Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 42 (1983)

Rubrik: Theme E: Acceptance criteria: accepted risk levels

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Theme E

Acceptance Criteria - Accepted Risk Levels

Critères d'acceptation - Niveaux de risques acceptables

Annehmbarkeitskriterien - Akzeptiertes Risiko

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Numerical Risk Acceptability and Mitigation Evaluation Criteria

Critères numériques d'évaluation et diminution des risques

Kriterien zur numerischen Risikoannehmbarkeit und -schätzung

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SUMMARY

The development of risk acceptance and risk mitigation evaluation criteria in various contexts is reviewed. Past and present attempts to derive such criteria for use by decision-makers on public hazards are noted. A structured presentation of the generic approaches that have evolved for the development of these criteria is provided in order to establish a foundation for their consideration as elements of potential risk-based decision-making on ship/bridge collision hazards and their possible mitigations.

RÉSUMÉ

Il s'agit d'une étude de la mise au point de critères permettant d'évaluer l'acceptation et la diminution des risques dans différents contextes. Celle-ci passe en revue les efforts passés et actuels pour définir de semblables critères devant servir aux preneurs de décision. Elle propose une présentation structurée des approches génériques développées pour mettre au point ces critères, et ce afin que soient fournies les bases qui permettront de tenir compte de ces derniers dans le cadre des décisions pouvant être adoptées dans le domaine des risques de collisions de navires avec des ponts, et de leur possible diminution.

ZUSAMMENFASSUNG

Die Entwicklung von Kriterien zur Risikoannehmbarkeit und -minderungsauswertung wird in verschiedenen Zusammenhängen überarbeitet. Frühere und gegenwärtige Versuche zur Ableitung solcher Kriterien für Entscheidungsträger bei öffentlichen Gefahren werden aufgeführt. Eine strukturierte Darstellung der artmäßigen Annäherungen, die zur Entwicklung dieser Kriterien entstanden sind, soll eine Grundlage für ihre Berücksichtigung als Elemente potentieller, risiko-abhängiger Entscheidungen bei Gefahren durch Schiffskollisionen mit Brücken und deren mögliche Minderung bieten.

1. INTRODUCTION

The fundamental question, "How safe is safe enough?" or, equivalently, "Is a given risk acceptable?" continues to exercise policy makers and decision makers. Unless the question can be answered on some basis, no limits can be assigned to the expenditure of resources for safety improvement in any given activity. Since resources are finite, critically important mis-applications of resources will be (and are being) made to attain smaller and smaller improvements in the safety of some activities that happen to receive attention, while others with more significant safety problems must go ignored. But any postulation that "enough" safety has been established is clearly subjective, and, as is very apparent in all western societies at present, subject to controversy.

This paper attempts to provide a brief assessment of the pro's and con's of the generic numerical approaches to risk acceptability and risk mitigation evaluation that have evolved in response to this problem. It is intended that this will establish a basis for the consideration of similar approaches and the numerical criteria they may provide in the context of ship/bridge collision hazards. The generic approaches that are discussed are:

- Comparisons to ambient risks
- Comparisons to revealed preferences
- Risk-cost-benefit evaluations

2. COMPARISONS TO AMBIENT RISKS

Many catalogs, tabulations and graphs have been published that exhibit the risks from existing natural and technological hazards, based on past experience or modeling estimates. It is argued that if the risk from a new hazardous activity is lower than the "standards" implied by society's acceptance of these ambient risks, everyone should be satisfied that the new hazardous activity is "safe enough."

The Environmental Impact Report for a proposed liquefied natural gas terminal at Oxnard, California (Socio-Economic Systems, 1977), illustrates this concept. The various cumulative risk curves shown in Figure 1 apply to the total population exposed to potential LNG terminal accidents at Oxnard. Note the shaded area in Figure 1. It shows the effects of uncertainty in the predictions of the LNG facility applicant's estimate (the SAI (Science Applications Incorporated) curve); the upper boundary is that of a "reasonable worst case" estimate established from a review of other analyses that had been made of the risks and of the applicant's process of estimating them. It is seen that the LNG terminal <u>might</u> not meet a standard of acceptance based on comparisons to ambient risks, in view of the uncertainties in the estimates of the LNG risk. Uncertainty in the risks of an activity essentially add to the predicted risks in considerations of their acceptability, and may well be an important such addition.

Many presentations of comparable ambient natural and technological risks have been published, e.g., in Starr (1971), and Cohen and Lee (1979). An early argument for considering natural hazards as sources of risk acceptance criteria is that of Libby (1971). A most extensive compendium of ambient risk data is given in a recent Brookhaven National Laboratory report (Coppola and Hall, 1981). Kletz (1977) considers such risks in the United Kingdom and argues that they set standards of acceptability for U.K. industry. McGinty and Atherly (1977) rebut this argument, however, indicating that acceptability decisions must be made more democratically, and not merely on the basis of someone's views of past risk acceptance. After all, the past risks may have been accepted in ignorance, or

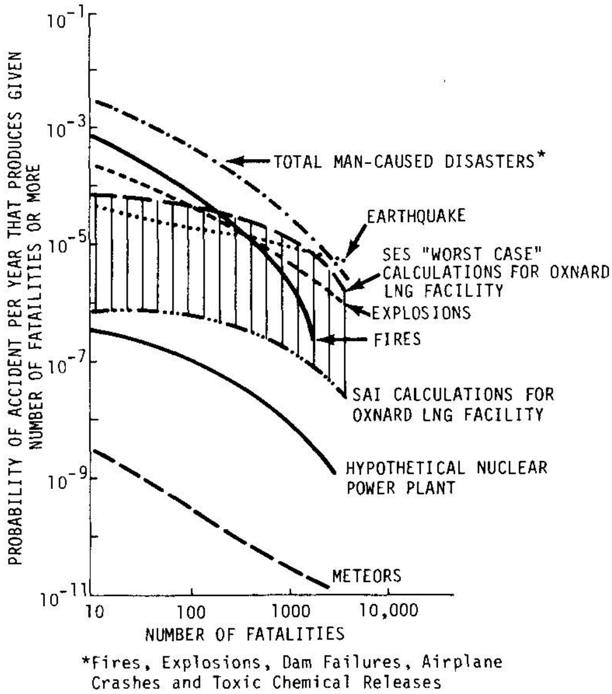

Fig. 1 Fatalities Risks from Potential Accidents in Oxnard Area (Socio-Economic Systems, 1977)

they may have been relatively unavoidable with the technology and economics that obtained in the past. Improved understanding of these risks and improved means to avoid them may now mean they are no longer "acceptable."

3. COMPARISONS TO REVEALED PREFERENCES

The notion that acceptable risk levels can be revealed by data on the relationship of the losses from past hazardous activities with the benefits associated with them was first put forward by Starr in an article in Science in 1969. The debate on risk acceptance criteria may be said to have originated with this article, and has since expanded in many directions and with growing intensity. Starr attempted to show, more or less quantitatively, what apparent past risk acceptance behavior was in U.S. society, and, due to its apparent consistency in certain ways, how it could provide a basis for judging what risks could be acceptable in the future. Otway and Cohen (1975), however, have critiqued Starr's findings and argued against the existence of the consistencies he claims. Baldewicz (1976), Pochin (1975, 1978), and others have extended Starr's data developments into occupational activities, where "voluntary" risk acceptance is presumably obtained by the relatively clear job benefits that are associated with it. Special concerns with catastrophic group or societal risks, as distinct from average individual risks, have been assessed, and arguments put forward on how society evaluates them, by Wilson (1975) and Ferreira and Slesin (1976), among others.

Figure 2 presents Starr's original curves of historically-accepted risks (i.e., "revealed risk preferences") versus the actual or perceived benefits he estimates accrue from their acceptance in society, from various types of hazardous man-made activities and possible natural events. These curves derive from statistical data on the average numbers of fatalities that resulted from the hazards in these activities per hour of individuals' exposures to these hazards in the past, versus dollar equivalents of the benefits (estimated in various direct and indirect ways) of such exposures.

Numerous arguments have been made against the application of Starr's conclusions, however. First, it is put that many past (and, for that matter, present) risk takers did not understand the risks they were accepting, so that the fact that they accepted them does not validate their or others' continuing to do so. Secondly, "voluntary" risk takers may not actually have accepted them "voluntarily," but because they had no viable alternative. As society and technology evolve, such alternatives may become more available, and, certainly, hazards may be reducible even for the same activity. Third, the use of average risks and benefits obviates the differences among specific risk takers and benefitters. Individuals accepting the highest risks may not be the same as those gaining the highest benefits. Finally, the use of averages "washes out" the disproportionate potential societal impacts of catastrophic hazards. Nevertheless, comparisons to relevant ambient risks remains a favored approach to acceptable risk criteria development in many specific contexts, as will be seen most particularly for nuclear power, below.

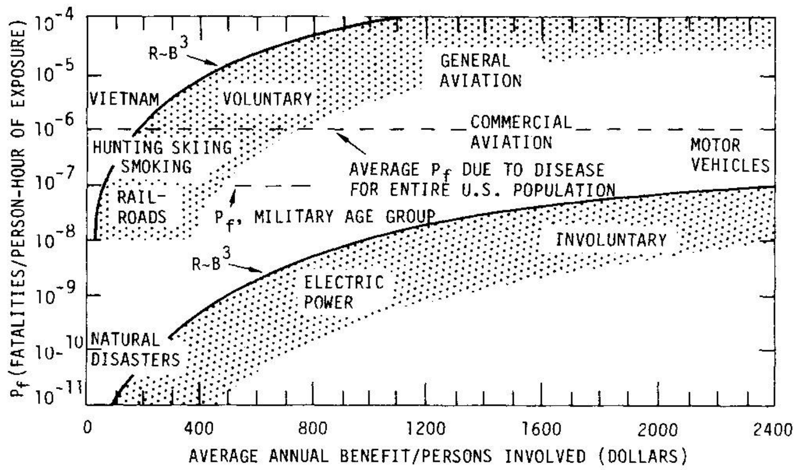

Fig. 2 Starr's Risk vs. Benefit Curves (1969)

4. RISK-COST-BENEFIT EVALUATIONS

Another basic approach to evaluation of the significance of the risks of a hazardous activity is to assess these risks in relation to the specific benefits the activity provides (Wilson, 1975ii). Three variations in this approach are considered.

First, and quite simply in principle, if alternative means are or can be made available to provide the desired benefits, the alternative that does this at the lowest risk is to be preferred (see, e.g., Figure 3, for alternative energy sources). It is assumed in this that costs of the alternatives are all more or less equally acceptable. This procedure is referred to as that of equi-benefit risk comparison. The risk of the lowest risk alternative defines the de facto level of acceptable risk (provided it is agreed that one of the alternatives must be selected).

Second, and more generally, the risks and benefits of an activity can be compared in some common terms, and the risks be deemed acceptable if, in these terms, they are not greater than the benefits. This is referred to as the balancing of risks and benefits. (Costs are assumed able to be neglected or subsumed as negative benefits.)

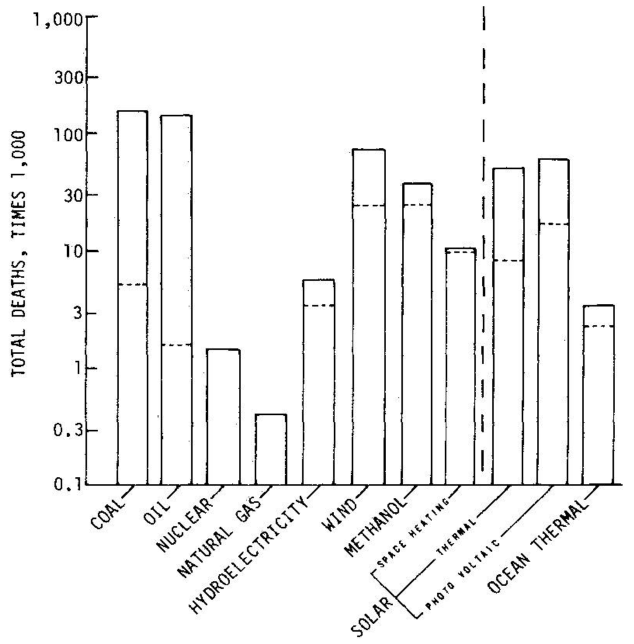

Fig. 3 Upper (U) and Lower (L) Bounding Estimates of Total Deaths (Public and Occupational), Times 1000, per Megawatt-Year, as a Function of Energy System (Total Fuel Cycle) (Inhaber, 1979)

Third, and perhaps most applicable to a risk management process employed in the optimization of ship/bridge safety decisions, resources can be applied to safety improvements until the value of the marginal risk decrease attained for an additional unit cost (in common terms with risk) becomes less than the cost. The residual risk remaining when optimality is reached is then the de facto acceptable risk level, in the sense that it would be an inefficient use of resources to attempt to reduce it further. This argument is best made when resources are limited and several hazards are competing for them so that it is accepted that they must be employed efficiently.

It is to be noted that the implementation of the second or third process requires a common scale of measurement of risks, benefits and costs. This has given rise to many attempts to establish an economic (e.g., dollar) "value-of-a-life" (Linnerooth, 1975; Jones-Lee, 1976) or, because of the evident problems in this, an economic value of the avoidance of a risk of loss of life. (Alternatively, the application of utility theory has been attempted in order to assess risks, benefits and costs on a common scale provided by a "decision maker's" utility function; see, e.g., Keeney, 1980.)

5. SYNTHESES OF SPECIFIC NUMERICAL ACCEPTANCE CRITERIA

Attempts have been made to develop on the basis of the concepts that have been discussed, and especially through comparisons with ambient risks, generally applicable numerical acceptability criteria that it is then hoped will be adopted by sufficient authority. A primary example is the present effort to convince the U.S. Nuclear Regulatory Commission (USNRC) to accept a specified set of reactor safety goals defined in terms of acceptable risk levels (Nuclear Regulatory Commission, 1980; Griesmeyer and Okrent, 1981; O'Donnell, 1981; and others). Farmer (1967), Gibson (1977) and Bowen (1975) have previously developed such criteria for use in the United Kingdom.

While various qualifications apply, the basic idea in the nuclear power case is as follows. Table 1 and Table 2 present a set of individual and group risk criteria, respectively (O'Donnell, 1981). Consider, for example, the Atomic Industrial Forum (AIF) committee's values. The individual fatality risk criterion of 10^{-5} per exposed person per year is justified in that it equates to 0.1% of the total ambient mortality risk (10^{-2} per year) of individuals in the U.S. and about 1% of the total ambient accident risk. The AIF committee's preferred group or population acceptable risk level (median value) is 0.1 fatalities per year per 1000 megawatts - electric of nuclear power capacity. This number is justified by comparison to the total ambient mortality risk and the total ambient cancer risk in the U.S. Assuming a total of 200,000 MWe of capacity, the number translates to about 0.01% of the total mortality risk and 0.05% of the total cancer risk. The AIF's individual and group criteria are further justified by their comparability to the other proposed criteria or risk estimates given in the two tables.

NRC - RES	10^{-5}/YR UNACCEPTABLE 10^{-6} - 10^{-5}/YR WARNING RANGE (CASE BY CASE EVALUATION)
WILSON	10^{-5}/YR NEAR SITE 10^{-6}/YR NEXT TOWNSHIP
OKRENT	2 x 10^{-4}/YR ESSENTIAL ACTIVITY 10^{-5}/YR BENEFICIAL ACTIVITY 2 x 10^{-6}/YR PERIPHERAL ACTIVITY ASSESS RISK AT 90% CONFIDENCE LEVEL
CORKERTON ET AL (CEGB)	10^{-5}/YR PUBLIC 10^{-4}/YR WORKER
WASH 1400	8 x 10^{-7}/YR
GERMAN RISK STUDY	1 x 10^{-6}/YR
AIF	10^{-5}/YR

Table 1 Some Proposed Numerical Values for Individual Risk Criterion (O'Donnell, 1981)

LEVINE	0.2	FATALITIES/YR
WASH 1400	0.02	FATALITIES/YR
GERMAN RISK STUDY	0.4	FATALITIES/YR
AIF	0.1	FATALITIES/YR

Table 2 Some Proposed Numerical Values for Population Risk Criterion (Implied from Risk Curves) (O'Donnell, 1981)

The latest criteria under consideration by the USNRC are less conservative variations on the theme of the foregoing concepts: nuclear risks should not exceed 0.1% of the risks that might accrue if equivalent non-nuclear power generation were substituted for the nuclear power plants, and also 0.1% of the cancer fatality risks from all other sources.

6. CONCLUSIONS

After a risk accruing from an existing or proposed hazardous activity, such as ship operations requiring passage under bridges, has been estimated, a decision must be made on whether it should be accepted, or some alternative action taken that will mitigate the risk. Whether the original risk may be able to be decided to be acceptable may depend on whether it is small relative to ordinarily accepted "ambient" risks or whether the benefits in accepting it are sufficiently great, which may be able to be assessed by direct comparison of the risk and benefits in common terms, or by comparison to risk-benefit preference relationships in the past. Whether, on the other hand, some alternative action, such as a bridge or ship channel design change or a variation in ship operating procedures, should be decided upon may depend on whether its cost is justified by the risk decrease that it would provide.

These decisions may impact specific exposed individuals or groups. Risks and benefits may directly accrue differently to different individuals and groups, and also may accrue indirectly to others, as well, including the decision maker, such as an activity operator; ship crews; a regulator; an insurer; society as a whole, insofar as harm to affected individuals or groups (especially from a catastrophic accident) could detract from society's present and future values.

This paper has attempted to assess some highlights of the very extensive and growing literature on these considerations. A more complete assessment is also available (Philipson, 1982). It is intended that the understanding of their potential applicability to the specific risk decision problems arising in the presence of ship/bridge collision hazards will thereby be advanced.

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Ship Collisions with Bridges in Sweden

Collisions de bateaux avec des ponts, en Suède

Schiffskollisionen mit Brücken in Schweden

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SUMMARY
The article gives some facts about five ship collisions with bridges, which have happened in Sweden since 1965. The road administration has developed design rules, which have been applied for new bridges since 1967. After the Tjörn bridge disaster in 1980 a complete survey was made of the collision risk at all Swedish bridges.

RÉSUMÉ
L'article décrit cinq collisions de navires avec des ponts, qui se sont passées en Suède depuis 1965. La Direction Nationale des Routes de Suède a établi des règles de calcul concernant le risque de collision, et ces règles ont été utilisées pour la construction de nouveaux ponts depuis 1967. Après le désastre du pont de Tjörn en 1980, une étude complète à l'égard du risque de collision a été réalisée pour tous les ponts suédois.

ZUSAMMENFASSUNG
Der Artikel gibt einige Daten über fünf Schiffskollisionen mit Brücken, die seit 1965 in Schweden eingetreten sind. Die Verwaltung verwendet seit 1967 für neue Brücken Dimensionierungsregeln, die Schiffsstöße berücksichtigen. Nach dem Tjörnbrückenunglück 1980 wurde eine Untersuchung des Risikos für Schiffskollisionen an allen schwedischen Brücken ausgeführt.

1. SHIP COLLISIONS WITH BRIDGES IN SWEDEN

1.1 Short reports on five collisions

1.11 The Tjörnbridge (Almö-bridge) near Gothenburg

The former Tjörnbridge was a 278 m steel-arch bridge, built about 1960, fig 1. The arch consisted of two 3,8 m circular tubes with 14-22 mm platethickness ("tube-tandem"). The navigation channel under the arch had a height of 41 m on 50 m width.- On the 18th of Januari 1980 at 1.30 a.m. in bad weather the arch was struck by a 27.000 dwt ship and collapsed totally. 8 people lost their lives, driving in their motorcars into the water over the edges of the remaining viaducts, before the road traffic could be stopped. The arch fell partly upon the ship without causing any injuries , fig 2. For further details, see /1/.

The collaps of the bridge interrupted a very important communication for the people on the islands of Orust and Tjörn, prolonging the roadway distance to Gothenburg by about 80 km. A provisional ferry-lane was therefore established immediately after the disaster, giving a capacity of about 2000 vehicles/day. At the same time a new bridge was planned and - after an international competition - the order for its construction was given in medio July 1980 to a Swedish-German consortium. The new bridge was built on the same place,but as a cable-stayed steel boxbeam of 366 m theoretical span. The about 100 m high pylons and the approaches were made in reinforced concrete. In order to make sure that the new bridge not could be hit by a ship, the pylons were placed on the rocks about 25 m on land. The free height was chosen to 45,3 m on 110 m width.-

About 16 month after the order - certainly a remarkable record in construction time - two lanes of the new bridge could be taken into use on Nov. 9th 1981, fig 3. The total cost for the new bridge and the temporary ferries was about 210 M SEK.

1.12 Tingstad Bridge, Gothenburgs harbour

The Tingstad railwaybridge is a steel truss-bridge with a 56,7 m swingspan, giving 2 navigation openings of 15,7 m width. There are two fixed approach spans of 31 m length.- On the 10th of September 1977 a 1600 dwt tankship hit a side span of the bridge, causing serious damage on the superstructure and the abutment. - The bridge was repaired for a cost of about 2 M SEK. For further details,see the introductory report, page 22.

1.13 Bridge over lake Mälaren at Hjulsta

The Hjulsta roadbridge is a two lane steel truss-swingbridge of 87,6 m length, giving 2 navigation channels of about 35 m width. On each side of the swing-span there are approaching bridges of 152 m resp. 266 m total length, consisting of 3 plategirders with concrete slabs of about 38 m spans, arranged continuously over two or three spans.- On the 12th Dec 1965 a 1.500 dwt ship hit the approaching bridge about 50 m south of the navigationchannel. Two spans of the superstructure were destroyed and fell into the water.

A provisional military-bridge was erected immediately on the undamaged piers. The superstructure was then rebuilt beside the military-bridge. Repairing cost about 2,5 M SEK.

1.14 Bridge over Göta river near Kungälv (Jordfallet)

The bridge over Göta-river near Kungälv is a double bascule bridge of about 44 m free width, giving a navigation channel of 42 m between guard-railings.

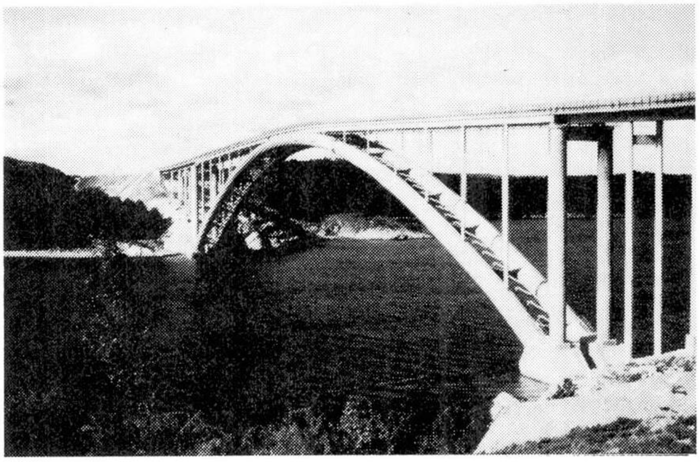

Fig 1. The former Tjörnbridge

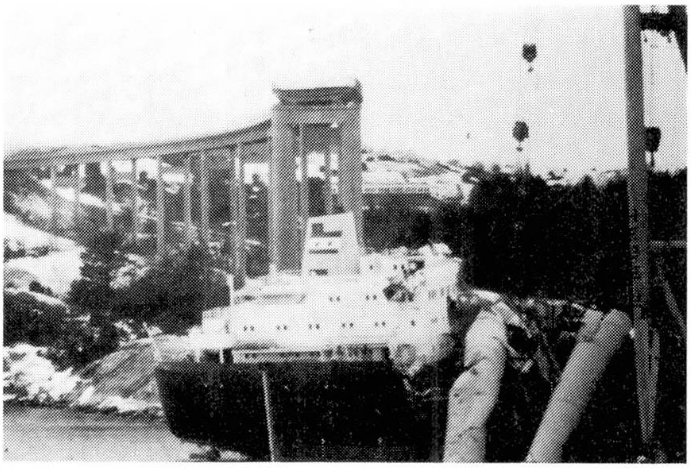

Fig 2. The collapsed Tjörnbridge

Fig 3. The new Tjörnbridge

There are about 20 m long bascule-piers of reinforced concrete on each side and then approaching bridges of about 240 respectively 362 m length. - On 30th April 1979 a 3.000 dwt ship hit the superstructure of one bascule, which could not be opened because of an electrical failure on the bridge. The ship,which had a breadth of only 12 m,could have passed in the remaining channel of 21,0 m, but she touched the bascule with her deckshouse. - A triangular bit of the bascules´ortotropic deck,with the bending points about 9 resp. 9 m from the corner,was bent downward at an angle of about 70°, fig 4. In spite of the very strong impact,the machinery and the counter-weight-arm of the bascule were undamaged. - The repair consisted of replacing of the damaged parts of the bridge deck and the maingirder. Repair-cost about 1,5 M SEK.

The same bridge was hit again on 28th Okt 1981 by a 480 dwt ship, which missed the channel and struck into the bascule-piers concrete wall about 8 m behind the guard-railing. The vessel went at about 3 knt speed uppstream. The shipsbow passed through the 0,45 m thick wall, reinforced by ø 12 c 300 on both sides , leaving a triangular hole of about 3 x 3,5 m just above the water level, fig 5.A concrete stiffener with 0,3 x 1,00 m cross-section was also destroyed. The bow never reached the backarm of the bascule. - The repair consisted of rebuilding of the concrete wall. Cost about 0,2 M SEK.

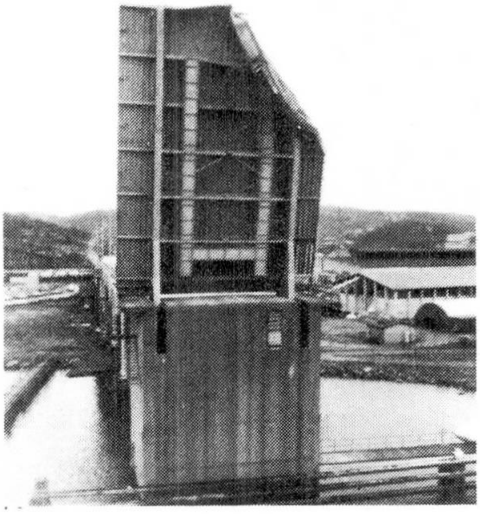

Fig 4. Damage on the Jordfallet bascule-bridge

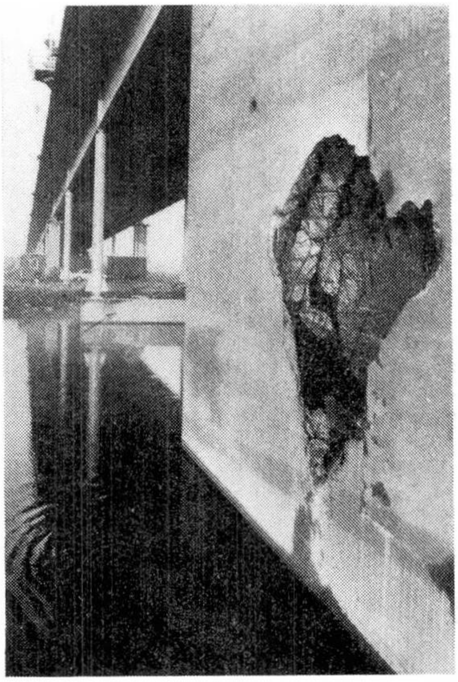

Fig 5. Damage on a bascule-pier at the Jordfallet-bridge

1.2 Some lessons from the Swedish accidents

The Swedish accidents are not very special, compared with others, which have happened all over the world. Nevertheless,they elucidate some elementary facts:

a. The threat of ship collisions with bridges is a real risk, which has to be regarded in the design concept. If possible, all piers and the super-structure should be placed out of the navigation area.

b. The superstructure of a bridge can not withstand any important ship collision. Normal steelplate structures are extremely sensitive for im-pact. The energy absorption of the bridge structure is very small.

c. Concrete structures must be made very thick and heavy reinforced if they shall withstand the collision impact from ships.

d. Guard-railings and other common navigation-aids are not enough effective, to prevent a collision. A colliding ship often hits the bridge besides of the navigationchannel. All piers in deep water must be regarded as threatened.

The Swedish design rules for ship collisions try to observe the above mentioned facts. It is supposed that the piers, if placed in deep water, are totally rigid and that all the collision energy must be absorbed by movements or deformations (damage) of the ship.

2. SWEDISH DESIGN REGULATIONS REGARDING SHIP COLLISIONS WITH BRIDGE PIERS

2.1 General remarks

In connection with the discussions about a planned new 18 km long bridge over the Öresund between Sweden and Denmark 1964-65 a study of measures against the risk of ship collisions with bridge piers was made in the Swedish Road Administration /2/. The need of regarding such risks was pointed out by the above mentioned accident at the Hjulsta bridge and became strongly stressed by the Maracaibo disaster, which had happened in May 1964.

The study, which was based on the Minorsky-analysis of energy absorption at collisions between two ships, resulted in the statement, that it is possible to design the piers of large bridges against the forces, which can arise by collisions with small ships (5000 dwt) and with mediumsized (40.000 dwt) ships. A series of some basic design rules were made up for the Öresund bridge, which then have been modified and applied to about a dozen of new Swedish bridges since 1965. Later on, these rules have been developed to the unified Nordic Recommendations, given in /3/.

2.2 Design rules for the Öresundbridge project

The proposed design forces for the piers of the Öresundbridge are given in fig 6. They have a maximum of 150 MN and are based on the assumption of a full speed collision (16 knt) of an ordinary 40.000 dwt tanker. It was even stated, that a 100.000 dwt tanker gives forces of about 240 MN (dotted extrapolition of the main line to a draught of about 16 m).

The design rules prescribed furthermore:

that piling foundations - if possible - should be avoided,

that the pier shafts should be made of reinforced concrete with at least 2 m walls, surface reinforcement ø 25 # c 200,

that the cross section of the pier should be given a form which would be capable to break through the shipshull,

that the pier shafts should be made in one piece, if minor than 10 m wide,

that the bearings should be properly fixed on the top of the pier,

that the collision forces may be regarded as ultimate loads, safety factor 1,0.

The above mentioned regulations have been made especially for the Öresunds-bridge. They had to be modified for other bridges with regard to ship size, speed and other conditions.

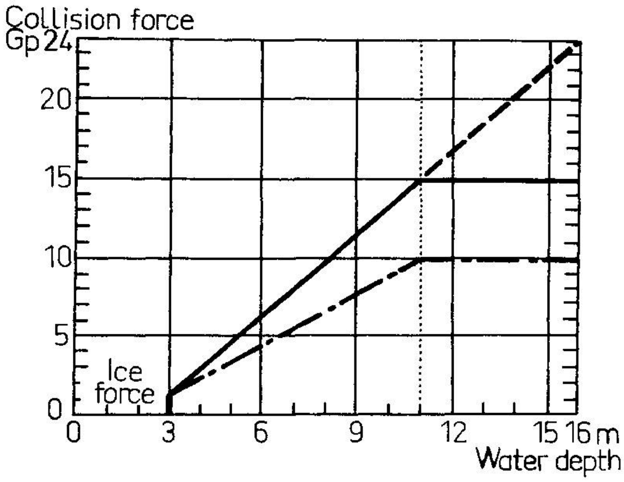

Fig 6. Design-forces due to ship-collisions for planned Öresundsbridge

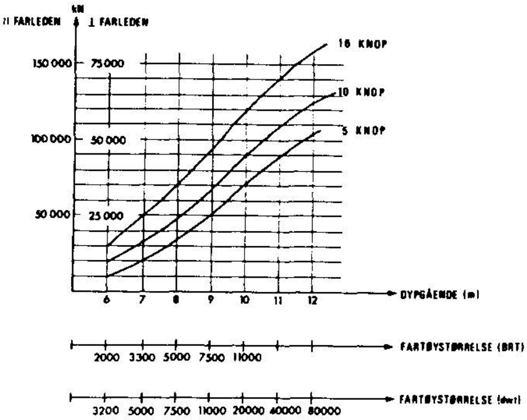

Fig 7. Shipcollision forces according to Nordic Recommendations

2.3 Nordic Recommendations of 1975

The Nordic Recommendations /3/ have been developed from the Swedish rules. The proposed collision forces are based on the same study /2/, using Minorskys energy-formula. But there are made some justifications in order to take account of the risk that big ships can sail in shallow water when unloaded. Moreover the risk of side-way collisions at low speed has been observed. Last not least some extrapolation has been made into the low energy area, which not can be handled according Minorskys formula, but which is of great importance for small bridges passed by minor ships.

The recommended design forces are shown in fig 7. They are applied as statical loads in the water level on the piers on each side of the navigation channel, perpendicular to the bridge´s axis (parallell to the channel). In the direction of the bridge´s axis and on other piers,far away from the channel, minor forces (e.g. 50 %) can be chosen as design loads. The characteristic ship size, when using the diagram fig 7, is that size of ships, which can be expected to pass the channel a certain number of passages a year, depending on the nautical difficulties in the channel, e.g. 100 passages/year in a easy navigated channel.

The general detailing rules, as mentioned in 2.2 above are valid also according to the Nordic Recommendations. The pier-dimensions may be changed in relation to the various impact forces. The large forces might be handled as surface loadings of about 2000 kN/m^2, which corresponds to the strength of the hull of large ships. For minor impact about 500 kN/m^2 is adequate.

2.4 Decisions of the administration

The Swedish design rules as well as the Nordic Recommendations lead normally to severe economical consequenses. They are therefore reconsidered by the administration in every special case in order to get both an acceptable cost and risk-level. It seems not yet possible to give correct figurs of the probability of ships collisions and of the consecuting risk- and safety levels. If figures are available they are valid only for very special circumstances, or they are the result of very grove assumptions. In normal cases is it neccessary to specify the collision design forces by a deterministic process after discussion of the costs and benefits of various possible measurements intended to increase the collision safety and the total safety.

The first question in this process is: Is it possible to avoid all collision problems, e.g. by altering the position of the bridge or of the piers?

If not, the next question is: What is the largest shipsize, which will pass the bridgesite and with what speed and frequency? Can piers of normal design withstand the possible impact forces?

If not, the next question might be: Is it reasonable to take a higher risk and to design the piers for a smaller ship size, which is more frequent? In Sweden we accept a number of between 20 and 100 passages/year, depending on the navigational difficulties of the channel.

The next question is usually: What is the cost of strengthening the piers for the actual design forces? Or what is the cost of alternative guard measures as protective piers, fenders or guard railings? In Sweden we have accepted a cost increase of between 5 to 20 % of the whole bridgecost, depending on the importance of the roadway.

The next question might be: What can be done by navigational means for improving of safety? Possible means in this field are: Speed limit, pilot-duty, direction-division, limit of ship-size, limit of admitted time, sight or weather-conditions, prescription of tug-aid, bridge-to bridge-contact and others.

A very important question is also: What can be done for safety by installing navigational aids? Here might be mentioned: Racon for marking of the channel, the superstructure or the piers. Lighthouses, beacons and buoys as manoeuvre-aids. Signals, lights, colour-marks for improving of visibility.

These questions are regularly discussed between the road administration and the navigational administration, which nearly always can come to an agreement. If there is a conflict of various interests, it is possible to get a special courts judgement. In that case often the nautical interests will win, as they can rely on "the elder legal right". The case may then go to the government, which can judge with regard to the total national-economical background of the project.

When all the premises have been clarified, the road administration, which normally is responsible for the planning, the construction and the maintenance of the roadway and the bridge, has to make the final decision regarding spans, piers, design-assumptions and protective measures of the bridge. In that way a certain risk-level is determined, but it is not very well-defined.

The risk-level, without taking special regard to the collision-risks in design, may be estimated from the Swedish experiences to a collision-probability of $\frac{5}{200 \times 20} = 1{,}25 \cdot 10^{-3}$, based on 5 collisions at 200 bridges during 20 years. (All movable bridges are included here). This ratio is higher than what can be obtained from the fact that about 1,5 collisions occur each year on about 10.000 risky bridges all over the world. The Swedish ratio might be reduced to the order of 10^{-4} to 10^{-5} p.a. by adopting all the above mentioned risk reducing measures. The international ratio may increase, if <u>all</u> accidents should be reported.

3. REVIEW OF EXISTING BRIDGES

3.1 Guidelines

After the Tjörn-accident the Swedish Road Administration made a survey of all Swedish bridges across shipping channels in order to detect especially exposed bridges. In the study the following presumptions were made:

- all bridges designed for ships-collision-forces (as above) are regarded as safe
- all bridges with a navigation-channel used by ships of 500 dwt or less are regarded as safe
- all movable bridges are regarded as safe, as they are under permanent observation by the operator, they are always equipped with special guard-railings and they are passed by the vessels with outmost caution.

3.2 Results

The Swedish Road Administration is responsible for totally about 11.300 road-bridges, about 140 of these are movable. The first round of the study brought about 60 bridges to discussion, beside of the movable bridges. The above mentioned presumptions left in the last round only six bridges for special considerations of measures.

At two bridges the thin-walled box-piers have been filled with reinforced concrete. At one bridge artificial islands were established around two of the piers. At one bridge a by-pass channel was closed for all navigation. At four bridges the navigational channel and the visibility of the bridge piers were improved and on three bridges radar-echo equipment of the most effective modern type was installed.

There are remaining risks on two big bridges, where the collision-force capacity could be improved to about 10 MN, but it seems for technical and economical reasons impossible to improve it more. Because of large depth of water over large areas, protective piers become very expensive and uneffective. Floating protective equipment seems not adequate as it must be removed during winter, when there is heavy ice on the sea and no navigation. Everything is done to improve navigational safety. As the frequency of big ships is low, we are going to accept the remaining risks for the moment. In future the complete closure of the navigation channel for ships over 2000 dwt is discussed for one bridge. At the other bridge the need of navigation with big ships is decreasing because of changes in the industrial structure of the area.

On the actual bridges also permanent warning systems have been discussed, which can stop the road circulation if something happens to the bridges.

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Ship and Bridge Collisions - The Economics of Risk

Collisions: risque du point de vue de la science économique

Kollisionen: Risikofaktoren aus wirtschaftlicher Sicht

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SUMMARY

Bowen Bridge, Hobart, Australia is being constructed as a back-up to Tasman Bridge, which was disrupted for three years following a ship collision in 1975. The economic evaluation of Bowen Bridge illustrates the objective analysis of the risk of bridge collapse, the disruption costs which can be avoided, and the initial costs of measures which reduce disruption costs. The cost/economic/risk equation is illustrated by a powerful graphical method developed for this case. The method is suitable for general use in evaluating a new bridge across shipping lanes.

RÉSUMÉ

Le pont Bowen de Hobart, Australie, a été construit comme complément au Pont Tasman, dont l'usage a été interrompu par suite de la collision d'un navire en 1975. L'évaluation économique du pont Bowen explique l'analyse objective du risque d'écroulement des ponts, le coût d'interruption qui pourrait être évité et le coût initial de mesures réduisant le coût d'interruption. L'équation coût/économie/risque est illustrée par une méthode graphique et dynamique qui a été développée pour ce cas. La méthode est destinée à un usage général pour évaluer un nouveau pont au travers de voies de navigation.

ZUSAMMENFASSUNG

Die Bowen-Brücke in Hobart, Australien, ist als Zusatzbrücke zur Tasman-Brücke gedacht, die nach einer Schiffskollision im Jahre 1975 drei Jahre lang verkehrsuntauglich war. Die vorliegende ökonomische Bewertung der Bowen-Brücke enthält eine objektive Analyse des Risikos von Brückeneinsturz, der Folgekosten, welche vermieden werden können, und der Kapitalkosten von Maßnahmen, welche die Folgekosten einer Verkehrsunterbrechung verringern. Das Verhältnis zwischen Kosten, Wirtschaftlichkeit und Risiko ist anhand einer überzeugenden graphischen Methode dargestellt, welche für die vorliegende Analyse entwickelt wurde. Diese Methode ist für die Bewertung einer neuen Brücke über Schiffahrtswege allgemein gültig.

1. INTRODUCTION

The possibility of ship bridge collisions is a matter that must be taken into consideration in the design of bridges over navigable waters. It is preferable that this possibility be incorporated in an explicit manner, and within a rigorous framework.

In an economic evaluation study [1] for a new river crossing following the collapse, due to ship collision, of the Tasman Bridge in Hobart, Australia, a methodology for incorporating the possibility of ship bridge collisions into the decision frame was established. The method considered simultaneously the probability of ship bridge collision and the uncertainty associated with the measurement of disruption costs.

The basic approach of that study is described in this paper. The way in which the methodology can be used in the general case as an aid to selecting the appropriate risk level and thence in setting design criteria is also explained. The importance of this research is that it shows how, even when the disruption cost associated with bridge collapse is uncertain, it is still possible to utilize cost-benefit analysis to derive the most appropriate design criteria.

2. BACKGROUND TO THE EVALUATION STUDY

Three spans of the Tasman Bridge in Hobart, Australia were demolished by ship collision in January 1975. There were substantial economic and social disruption costs as a result of this unexpected closure. Tasman Bridge, was re-opened in October 1977.

Bowen Bridge is now being constructed 7 kilometres upstream from the Tasman Bridge, at a cost of approximately $A35 million (1983 dollars). It will be opened to traffic in 1983. Economic analysis demonstrated that its primary purpose was to provide an alternate river crossing in the event of a future closure of Tasman Bridge. The principal economic benefit therefore, is the avoidance of disruption cost in the event of a future ship collision with the Tasman Bridge, an insurance benefit.

The population of Hobart (at June 1974) was approximately 150,000, persons of which some 45,000 persons lived on the eastern shore of the Derwent River and 105,000 on the western shore. Economic and social activities for the 150,000 Hobart residents were heavily dependent on the single transport link, the Tasman Bridge. The only alternative road link between the two shores was the Bridgewater Bridge involving a one-way trip of 43 kilometres (86 km round trip). Disruption costs associated with collapse of the Tasman Bridge therefore included the massive disruption of economic and social linkages within the city as well as the costs of temporary bridging and of rebuilding the Tasman Bridge.

The situation thus provided a unique opportunity to develop a methodology that would enable the risk of ship bridge collision to be included in the decision frame. The objective of the study was to evaluate the proposed second crossing (Bowen Bridge), of the Derwent River. To assess the proposed Bowen Bridge a means had to be developed to incorporate, in a rigorous way, the possibility of future closure of the Tasman Bridge and the wide variation in the estimates of avoidable disruption costs.

This work was undertaken prior to a decision to fund the construction of the second bridge and the results of the evaluation were a significant input into this decision.

3. COST AND BENEFITS

The economic evaluation was done at a time when the design of Bowen Bridge was substantially completed. The design involved full protection of all river piers [2]. As foreseeable river traffic involved only small vessels (up to 5000 tonnes displacement) the safety of the new bridge against serious damage arising from a ship collision could be guaranteed.

Thus the cost of Bowen Bridge including costs of supervision and approach roads could be accurately estimated and at mid 1978 prices was $28.5 million.

As construction expenditure would take place over 3 years the present value of these expenditures ranged from $28.5 million at zero rate of discount; $26.1 million at 5% rate of discount; $24.8 million at 7% rate of discount and $23.4 million at 10% rate of discount.

There are three major identifiable economic and social benefits accruing from the construction of Bowen Bridge. They are:

- o Reduction of disruption costs from a further collapse of Tasman Bridge - the insurance benefit
- o Traffic facilitation due to the additional traffic lanes across the Derwent River provided by Bowen Bridge
- o Cost reductions for new urban development

The largest of these is the insurance benefit which is discussed separately below. The other benefits are discussed briefly now.

The urban development benefit is the reduction in the cost of servicing new urban settlements. It is calculated by comparing the pattern of development and associated infrastructure budget if Bowen Bridge is constructed, with alternative budgets that are associated with other selected development patterns. The net present value of this benefit is within the range of $0 to $5 million.

Two types of traffic benefits were calculated:

- o Some existing trips will be reduced in length or cost by the availability of the bridge. The benefit is the saving in vehicle operating costs and travel time.
- o Some trips not now made will be made because the bridge is there. In this case the surplus value of the trip is equal to the difference between the cost of making the trip and the intrinsic value of the trip.

The net present value of traffic benefits for the three rates of discount were $6.6-10.8 million for a 5% discount rate, $4.7-7.7 million for a 7% discount rate and $3.3-5.4 million for a 10% discount rate. That is the present worth of the traffic benefit lies approximately in the range of $3m to $11m.

4. DISRUPTION COST ANALYSIS

It will be seen that even at high values of traffic and urban development benefit, these benefits cannot in themselves justify the cost of constructing the second crossing. It was therefore necessary to obtain an estimate for the third type of benefit, avoidance of disruption cost.

Disruption cost analysis [1] provides a methodology for assessing the benefits of projects designed to avoid or minimise future disruption costs caused by expected events. The methodology can be applied to both common and infrequent events. It is necessary to postulate two time series of disruption costs; one if the project is not undertaken and a second if the project that will reduce disruption costs is undertaken. These time series of expected disruption costs can be translated into present worth values once the probability of experiencing disruption in each year of the future is known and a discounting factor selected. The expected present value of the benefit is the difference between the two present worth estimates.

The analysis is developed as follows:

D_n = Disruption cost in year n given the disruption event occurs

Dk_n = Disruption cost in year n given the disruption event occurs and the project being evaluated has been implemented

DA = Avoidable disruption cost = D-Dk

p_n = Probability of disruption event occurring in time period n

i = Rate of discount

g = Real annual rate of growth of disruption costs

PW = Present worth

The present worth equations are:

$$PW\ (D) = \frac{D_1 p_1}{(1 + i)} + \frac{D_2 p_2}{(1 + i)^2} + ---- + \frac{D_n p_n}{(1 + i)^n} + ----$$

A similar equation may be written for PW (Dk)

If the following simplifications are made:

- o The probability of the disruption event is equal in each year i.e. $p_1 = p_2 = p_n$
- o Given that the project is not undertaken the disruption cost is the same in each future time period i.e. $D_1 = D_2 = D_n = D$
- o Given that the project is undertaken the disruption cost is the same in each future time period i.e. $Dk_1 = Dk_2 = Dk_n = Dk$

Then PW (DA) = PW (D) - PW (Dk) which by algebra reduces to:

$$PW\ (DA) = p\ [\frac{DA}{(1 + i)} + \frac{DA}{(1 + i)^2} + ---- + \frac{DA}{(1 + i)^n} + ----\ ]$$

If the growth factor g is now introduced

$$PW\ (DA) = p\ [DA\ \frac{(1 + g)}{(1 + i)} + DA\ \frac{(1 + g)^2}{(1 + i)^2} + ---- + DA\ \frac{(1 + g)^n}{(1 + i)^n} + ----]$$

For small values of g the infinite series reduces to

$$PW\ (DA) = \frac{p\ DA}{i-g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 1.$$

That is the present worth of future disruption cost avoidable by the specified project is equal to the probability of collapse in any year, multiplied by the disruption cost avoided when the disruption event occurs, divided by the discount rate less the rate of growth in disruption cost.

If the factor P/i-g is calculated for various values of p, i and g it is easily demonstrated that even for a relatively low probability event the present worth of the disruption cost is a significant percentage of the disruption cost when it occurs. Suppose the probability of the event occuring is once in every 100 years and that the net discount rate (i-g) is 3 percent then the present worth of future disruption cost is equal to 33 percent of the contingent disruption cost (in the year the disruption occurs). This indicates that for this probability, if the contingent disruption cost is high, it is appropriate to spend a considerable sum to avoid that disruption. One approach would be to consider the costs of decreasing to say 1 in 1000 or 1 in 10000 years, the probability of the disruption event occurring. This would, in many cases require the selection of new design criteria for the bridge. Another approach would be to consider projects that would reduce the magnitude of disruption costs in the event that a disruption occurs. (The

Bowen Bridge solution to the possibility of collapse of the Tasman Bridge is an example of the latter approach).

5. INSURANCE BENEFIT

The insurance benefit for Bowen Bridge was determined using disruption cost analysis as follows:

Avoidable government disruption costs (based on the Tasman Bridge experience) were calculated to cover items such as temporary bridging, additional government services, roads, ferries, ferry terminals, ferry subsidy, additional bus services. In 1978 dollars these were assessed at a lower bound estimate of $10 million and an upper bound estimate of $22 million.

Avoidable private disruption costs were calculated to cover three items; value of additional travel time, additional money costs of travel and value of trips foregone. In 1978 dollars these were assessed to have a lower bound estimate of $18 million and an upper bound estimate of $37 million.

Thus the total avoidable disruption costs were calculated to be in the range $28 million to $59 million.

The present worth of avoidable disruption costs was calculated using formula 1., given above.

The probability of a future collapse of Tasman Bridge was determined to have a recurrence interval of between 10 years and 40 years [3] [4]. The value of p which is the reciprocal of the recurrence interval was therefore assessed to be between 0.1 and 0.025.

The net rate of discount (i-g) was taken as a variable of 4%, 6% and 9% consistent with a rate of discount of 5%, 7% and 10% with a 1% rate of growth in disruption.

The present worth of avoidable disruption cost (the insurance benefit) was calculated using the above estimated ranges for avoidable disruption cost, net discount rate and probability of collapse of the Tasman Bridge, and was calculated to lie within the range of $8 million and $148 million; as shown in the table below:

1/p Recurrence Interval years	i					
	5%	7%	10%	5%	7%	10%
10	70	47	31	148	98	66
40	18	12	8	37	25	16
	DA = $28m lower bound			DA = $59m upper bound		

6. DECISION FRAMEWORK INCORPORATING PROBABILITY OF COLLAPSE

The aggregate total of present worth of benefits is therefore as follows.

Urban Development	-	$ 0 to $ 5 million
Traffic	-	$ 3 to $ 11 million
Insurance	-	$ 8 to $148 million
Total	-	$11 to $164 million

The range for present value of project benefit is extremely wide and this information as such is of limited value. Consequently techniques were developed for calculating the probability that project benefit is greater than project cost.

Each estimate of aggregate project benefit depends on the values assigned to nine parameters and a large number of estimates of project benefit is possible. A computer model was developed to calculate the probability that the aggregate benefit of the project exceeded any particular amount. This probability was calculated for various rates of discount and a range of assigned recurrence intervals (probability of collapse of Tasman Bridge); these two parameters having most influence on the calculated project benefit.

For each of the other seven parameters namely:

Unit cost of vehicle operation
Value of travel time in normal circumstances
Value of travel time in abnormal circumstances
Weeks to construct temporary crossing
Weeks to reconstruct Tasman Bridge
Government expenditure in year of collapse
Urban development benefit

the probability that the true but unknown value of the parameter lay at various points of the range was assessed and a probability distribution established for each of these parameters. The technique used is illustrated below for the value of travel time in abnormal circumstances.

Value of travel time (dollars per hour)	Probability that true value exceeds selected value
2	80%
3	40%
4	0%

(In this case the value of travel time was resticted to the integer values of 2, 3 and 4).

With the probability assessments for each parameter it was possible to calculate the probability that the aggregate benefit would exceed any value of aggregate benefit for each set of collapse probability and discount rate values. This provides a cumulative probability distribution. The results are presented graphically as shown in Figs. 1-4.

Conclusions from the graphs are easily drawn. For instance for a median project benefit (50% probability that project benefit is greater) and for discount rates of 5%, 7% and 10% project benefit exceeds project cost when probability of collapse is less than once in 80, once in 50 and once in 30 years respectively. The result of the evaluation therefore indicates that the aggregate benefits of the Bowen Bridge most likely exceeds its cost.

In this context it is noted that a separate study [3] showed that the cost of protecting the piers of Tasman Bridge against ship collision was far greater than the cost of constructing a back up bridge.

As the Tasman Bridge, which is undoubtedly vulnerable to further ship collisions, is not being protected the remaining matter to be resolved was that of protecting the public using the bridge. The restored bridge which carries 50,000 vehicles per day has computer controlled traffic lights, on gantries, for tidal flow of traffic in peak hours. This system was modified simply and cheaply to enable the bridge to be used in a manner similar to a railway level crossing. In peak road traffic periods ships are not permitted to navigate the bridge. At all other times the bridge deck is completely cleared of all traffic while a ship passes beneath the bridge. The traffic delay is about 3 minutes and the public have not objected.

7. RISK LEVELS USING DISRUPTION COST ANALYSIS

Risk models can be and usually are established by engineers, particularly for consideration of problems such as ships hitting bridges. Engineering parameters such as statistics of shipping, distribution of ship sizes, the fraction of passing ships which are uncontrollable (causation probability),

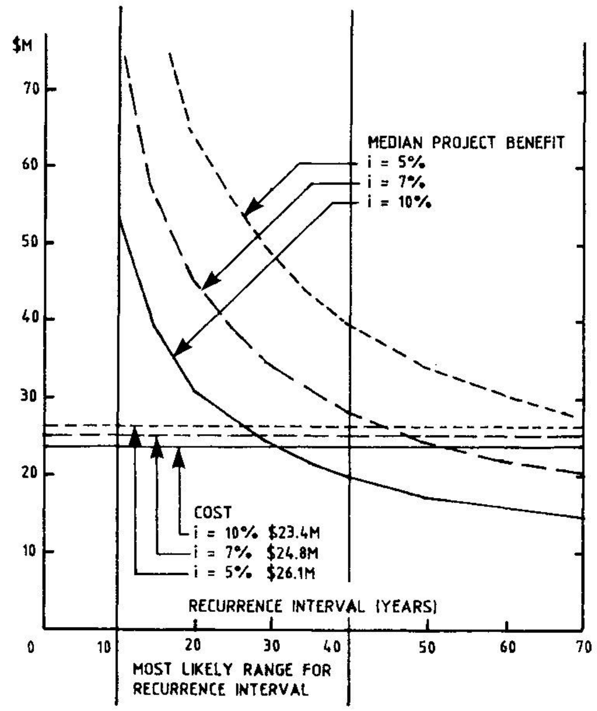

Fig. 1 Median Project Benefit by Recurrence Interval and Discount Rate (Million dollars, 1978 prices)

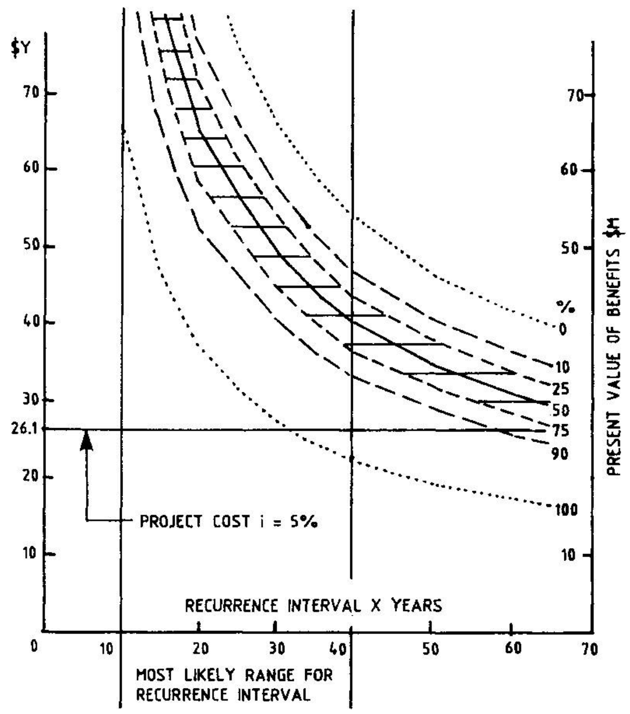

Fig. 2 Probability that Project Benefit is Greater than $Y for Recurrence Interval of X Years for Discount Rate 5%

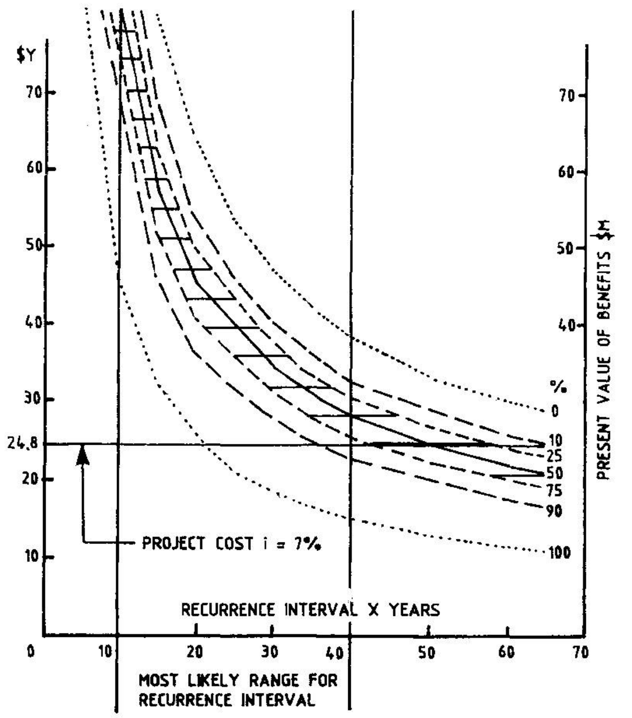

Fig. 3 Probability that Project Benefit is Greater than $Y for Recurrence Interval of X Years for Discount Rate 7%

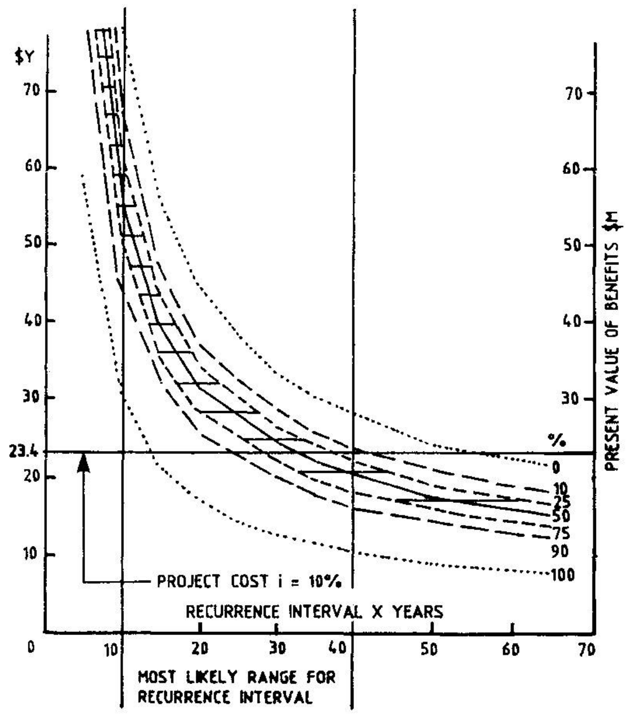

Fig. 4 Probability that Project Benefit is Greater than $Y for Recurrence Interval of X Years for Discount Rate 10%

the probability of a ship out of control hitting a pier (geometric probability) are available to estimate the biggest ship which can hit a pier in a given period. The difficult question is the choice of the acceptable recurrence interval or risk level of the structure.

Disruption cost analysis as described in this paper provides a framework based on cost benefit analysis which will aid the choice of risk level on a rational basis. In the past cost benefit analysis has not generally been used due to the variability of the parameters and the difficulties in evaluating the economic consequences. The method described in the paper which deals with the variability of parameters on a probability basis provides a satisfactory way of presenting the cost benefit data in graphical form so that the information is both comprehensive and easy to assess, thus leading to an informed decision on the risk level to be adopted.

The method can be applied to a "greenfields" site where a new major bridge is to be built across existing shipping lanes. Presumably the decision to construct such a bridge in the first place would be justified on economic grounds; that is the economic benefit derived from its construction exceeds its cost. In considering the design of the bridge the risk level to be adopted, the number of piers in navigable water versus the cost of longer spans etc. can be determined on the basis of disruption cost theory starting from the economic costs associated with the disruption of this benefit. In this context and in hindsight it is interesting to consider the design of Tasman Bridge (carried out in 1956). This bridge has 20 piers [5] in navigable water with spacings of generally 43m. The overriding consideration of the design at the time was capital cost. The authors suggest that if the bridge were designed today, using the disruption cost analysis described in this paper, the resulting design would have been totally different with longer spans and considerably higher initial capital cost, which would have been seen to be fully justified.

The disruption cost method might even be extended to the general level of safety for which major structures should be designed. With the advent of limit state design theory the concepts of the resistance R and load Q effect and are well established. Typically 5 and 95 percentile values are chosen for the characteristic values R_k and Q_k in specifying design values for checking ultimate (or collapse) limit states, while mean values are used in considering serviceability limit states. With most codes such an approach leads to a Safety Index (β) for individual elements of approximately 4. This is roughly equivalent to a probability of failure of 10^{-4}. Disruption cost analysis could help to provide an answer to the question (assuming that it is posed) of whether such typical levels of structural safety are satisfactory or desirable for a particular structure of major significance (and presumably substantial economic benefit) which is being designed.

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Bridge Risk Assessment and Protective Design for Ship Collision

Evaluation des risques de collision et plan de protection.

Risikoschätzung und Schutzentwurf gegen Schiffskollisionen.

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SUMMARY
Risk assessment procedures and assumptions for the 464 m cable stayed Annacis Island Bridge span at Vancouver, Canada, are described, preceded by a prescriptive account of a method for adopting a design return period that provides the appropriate balance between risk and initial cost.

RÉSUMÉ
Les procédés et hypothèses pour l'évaluation des risques dans le cas du pont haubanné Annacis Island de 464 m d'envergure à Vancouver, Canada, sont décrits. L'équilibre entre le risque et les coûts initiaux permet d'évaluer la durée de vie de l'ouvrage.

ZUSAMMENFASSUNG
Die Verfahren und Annahmen fur die Schätzung der Risiken der 464 m langen Schragseilbrücke Annacis Island in Vancouver, Kanada, werden beschrieben. Der Vergleich von Risiken und Initialkosten läßt die Lebensdauer des Werkes schätzen.

0. INTRODUCTION

Safety levels in structural engineering are generally established by precedent. However, in the case of ship collision on bridges there are few precedents, and economic sense requires that the choice of acceptable risk be site-specific because the cost of protection varies widely with circumstances. Acceptable risk should be that which minimizes the sum of present expected value of future collision consequences plus present protective expenditures. A methodology for adopting the appropriate risk level is presented in Section 1 using hypothetical numerical values for example purposes.

The choice of acceptable risk depends on the relations between risk (or return period for catastrophic collapse) and cost of protective works. In order to illustrate how this relationship can be developed and to demonstrate a practical risk analysis for an actual project, some aspects of the risk assessment study for the Annacis Island Bridge, Vancouver, are briefly presented. This 465 m span cable stayed bridge is being designed in both concrete and steel alternatives for the British Columbia Ministry of Transportation and Communications. CBA-Buckland and Taylor are responsible for the steel alternative and for the ship collision risk assessment. A model of the bridge appears in Figure 1, and an elevation and layout are shown in Figure 2.

1. CHOICE OF ACCEPTABLE RISK

The provision of protection to a bridge gives insurance against serious damage or catastrophic collapse of the bridge due to ship collision. Thus the amount spent initially should be related to the degree of protection, and to the consequences of collision. For the case of a heavily travelled bridge, the acceptable return period for catastrophic collapse is so great that the risk to human lives from ship collision will be relatively small compared with that accepted by the motorists in their normal travel over the bridge. Thus a focus on only economic consequences is realistic. Available traffic statistics can be used to verify this assumption, and traffic warning devices can be developed to improve the safety of motorists (1).

The objective is to choose the level of risk (in terms of return period for catastrophic collapse) that provides a minimum of the sum of protection costs and expected present value of future consequences. The bridge owner, or his consultant, is assumed to be the decision maker and will pay the costs of protective works and absorb losses due to catastrophic collapse.

Excluded are losses associated with the vessel, such as cargo spills. These losses cannot necessarily be mitigated by the bridge protective works, in fact the chance of collision may be increased by construction of protective works. Risk management decisions that include vessel or cargo losses must be made at the bridge conceptual stage because they may strongly affect the site selection and pier locations.

The methodology suggested here is based on well established principles of minimum expected cost optimization (2). Some useful commentary on issues raised by the approach is found in (3) and (4).

1.1 Present Value of a Future Loss

Assume that a loss C_f will occur at a definite time in the future. Then the present value of this loss is

$$C_o = C_f e^{-it} \qquad (1)$$

where

- C_o is the present value of the future loss
- C_f is the future loss in present monetary units (not inflated)
- i is the real interest rate (excluding inflation)
- t is the time to the future loss.

Note a number of assumptions, i.e. constant real rate of interest and known real cost of consequences. Inflation need not be considered because it is assumed that the rate of return on an investment will be the sum of the inflation rate and the real interest rate.

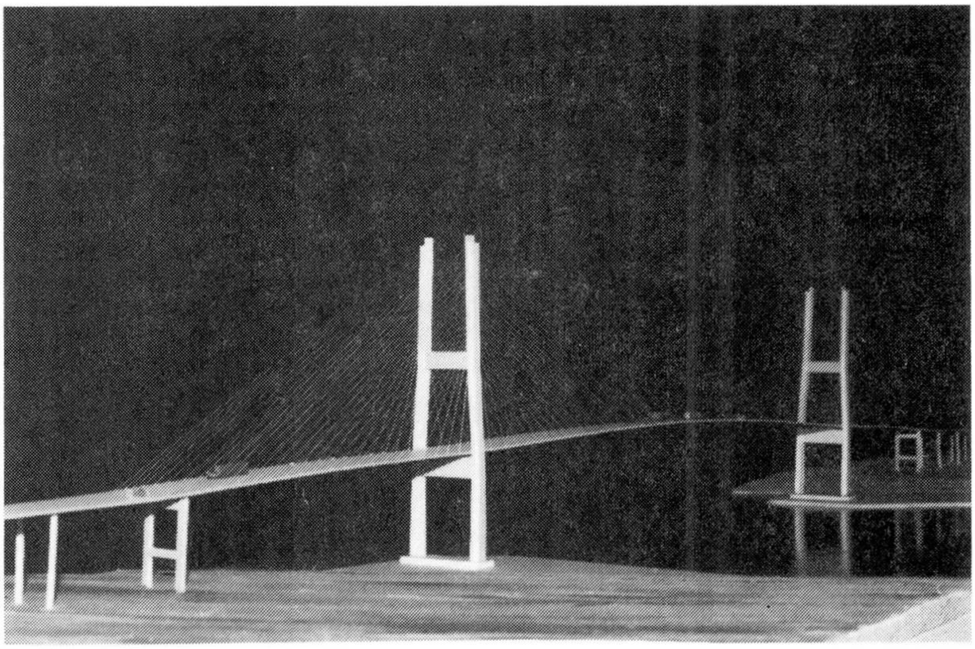

Figure 1 Model of the 465m Annacis Island Bridge

1.2 Present Value of a Series of Future Losses

Ship collision consequences are expected to have return periods of the order of hundreds or thousands of years. The present value of the consequences of a second loss in a series will be negligible compared with those of the first loss in the series. It is therefore reasonable to limit the problem to consideration of the first occurrence of catastrophic collision only.

1.3 Present Value of a Future Loss Occurring at Random Time

When the time to occurrence of the loss is a random variable, the present expected value, based on Equation 1, becomes

$$C_o = E[C_f e^{-it}] \qquad (2)$$

$$= C_f \int e^{-it} f(t)\, dt \qquad (3)$$

where

- C_o is the present expected value of the future loss
- E(.)is the expectation operator
- f(t)is the probability density function on t, the time to occurrence of the catastrophy.

Ship collision events are rare and are independent random events in time. They can therefore be considered as Poisson events. The time to first occurrence is therefore exponentially distributed.

$$f(t) = ue^{-ut} \qquad (4)$$

where u is the rate parameter (reciprocal of return period).

It follows that

$$C_o = \int C_f e^{-it} u e^{-ut}\, dt \qquad (5)$$

$$= C_f u/(i+u) \qquad (6)$$

As an illustrative example, assume a loss of $\$2x10^8$ in the event of catastrophic collapse due to ship collision and assume that such an event has annual probability u = 0.001 or return period T = 1000 years. Assume a real interest rate of 3% or i = 0.03. Then

$$C_o = \$2x10^8(0.001)/0.031 = \$6.5x10^6$$

It should be emphasized that the losses include direct economic losses associated with the bridge structure, as well as indirect losses that occur as a result of loss of use of the bridge. The latter may be considerable (1).

1.4 Acceptable Risk or Return Period

In the hypothetical example of the foregoing section, a return period of 1000 years is assumed. In fact it is more straightforward to develop some reasonable design for protection of the bridge, then estimate its cost and the resulting return period. Assume that the return period 1000 years in the example was for failure of a protective system with the ability to absorb 1000 MN-m of energy before collapse of the bridge. Assume that the cost of this protective system is $C_c = \$6x10^6$. Then the present value of total expected cost chargeable to ship collision is $C_o+C_c = \$6.5x10^6+\$6.0x10^6 = \$12.5x10^6$. The objective is to find the level of risk that minimizes this sum. Table 1 shows assumed values for two other trial designs, one for protection for 800 MN-m and another for 1400 MN-m. Each of these has a corresponding annual rate of exceedence (or probability that greater energy will be found). The calculations show that the design for the 1000 year return period is optimal in this example, because the total cost Co + Cc is a minimum. Only a few discrete trial designs need be prepared in order to find a near-minimum in the cost function, as it will normally not be sensitive to minor changes in the trial energy levels.

Table 1

Energy Capacity (MN-m)	Annual rate of Exceedence	Return period for Exceedence T	C_o	C_c	C_o+C_c
800	.002	500 yrs	$12.5x10^6$	$3.0x10^6$	$15.5x10^6$
1000	.001	1000 yrs	$6.4x10^6$	$6.0x10^6$	$12.5x10^6$
1400	.0002	5000 yrs	$1.3x10^6$	$12.0x10^6$	$13.3x10^6$

It is evident that when the consequences are very high, or when the cost of protection is relatively low, a high level of protection is optimal. Conversely, when the cost of protection is high or the consequences are low, little protection is justified. The analysis can properly weigh these variables to approach an optimal solution, but it is important to recognize the approximate nature of all the input variables. The calculation of risk, and the actual capacity of a proposed protective work, are very crude.

2. RISK ASSESSMENT FOR THE ANNACIS ISLAND BRIDGE - A CASE STUDY

2.1 The Site

The bridge site is at the east end of Mungo bend in the main channel of the Fraser River at the approach to the Port of New Westminster near Vancouver, Canada. The main piers proposed are outside the navigation channel but water depth is sufficient to expose the piers to possible ship collision. The superstructure is set to clear all shipping and is not considered to be at risk. Figure 2 indicates the shipping channel and clearances.

River current can be slightly upstream at flood tide and low flow periods, and reaches about 2 m/s downstream at ebb tide and high flow periods. Water elevation varies from -1.2 m to + 3.5 m.

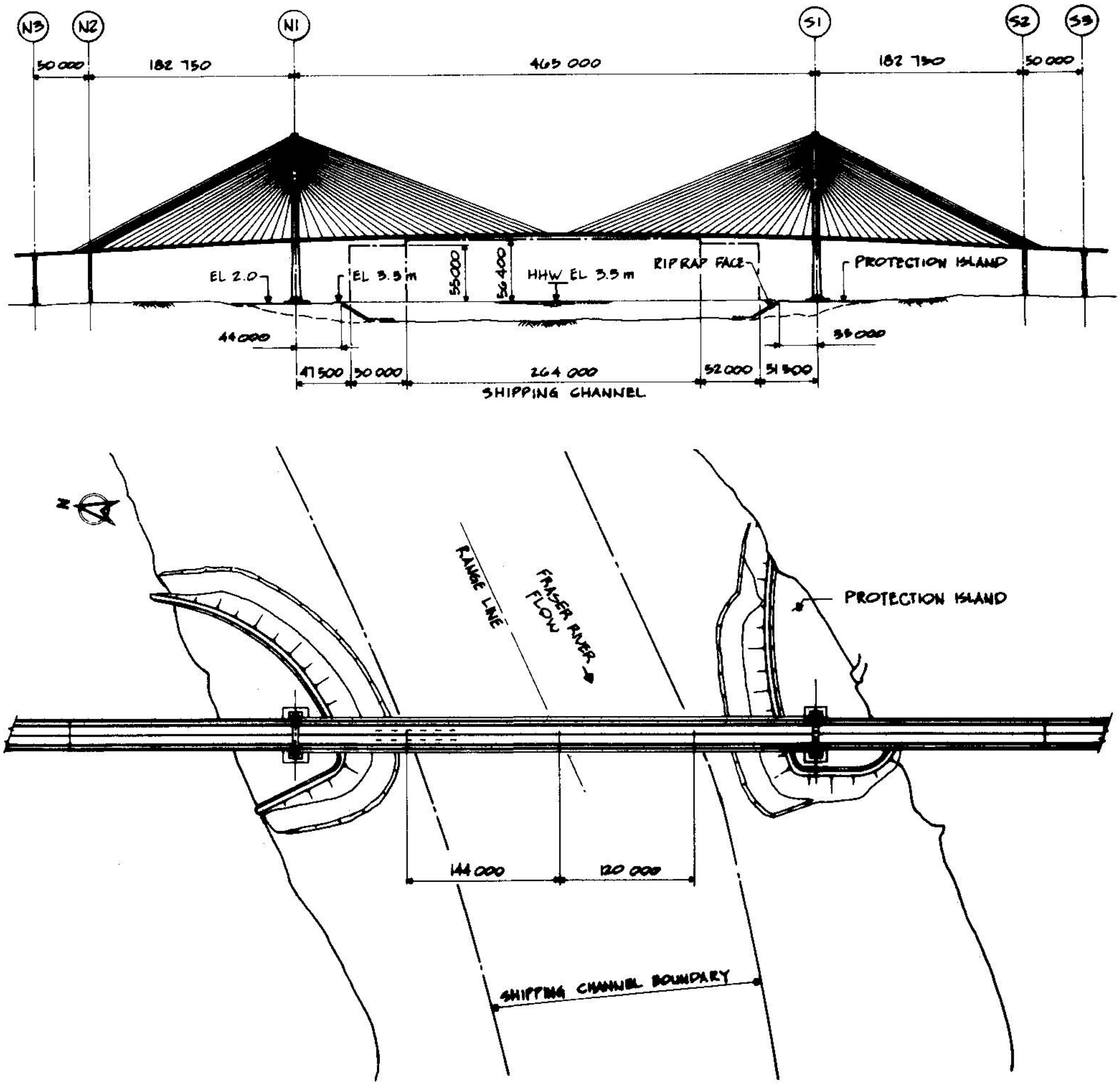

Fig. 2 Annacis Island Bridge, Arrangement of Protection Islands

2.2 Vessel Traffic Characteristics

Traffic types and volumes were obtained from logs of the Pacific Pilotage Authority, which supplies pilot services for vessels larger than 300 deadweight tons. One year of records were used to develop a discrete relative frequency model for vessel mass (Table 2). Accuracy of this model at the lower mass values is poor, but the larger vessels are well documented. The model has several limitations; prediction of the future from the chosen record is a major one. The rather coarse discrete increments chosen facilitate numerical work and are justified by the very approximate nature of the entire analysis.

It was assumed that vessel speed at collision is independent of mass. This is not likely to be true, but is a convenient simplification. Based on observation, discussion with the Pilotage Authority, and records of river current, a discrete probability mass function was constructed of vessel speed at collision with the bridge pier (Table 3).

Table 2

Vessel Mass (Tonne)	Relative Frequency
20000	0.25
30000	0.30
40000	0.30
50000	0.10
60000	0.05

Table 3

Vessel Speed (m/s)	Probability
1.8	0.7
2.5	0.1
3.2	0.1
6.0	0.1

The probabilities are subjective of necessity because a survey of speeds of normal traffic cannot represent the collision scenario. Data from actual collisions would aid in construction of this type of information, but reliable data is still sparse.

2.3 Rate of Collisions with the Piers

Ten years of records of River traffic indicated 20 incidents, or 2 per year, involving loss of control in the approach channel to the site. The fraction of these that might occur near the bridge is taken as 0.2 because the length over which reporting is done is about 40 km while the region near the bridge is taken as 8 km. Thus incidents of interest are 0.2(2) or 0.4 per year.

This is probably quite conservative, since the reported incidents are mostly docking incidents, which have little bearing on the chance of striking a pier. For comparative purposes, it is worth noting that the "causation probability" (probability that a given ship will be uncontrollable while passing the vicinity of the bridge pier) used in the risk analysis for the Great Belt Bridge, Denmark, was taken as 2×10^{-4} (1). The comparable value used herein for Annacis Island is 0.4 incidents per year for 1098 transits past the bridge, or $0.4/1098 = 3.6\times10^{-4}$.

The channel is about ten times the width of a large vessel. It is assumed that the pier or its protective system would be struck with probability 0.1 given that a loss of control incident has occurred. This is the geometrical probability. Thus the net rate of arrival of collisions on piers is 0.4(0.1) = 0.04 per year. This is the accident arrival rate as far as the bridge is concerned. It is reasonable to assume that accidents are independent events with a small chance of occurrence in any given short time period. It then follows that the value 0.04 may be assumed to be the rate parameter λ of the Poisson stochastic process. The arrival rate gives the accident frequency for all ships in the size category sampled, i.e. greater than 300 deadweight tons. Obviously most of these accidents will involve the smaller ships and low energy. This arrival rate corresponds to about 4 accidents per 100 years, or a return period of 25 years.

2.4 Energy Content of Accidents

Assume that a vessel chosen at random from the relative frequency distribution of vessel masses (Table 2) has a velocity (as it strikes the pier protective system) from the probability mass function of velocities (Table 3). Mass and velocity are assumed independent. Then the energy E_k is computed for each combination of mass and velocity, and a probability distribution of energy is constructed. Table 4 gives the resulting conditional probability mass function on kinetic energy of vessels given that impact has occurred. Note that the energy is low with very high probability. It takes the rare joint event of a large vessel moving at a high speed to produce large energy.

Table 4

Vessel Energy (MN-m)	Probability
100	0.855
300	0.070
500	0.030
700	0.030
900	0.010
1100	0.005

Denote the event Q_0 no energy of impact in a given year, Q_1 the lowest energy level and so on for each of the discrete energy levels. Let H_0 be the event of "no collision" and H_1 be the event of a collision, in a given year. $H_0 = 0.96, H_1 = 0.04$. Then, by the theorem of total probability,

$$P(Q_i) = P(Q_i/H_1)P(H_1)+P(Q_i/H_0)P(H_0) \qquad (7)$$

for all Q_i. Thus

$$P(Q_0) = 0.96 \text{ (no energy demand in a year)}$$

$$P(Q_1) = 0.855(0.04) = 0.0342 \text{ (energy 100 MN-m)}$$

$$P(Q_2) = 0.070(0.04) = 0.0028 \text{ (energy 300 MN-m)}$$

and so on.

The probability mass function on the energy demand in one year, and the probability of equalling or exceeding a given level of energy in one year, are given in Table 5.

Table 5

Energy (MN-m)	Probability of Equalling	Probability of Equalling or Exceeding
0	0.96	1.0000
100	0.0342	0.0400
300	0.0028	0.0058
500	0.0012	0.0030
700	0.0012	0.0018
900	0.0004	0.0006
1100	0.0002	0.0002

The Table reflects the very high probability that nothing will happen in a given year, thus zero energy will be found. The non-zero values represent the other possible occurrences in a given year.

For the Annacis Bridge several levels of energy capacity were considered, corresponding to several levels of pier protection. Table 6 is a representation of the risk predicted for the bridge assuming a design protection level and then finding the probability that it will be exceeded from Table 5. The variation of risk with protection level is thus portrayed. Table 6 is similar to Table 1 in the preceding discussion of acceptable risk levels.

Table 6

Energy Capacity (MN-m)	Annual Rate of Exceedence u	Return Period T
800	.0006	1667
1000	.0002	5000
1200	.0001	10000
1400	0	inf

The choice of acceptable risk could be made at this point by establishing cost of consequences and cost of protective works for each of the four energy levels in Table 6. This was not done for the Annacis Bridge. Based on estimates of feasible protective works and comparisons of risk with other risks, such as earthquake, a design energy capacity for pier protection of 1200 MN-m was adopted. The corresponding return period is 10000 years.

2.5 Design of the Protection System

A study of available literature on protective systems shows great variety, depending on the magnitude of the energy demand and the circumstances at the site (6). The Annacis design energy demand is relatively large, but the superstructure is not exposed, and the piers are located well out of the navigation channel. After examination of several alternatives including collision dolphins (which cause extreme scour problems), a protective frangible concrete shell at the piers, and protection islands, it was decided to construct protection islands. The proposed design indicated in Figure 2, provides sand-filled protection islands faced with rip-rap. The elevation of the islands is 2.0 m, thus water level is above the islands only for a small percentage of the time. The islands provide a sliding or ploughing distance for a moving ship of about 25m through which energy of at least 1200 MN-m can be absorbed. Calculations were similar to those suggested in (5) for a ship grounding and sliding over the island.

Because the risk was assessed with approximate conservative assumptions, no experimental work on energy absorption in the islands was considered justified. Extensive hydraulic studies were performed at Western Canada Hydraulic Laboratories to assess scour and flow characteristics around the protection islands.

The protection islands provide technical advantages for the bridge foundations regarding settlement and access, thus their cost is partially justified on these grounds. The cost of the islands is of the order $\$5x10^6$, an expenditure that appears to be in the correct order of magnitude for protection of an investment with failure consequences in excess of $\$2x10^8$.

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Faisabilité économique des protections des piles de pont

Wirtschaftliche Durchführbarkeit von Brückenpfeilerschutz
Economic Feasibility of Bridge Pier Protection

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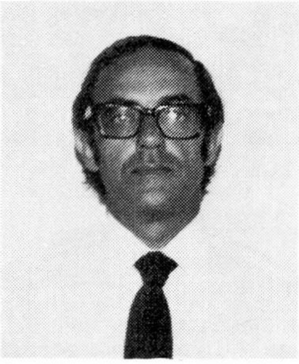

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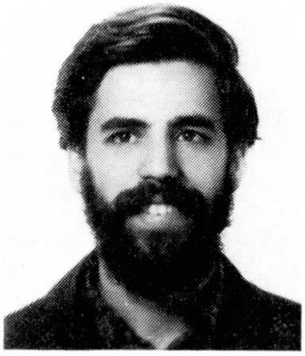

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RÉSUMÉ
L'article présente une méthode de calcul de faisabilité des protections et une méthode simplifiée d'estimation du coût d'interruption du service.

ZUSAMMENFASSUNG
Es wird eine Berechnungsmethode vorgeführt für die Durchführbarkeit von Schutzbauten und eine vereinfachte Kostenkalkulations-Methode im Fall einer Betriebsunterbrechung.

SUMMARY
A method for the calculation of the feasibility of protections and a simplified method for the estimation of the service interruption cost are presented.

1. PRESENTATION

Le risque économique dérivé du passage de bateaux sous les ponts doit etre évalué souvent pour les ponts existants et l'est, aussi,pour ceux qui sont en cours d'étude. On suppose ici que le pont existe déjà, mais le problème posé pour des ponts futurs peut être analisé d'une manière similaire. La question est de savoir s'il convient de prévoir des dispositif ou des renforcements spécialmente conçus.

La décision dépend de la comparaison entre le coût du risque et le coût des protections.

On présente una méthode de calcul et on introduit les notions d'éfficacité de la protection et de systeme de protection.

2. COÛT DÉRIVÉ DE LA VULNERABILITÉ D'UNE PILE

Si une pile est détruite il y a deux genres de dépenses:

C_1 = coût de réparation de la partie affectée.

C_2 = coût anmuel d'interruption du service (allongement des distances de transport, changement des moyens de transport, etc., voir 8)

Le coût total dérivé d'une pile détruite est donc:

$$CA = C_1 + C_2 \cdot t \qquad (2.1.)$$

où t est le temps de la réparation.
Si r est la durée de la vie utile de l'ouvrage et q l'année où se produit l'accident, le coût actuel est:

$$CA_q = Ca\ (1 + i)^{-q} \qquad (2.2.)$$

ou i = intéret annuel de l'argent

Si le flux de bateaux est constant, toutes les années ont la meme probabilité d'accident et le coût moyen d'un accident est:

$$CAP = \frac{CA}{r} \sum_{q=1}^{r} (1+i)^{-q} \qquad (2.3.)$$

Si m est le nombre total d'accidents pendant le temps r, le coût total des accidents est:

$$CV = m \cdot CAP \qquad (2.4.)$$

Si n est le nombre total de passages pendant le temps r, on sait que $0 \leq m \leq n$, et on peut atribuer une probabilité p_m a chaque valeur m.

L'espérance mathématique du coût total des accidents, c'est à dire ce que'on peut s'attendre a avoir a dépanser, est:

$$E_{(CV)} = \sum_{m=0}^{n} p_m \cdot m \cdot CAP \qquad (2,5.)$$

3. PROBABILITÉ DE QUE LA PILE SOIT DÉTRUITE m FOIS

Si p_d es la probabilité de que la pile soit détruite quand il y a un seul passage, on a:

$$p_m = p_d^{\,m} \cdot (1 - p_d)^{n-m} \cdot \frac{n!}{m!\,(n-m)!} \qquad (3.1.)$$

Mais, en réalité, il y a une limite supérieure N pour m, $N < n$, étant donné que, après un accident qui détruit une pile, il existe un temps mort pendant lequel il ne nous intéresse pas s'il y a d'autres chocs contre la même pile. Nous supposons que nous pouvons considérer comme temps mort le temps t de réparation de la pile. Le nombre maximum d'accidents pendant la vie utile r est donc: $N = r/t$, et $0 \leq m \leq N$.

Si le flux de bateaux est constant, le nombre des bateaux qui passent dans la période t est $N_t = n \cdot (t/r)$. La probabilité de que la pile soit détruite au moins une fois dans cette période est:

$$p_{dt} = 1 - (1 - p_d)^{N_t} \qquad (3.2.)$$

La probabilité p_m doit donc etre calculée au moyen de la formule:

$$p_m = p_{dt}^{\,m}\,(1 - p_{dt})^{N-m} \cdot \frac{N!}{m!\,(N-m)!} \qquad (3.3.)$$

et la limite de la somme (2.5.) est N au lieu de n.

La probabilité p_d de qu'une pile soit détruite est le produit de la probabilité de qu'elle soit heurtée p_{ch} par la probabilité de que le bateau ait una énergie suffisante pour la détruire p_e:

$$p_d = p_{ch} \cdot p_e \qquad (3.4.)$$

La probabilité p_{ch} est le produit de la probabilité de qu'un bateau soit "sans contrôle" (causation probability) p_c par la probabilité de que ce bateau heurte effectivement la pile (probabilité geómétrique) pg:

$$p_{ch} = p_c \cdot p_g \qquad (2.5.)$$

La probabilité p_c est de l'ordre de 10^{-4} [1]; la probabilité p_g peut être estimée par exemple, d'après Buffon, sous la forme:

$$p_g = \frac{L}{2\pi \cdot d} \qquad (L \leq 2d) \qquad (3.6.)$$

où L est la longueur moyenne des bateaux qui passent et d la distance de la pile considérée au centre du canal de navigation.

La probabilité p_e, à vitesse de passage uniforme pour tous les bateaux, est égale a la probabilité de que la masse du bateau soit supérieure a une valeur donnée (par une analyse dynamique de la structure). Si la vitesse n'est pas uniforme la probabilité p_c sera une fonction des probabilités de la vitesse et de la masse.

4. COÛT DE LA PROTECTION

Le coût de la protection est function de trois paramètres:

C_3 = coût d'installation de la protection

C_4 = coût de maintenance

C_5 = coût de réparation dans le cas où la protection soit heurtée

Si le coût annuel de maintenance est M_a, on a:

$$C_4 = M_a \cdot \sum_{q=1}^{r} (1 + i)^{-q} \qquad (4.1.)$$

Si nous appelons C_r le coût de réparation de la protection chaque fois qu'elle est heurtée, le coût moyen pendant la durée de vie utile r est:

$$C_5 = \frac{C_r}{r} \sum_{q=1}^{r} (1 + i)^{-q} \qquad (4.2.)$$

En supposant que le temps mort après un choc est du même ordre que celui du pont, l'espérance mathématique du coût de la protection est:

$$E_{(CD)} = C_3 + C_4 + \sum_{m=0}^{N} p_m \cdot m \cdot C_5 \qquad (4.3.)$$

5. EFFICACITÉ DE LA PROTECTION

Toutes les solutions techniques au problème de la protection des piles ne sontpas 100 % efficaces, soit parce qu'elles n'entourent pas complètement la pile, soit parce qu'elles ne supportent pas nécessairement l'énergie cinétique maximun que l'on peut espérer. En effet, on peut démontrer que, en général, il n'est pas économique de prévoir des protections pour cette énergie a moins qu'elle ait una probabilité d'occurrence suffisament élevée. On doit donc, en général, attribuer à la protection une efficacité e ($0 < e \leq 1$) qui réprésente la proportion des chocs qui seront effectivement évités.

Le coût de la protection doit etre majoré du coût des chocs contre la pile qui ne serontpas évités:

$$E_{(CDC)} = E_{(CD)} + (1 - e) \cdot E_{(CV)} \qquad (5.1.)$$

6. LES SYSTÈMES DE PROTECTION DE PLUSSIEURS PILES

En général il faut étudier la faisabilité des protections de plusieur piles, ce que nous appelons un système. La probabilité p_{dj} de qu'une quelconque des j piles protégées par le systeme soit détruite (si le système n'est pas installé), est function de la probabilité p_{di} de chaque pile:

$$p_{dj} = \sum_{i=1}^{j} p_{di} \qquad (6.1.)$$

En ayant compte du temps mort, l'équation (3.2.) s'écrit avec:

$$p_{dt} = 1 - (1 - p_{dj})^{N_{tj}} \qquad (6.2.)$$

où le temps t_j qui définit N_{tj} est une moyenne pondérée des temps t_i de réparation des différentes piles protégées par le système:

$$t_j = \sum_{i=1}^{j} t_i \cdot \frac{p_{di}}{p_{dj}} \qquad (6.3.)$$

Les coûts du risque et de la protection doivent etre corrigés, pour le système, de façon qu'il puisse etre traité comme s'il s'agissait d'une seule pile, comme suit:

$$C_{1j} = \sum_{i=1}^{j} C_{1i} \cdot \frac{p_{di}}{p_{dj}} \qquad (6.4.)$$

$$C_{2j} = C_2 \qquad (t = t_j) \qquad (6.5.)$$

$$C_{3j} = \sum_{i=1}^{j} C_{3i} \qquad (6.6.)$$

$$C_{4j} = \sum_{i=1}^{j} C_{4i} \qquad (6.7.)$$

$$C_{5j} = \sum_{i=1}^{j} C_{5i} \cdot \frac{p_{di}}{p_{dj}} \qquad (6.8.)$$

et l'efficacité du systeme devient:

$$e_j = \sum_{i=1}^{j} e_i \cdot \frac{p_{di}}{p_{dj}} \qquad (6.9.)$$

Les espérances mathématiques du coût du risque et du coût de la protection seront différentes de celles qu'on obtiendrait de la somme des valeurs obtenues pour les piles considérées séparément, étant donné que la formule 6.1. n'est pas valable por m chocs, ce qui justifie la notion de système.

7. FAISABILITÉ

Le système de protection j est "faisable" s'il remplit la condition:

$$E_{(CV)j} - E_{(CDC)j} > 0 \qquad (7.1.)$$

et le nombre des piles a protéger, j, est celui pour lequel cette différence est maximun. On trouve qu'on peut aussi maximiser cette différence and mélangeant des différentes solutions techniques en function des conditions locales à l'endroit de chaque pile (profondeur, distance du canal de navigation, etc.) et que, évidemment, les protections son d'autant plus faisables que le flux de bateaux est plus grand.

8. COÛT D'INTERRUPTION DU SERVICE

L'estimation de la valeur C_2 est très compliquée, en général, sinon impossible pour una durée suffisament longue r. On présente donc un critère de calcul basé sur l'hypothèse de que la construction du pont était ou est justifiable économiquement. Si ceci est vrai alors la valeur minimun du bénéfice économique apporté par l'ouvrage annuellement doit etre suffisant pour amortir l'investissement plus les dépenses de la maintenance. Si nous appellons B le bénéfice annuel minimun, I l'investissement, αI la dépense annuelle de la maintenance et a le taux de retour minimun, on a:

$$B = I \left[1 / \sum_{q=1}^{r} (1+a)^{-q} + \alpha \right] \qquad (8.1.)$$

Il peut être suffisant de calculer cette valeur pour définir la faisabilité des protections.

On a:

$$C_2 = B \qquad (8.2.)$$

9. RÉFÉRENCES

1. MACDUFF, T. Probability of Vessel Collisions. Ocean Industry, Vol. 9, N° 9, Sept. 1974, pag. 144-148.