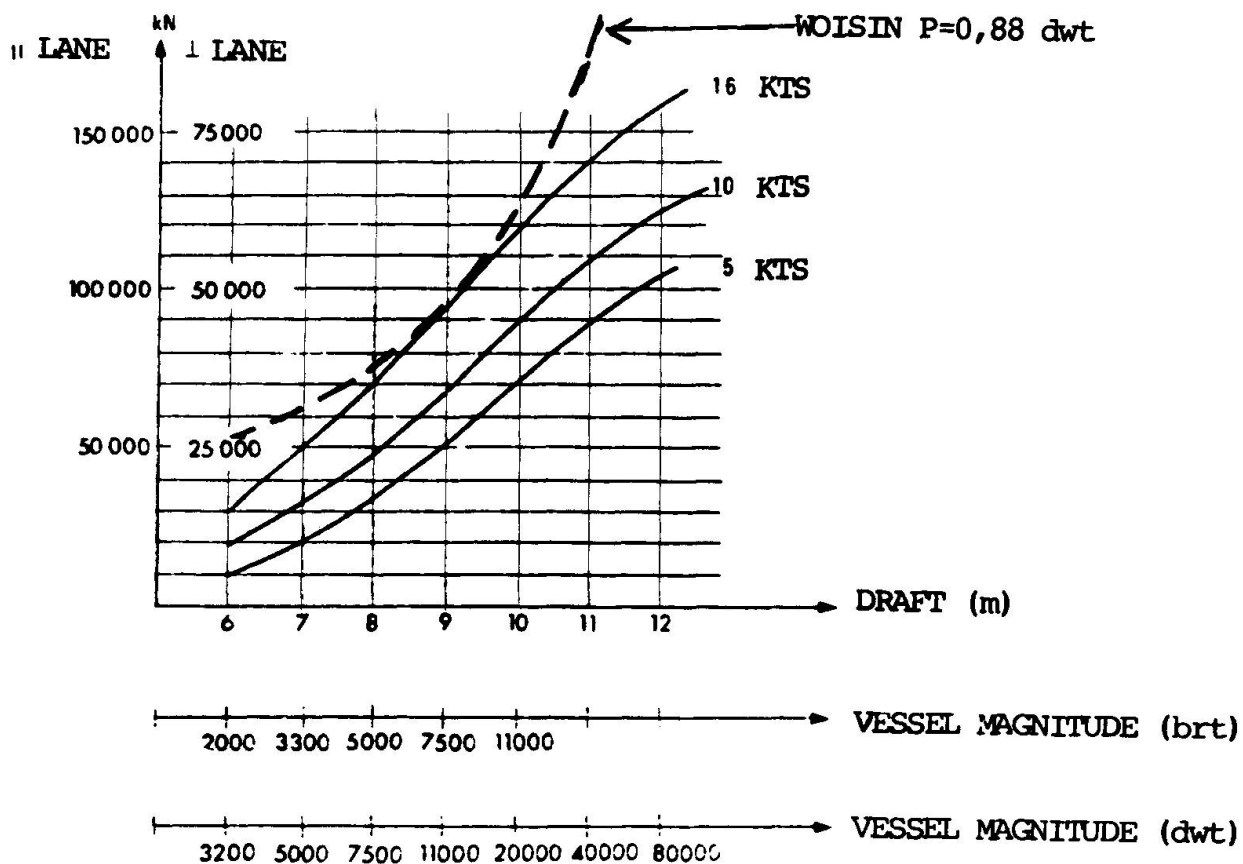


Fig. 1. Nordic Road Federation. Recommended impact force.



	Ship size Displacement if not other- wise specified	Ship Velocity	Design force	Procedure for force estimate, comment
<u>Bridge codes and practice</u>				
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	
<u>Offshore structures</u>				
<u>codes and practice</u>				
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	5000 t	2 m/s		PLS, energy considerations
(20)(10)(18)	5000 t	0,5 m/s		ULS
<u>Bridge projects:</u>				
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	
Bahrain/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 bridge 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 $v \text{ (m/s)} = 1/2 H_s \text{ (m)}$.

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.

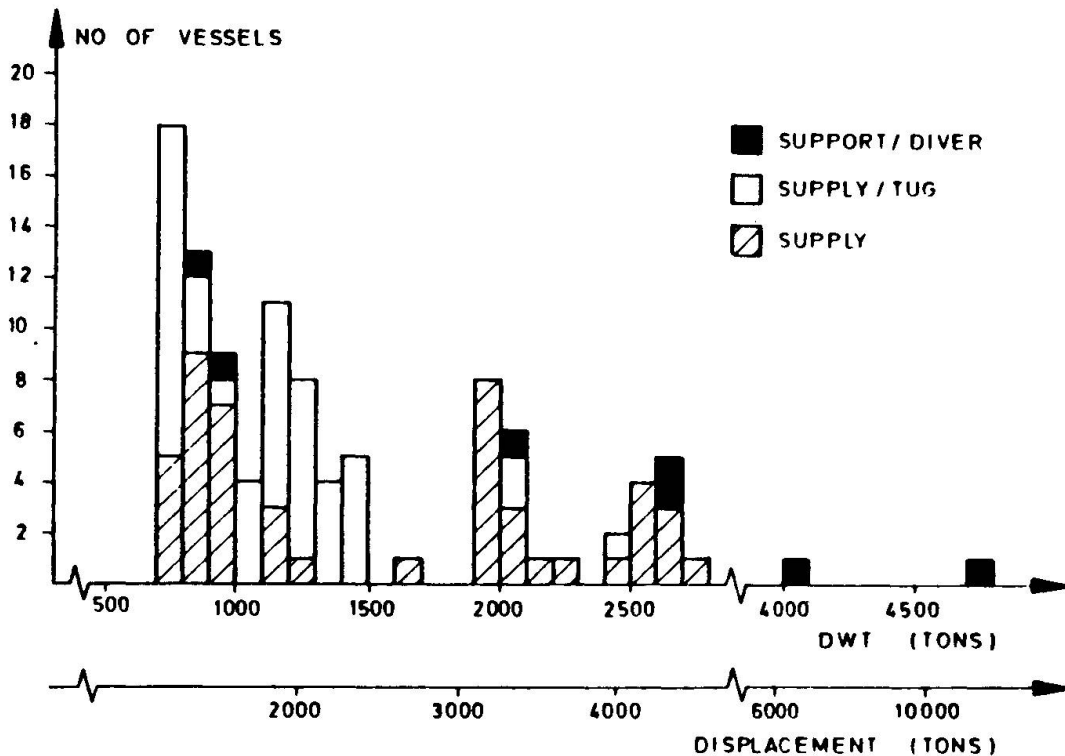


Fig 2. Supply vessels in the Veritas class - 1980.

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 $V = 0.5 \text{ m/s}$. 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]:

- The ship striking the platform which experience plastic deformations, and
- 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.

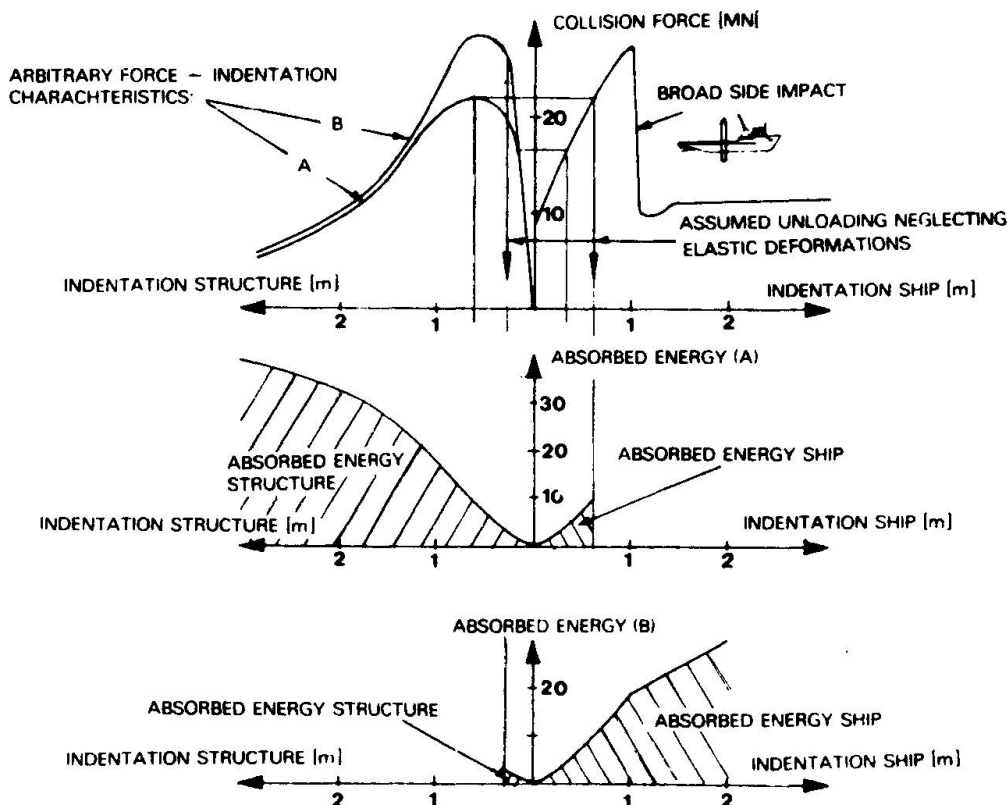


Fig. 3. Energy distribution ship/structure.

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

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:

$$P_{\max} \approx 0.88 \text{ dwt} \pm 50\%$$

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

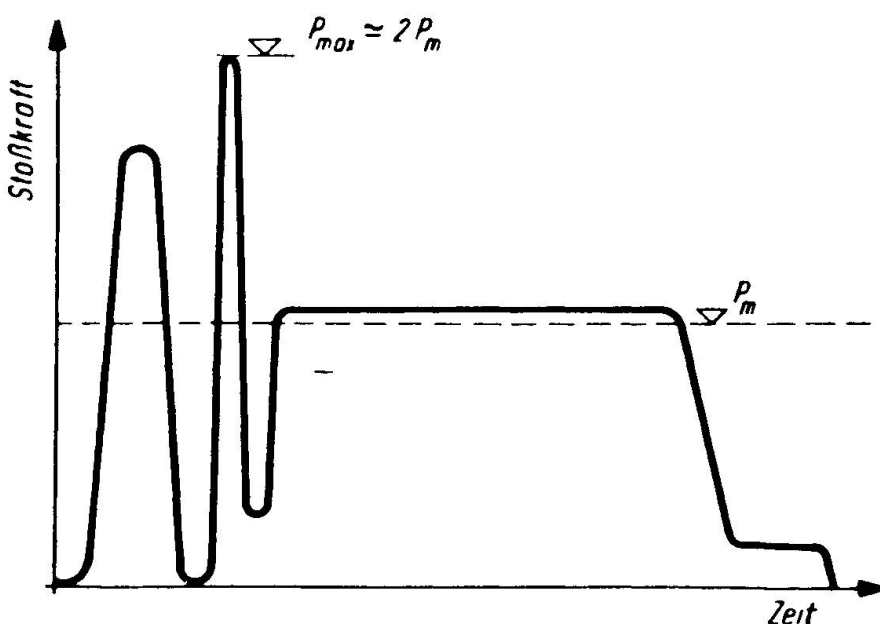


Fig. 4. Time-force relation for ship impacts [14].



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].

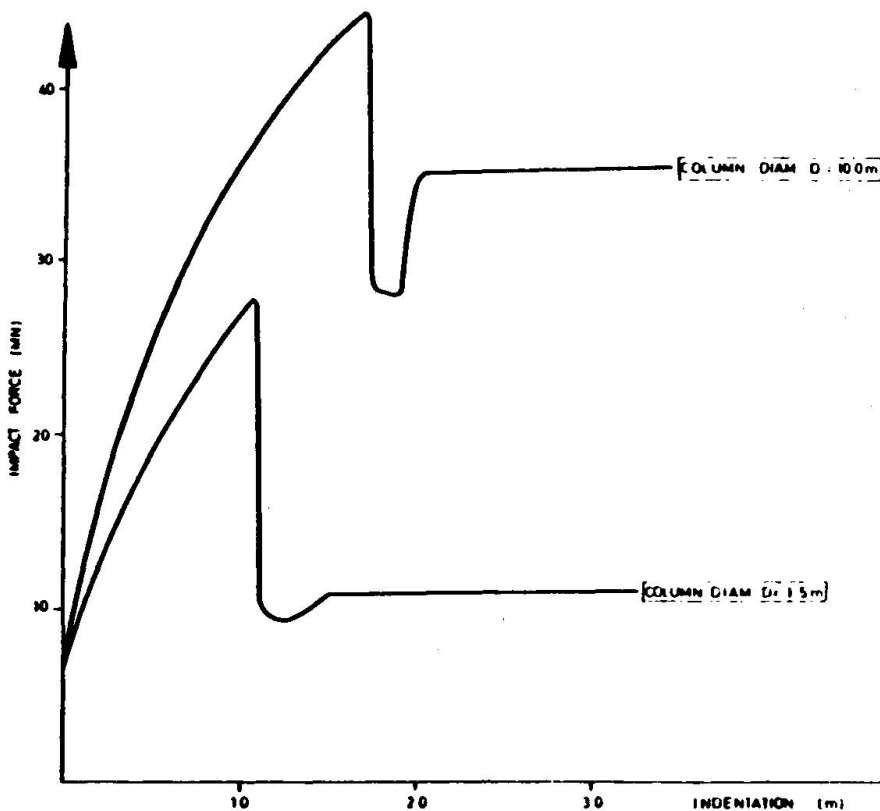


Fig. 5. Force/indentation curve for broadside impact with infinitely stiff cylinder. Boat displacement 5000 t.

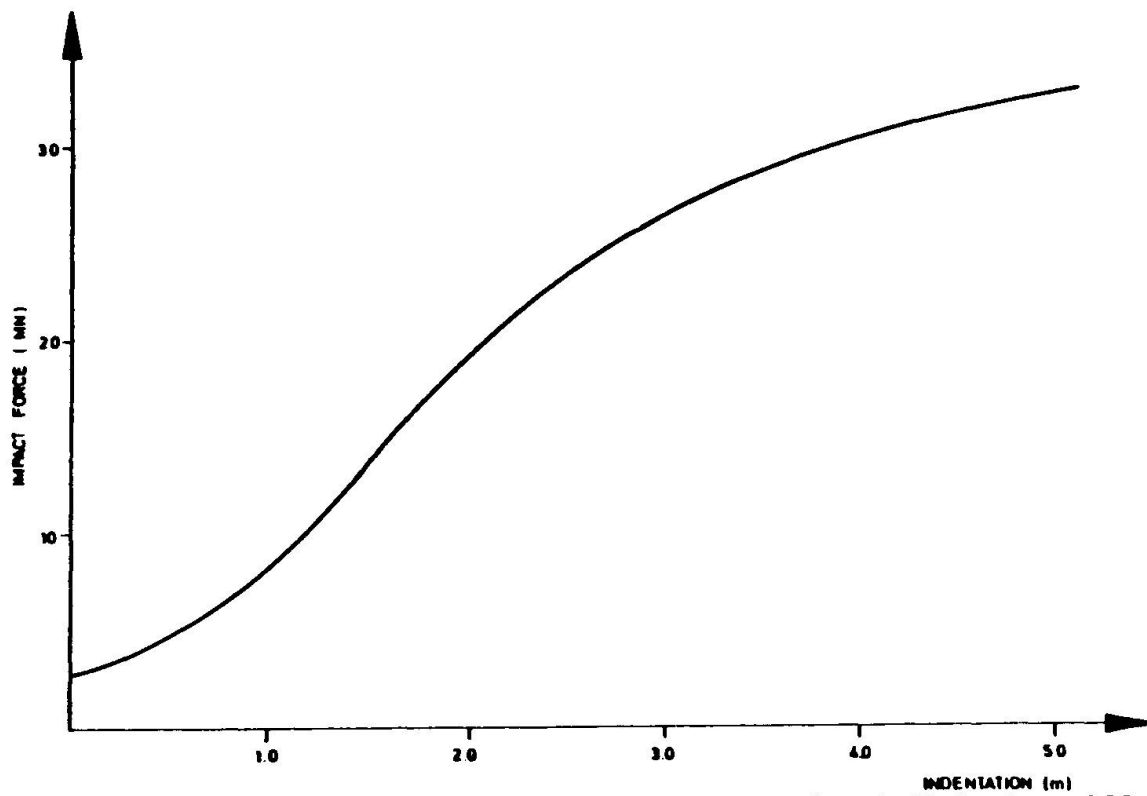


Fig. 6. Force/indentation curve for bow impact with infinitely stiff cylinder. Boat displacement 5000 t.

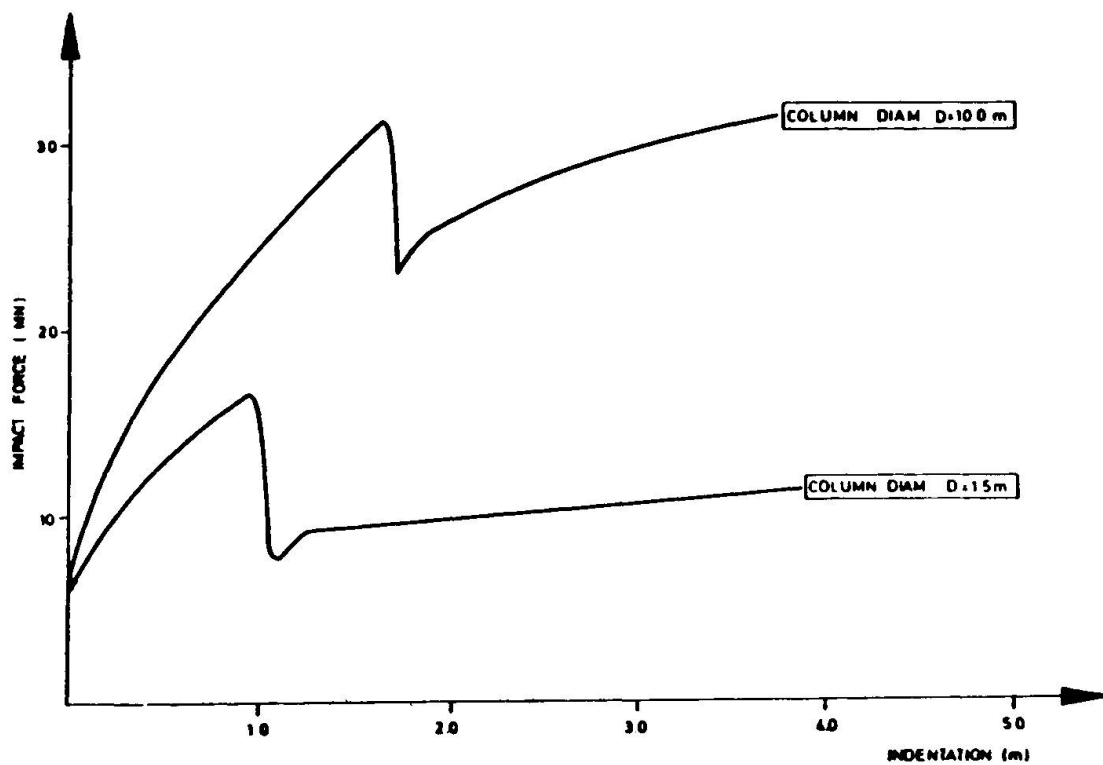


Fig. 7. Force/indentation curve for stern impact with infinitely stiff cylinder. Boat displacement 5000 t.

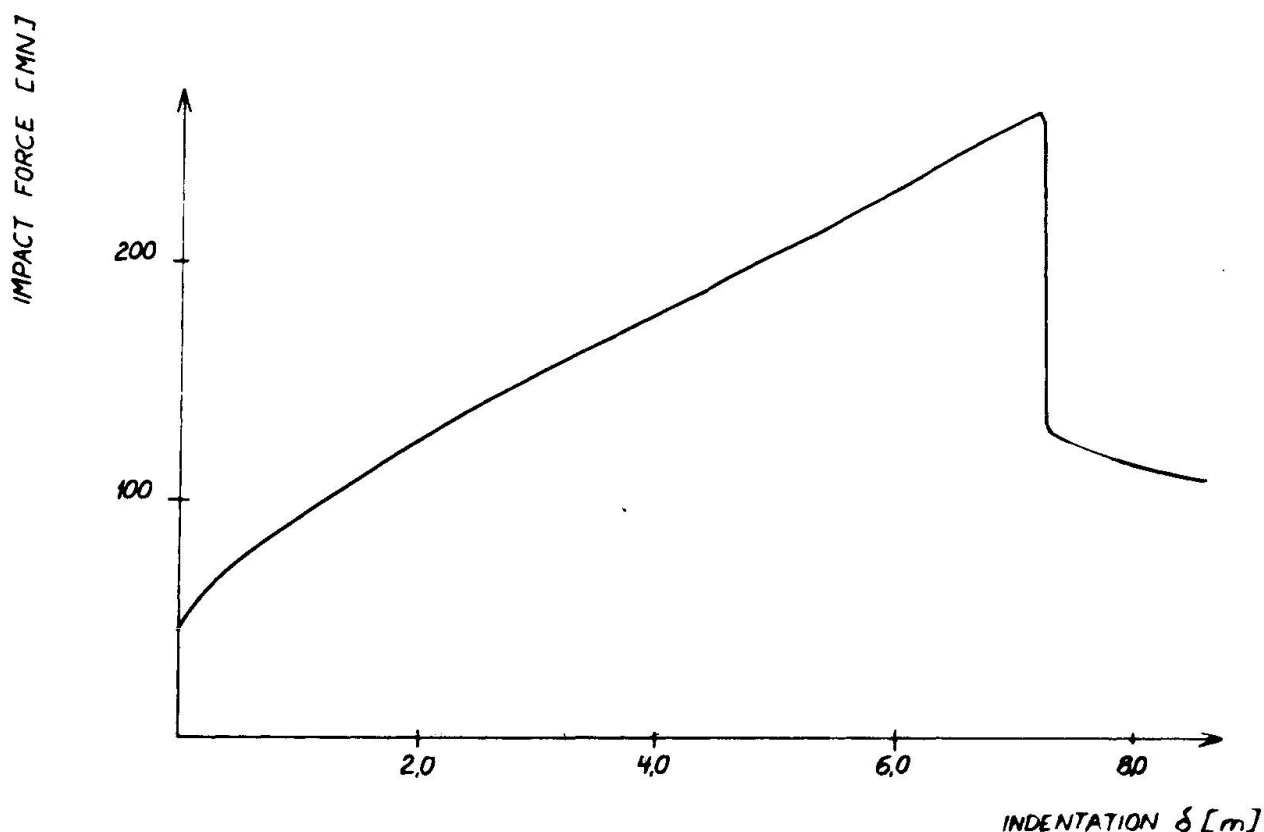


Fig 8. Force/indentation curve for broadside impact with infinitely stiff cylinder. Boat displacement 150,000 t. Diameter 13.3 m.

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].

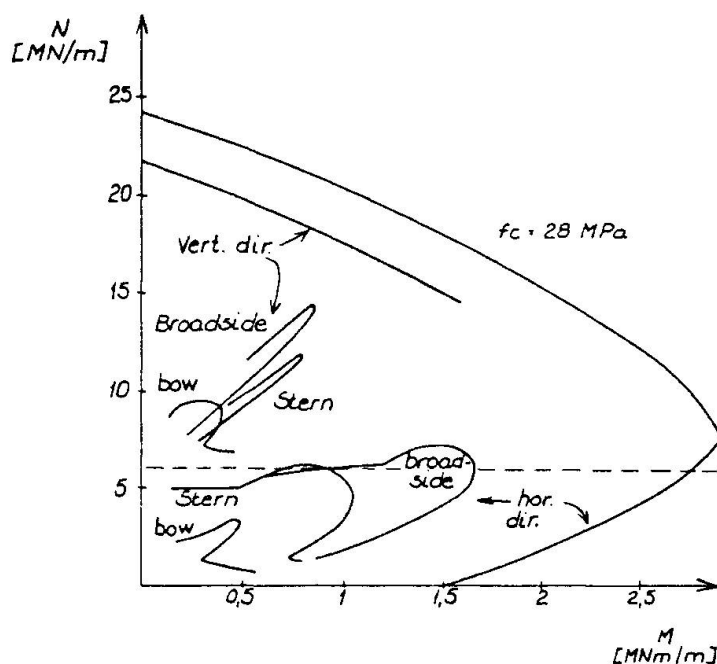


Fig. 9. Load effects and strength of 65 cm thick concrete shaft.

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.

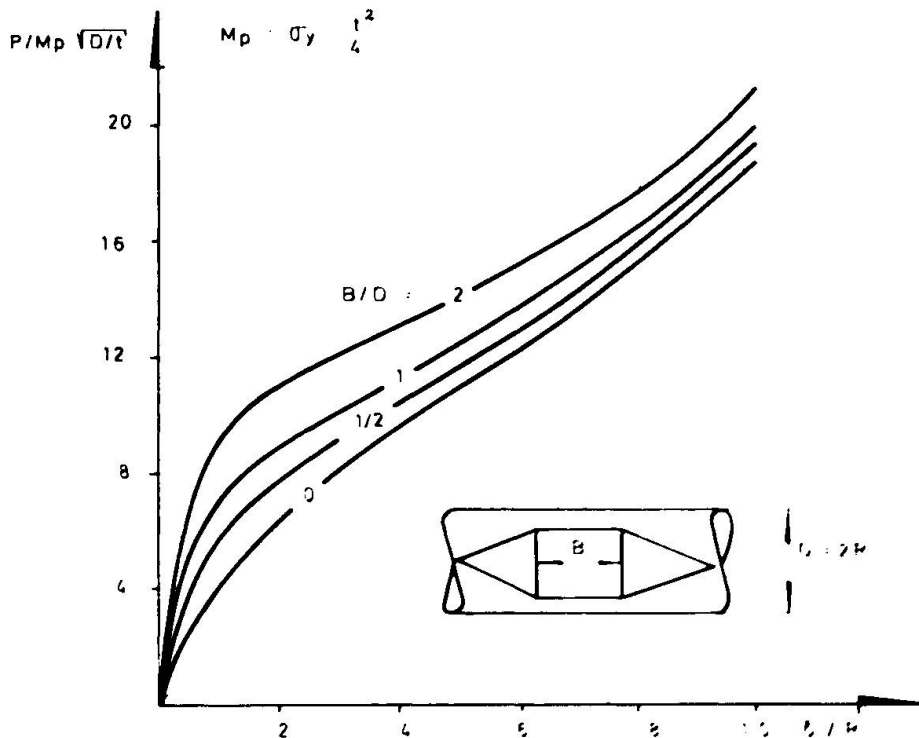


Fig. 10. Load indentation relation for steel tubes.

- 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].

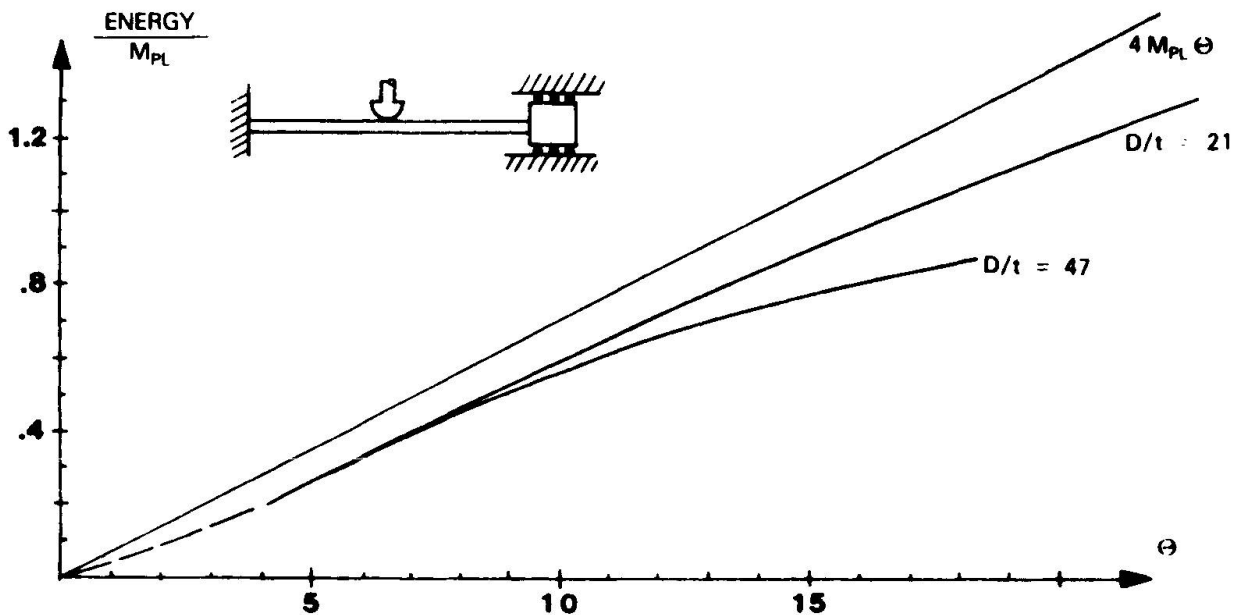


Fig. 11.- Plastic energy absorption in a three hinge mechanism.

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.

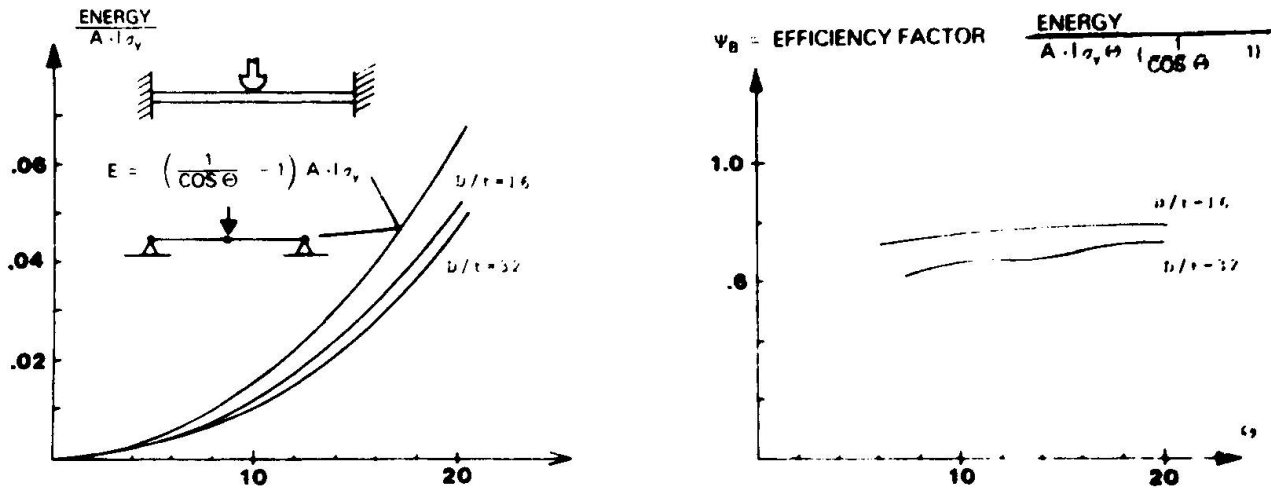


Fig. 12. Energy absorption in axially restrained members [29].

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

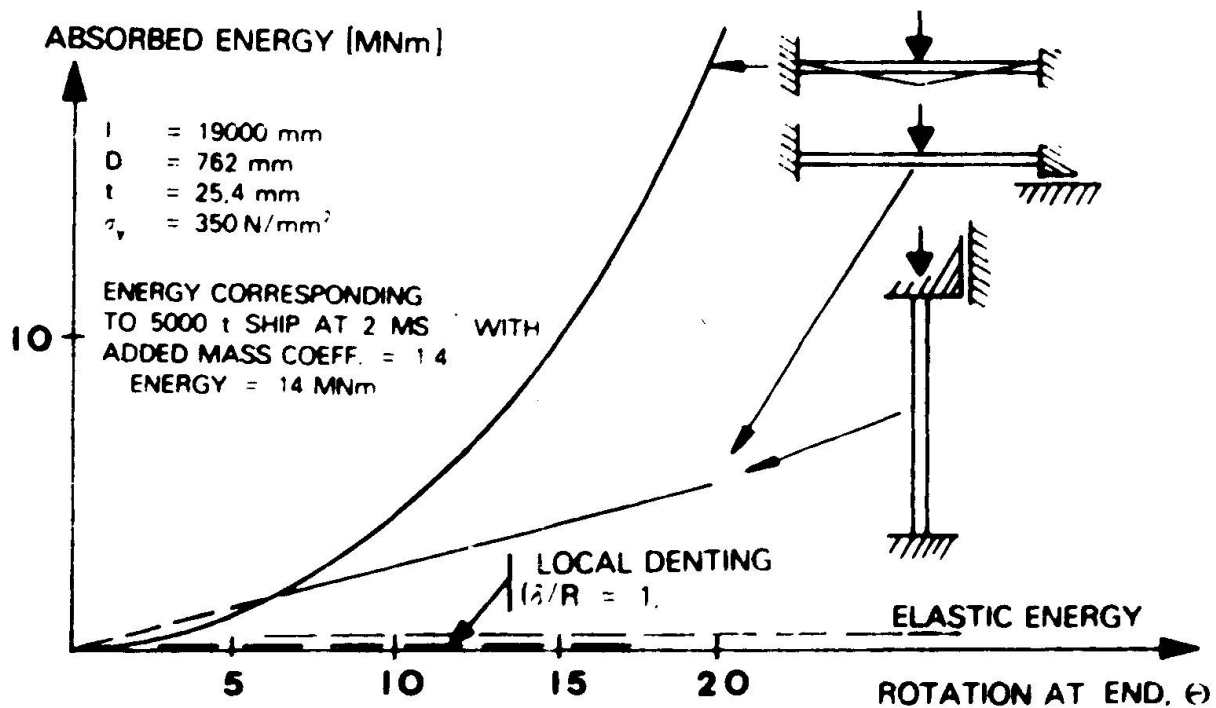


Fig. 13 Comparison of energy absorption capabilities [29].

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 indispensable 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.

REFERENCES

1. JOINT COMMITTEE ON STRUCTURAL SAFETY., Common Unified Rules for Different Types of Construction and Materials, Paris, 1976.
2. ECCS., European Recommendations for Steel Construction, 1978.



3. CEB-FIP., Model Code for Concrete Structures, 1978.
4. NPD., Guidelines for Safety Evaluation of Platform Conceptual Designs, 1981.
5. VERITAS., Design against Accidental Loads TNA 101, 1981.
6. FRANDSEN AND LANGSØ., Ship Collisions Problems, IABSE Proceedings, P-31/80.
7. DEUTSCHE BUNDESBahn., Vorschrift für Eisenbahnbrücken und Sonstige Ingenieurbauwerke Voraussgabe der DV 804, 1979.
8. DIF., Norm for Pælefunderede Offshore Stålkonstruksjoner (Code for Piled Offshore Steel Structures), in Danish, Proposal, January 1982.
9. DEPARTMENT OF ENERGY., Offshore Installations: Guidance on Design and Construction, 1977.
10. VERITAS., Impact Loads from Boats TNA 202, 1982.
11. NORDIC ROAD FEDERATION., Lastbestemmelser for Vegbruer (Load Code for Road Bridges) in Norwegian, 1980.
12. SAUL AND SVENSSON., Zum Schutz von Brückenpfeilern gegen Schiffsanprall, Dargestellt am Beispiel der Brücken Zarate-Brazo Largo über den Parana (Argentinien). Die Bautechnik 10, p. 326, 1981.
13. MINORSKY., An Analysis of Ship Collision with Reference to Protection of Nuclear Power Plants, Journal of Ship Research 3, 1959.
14. WOISIN., Konstruktion gegen Kollisionsauswirkungen, Schiff und Hafen/Kommandobrücke 31/1979.
15. WOISIN AND GERLACH., Beurteilung der Kräfte aus Schiffsstößen auf Leuchttürme, in Sec. 8. International Seamark Conference, Stockholm, 1979.
16. RINGVOLD., SR-24: Collisions, Risk Analysis of an Offshore Petroleum Production Platform. Veritas Report No. 81-1247.
17. PETERSEN., Beregning av hydrodynamisk tillægsmasse for forsyningsskib ved sideværts stød. (Calculation of Hydrodynamic Added Mass for Supply Ships by Sideways Impact). Executed upon request from Det Norske Veritas, in Danish, 1981.
18. NPD., Regulations for the Structural Design of Fixed Structures on the Norwegian Continental Shelf, 1977.
19. API., Planning, Designing and Constructing Fixed Offshore Platforms, RP2A, 12 ed., January, 1981.
20. VERITAS., Rules for the Design Construction and Inspection of Offshore Structures, 1977.
21. RØLAND, SKÅRE AND OLSEN., Ship Impact on Concrete Shafts. Nordisk Betong 2-4, 1982.



22. NPD., Forskrifter for beregning og dimensjonering av faste bærende konstruksjoner på den norske kontinentalsokkel. Vegledning om kontroll av grensetilstand for progressivt brudd. (Regulations for the structural design of fixed structures on the Norwegian Continental Shelf. Guidelines about control and limit state of progressive collapse). Enclosure to letter from NPD dated 22.6.1979.
23. SHERMAN, D.B., Test of Circular Steel Tubes in Bending. ASCE Journal of Struc. Div., Vol. 102, No. ST 11, 1976.
24. AMDAHL, HYSING AND VALSGÅRD., Impact and Collisions Offshore: Progress Reports 1-13 (1977-1982).
25. FCB., Lokalbelastning på forspente sylinderskall. Beregningsformel. (Local loads on pre-stressed cylindrical shells. Design formula). STF 65 F78021, in Norwegian, 1978.
26. BKF-SENTRALEN., Marine Betongkonstruksjoners Stødbæreevne. (Impact resistance of marine concrete structures). Report No. 079-1978, in Danish, 1978.
27. BRAKEL AND OSTLANDER., Concentrated Loading on a Thick Walled Concrete Cylinder. BOSS-Conference, London, 1979.
28. BSRA., A Feasibility Study on the Establishment of Force Time Curve for the Structural Deformation of an Ocean-going Tug in Collision with a Rigid Structure. Wallsend Research Station, 1975.
29. FOSS AND EDVARDBSEN., Energy Absorption during Ship Impact on Offshore Steel Structures. OTC Paper 4217, 1982.
30. VERITAS., Rules for Design, Construction and Inspection of Offshore Structures. Appendix E. Hydrostatic Stability and Anchoring.
31. PETERSEN AND PEDERSEN., Collisions between Ships and Offshore Platforms. OTC Paper 4134, 1981.
32. DHALLA, A.K., Discussion to [23]. ASCE Journal of Struc. Div., Vol. 103, No. ST 8, 1977.
33. ALLEN, D., Discussion to [23]. ASCE Journal of Struc. Div., Vol. 103, No. ST 7, 1977.