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# Theme A

## Case Stories of Recent Ship Collision Accidents
Rapports d'accidents de collisions de bateaux
Fälle von neueren Schiffskollisionen

# Ship Collisions with Danish Lighthouses

## Collisions de navires avec des phares danois

## Schiffskollisionen mit dänischen Leuchttürmen

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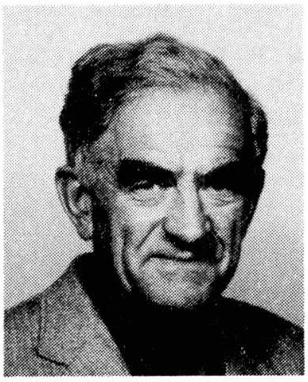

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**SUMMARY**
Since 1972, fourteen small lighthouses have been built in the Great Belt at water depths ranging between 8 and 17 m. Many of them have since then been exposed to ship collisions. Two of them, situated closely to the planned site of the Great Belt Bridge, have for instance been run into three and four times, respectively. The conclusion of the collision cases must be said to be negligence.

**RÉSUMÉ**
Depuis 1972, quatorze petits phares ont été construits dans le Grand Belt à des profondeurs d'eau de 8 à 17 m. Nombre d'entre eux ont été entretemps exposés à des collisions de navires. Ainsi, deux de ces phares, implantés à proximité du site prévu pour le pont du Grand Belt, ont fait l'objet de trois resp. quatre collisions. Il a été conclu que ces accidents étaient le résultat de négligences.

**ZUSAMMENFASSUNG**
Seit 1972 sind im Großen Belt in Wassertiefen von 8 bis 17 m 14 kleine Leuchttürme gebaut worden. Viele von ihnen sind seither in Schiffskollisionen verwickelt gewesen. Zwei von ihnen, die nahe der geplanten Linienführung der Großen-Belt-Brücke liegen, unterlagen 3 bzw. 4 Zusammenstößen. Diese Kollisionen sind offensichtlich auf Fahrlässigkeit zurückzuführen.

## 0. INTRODUCTION

Danish lighthouses, lightships and buoys have before been the object of collisions, and one of them, with the 'Drogden Lighthouse' on 2/12-1946, was described in 'Ingeniørens Ugeblad' (Engineers' Weekly) No. 6 of 6 February 1965.

In the period between 1972 and 1977, fourteen small lighthouses were built along the so-called 'deep water route' of the Great Belt, more or less to replace lightbuoys.

With these lighthouses there have - so far - been ten collisions, of wich seven comprised the two lighthouses nearest the planned position of the Great Belt Bridge.

## 1. LOCATION OF LIGHTHOUSES (FIG. 1)

On account of the growing traffic with larger and more powerful vessels which without much difficulty have been able to force there way through the ice occurring in Danish waters, it has become increasingly desirable to have the floating buoyage, which had to be withdrawn during winter, replaced by light houses made fast on the sea bed.

The placing of the lighthouses became a compromise between the best position for navigation purposes and the most advantageous technical solution. The lighthouses are placed as close to the deep fairway as possible, and preferably where there are bends or turns of the route.

In the placing of the two lighthouses, Halskov Rev and Sprogø N. Ø., it was taken into consideration - following consultation with the Great Belt Commission at that time - that the lighthouses were to assist navigation when vessels had to pass under the Great Belt Bridge, if this should become a reality.

## 2. LIGHTHOUSE CONSTRUCTION AND EQUIPMENT (FIG. 2)

On account of the rather heavy drifting of ice in Danish waters, the lighthouses have been made as slender as possible in the area near the waterline. At a water depth of between 5 and 7 m, the steel lighthouse structure has been anchored in a round concrete caisson placed on a level layer of gravel on the sea bed.

At a height of approx. lo m above sea level, a glasfiber reinforced plastic lantern house has been placed containing the lighthouse equipment which is based on gas and a battery-operated racon.

With these sources of energy it has not been possible to establish flood light on the lighthouse. This would either require an independent power plant for each lighthouse or a land-connected submarine cable.

Either of these solutions would have multiplied the construction costs, but it must be admitted that a good flood light of the facade would undoubtedly have reduced the number of collisions.

The lighthouses have been designed in such a way that if they are run into, damages to the ship should be as insignificant as possible, as this would be less costly, less dangerous to the crew, and reduce the risk of oil pollution.

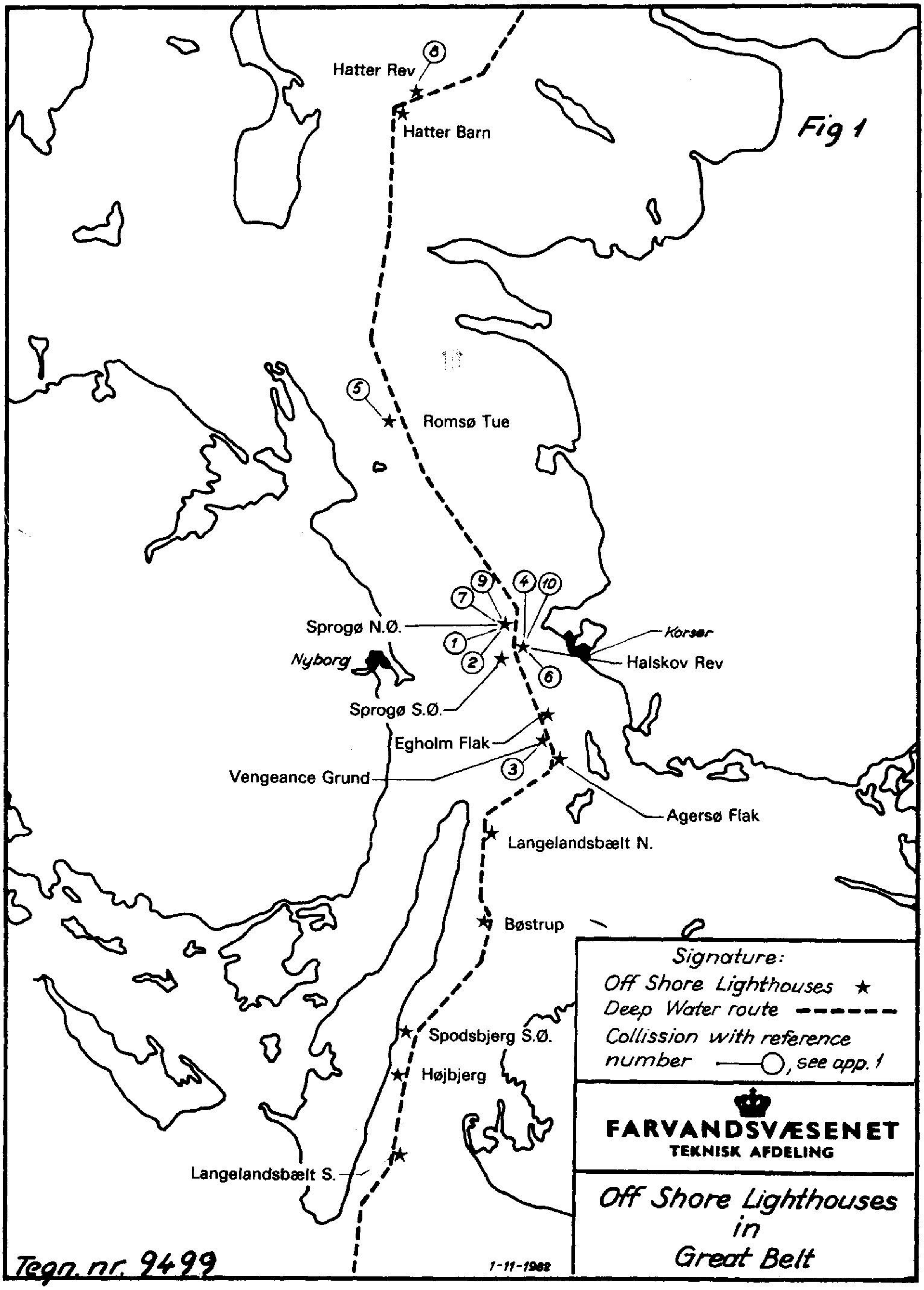


Fig. 1 Location of lighthouses in Great Belt

FIG. 2

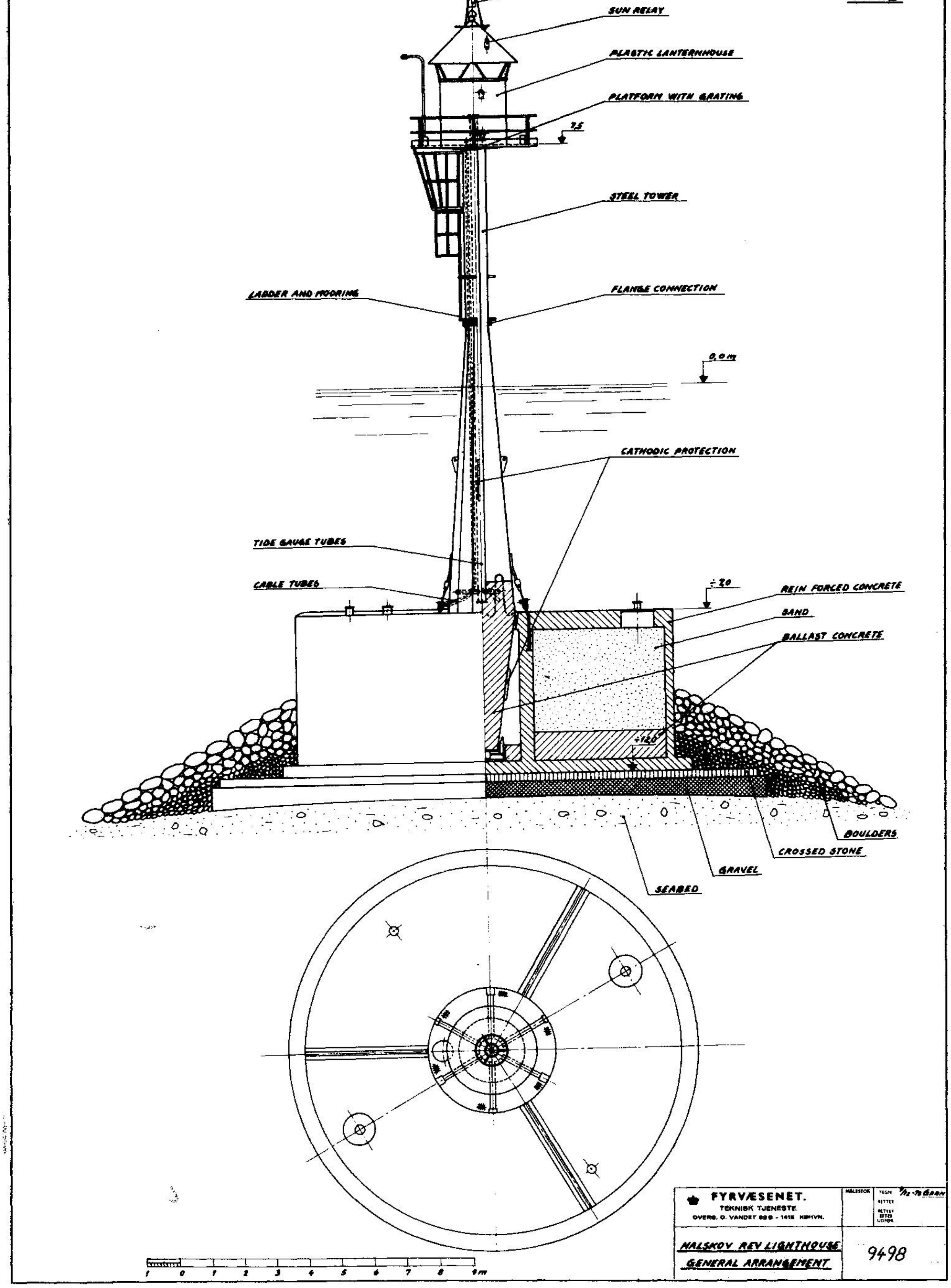


Fig. 2 Lighthouse. General arrangement

In return, it has been necessary to carry a stock of various spare parts for the lighthouses, in order that following a collision they can be promptly re-established.

3. SUMMARY OF REGISTERED COLLISIONS WITH THE LIGHTHOUSES

In appendix 1 a schematic presentation has been made of the ten known collisions with the Great Belt lighthouses.

In seven cases information as to the name of ship involved in collision and the time of it are available, but unfortunately not all the required information has been received which would enable us to make a detailed analysis of the causes, but the following information can be given:

Ref. 1. The ship was heading north. Unexpected strong current was stated as the cause of the collision. The ship hit the lighthouse abaft the beam, and the lighthouse broke and crashed into the sea.

2. The ship merely grazed the lighthouse, and no great damage was done.

3. The lighthouse had only been in service for four months when it was hit by an unknown ship, the draught for which must have been smaller than 5.3 m.

4. Reason given for the collision was that the distance had been misjudged. The ship was badly damaged, and had to go straight to a shipyard, The upper part of the lighthouse broke off, and the whole lighthouse was displaced about 1 m.

5. The upper part of the lighthouse was hit and thrown into the sea by the stem of the vessel which sustained minor damages, the ship had to put in for repairs.

6. An unknown ship has grazed the lighthouse.

7. Central collision with the lighthouse, the superstructure broke and the lighthouse was displaced.

   The stem of the ship was badly damaged.

   The captain, who was standing on the bridge when the collision happened, states that he was dazzled by the strong lights of a temporary drilling rig which was examining the sea bed in connection with the projected bridge siting. The lighthouse was only noticed at the moment the collision took place.

8. The unknown vessel has scraped its way heavily over the concrete caisson found at a depth of 6.2 m and compleatly destroyed the steel-structure.

9. The southbound Russian vessel 'General Shkodunovice' made a sweeping turn to avoid colliding with the northbound Russian ship which was allegedly sailing in the wrong side of the fairway. The two vessels collided nevertheless, and when 'General Shkodunovice' subsequently tried to back away, it was led towards the Sprogø N.Ø. lighthouse by the current.

   On the part of the pilot it was stated that it was realized that a collision with the lighthouse would take place, but that it was considered that the material damage would be smaller through this manoeuvre.

This collision, incidentally, resulted in the most extensive damage found so far, as the concrete caisson at a depth of 7.8 m was also damaged and displaced.

lo. It is stated that in order to save an alternation of course the ship intended to sail along the inside of the lighthouse, but the current drew the ship against it.

The steel structure of the lighthouse was spoilt completely, and the caisson was shifted 2 m.

The ship sprang a leak and was grounded, causing oil pollution.

## 4. COLLISION CAUSES

No failures of the steering gear have been ascertained in the registered collisions.

In one single case it has been mentioned, but it could not be proved. In another case a member of the crew stated that no one was on the bridge at the time of the collision, but this has not been officially confirmed, either.

It is possible that some of the involved ships have had charts wich were not up to date.

It is remarkable that all the registered collisions took place in the dark. A good flood lighting would have improved the possibility of judging the distance correctly.

Only in the collision, ref. 9, was there a pilot on board the vessel, and in this case the whole blame for the accident was placed on the ship approaching in the opposite direction.

All in all, it would appear that the causes of the collisions must be said to be human errors, such as negligence, misjudgements and ignorance of the special conditions in the Great Belt.

# Ship Collisions with Lighthouses in the Great Belt. Appendix 1.

| Ref. No. | Name of Lighthouse | Collision date | hour | Ship's name | Tonnage | Damage to Lighthouse | Comments |
|---|---|---|---|---|---|---|---|
| 1. | Sprogø N.Ø. | 27/2-74 | 19.35 | M/V Mulde | c.300 | Broken flange | Strong current |
| 2. | Sprogø N.Ø. | 11/3-75 | 04.15 | Dorothea | c.300 | Minor damage to ladder | |
| 3. | Vengeancegrund | 4/11-77 | | | | Broken flange | |
| 4. | Halskov Rev | 5/11-77 | 03.25 | M/V Windblow | c.400 | Broken flange. Displaced. | Misjudged distance |
| 5. | Romsøe Tue | 24/1-78 | 05.15 | M/S Anda | 578 | Broken flange | |
| 6. | Halskov Rev | 1/2-78 | | | | Minor damage to ladder | |
| 7. | Sprogø N.Ø. | 16/5-78 | 23.55 | M/S Eva Bress | 394 | Broken flange. Displaced. | Dazzled by lights from temporary drilling rig |
| 8. | Hatter Rev | 30/9-78 | Evening | - large ship | | Steel structure completely destroyed | |
| 9. | Sprogø N.Ø. | 31/10-79 | 06.00 | General Shkodunovice | 12,000 | idem. - and concrete caisson damaged | |
| 10. | Halskov Rev | 12/12-79 | Night | M/T Tine | c.600 | idem. | |

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# Newport Bridge Collision

## Collision au pont de Newport

## Newport Brückenzusammenstoß

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Thomas R. Kuesel, born 1926, received two degrees in civil engineering from Yale University. Since 1947, he has been with Parsons Brinckerhoff Quade & Douglas in New York, where he is senior vice president and director of engineering and technology. He directed the preliminary design of the Newport Bridge from 1959 to 1963.

**SUMMARY**
In 1981 a main tower pier of the 488-m Newport suspension bridge in Rhode Island, USA, was struck head on by a fully laden 45,000-ton tanker. The ship was shortened 3.5 m through bow crushing, but the bridge pier suffered only superficial damage. Details of bridge design and accident are given, and forces developed during the collision are derived.

**RÉSUMÉ**
En 1981, un pétrolier de 45,000 tonnes en pleine charge, est entré en collision avec la partie inférieure d'une des tours principales du pont suspendu (488 m de longeur) de Newport, Rhode Island, USA. Cette collision, de plein fouet, a eu pour résultat de raccourcir de 3.5 m l'avant du navire. La pile cependant n'a subi que des dégâts superficiels. Les détails du projet du pont, et de l'accident sont présentés, de même que les forces développées pendant cette collision.

**ZUSAMMENFASSUNG**
Der Pfeiler der 488-Meter langen Hängebrücke in Newport, Rhode Island, USA, wurde 1981 von einem vollbeladenen, 45 000-Tonnen Tanker frontal gerammt. Durch das Eindrücken des Bugs wurde das Schiff zwar um 3,5 Meter verkürzt, dagegen erhielt jedoch der Brückenpfeiler nur Oberflächen-Schaden. Einzelheiten über den Brückenentwurf und über den Unfall sind dargestellt, und Berechnungen der Anprall-Kräfte sind angegeben.

## 1. THE BRIDGE

The Newport Bridge is approximately 3 km long, crossing the Eastern Passage of Narragansett Bay at Newport, Rhode Island (Fig. 1). The main shipping channel is bridged by a suspension span 488 m long, with a clear height of 66 m at the center of the channel. The water depth beneath the suspension spans ranges from 30 to 45 m. The tower piers are of "Potomac type" caisson construction, founded on steel H-piles driven into sands that fill the glacial gorge underlying the bay.

The bridge was completed in 1969. It was designed to provide clearance for aircraft carriers proceeding to U.S. Navy installations further up Narragansett Bay, as well as commercial shipping bound for Providence, at the head of the bay. Owing to the large expanse of very deep water, there is no defined shipping channel. Large vessels entering the bay from the ocean generally make a 45$^{o}$ port turn about 3 km below the bridge and a 15$^{o}$ starboard turn about 1 km below the bridge.

## 2. THE COLLISION

Shortly after noon on February 19, 1981, the bridge was struck by the tanker Gerd Maersk, which was proceeding up the bay toward Providence. The ship was fully laden with a cargo of fuel oil, and displaced 45,000 (metric) tons at the time of the accident. Navigating in a dense fog, the pilot had no warning

Fig. 1. Newport Bridge, Newport, Rhode Island, USA.

of the collision until the bow lookout cried out that the pier was dead ahead. The captain called for hard left rudder and full steam ahead (to increase steerageway), but there was insufficient room to affect the ship's course appreciably. The ship struck main tower pier 1E head on, nearly in its center and normal to the bridge axis. The estimated speed of the ship at impact was 6 knots.

## 2.1 Damage

The bow of the ship was crushed in to the extent that the ship was shortened 3.5 m (Fig. 2). The ship came to a complete stop and then drifted off. Although it took on some water through sprung plates, no oil was spilled, and the ship was never in danger of sinking.

Damage to the bridge was negligible. The pier suffered no displacement or rotation, and there was no misalignment of the roadway deck joints or any other superstructure elements. The side of the pier suffered extensive surface scrapes, gouges, and spalls of the concrete (Fig. 3) to a depth of 2 to 5 cm over an area roughly 7 m wide by 20 m high, approximately equally above and below the water line (Fig. 4).

There were a number of superficial tears in steel plating of the underwater caisson structure, but no damage to the concrete behind the plating. A spectacular blotch of gray paint decorated the side of the pier for several days before falling off.

Owing to the dense fog, the bridge operators, stationed at the administration building and toll booths about 2 km from the site of the collision, were unaware of the accident for some time. It is not known whether any motorists were on the suspension spans at the moment of impact. There were no eyewitness reports from the bridge.


Fig. 2. Collision damage to tanker.

## 3. STRUCTURAL DESIGN

Details and dimensions of the pier are shown in Figs. 5a and 5b. The dead weight of the pier (neglecting 12,000 tons buoyancy) is 32,000 tons, and the superstructure reaction to the piertop is an additional 11,000 tons.

The top of the pier pedestal is 8.5 m above sea level, and the bottom of footing is 37 m below sea level, or about 8 m below the natural bottom. This top layer of natural material was dredged out to permit placing the footing form on the excavated bottom. The overexcavated space around the footing was filled with dumped sand and covered with heavy stones for scour protection.

The pier is supported on 512 steel H-piles, driven an average of 28 m into the sand. The design capacity is 72 tons/pile. The footing and shaft are composed of reinforced structural tremie concrete, placed within steel shell forms that remain in place. The distribution slab and pedestal are composed of normal reinforced concrete placed in the dry in dewatered cofferdams.

It was considered impractical to provide free-standing fenders in 30-m water depth, and so the pier was designed for an arbitrary ship impact force of 1,650 tons, applied at Elev. +3 m. This was intended to represent the effect of a 20,000-ton ship traveling with a velocity component normal to the pier of 3 knots.

The (buoyant) dead load pile load is 60 tons/pile, and the design ship impact force applied normal to the bridge axis produced forces of $\pm$ 10 tons/pile in the outer rows. (The governing design case was dead load + quartering hurricane wind).

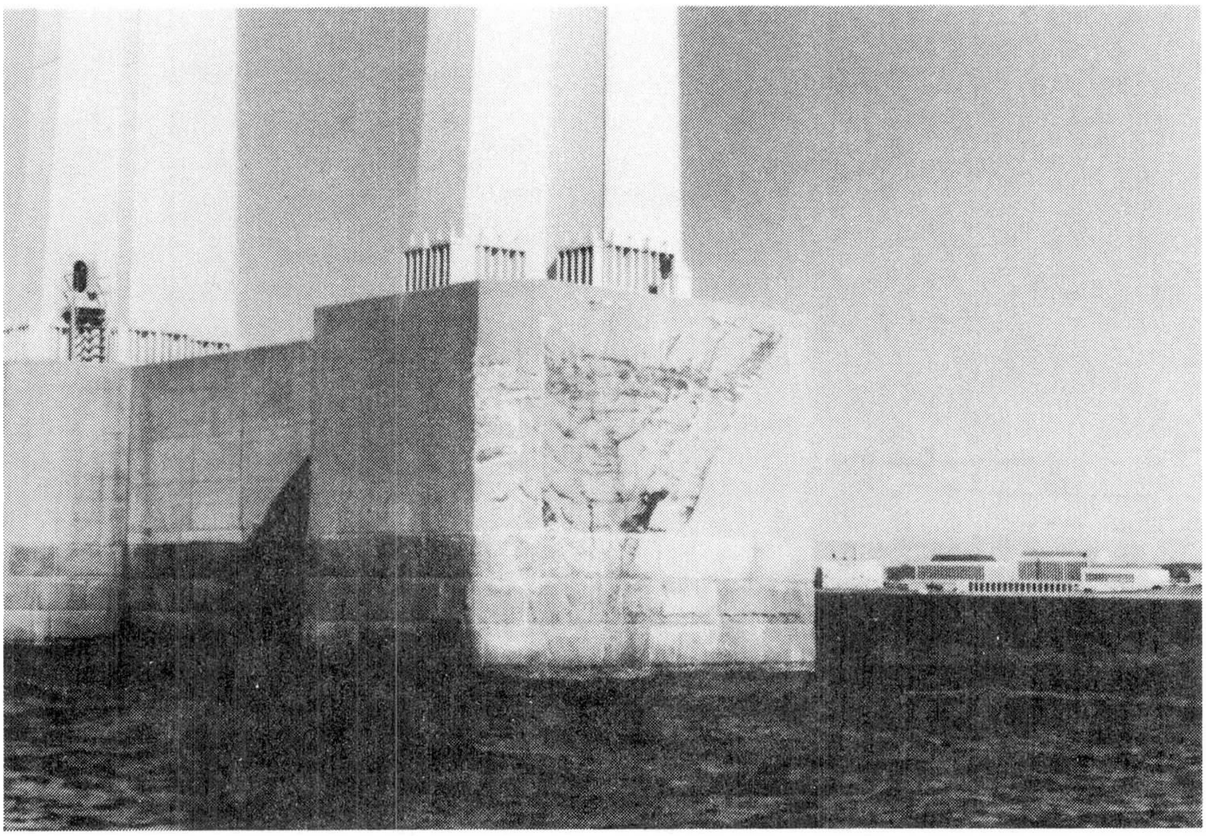

Fig. 3. Bridge pier damage.

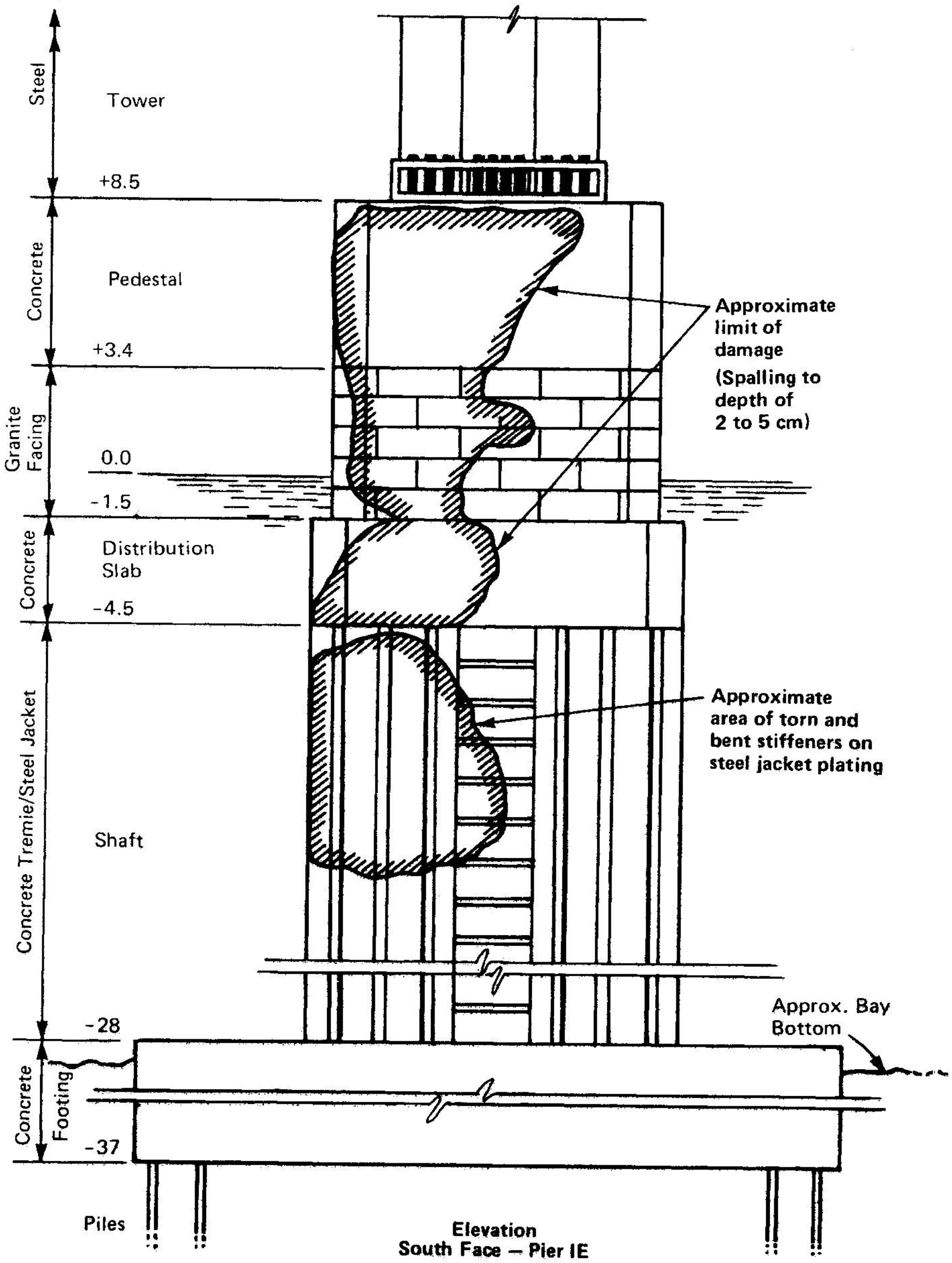


Fig. 4. Pier damage above and below waterline.

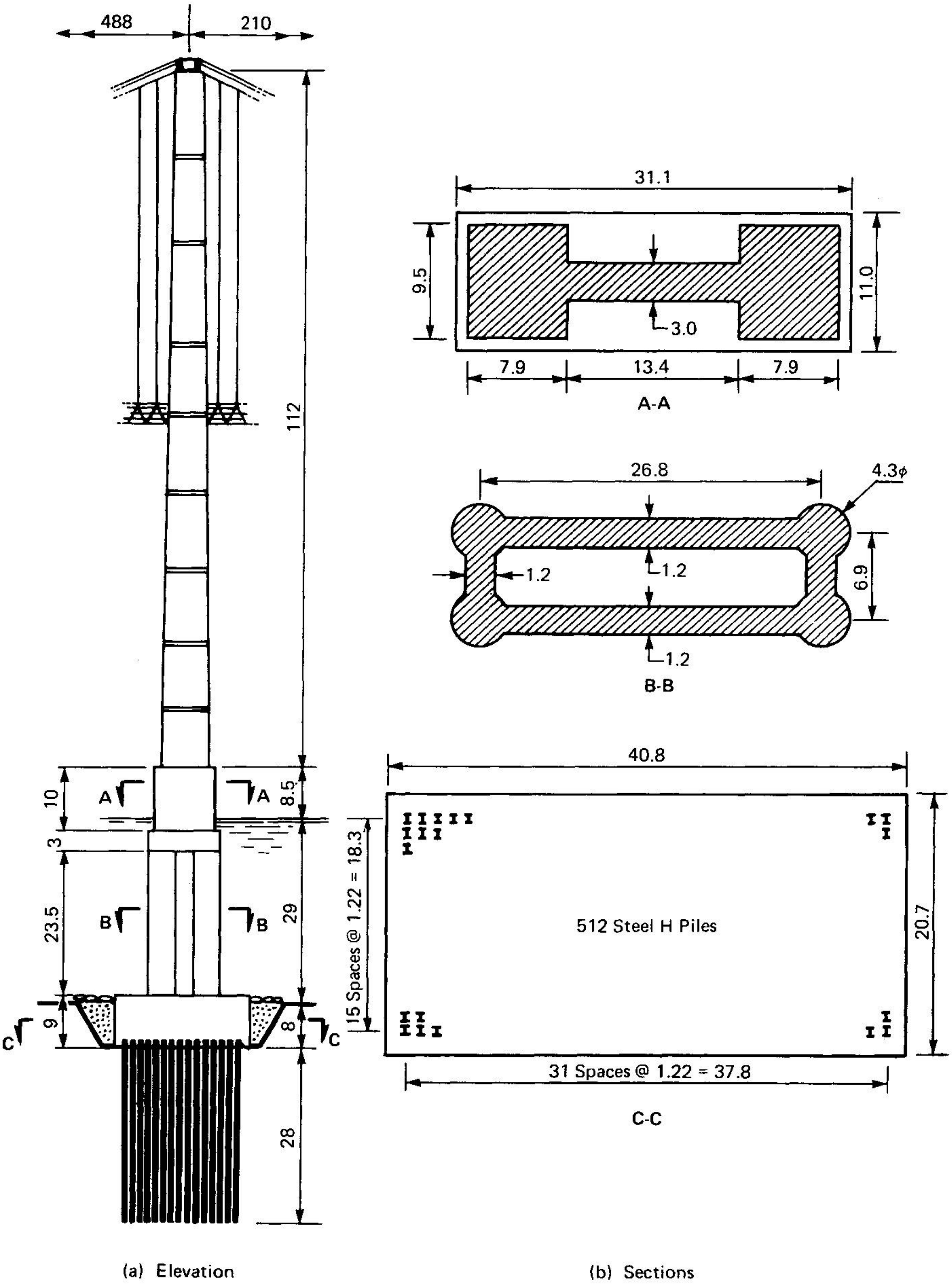


Fig. 5. Details and dimensions of pier.

## 4. COLLISION FORCES

For the actual collision, as a first approximation it may be assumed that the ship was decelerated from 6 knots (3 m/sec) to a dead stop in 3.5 m by a constant average force. For a 45,000-ton ship, this requires an average force of 6,000 tons applied for a duration of 2.3 seconds. The actual force-time-motion relation is of course more complex, and the instantaneous maximum force may have been 50% to 100% greater. Considering inertia and time effects, analysis of the stresses produced in the pier by a static horizontal force of 6,000 tons may give a feel for the expected performance of the pier.

Such an analysis yields maximum overturning forces of $\pm 70$ tons/pile, compared to the dead load of 60 tons/pile. Since the piles were driven to resistance based on load tests to twice the 72-ton design load, and the uplift resistance of piles driven 28 m into sand is very large, it is reasonable to expect that the actual impact would not produce permanent tilting of the pier. Neglecting any passive pressure of the backfill around the footing, the average horizontal shear of 12 tons/pile would not be expected to produce translation of the pier.

Analysis of stresses at the bottom of the shaft (top of footing) indicates that under a 6,000-ton collision load the resultant force lies close to the edge of the kern of the section. The maximum compressive stress in the concrete is about 35 kg/cm$^2$, and a very slight tension exists on the impacted side. This is well within the capacity of the reinforcing steel, without counting on the steel shell form plate.

The analysis thus indicates that although the actual collision load substantially exceeded that assumed for design, the observed lack of damage to the pier is consistent with the results to be expected from an approximate rational analysis. It further indicates that it is possible to design and construct deep water bridge piers to absorb the effects of collision from substantial large ships.

The Newport Bridge is fortunate that it has good foundations, and that the requirement to design for hurricane winds gives it an extra margin of safety against ship collisions, beyond that assumed for design.

It is of course preferable, and for extremely large ships mandatory, to attempt to divert the ship and absorb some of the collision energy in a deformable fender system, where site conditions permit such construction.

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# Recovery and Repair of the Second Narrows Railway Bridge

## Réhabilitation et réparation du pont ferroviaire de Second Narrows

## Wiederherstellung und Reparatur der zweiten Eisenbahnbrücke von Second Narrows

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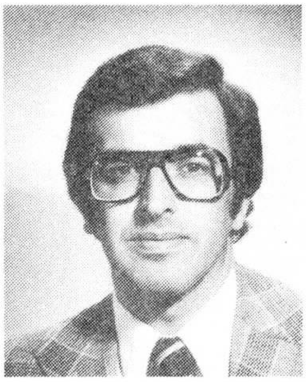
Donald Jamieson is a graduate of the University of British Columbia who, in the course of his professional career, has supervised the fabrication and erection of many large Canadian Bridges. He was in charge of the salvage and repair of the two bridges over the Second Narrows in Vancouver, Canada.



David Calder is a graduate of Glasgow University and Imperial College, London. He was the project engineer for the erection of the Second Narrows Railway Bridge in 1969 and is now Manager of the Civil Division of Swan Wooster Engineering.

**SUMMARY**
The paper describes the damage caused by a ship collision with the Canadian National Railway Lift Span in Vancouver, British Columbia. The theoretical and practical consideration of the stabilisation and recovery schemes are discussed and the equipment and procedures used in the twenty weeks it took to restore the train service are described in detail.

**RÉSUMÉ**
L'article décrit les dégâts causés par un navire entré en collision avec le pont de la société nationale des chemins de fer canadiens, à Vancouver, Colombie britannique. Les aspects théoriques et pratiques de la méthode de stabilisation et de réhabilitation sont abordés. Le matériel et les techniques mis en oeuvre durant les 20 semaines nécessaires au rétablissement des liaisons ferroviaires font l'objet d'une description détaillée.

**ZUSAMMENFASSUNG**
Der Artikel beschreibt den durch einen Schiffsaufprall auf die Brücke der Canadian National Railway in Vancouver, British Columbia, verursachten Schaden. Die theoretische und praktische Erwägung der Stabilisierungs- und Wiederherstellungspläne werden erörtert, und die verwendeten Anlagen und Verfahren werden genau beschrieben, welche zur Wiederherstellung des Zugverkehrs innerhalb von zwanzig Wochen benötigt wurden.

On October 12, 1979 the Canadian National Railway Bridge over the Second Narrows of Burrard Inlet, British Columbia, was extensively damaged when struck by the heavily laden vessel "Japan Erica". The ship was outward bound from Port Moody in a dense fog. The vessel struck the 77 m north tower span near its mid-point while the 152 m lift span was in the raised position at the top of the towers.

Although the ship was moving dead slow the impact knocked the north end of the span clear of the pier and displaced it laterally the width of the bridge (Fig. 1) by pivoting around the central wind post of the main pier.

The wind post prevented complete displacement of the span as the 12-64 mm anchor bolts in the pier members were sheared. The span was skewed approximately $9^{o}$ to the axis of the bridge and the north end dropped to a gravel bar at the foot of the pier.

The tower lost substantial support as the bearings were displaced to the extent that they were partially projecting over the N.E. and S.W. edges of the main pier. In this precarious state of equilibrium the lift span, which had been torn from the vertical tower guides (Fig. 3) was left hanging, hammock-like, from the lift ropes.

On the basis of a hastily prepared examination and submission the firm of Canron Inc., Western Bridge Division, was commissioned to restore the bridge to service. The plan, as submitted, envisaged three phases of the work: stabilization of the structure, recovery of the fallen span and restoration to service.

## Appraisal of Damage

Figure 1

The initial appraisal of damage indicated a dramatic series of instantaneous failure mechanisms. The severe damage was limited to the bottom chord and web system between panel points (p.p.) L2 and L4 (see Fig. 2 for p.p. numbering). This damage permitted the remaining portion of the span to rotate against a plastic hinge which formed in the top chord at U2. An examination of the point of impact of the vessel at L3 failed to show any vertical striations in the paintwork indicating the explosive suddenness of the lateral displacement. Except for one member in the immediate area of impact, the top lateral system was undamaged, as was the portal bracing system.
All the bottom laterals which were visible from L0 to L6 displayed compression failure, although the floor beams and stringers in the undamaged panels did not exhibit any distress.

The top chord members from U7 to U2 were virtually undamaged except for two significant areas. The first was the plastic hinge in the chords and the second was the compression failure of the diagonal brace from U3E to U2W. The latter was of great importance during truss separation.

The tower span was thus acting as a bent shore resisting the over-turning thrust from the northwest corner of the lift span which was bearing against the west leg of the tower. The intensity of bearing was a matter of considerable conjecture. No visible distortion of the tower leg was apparent nor were there any abrasion scars on the south face. The jacking girder in the north tower diplayed some damage due to the reaction on the wind post. Otherwise, the main members in the north tower were undamaged.

There were no outward signs of distress in the winding machinery. The sheaves were free of galling and the enamelled mechanical components did not exhibit any paint crazing. Couplings, drives, pinions and gears were inspected and were found to be free of any external indication of strain.

Examination of the lift span was not possible during the initial appraisal due to inaccessibility. Inspection the following week, however, indicated that while there was substantial crushing of the 60 mm diameter lift ropes where they passed through the lift girder, and some surface abrasion caused as the ropes were torn through the weather skirting of the towers, they were otherwise capable of carrying the 1000t reaction of the span during the salvage operations. The top flanges of the lift girders, however, which were normally stayed against lateral deflection by a tie to the top lateral system, were bowed outward by the horizontal component of the 40 lift ropes at each end. This reaction sheared the connection of the girder tie and bowed the north girder outward 76 mm and the south girder 44 mm. The top flange of the lifting girder, fortunately, was the tension flange and such distortions did not create a buckling problem. The condition of the north girder was considered to be inimical to the safety of the structure.

Immediately following the accident the Harbour Master closed the passage to heavy marine traffic but permitted smaller vessels such as tow boats and pleasure craft to use the channel under the south tower span. The wisdom of this order became apparent during the initial damage survey when it was noticed that as each vessel made the transit of the south channel the vibration of the vessel's screw could be felt in the higher elevations of the south tower. It was abundantly clear that the equilibrium of the system was extremely sensitive, so on the second day after the accident a series of survey check points was established on the north and south towers and main piers. Concurrently with the survey, divers were engaged to evaluate and report on the conditions of bearing of the north end of the fallen tower span on the gravel bar.

At this time the philosophy of the salvage and recovery system was firming up and it was determined that the first stage of the work should be the stabilization of the structure.

STABILIZATION MEASURES

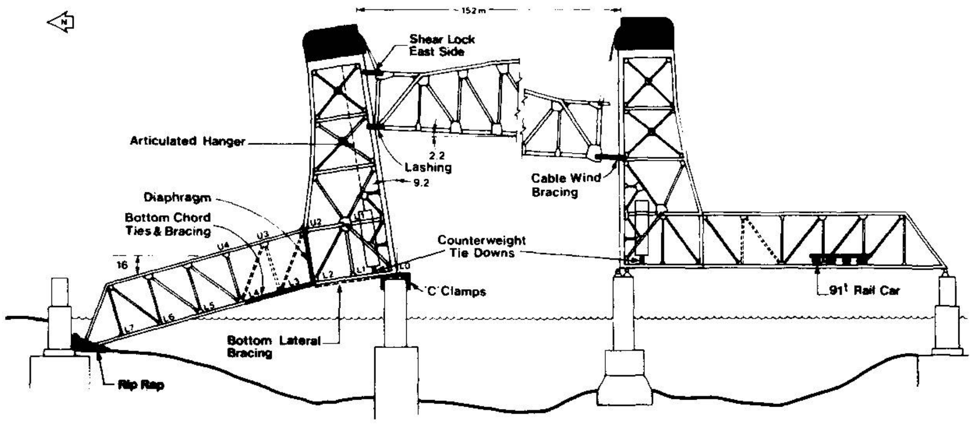



Figure 2

## Stabilization

Figure 3

Upon receipt of the instruction to undertake the work the Canron engineering group directed their efforts toward the predominant need to stabilize the structure and to prepare recovery procedures. Stabilization included the immediate installation of wire rope diagonal bracing between the lift span and the south tower and shear devices between the lift span and the north tower. Concurrently with this work the north lift girder was stayed against further movement and the 1000t counterweight in the north tower was prevented from further downward movement by the installation of articulated platework hangers capable of carrying the total weight. Upward movement of north and south counterweights was prevented by the rigging of heavy down-haul tackle.

To ensure that the south tower span did not tip under any sudden increase in the horizontal component of cable forces, a 91t railcar was spotted over the south end of the south tower span.

In the meantime crews were installing heavy C-clamp type weldments between the foot of the tower and the main pier while others were clearing away the span locking equipment of the north main pier and removing the deck and damaged steel in the way of temporary works.

Due to the loss of the web members in panel L2 to L4 it was necessary to inhibit any further tendency to hinge at U2. Accordingly, heavy structural sections were cut and fitted on site to make the bottom chord continuous from L2 to L4. A temporary vertical U3-L3 was installed and the vertical bracing in the plane of the north face of the tower was extended down through the truss to L2. The bottom lateral bracing system was restored from L0 to L2 and so transformed the tower portion of the structure into an integrated vertical box truss, a condition which was to present some problems during subsequent operations.

Upon completion of stabilization the passage was re-opened to deep water shipping and the upper docks resumed normal activity. From the outset of the work divers carried out a daily check on the stability of the gravel bar on which the north end of the span rested. The bar was subject to heavy tidal currents of the flood tide and hence destructive scouring. To prevent scouring, rip-rap was placed at L8. The underwater survey was carried out daily until completion of the salvage operation.

During the work of stabilization a final appraisal of the damaged structure provided the information necessary to complete the engineering analysis and design.

## Engineering

The scope of the work and complexity of analysis engaged not only Canron's staff of seventeen engineers and draughtsmen but also required the services of nine consulting engineers of various disciplines.

Paralleling the work of stabilization, loads and stresses were being analysed from calculations made from the original centres of gravity of the structure, physical measurement of loads in the lift ropes and, by inference, from the effect of the bearing of the bottom chord against the west leg of the north tower.

These calculations were verified in sense and in degree by a series of readings of residual stresses in sensitive members and joints which were obtained by the "blind hole" drilling method of photo-elastic strain analysis.

Design of falsework and procedures, however, did not await the receipt of this information but, instead, were designed on the basis of upperbounds and verified by model studies.

The scope of this paper does not allow for a detailed description of the analytical steps. However, the interrelation of model tests, computer analysis, field measurement and upper and lower bound scenarios was germane to the fast tracking of the recovery scheme. The total elapsed time from accident to first train crossing was only 20 weeks and, from the amount of falsework and equipment that had to be designed and fabricated, it is obvious that engineering was on the critical path.

Figure 4 shows the accuracy that the various analyses achieved by plotting the jacking forces at L2 as the tower rotated to the vertical. The lower bound calculations were derived from measured forces and theoretical analysis and, as can be seen, are very close to the forces predicted by model analysis.

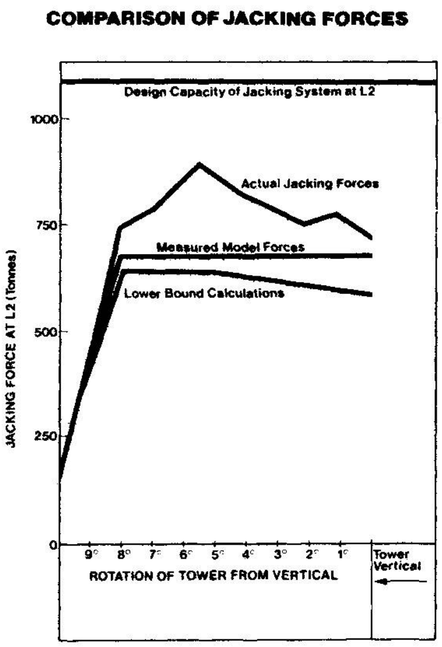



Figure 4

The actual values had a disturbing peak in them which was caused by a jammed shear lock. Without this occurence it is believed that the measured values would have paralleled the model values closely. Even with the jamming, the maximum jacking forces did not come close to the upper capacity provided in the recovery scheme.

The design capacity had been conservatively estimated by considering the tower weight and ignoring the restraining forces from the inclined cables.

## Recovery

The philosophy of the recovery procedure envisaged isolating the lift span and tower from the partially submerged portion of the span. This required the provision of falsework under the back legs of the tower of sufficient capacity to carry the heaviest vertical reaction and provide a base large enough to carry a jacking frame, and a transversing system which would be required to re-align the tower after plumbing (Fig. 10). Support for this falsework, which was designed for 1091t, was provided by two groups of 12 - 500 mm dia. pipe piles supporting a transverse girder system of adequate size to

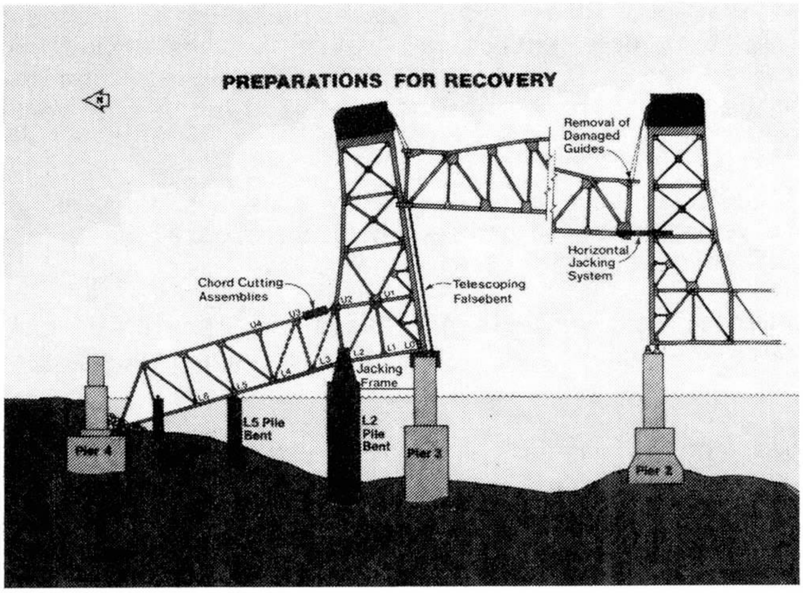


Figure 5

carry the jacking-skidding system (Fig. 11). This platform braced to Pier 3 with 610 mm dia. pipes. The attachment to the pier was made with Dywidag bars in holes cored through the foundation legs. 914 mm dia. pipe piles supported falsework of 455t capacity at L5, and 91t capacity at L7. The L5 and L7 bents were necessary to support the partially submerged span after separation from the tower. Bent L5 was designed to act as a pivot bent on which the severed truss span would rotate as it was raised to the horizontal for subsequent dismantling in large sections.

## Construction of Pile Bents

Installation of the piles by a vibratory hammer was elected due to the sensitivity of equilibrium of the system which could have been upset by the dynamic action of a reciprocating pile hammer.

During the driving of the 500 mm dia. pipe piles the tides produced currents of up to 8.4 km/h which engendered destructive vortex shedding in the piling and produced several modes of vibration.

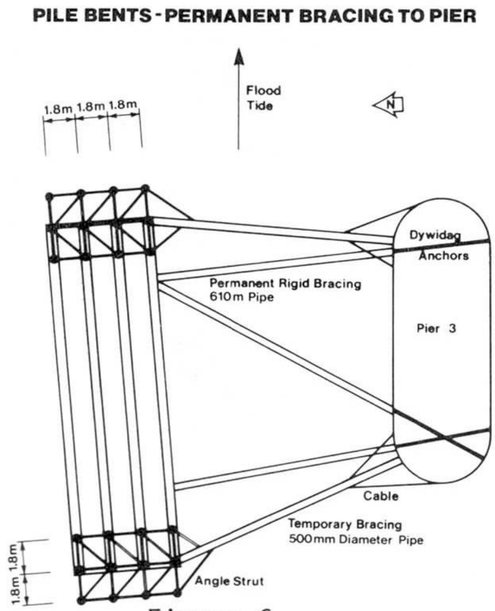


Figure 6

Figures 6 and 7 show the final configuration of the L2 bent. The table beside Figure 7 lists the variety of oscillations observed in the bent during construction. The amplitudes in most cases are visual estimates and therefore, are not too accurate. The measurement that is guaranteed, however, is the Stage 1 plus or minus 900 mm value. In this case the piles were driven at 1800 mm centres and during oscillations actually made contact with each other. Tidal flow and frequency were accurately measured. Vibrations only occurred for about 1 to 2 hours during every other tide during a two week period. It was unfortunate that the construction schedule did not fully coincide with the tidal cycle. Attempts to damp out the oscillations with ropes attached to the piers were unsuccessful. It was fortuitous that the oscillations almost always occurred after the dayshift was complete (7 p.m. to 9 p.m.) or before it started (3 a.m. to 5 a.m.) and work could always continue on bent completion during daylight hours with no oscillations. The Stage II oscillations came as a considerable surprise since group oscillation was considered to be unlikely. The east pile group survived the first set of oscillations, but top bracing of the west group was partially destroyed and was subsequently re-installed.

Temporary 500 mm dia. tube struts were installed from the bents to the piers which eliminated the violent crossflow vibrations but replaced them with plus or minus 76 mm in-line vibrations.

The jacking bent was completed as designed with 600 mm dia. pipe bracing and no serious vibrations were detected thereafter.

**TYPICAL SECTION THROUGH PILE BENT**

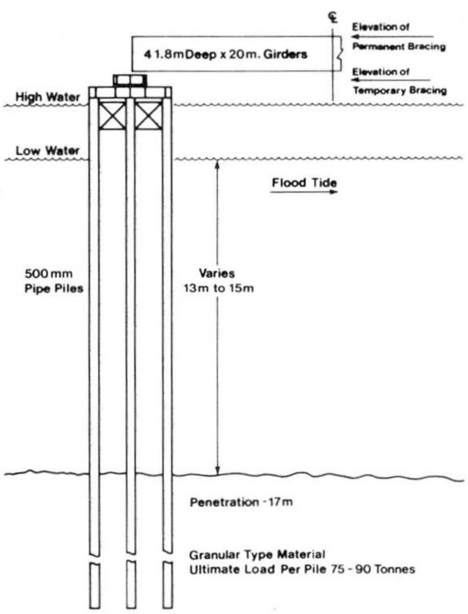



Figure 7

SUMMARY OF PILE VIBRATIONS

| STAGE | HALF AMPLITUDE | DIRECTION | CYCLES/ MINUTE | CURRENT (KNOTS) |
|---|---|---|---|---|
| I | 900 mm | N/S crossflow | 35 | 4 to 6 |
| II East | 220 mm | N/S crossflow | 50 | 4 to 5 |
| II West | 450 mm | Do | 35 | 4 to 5 |
| III | 75 mm | E/W in line | 50 | 2 to 5 |
| IV | 75 mm | Do | 50 | 2 to 5 |
| V | 7.5 mm | E/W | 180 | 5.4 |
| | 2.5 mm | Bowstring | 180 | 4.0 |
| | .75 mm | E/W Random | Random | 4.0 |

The jacking bent was capped by 4 - 1800 mm deep plate girders which supported the jacking frames during the jacking stage and later supported the skidding frames as the tower was re-aligned. The jacking frames each contained two welded channel members braced in position. A box beam which spanned between these members supported the truss which was raised through the box beam by two 318t jacks in each jacking frame.

## Structural Separation and Re-alignment (Fig. 9, 10, 11)

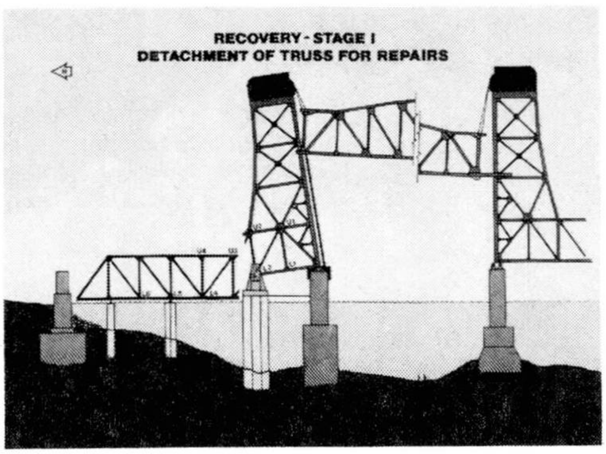



Figure 8

Prior to separating the tower from the horizontal truss, the tie-downs were installed to the box beam and the 318t jacks at L2 were loaded to provide a positive reaction of 45t per side. This ensured no sudden movement of the tower when the chords were cut. The pivot bearings on bent L5 were brought into contact by flat jacks and shimmed in place and thus prevented sudden movement during chord cutting.

The lift span was supported by a pin-connected telescopic jacking bent supported on the main pier in front of the north tower. This bent did not carry load initially but was brought into light contact only with the lift span by two 363t jacks in each leg of the bent. Support under the lift span was through a teflon-coated rocker bearing on a traversing girder. This bearing allowed easterly traversing of the lift span as the tower was plumbed.

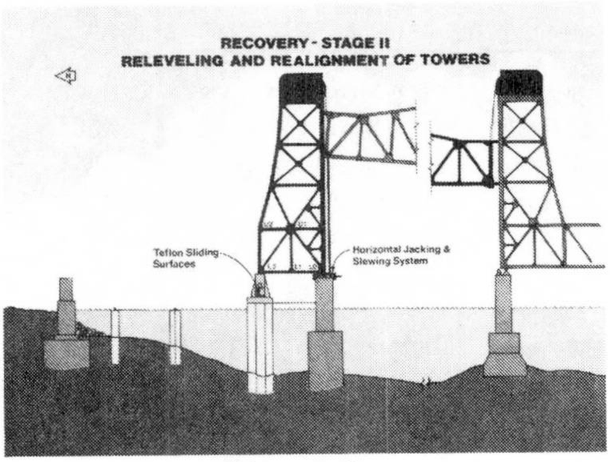



Figure 9

Cutting frames were installed on the east and west top chords and the U3W-U2E diagonal. The chord cutting frames contained four 180t jacks and the diagonal frame two 90t jacks.

The temporary bottom chords and the crippled top lateral bracing from U2 to U3 were subsequently parted. The U3W-U2E diagonal was cut using oxy-acetylene torches and the load released by retracting the jacks. The top chords were then cut simultaneously by use of oxygen lances. The east chord was the first to be cut completely and showed to be in a state of zero stress. The load in the west chord was released after cutting by retracting the jacks. The maximum measured compression load was 220t as the tower reaction was transferred to the jacking frame.

The jacking frame supported the tower through a teflon-coated rocker contained between two 55t jacks.

As the bottom chord was raised the jacks were activated to ensure that the frame remained plumb as the locus of the point of contact moved first to the south and secondly to the north.

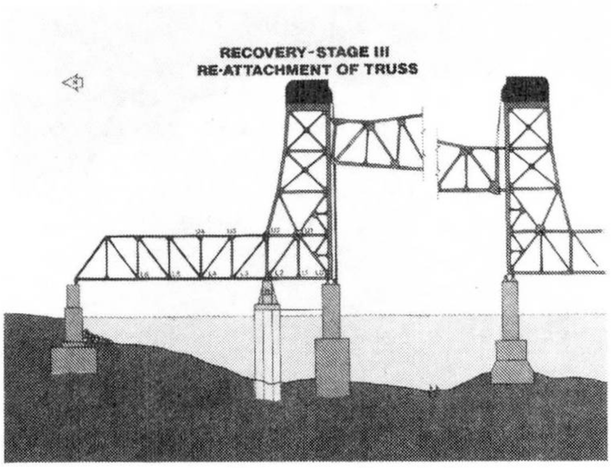



Figure 10

Total lift required to bring the chord to geometric elevation was 3 m. The box beam was raised in 400 mm increments using a climbing system employing 190 mm diameter retaining pins. Controls during jacking monitored both individual jack loads and, by an ingenious system of lights, the level of the four corners of the box beam. Maximum movement of the head of the jacking frame relative to the node point was 150 mm to the south, thence 25 mm to the north.

As the tower was jacked to the vertical the effect of the rack in the span, which was now contained by the vertical bracing, caused the jacks to be more heavily loaded on the east side than those on the west. At one time this disparity in loads increased uncomfortably close to the design loading of the easterly pile group.

The locus of the tower top during jacking was south east while the lift span was supported N-S. Therefore, during the jacking of the tower it was necessary to traverse the lift span to the east as the weight of the lift span was transferred to the bent.

The pier reaction of 2636t from the tower was subsequently transferred to two skidding sleighs each containing three 509t jacks. The jacking frame at L2 was replaced by a skidding frame which enabled the tower to be slewed into alignment and on to chainage. The tower was pivoted through a structural system on the main pier. The skidding sleighs, which were controlled by opposing jacks on a fixed radius from the pivot, were carried on a teflon bearing surface against a polished steel sheet.

Similarly, the skidding frame at L2 was supported on teflon bearings moving on a polished steel surface. Total movement at L2 was 3100 mm.

Concurrently with the jacking-slewing operations, the submerged portion of the span was raised level, stripped of damaged members and dismantled as two trusses. These trusses were set up vertically on two scows and towed to a yard where replacement members were erected. The two 155t trusses were then returned to site and erected to the tower span using a 364t capacity marine crane. The floor system and chord bracing were installed concurrently with the removal of the bent at L2.

With the completion of the north tower the false bent under the lift span was lowered in decrements of 400 mm as for the jacking at L2, transferring the reaction of the lift span back to the lift ropes.

On the morning of March 3 the lift span was lowered, using a combination of tackle and the winding machinery. On March 4 the span was opened to rail traffic.

Although the ropes had suffered only minimal damage it was deemed to be prudent to replace them while it was opportune to do so without any great inconvenience to marine or rail traffic. Therefore, while the bridge was open to rail traffic and with the harbour closed for the next 16 days, crews worked around the clock to replace 80 - 60 mm diameter lift ropes.

Figure 11

Only twenty weeks elapsed from the date of the accident until restoration of rail service. It was a feat made possible by the enthusiastic support of all who participated in the work; the engineering community, draughtsmen, fabricators, sub-contractors and the transportation groups; the marine contractors who, in recognition of the exigency of the work, made available their heavy lift water borne equipment, and above all the ironworkers, whose performance under severe conditions of weather and risk carried the work through to a successful conclusion.

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# Case Stories of Dolphin Accidents and Remedies

## Accidents aux postes d'amarrage et remèdes

## Zusammenstöße mit festen Bojen und Lösungen

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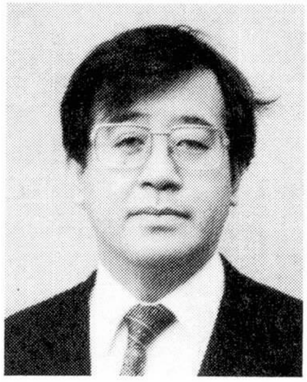

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**SUMMARY**
The report outlines recent ship collision accidents with dolphins in Iwakuni, Japan and their remedies.

**RÉSUMÉ**
Ce rapport traite d'accidents causés par la collision des cargos avec les postes d'amarrage à Iwakuni, Japon et ses remèdes.

**ZUSAMMENFASSUNG**
Es wird über Zusammenstöße mit festen Bojen in Iwakuni, Japan berichtet. Lösungen werden vorgeschlagen.

## 1. GENERAL

In and around the Japanese waters, marine accidents have occured frequently. About 2,300 vessels of which 50-60 % are foreign vessels meet with accidents and about 400 passengers and crewmen die or are missing annually.

Marine accidents are classified into ten types i.e. collision, stranding, engine trouble, fire, etc. and ship collisions are about 16 % of the total number of these marine accidents.

We have investigated ten cases of marine accidents, in which oil tankers and cargo vessels collided with dolphins. In the following pages two collisions which occured at Iwakuni, Yamaguchi Prefecture, Japan in 1978 and 1979 are investigated.

The harbour structure of concern here has a landing berth for wooden chips for paper-making and the berth was designed for foreign chip carriers with 53,600 dwt. As shown in Fig.1 the berth consists of 1 main-breasting dolphin, and 2 sub-breasting ones, and 4 mooring ones.

The north sub-breasting dolphin (B-3 in Fig.1) was damaged in the first collision in 1978, and the south mooring dolphin (M-1 in Fig.1) was again damaged in 1979.

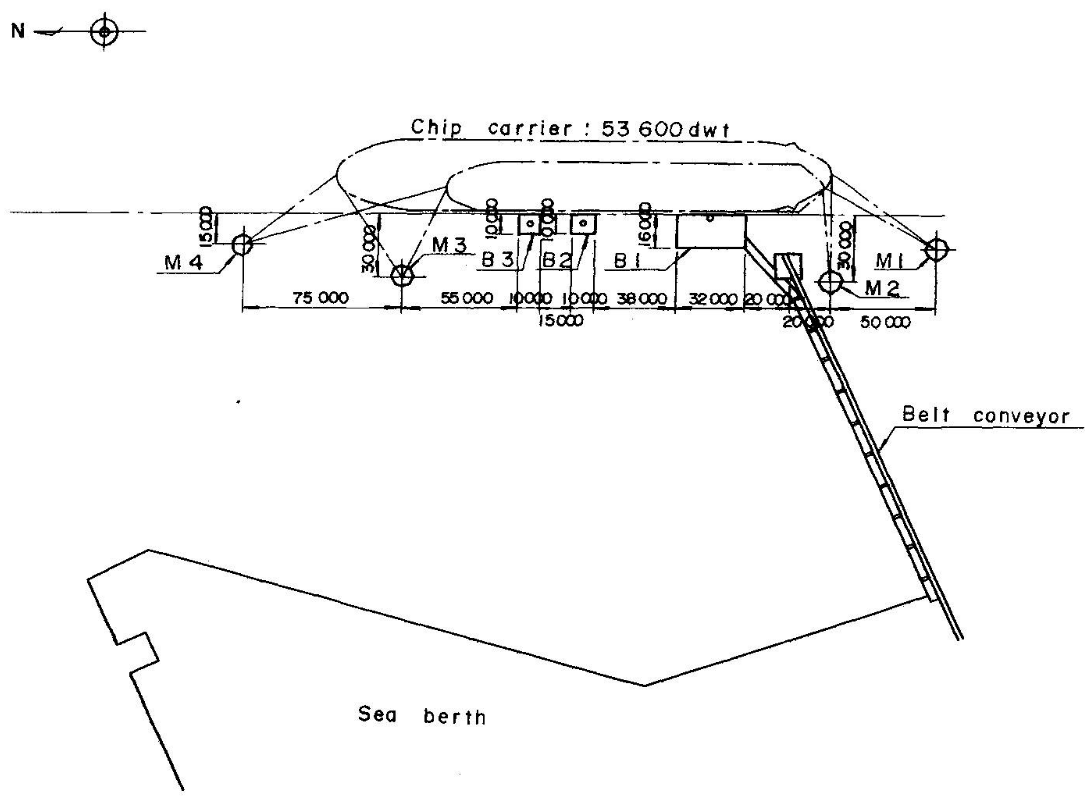



Fig.1 Outline Plan of the Berth

## 2. STRUCTURE OF THE DOLPHINS

### 2.1 Breasting Dolphin B-3

As shown in Fig.2 the Dolphin B-3 consists of 9 steel pipe piles (914.4 mm in diameter, 16.0 mm in thickness, and 41.5 m in length) and the top concrete deck. The structure has the flexibility to absorb the kinetic energy of ship berthing for an approaching velocity of 15 cm/sec. That is, the resisting capacity against impact is maintained by the mutual effect between the rigidity of the steel pipe piles and the viscoelasticity of the foundation strata.

### 2.2 Mooring Dolphin M-1

In the structure of the Dolphin M-1, 9 steel pipe piles (609.6 mm in diameter, 12.7 mm in thickness, and 40.5 m in length) composed of 8 radial battered piles and 1 vertical pile are rigidly connected with the top concrete deck. This mooring dolphin has a rigid structure in contrast with the breasting dolphin, that is, the static load worked through mooring ropes is borne by the structure so that a ship can keep a certain distance with the berth.

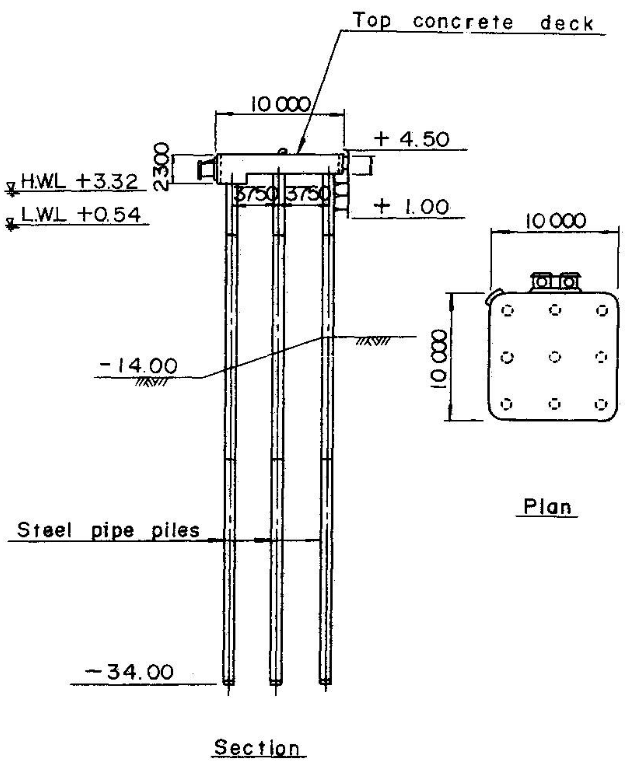



Fig.2 Structure of Dolphin B-3

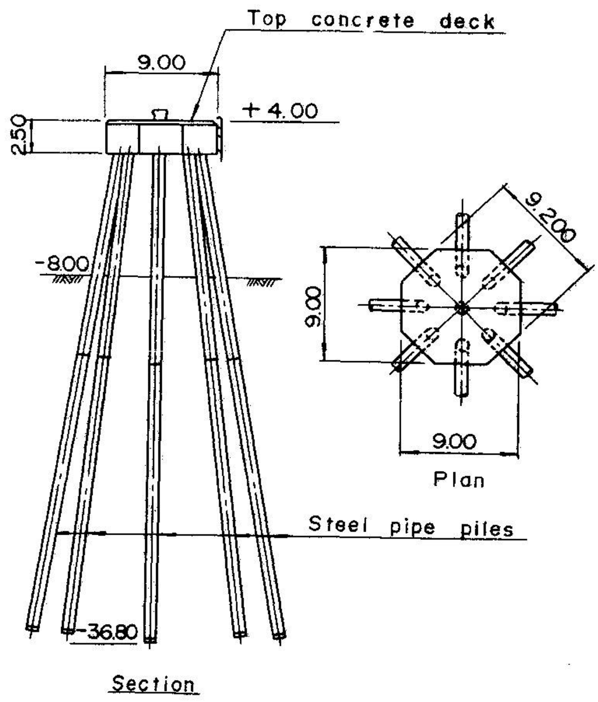



Fig.3 Structure of Dolphin M-1

## 3. CIRCUMSTANCES OF THE ACCIDENTS

### 3.1 The First Accident

The circumstances of the first collision accident in 1978 is described as follows: (See Fig.4)

While the vessel was approaching on her starboard side to the sea berth, the ship swung her stern towards the right side in order to maintain the correct direction for her berthing. After that, the side of the vessel collided with the fender on the corner of the north sub-breasting dolphin B-3.

The fender itself had no function to absorbe the impact energy of ships, therefore, the inertia force of the moving ship was directly loaded on the dolphin structure, and the foundation piles were deformed.

The particulars of the vessel were as follows:

| | |
|---|---|
| Tonnage: | 40,000 GRT |
| Length O.A.: | 210 m |
| Breadth mld.: | 30 m |
| Depth mld.: | 20 m |

### 3.2 The Second Accident

As in the first accident the same cargo vessel, in 1979, lost proper speed while she was approaching the berth from the north, then collided with the south mooring dolphin M-1.

Both collisions were caused by the mistakes of the operators. But, as described in the beginning of the article, Japanese coastal/offshore structures are located in very confused sea areas and it is one of the reasons of the frequent occurence of ship collision casualties.

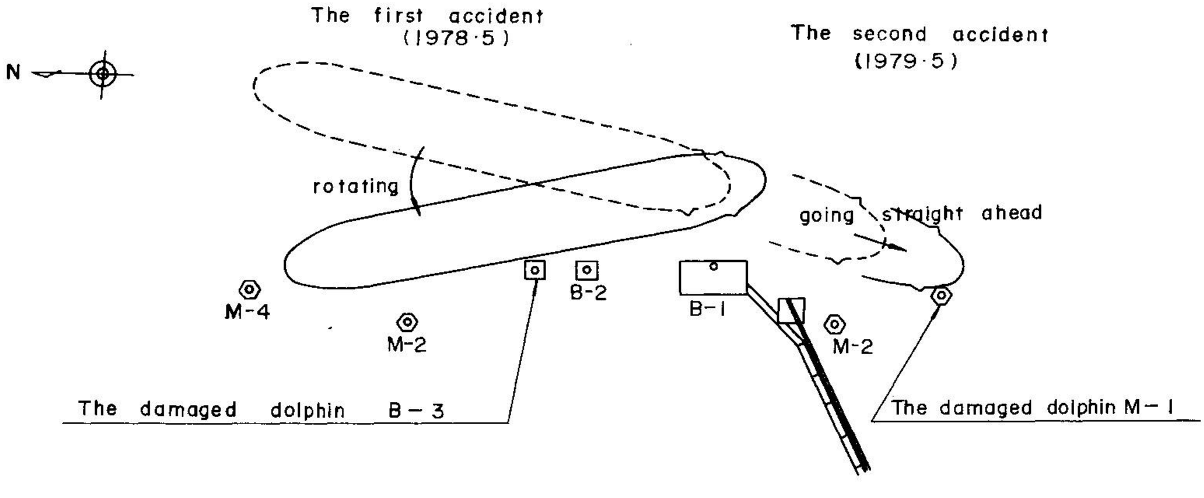



Fig.4 Geometry of the Collisions

## 4. DEFORMATION BY THE COLLISIONS

### 4.1 Breasting Dolphin B-3

The results of our inspection carried out immediately after the accidents are briefly as follows: (See Fig.5)

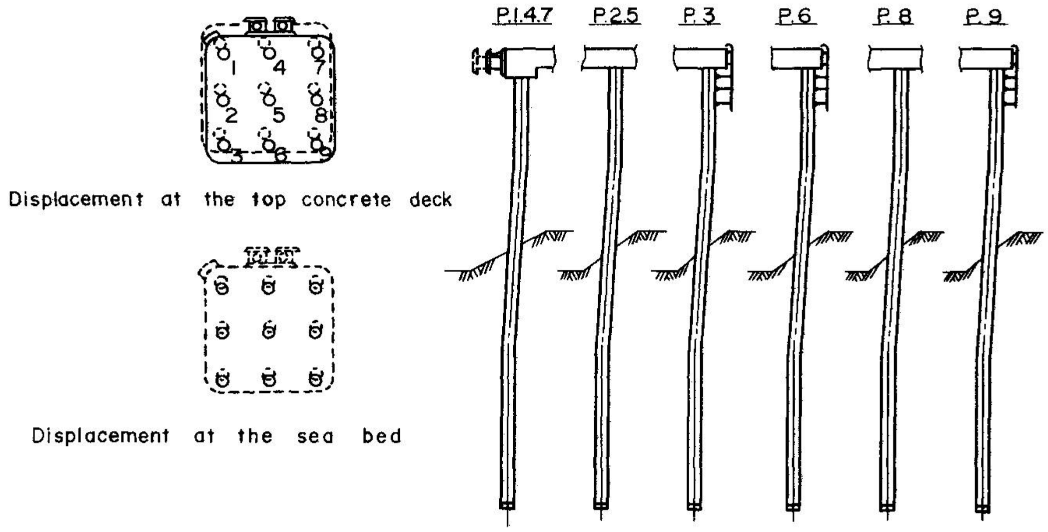



Fig.5 Deformation of Dolphin B-3

#### 4.1.1 Deformation of the Top Concrete Deck

Upon measuring the damaged dolphin, the deformation was found to be the residual displacement of the top concrete deck:

Horizontal displacement:

In the direction of the berthing line ----- 25.2 cm
In the direction of right angle to the berthing line ----- 97.2 cm

Vertical displacement:

Inclination towards the shore on the diagonal line ----- 0° 20'

#### 4.1.2 Deformation of the Steel Pipe Piles

Upon inspecting the piles in the water, the buckling on the piles were found near the welded joints of No.3,6,8 & 9 piles 2 m deep in the water and the shells thereon had been torn off.

The deformation by the collision was more severe on the opposite piles of the collision side. From that fact it was presumed that the dolphin was rotated by the collision impact (the horizontal force) so that those opposite piles were loaded by both the force at a right angle to the axes and the axial force (the compressive forces).

Further, the maximum deformation was found on the middle piles (12.7 mm in thickness) while the upper piles above D.L. -0.50 m were still vertical with little deformation. The reason for this is that the materials of the upper piles were thicker (16.0 mm in thickness) and they were stuffed with filling concrete.

#### 4.1.3 Deformation of the Ground

After the diving inspection, voids of 40 cm in width and 100 cm in depth were observed in the sea bed on the offshore side of the piles. It indicated that plastic deformation in the ground occured by the energy exerted by the collision.

### 4.2 Mooring Dolphin M-1

#### 4.2.1 Deformation of the Top Concrete Deck

An opening made by the collision was found at the junction of the top concrete deck with the steel pipe piles. This opening was 20 cm at its maximum in a direction of north-west. At the same time a wedge from the shearing force (1.6 m in width, 0.2 m in height, and 0.4 m in depth) appeared in the concrete deck.

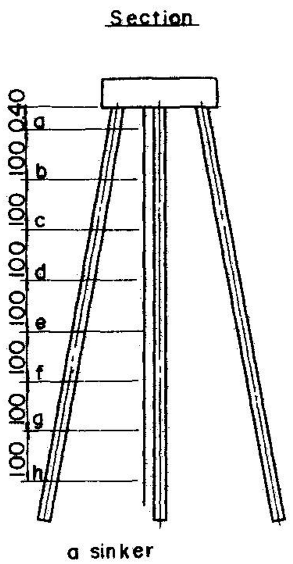
#### 4.2.2 Deformation of the Steel Pipe Piles

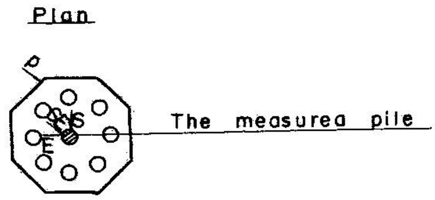
Upon measuring the deformation of the central vertical pile in three directions, the maximum displacement of 25 cm was observed in the direction of south-east. (See Fig.6)

Shells of barnacles and sea mussels had been torn off around the tidal zone at the top of the piles, and it indicated the hysteresis of deformation.

#### 4.2.3 Deformation of the Ground

The sea bed was covered with mud so that deformation of the ground could not be inspected there.




Displacement

(Unit: m)

| | S | E | SE |
|---|---|---|---|
| a | 0.480 | 0.505 | 0.508 |
| b | 0.455 | 0.485 | 0.490 |
| c | 0.425 | 0.460 | 0.440 |
| d | 0.400 | 0.440 | 0.400 |
| e | 0.370 | 0.410 | 0.360 |
| f | 0.345 | 0.400 | 0.330 |
| g | 0.320 | 0.360 | 0.300 |
| h | 0.290 | 0.350 | 0.260 |
| | 0.190 | 0.153 | 0.248 |

Fig.6 Deformation of Dolphin M-1

## 5. METHOD OF RESTORATION

In the case when a structure receives an impact of excessive energy over the designed value, the structure cannot perfectly rebound to its original state but will have residual deformation and stress in itself even after removal of the load. It seems that the residual deformation is caused by both the change in the steel material which reaches the plastic range over its yield point and the plastic deformation of the ground.

From a technical point of view, as usual, it used to be said that a structure with residual deformation, whose material suffered stress hysteresis over its yield point, would be very dangerous if re-used. Especially for a remedy of a damaged dolphin which is constantly loaded with forces again and again, a dismantlement and re-construction method is generally adopted.

However, on the basis of an accurate stress analysis of the materials, we selected a reinforcement method with additional piles, in which the structural system of the dolphin was not changed but improved in order to keep its stability. That is, if the residual yield strength of a damaged structure with residual deformation is accurately estimated, the structure can be restored by redistribution of the stress from its reinforcement. (See Fig.7 & 8)

In conclusion, we assume that it will be more valid and of more general applicability to accurately estimate the residual function of the damaged structure in order to select a restoration method for the same structure, when the structure is not entirely destroyed by a collision.

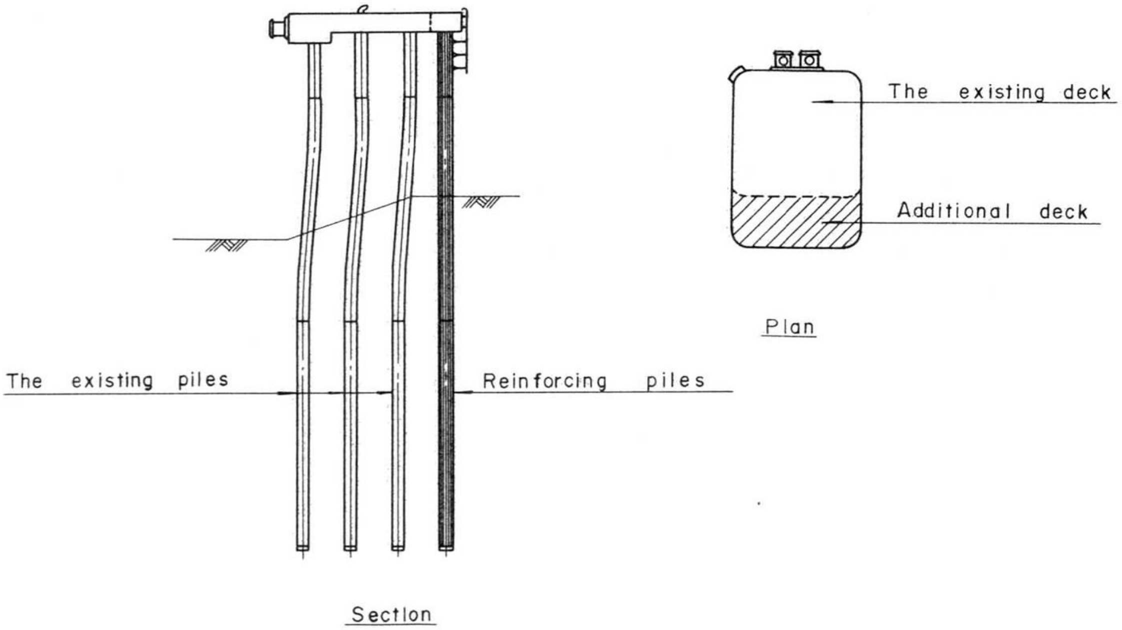



Fig.7 Restoration of Dolphin B-3

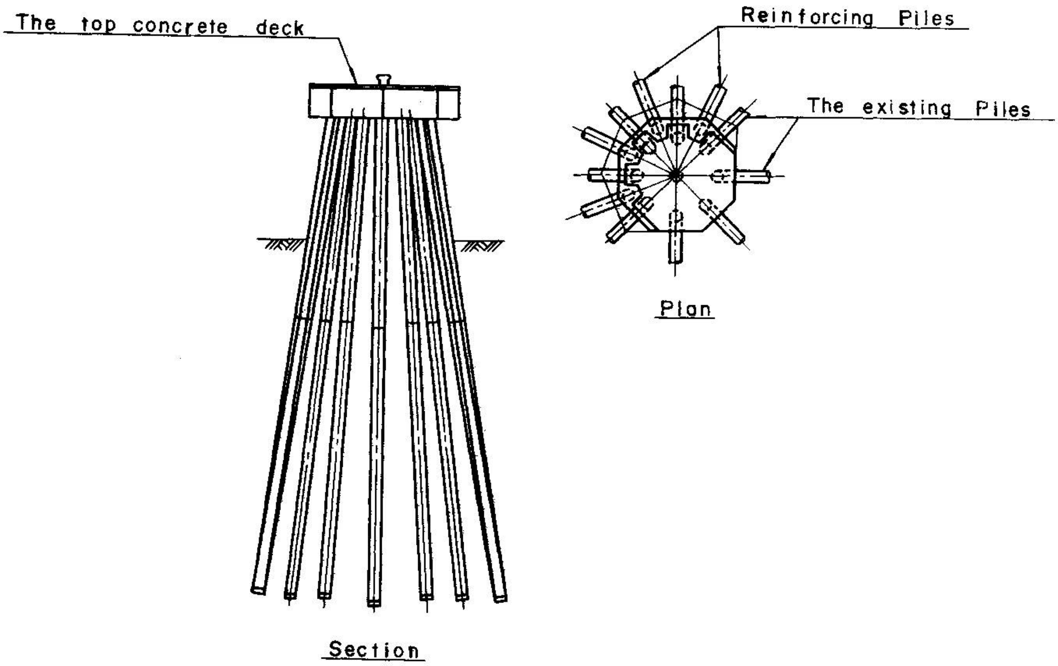



Fig.8 Restoration of Dolphin M-1

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# Vulnerability of Norwegian Bridges across Channels

Vulnérabilité des ponts enjambant des canaux de navigation, en Norvège

Verwundbarkeit von Wasserstraßen überquerender Brücken in Norwegen

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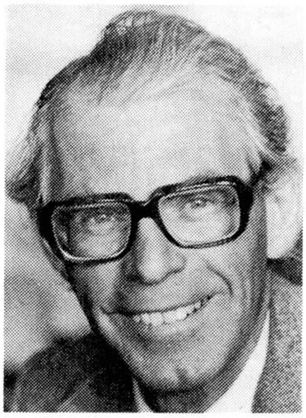

Per Tambs-Lyche, born in 1920, got his M. Sc. in civil engineering at the Technical University of Norway 1946. He has worked in the Public Roads Administration since 1946, as director of the Bridge Division from 1973.

**SUMMARY**
The paper describes the procedures for protective actions taken for securing bridge foundations against ship collisions. Relevant loads used in the design are given. No major catastrophe has occured. Some minor collisions causing relatively serious damage are described. A survey undertaken in 1982 to spot and evaluate the bridges that could be in danger of ship collision are presented.

**RÉSUMÉ**
L'article décrit les procédés de mise en place de systèmes de sécurité pour la circulation maritime et la manière de construire les fondations de ponts pour prévenir d'éventuelles collisions. L'article donne également les forces d'impact retenues pour le calcul. Aucun accident grave n'est survenu. Quelques collisions causant d'assez graves dégâts sont mentionnées. Une étude des ponts les plus exposés à des collisions a été réalisée en 1982.

**ZUSAMMENFASSUNG**
Das Vorgehen beim Festlegen von Sicherheitsvorkehrungen für den Schiffsverkehr und für den Brückenbau in bezug auf Zusammenstöße wird beschrieben. Die Aufprallasten für die Bemessung werden angegeben. Von größeren Katastrophen ist man verschont geblieben, es wird jedoch von einigen Auffahrunfällen mit erheblichem Sachschaden berichtet. Das Ergebnis einer im Jahr 1982 durchgeführten Überprüfung der am meisten gefährdeten Brücken wird vorgelegt.

## 1. INTRODUCTION

The Norwegian topography, with the narrow and deep fjords and numerous islands has resulted in a considerable amount of bridges crossing ship channels. The survey has detected 102 bridges of the kind. In each single case the criteria are worked out together with the Coast Directorate, which is the main authority for marine traffic. Requirements for sailing height, channel width and necessary actions for directions of ship traffic and bridge protections are worked out.

It is a difficult task to determine the load acting on a bridge during a ship collision. The actual force is depending upon the size of the vessel, its construction that determines the deformation length, velocity, deformation of the bridge foundation, the angle of collision etc.

In Norway it has, for practical and economical reasons been the rule to protect only the main foundations adjoining the ship channel, and only to a lesser degree protect the other foundations. However, by proper design these are given the best possible protection.

In the Norwegian ship navigation instructions, spesifications of the permissible size of the ship and width and height of the sailing channel are given.

Even with the rules described above, it is possible that larger ships accidentally may hit a bridge foundation. This might be due to navigation errors, bad weather, engine troubles etc.

The possibility of a ship collision is therefore always present. The degree of protection must be a compromise between the acceptable risk and the cost of establishing protection.

As part of the work done by the association of nordic road administrations, spesifications for the static forces to be used in the design has been worked out. These are related to the size of the ship, the ship depth and velocity. See fig. 1. (1).

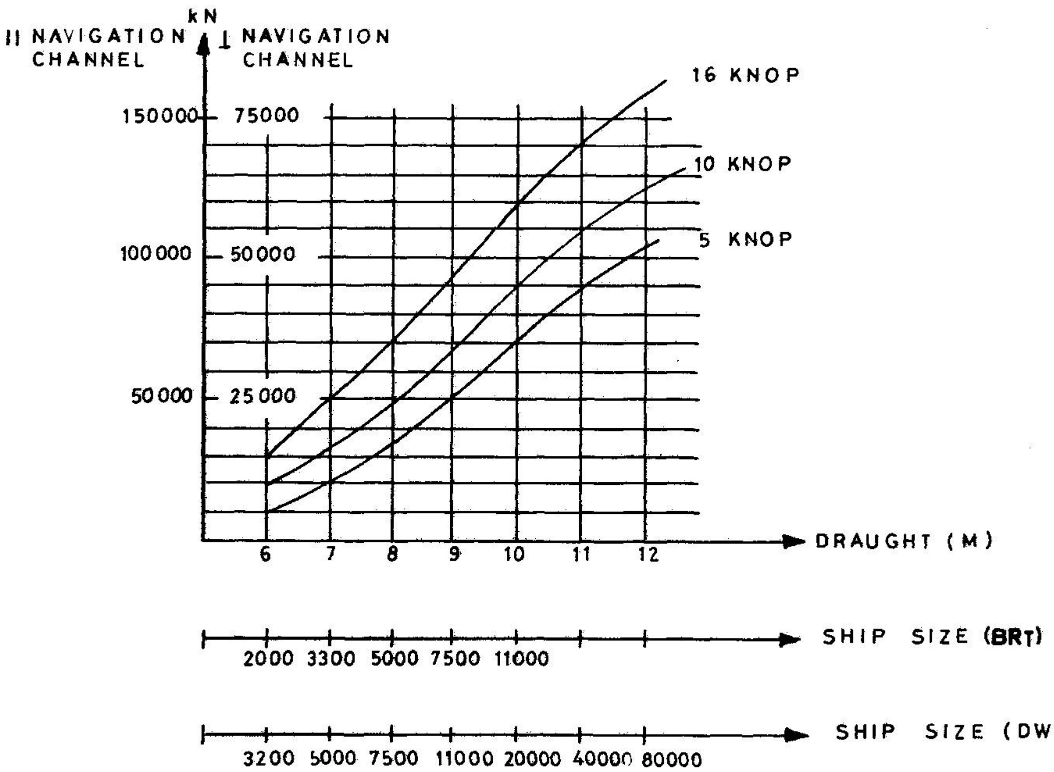


Fig. 1 Magnitude of ship collision force as a function of ship size and speed

It is for this matter refered to the article by Werner von Olnhausen, head of the bridge section, Sweden Road Administration.

Norwegian bridge foundations adjoining a ship channel are as a rule designed for a static force of 3000 MP perpendicular to the bridge's axis (parallel to the channel) and 1500 MP parallel to the bridge. The foundations are usually supposed to be rigid. The size of this force is approxemately supposed to represent a ship of 8000 dwt at a speed of 5 knot.

## 2. SHIP COLLISION ACCIDENTS IN NORWAY

Fortunately, there has been no disasters caused by ship collision involving Norwegian bridges. However, some major accidents, requireing large repairs have occured, as will be described briefly as follows:

### 2.1 Tromsø Bridge

This bridge, which was opened to traffic in 1960, was one of the most remarkable concrete bridges in Norway at that time.

With its 1016 metres long superstructure resting on slender columns above Tromsø Sound, it is, together with the near situated "Artic Cathedral", perhaps the most well-known landmark in Northern Norway. Fig. 2.

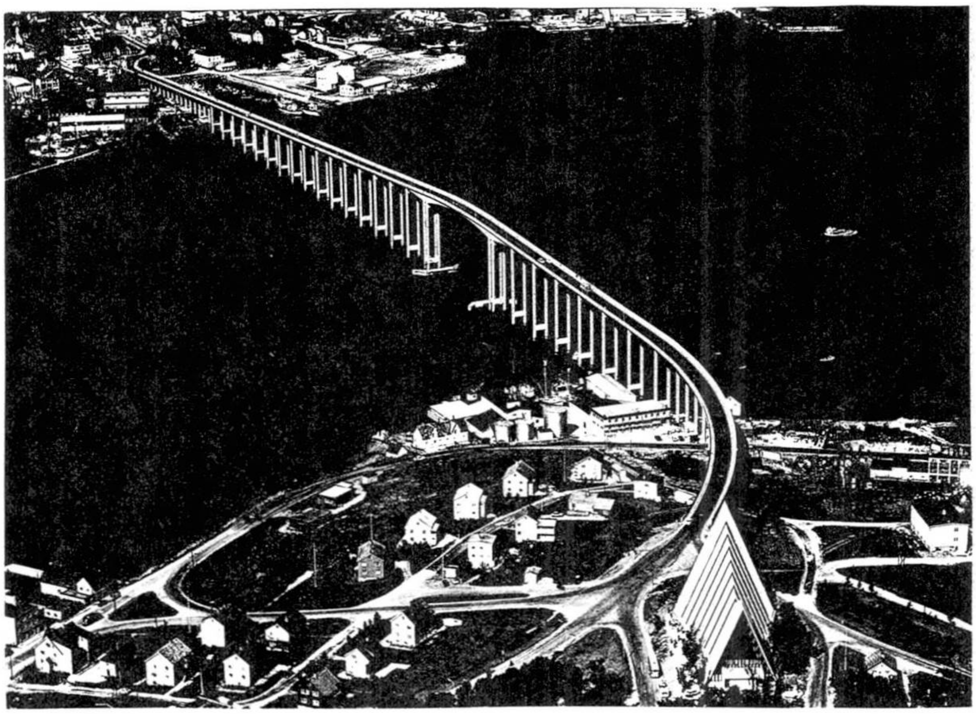

Fig. 2 Tromsø Bridge

To day, having learned by experience from several serious accidents around the world, and also a great number of smaller accidents in Tromsø Sound, we must state that the risk of collision between passing ships and the columns of Tromsø Bridge, was not satisfactorily judged when the bridge was planned.

The center to center distance between the two pairs of columns on each side of the main span is only 80 metres, and between fenders protecting these columns, the channel is only 60 metres wide. Between the main span and the abutments on the Tromsø side and the Tromsdal side respectively, the distance between the pairs of columns varies from 24 to 10 metres.

Especially against the Tromsø side, i.e. about 600 metres of the bridge length, the depthof water varies from about 12 to 7 metres. In July 1977, M/S May Veronica, a large fishing vessel, ran into a column 134 metres from the channel. Fortunately the damage was limited and not serious.

The tidal stream can reach a speed of 4-5 knots in the channel, and ships passing should then keep 8-10 knots to have steering control. Each of the original fenders were designed for a static force of only 100 tons acting parallel to the channel, and for 5 tons pr. meter of length of the fender structure acting parallel to the axis of the bridge. (The fender was about 20 metres long, i.e., the force used in the calculations was approximately 100 tons.)

In a letter from July 1957 one can read that concrete piles were considered as more longlasting than steel piles and therefore chosen in the first fender structures.

Danger of cracks in these stiff piles would occur if, during a collision, the fender slab was given a horizontal deformation of 30 centimetres only.

On the 21 November 1961, the 10 000 dwt. S/S "Gloria" ran into the eastern fender. The fender slab above the water as well as most of the concrete piles carrying the slab were completely destroyed and ended on the bottom of the sea. Fig. 3 shows the dimensions of the ship, the channel and the fenders.

On the 21 May 1963 the western fender suffered the same fate as its easterly twin. M/S "Rotesand", a 1560 dwt ore-ship, then ran foul of the fender which collapsed and had to be removed. Fig. 4 shows the fender structure after the breakdown.

When the two fenders were rebuilt in 1962-63, hollow steel piles KP 35, filled with concrete, were used. The clearance between the fender slab and the nearest columns was 5 metres. Fracture of the piles were expected at a horizontal deformation of the fender of 4.3 metres. This would, approximately, be sufficient to stop a 10 000 dwt ship drifting into the fender at a speed of 0.5 m/sec.

In its first years of service, the Tromsø Bridge was a toll bridge under local administration. When the Public Road Administration (PRA) later was asked to take over the responsibility for the bridge, the Administration made demands for stronger fender structures.

The port authorities informed that the yearly traffic in Tromsø Sound included about 100 ships in the class 8-10 000 grt and 25 ships in the class 10-15 000 grt.

The PRA therefore worked out tender spesifications for new fender structures, each designed to give satisfactory protection of the main columns of the bridge even if a ship representing a weight of 12 000 tons (calculated as the sum of displacement and hydraulic mass) ran into the fender at a speed of 4 m/sec (8 knots). Such fenders would be 10 times as strong as the existing ones.

The tenders' offers were so expensive that the Administration reviewed the whole project, taking into consideration that a neighbouring channel, the Sandnes Sound, in the future could serve as fairway for larger ships and thereby relieve the Tromsø Sound.

The existing fender structure which were built in 1974-75, were therefore designed to withstand the impact of a 7000 tons body (ship plus hydraulic mass) with a speed of 4 m/sec. The design was based on assumptions so chosen that the resulting forces should be on the safe side as far as the bridge structure is considered. (2), (3).

Ring-shaped reinforced concrete structures resting on steel piles, now encircles the groups of four columns on each side of the main span. The clearance between concrete ring and the nearest pair of columns, is 5.25 metres measured at right angle to the channel, and 7.05 metres measured parallel to the channel. Fig. 5.

The ability of the fenders to protect the bridge was demonstrated in av very re-

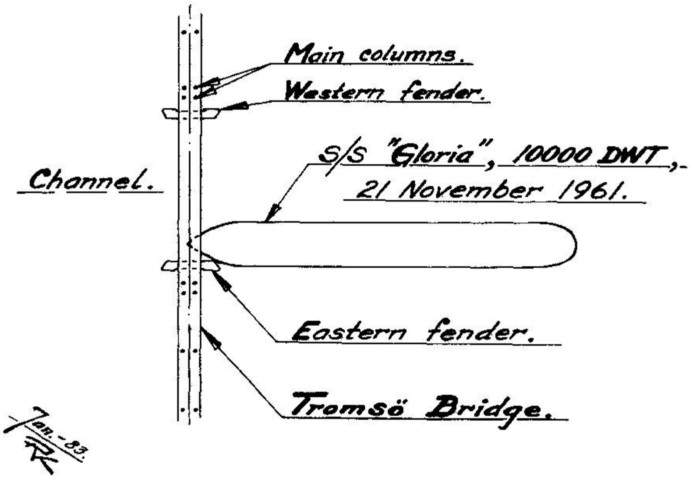


Fig. 3 Tromsø Bridge:
Collision, November 1961.
Dimensions of ship,
channel and fenders.

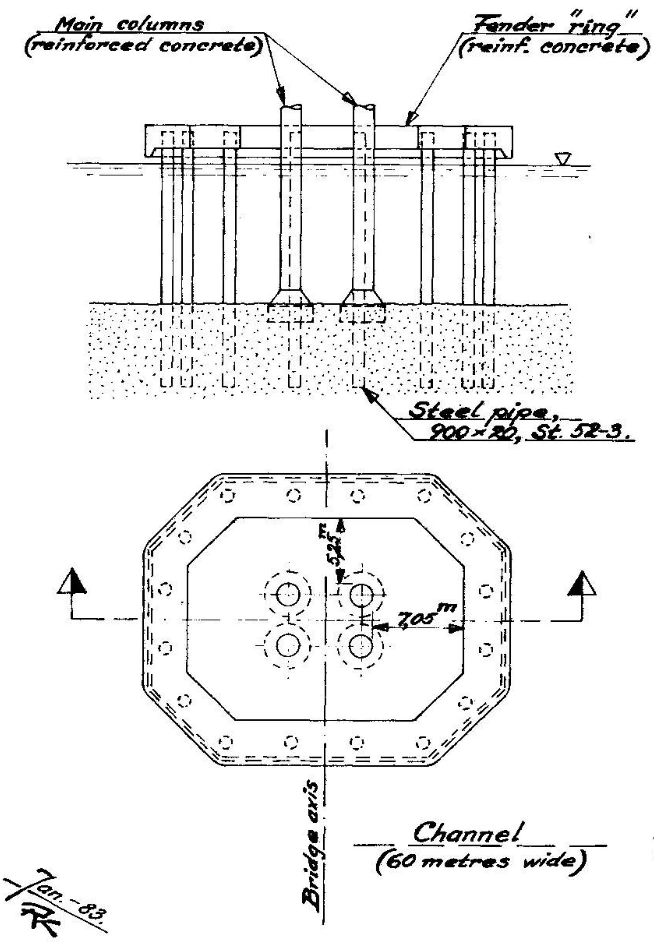


Fig. 4 Tromsø Bridge:
Western fender before and
after having been destroyed.
(The destroyed fender was
partly resting on heaps of
broken piles, which are not
shown on the figure).

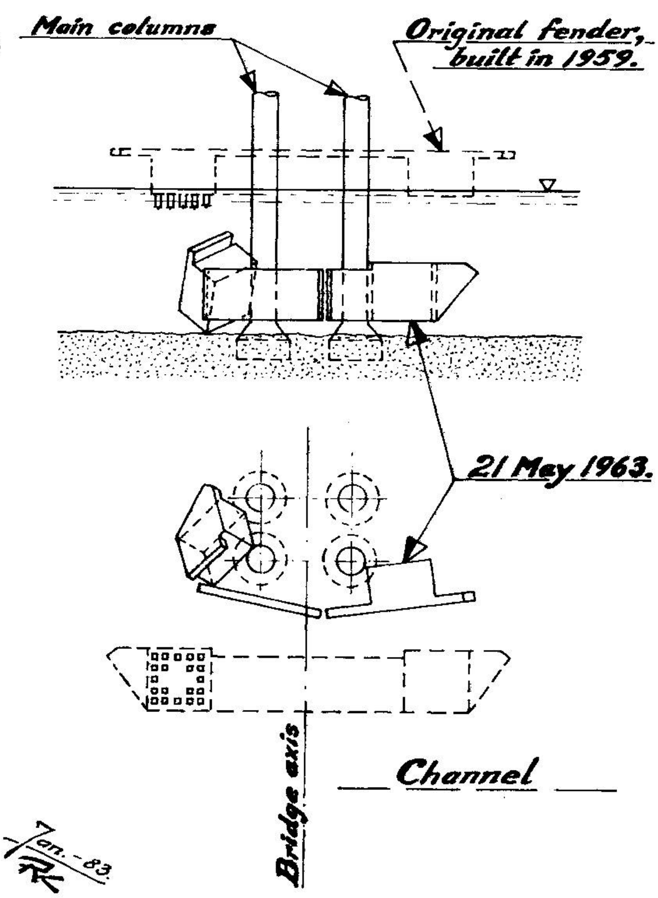


Fig. 5 Tromsø Bridge:
New fender.
Ring-shaped reinforced concrete
structure resting on steel piles.

alistic manner on the 5 July 1975. Then the passenger ship "Ragnvald Jarl" ran into one of the fenders resulting in a big crack in the side of the ship and four damaged cabins.

The wooden planks along the concrete fender were more or less crushed and torn off, but the concrete and the steel parts of the structure were not damaged.

If the new fenders had not been built, the whole mid-section of the bridge would probably have fallen down.

A better protection of most of the columns carrying the smaller spans of the bridge could have been desirable, but very expensive. Obviously, a new bridge might be a better solution.

With the limited resources at our's disposal, the following precautions are recommended in the report on vulnerable Norwegian bridges across channels. (4):

I Installation of radar echo equipment along the main channel (on buoys of skerries).

II Installation of navigation lamps and/or improving existing lighting systems on the bridge.

III Installation of special warning devices to stop all the traffic across the bridge in case of serious damage to the structure.

## 2.2 Brevik Bridge

The Brevik bridge is situated on the main route E18, approximately a hundred miles southwest of Oslo.

The 677m long structure consists of suspended main- and sidespans of 272m and 85m respectiveliy in addition to 16 viaducts. Fig. 6.

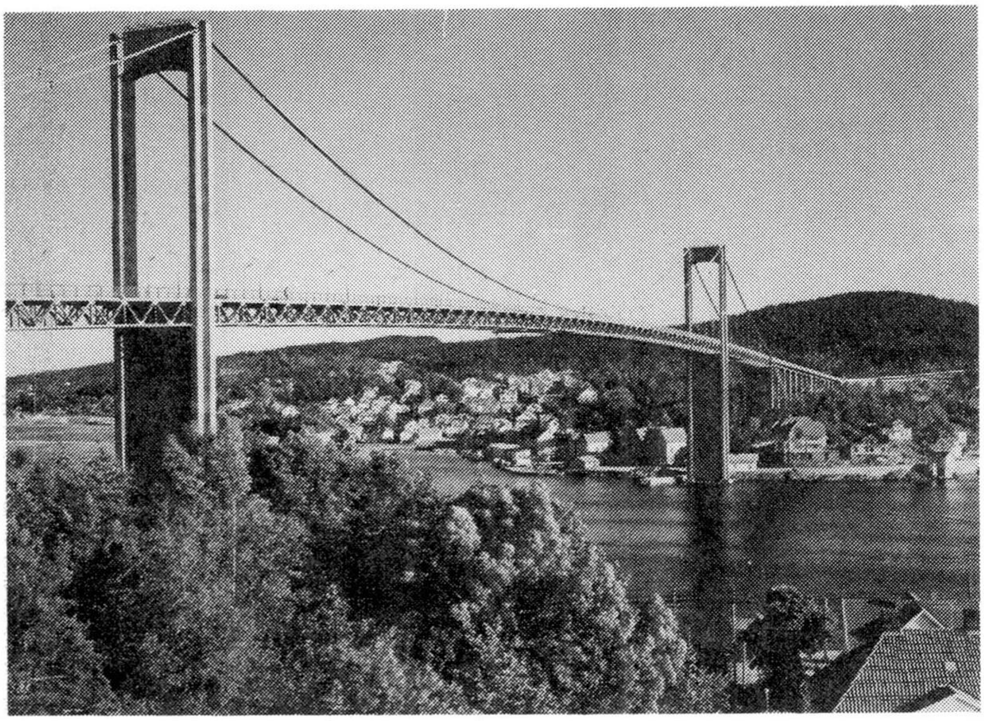

Fig. 6 Brevik Bridge

It was opened to traffic in 1962 and spans the inlet to the Frierfjord where the harbour of Porsgrunn and the largest chemical industrial centre of the country are situated.

The southern tower of the main span has been regarded as vulnerable to the ship-traffic taking place through the channel. The navigational conditions are not the best ones through the shallow, narrow and tidal waters of the inlet, though certain equipment with regard to surveillance of the ships concerned has been in operation for some time.

Even though, incidents are now and then taking place in the channel including two cases of running aground close by the tower. One of the towerpiers was left with spalled concrete after one of these episodes. In addition three other close by incidents (agrounds within 20m from the tower) have been registered. All the run-agrounds in the bridge area have taken place while ships were leaving the harbour. According to attached figure 7 the tower is obviously most exposed to collision from ships moving in that direction. It should be emphasized that part of the tonnage passing the sound carries dangerous cargo, e.g. gastankers.

Some years ago the stiffening truss system of the suspended mainspan was hit by a floating crane. The cantilevered cranearm struck the bridge about midspan 48m above sea level.

The bottom girder of the stiffening truss was locally bent without causing serious damage to the structure. Hence the bridge was open to one lane traffic througout the replacement operation of the girder and bracings.

The largerst ship having passed the bridge is said to be about 35 000 dwt. Permissible speed is set to 5 knots but might be slightly higher due to heavy tidal streams.

In order to protect the bridge and increase the safety generally, the following precautions are recommended in the report on vulnerable Norwegian bridges across channels. (4):

I A proposed filling up zone as indicated in figure 7, will reduce sailing depths at the southern tower and will also provide an energy absorbing cushion for ships running aground.

II Installation of warning devices to stop road traffic in case of serious damage to the structure.

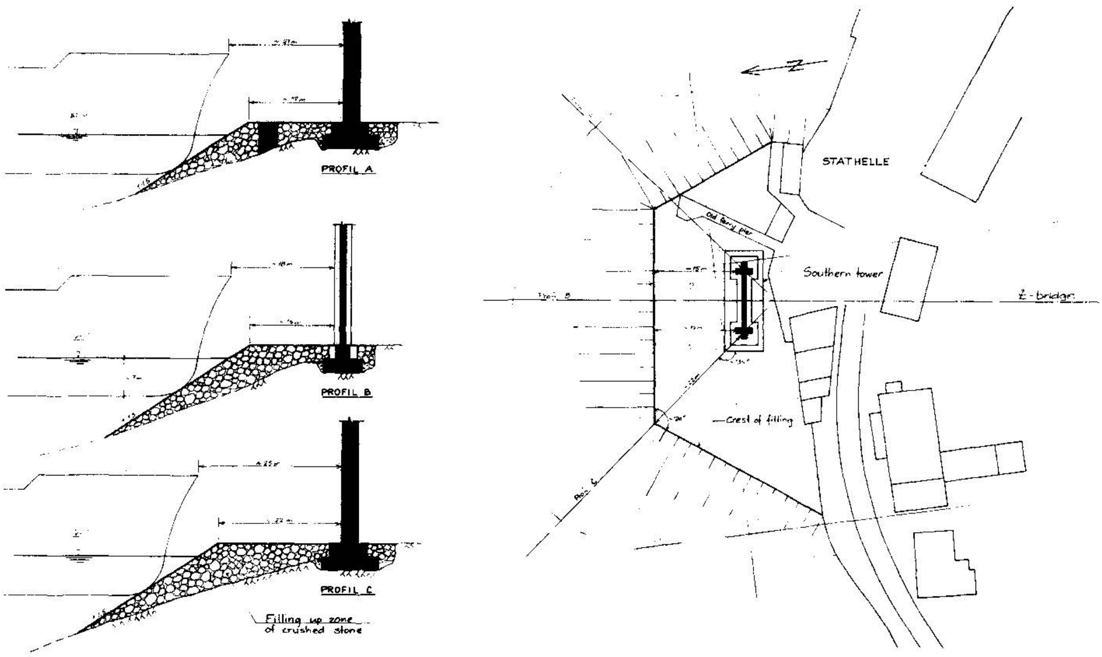


Fig. 7 Brevik Bridge; Filling up zone, southern tower

### 2.3 Sørsund Bridge, Kristiansund harbour

The bridge was opened in 1963. This is a free cantilever construction with a prestressed 100m main span, and 2 adjoining span of 50m. The 16 sidespans with span width of 13m are reinforced concrete, supported by a pair of circular columns with a diameter of 140cm. Sailing height is 35m over a width of 50m.

The regular ship traffic is directed through the main span. However, in 1963 a Russian ship lost control and hit the innermost of the sidespan columns. The depth of water was only 2.5m, and thus the ship was slowed down by hitting the bottom. The velocity at the moment of collision is estimated to have been 0.5 knot.

The column, having a height of approx. 38m from bottom, was hit directly by the ship and failed both at the bottom and at the point of collision. The deformation of the column was approximately 65cm. The expansion joint at the bridge deck was deformed by 3cm at one side and 8cm at the other.

The bridge was repaired by construction of two supports on the rock giving a triangle for jacking directly opposite of the point of collision. The repair was successful. The ship received a 35cm dent in the front.

### 2.4 Kjøkøysund Bridge, Hvaler

This bridge was opened in 1971. It represents a different type of ship collision. The bridge is a free cantilever prestressed construction. The main span is 100m. The sailing height is 25m over a width of 80m. In 1976 it was hit in the middle of the main span by the crane of a boat in regular traffic under the bridge.

The lower part of the box section in an area of 2m width, and 3.6m length at right angles to the bridge (approx. 70 degr.), the concrete was cracked, and holes developed. At 9m to both sides there were cracks between the bottom plate and the walls. Cracks developed in the walls, and the corner of the box-section was destroyed. Due to the shock, the concrete around the prestressing anchorages was knocked off. No damages were observed in the top plate. The bridge was not closed, and the repair was done in steps. First a 1m section of the hole in the bottom web was repaired to secure the carrying capasity. Then the damaged concrete was removed and replaced. Epoxy injection was used for the cracks, and glassfibers and epoxy was used approx. 10m on the bottom side for protection. No injection of the cracks in the bottom plate was done.

### 2.5 Gisund Bridge

During construction, the Bailey platform used for the construction of one of the main foundations for this free cantilever construction, was hit by a ship. Damages made it necessary to replace the platform. The requirement for lighting and other navigation directions had been followed, and thus the shipowner had to pay the bill.

### 2.6 Drammen Bridge

This motorway bridge was finished in 1975. It is a concrete box-section construction with spans around 50m. Close to the bridge is a quay in use for larger ships. In 1978 a ship of 4572 dwt was not able to reverse the engine. However, by very good seamanship the ship was grounded after manouvering between two columns. The superstructure of the ship only caused minor damages to the edges of the box-section. Fig. 8.

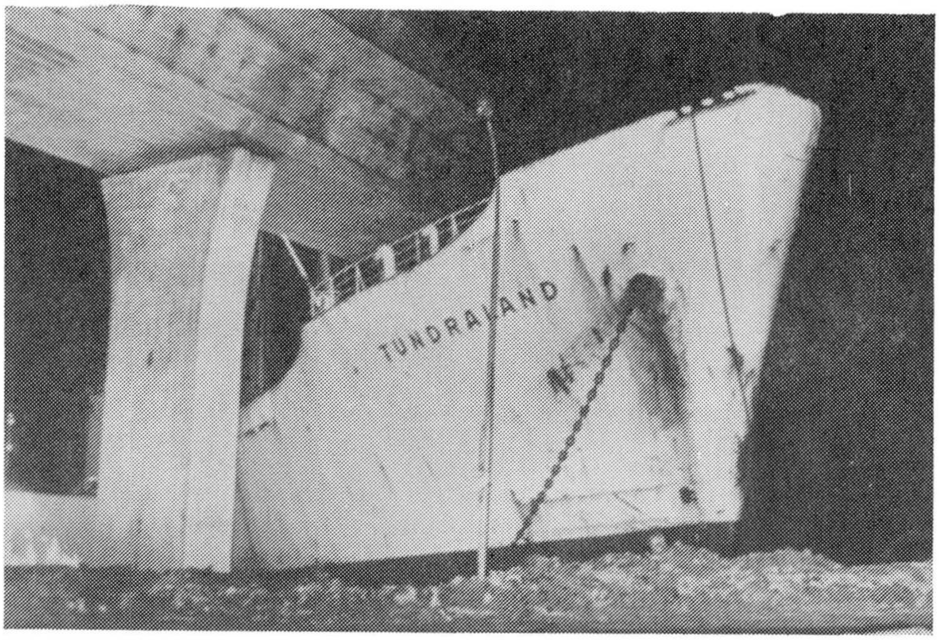


Fig. 8 Drammen Bridge; Nearly catastrophe

3. THE VULNERABILITY OF NORWEGIAN BRIDGES ACROSS CHANNELS
A SUMMARY OF THE REPORT DATED JULY 1982 (4)

The question of safety with respect to the Norwegian channel bridges was brought up by the Tjörn bridge collapse in Sweden, January 1980.

Shortly afterwards the Norwegian Public Roads Administration, through its Bridge Division, prepared a preliminary survey on bridges which were supposed to be classified as a channel crossing. The actual size of the ships which possibly could pass through the waters - and their frequency - was not taken into consideration on this stage of the proceedings.

The list of the 102 bridges thus brought forward was based on selections made by the local road authorities in the coastal counties concerned.

Fairly early in the subsequent examination of the listed bridges it became evident that a realistic number of vulnerable constructions was much smaller when importance was attached to the following conditions:

- Vulnerability of piers and superstucture against collision.
- Expected size of ships and traffic intensity through the channel.
- Intensity of motor traffic and pedestrians across the bridge.

The report was only aiming at different means of secureing ships and structures by improving navigational conditions and pier protection in the main channel area. A reinforcement of bridge structures outside this area even though spanning across navigable waters, has been ignored from technical and economical reasons. The safety of these structures has been taken care of we believe by the road-traffic warning system recommended in the report.

The navigational conditions at the bridges in question were reviewed by the Coast Directorate. Their evaluation of the naval problems together with conclusions drawn up by the Bridge Division led to the presentation of the final report containing 20 presumably vulnerable bridges. The report states that increased safety can be achieved by applying one or more of the suggested improvements listed below:

I Better pier protection by reducing sailing depths, i.e. a filling up zone around main foundations to an acceptable level.
Alternatively - construction of separate fenders. (4 bridges).

II Installation of radar echo equipment along the main channel (on buoys or skerries), (6 bridges).

III Installation of navigation lamps and/or improving existing lighting systems on the bridge. (13 bridges).

IV Installation of special warning devices to stop all the traffic across the bridge in case of serious damage to the structure. (8 bridges).

Though the existing procedures for designing adequate pier protection - as well as positioning of structure according to navigational requirements - appears well established, we believe that this report will pinpoint new aspects of the question of safety for channel crossings.

## 4. ACKNOWLEDGEMENTS

My thanks to following contributors to the paper; Erik Lie, chief engineer; Reidar Klinge, chief engineer and Per H. Berg, chief engineer, all Bridge Division, Public Roads Administration, Oslo, Norway.

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